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Subject: **Response to Portion of NRC RAI Letter No. 323 Related to ESBWR Design Certification Application – DCD Tier 2 Section 3.8 – Seismic Category I Structures; RAI Number 3.8-94 S04**

The purpose of this letter is to submit the GE Hitachi Nuclear Energy (GEH) response to a portion of the U.S. Nuclear Regulatory Commission (NRC) Request for Additional Information (RAI) letter number 323 sent by NRC letter dated April 6, 2009 (Reference 1). RAI Number 3.8-94 S04 is addressed in Enclosure 1. Enclosure 2 contains the DCD changes to Tier 2 as a result of GEH's response to this RAI. Verified DCD changes associated with this RAI response are identified in the enclosed DCD markups by enclosing the text within a black box.

If you have any questions or require additional information, please contact me.

Sincerely,

Richard E. Kingston

Richard E. Kingston
Vice President, ESBWR Licensing

Reference:

1. MFN 09-245 Letter from U.S. Nuclear Regulatory Commission to J. G. Head, GEH, *Request For Additional Information Letter No. 323 Related to ESBWR Design Certification* dated April 6, 2009

Enclosures:

1. Response to Portion of NRC RAI Letter No. 323 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 – Seismic Category I Structures; RAI Number 3.8-94 S04
2. Response to Portion of NRC RAI Letter No. 323 Related to ESBWR Design Certification Application - DCD Markups for RAI Number 3.8-94 S04

cc:	AE Cubbage	USNRC (with enclosures)
	JG Head	GEH/Wilmington (with enclosures)
	DH Hinds	GEH/Wilmington (with enclosures)
	eDRF Section	0000-0101-1572 (RAI 3.8-94 S04)

ENCLOSURE 1

MFN 09-388

Response to Portion of NRC RAI Letter No. 323
Related to ESBWR Design Certification Application[1]

DCD Tier 2 Section 3.8 – Seismic Category I Structures

RAI Number 3.8-94 S04

[1] Original Response, Supplement 1, Supplement 2 and Supplement 3 previously submitted under MFNs 06-407; 06-407, Supplement 2; 06-407, Supplement 3 and 06-407, Supplement 13 without DCD updates are included to provide historical continuity during review.

NRC RAI 3.8-94

DCD Section 3.8.5.4 indicates that the design incorporates an evaluation of the worst loads resulting from the superstructures and loads directly applied to the foundation mat, due to static and dynamic load combinations. However, the DCD does not identify the maximum allowable toe pressure that is acceptable for the basemat design, under the worst-case static and dynamic loads. This information is needed so that evaluations can be made at the COL state for site-specific conditions. Include the maximum toe pressure used in the basemat design in DCD Table 3.8-13.

GE Response

Maximum soil bearing stresses involving SSE are summarized in DCD Tier 2 Table 3G.1-58 for soft, medium and hard site conditions. Maximum soil bearing stress due to dead plus live loads is 699 kPa as shown in DCD Tier 2 Appendix 3G.1.5.5. The site-specific allowable bearing capacities need to be larger than the maximum stress depending on its site condition.

The values indicated in DCD Tier 2 Table 3G.1-58 are evaluated by using the Energy Balance Method, which is described in the Reference cited in response to NRC RAI 3.7-48, Supplement 1. In the evaluations, the basemat is assumed to be rigid, and uplift of the basemat is considered.

The soil pressures obtained from the RB/FB global FE model analyses used for the basemat section design are summarized in Table 3.8-94(1). This table also includes the results of the basemat uplift analyses, which were performed to respond NRC RAI 3.8-13. Seismic loads used for the FE analyses are worst-case loads, i.e., the enveloped values for all site conditions included in DCD Tier 2 Table 3G.1-58. In the FE analyses, the basemat is assumed to be flexible.

As shown in Table 3.8-94(1), the bearing pressures obtained by the FE analyses are less than the worst case maximum bearing pressure in DCD Tier 2 Table 3G.1-58, which is 5.33 MPa for the hard site. Therefore, it can be concluded that the maximum bearing pressures in DCD Tier 2 Table 3G.1-58 are evaluated conservatively.

Table 3.8-94(1) Maximum Bearing Pressure

Seismic Direction	Case	Max. Pressure (MPa)	Location	Combination
NS	DCD	4.18	Northeast	1.0NS+0.4EW+0.4V
	Uplift*1	4.56	Northeast	1.0NS+0.4EW+0.4V
EW	DCD	4.16	Northeast	0.4NS+1.0EW+0.4V
	Uplift*1	4.49	Northeast	0.4NS+1.0EW+0.4V

Note *1: See response to NRC RAI 3.8-13, Supplement 1.

NRC RAI 3.8-94, Supplement 1

NRC Assessment Following the December 14, 2006 Audit

GE's response refers to Table 3G.1-58 which provides the maximum soil bearing stress involving SSE. GE needs to clarify that the values in Table 3G.1-58 represent the maximum soil bearing stress for all load combinations. GE also needs to explain whether the comparisons to the bearing pressures in Table 3.8-94(1) are for the same load combinations.

During the audit, GE provided a draft supplemental response to address the above. Regarding the first question, GE provided an acceptable response. GE needs to clarify the RAI response and the draft supplemental response regarding the comparison of the maximum bearing pressures reported in Table 3.8-94(1) to Table 3.G.1-58. GE also needs to explain why the toe pressures reported in Table 3G.1-58 are conservative when considering the variation of horizontal soil springs as discussed in RAI 3.8-93.

GE Response

The values in DCD Tier 2 Table 3G.1-58 represent the maximum soil bearing stress for all combinations calculated using the Energy Balance Method for the RB/FB (Reference 1). They are the maximum bearing stresses for the three generic soil conditions. The toe pressures presented in Table 3.8-94(1) are calculated using the global FE model for design seismic forces which envelope the responses of three soil conditions. The methods of analysis are different in the two calculations. Table 3.8-94(2) compares the maximum soil bearing pressures calculated by the Energy Balance Method and the linear FEM analysis. The results show that the Energy Balance Method is a more conservative method to use for the determination of soil bearing pressures. Note that the values obtained by the Energy Balance Method shown in Table 3.8-94(2) are the updated values for DCD Tier 2 Table 3G.1-58, due to the changes in seismic design loads, which have been included in DCD Tier 2 Revision 3.

Reference 1: Tseng, W.S. and Liou, D.D., "Simplified Methods for Predicting Seismic Basemat Uplift of Nuclear Power Plant Structures, Transactions of the 6th International Conference on SmiRT", Paris, France, August 1981

Table 3.8-94(2) Comparison of Maximum Bearing Pressure

		Site Condition (MPa)		
		Soft	Medium	Hard
Energy Balance Method		2.7	7.3	5.4
FEM	Linear	2.6	4.8	5.4
	Uplift*	-	-	5.4

* See response to NRC RAI 3.8-13, Supplement 1. The tension springs of linear cases are eliminated.

The variations of horizontal soil spring ("Hard Spot" and "Soft Spot" as shown in the response to NRC RAI 3.8-93, Supplement 1) are also considered in this study. Note that the DCD envelope is based on uniform soil conditions. Despite the fundamental difference in the treatment of the soil stiffness distribution, the maximum soil bearing pressures of the non-uniform soil condition are similar to those of the uniform soil condition.

Table 3.8-94(3) Maximum Bearing Pressure Under Non-Uniform Soil Condition

	Case	Max. Pressure (MPa)
FEM	Hard Spot*	3.8
	Soft Spot*	4.9

* See response to NRC RAI 3.8-93, Supplement 1.
Stiffer area is Softx3 condition.

DCD Tier 2 Subsections 3G.1.5.5, 3G.1.6, Table 3G.1-58 and Table 3G.2-27 have been revised. The pages (pp. 3G-16, 3G-18, 3G-123 & 3G-215) revised in DCD Tier 2 Revision 3 for this response are attached.

DCD Impact

No DCD change was made in response to this RAI Supplement.

NRC RAI 3.8-94, Supplement 2

NRC Assessment from Chandu Patel E-mail Dated May 24, 2007

The staff requests the applicant to address the following:

(1) The bearing stresses reported in DCD Tier 2 Table 3G.1-58 for soft, medium and hard site conditions are 2.7 MPa (56.4 ksf), 7.3 Mpa (152.6 ksf) and 5.4 MPa (112.9 ksf). These values are extremely large compared to known soil and rock capacities. Explain how the COL applicant will satisfy this criteria. Also explain why the bearing stress reported for the medium site condition (7.3 MPa) is higher than the hard site condition (5.4 MPa).

(2) Explain how the COL applicant is to use the maximum bearing pressures reported in DCD Tier 2 Table 3G.1 58 and Table 3G.2 27 when conditions for a specific site fall between the tabulated values for soft, medium and hard site conditions.

(3) Footnote 7 to DCD Tier 2 Table 2.0-1 references DCD Tier 2 Subsections 3G.1.5.5, 3G2.5.5 and 3G.3.5.5 for the minimum dynamic bearing capacities for the Reactor, Control and Fuel Building, respectively. However, Footnote 7 to the corresponding DCD Tier 1 Table 5.1-1 only states "At foundation level of Seismic Category I structures." Explain why the minimum dynamic bearing capacities are not clearly specified as Tier 1 information.

(4) The response to RAI 3.8-94 states that variations in the horizontal soil spring were considered and concludes that the maximum soil bearing pressures of the nonuniform soil condition are similar to those of the uniform soil condition. Results for maximum bearing pressure under non-uniform soil conditions are presented in Table 3.8-94(3). To complete the response, for the nonuniform soil conditions considered in Table 3.8-94(3), provide comparisons of the bending moments across the basemat in both directions that demonstrate that the DCD design moments bound the moments for the nonuniform soil condition.

GEH Response

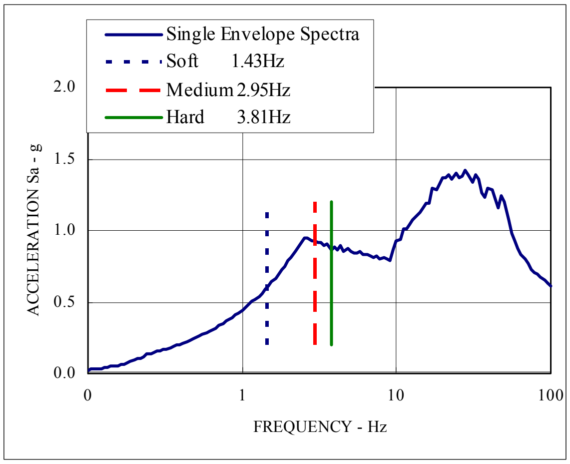
(1) Confirmation of bearing capacity is a COL item as stated in DCD Tier 2 Table 2.0-1. The higher bearing stress at the medium site condition is due to the higher spectral acceleration of the input ground motion response spectra at the SSI fundamental frequencies as shown in Figure 3.8-94(1) in comparison with other site conditions for each direction. Consequently, the envelope of the soil reaction forces, which are the basis for calculating the bearing pressures, are the largest at the medium site as shown in Table 3.8-94(4).

(2) When specific site conditions fall between the cases specified, the larger value within the applicable range applies. Alternatively, a linearly interpolated value

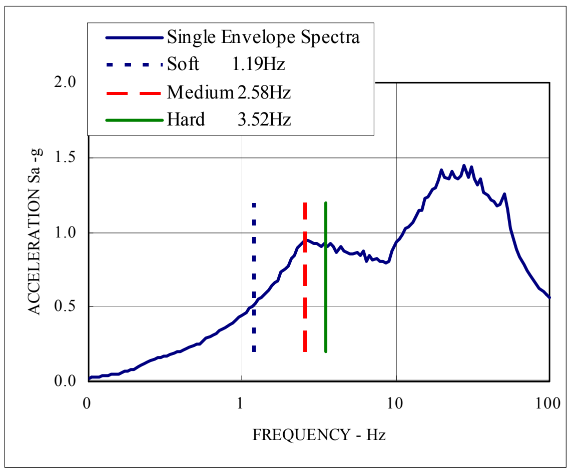
may be used and is clarified in footnotes to DCD Tier 2 Revision 4 Tables 3G.1-58 and 3G.2-27. The revised pages 3G-123 and 3G-228 in DCD Tier 2 Revision 4 are attached.

(3) Minimum dynamic bearing capacities have been included in DCD Tier 1 Revision 4 Table 5.1-1. The revised page 5.1-3 in DCD Tier 1 Revision 4 is attached.

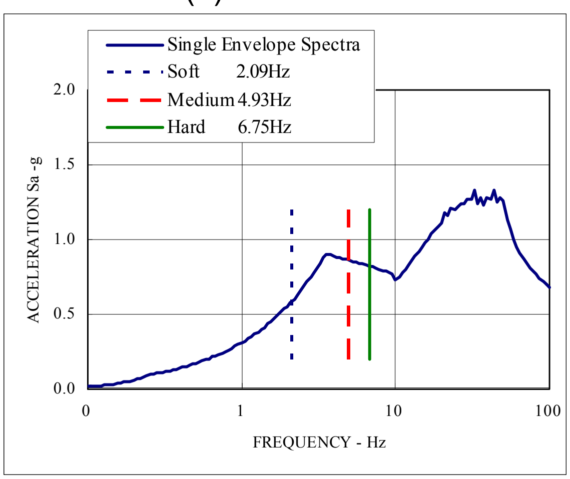
(4) Table 3.8-94(3) is a summary of the analyses results presented in the response to NRC RAI 3.8-93, Supplement 1. The comparisons of the bending moments across the basemat were provided in Figure 3.8-93(16)-c. In that figure Hard Spot case is higher than DCD condition. The allowable bending moment at the top surface of the basemat is 16.7 MNm/m using the rebar ratio (0.321%) shown in DCD Tier 2 Table 3G.1-50. Therefore, it is concluded that the hard spot results do not affect section design in the DCD. Also, DCD Tier 2 Tables 3G.1-51 through 3G.1-55 show rebar and concrete stresses. These calculated stresses are sufficiently lower than Code allowable limits.



(a) NS Direction



(b) EW Direction



(c) Vertical Direction

Figure 3.8-94(1) Input Motion Spectra and RBFB SSI Fundamental Frequencies

Table 3.8-94(4) Soil Spring Reaction for RBFB Seismic Model

	Envelope Soil Reaction	
Soft	N(MN)	M(MNm)
NS	-	22094
EW	-	31999
V	676	-
Bearing Pressure	2.7 Mpa	
Medium	N(MN)	M(MNm)
NS	-	48131
EW	-	58908
V	1148	-
Bearing Pressure	7.3 Mpa	
Hard	N(MN)	M(MNm)
NS	-	50238
EW	-	47061
V	1003	-
Bearing Pressure	5.4 Mpa	

DCD Impact

No DCD change was made in response to this RAI Supplement.

NRC RAI 3.8-94, Supplement 3

The RAI Supplement 2 response, transmitted in GEH letter dated November 28, 2007, provided information to address five items related to the soil bearing capacities. GEH is requested to addresses the following items:

(1) The staff agrees with the statement made in the GEH response that confirmation of the bearing capacity is a COL item. However, the development of the required bearing capacities is part of the DCD review and if the values are extremely large compared to known soil and rock capacities, the staff needs to have a reasonable assurance that these bearing capacities can be met. Therefore, GEH is requested to explain why these extremely large bearing capacities are considered to be reasonable values which can be met at various potential plant sites.

(2) GEH is requested to explain why it is acceptable to use a linearly interpolated value for the soil bearing capacities between the three sets of values (soft, medium, and hard). Using the information presented in Figure 3.8-94(1) (c) of the response (as an example), this would underpredict the required bearing capacity.

(3) Footnotes are still missing in the revised Table 5.1-1 in DCD Tier 1 Revision 4.

Revised GEH Response

(1) The large RB/FB and CB minimum dynamic soil bearing capacities in DCD Revision 5 are considered to be conservative and have been reduced in DCD Revision 6 based on the below recalculation.

The minimum dynamic soil bearing capacities in DCD Revision 5 were determined from bearing pressure demand calculations for foundation stability analyses. These analyses contained conservatisms as follows:

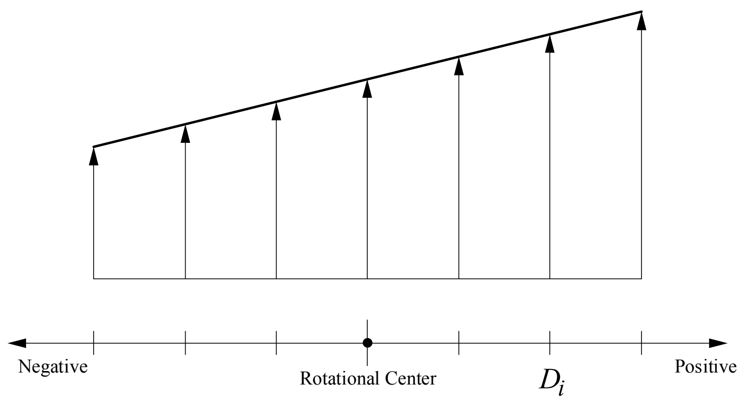
a) Although the RB/FB and the CB are deeply embedded structures, the seismic soil reactions calculated by the DAC3N soil-spring SSI analyses without the embedment effect were used for the stability analysis for these buildings. This extra conservatism is removed in the below bearing pressure demand recalculation in which the seismic soil reactions obtained from the SASSI2000 analyses, which take into account embedment, are used.

b) The bearing forces were calculated by the Energy Balance Method for three cases NS+UD, EW+UD, and UD (vertical). The maximum toe pressures from these cases were then combined by the 100/40/40 method. In this approach the dead weight of the building and vertical seismic load were included in vertical “UD” in all three cases resulting in triple counting of the vertical load effect.

Therefore, the minimum dynamic soil bearing capacities (maximum dynamic soil bearing stress involving SSE plus static) are recalculated as follows:

1. Calculation of Overturning Moment from the SASSI2000 Results

Vertical soil reaction force time histories from the separate NS, EW and UD (vertical) SASSI2000 analyses at each node of the SASSI2000 basemat model are first added by the algebraic sum method since the input motions of the three components are mutually statistically independent. The overall vertical force time history for the basemat is calculated by summing up the reaction forces at all nodes. The overturning moment time histories for both directions are then calculated from the nodal vertical time histories by using the following equations:



$${}_{time=T}M = \sum D_i \cdot {}_{time=T}V_i$$

$${}_{time=T}V_i = {}_{time=T}V_{NS} + {}_{time=T}V_{EW} + {}_{time=T}V_{UD}$$

${}_{time=T}V_{NS}$: Vertical seismic force at T sec due to NS (X-dir) excitation
${}_{time=T}V_{EW}$: Vertical seismic force at T sec due to EW(Y-dir) excitation
${}_{time=T}V_{UD}$: Vertical seismic force at T sec due to UD (Z-dir) excitation

The bearing pressures are evaluated at the possible three timings when the NS (M_x) moment, the EW (M_y) moment or the vertical force (V) each becomes maximum, i.e.:

M_{x_max}, M_y @ time T of M_{x_max}, V @ time T of M_{x_max}
M_{y_max}, M_x @ time T of M_{y_max}, V @ time T of M_{y_max}
V_{max}, M_x @ time T of V_{max}, M_y @ time T of V_{max}

And then the three bearing pressures are enveloped.

2. Evaluation Method of Bearing Pressure from Three Forces, $_TM_x$, $_TM_y$, $_TN$

As for the vertical loads, $_TN$, the following two cases are considered:

Max. $_TN = W + {}_TV$
Min. $_TN = W - B - {}_TV$

where, "W" is the building weight and "B" is the buoyancy force

If $M_x/Z_x > M_y/Z_y$, the following procedure is used (if $M_x/Z_x < M_y/Z_y$, switch M_y for M_x):

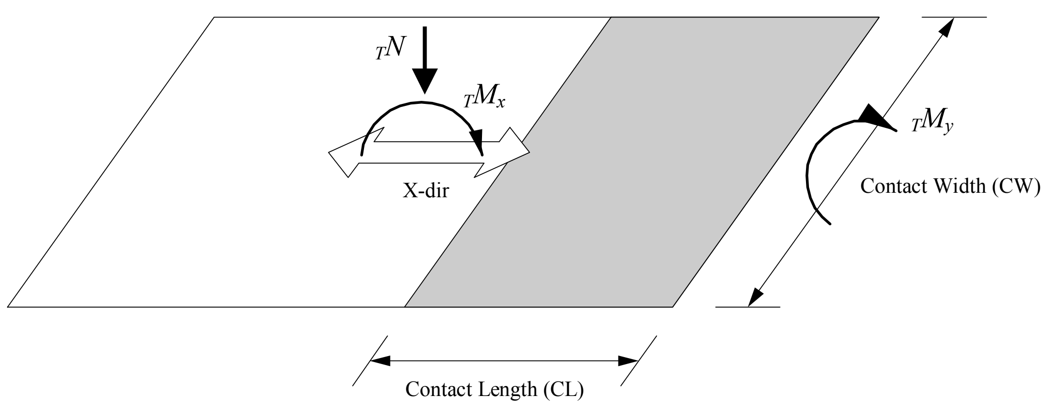
a) Calculate bearing pressure, $_TBP_x$, per the Energy Balance Method using $_TM_x$ and $_TN$

b) Calculate bearing pressure, $_TBP_y$, per the following equation using $_TM_y$:

$_TBP_y = {}_TM_y / Z_y$
$Z_y = CL \cdot CW^2 / 6$ (considering the contact area)

c) Calculate total bearing pressure $_TBP$:

$_TBP = {}_TBP_x + {}_TBP_y$



3. Evaluation Results

Bearing pressures are evaluated by the above method using the SASSI2000 results for three uniform sites (RU-8 with embedment for RB/FB, CU-4 with embedment for CB and FU-2 without embedment for surface founded FWSC) and the layered sites considered in the DCD. DCD Tier 2 Subsections 3.7.1.1.3, 3A.5, 3A.5.2, 3A.6, 3C.7.1.3, 3C.7.2.1 and 3C.7.2.3 and Tables 3.7-3 and 3A.6-1 will be updated in Revision 6 accordingly.

The layered site cases L-2 and L-4 are excluded for the RB/FB and CB in the stability evaluation. The calculated basemat interface loads with the supporting soil for these two sites are large as compared to those for other generic site conditions analyzed. This may be attributed to the large contrast in shear wave velocities in adjacent soil layers assumed for these two layered sites for which the shear wave velocity ratio of the soil layer below the foundation to the soil layer above the foundation is larger than 2.5.

To be consistent with this limitation, a new site interface parameter for maximum ratio of soil shear wave velocity in adjacent layers will be added in DCD Tier 2 Table 2.0-1 in Revision 6 to ensure that the site soil layering does not have as large a contrast in shear wave velocities as the generic layered sites L-2 and L-4 (see DCD Tier 2 Table 3A.3-3 for description of layered sites) as follows:

Bottom 20 m (66 ft) layer to top 20 m (66 ft) layer: 2.5 ratio
Bottom 40 m (131 ft) layer to top 20 m (66 ft) layer: 2.5 ratio

Adjacent layers are the two layers with a total depth of 40 m (131 ft) or 60 m (197 ft) below grade. The first layer, termed top layer, covers the top 20 m (66 ft). The second layer, termed bottom layer, covers the next 20 m (66 ft) or 40 m (131 ft). The ratio is the average velocity of the bottom layer divided by the average velocity of the top layer. Either the lower bound seismic strain (i.e., strain compatible) profile or the best estimate low strain profile can be used because only the velocity ratio is of interest. This velocity ratio condition does not apply to the FWSC nor to the RB/FB and CB if founded on rock-like material having a shear wave velocity of 1067 m/sec (3500 ft/sec) or higher.

The minimum dynamic soil bearing capacities (maximum dynamic soil bearing stress involving SSE plus static) obtained are shown in Table 3.8-94(5). DCD

Tier 1 Table 5.1-1, DCD Tier 2 Tables 2.0-1, 3G.1-58, 3G.2-27 and 3G.4-23 will be revised in Revision 6 with these updated capacities.

The SASSI2000 results of uniform sites (RU-8 for RB/FB, CU-4 for CB and FU-2 for FWSC) are compared with the DAC3N results (RU-3 for RB/FB, CU-3 for CB and FU-1 for FWSC) for floor response spectra as discussed below.

Comparisons of the response spectra are shown in Figures 3.8-94(4) through 3.8-94(15), Figures 3.8-94(16) through 3.8-94(27), and Figures 3.8-94(28) through 3.8-94(39), respectively for the X direction, Y direction, and Z direction. These comparisons will be added in DCD Revision 6 as DCD Tier 2 Figures 3A.8.7-1a through 3A.8.7-3l.

As for the RB/FB, it is found from the results that the responses for the SASSI2000 uniform cases are bounded by the broadened envelope responses of the DAC3N cases in the whole frequency range. The responses of the RU-8 uniform hard site at the vent wall top (X direction per Figure 3.8-94(6)) and the refueling floor (Z direction per (Figure 3.8-94(28)) at around 20 Hz are slightly higher around 20 Hz but the exceedance is negligibly small.

On the other hand, the response spectra of a portion of the CB above ground and the FPE in the FWSC exceeded greater than 10% at the broadened envelope responses of the DAC3N cases in the higher frequency range.

Thus, the SASSI2000 uniform site results of the CB and the FWSC are included where appropriate to obtain the enveloping design spectra. DCD Tier 2 Figures 3A.9-1g, 3A.9-1l, 3A.9-2g and 3A.9-3g will be revised in Revision 6 accordingly.

The uniform site SASSI2000 results for seismic forces of building structural members are less than the DAC3N results, thus there is no impact on the design envelope loads.

DCD Tier 2 Subsection 3A.8.7 and Table 3A.8.7-1 will be updated and DCD Tier 2 Subsection 3A.9.3 will be added in Revision 6 to incorporate the above discussion.

(2) The linear interpolation method is adequate to evaluate maximum dynamic soil bearing pressures for sites within the applicable range of shear wave velocities considered in the DCD.

In accordance with NRC RAI 3.8-94, Supplement 2, Figure 3.8-94(1) (c), the vertical input response spectrum peaks at 3.57 Hz in the frequency range between 2.09 Hz for the soft site and 4.93 Hz for the medium site. This peak vertical frequency is 73% of the vertical SSI fundamental frequency at the medium site.

Applying the same frequency ratio to the horizontal SSI frequencies of the medium site, the horizontal SSI fundamental frequencies for the 3.57 Hz vertical frequency site (termed “intermediate site” hereafter) are found to be 2.14 Hz and 1.87 Hz in the NS and EW directions, respectively. The corresponding spectral accelerations are 0.85g, 0.76g and 0.9g in the NS, EW and vertical directions, respectively, and the corresponding ratios to the medium site spectral accelerations are 0.91, 0.8 and 1.05, as shown in Figure 3.8-94(2).

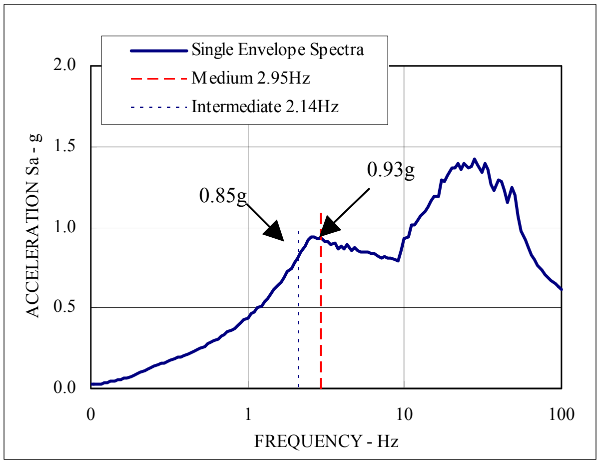
These spectral acceleration ratios are then applied to the SSE base moment and SSE vertical load of the medium site to obtain the corresponding loads for the intermediate site in the bearing pressure calculation and the maximum bearing pressure for the intermediate site is found to be 1.39 MPa.

This calculated value agrees with the value obtained by the linear interpolation of bearing pressures between the soft and medium sites as illustrated in Figure 3.8-94(3), in which the shear wave velocity value of 561 m/sec for the intermediate site (3.57 Hz) is linearly interpolated from 300 m/sec soft site (2.09 Hz) and 800 m/sec medium site (4.93 Hz).

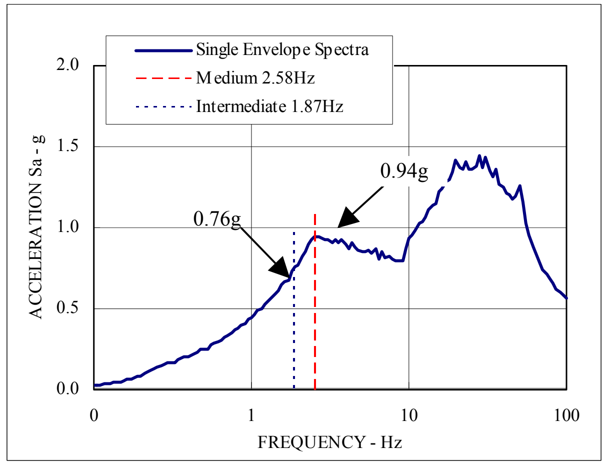
(3) DCD Tier 1 Table 5.1-1 has been revised in Revision 5 to retain only those footnotes in DCD Tier 2 Table 2.0-1 that are intrinsic to the description of the ESBWR Standard Plant site design parameter and are not background information for the parameter. Please see the GEH response to NRC RAI 2.0-1 transmitted to the NRC on March 24, 2008 via MFN 08-248.

Table 3.8-94(5) Maximum Dynamic Soil Bearing Stress Involving SSE + Static (MPa)

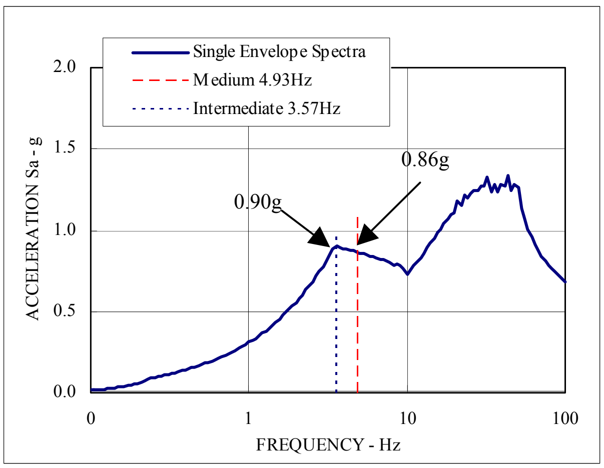
Building	Site Condition		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
RB/FB	1.2	1.5	1.1
CB	0.44	2.2	0.42
FWSC	0.46	0.69	1.2



(a) NS Direction

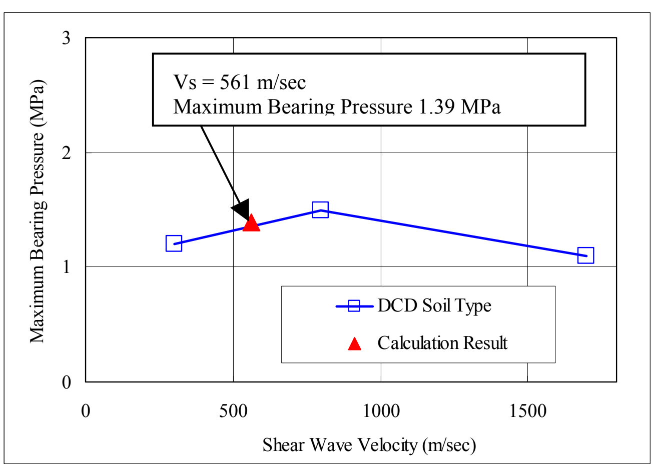


(b) EW Direction



(c) Vertical Direction

Figure 3.8-94(2) Input Motion to Intermediate Site



V_s Interpolation:

Medium Site: f = 4.93 Hz, V_s = 800 m/sec
Soft Site: f = 2.09 Hz, V_s = 300 m/sec

Hence,

Intermediate Site: f = 3.57 Hz, V_s = 561 m/sec

Figure 3.8-94(3) Calculated vs. Interpolated Bearing Pressures

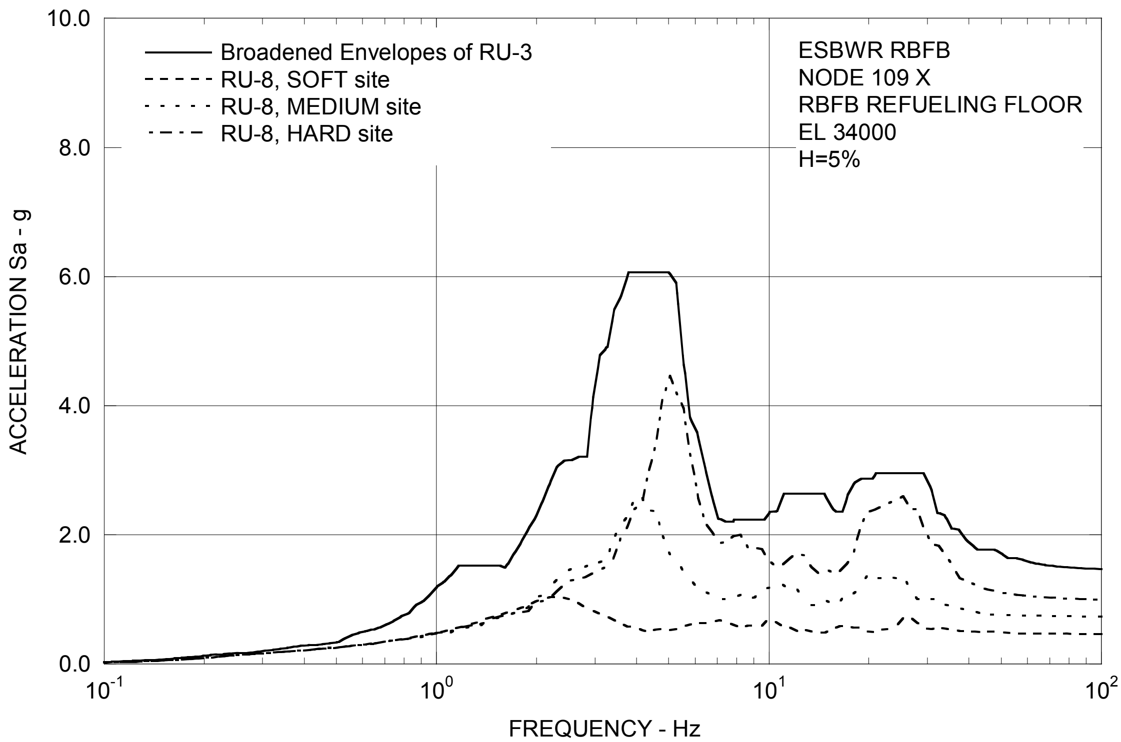


Figure 3.8-94(4) FRS (Compared with the DAC3N) – RBFB Refueling Floor X

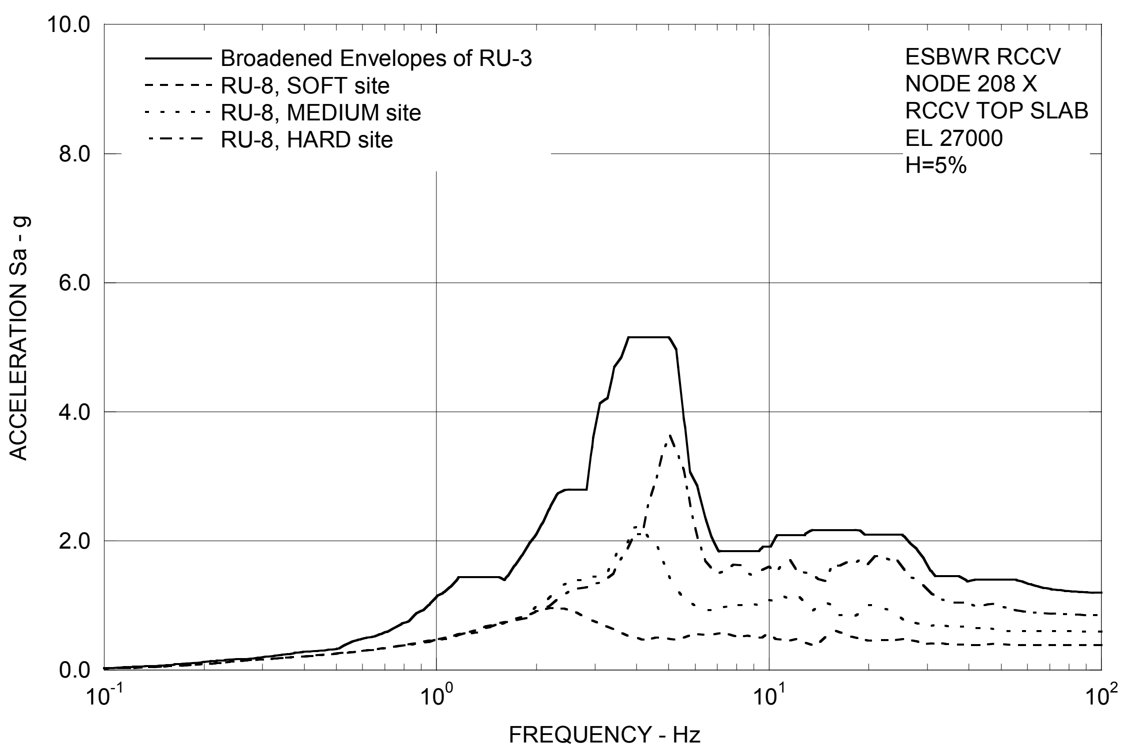


Figure 3.8-94(5) FRS (Compared with the DAC3N) – RCCV Top Slab X

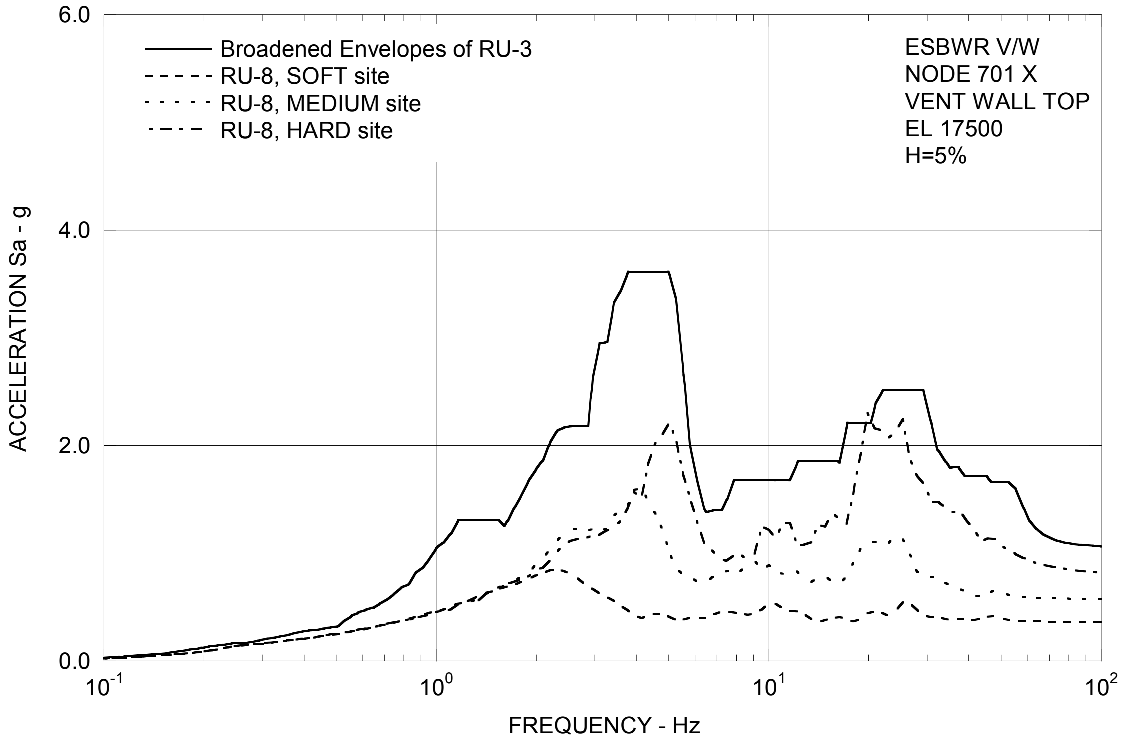


Figure 3.8-94(6) FRS (Compared with the DAC3N) – Vent Wall Top X

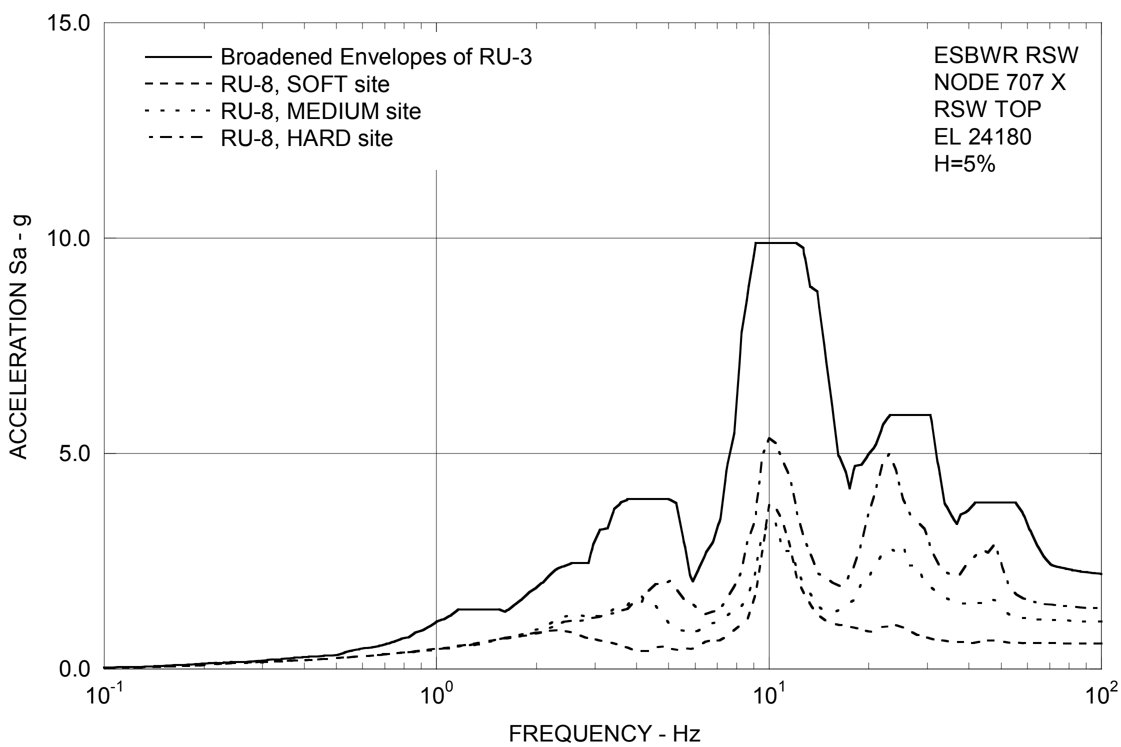


Figure 3.8-94(7) FRS (Compared with the DAC3N) – RSW Top X

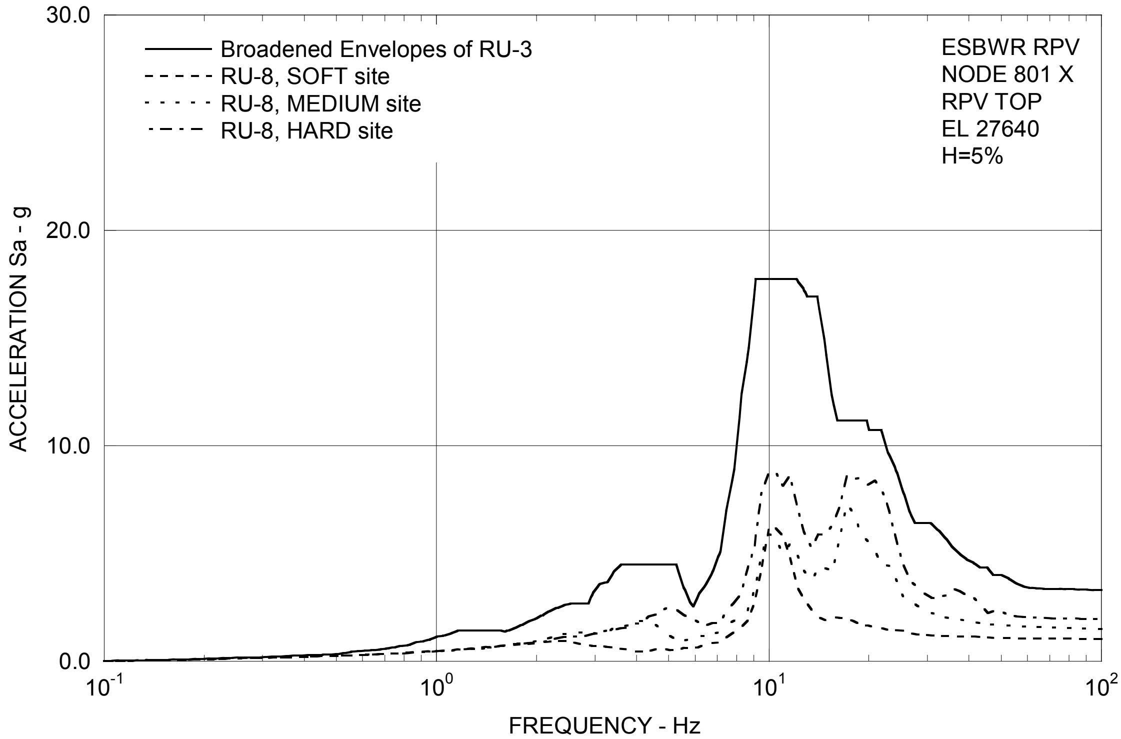


Figure 3.8-94(8) FRS (Compared with the DAC3N) – RPV Top X

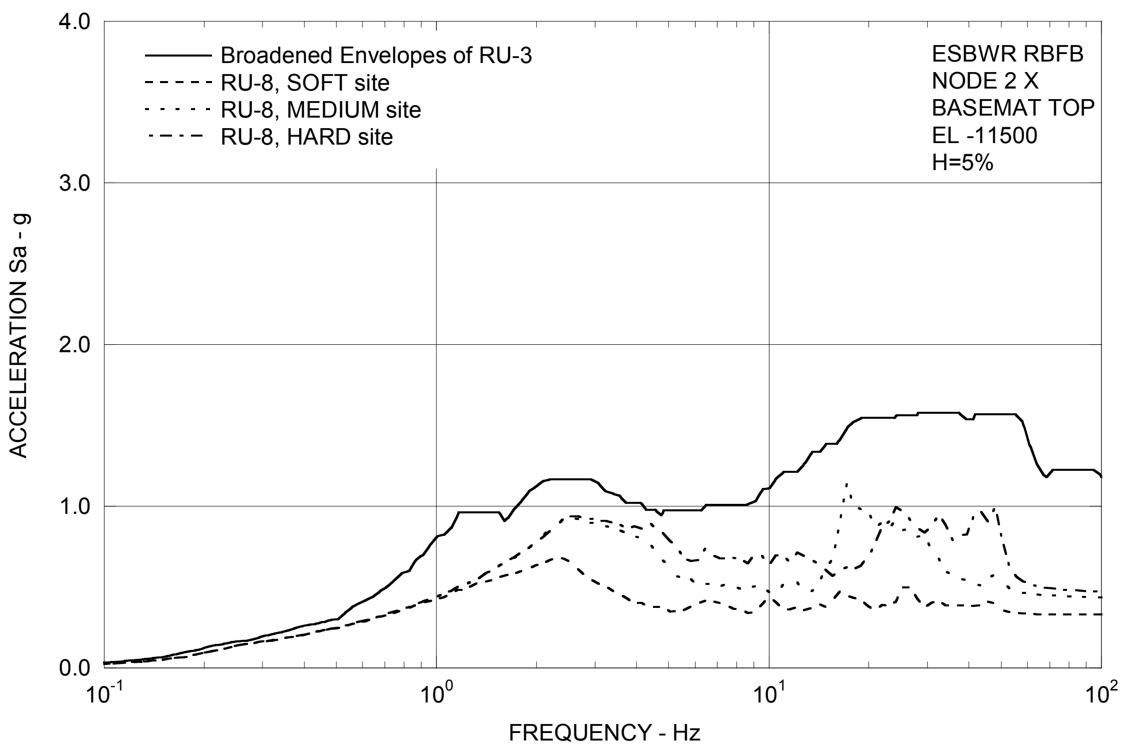


Figure 3.8-94(9) FRS (Compared with the DAC3N) – RBFB Basemat X

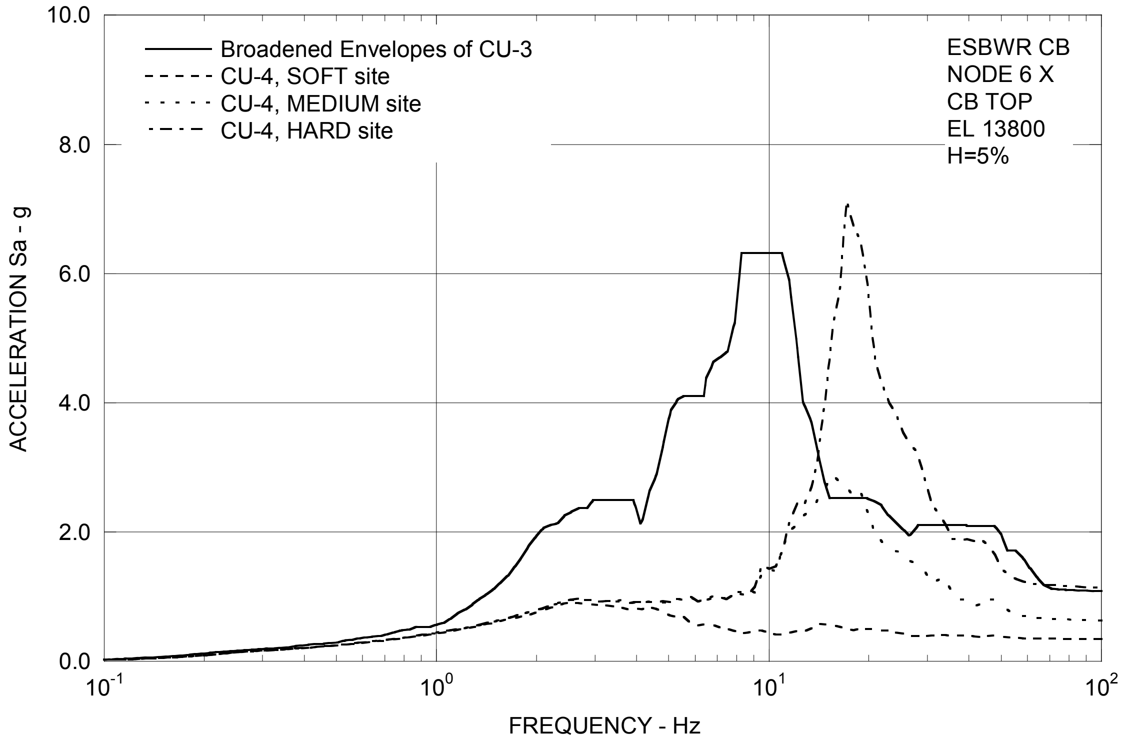


Figure 3.8-94(10) FRS (Compared with the DAC3N) – CB Top X

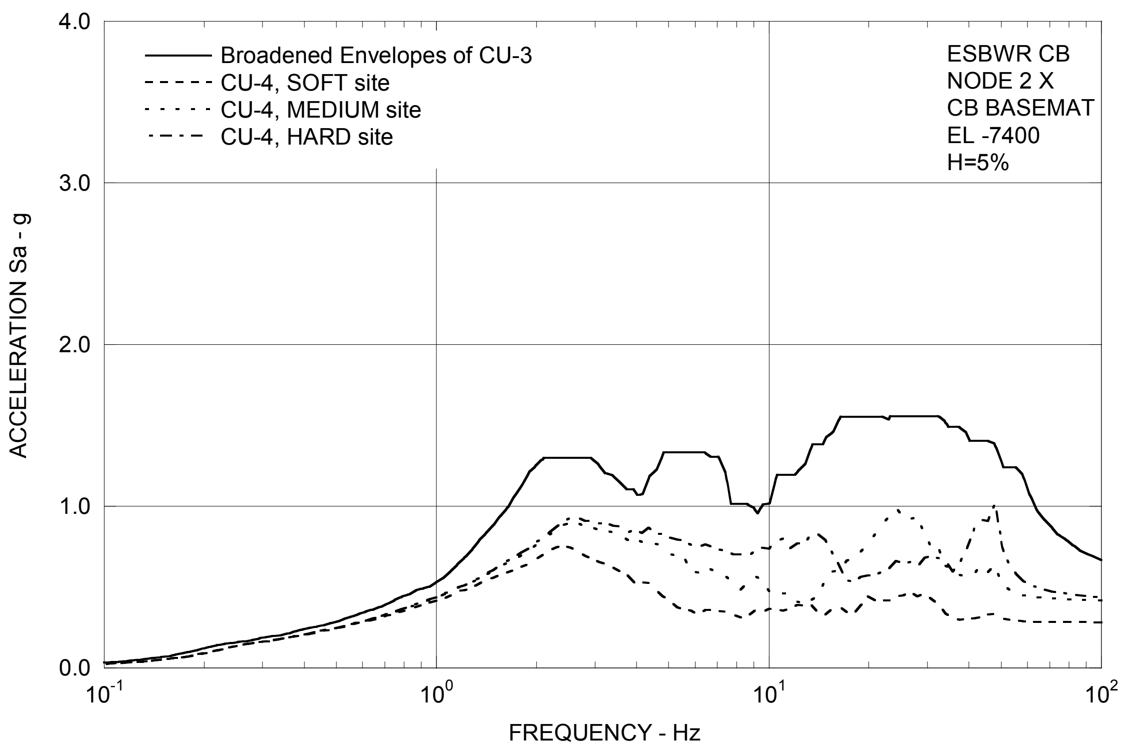


Figure 3.8-94(11) FRS (Compared with the DAC3N) – CB Basemat X

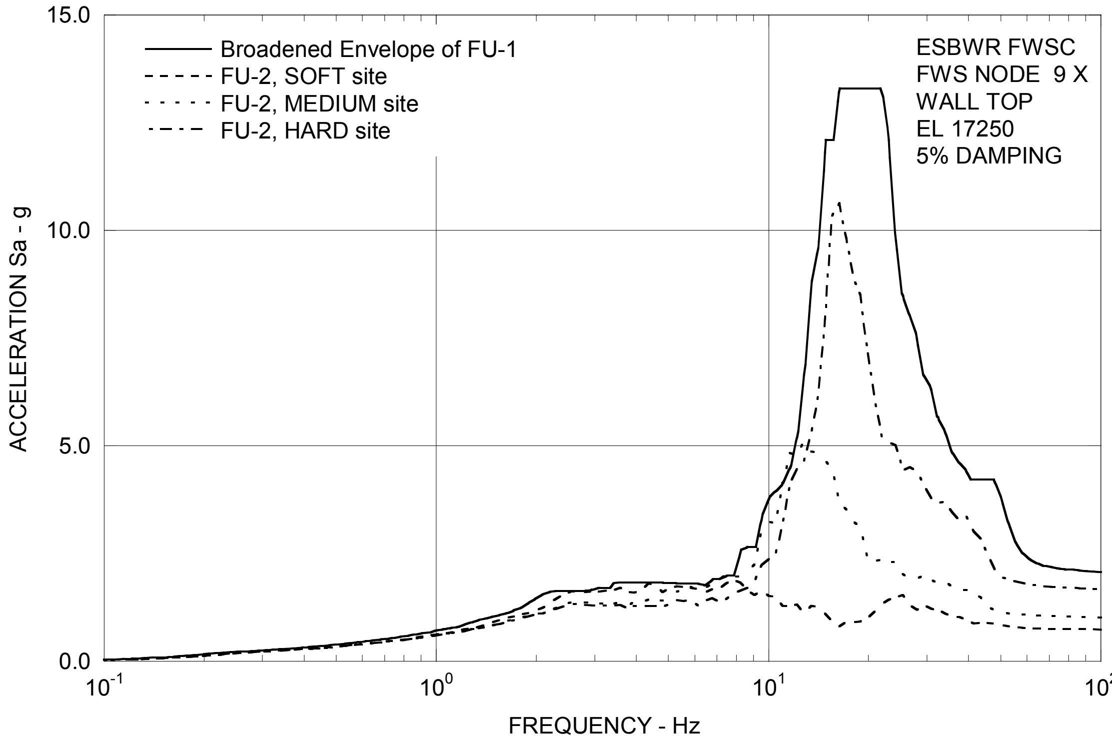


Figure 3.8-94(12) FRS (Compared with the DAC3N) – FWS Wall Top X

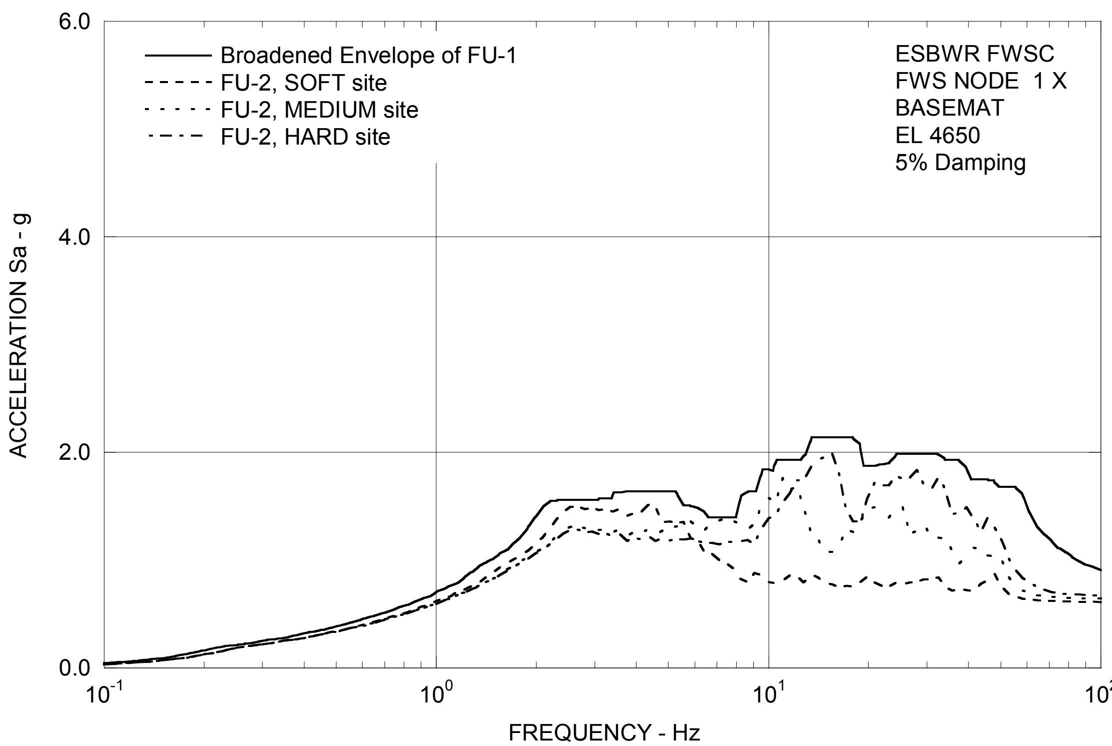


Figure 3.8-94(13) FRS (Compared with the DAC3N) – FWS Basemat X

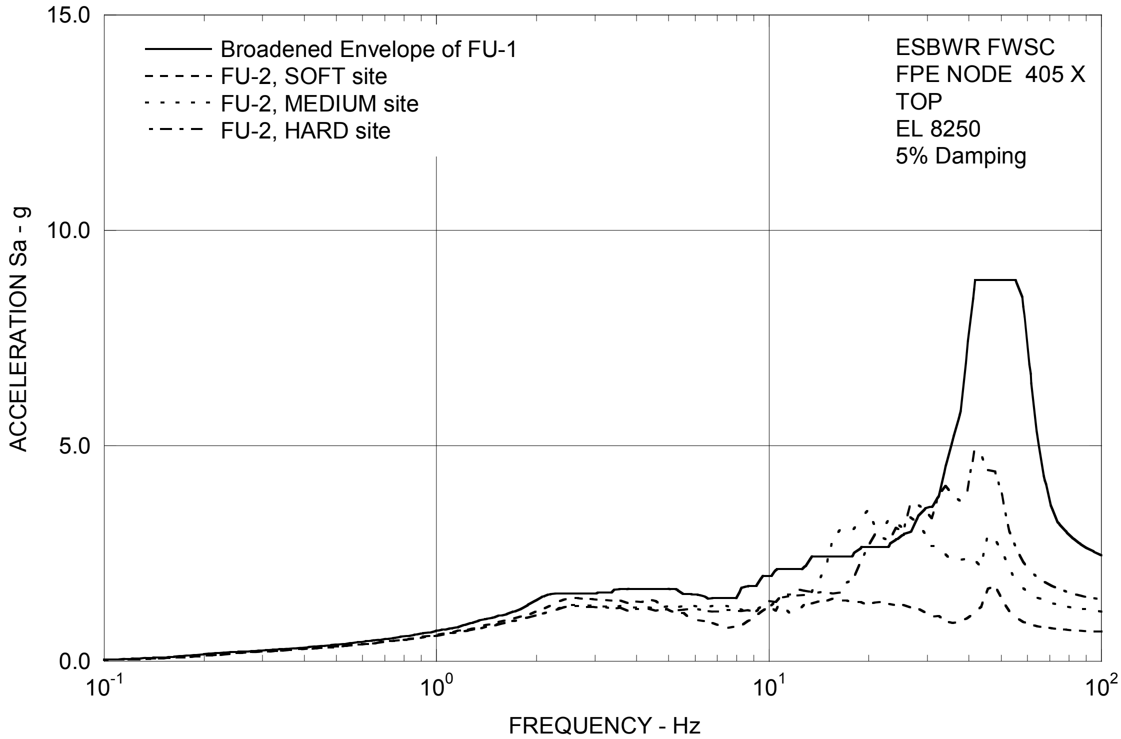


Figure 3.8-94(14) FRS (Compared with the DAC3N) – FPE Top X

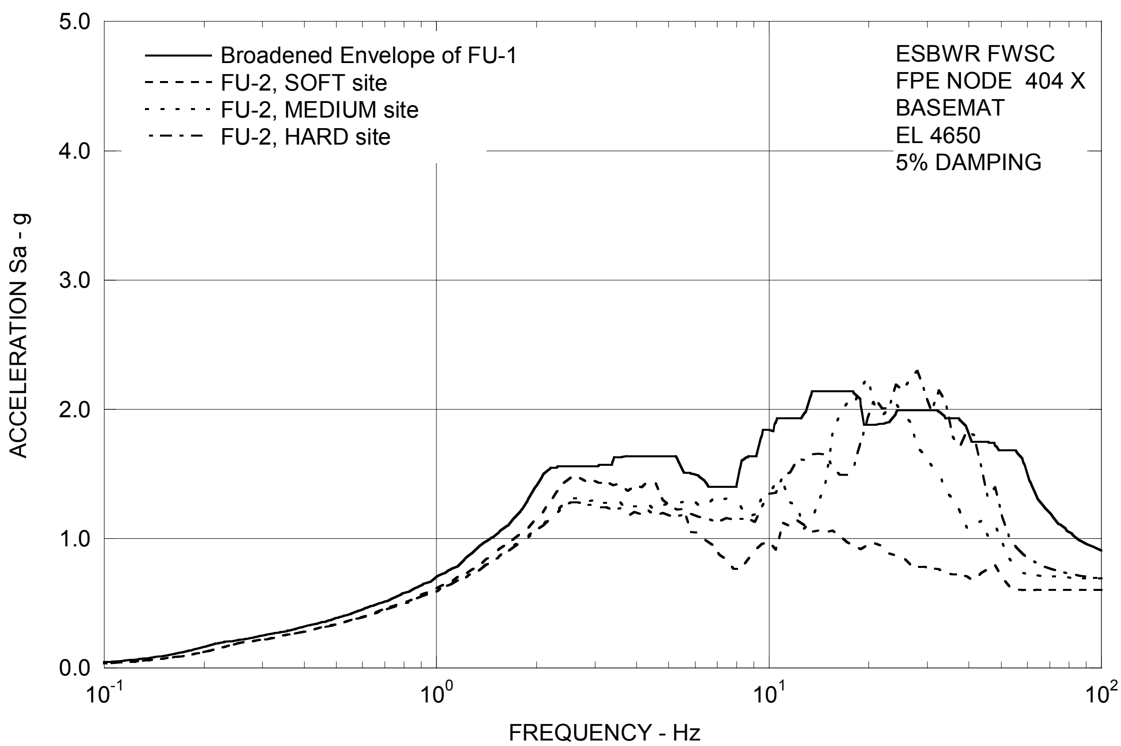


Figure 3.8-94(15) FRS (Compared with the DAC3N) – FPE Basemat X

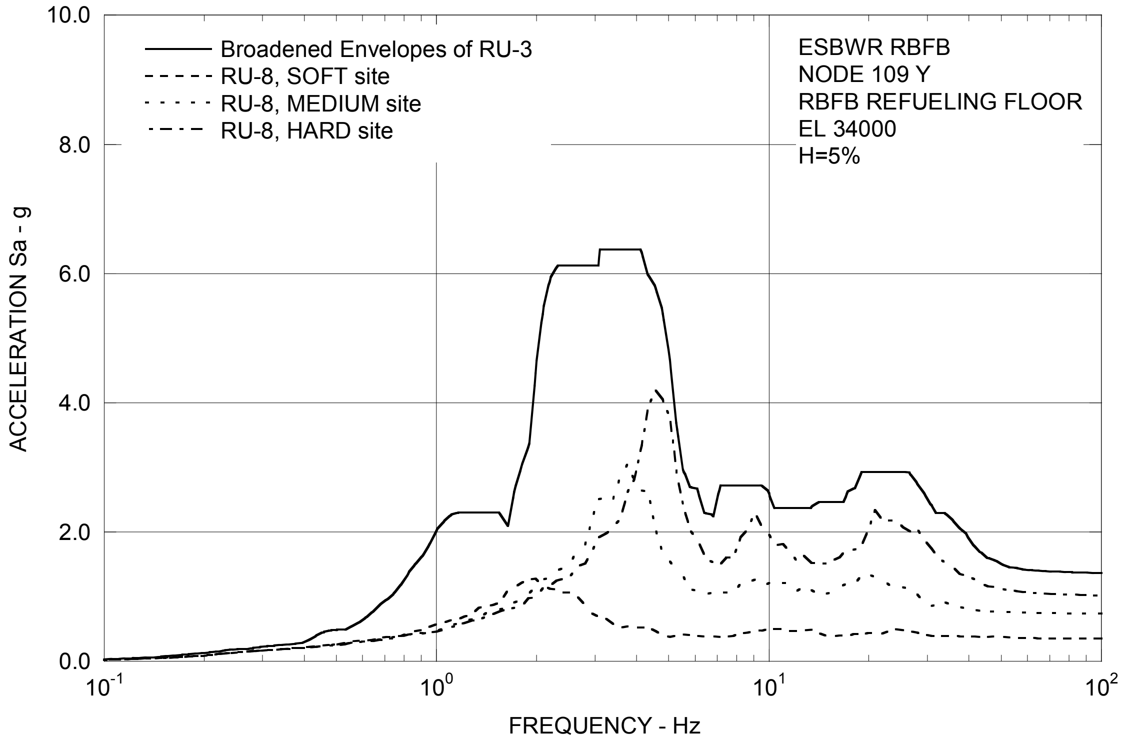


Figure 3.8-94(16) FRS (Compared with the DAC3N) – RBFB Refueling Floor Y

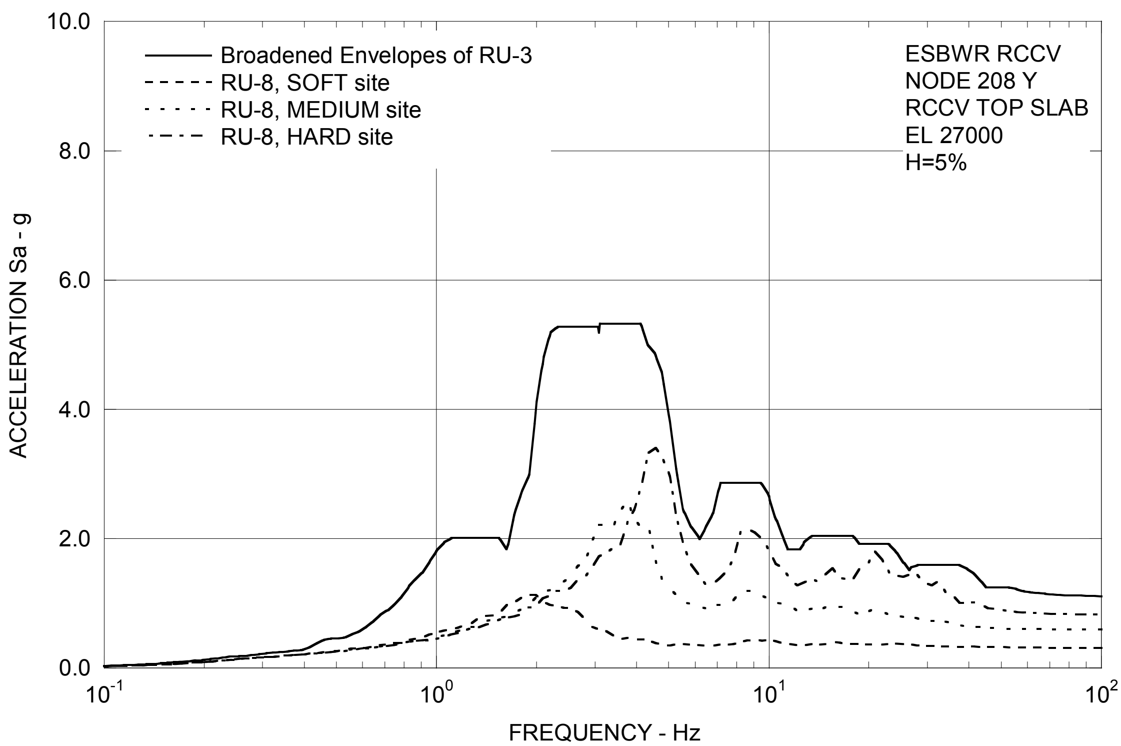


Figure 3.8-94(17) FRS (Compared with the DAC3N) – RCCV Top Slab Y

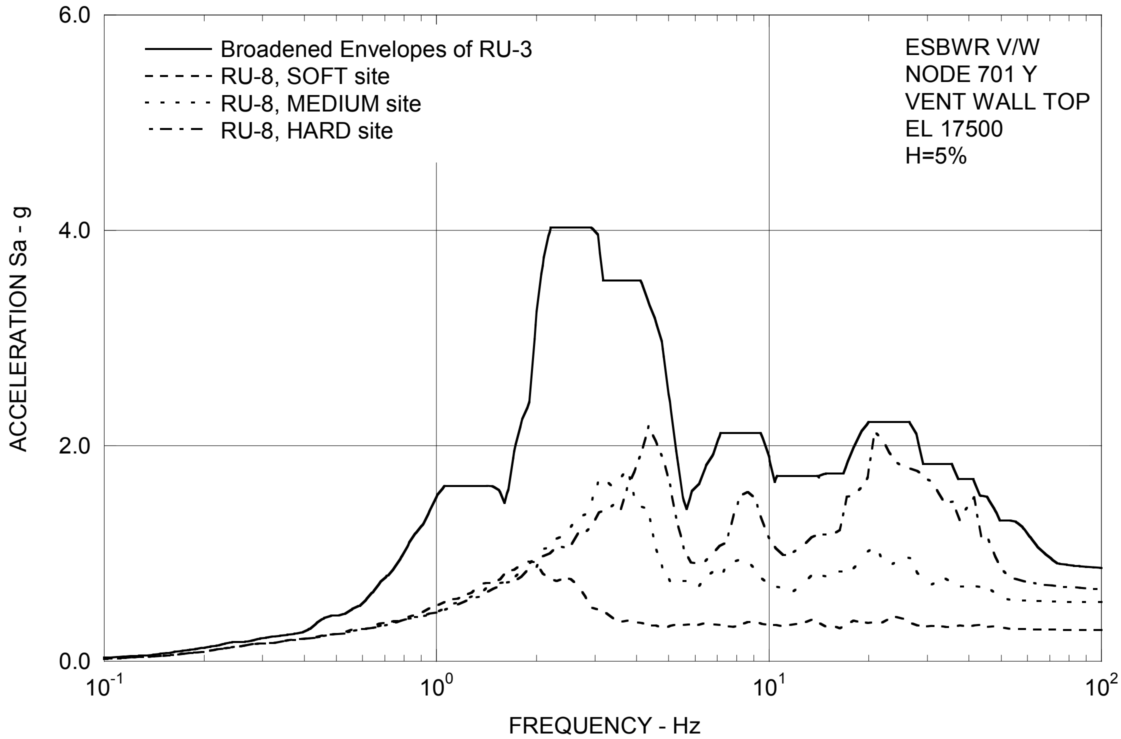


Figure 3.8-94(18) FRS (Compared with the DAC3N) – Vent Wall Top Y

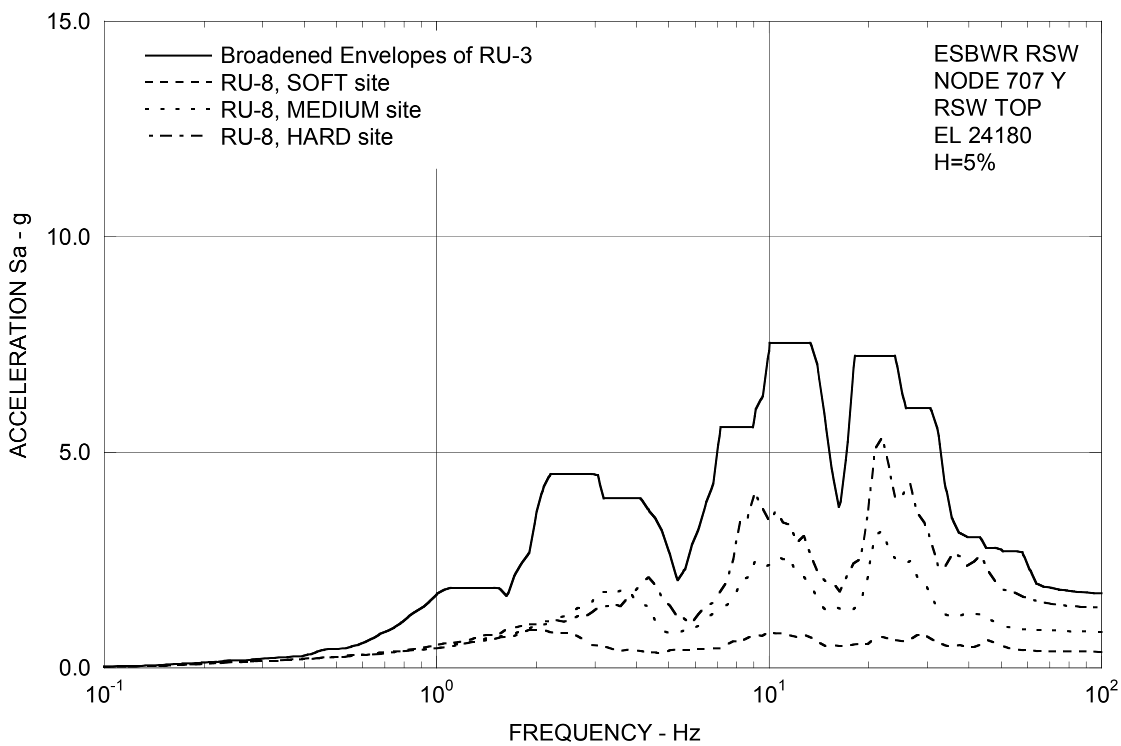


Figure 3.8-94(19) FRS (Compared with the DAC3N) – RSW Top Y

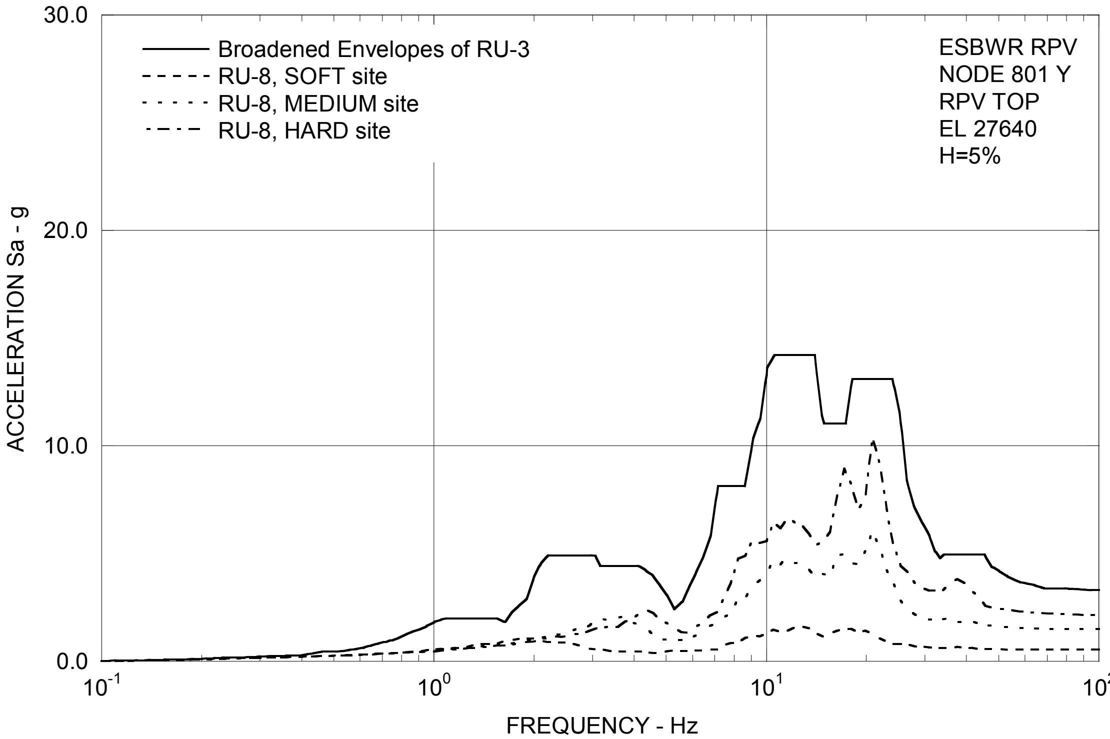


Figure 3.8-94(20) FRS (Compared with the DAC3N) – RPV Top Y

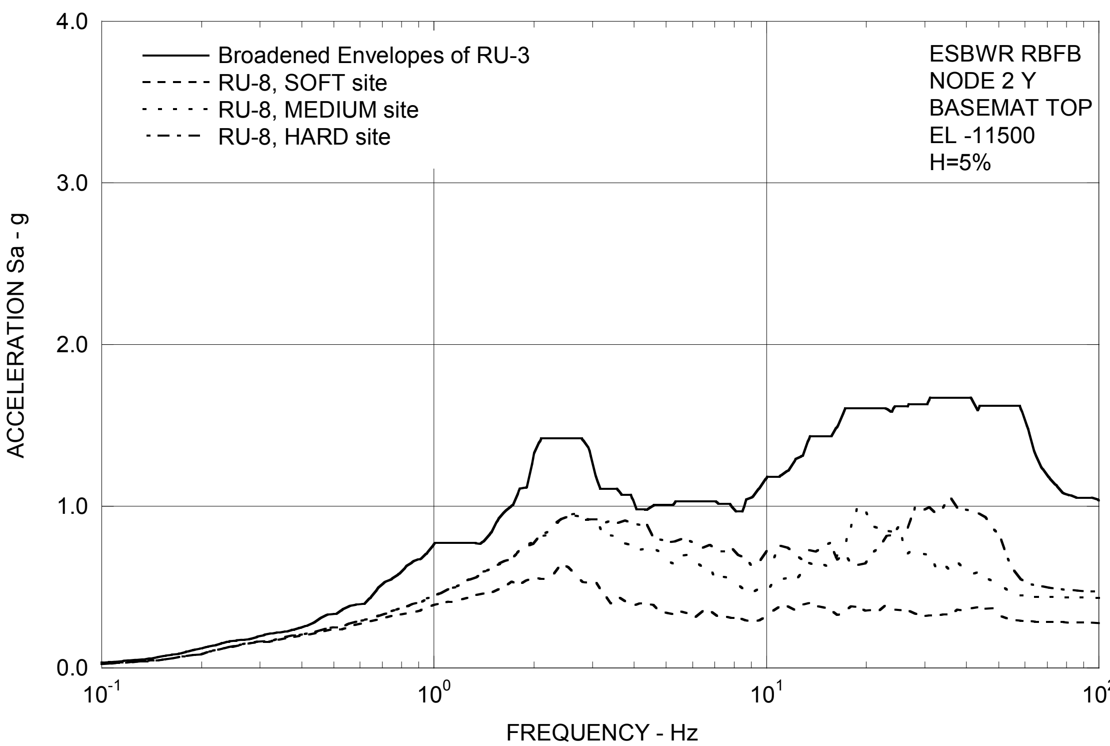


Figure 3.8-94(21) FRS (Compared with the DAC3N) – RBFB Basemat Y

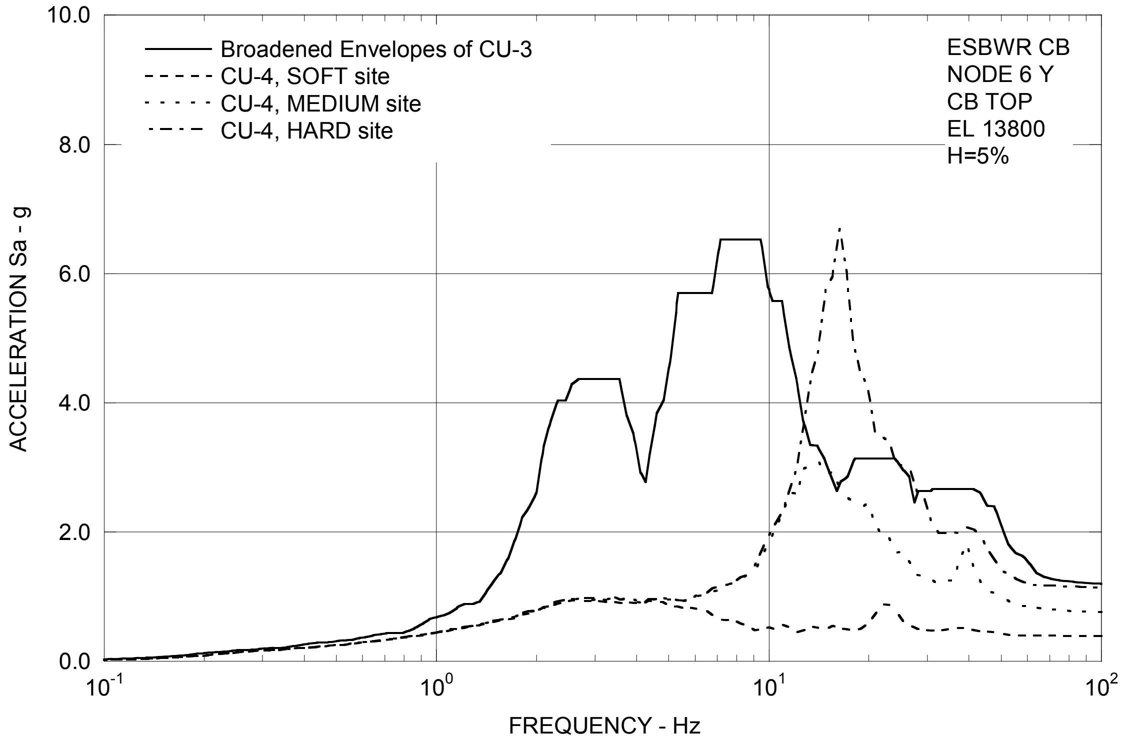


Figure 3.8-94(22) FRS (Compared with the DAC3N) – CB Top Y

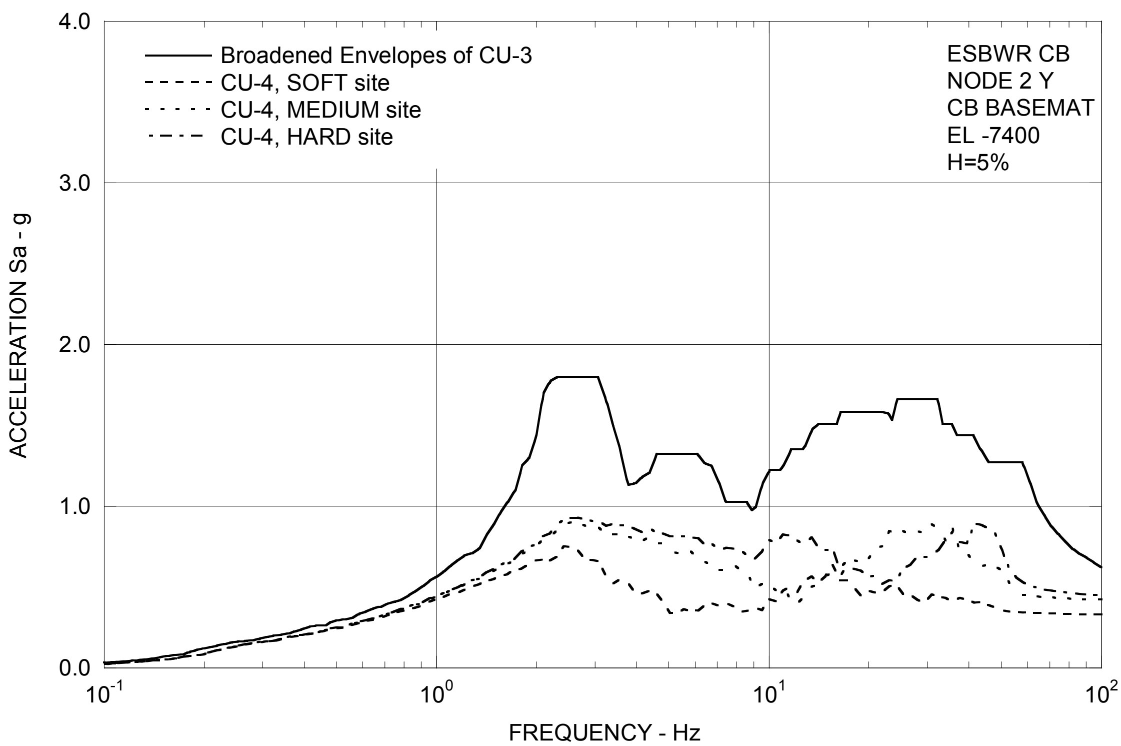


Figure 3.8-94(23) FRS (Compared with the DAC3N) – CB Basemat Y

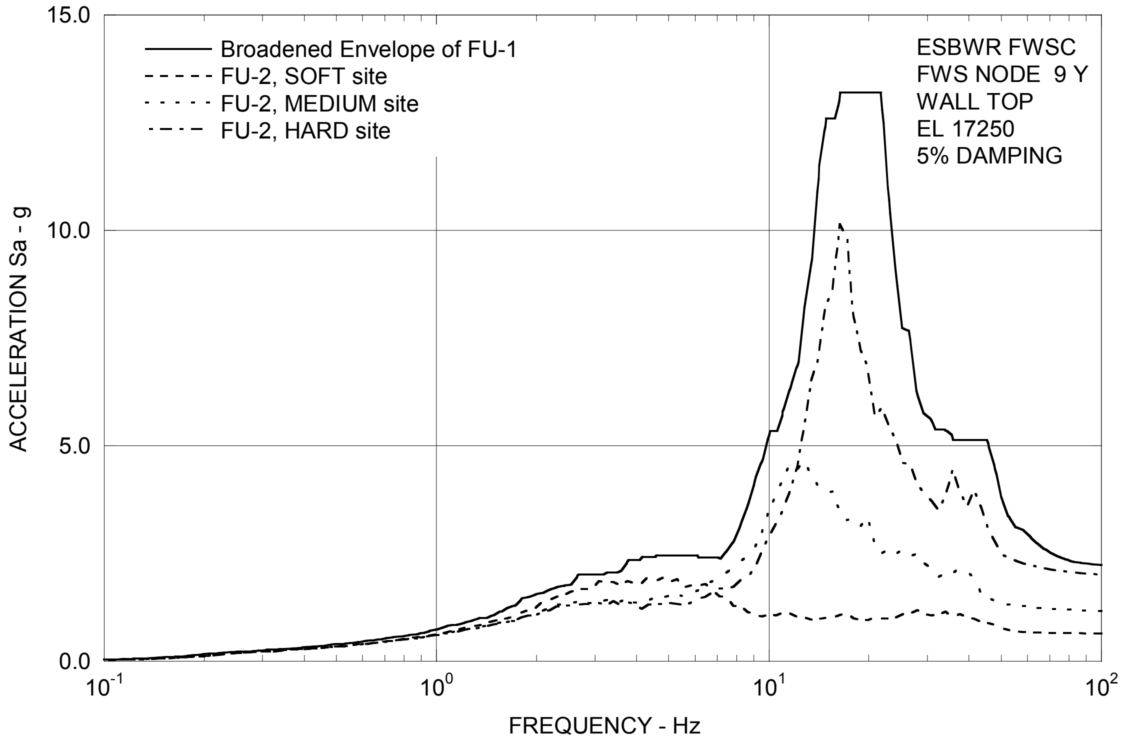


Figure 3.8-94(24) FRS (Compared with the DAC3N) – FWS Wall Top Y

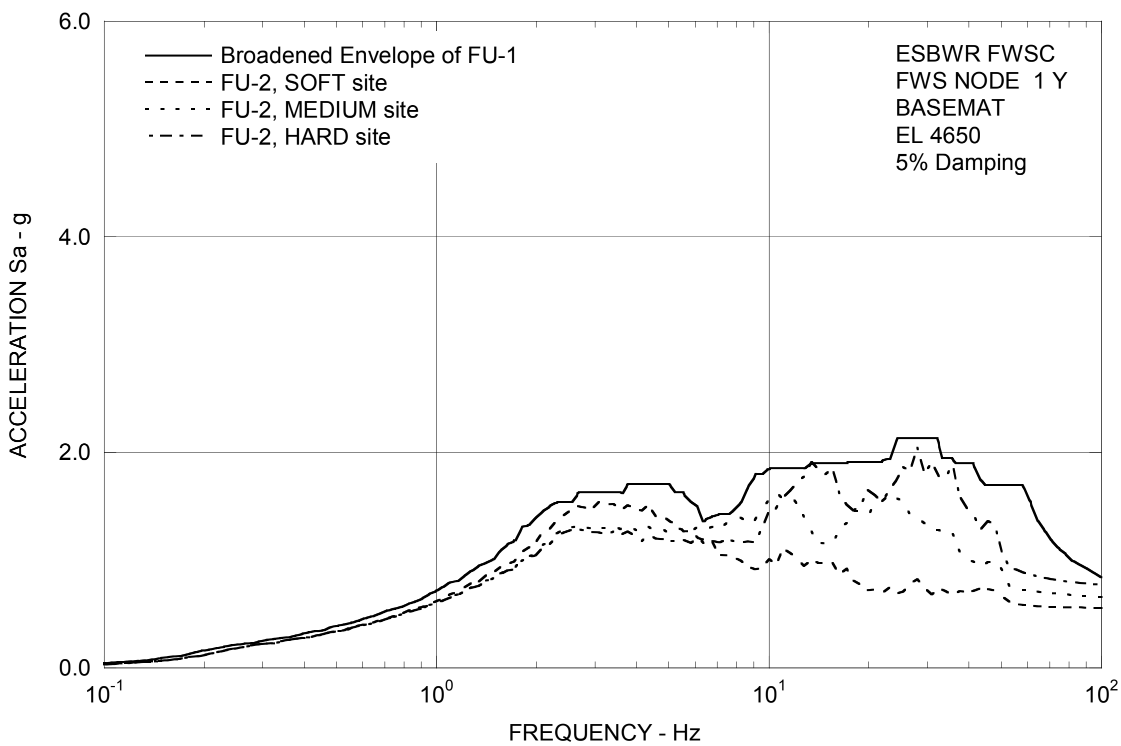


Figure 3.8-94(25) FRS (Compared with the DAC3N) – FWS Basemat Y

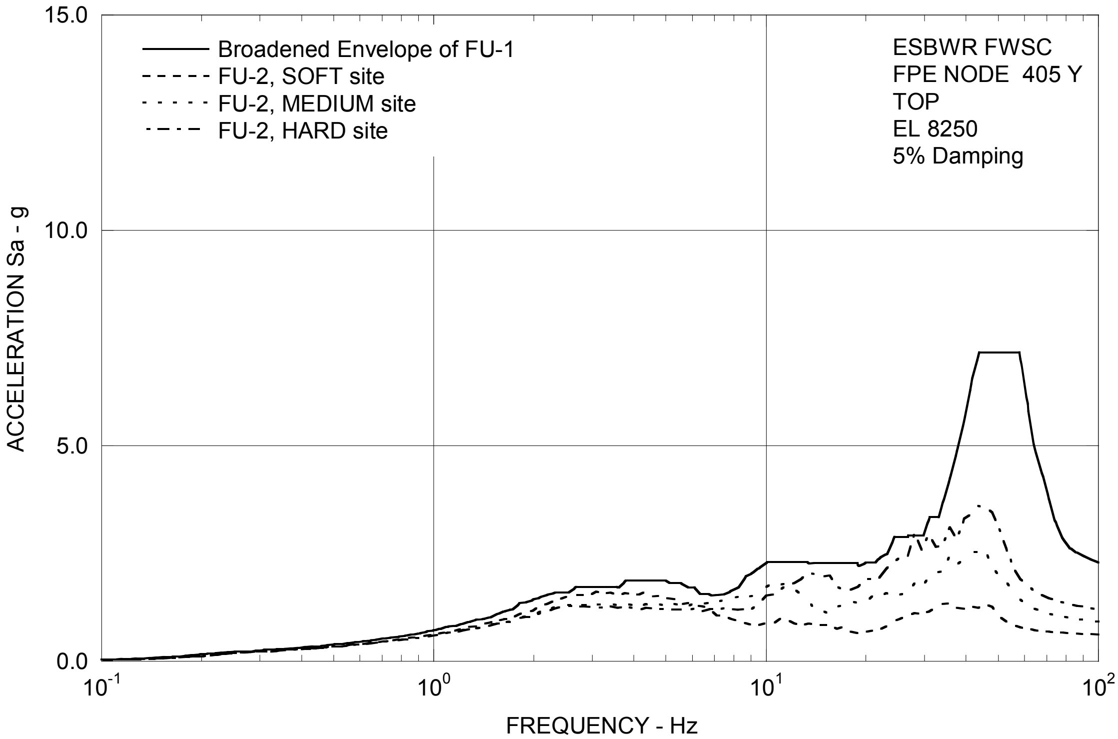


Figure 3.8-94(26) FRS (Compared with the DAC3N) – FPE Top Y

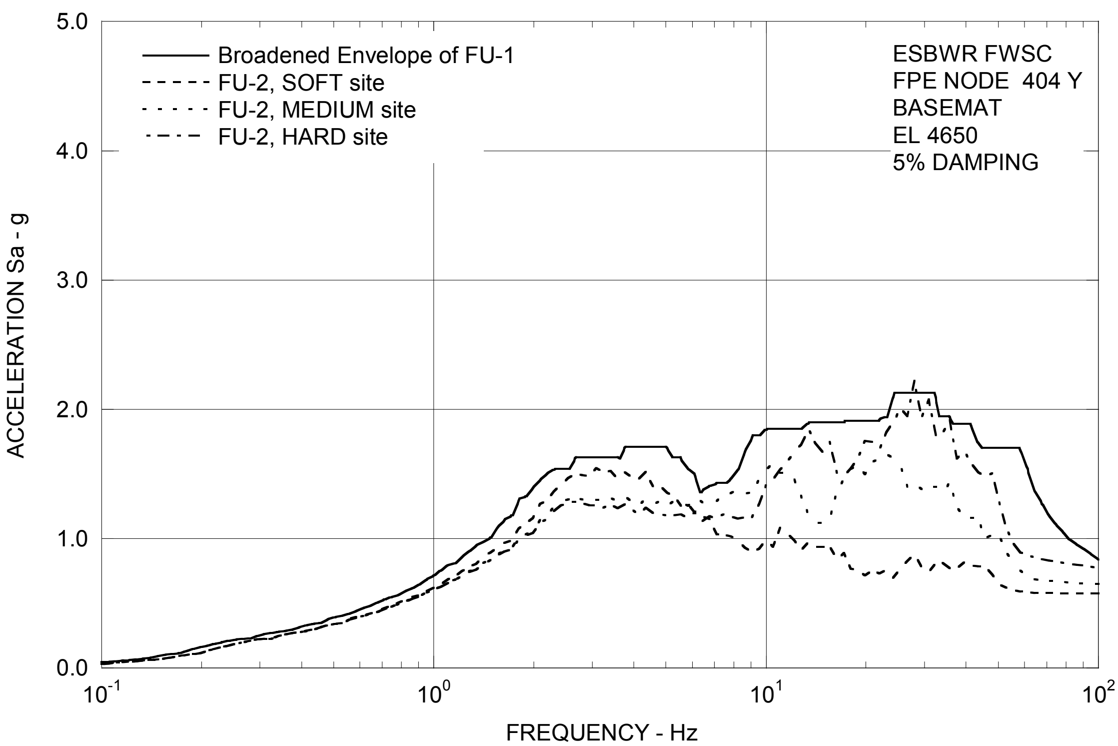


Figure 3.8-94(27) FRS (Compared with the DAC3N) – FPE Basemat Y

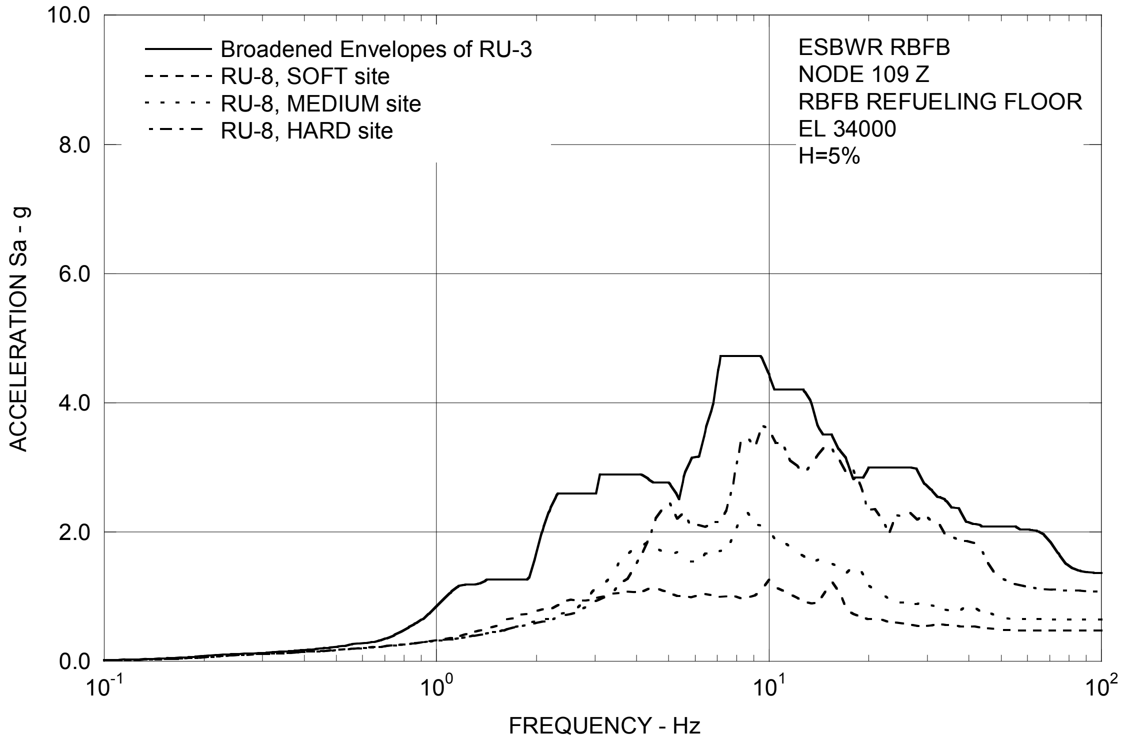


Figure 3.8-94(28) FRS (Compared with the DAC3N) – RBFB Refueling Floor Z

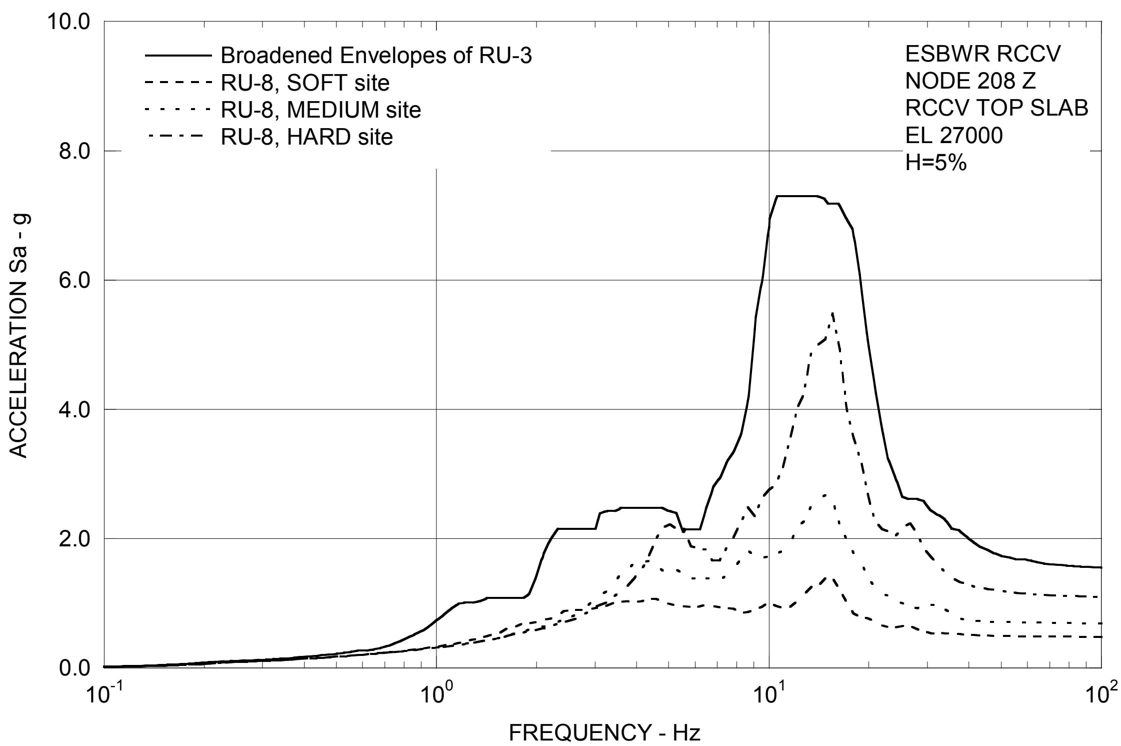


Figure 3.8-94(29) FRS (Compared with the DAC3N) – RCCV Top Slab Z

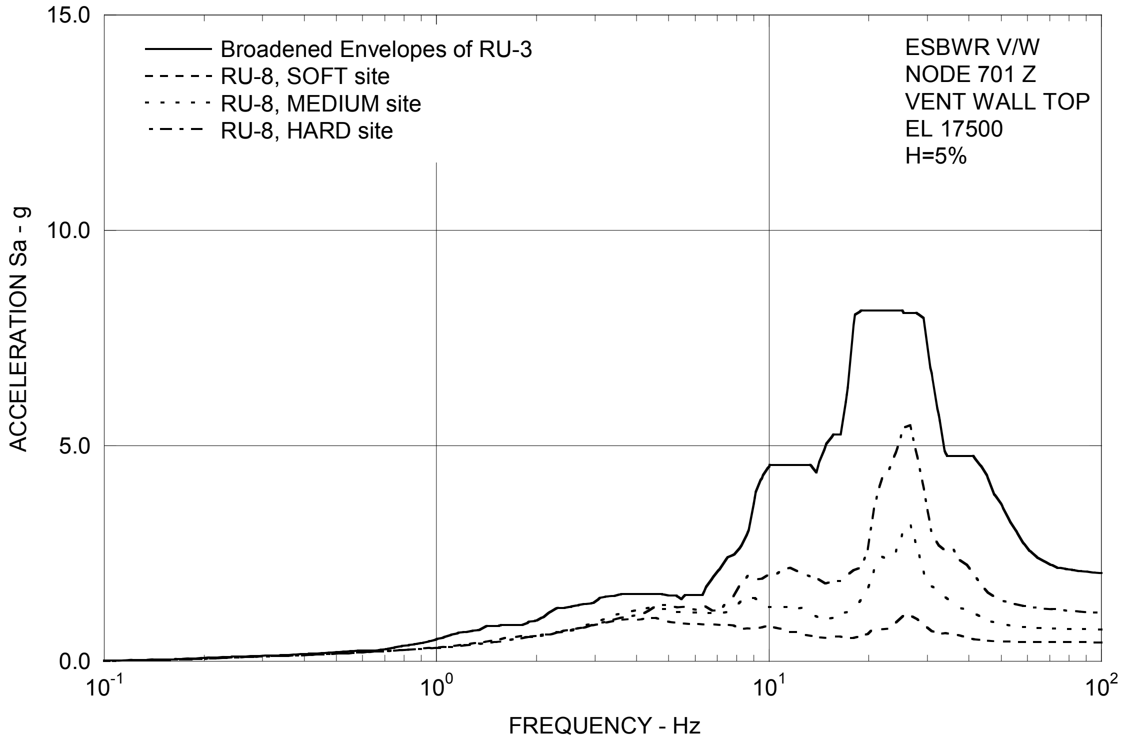


Figure 3.8-94(30) FRS (Compared with the DAC3N) – Vent Wall Top Z

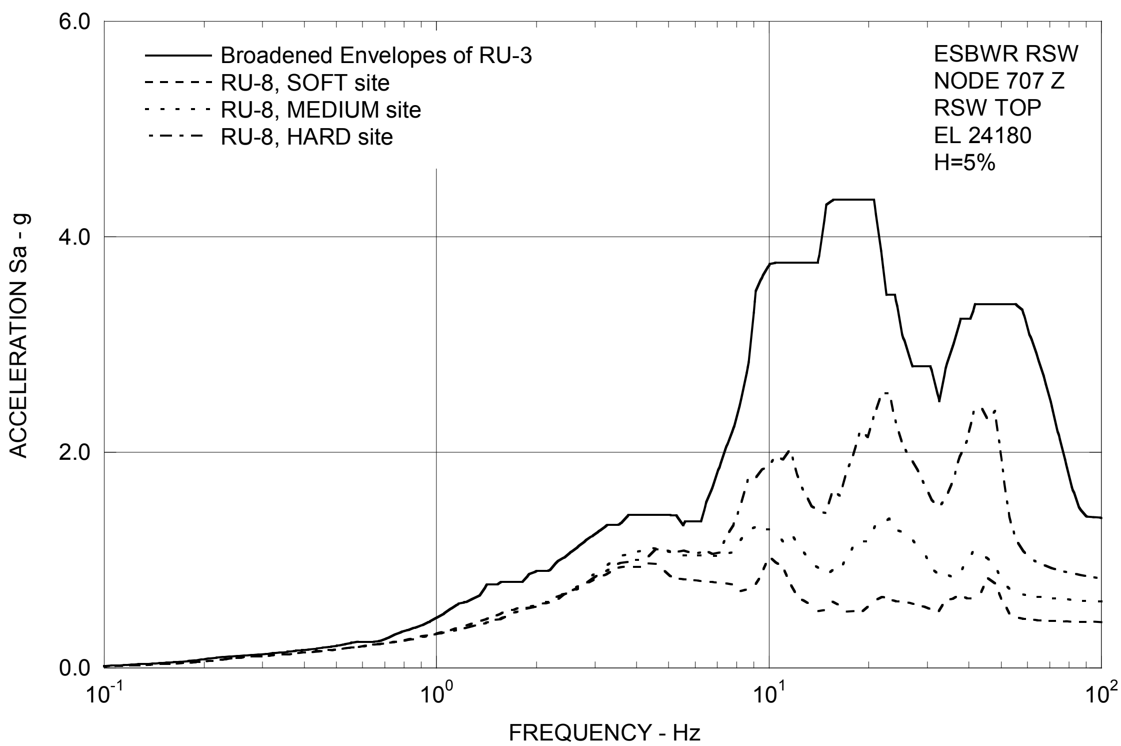


Figure 3.8-94(31) FRS (Compared with the DAC3N) – RSW Top Z

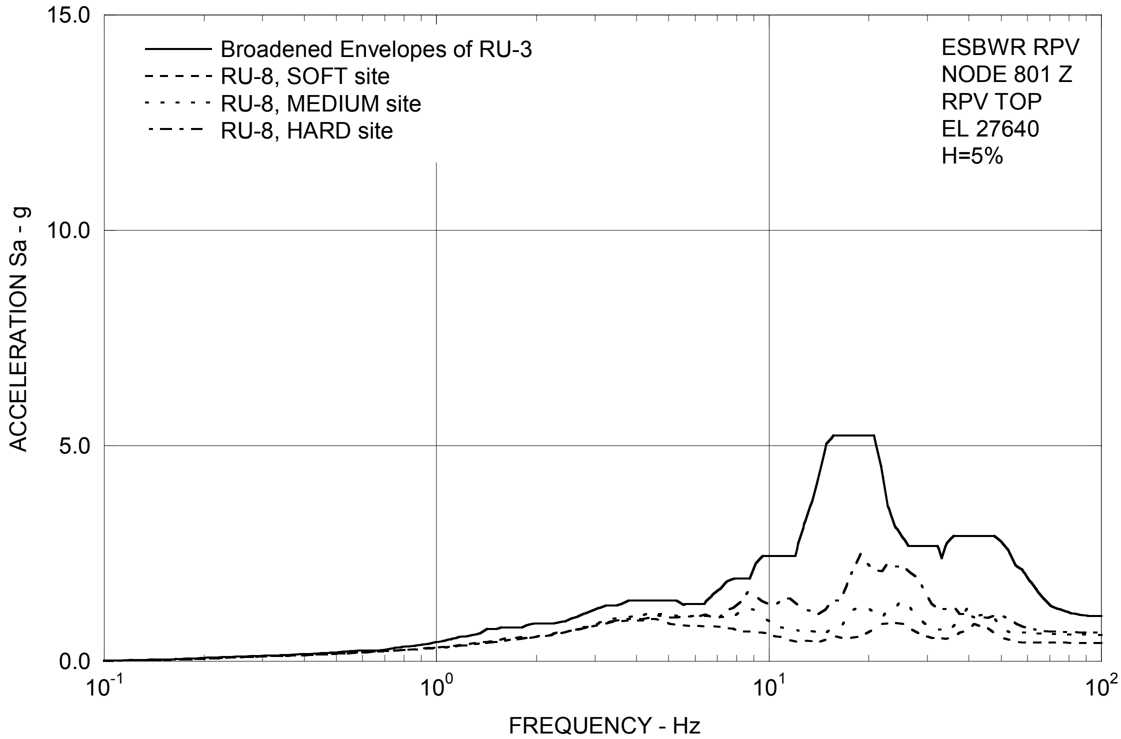


Figure 3.8-94(32) FRS (Compared with the DAC3N) – RPV Top Z

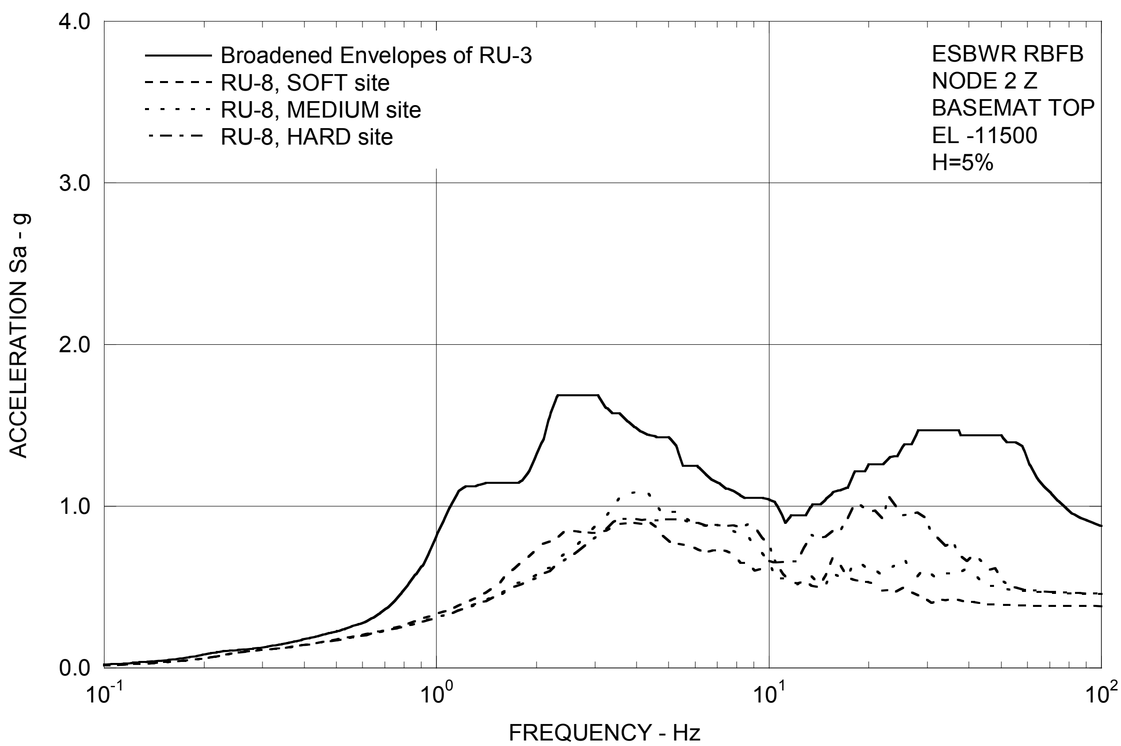


Figure 3.8-94(33) FRS (Compared with the DAC3N) – RBFB Basemat Z

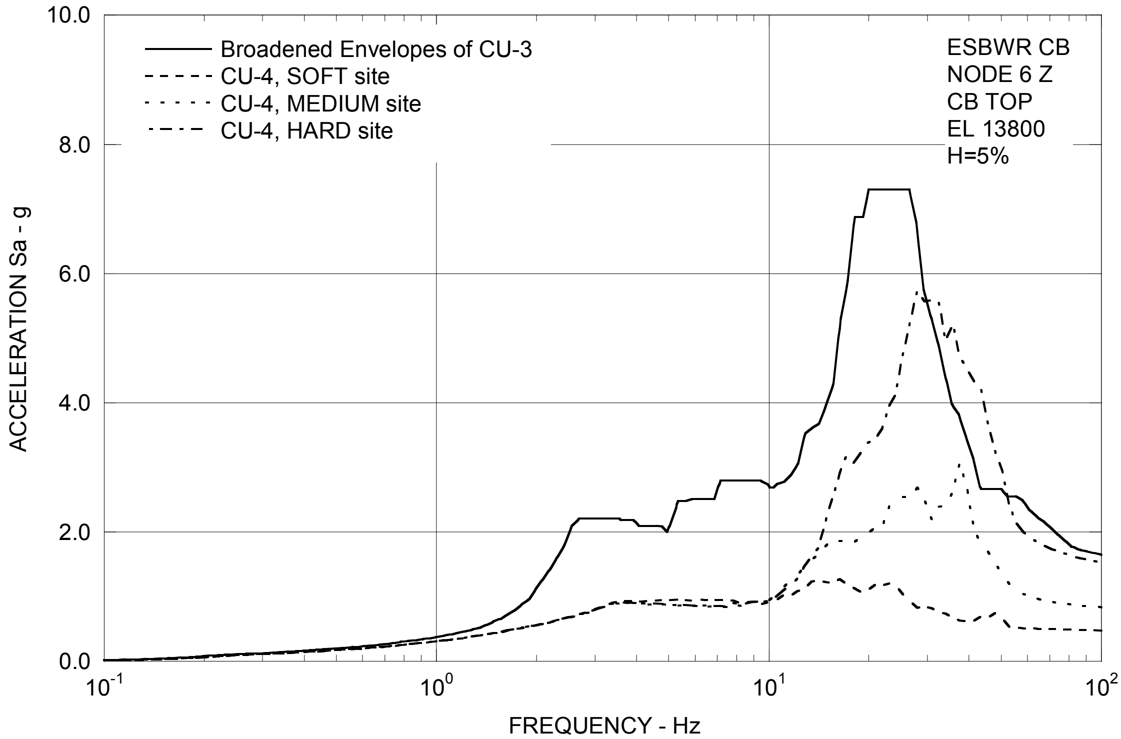


Figure 3.8-94(34) FRS (Compared with the DAC3N) – CB Top Z

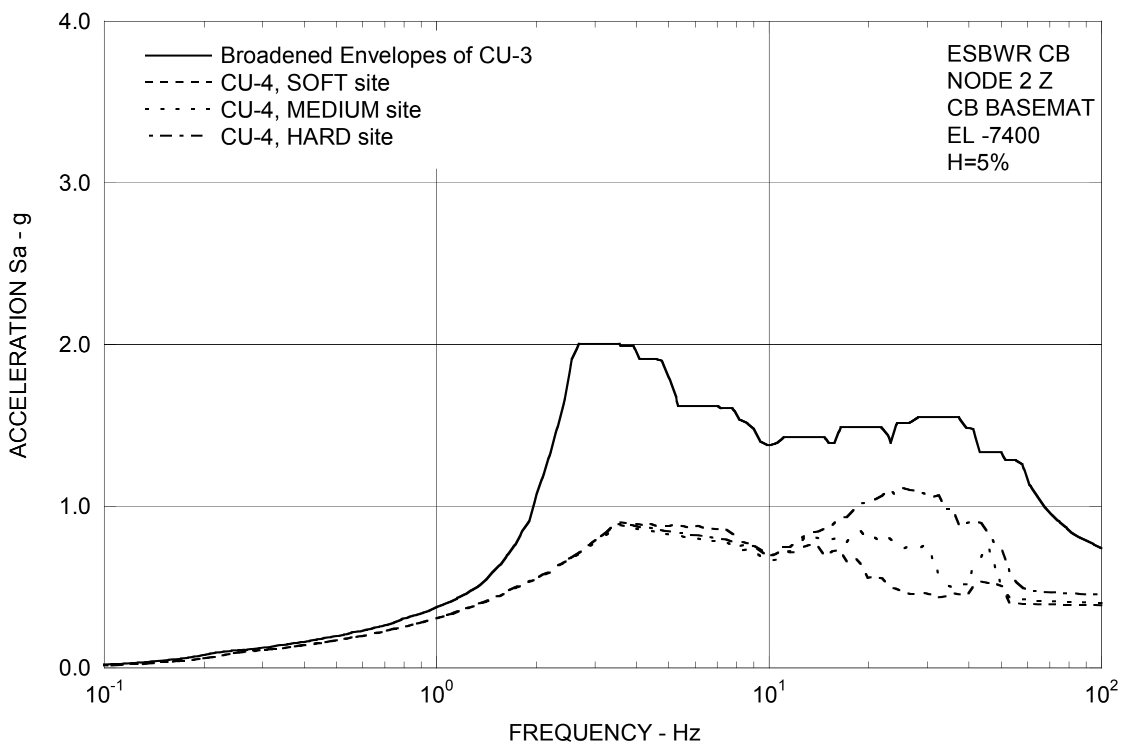


Figure 3.8-94(35) FRS (Compared with the DAC3N) – CB Basemat Z

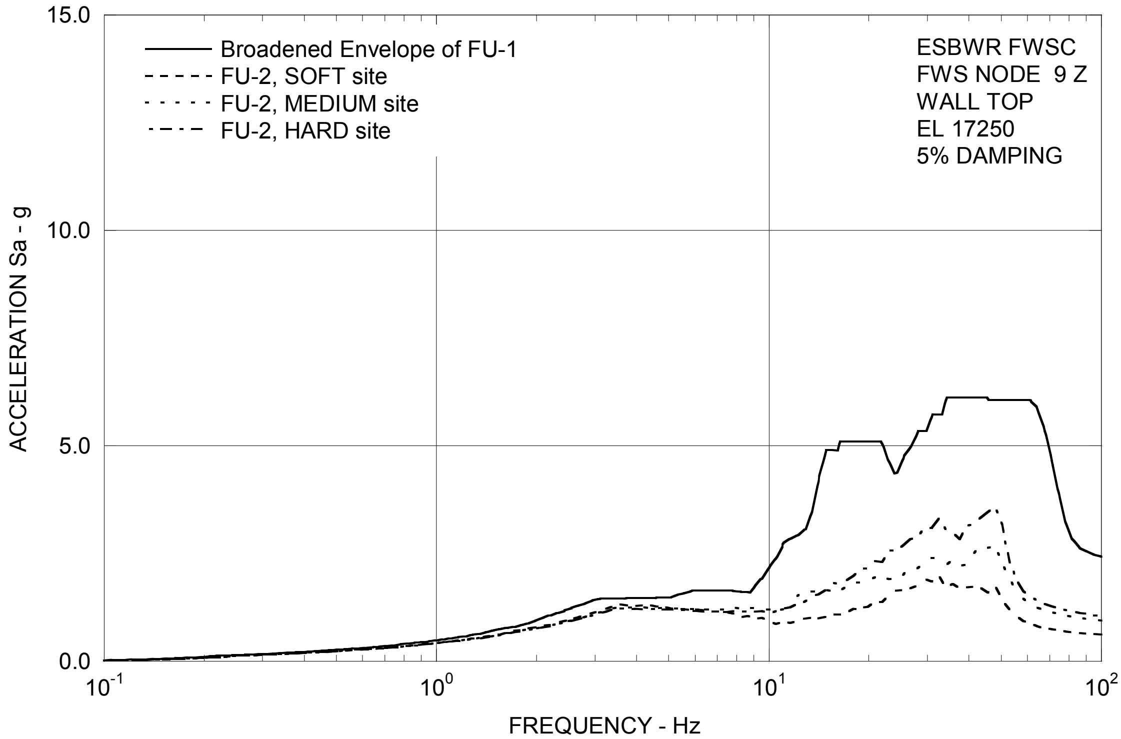


Figure 3.8-94(36) FRS (Compared with the DAC3N) – FWS Wall Top Z

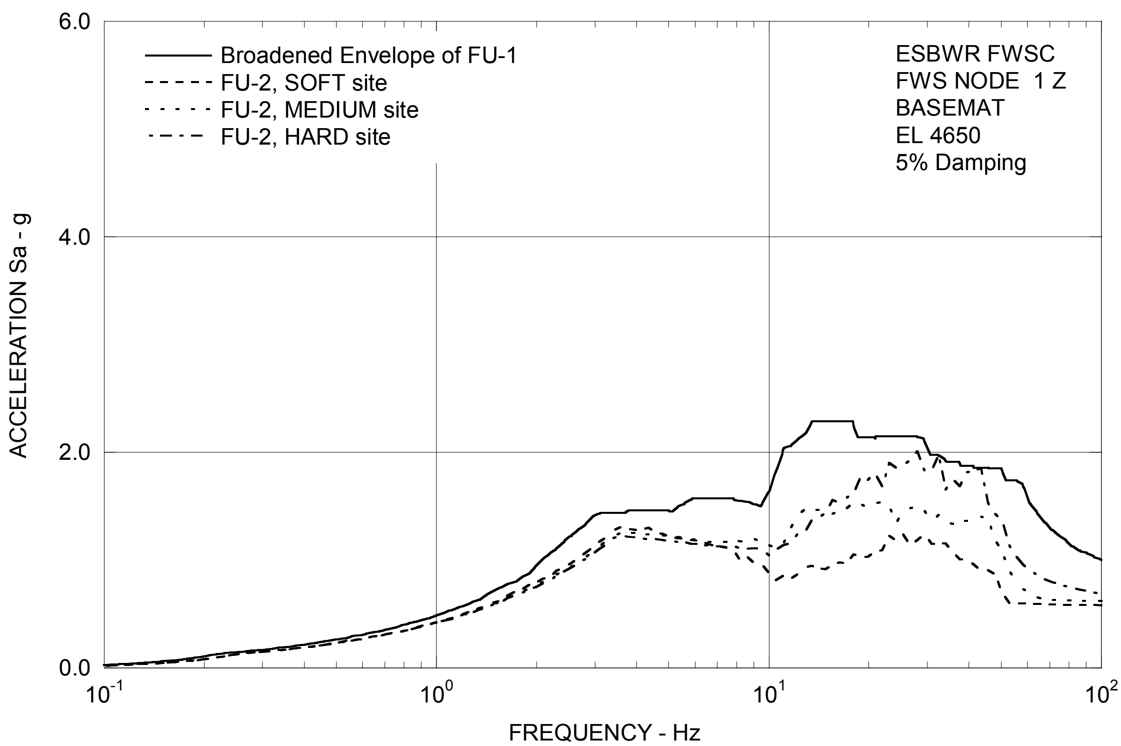


Figure 3.8-94(37) FRS (Compared with the DAC3N) – FWS Basemat Z

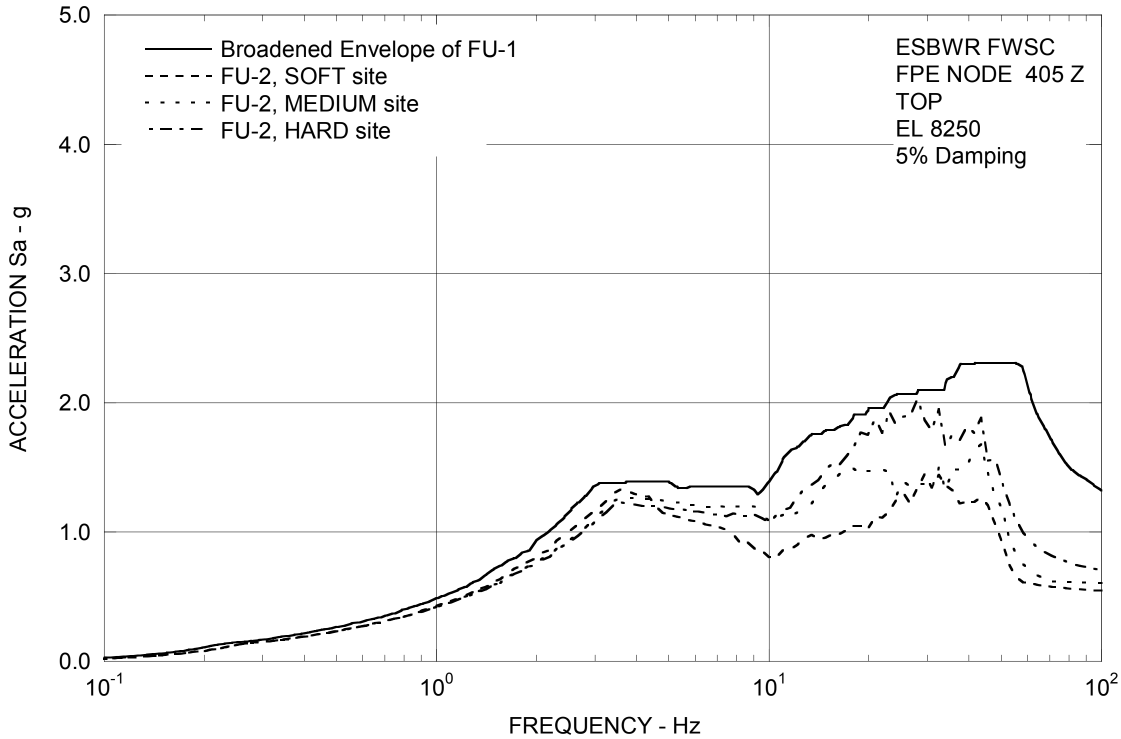


Figure 3.8-94(38) FRS (Compared with the DAC3N) – FPE Top Z

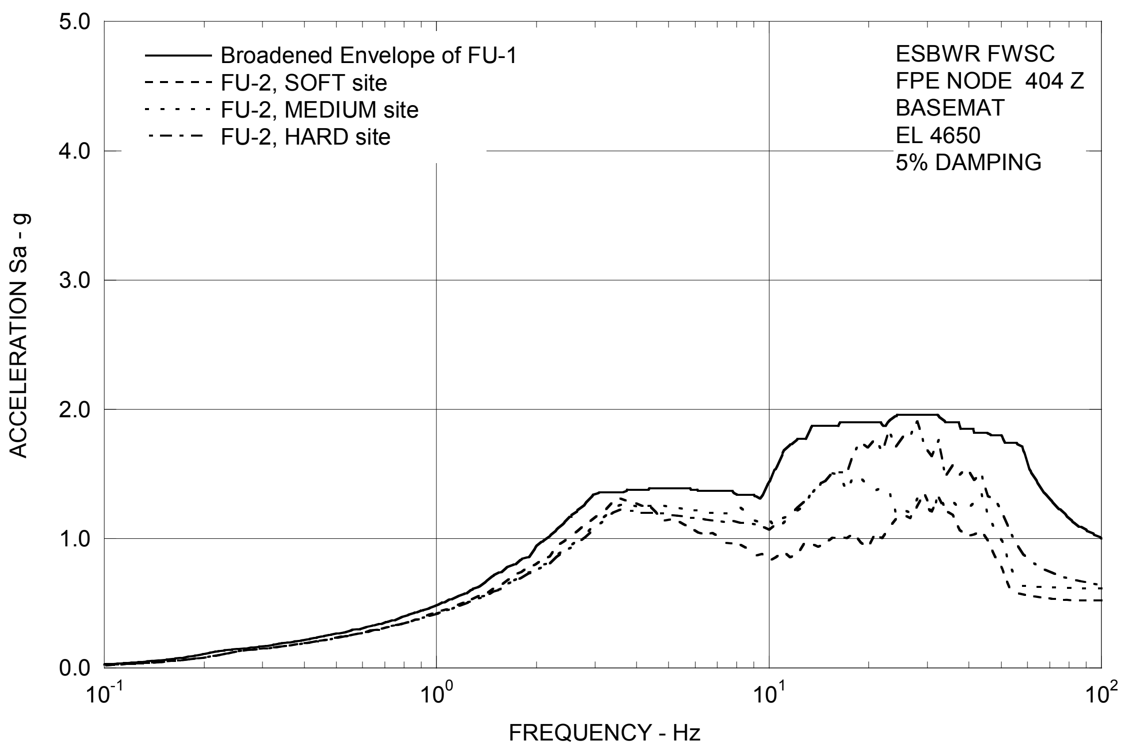


Figure 3.8-94(39) FRS (Compared with the DAC3N) – FPE Basemat Z

DCD Impact

Markups of DCD Tier 1 Table 5.1-1, DCD Tier 2 Subsections 3.7.1.1.3, 3A.5, 3A.5.2, 3A.6, 3A.8.7, 3A.9.3, 3C.7.1.3, 3C.7.2.1 and 3C.7.2.3, Tables 2.0-1, 3.7-3, 3A.6-1, 3A.8.7-1, 3G.1-58, 3G.2-27 and 3G.4-23 and Figures 3A.8.7-1a through 3A.8.7-1l, 3A.8.7-2a through 3A.8.7-2l, 3A.8.7-3a through 3A.8.7-3l, 3A.9-1g, 3A.9-1l, 3A.9-2g and 3A.9-3g were provided to the NRC in MFN 06-407 S13, dated 2/20/09.

NRC RAI 3.8-94, Supplement 4

Based on the review of GEH RAI 3.8-94 S03 response, presented in GEH letter dated February 20, 2009, GEH is requested to address the items described below.

A) As described on page 10 of 34 in the RAI response, the evaluation of peak toe pressure is made considering the bearing pressures due to the three perpendicular earthquake directions at only three time steps and not at every time step throughout the time history. The three time steps correspond to the time when Mx is maximum, when My is maximum, and when V is maximum. At each of these three time steps, the other two corresponding forces are utilized. GEH is requested to provide the technical basis why this approach is considered to be acceptable since at other time steps, where Mx, My, and V may not be maximum values, the resulting bearing pressures may actually be higher. Typically, a bearing pressure time history analysis should be performed at every time steps using algebraic summation or alternatively, the bearing pressures due to the three maximum forces may be combined by the SRSS method.

B) As described on page 11 of 34 in the RAI response, the calculation of bearing pressure is performed for one horizontal and vertical directions (i.e., two dimensional evaluation) using the "Energy Balance Method." Then another calculation is performed for the maximum bearing pressure contribution from the other horizontal earthquake direction. The total bearing pressure is then determined by the addition of these two values. GEH is requested to describe and identify the source of the specific "Energy Balance Method" being used to calculate the bearing pressure for this evaluation. Also, explain how the contact lengths CL and CW shown on page 10 of 33 are determined.

C) The forces used to calculate the maximum soil bearing pressures were obtained from the SASSI analyses. These analyses consider that the soil and foundation are integrally connected. However, the bearing pressure calculations on page 11 of 34 show that uplift occurs. Describe the extent of the maximum uplift that occurs in SASSI (denoted by tension in the soil springs), recognizing that this region could expand further if the tension springs would be released using a different computer code. Provide the technical basis for using these seismic loads in the bearing and sliding calculations from the SASSI analyses without consideration of the effects of uplift on the seismic demand loadings. Alternatively, an analysis that considers the nonlinear effect of liftoff due to the three input directions applied simultaneously can be considered.

D) In Item 3 on page 12 of 34 (Evaluation of Results), GEH indicates that the resulting toe pressures from the two layered soil cases (L2 and L4) are large as compared to those of the other generic soil cases. GEH deduces that this result may be due to the fact that large velocity contrasts exist in the layers of these cases (greater than 2.5). In the last paragraph on this page, the statement is made that "the best estimate low strain profile can be used because only the velocity ratio is of interest". However, it is not clear if this difference is in fact the only cause or even the primary *cause of the computed large peak responses. Other issues (such as the reduction in site radiation damping due to layering effects, ratios of the velocity to the bedrock velocities below, impact of layer thickness on the computed site amplification, etc.) may in fact contribute to such large responses. Provide detailed information to indicate that (a) the velocity ratio is the primary parameter controlling such high amplifications in toe pressure, and (b) the value of the site parameters that will lead to acceptable levels of peak toe pressure. Also see requested information in the new RAIs 3.7-70 and 71, which relate to this issue.*

E) *In the same discussion (Item 3 on page 12 of 34 - Evaluation of Results), a new site interface parameter, for the maximum ratio of soil shear wave velocity in adjacent layers, will be added to the DCD. The RAI response states that "The ratio is the average velocity of the bottom layer divided by the average velocity of the top layer." Provide a description of how the average shear wave velocity is calculated and include it in the appropriate locations in the DCD (i.e., DCD Tier 2, Section 2 as well as Table 2.0-1, and DCD Tier 1, Table 5.1-1). Also, provide the technical basis for this definition of the average shear wave velocity and explain how it would compare to the results obtained by properly treating multiple layers of varying shear wave velocities.*

F) *In the same discussion (Item 3 on page 12 of 34 - Evaluation of Results), the statement is made that "this velocity ratio condition does not apply to the FWSC nor to the RB/FB and CB if founded on rock-like material having a shear wave velocity of 1067 m/sec (3500 ft/sec) or higher." The definition of "rock" material, following the guidance of the SRP Section 3.7.2, associated with SSI evaluations is not 3,500 fps but 8,000 fps after which SSI effects are considered small. Provide the numerical results available to indicate that the computation of maximum toe pressure is not impacted by the velocity ratios for cases where the layer beneath the basemat has velocities greater than 3,500 fps.*

G) *In DCD Tier 1, Table 5.1-1 and DCD Tier 2, Table 2.0-1, the descriptions provided for minimum static and minimum dynamic bearing capacity are not clear. These requirements should be specified as "maximum bearing demand" not "minimum bearing capacity" since these values were obtained from the envelope of the elastic SASSI results applied to the liftoff calculations. The COL applicant then needs to determine the allowable bearing pressure based on the site-specific soil "bearing capacity" divided by the factor of safety appropriate for the design load combination. Therefore, the DCD should be revised to capture the following items: (1) present the values as maximum static bearing demand and maximum dynamic bearing demand, and (2) expand the footnote applicable to these values to state that the allowable bearing pressure shall be developed from the site-specific bearing capacity divided by a factor of safety appropriate for the design load combination.*

H) *In Section 3A.8.8, there is no indication that the torsional seismic effects are included in developing the soil pressures for the design of the foundation walls, along with the wall pressures from the translational seismic loadings. Explain how* *the torsional seismic effects have been included in the design of the foundation walls.*

I) *In several of the enveloping floor response spectra (e.g., Figures 3.8-94(19), (20), and (22), the staff noted that "valleys" exist between successive peaks in the low frequency region, up to approximately 5 Hz. If the spectral peaks are influenced by site conditions, it is the staff's position that these "valleys" should be filled-in to accommodate the expected variability in site shear wave profiles that may be encountered for this generic design. GEH is requested to provide an explanation why these valleys have not been filled-in.*

J) *In item (3) on page 14 of 34, Table 3.8-94(5) – Maximum Dynamic Soil Bearing Stress Involving SSE + Static, the bearing pressures under soft, medium and hard soils for each of the three structures (RB/FB, CB, and FWSC) are presented. For the CB, the tabulated bearing pressures are 0.44 MPa for soft, 2.2 MPa for medium, and 0.42 MPa for hard soils. These values show a very large variation between the medium soil values and the other two values, unlike the RB/FB and FWSC, where a more gradual variation exists. Therefore, GEH is requested to explain why the bearing pressure for the CB medium soil case varies by a factor of five times from the soft and hard soil cases.*

<u>GEH Response</u>

A) A refined bearing pressure time history analysis for all time steps has been performed. The maximum dynamic soil bearing pressures obtained from this analysis are shown in Table 3.8-94(6). The results of all SASSI cases including layered site soil Cases L-2 and L-4 are included in the analysis. DCD Tier 1 Table 5.1-1 and DCD Tier 2 Tables 2.0-1, 3G.1-58 and 3G.2-27 will be revised in Revision 6 to include these maximum dynamic soil bearing pressures. Please refer to GEH's response to NRC RAI 3.8-94 S04, Item D) below.

B) The Energy Balance Method is based on the assumption that the soil strain energy associated with the rocking of the basemat is the same for the linear response ignoring the uplift effect and for the nonlinear response considering the uplift effect. The source of the Energy Balance Method is a technical paper entitled "Simplified Methods for Predicting Seismic Basemat Uplift of Nuclear Power Plant Structures," by W.S. Tseng and D.D. Liou, presented at the 6th International Conference on SMiRT, August 1981. This technical paper is DCD Tier 2 Reference 3G.1-2.

   Contact length CL is a calculated quantity from the Energy Balance calculation. Contact length CW is the width of the foundation.

C) The Energy Balance Method takes into account the effects of uplift on the linear response calculated by SASSI.

   In the technical paper "Simplified Methods for Predicting Seismic Basemat Uplift of Nuclear Power Plant Structures," by W.S. Tseng and D.D. Liou in GEH's response to NRC RAI 3.8-94 S04, Item B), the Energy Balance Method was developed on the basis of a comparison of results of both linear and nonlinear time history analyses. It is concluded that the proposed simplified method provides a reasonable estimate using the linear analysis results when the amount of uplift is within 50% of the basemat dimension.

The SASSI results for the RB/FB layered site soil Case RL-2 are examined for the extent of maximum uplift when the bearing pressure reaches maximum in combination with dead loads and buoyancy. As shown in Figure 3.8-94(40), the extent of uplift is not extensive and a large portion of the basemat remains in contact with the soil.

D) Layered site soil Cases L-2 and L-4 are no longer excluded from the foundation stability evaluation, and all layered site soil cases are considered in the ESBWR Standard Plant design. DCD Tier 2 Subsections 3A.8.7 and 3A.9.3 will be revised in Revision 6 to include layered site soil Cases L-2 and L-4. Please refer to GEH's response to NRC RAI 3.8-94 S04, Item A).

E) The new ESBWR Standard Plant site interface parameter for the maximum ratio of soil shear wave velocities in adjacent layers, as proposed in GEH's response to NRC RAIs 3.8-94 S03 (MFN 06-407 S13, dated 2/20/09) and 3.8-96 S03 (MFN 06-407 S14, dated 2/20/09), will not be included in Revision 6 of DCD Tier 2 Table 2.0-1. Please refer to GEH's response to NRC RAI 3.8-94 S04, Item D).

F) Please refer to GEH's response to NRC RAI 3.8-94 S04, Item E).

G) In Revision 6 of DCD Tier 1 Table 5.1-1, DCD Tier 2 Section 2.0 and DCD Tier 2 Table 2.0-1, the descriptions "Minimum Static Bearing Capacity" and "Minimum Dynamic Bearing Capacity" will be changed to "Maximum Static Bearing Demand" and "Maximum Dynamic Bearing Demand", respectively. DCD Tier 1 Table 5.1-1 Note (2) and DCD Tier 2 Table 2.0-1 Note (7) will be expanded to state that the allowable bearing pressure is developed from the site-specific bearing capacity divided by a factor of safety appropriate for the design load combination.

H) Torsional seismic effects have been included in the design of the foundation walls. The SASSI model used is a 3D model including the torsional degree of freedom. Therefore, the calculated soil pressures include the torsional effects. To evaluate the torsional contribution, the SASSI calculated pressures at two ends and at the center of the wall are shown in Figures 3.8-94(41) and 3.8-94(42) for RB/FB layered site soil Case RL-2 and the CB layered site soil Case CL-2, respectively. As shown, the torsional contribution is not significant, since pressures are similar along the wall width. For wall design the SASSI calculated pressures along the wall width at a given elevation are averaged and applied as uniform pressures. Furthermore, as stated in DCD Tier 2 Section 3A.8.8, the wall design pressures are the envelope of the SASSI results and the ASCE 4-98 elastic solution. Although the torsional effect is not addressed in the ASCE 4-98 elastic solution, the inclusion of the ASCE 4-98 results in the design envelope provides additional margins above the SASSI results.

Hence, the foundation walls are adequately designed for torsion-induced seismic soil pressures.

I) Sufficient levels of conservatism are already built into the design floor response spectra in the form of enveloping and peak broadening the results of a sufficient number of generic sites under a bounding Certified Seismic Design Response Spectra (CSDRS) ground input motion. To provide additional margins for sites that might have predominant SSI frequencies at the spectral valleys, the spectral valleys of the site enveloping floor response spectra are further filled-in for the generic design floor response spectra. As an example, the original enveloping floor response spectra at the Vent Wall top, which is the same location as shown in Figure 3.8-94(18), is shown in Figure 3.8-94(43) together with its respective enveloping floor response spectra having the spectral valleys filled-in. The site enveloping floor response spectra in DCD Tier 2 Subsection 3A.9.2 will be replaced in Revision 6 with enveloping floor response spectra having the spectral valleys filled-in. DCD Tier 2 Subsections 3A.8.7 and 3A.9.2 will be revised in Revision 6 for clarification with regards to the spectral valleys being filled-in.

J) According to the SASSI results, the vertical seismic forces for the CB medium site condition are larger than those for the soft and hard site conditions. When a large vertical seismic force is applied upward, the contact length is reduced since the net downward vertical load is decreased. As a result, the bearing pressure at the toe is increased. Table 3.8-94(7) shows the seismic forces when the maximum bearing pressure for the CB occurs. The medium case has a large vertical response. This is the reason why the bearing pressure for the CB medium site condition is larger than other site conditions.

To gain more insight about the vertical load contribution to bearing pressures for each site condition, Table 3.8-94(8) has been prepared to show the seismic forces at the time when the vertical seismic load is maximum (Vmax). The value of Vmax for the medium site condition is close to that for the hard site. However, Mx and My for the medium site condition are much larger than the hard site condition. Thus, the bearing pressure for the medium site condition becomes larger.

Table 3.8-94(6) Maximum Dynamic Soil Bearing Pressure Involving SSE + Static (MPa)

Building	Site Condition		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard (V_s ≥ 1700 m/sec)
RB/FB	1.1	2.7	1.1
CB	0.50	2.2	0.42
FWSC	0.46	0.69	1.2

Table 3.8-94(7) Seismic Forces at the Time of Maximum Bearing Pressure for CB

	Soft		Medium		Hard	
	Downward	**Upward**	**Downward**	**Upward**	**Downward**	**Upward**
Time (sec)	7.355	7.340	7.155	7.150	6.685	13.60
Mx (MN-m)	302.9	345.6	161.5	160.2	80.5	109.5
My (MN-m)	249.0	157.8	117.3	114.1	147.0	112.8
V (MN)	40	43	91	92	56	10
Total Vertical Load (MPa)	235	51	286	2	251	84
Bearing Pressure (MPa)	0.50	0.28	0.48	2.19	0.42	0.19

Table 3.8-94(8) Seismic Forces at the Time of Maximum Vertical Seismic Load (Vmax) for CB

	Soft		Medium		Hard	
	Downward	**Upward**	**Downward**	**Upward**	**Downward**	**Upward**
Time (sec)	7.150		7.150		7.145	
Mx (MN-m)	85.8		160.2		6.7	
My (MN-m)	69.0		114.1		47.1	
Vmax (MN)	79		92		90	
Total Vertical Load (MPa)	274	15	287	2	285	5
Bearing Pressure (MPa)	0.43	0.07	0.48	2.19	0.41	0.02

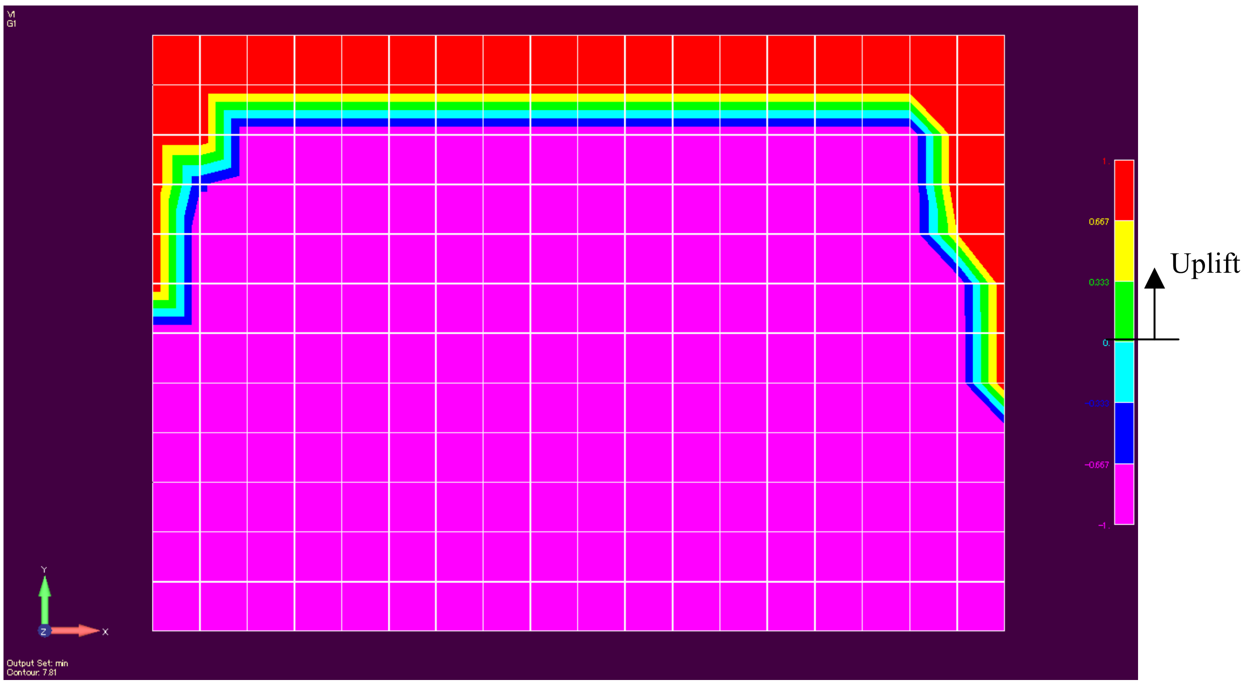


Figure 3.8-94(40) RB/FB Layered Site Soil Case RL-2, Time = 7.810 sec

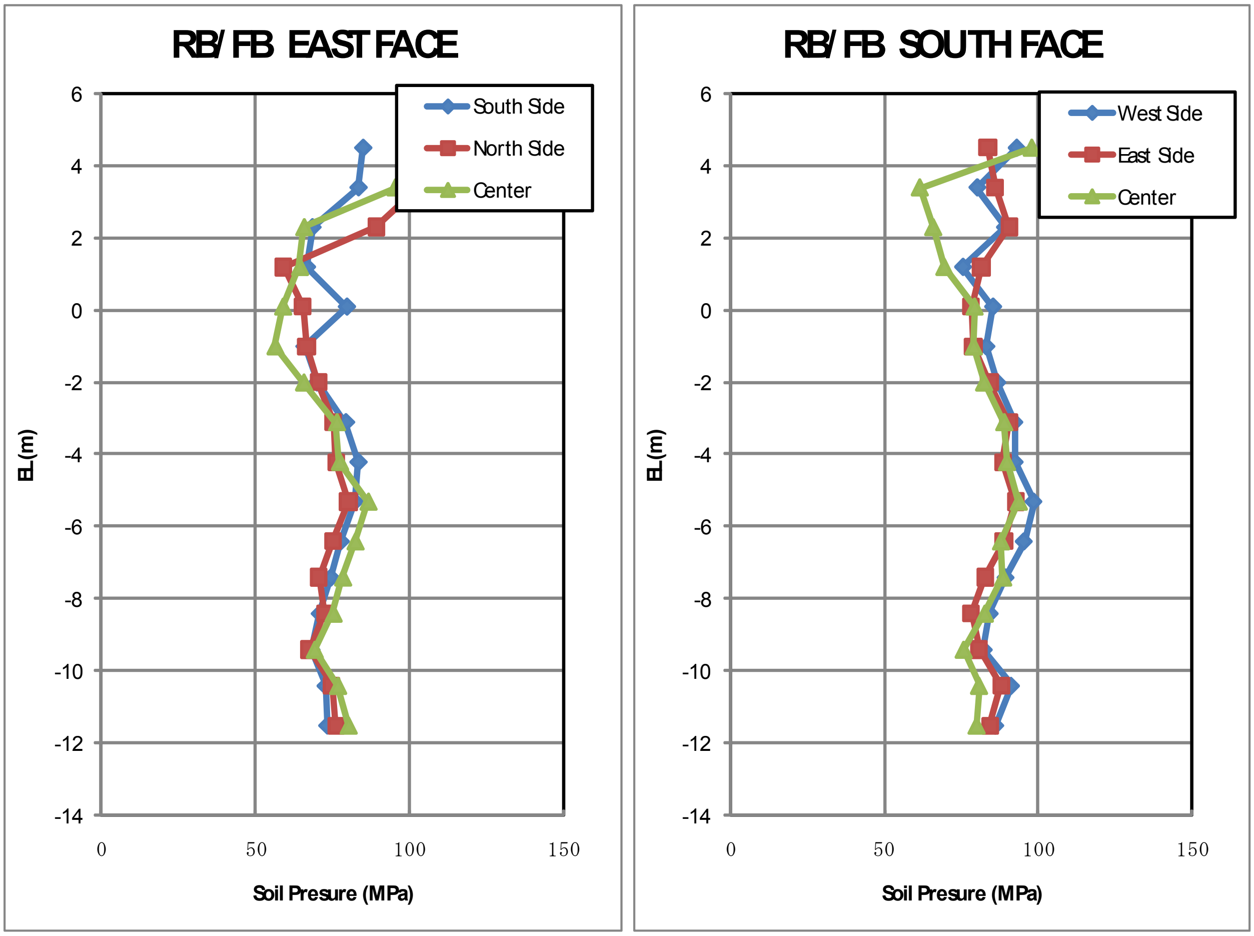


Figure 3.8-94(41) Soil Pressure in SASSI Analysis – RB/FB Layered Site Soil Case RL-2

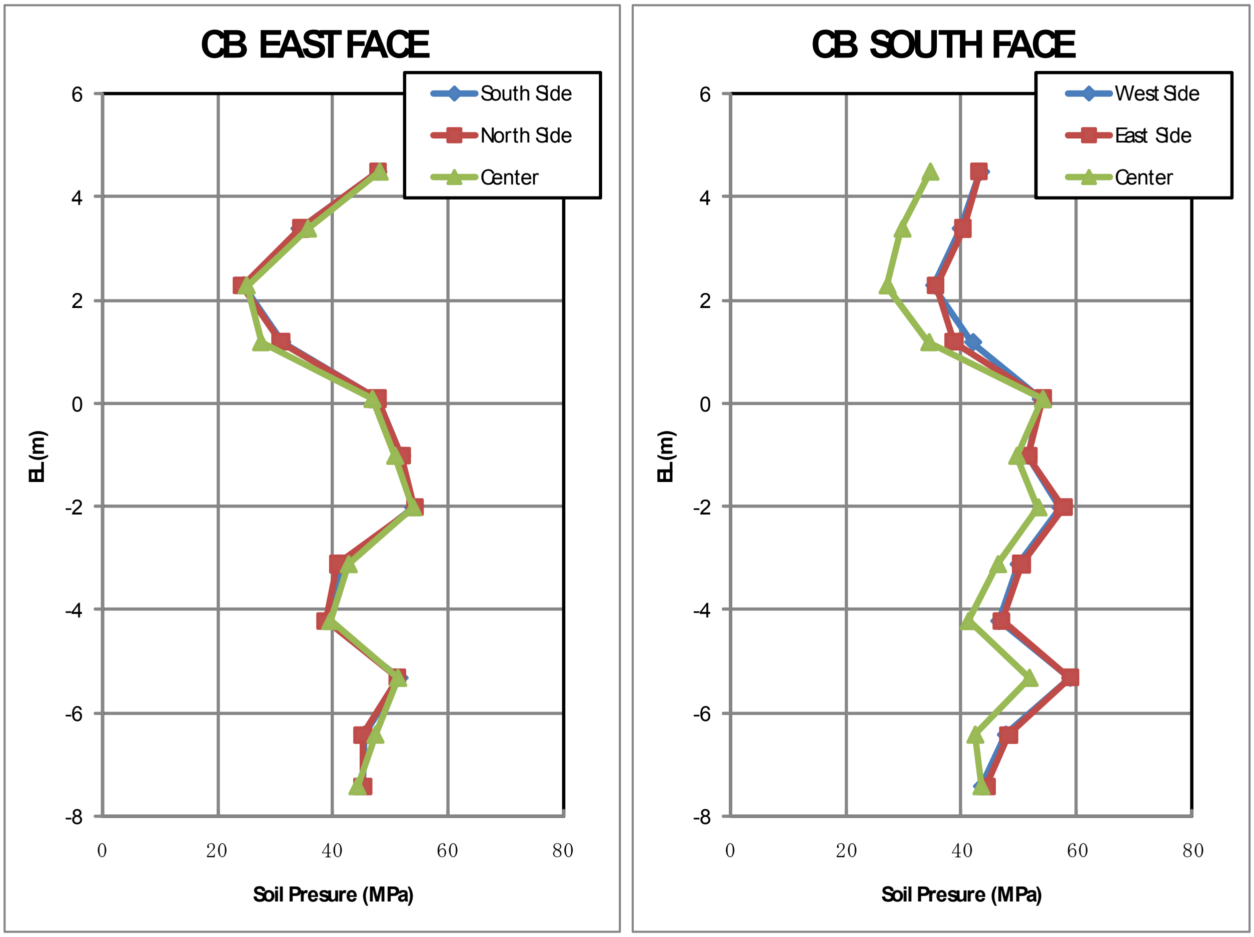


Figure 3.8-94(42) Soil Pressure in SASSI Analysis – CB Layered Site Soil Case CL-2

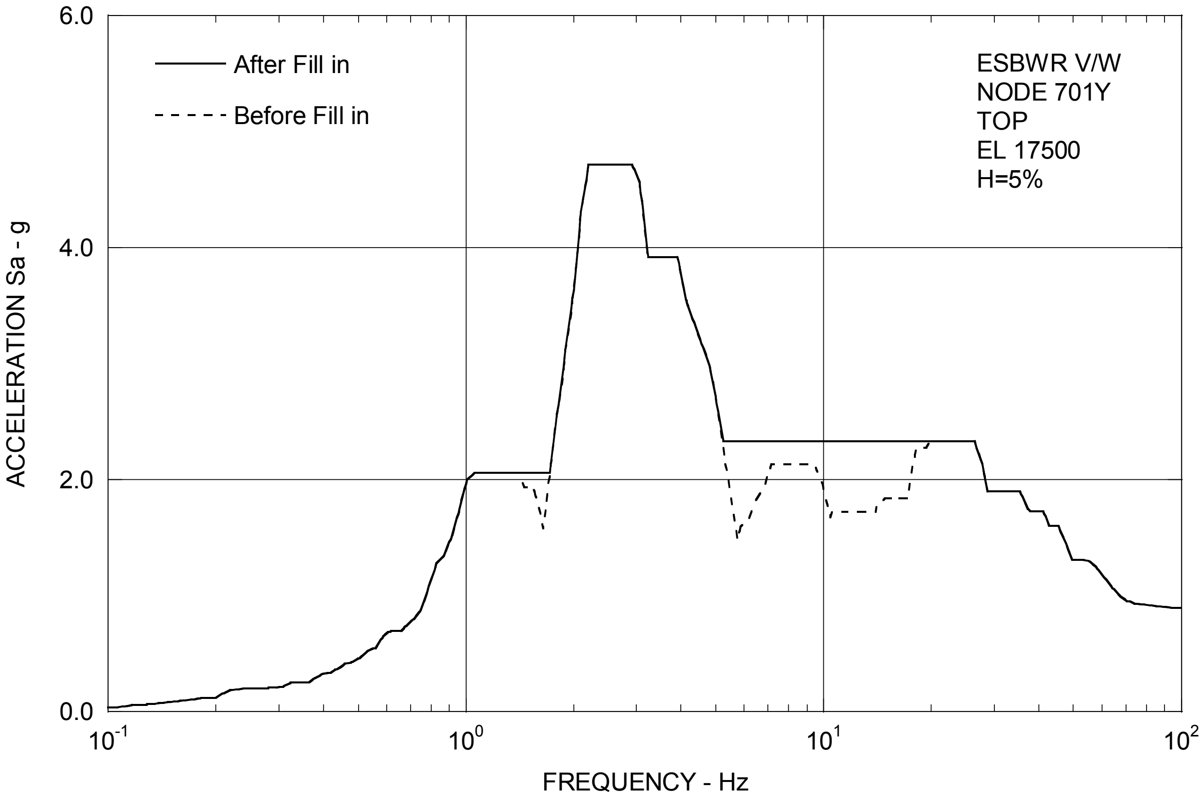


Figure 3.8-94(43) Enveloping Floor Response Spectra – Vent Wall Top Y

DCD Impact

DCD Tier 1 Table 5.1-1 will be revised in Revision 6 as noted in the attached markups.

DCD Tier 2 Section 2.0, Subsections 3A.8.7, 3A.9.2 and 3A.9.3, Tables 2.0-1, 3G.1-58, 3G.2-27 and 3G.4-23 and Figures 3A.9-1a through 3A.9-1l, 3A.9-2a through 3A.9-2l and 3A.9-3a through 3A.9-3l will be revised in Revision 6 as noted in the attached markups.

Enclosure 2

MFN 09-388

Response to Portion of NRC Request for

Additional Information Letter No. 323

Related to ESBWR Design Certification Application

DCD Markups for RAI Number 3.8-94 S04

Table 5.1-1
Envelope of ESBWR Standard Plant Site Parameters (continued)

Soil Properties: (6)	- ~~Minimum~~ Maximum Static Bearing ~~Capacity~~Demand: (2) Reactor/Fuel Building: 699 kPa (14,600 lbf/ft^2) Control Building: 292 kPa (6,100 lbf/ft^2) Fire Water Service Complex: 165 kPa (3,450 lbf/ft^2) - ~~Minimum~~ Maximum Dynamic Bearing ~~Capacity~~Demand (SSE + Static): (2) **Reactor/Fuel Building:** Soft: ~~27~~1100 kPa (23~~56~~,4000 lbf/ft^2) Medium: ~~73~~2700 kPa (~~152,5~~56,400 lbf/ft^2) Hard: ~~54~~1100 kPa (~~11~~23,0~~8~~00 lbf/ft^2) **Control Building:** Soft: ~~280~~500 kPa (~~58~~10,500 lbf/ft^2) Medium: 22~~5~~00 kPa (46~~52~~,0~~3~~00 lbf/ft^2) Hard: ~~2~~42~~0~~0 kPa (8~~50~~,8~~2~~00 lbf/ft^2) **Firewater Service Complex (FWSC):** Soft: 4~~4~~60 kPa (9,6~~2~~00 lbf/ft^2) Medium: 69~~54~~0 kPa (14~~1~~,~~3~~400 lbf/ft^2) Hard: 120~~67~~0 kPa (25~~14~~,1~~0~~00 lbf/ft^2) - Minimum Shear Wave Velocity: (3) 300 m/s (1000 ft/s) - Liquefaction Potential: Seismic Category I Structures: None under footprint of Seismic Category I structures resulting from site-specific SSE. - Angle of Internal Friction ≥ 35~~0~~ degrees
Seismology:	- SSE Horizontal Ground Response Spectra: (4) See Figure 5.1-1 - SSE Vertical Ground Response Spectra: (4) See Figure 5.1-2
Hazards in Site Vicinity:	- Site Proximity Missiles and Aircraft: < about 10^{-7} per year - Volcanic Activity: None - Toxic Gases: None *
* Maximum toxic gas concentrations at the Main Control Room (MCR) HVAC intakes:	< toxicity limits
Required Stability of Slopes:	- Factor of safety for static (non-seismic) loading 1.5 - Factor of safety for dynamic (seismic) loading due to site-specific SSE 1.1

Table 5.1-1
Envelope of ESBWR Standard Plant Site Parameters (continued)

Meteorological Dispersion (X/Q): (continued)	Technical Support Center X/Q:*		
	Reactor Building		
	0-2 hours:	1.00E-03 s/m^3	1.00E-03 s/m^3
	2-8 hours:	6.00E-04 s/m^3	6.00E-04 s/m^3
	8-24 hours:	3.00E-04 s/m^3	3.00E-04 s/m^3
	1-4 days:	2.00E-04 s/m^3	2.00E-04 s/m^3
	4-30 days:	1.00E-04 s/m^3	1.00E-04 s/m^3
	Turbine Building		
	0-2 hours:	2.00E-03 s/m^3	2.00E-03 s/m^3
	2-8 hours:	1.50E-03 s/m^3	1.50E-03 s/m^3
	8-24 hours:	8.00E-04 s/m^3	8.00E-04 s/m^3
	1-4 days:	6.00E-04 s/m^3	6.00E-04 s/m^3
	4-30 days:	5.00E-04 s/m^3	5.00E-04 s/m^3
	Passive Containment Cooling System / Reactor Building Roof		
	0-2 hours:	2.00E-03 s/m^3	2.00E-03 s/m^3
	2-8 hours:	1.10E-03 s/m^3	1.10E-03 s/m^3
	8-24 hours:	5.00E-04 s/m^3	5.00E-04 s/m^3
	1-4 days:	4.00E-04 s/m^3	4.00E-04 s/m^3
	4-30 days:	3.00E-04 s/m^3	3.00E-04 s/m^3

Notes:

(1) ~~The design of the Radwaste Building uses a set of design parameters that are specified in Regulatory Guide 1.143, Table 2, Class RW-IIa instead of the corresponding values given in this table for all parameters except as follows: (1) Tornado: Wind Speeds, Radius, Pressure Drop, and Rate of Pressure Drop; (2) Seismology: Horizontal and Vertical Ground Spectra: See Figures 5.1-1 and 5.1-2.~~(Deleted)

(2) At the foundation level of Seismic Category I structures. The static bearing pressure is the average pressure. The dynamic bearing pressure is the toe pressure. To compare with the maximum bearing demand, the allowable bearing pressure is developed from the site-specific bearing capacity divided by a factor of safety appropriate for the design load combination. ~~For~~ The maximum ~~minimum~~ dynamic bearing ~~capacity~~ demand to be compared with the site-specific ~~application, use~~allowable dynamic bearing pressure is the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level. The shear wave velocities of soft, medium and hard soils are 300 m/sec (1000 ft/sec), 800 m/sec (2600 ft/sec) and greater than or equal to 1700 m/sec (5600 ft/sec), respectively.

(3) This is the ~~equivalent uniform~~minimum shear wave velocity of the supporting foundation material ~~(Veq) over the entire soil column at seismic strain, which is a lower bound value after taking into account uncertainties. Veq is calculated to achieve the same wave traveling time over the depth equal to the embedment depth plus 2 times the largest~~

2. SITE CHARACTERISTICS

2.0 INTRODUCTION

This chapter defines the envelope of site-related parameters that the ESBWR Standard Plant is designed to accommodate. These parameters envelope most potential sites in the U.S. A list of the site envelope design parameters is given in Table 2.0-1.

Table 2.0-2 references the guidance in NUREG-0800 Standard Review Plan (SRP). Table 2.0-2 defines the limits imposed on the acceptance criteria in Section II of the various SRPs by (1) the envelope of site-related parameters that the ESBWR plant is designed to accommodate, and (2) the assumptions, both implicit and explicit, related to site parameters that were employed in the evaluation of the ESBWR design.

The requirements for site parameters for a standard design are contained in 10 CFR 52.47(a)(1)~~(iii)~~. A design certification applicant provides postulated site parameters for the design, and an analysis and evaluation of the design in terms of such parameters. The following demonstrate that the standard design meets the above criteria.

The specified site parameters are the top-level bounding site parameters useful in the selection of a suitable site for a facility referencing the ESBWR certified design. Because they were used in bounding evaluations of the certified design, they define the envelope of site parameters used for the design that must be considered for a site. When the site characteristics fall within the site parameter values, a facility built on the site is in conformance with the design certification. Appropriate values for site parameters have been selected that make the design suitable for many sites. All site parameters specified in Tier 1 have the same values as those presented in this chapter.

The analyses and evaluations of the design, considering the site parameters of Table 2.0-1, are contained in the various sections of this document. For example, the safe shutdown earthquake (SSE) parameters are used in structural and piping analyses in various sections of Chapter 3, atmospheric dispersion parameters are used in radiological analyses throughout Chapter 15, and the elevation parameter is used in the flooding analyses in Section 3.4.

Site parameters are specified for the following parameters:

- Maximum Ground Water Level
- Maximum Flood (or Tsunami) Level
- Precipitation (for roof design)
- Ambient Design Temperature
- Extreme Wind
- Tornado (maximum speed, pressure drop, missile spectrum, etc.)
- Maximum Settlement Values for Seismic Category I Buildings
- Soil Properties (maximum~~minimum~~ static bearing demand~~capacity~~, maximum~~minimum~~ dynamic bearing demand~~capacity~~, minimum shear wave velocity, liquefaction potential, angle of internal friction)

Table 2.0-1

Envelope of ESBWR Standard Plant Site Parameters (continued)

Soil Properties: [16]

- ~~Minimum~~ Maximum Static Bearing ~~Capacity~~Demand: (7)
  - Reactor/Fuel Building: 699 kPa (14,600 lbf/ft^2)
  - Control Building: 292 kPa (6,100 lbf/ft^2)
  - Firewater Service Complex: 165 kPa (3,450 lbf/ft^2)
- ~~Minimum~~ Maximum Dynamic Bearing ~~Capacity~~ Demand (SSE + Static): (7)
  - Reactor/Fuel Building:
    - Soft: ~~27~~1100 kPa (2~~5~~3~~6~~,~~4~~000 lbf/ft^2)
    - Medium: ~~73~~2700 kPa (~~152,5~~56,400 lbf/ft^2)
    - Hard: ~~54~~1100 kPa (~~11~~23,0~~8~~00 lbf/ft^2)
  - Control Building:
    - Soft: ~~280~~500 kPa (~~58,5~~10,500 lbf/ft^2)
    - Medium: 22~~5~~00 kPa (46~~52~~,0~~3~~00 lbf/ft^2)
    - Hard: ~~2~~42~~0~~0 kPa (8~~50~~,8~~2~~00 lbf/ft^2)
  - Firewater Service Complex (FWSC):
    - Soft: 4~~4~~60 kPa (9,6~~2~~00 lbf/ft^2)
    - Medium: 69~~54~~0 kPa (14~~1~~,~~3~~400 lbf/ft^2)
    - Hard: 120~~67~~0 kPa (25~~14~~,1~~0~~00 lbf/ft^2)
- Minimum Shear Wave Velocity: (8) 300 m/s (1000 ft/s)
- Liquefaction Potential:
  - Seismic Category I Structures: None under footprint of Seismic Category I structures resulting from site-specific SSE.
  - Other than Seismic Category I Structures: See Note (14)
- Angle of Internal Friction: ≥ 35~~0~~ degrees
- Backfill on sides of Seismic Category I structures (not applicable if the fill material is concrete)
  - Product of peak ground acceleration, Poisson's ratio and density: $\alpha(0.95\nu+0.65)\gamma$: 1220 kg/m^3 (76 lbf/ft^3) maximum
  - Product of at-rest pressure coefficient and density: $k_0\gamma$: 750 kg/m^3 (47 lbf/ft^3) minimum
  - Product of the difference of passive and active pressure coefficients and density: $(k_p-k_a)\,\gamma$: 1100 kg/m^3 (69 lbf/ft^3) minimum
- Backfill underneath FWSC against shear keys (not applicable if the fill material is concrete)
  - At-rest pressure coefficient: k_0' : 0.36 minimum
  - Difference of passive and active pressure coefficients: $(k_p-k_a)'$: 2.5 minimum

Notes for Table 2.0-1:

(1) The site parameters defined in this table are applicable to Seismic Category I, II, and Radwaste Building structures, unless noted otherwise.~~The design of the Radwaste Building uses a set of design parameters that are specified in Regulatory Guide 1.143, Table 2, Class RW-IIa instead of the corresponding values given in this table for all parameters except as follows: (1) Tornado: Winds Speeds, Radius, Pressure Drop, and Rate of Pressure Drop; (2) Seismology: Horizontal and Vertical Ground Spectra: See Figures 2.0-1 and 2.0-2.~~

(2) Probable maximum flood level (PMF), as defined in Table 1.2-6 of Volume III of Reference 2.0-4.

(3) Maximum speed selected is based on Attachment 1 of Reference 2.0-5, which summarizes the NRC Interim Position on Regulatory Guide 1.76. Concrete structures designed to resist Spectrum I missiles of SRP 3.5.1.4, Rev. 2, also resist missiles postulated in Regulatory Guide 1.76, Revision 1. Tornado missiles do not apply to Seismic Category II buildings. For the Radwaste building, the tornado missiles defined in Regulatory Guide 1.143, Table 2, Class RW-IIa apply.

(4) Based on probable maximum precipitation (PMP) for one hour over 2.6 km^2 (one square mile) with a ratio of 5 minutes to one hour PMP of 0.32 as found in Reference 2.0-3. ~~Roof scuppers and drains are designed independently to limit water accumulation on the roof to no more than 100 mm (4 in) during PMP conditions.~~The 48-hour probable maximum winter precipitation (PMWP) is based on Reference 2.0-6. See also Table 3G.1-2.

(5) ~~Maximum design roof load accommodates snow load and 48-hour probable maximum winter precipitation (PMWP) in References 2.0-2 and 2.0-6. Roof scuppers and drains are designed independently to limit water accumulation on the roof to no more than an average depth of 100 mm (4 in) during PMWP conditions.~~ See Reference 2.0-9 for the definition of normal winter precipitation and extreme winter precipitation events. See also Table 3G.1-2.

(6) Zero percent exceedance values are based on conservative estimates of historical high and low values for potential sites. Consistent with Reference 2.0-4, they represent historical limits excluding peaks of less than two hours. One and two percent annual exceedance values were selected in order to bound the values presented in Reference 2.0-4 and available Early Site Permit applications.

(7) At the foundation level of Seismic Category I structures. The static bearing pressure is the average pressure. The dynamic bearing pressure is the toe pressure. To compare with the maximum bearing demand, the allowable bearing pressure is developed from the site-specific bearing capacity divided by a factor of safety appropriate for the design load combination. The~~For~~ maximum~~minimum~~ dynamic bearing demand~~capacity~~ to be compared with the site-specific allowable dynamic bearing pressure is~~application, use~~ the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level. The shear wave velocities of soft, medium and hard soils are 300 m/sec (1000 ft/sec), 800 m/sec (2600 ft/sec) and greater than or equal to 1700 m/sec (5600 ft/sec), respectively.

from the mean. ~~(V_{eq}) over the entire soil column at seismic strain, which is a lower bound value after taking into account uncertainties. V_{eq} is calculated to achieve the same wave traveling time over the depth equal to the embedment depth plus 2 times the largest foundation plan dimension below the foundation as follows:~~ $\cancel{V_{eq} = \frac{\sum d_i}{\sum \frac{d_i}{V_i}}}$

~~where d_i and V_i are the depth and shear wave velocity, respectively, of the ith layer.~~ The ratio of the largest to the smallest shear wave velocity over the mat foundation width ~~at~~ of the supporting foundation ~~level~~ material does not exceed 1.7.

(9) Safe Shutdown Earthquake (SSE) design ground response spectra of 5% damping, also termed Certified Seismic Design Response Spectra (CSDRS), are defined as free-field outcrop spectra at the foundation level (bottom of the base slab) of the Reactor/Fuel and Control Building structures. ~~For ground surface founded Firewater Service Complex structures, the CSDRS is 1.35 times the values shown in Figures 2.0-1 and 2.0-2.~~ For the Firewater Service Complex, which is essentially a surface founded structure, the CSDRS is 1.35 times the values shown in Figures 2.0-1 and 2.0-2 and is defined as free-field outcrop spectra at the foundation level (bottom of the base slab) of the Firewater Service Complex structure.

(10) Values reported here are actually design criteria rather than site design parameters. They are included here because they do not appear elsewhere in the DCD.

(11) If a selected site has a X/Q value that exceeds the ESBWR reference site value, the COL applicant will address how the radiological consequences associated with the controlling design basis accident continue to meet the dose reference values provided in 10 CFR ~~50.34~~52.79(a)(1)(vi) and control room operator dose limits provided in General Design Criterion 19 using site-specific X/Q values.

(12) If a selected site has X/Q values that exceed the ESBWR reference site values, the release concentrations in Table 12.2-17 would be adjusted proportionate to the change in X/Q values using the stack release information in Table 12.2-16. In addition, for a site selected that exceeds the bounding X/Q or D/Q values, the COL applicant will address how the resulting annual average doses (Table 12.2-18b) continue to meet the dose reference values provided in 10 CFR 50 Appendix I using site-specific X/Q and D/Q values.

(13) Value was selected to comply with expected requirements of southeastern coastal locations.

(14) Localized liquefaction potential under other than Seismic Category I structures is addressed per SRP 2.5.4 in Table 2.0-2.

(15) Settlement values are long-term (post-construction) values except for differential settlement within the foundation mat. The design of the foundation mat accommodates immediate and long-term (post-construction) differential settlements after the installation of the basemat.

(16) For sites not meeting the soil property requirements, a site-~~-~~specific analysis is required to demonstrate the adequacy of the standard plant design.

soil layers. Regarding the lateral soil pressure, the analysis results are shown in Subsection 3A.8.8.

As shown in Table 3A.8.7-1 the basemat reaction shear forces calculated by DAC3N for case RU-3 without the embedment effect are conservative. To better predict interface loads with the supporting foundation medium, uniform sites are further analyzed using SASSI2000 with embedment included for RB/FB and CB and without embedment for surface founded FWSC. These SASSI2000 cases are designated as RU-8, CU-4 and FU-2 in Table 3A.6-1. The results of case RU-8 are also shown in Table 3A.8.7-1 and they are lower than RU-3 results as expected. The basemat interface loads for uniform sites considered in the foundation stability evaluation are those calculated by SASSI2000.

The SASSI2000 results of uniform sites are also compared with the DAC3N results for floor response spectra as discussed below.

Comparisons of response spectra are shown in Figures 3A.8.7-1a through 3A.8.7-1l, Figures 3A.8.7-2a through 3A.8.7-2l, and Figures 3A.8.7-3a through 3A.8.7-3l, respectively for X direction, Y direction, and Z direction.

As for the case of the RB/FB, it is found from the results that the responses for SASSI2000 cases are bounded by the broadened envelope responses of DAC3N cases in the whole frequency range. The responses of RU-8 hard site at vent wall top X direction (Figure 3A.8.7-1c) and refueling floor Z direction (Figure 3A.8.7-3a) are slightly higher around 20 Hz but the exceedance is negligibly small. Furthermore, the slightly higher responses at these two locations are bounded by the design envelope response spectra after the spectral valleys are filled-in as described in Section 3A.9.

On the other hand, the response spectra of a portion of the CB above ground and the FPE in the FWSC exceeded greater than 10% at the broadened envelope responses of the DAC3N cases in the higher frequency range.

Thus the SASSI2000 uniform site results of the CB and FWSC are included to obtain the enveloping design spectra (Section 3A.9).

The uniform site SASSI2000 results for seismic forces of building structural members are less than the DAC3N results, thus there is no impact on the design envelope loads.

3A.8.8 Effect of Lateral Soil Pressures

The lateral pressure computed from the equivalent static pressure analysis listed in ASCE 4-98 is used for the design soil pressure. To confirm that the ASCE 4-98 method is conservative, the soil pressures calculated from the SASSI2000 analysis for the layered sites described in Subsection 3A.8.6 are compared with the ASCE 4-98 method soil pressures in Figures 3A.8.8-1 through 3A.8.8-4.

It is found from the results that the SASSI2000 soil pressures are generally bounded by the ASCE 4-98 soil pressures; however, at the elevation close to the ground surface and the basemat elevation, the SASSI2000 soil pressure exceeds the ASCE 4-98 soil pressure. The design soil pressure loads for the exterior walls are calculated by averaging soil pressures which each wall is subjected to. The calculated design soil pressures are summarized in Tables 3A.8.8-1 and 3A.8.8-2, comparing the SASSI2000 soil pressures and the ASCE 4-98 soil pressures. The

3A.9 SITE ENVELOPE SEISMIC RESPONSES

The site-envelope seismic loads are established from the envelopes of all analysis results from SSI cases summarized in Table 3A.6-1. The site-envelope seismic loads obtained are applicable for the design of Seismic Category I and II structures, systems and components housed in the ESBWR Standard Plant.

3A.9.1 Enveloping Maximum Structural Loads

The enveloping maximum shear and moment distributions along the RB/FB walls, RCCV, vent wall/pedestal, RSW, key RPV/internals, CB walls, and FWSC walls are shown in Tables 3A.9-1a through 3A.9-1h. These shears and moments are the envelope of all SSI cases, except for the LOCA flood case (RU-6). Tables 3A.9-2a through 3A.9-2e show enveloping maximum responses for the RB/FB LOCA flood case (RU-6). The torsional moments for building structures are due to geometric eccentricities only. Additional torsion due to an accidental eccentricity of 5% of maximum floor dimension under consideration is added for the design of building structures.

The vertical loads are expressed in terms of enveloping absolute acceleration. The enveloping maximum acceleration values are shown in Tables 3A.9-3a through 3A.9-3i for all cases except for LOCA flood case (RU-6). Tables 3A.9-4a through 3A.9-4e show enveloping maximum responses for LOCA flood case (RU-6). These acceleration values do not include the coupling effect and are only applicable for structural analysis in combination with the seismic loads due to horizontal shakings.

3A.9.2 Enveloping Floor Response Spectra

The site-envelope SSE floor response spectra are obtained according to the following steps:

- For each soil case analyzed, the calculated co-directional FRS in X, Y, and Z directions are combined by the SRSS method to obtain FRS at the building edges considering the coupling effects between vertical and rocking and between lateral and torsion motions.
- Individual site responses are enveloped to form the site-envelope response spectra in each of the 3 directions.
- The envelope spectra are subsequently peak broadened by ±15%.
- The spectral valleys are filled-in.

The site-envelope peak-broadened SSE floor response spectra at critical damping ratios 2, 3, 4, 5, 7, 10, and 20% for the RB/FB, CB and FWSC are shown in Figures 3A.9-1a through 3A.9-1l for the X direction, in Figures 3A.9-2a through 3A.9-2l for the Y direction, and in Figures 3A.9-3a through 3A.9-3l for the vertical direction. For seismic design of equipment and piping, the alternative seismic input can be the individual FRS of each site condition considered in generating the site-envelope spectra.

3A.9.3 Basemat Interface Loads with Foundation Medium for Foundation Stability Evaluation

The base shears, base moments and base vertical forces for consideration of foundation stability evaluation in Subsections 3G.1.5.5, 3G.2.5.5, 3G.3.5.5, and 3G.4.5.5 are the enveloping results of all cases except for the DAC3N cases for uniform sites (see Subsection 3A.8.7 for details).

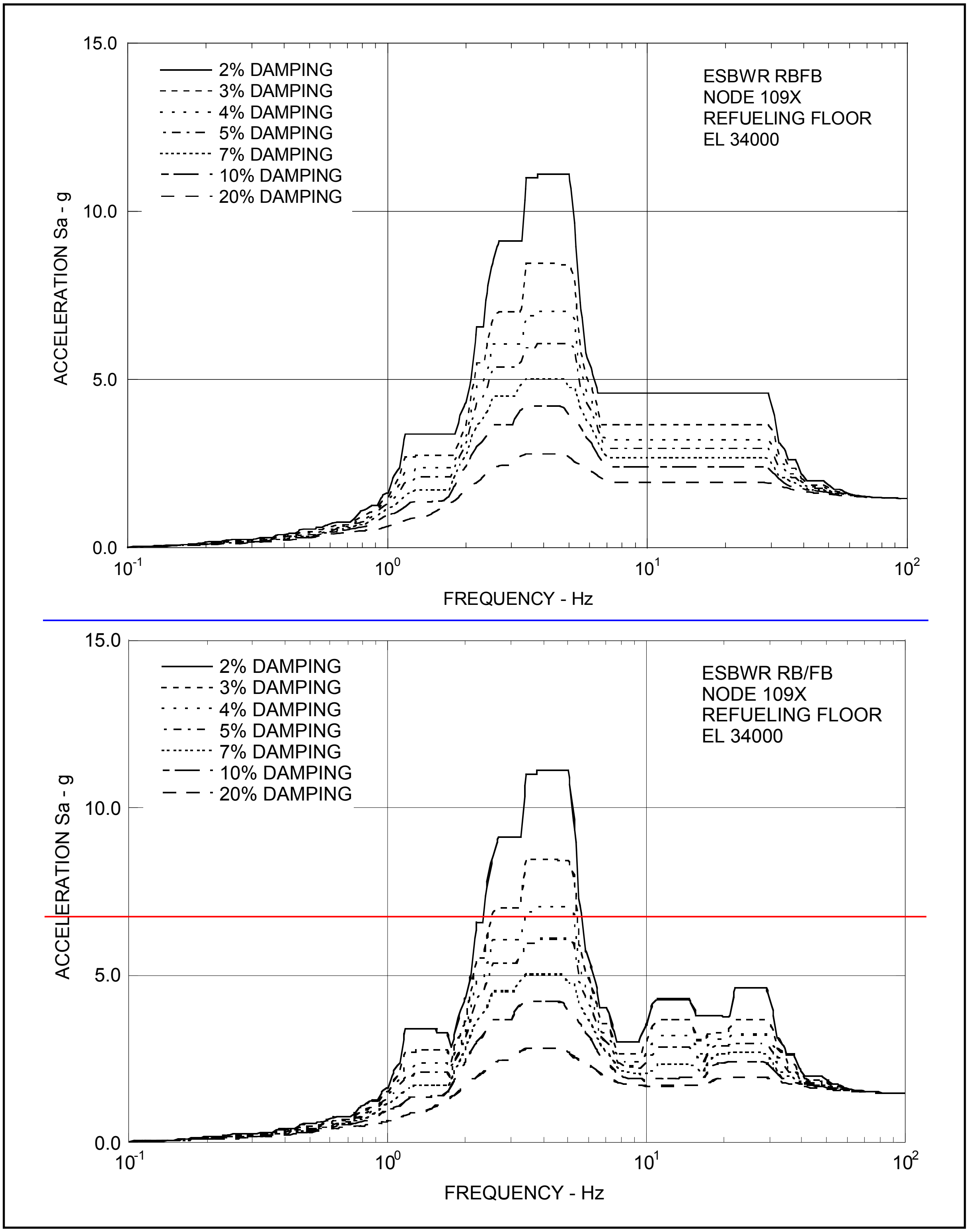


Figure 3A.9-1a. Enveloping Floor Response Spectra – RB/FB Refueling Floor X

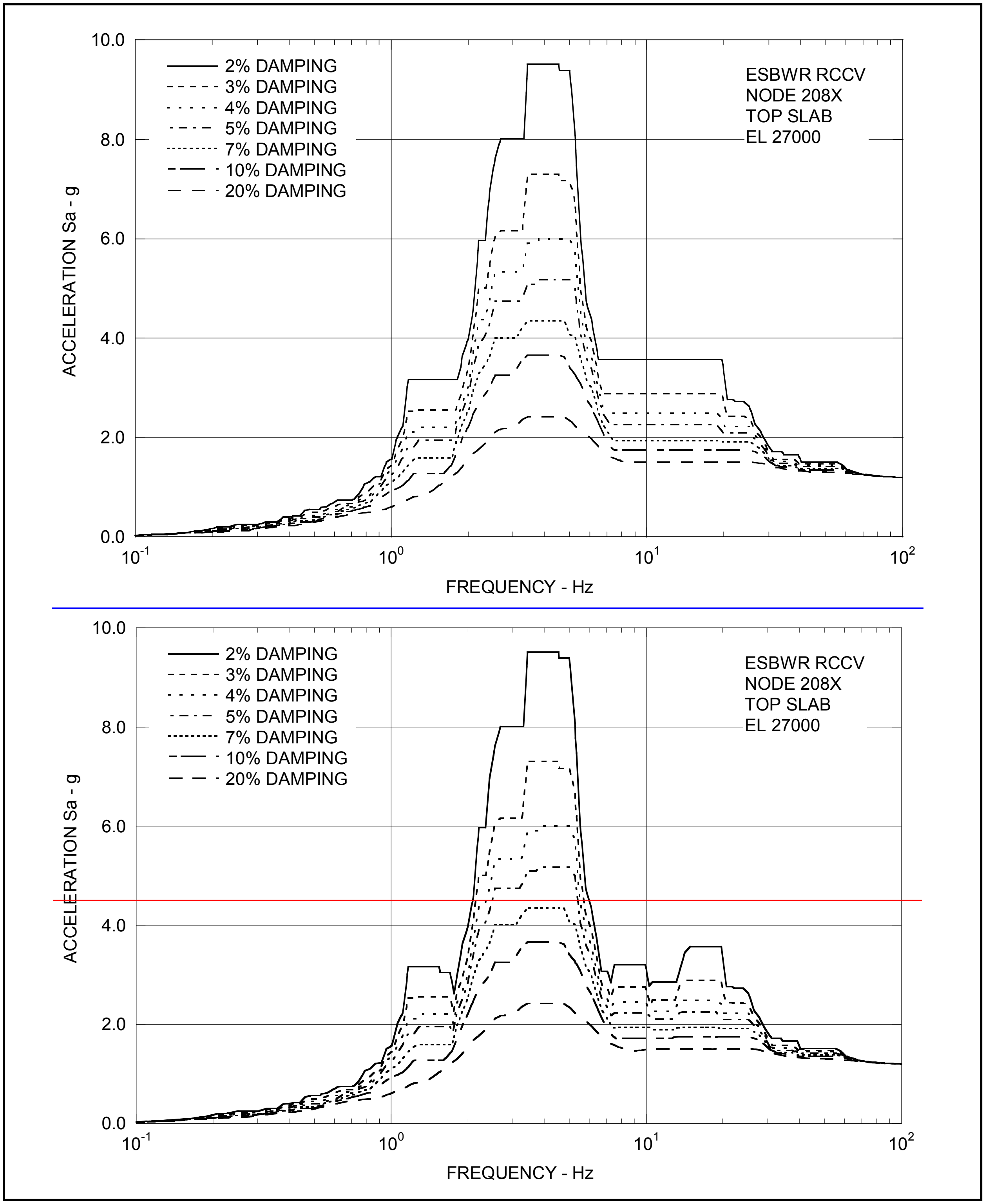


Figure 3A.9-1b. Enveloping Floor Response Spectra – RCCV Top Slab X

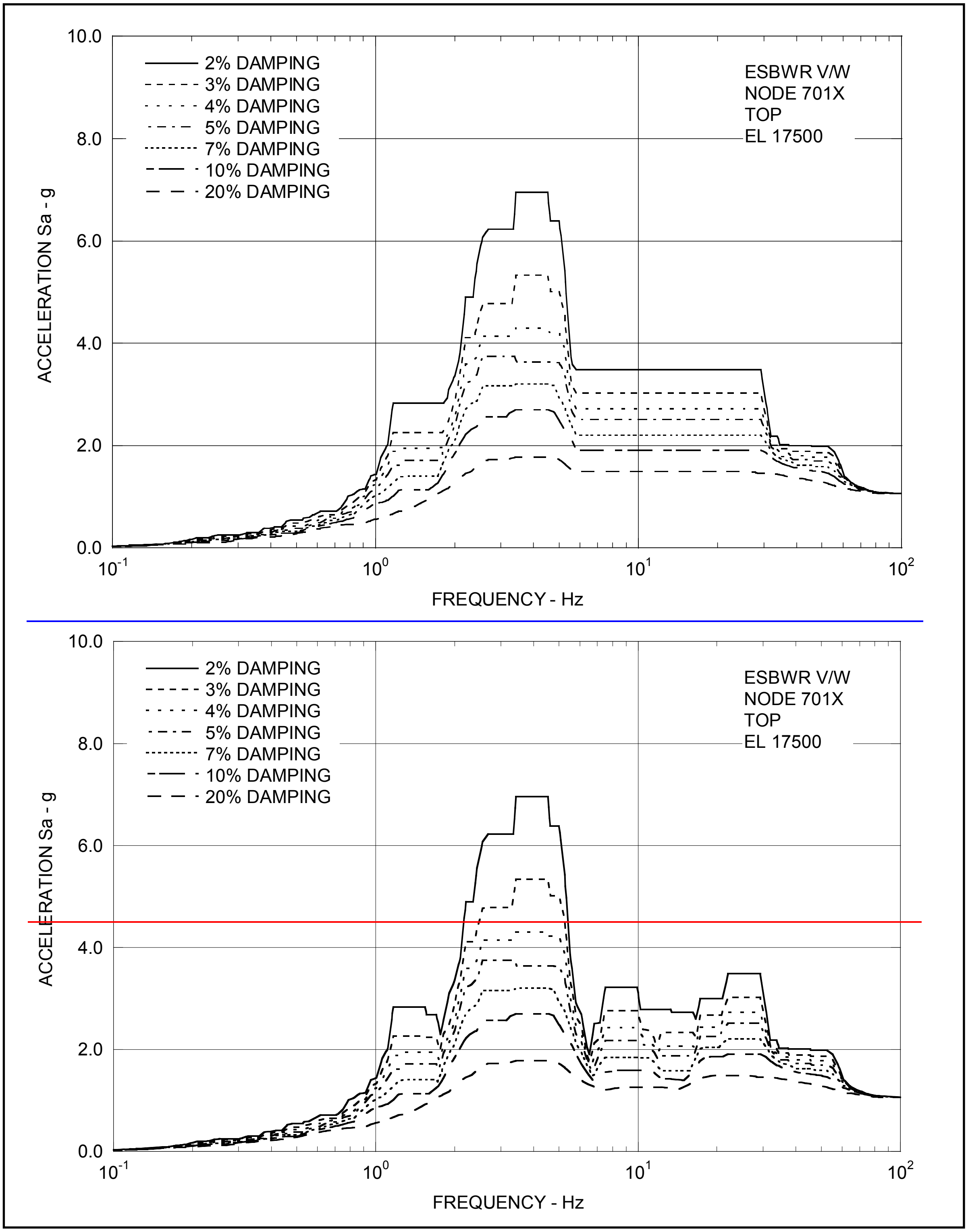
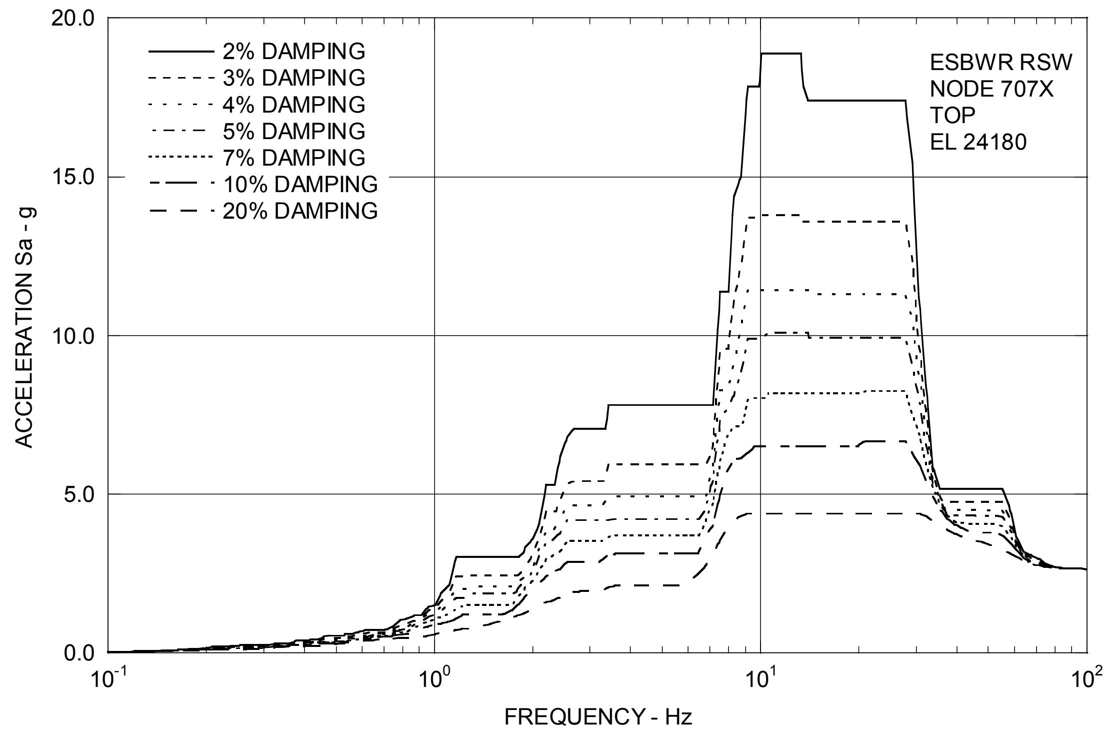


Figure 3A.9-1c. Enveloping Floor Response Spectra – Vent Wall Top X



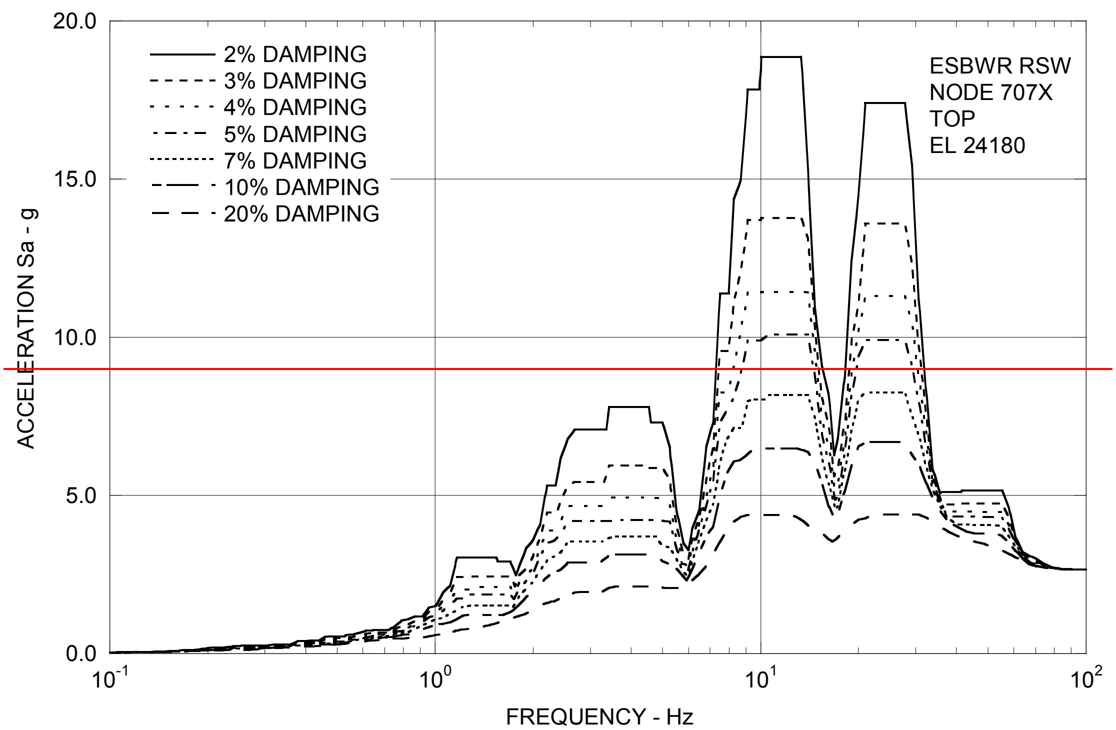


Figure 3A.9-1d. Enveloping Floor Response Spectra – RSW Top X

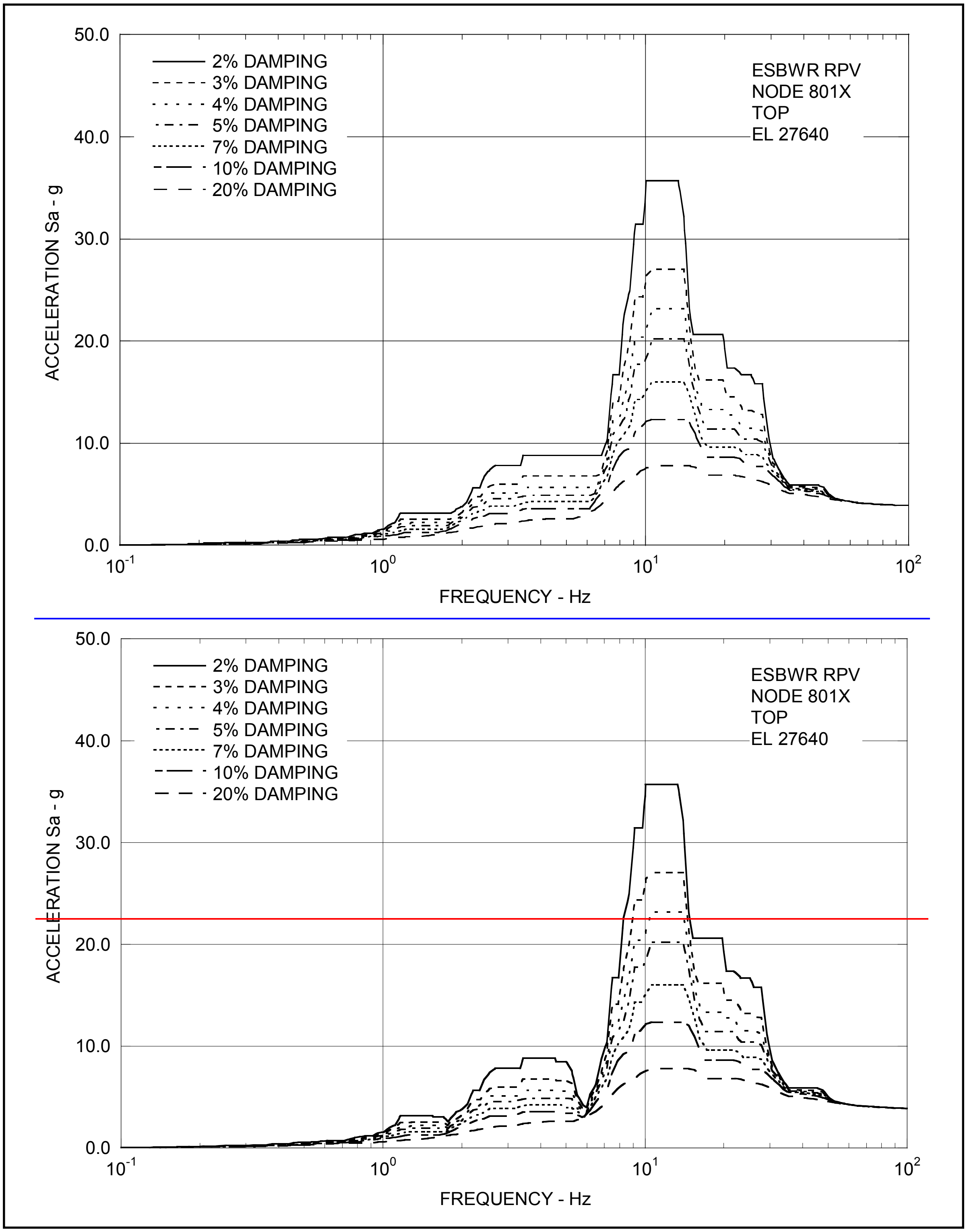


Figure 3A.9-1e. Enveloping Floor Response Spectra – RPV Top X

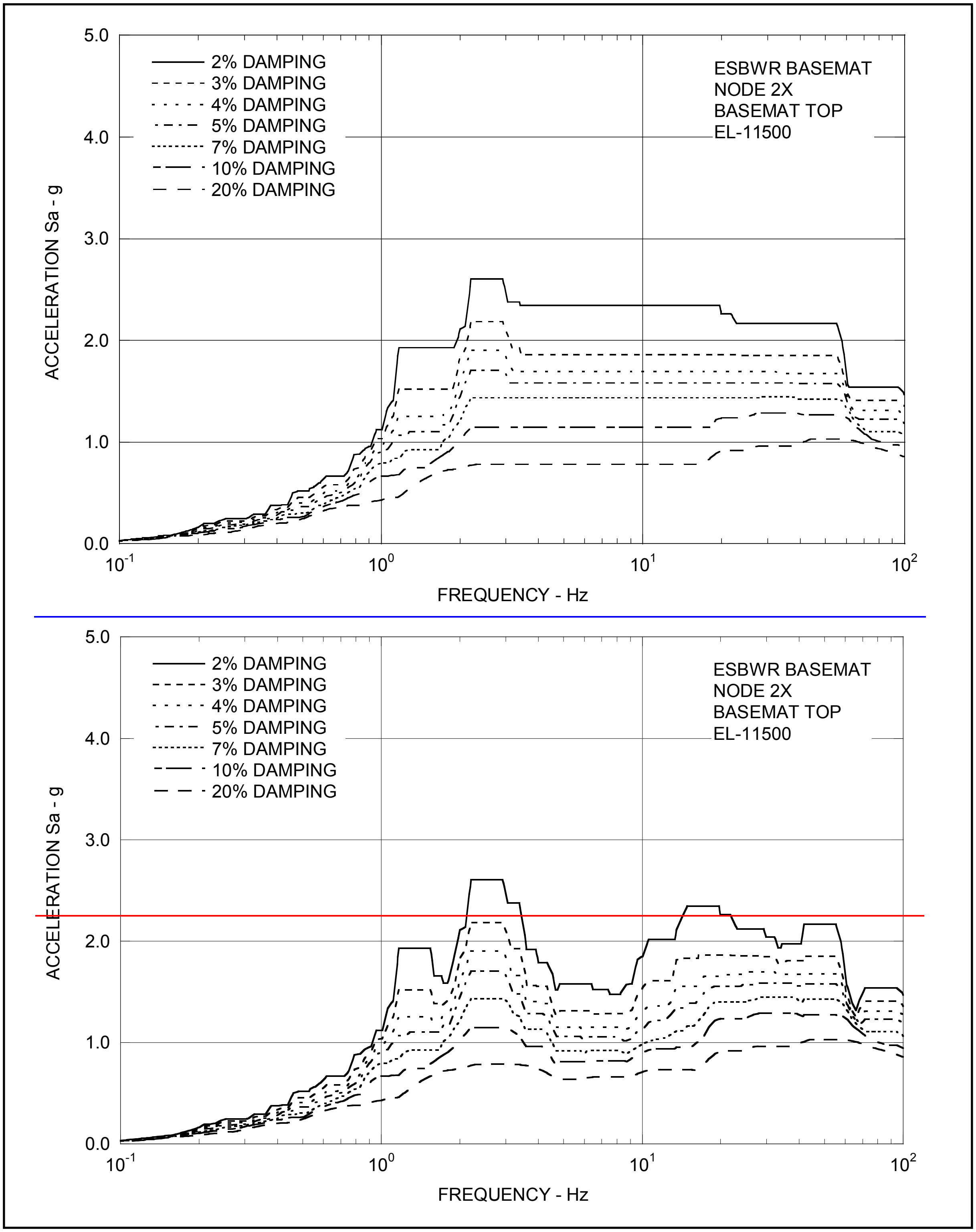


Figure 3A.9-1f. Enveloping Floor Response Spectra – RB/FB Basemat X

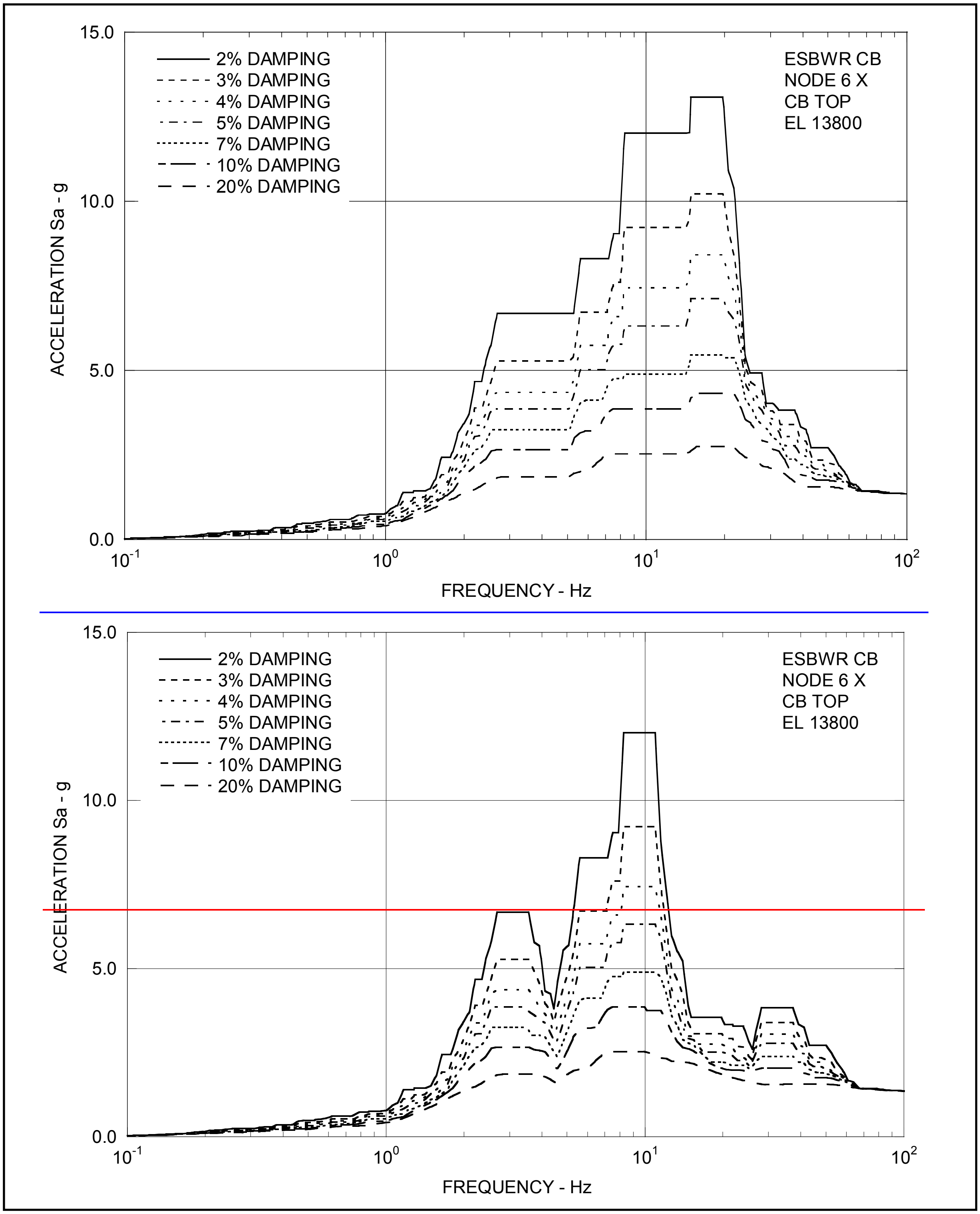


Figure 3A.9-1g. Enveloping Floor Response Spectra – CB Top X

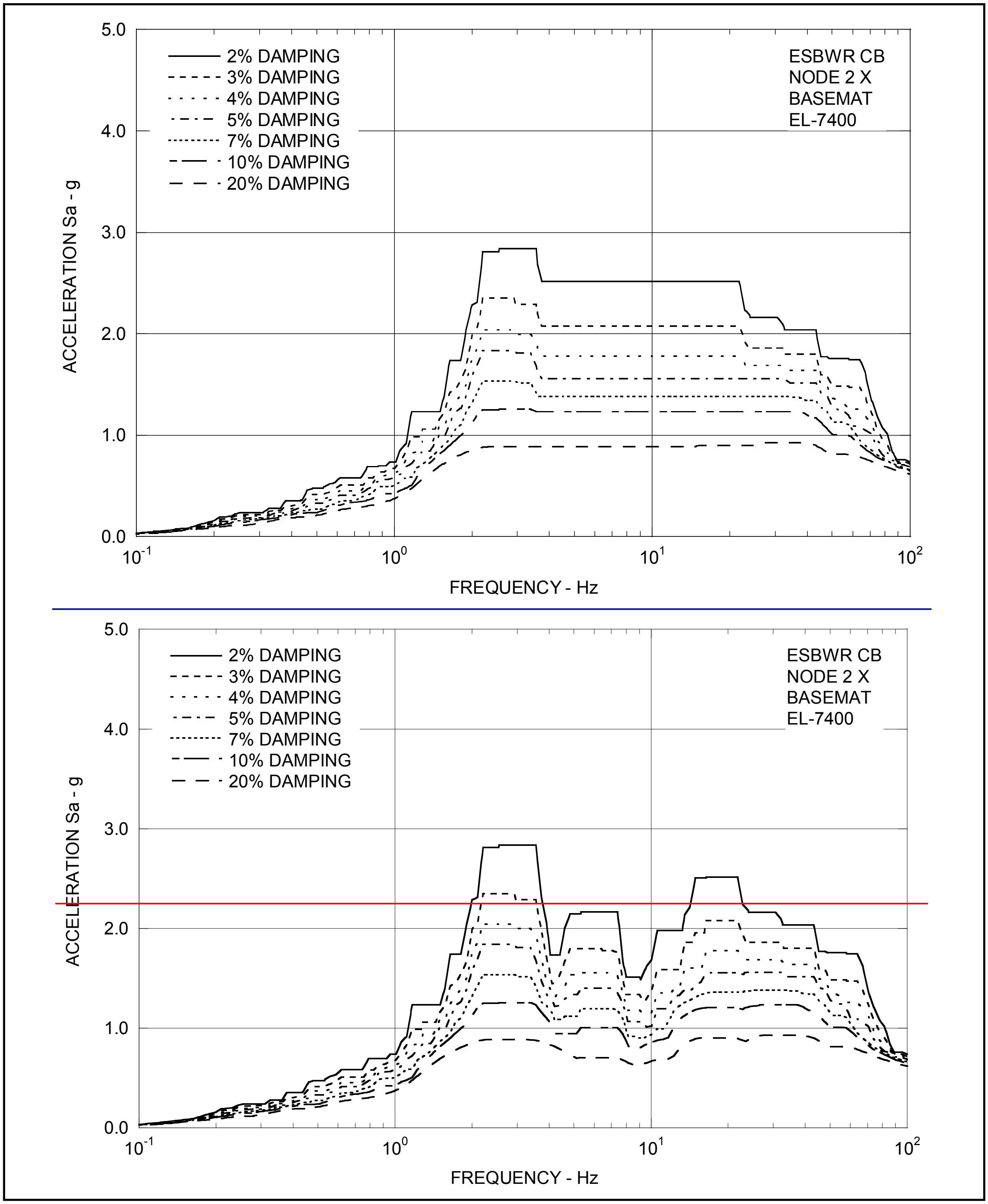
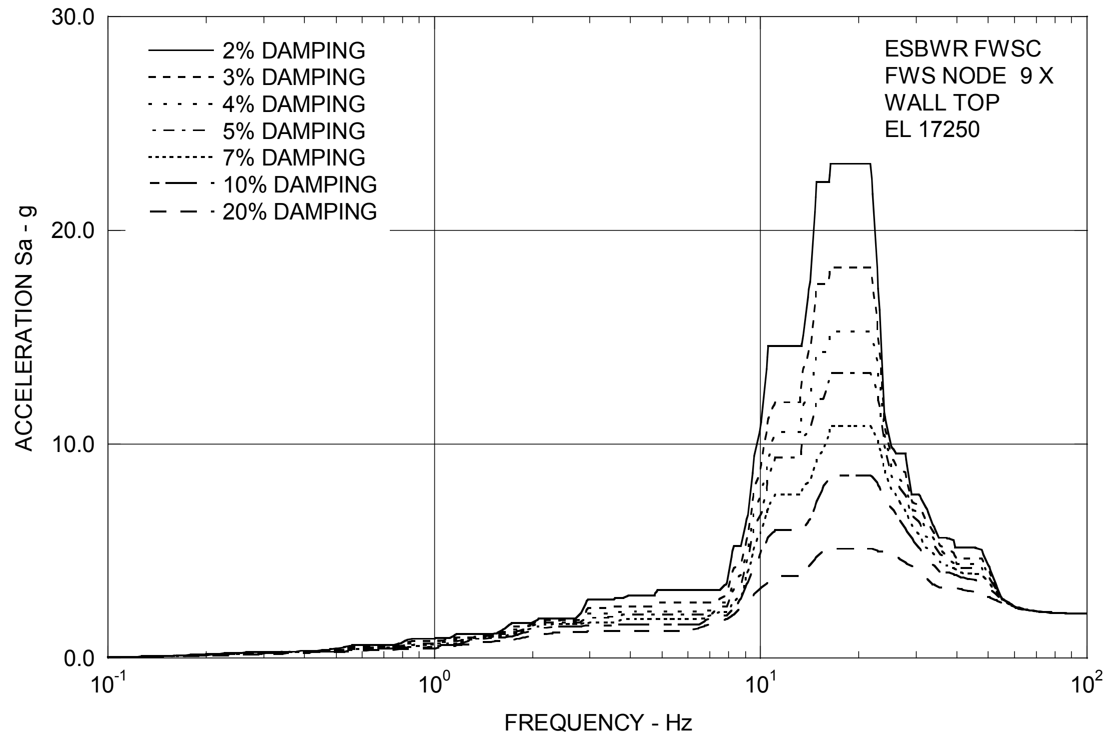


Figure 3A.9-1h. Enveloping Floor Response Spectra – CB Basemat X



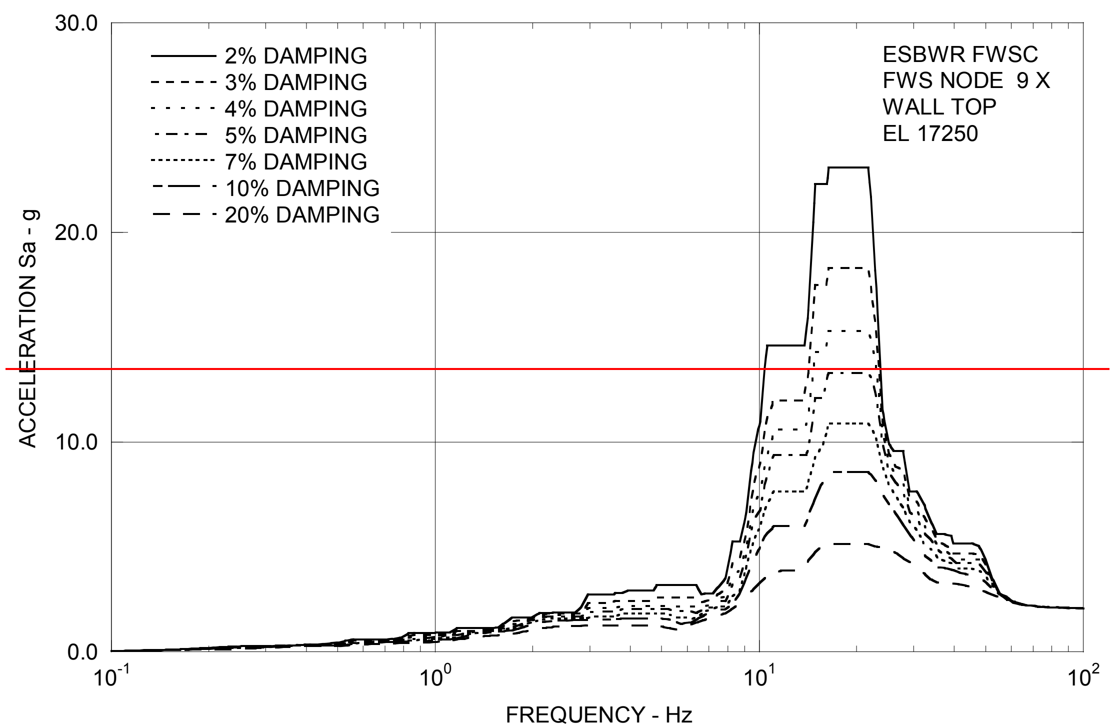


Figure 3A.9-1i. Enveloping Floor Response Spectra – FWS Wall Top X

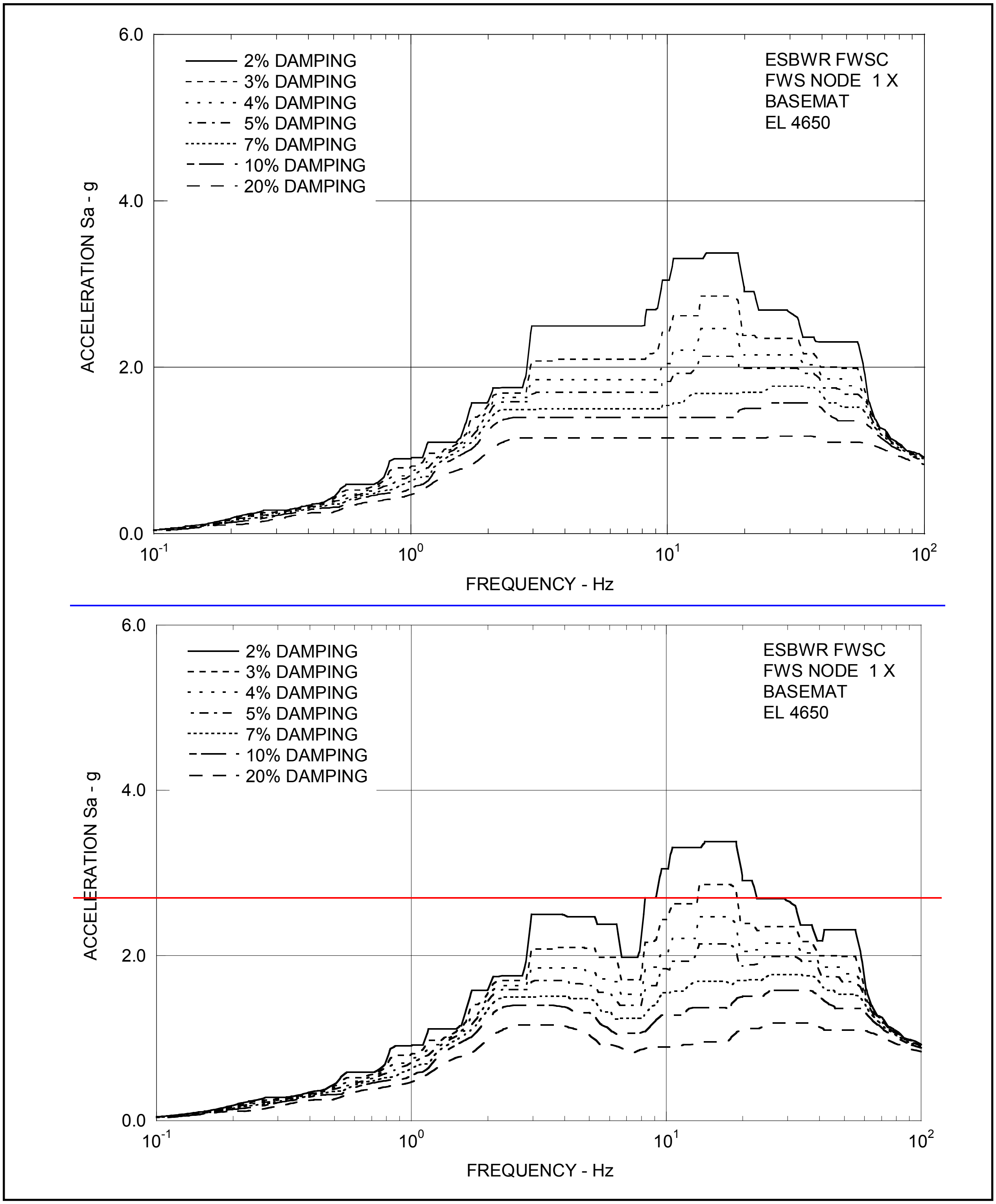
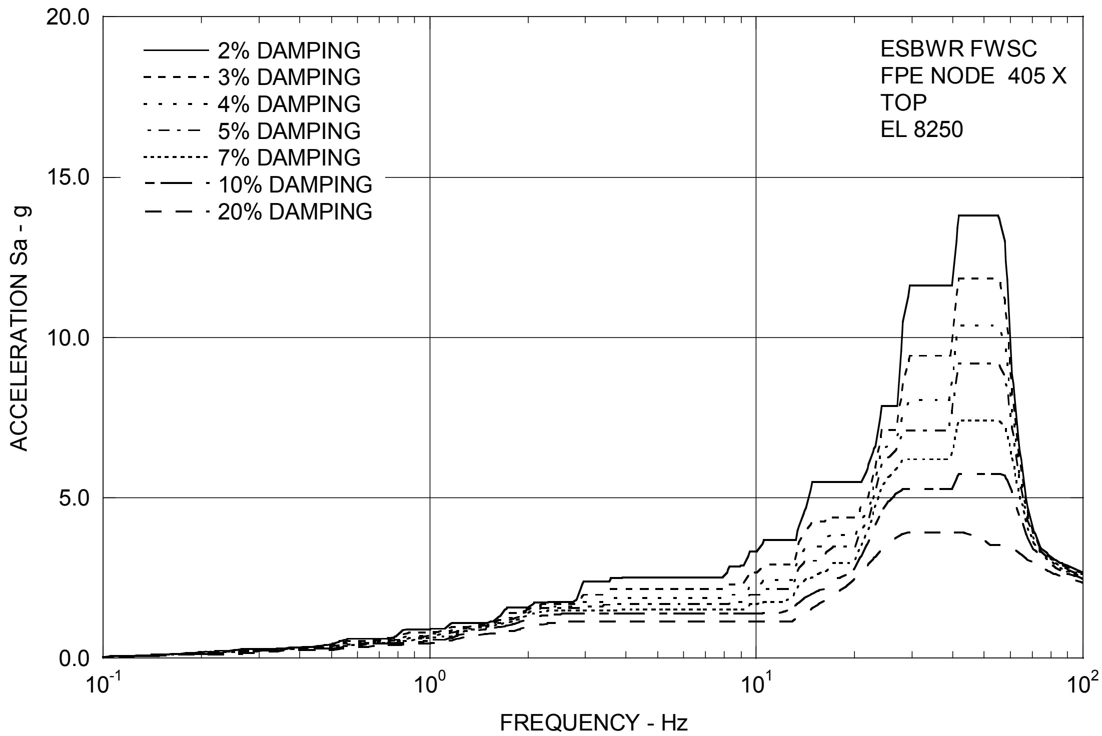


Figure 3A.9-1j. Enveloping Floor Response Spectra – FWS Basemat X



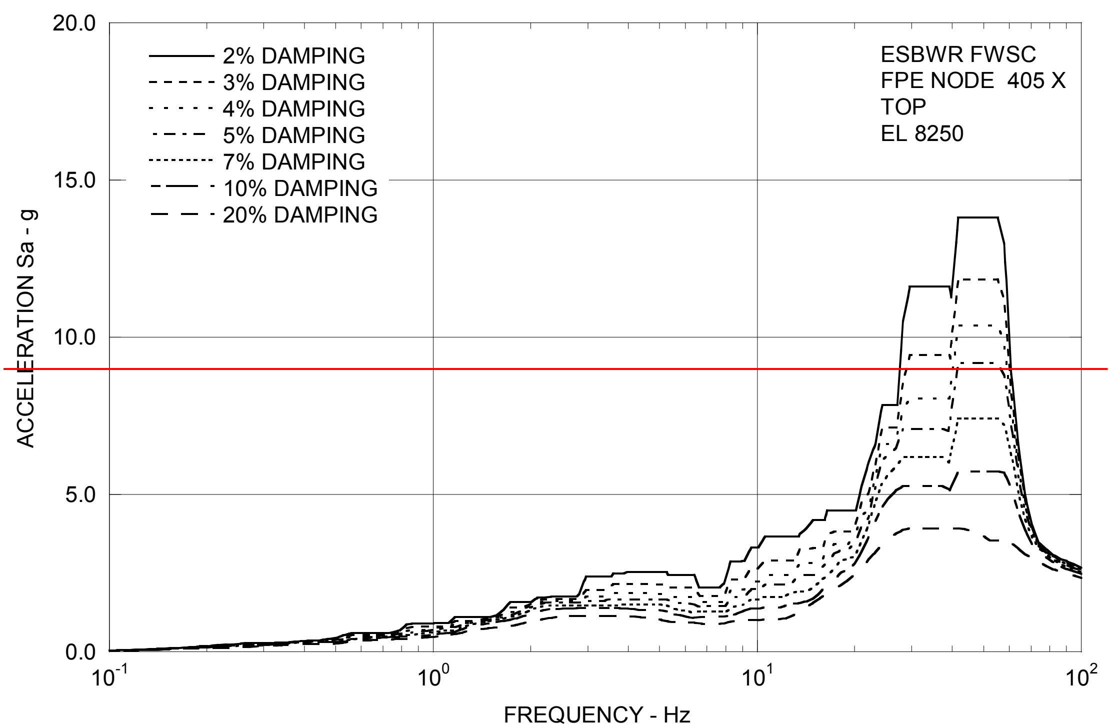


Figure 3A.9-1k. Enveloping Floor Response Spectra – FPE Top X

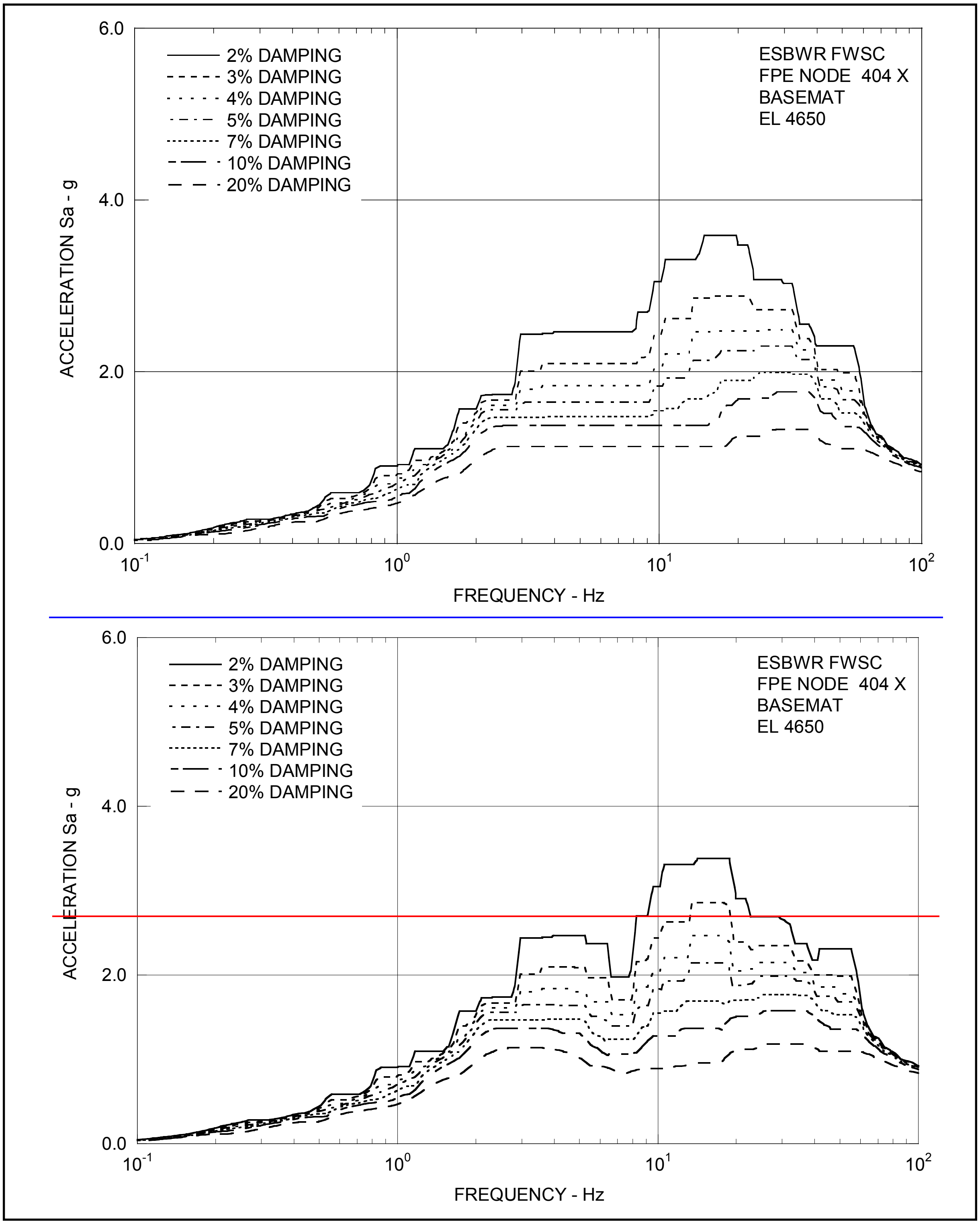


Figure 3A.9-1l. Enveloping Floor Response Spectra – FPE Basemat X

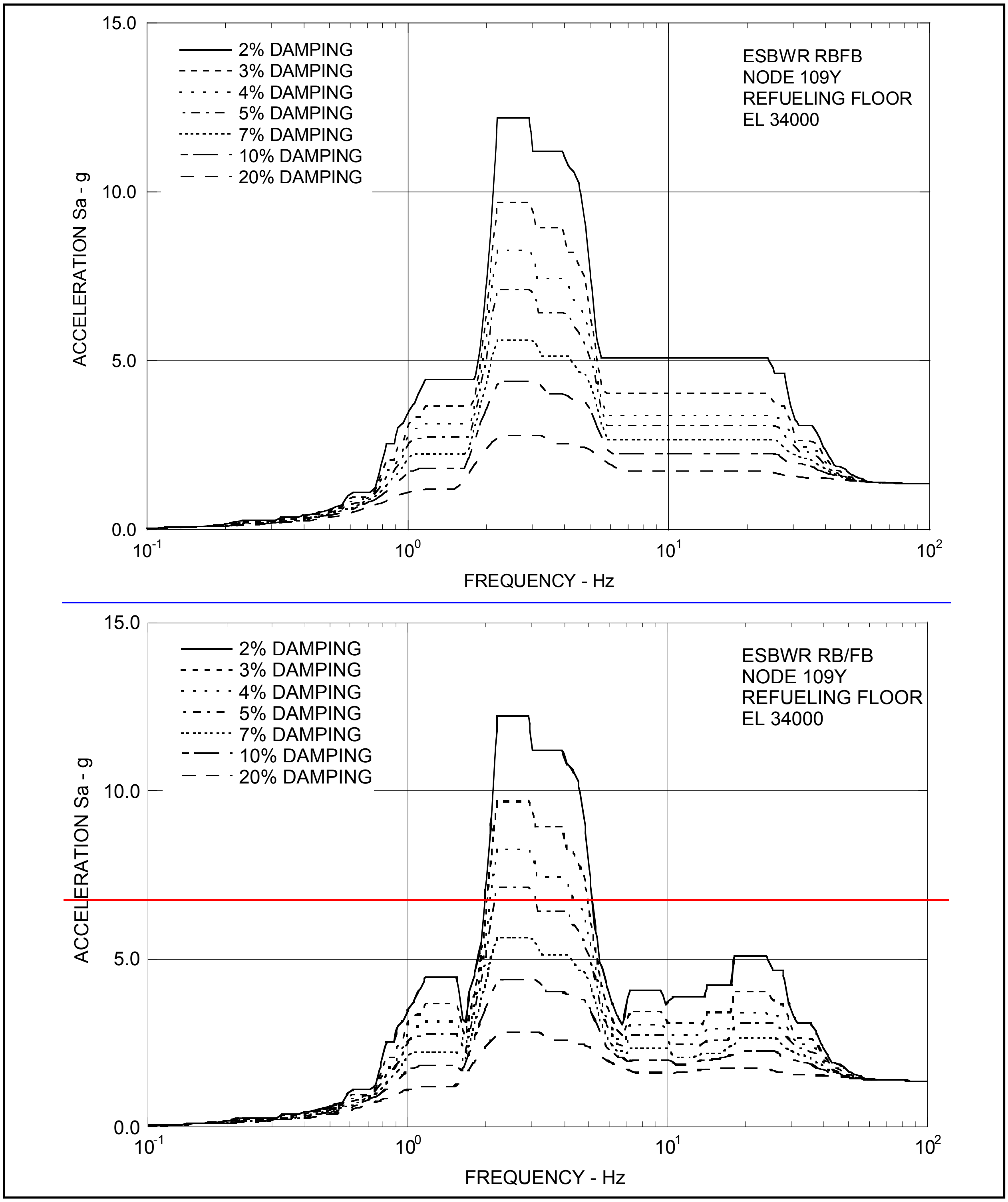
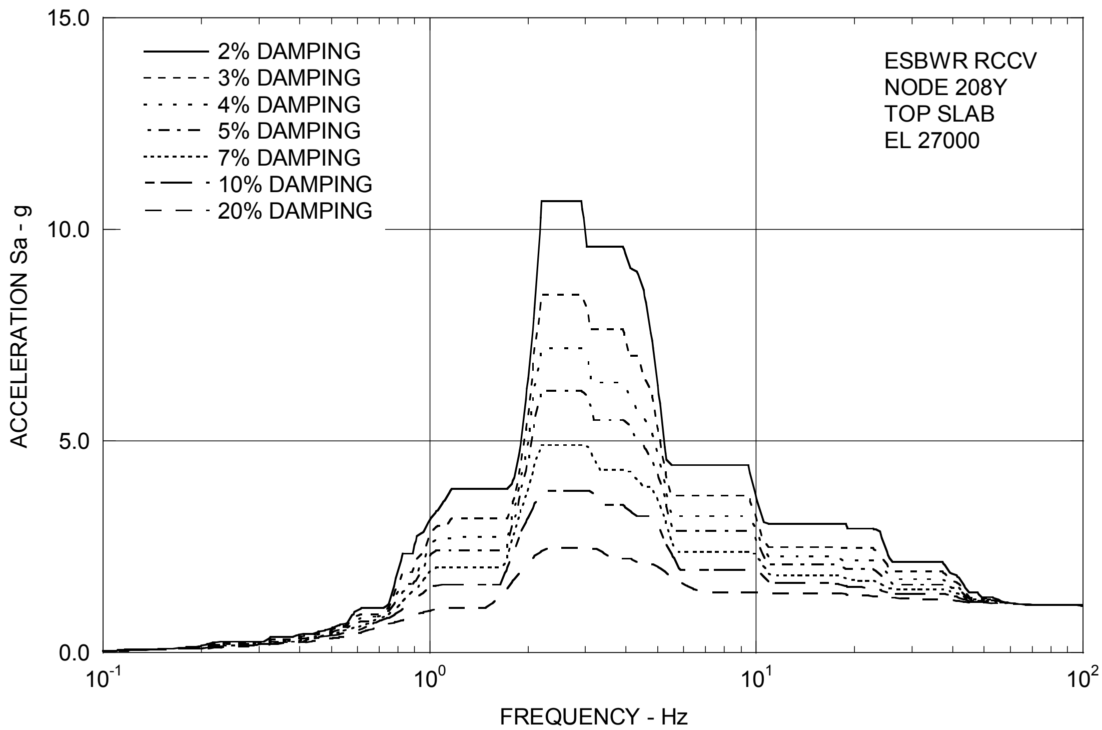


Figure 3A.9-2a. Enveloping Floor Response Spectra – RB/FB Refueling Floor Y



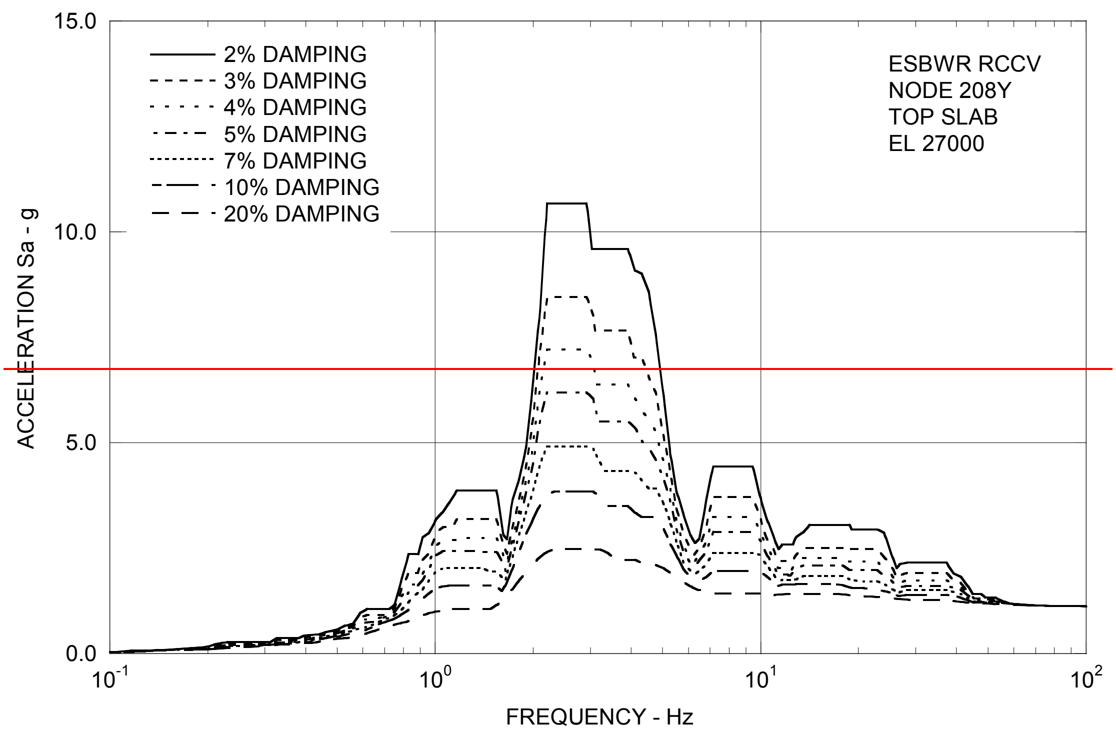
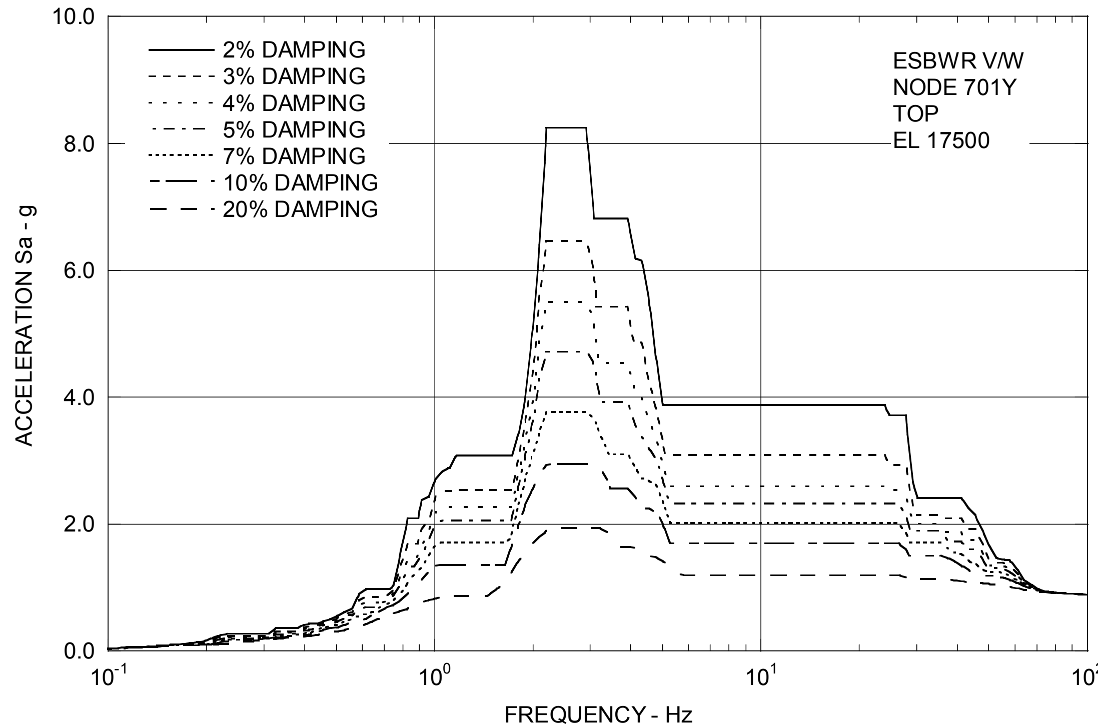


Figure 3A.9-2b. Enveloping Floor Response Spectra – RCCV Top Slab Y



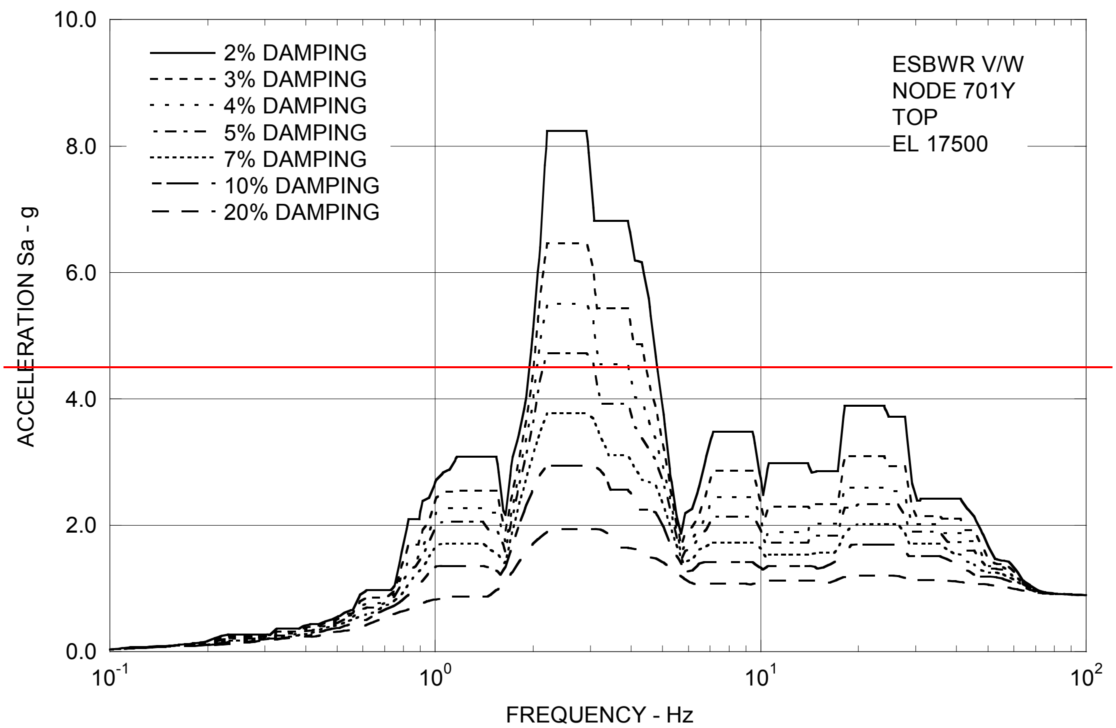


Figure 3A.9-2c. Enveloping Floor Response Spectra – Vent Wall Top Y

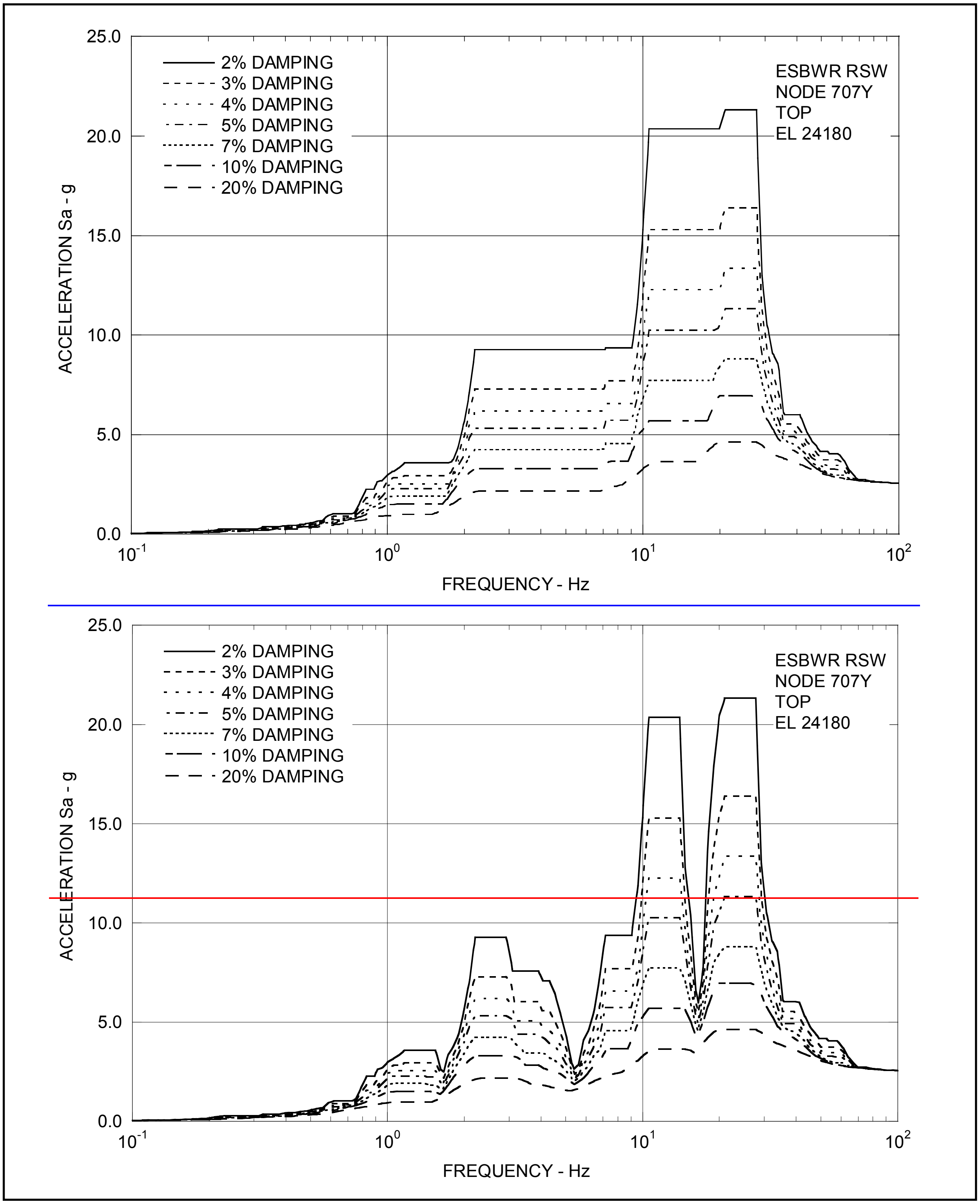


Figure 3A.9-2d. Enveloping Floor Response Spectra – RSW Top Y

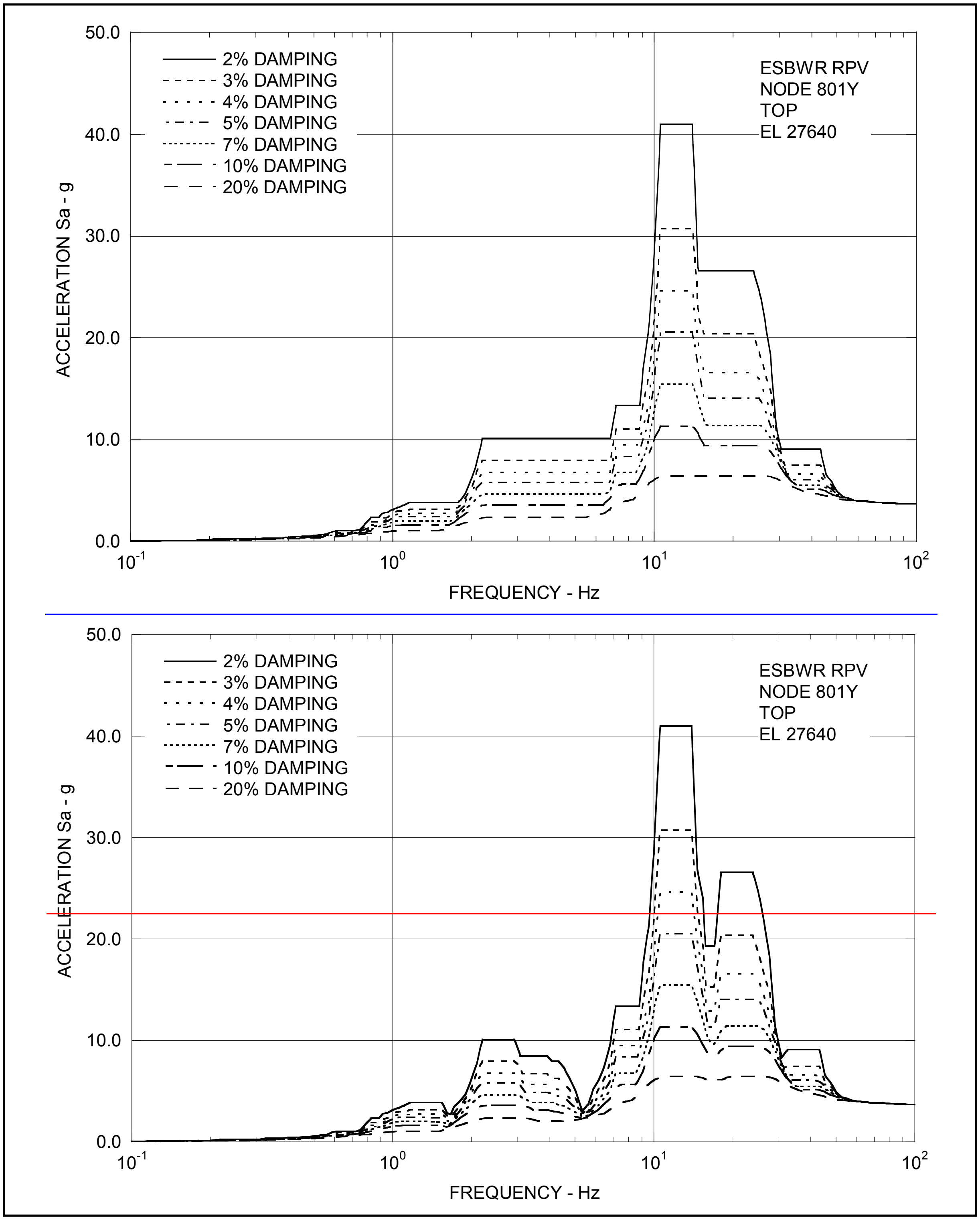


Figure 3A.9-2e. Enveloping Floor Response Spectra – RPV Top Y

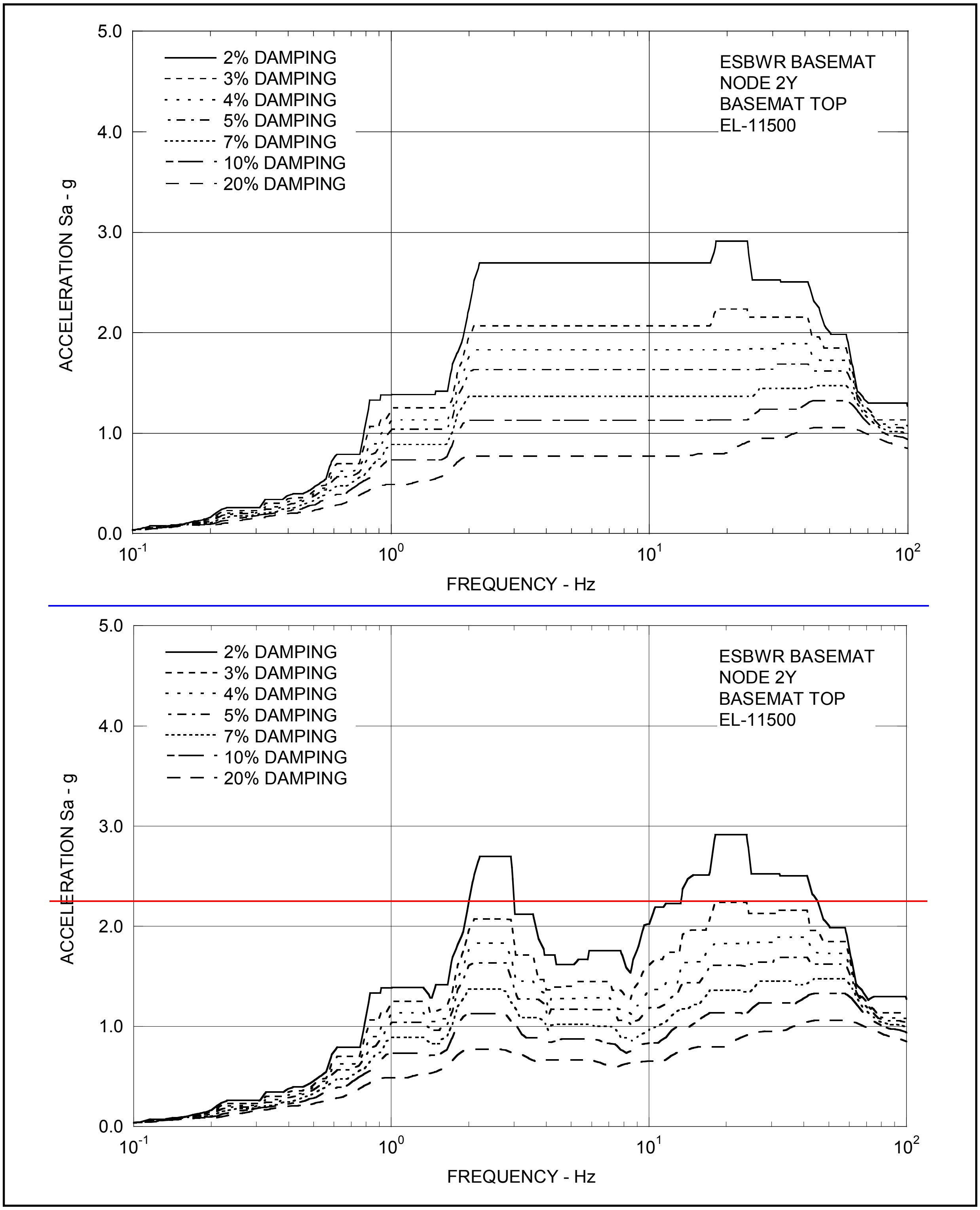
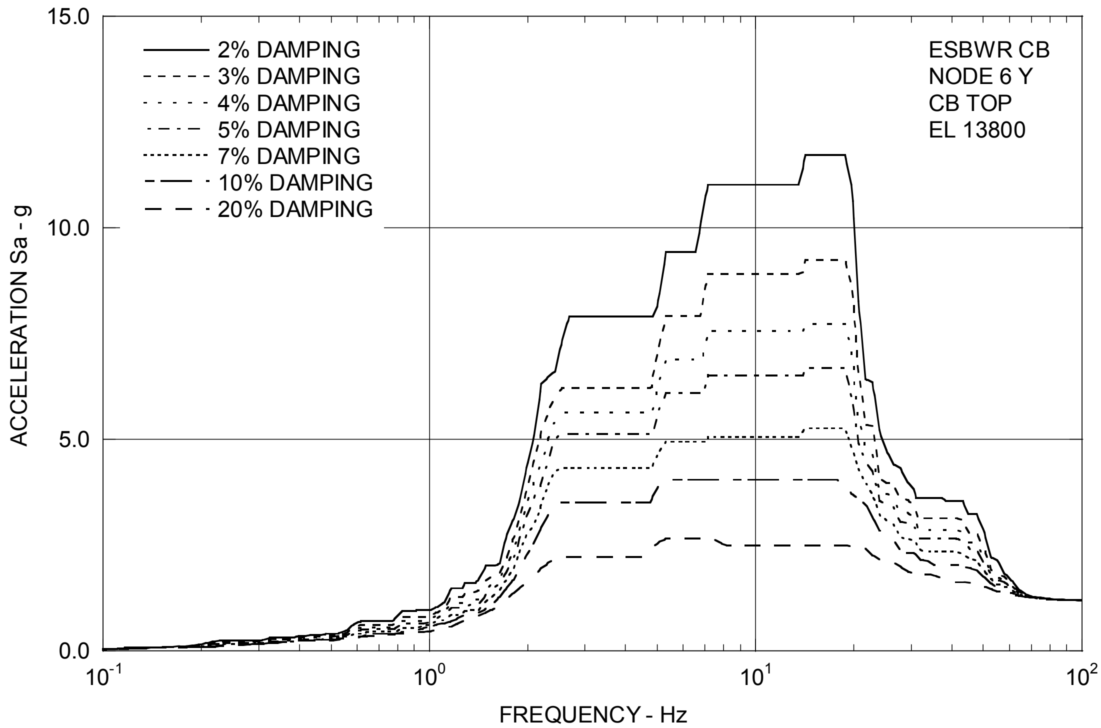


Figure 3A.9-2f. Enveloping Floor Response Spectra – RB/FB Basemat Y



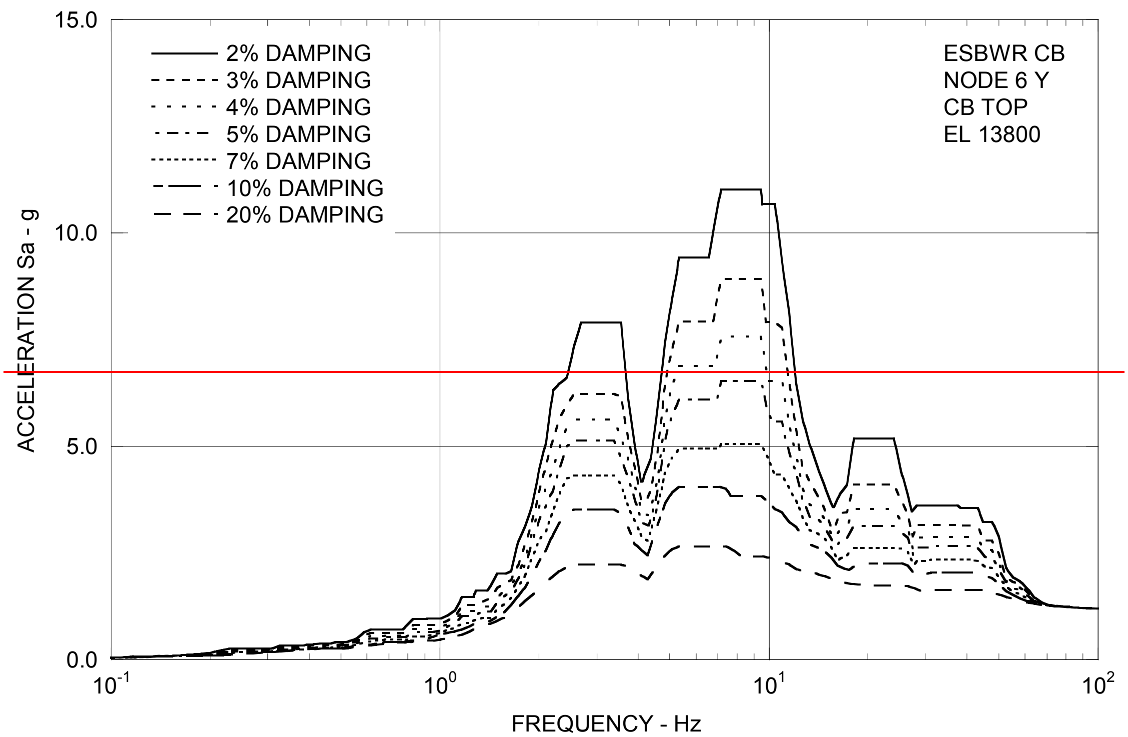
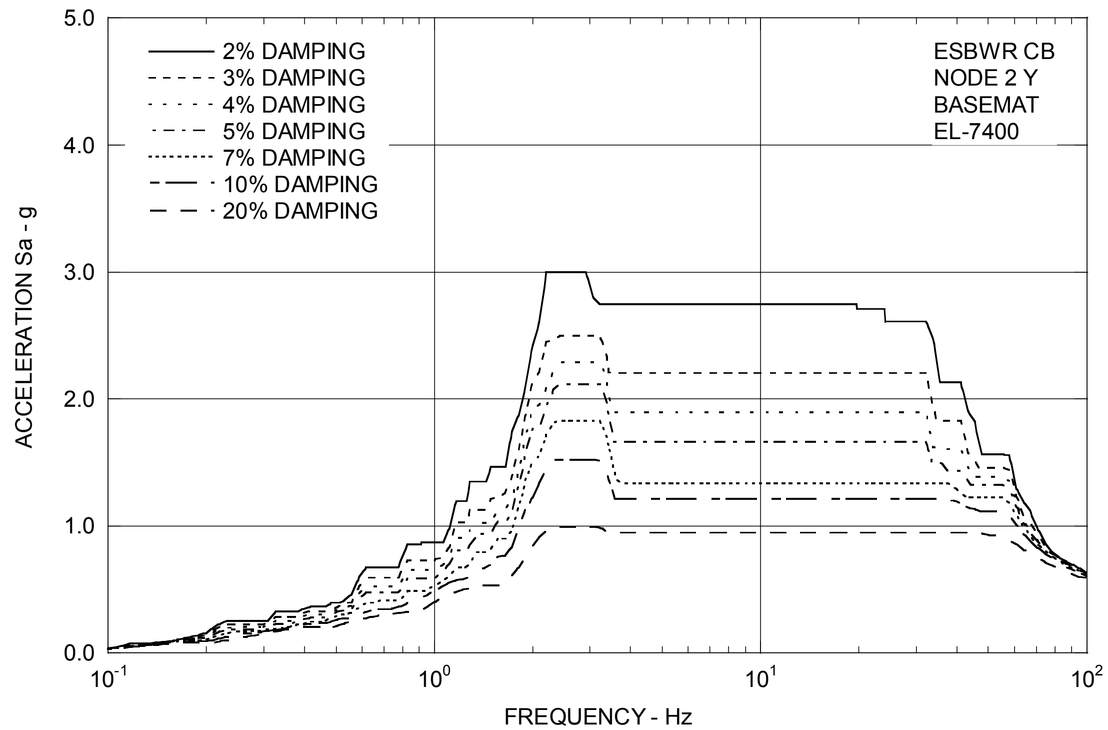


Figure 3A.9-2g. Enveloping Floor Response Spectra – CB Top Y



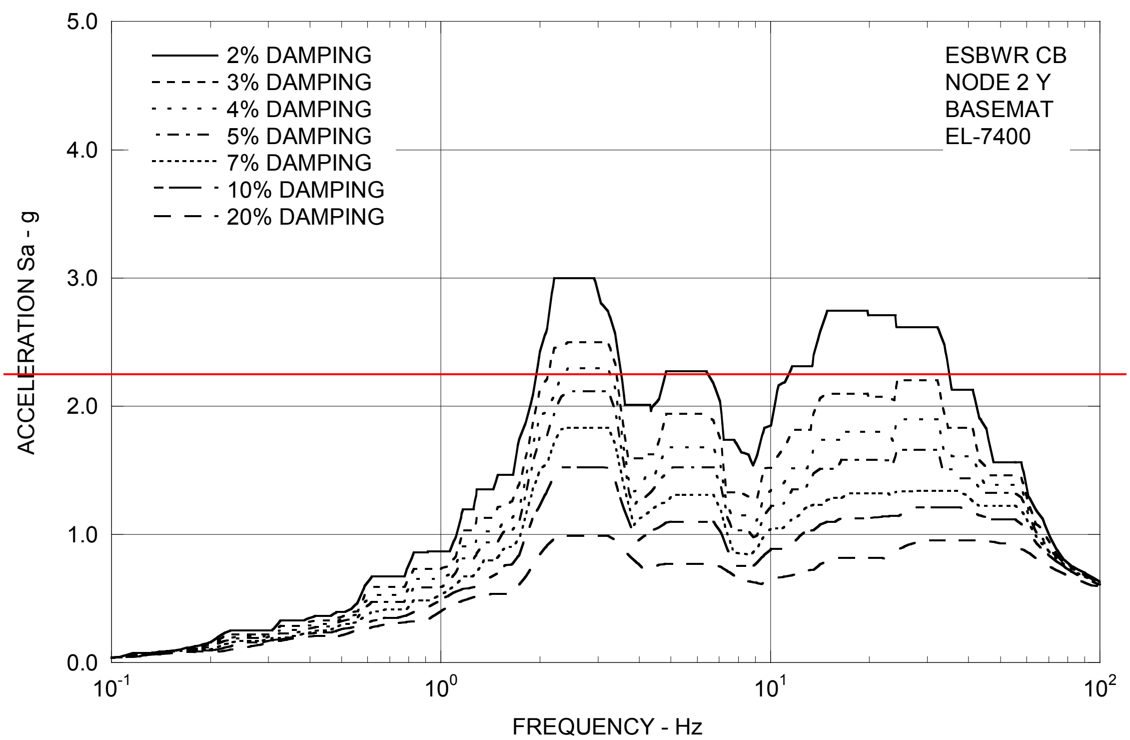
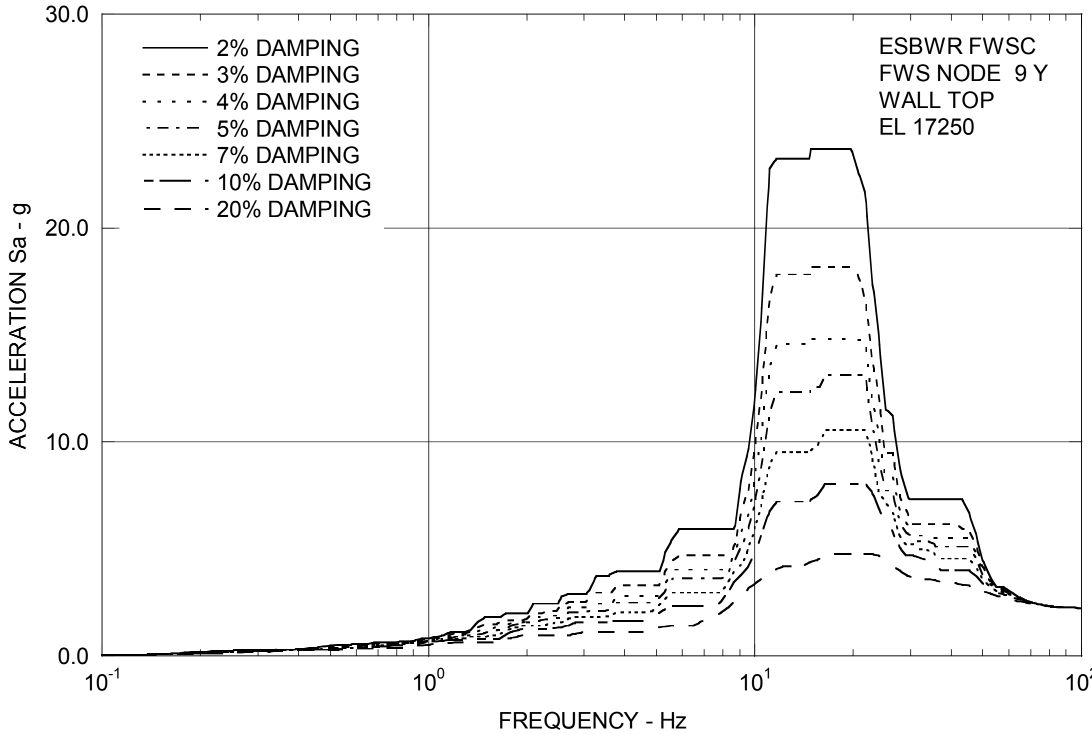


Figure 3A.9-2h. Enveloping Floor Response Spectra – CB Basemat Y



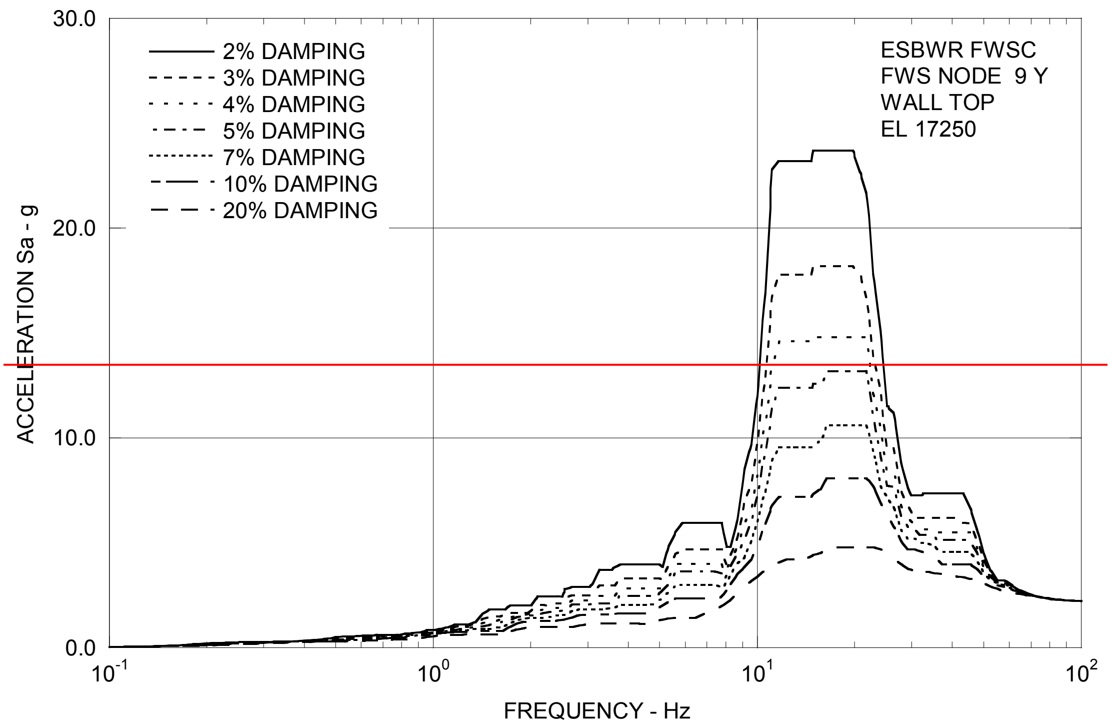


Figure 3A.9-2i. Enveloping Floor Response Spectra – FWS Wall Top Y

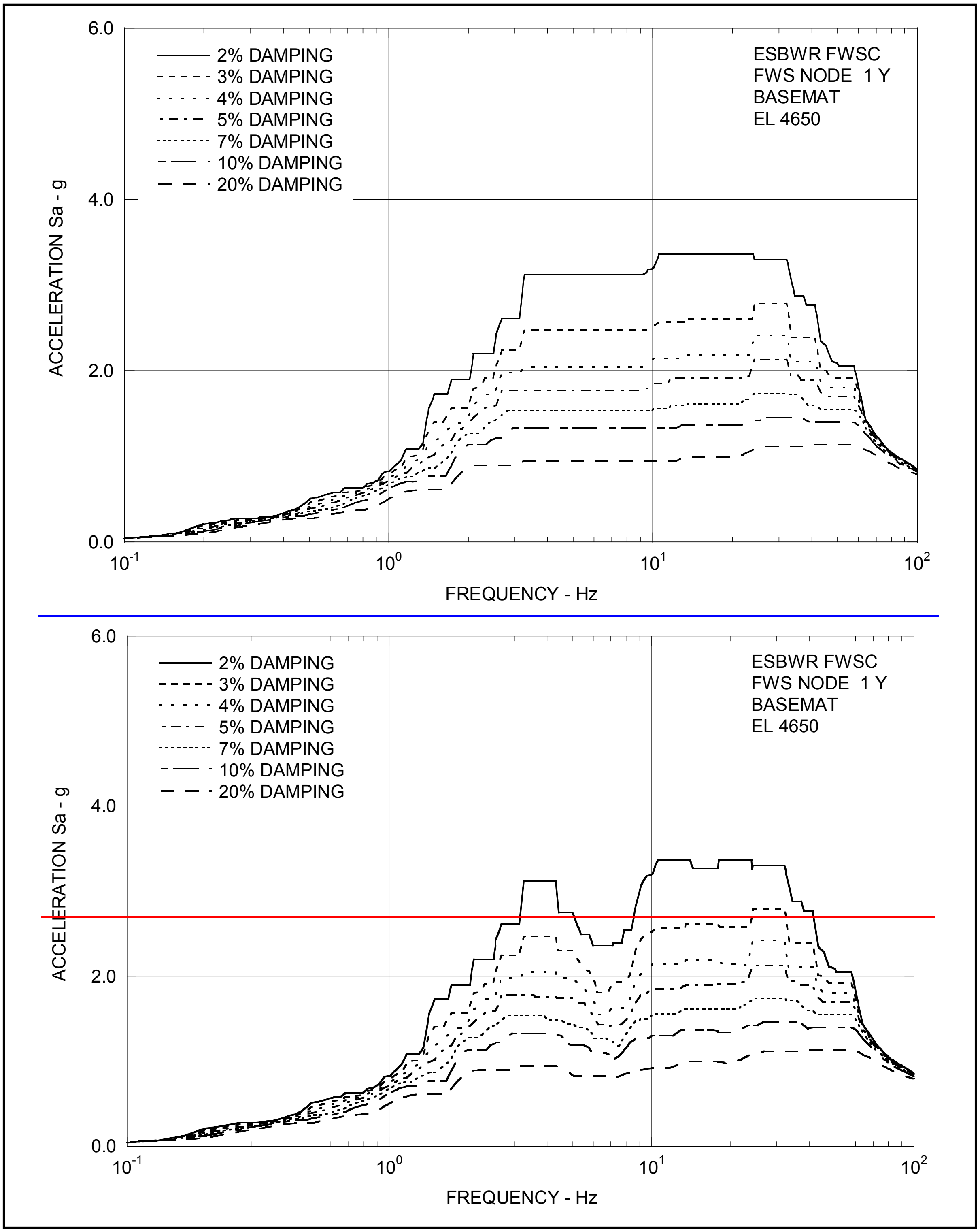


Figure 3A.9-2j. Enveloping Floor Response Spectra – FWS Basemat Y

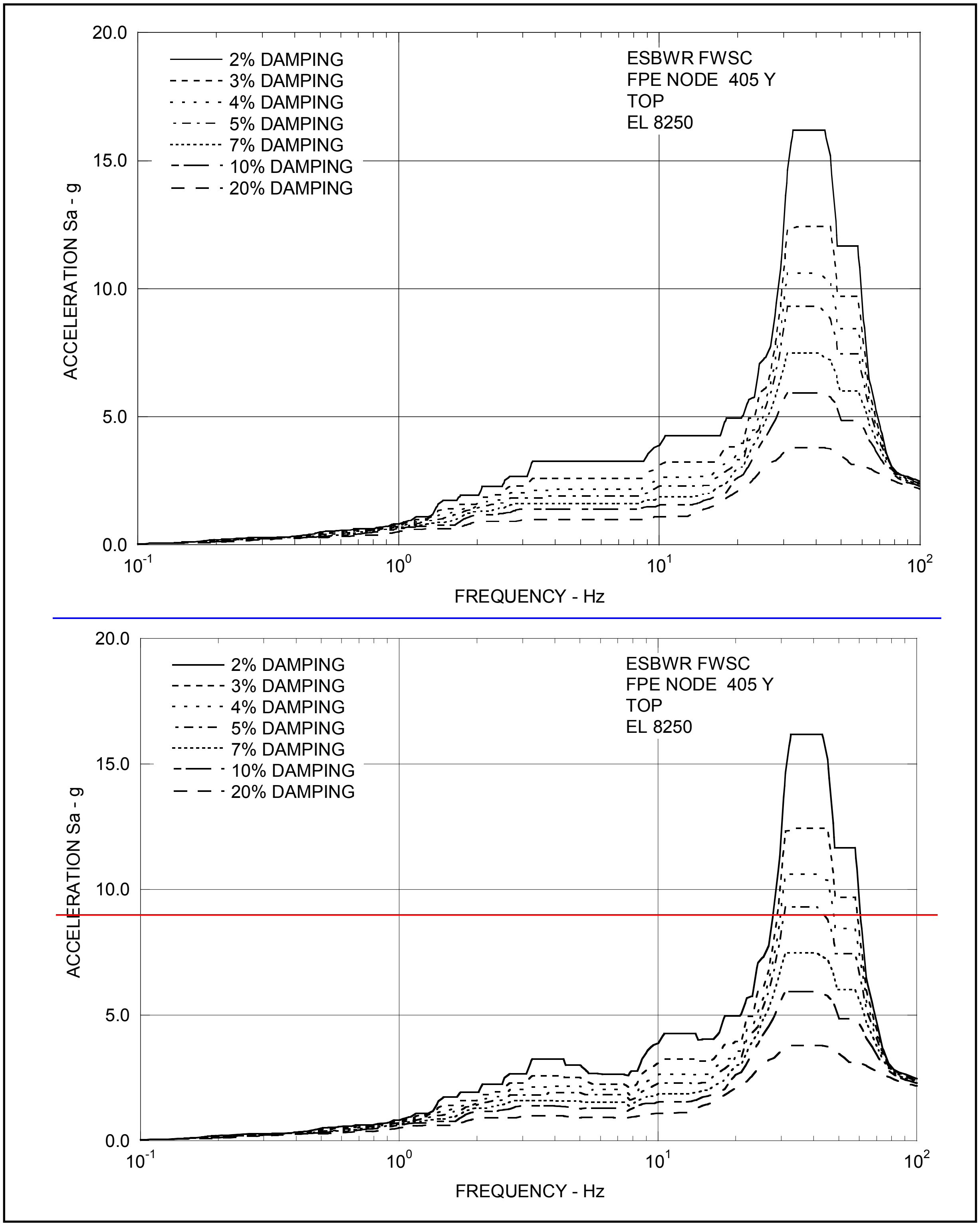


Figure 3A.9-2k. Enveloping Floor Response Spectra – FPE Top Y

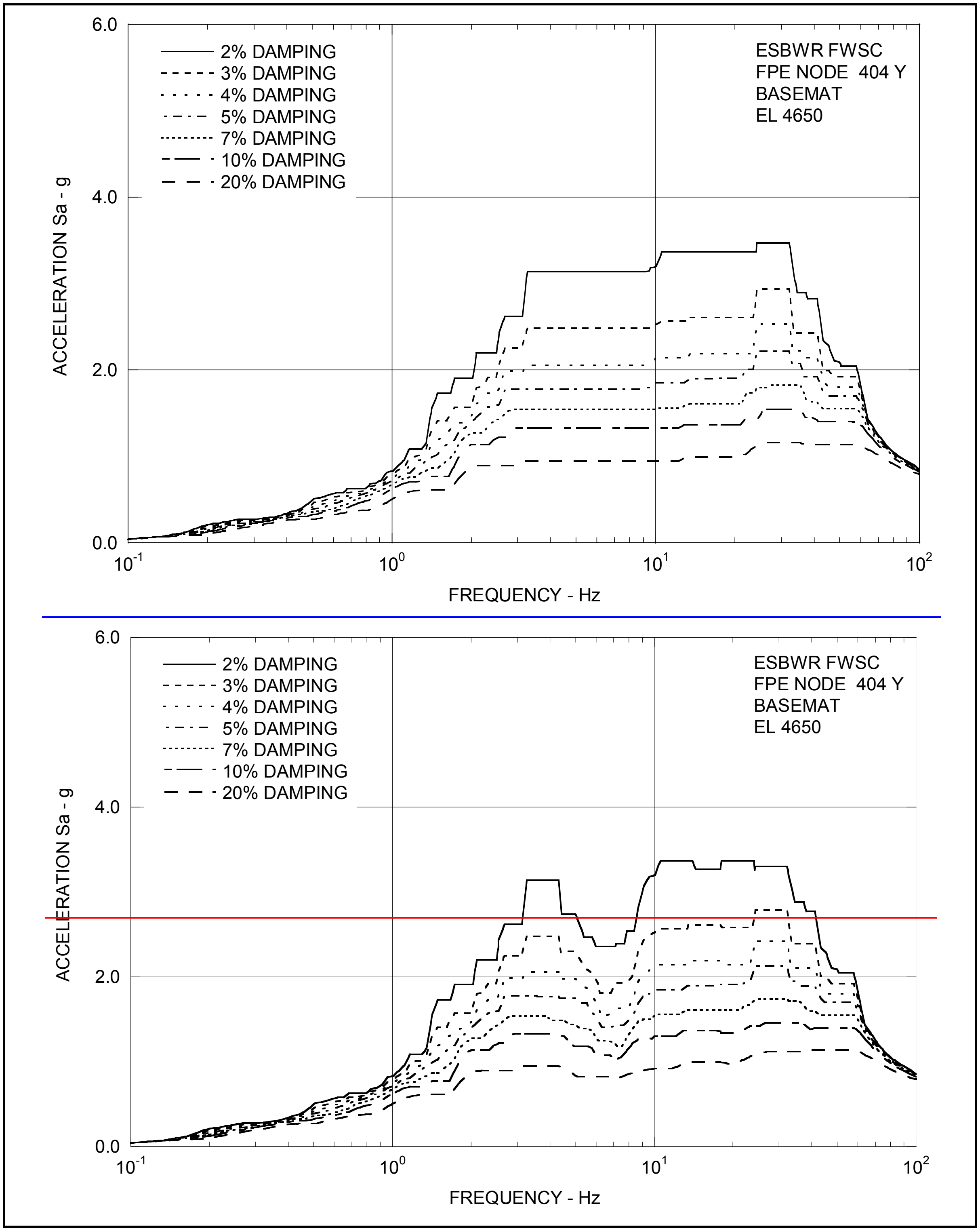
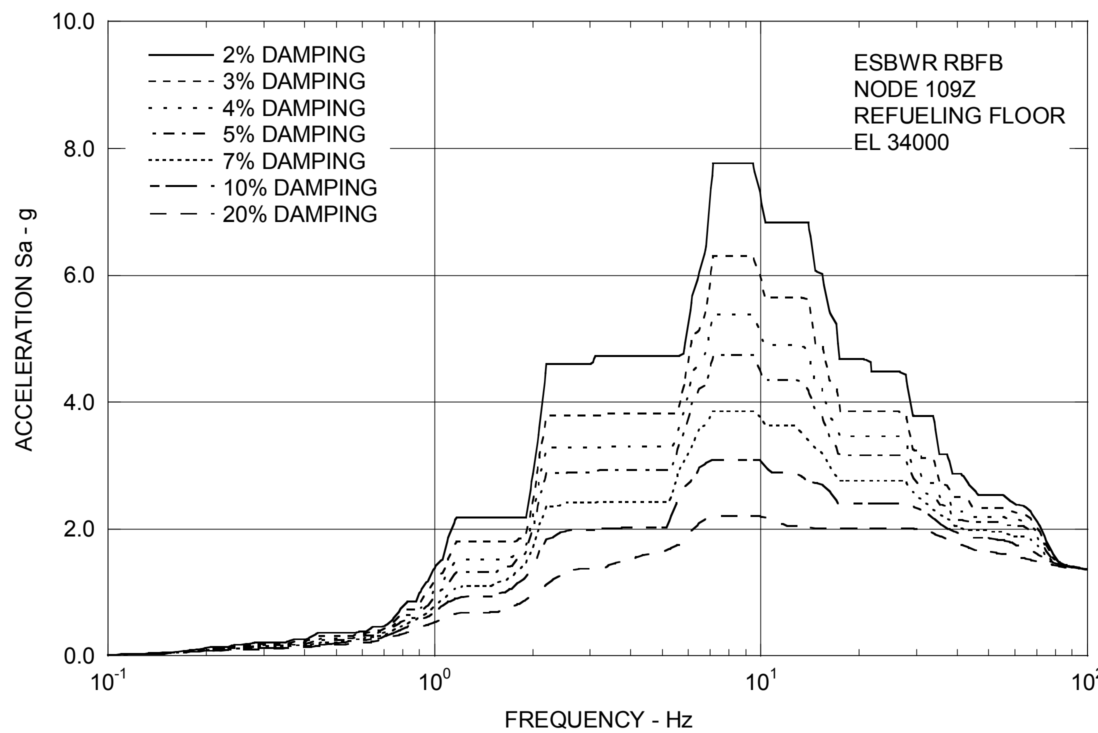


Figure 3A.9-2l. Enveloping Floor Response Spectra – FPE Basemat Y



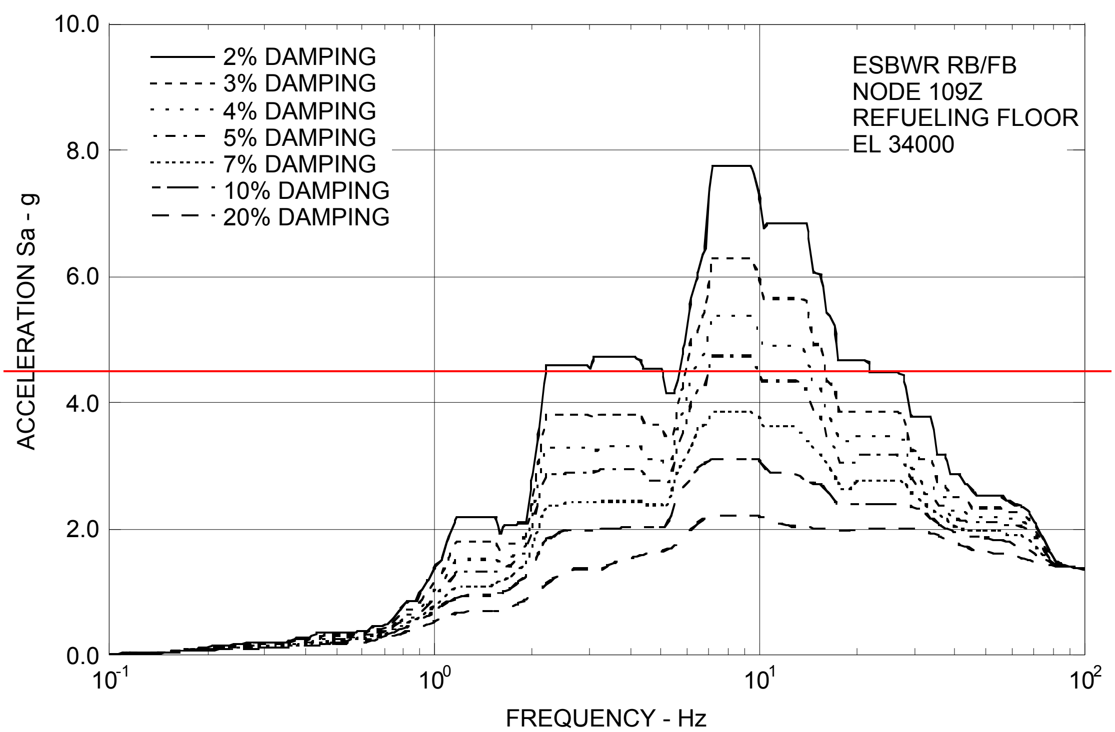


Figure 3A.9-3a. Enveloping Floor Response Spectra – RB/FB Refueling Floor Z

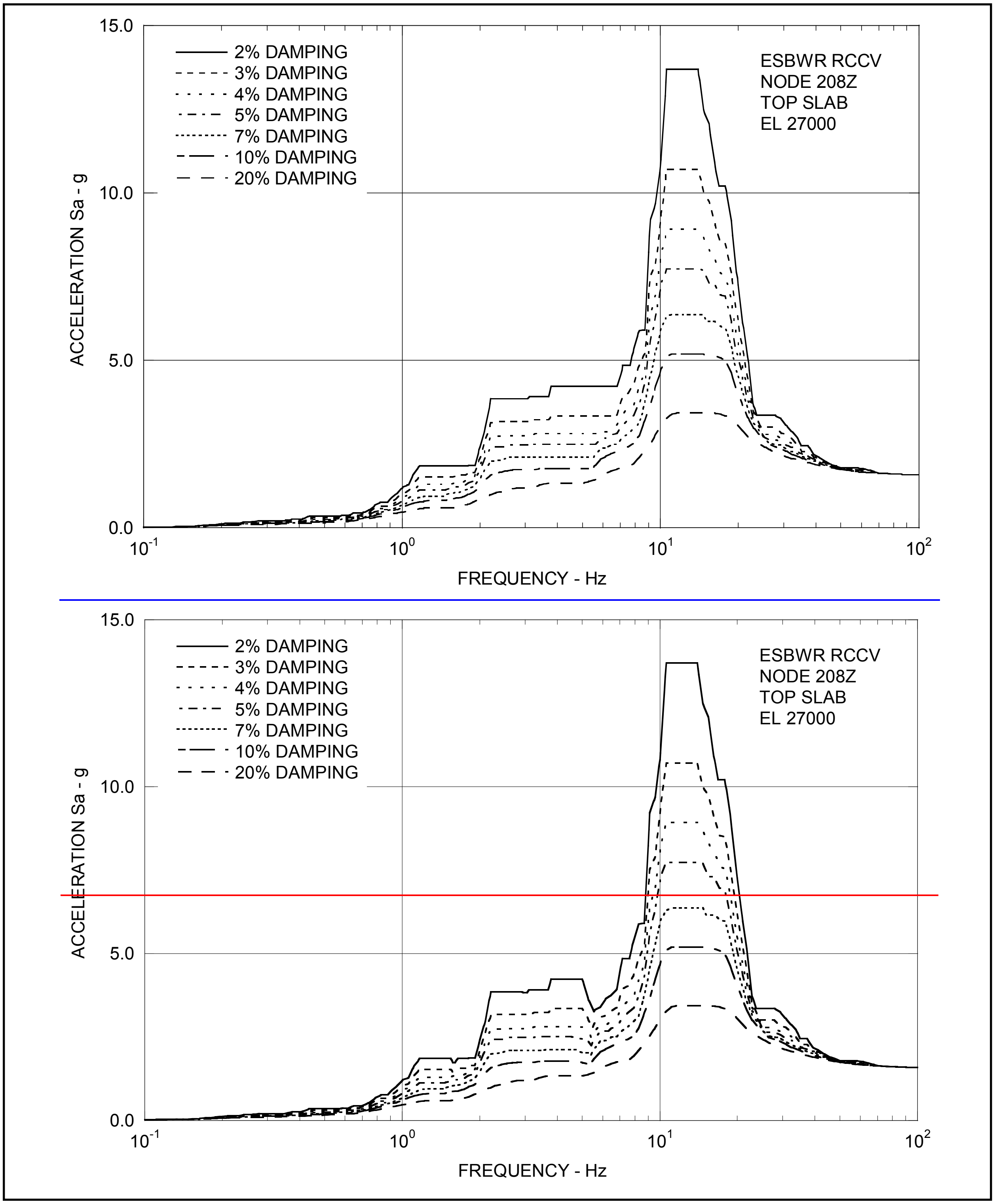


Figure 3A.9-3b. Enveloping Floor Response Spectra – RCCV Top Slab Z

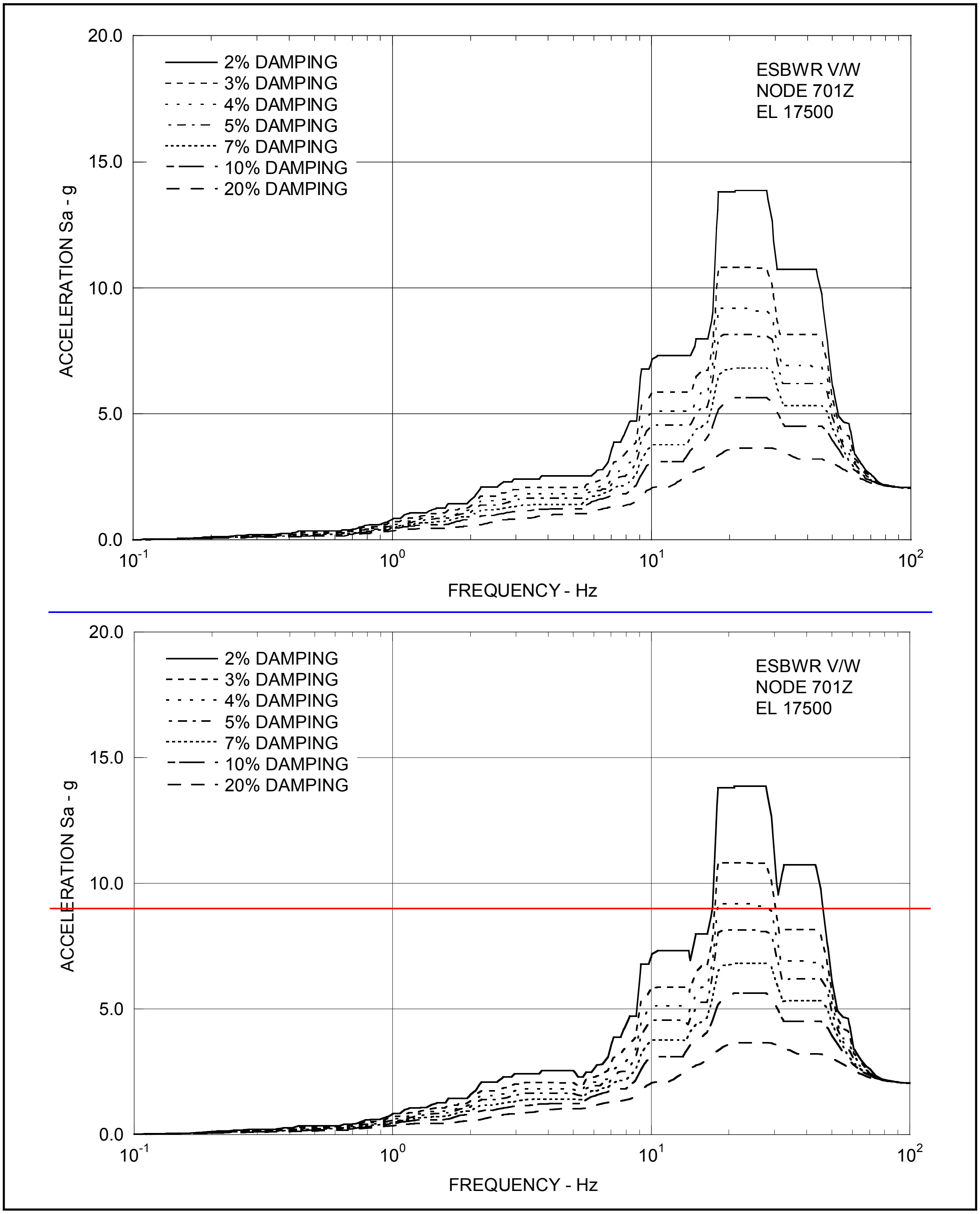


Figure 3A.9-3c. Enveloping Floor Response Spectra – Vent Wall Top Z

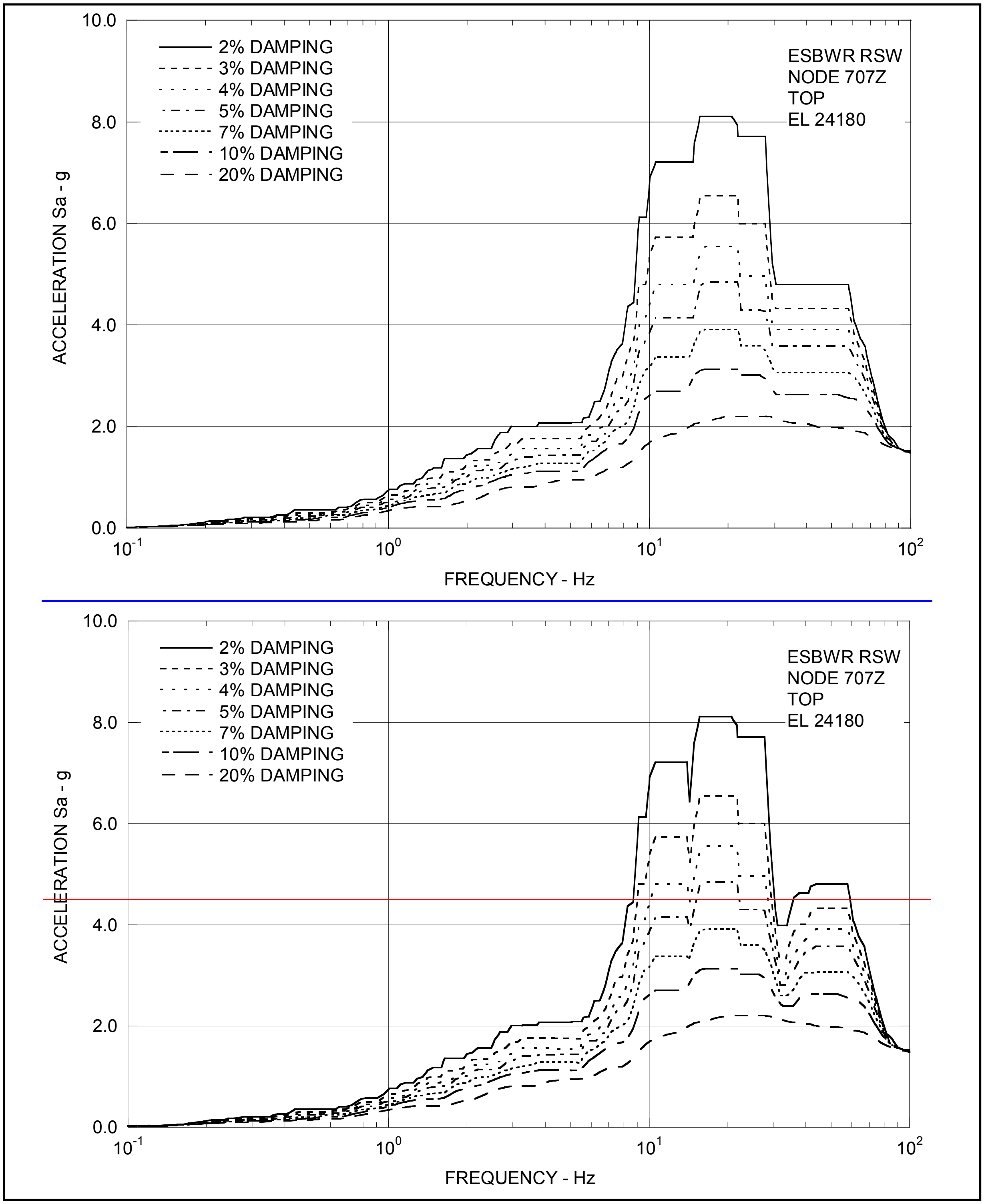


Figure 3A.9-3d. Enveloping Floor Response Spectra – RSW Top Z

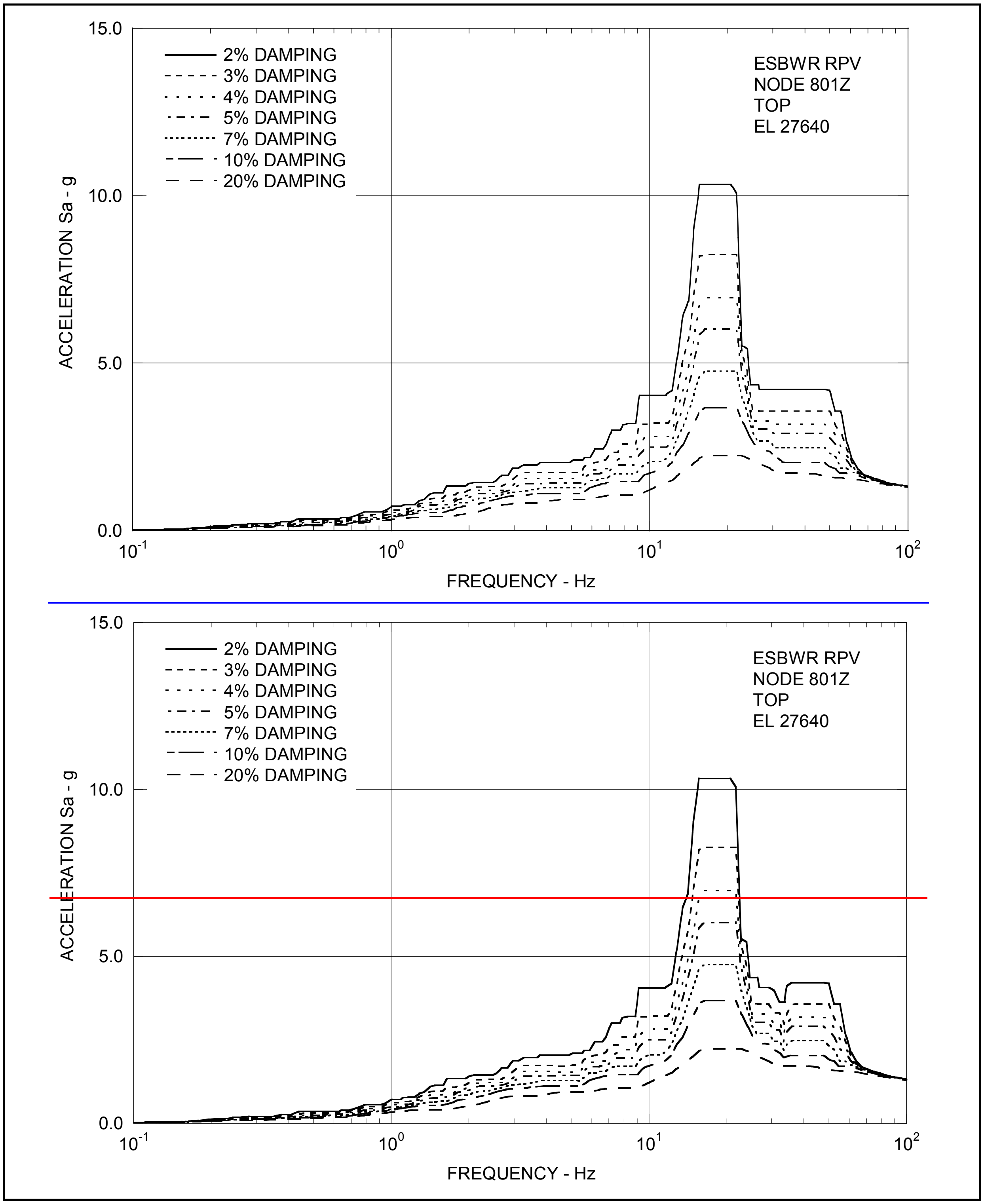


Figure 3A.9-3e. Enveloping Floor Response Spectra – RPV Top Z

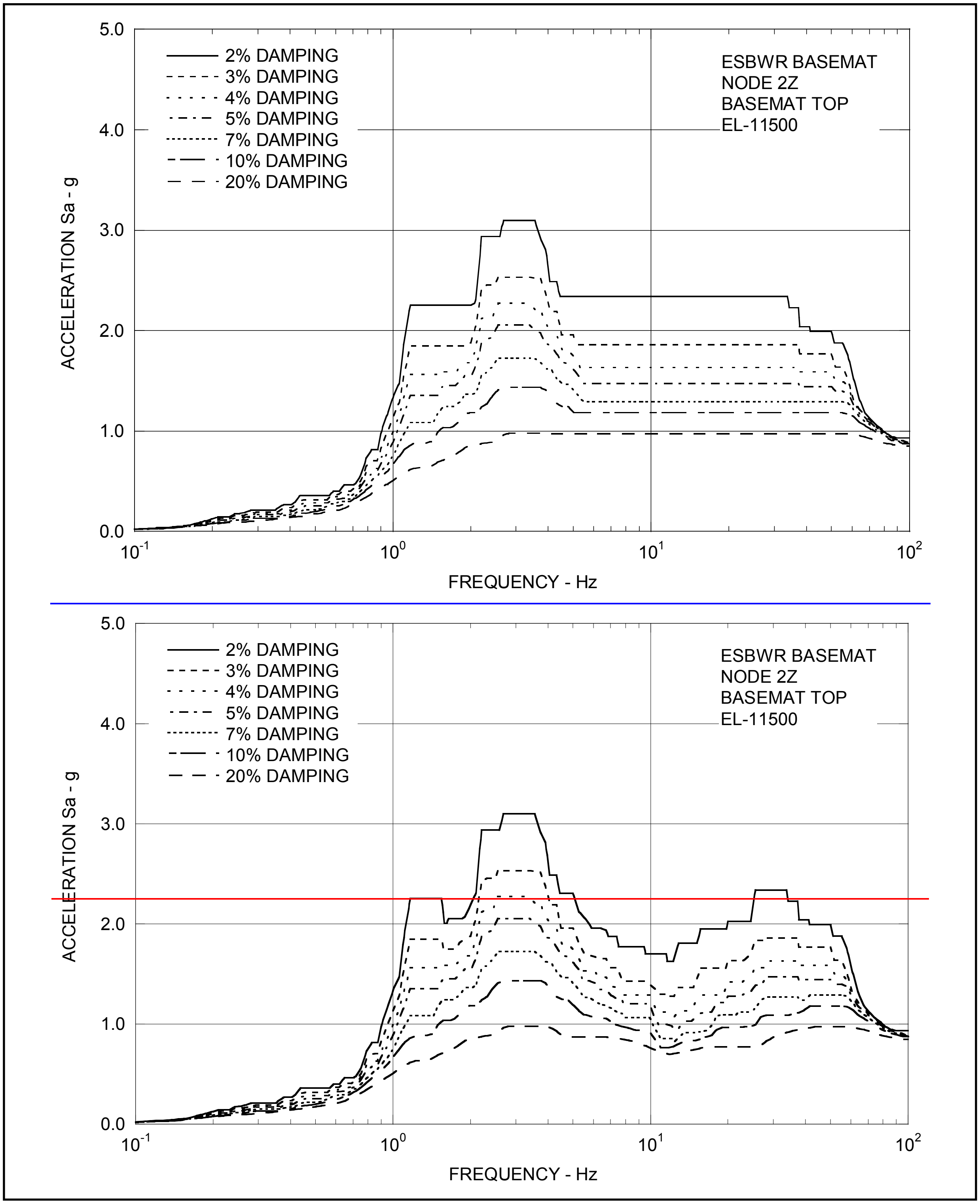
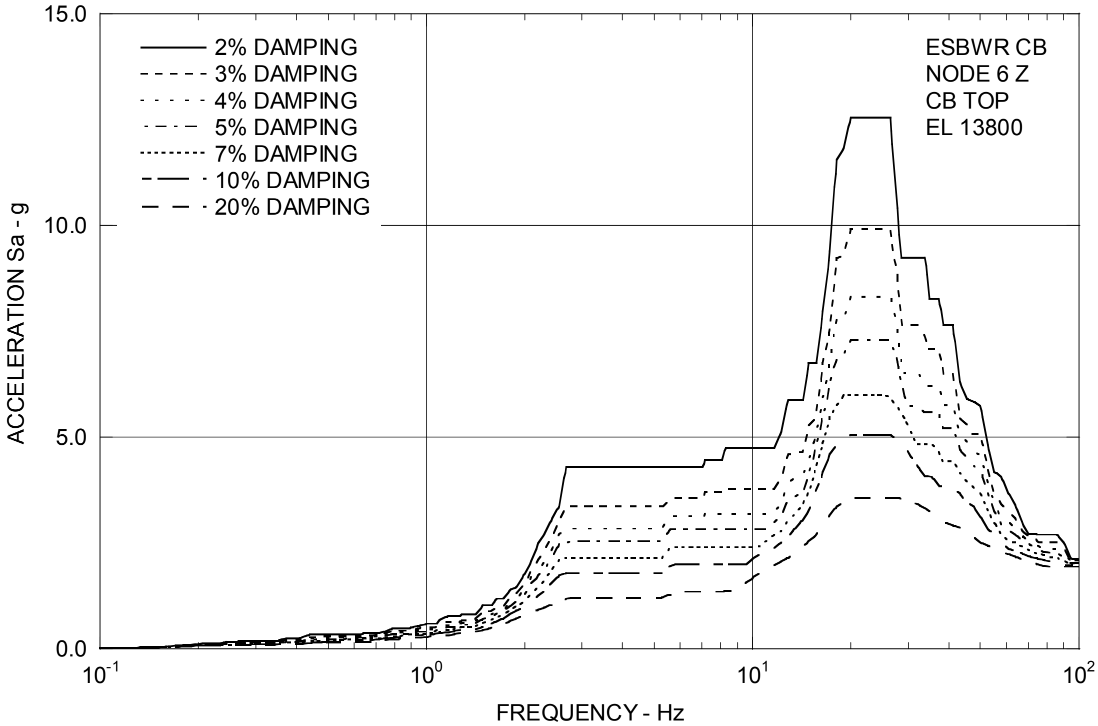


Figure 3A.9-3f. Enveloping Floor Response Spectra – RB/FB Basemat Z



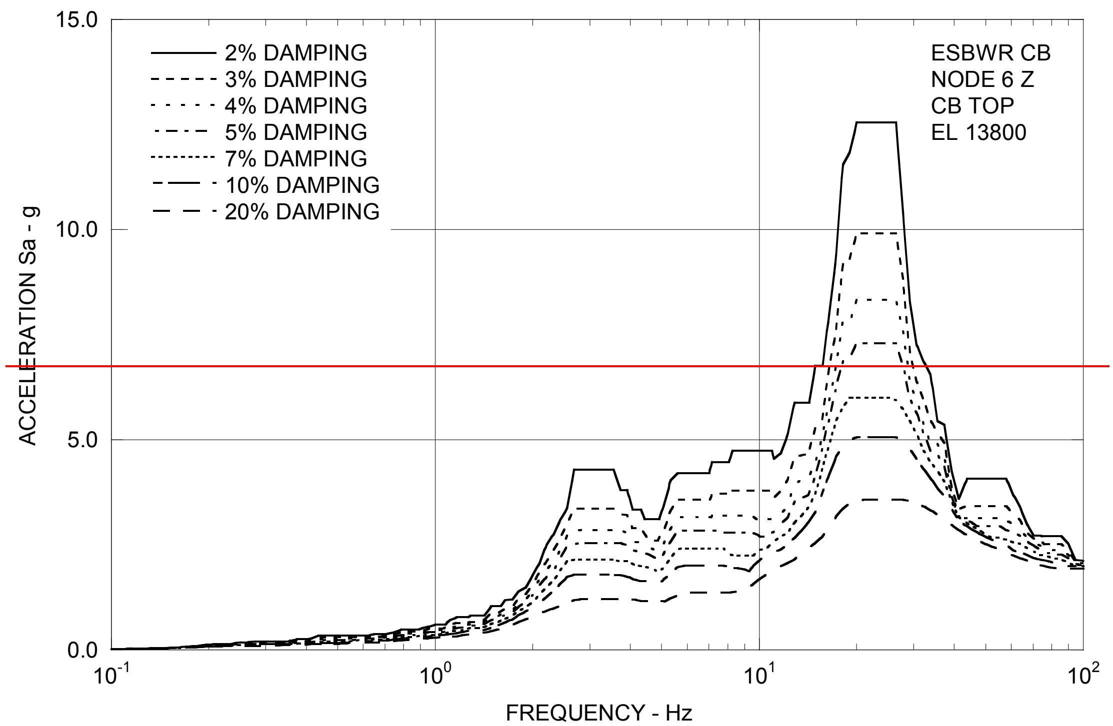


Figure 3A.9-3g. Enveloping Floor Response Spectra – CB Top Z

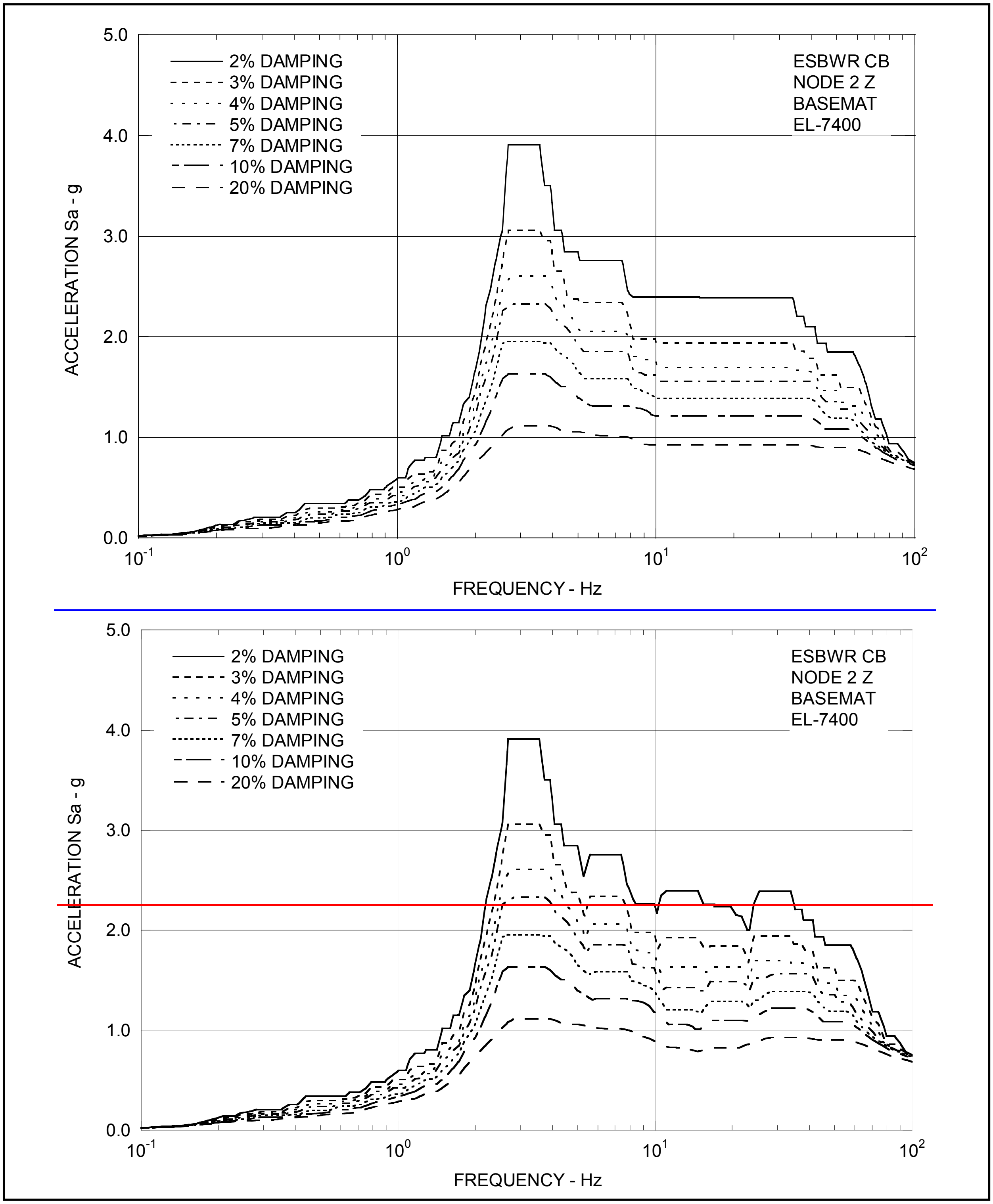
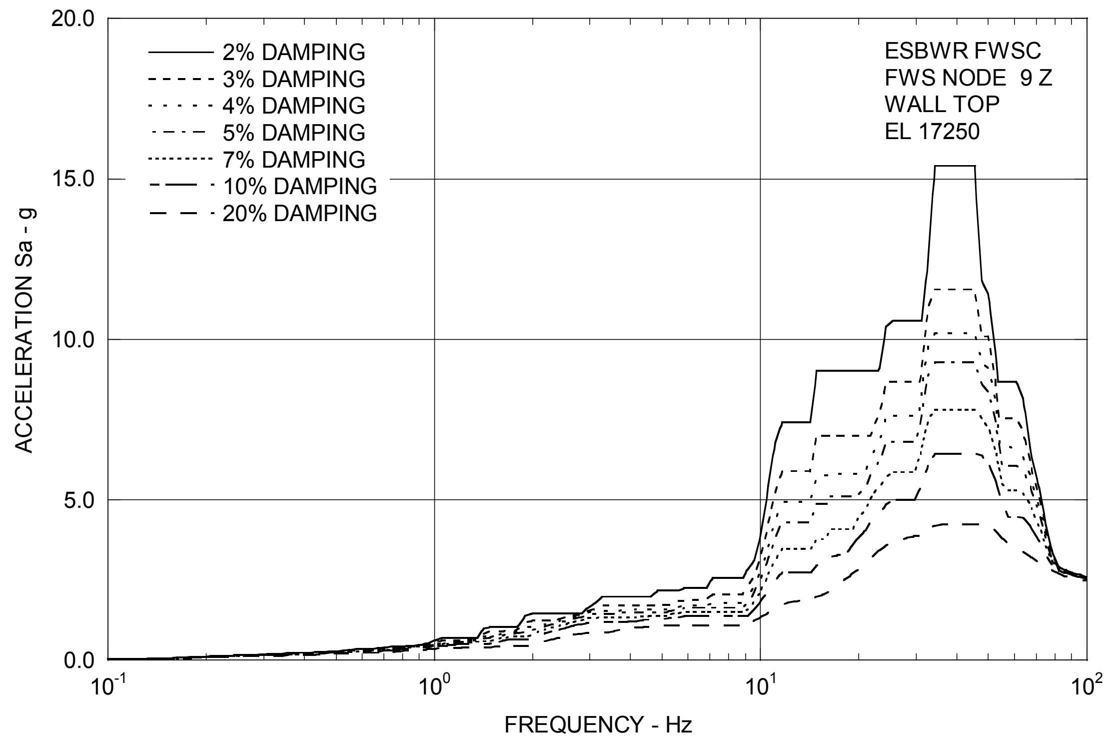


Figure 3A.9-3h. Enveloping Floor Response Spectra – CB Basemat Z



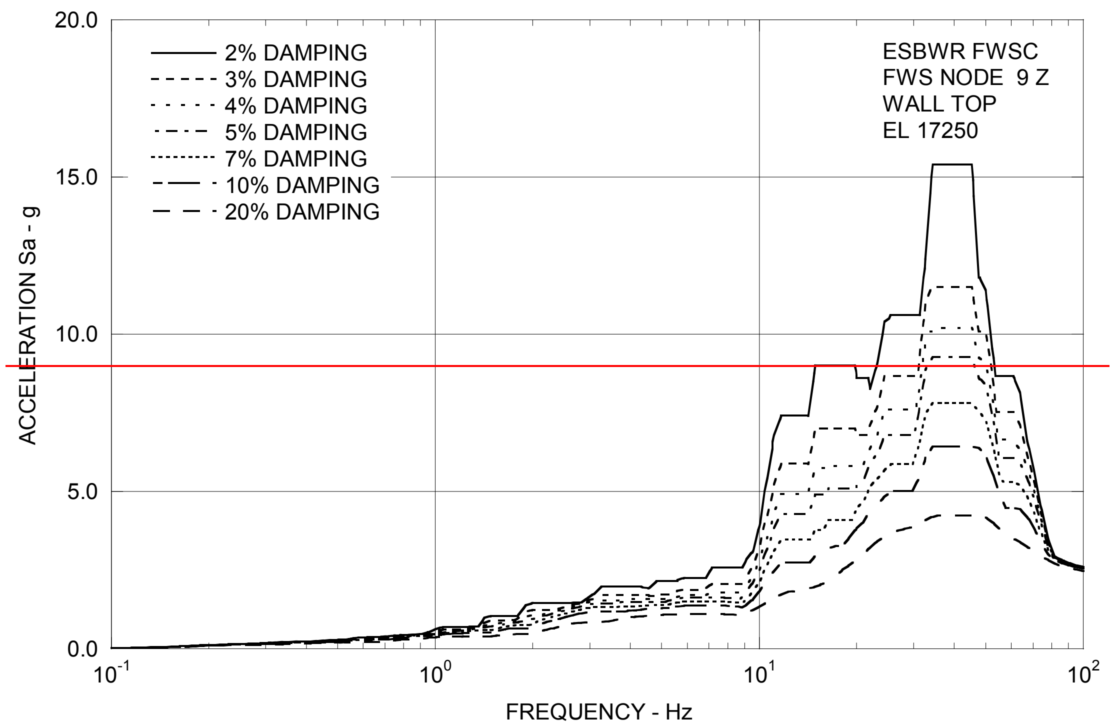


Figure 3A.9-3i. Enveloping Floor Response Spectra – FWS Wall Top Z

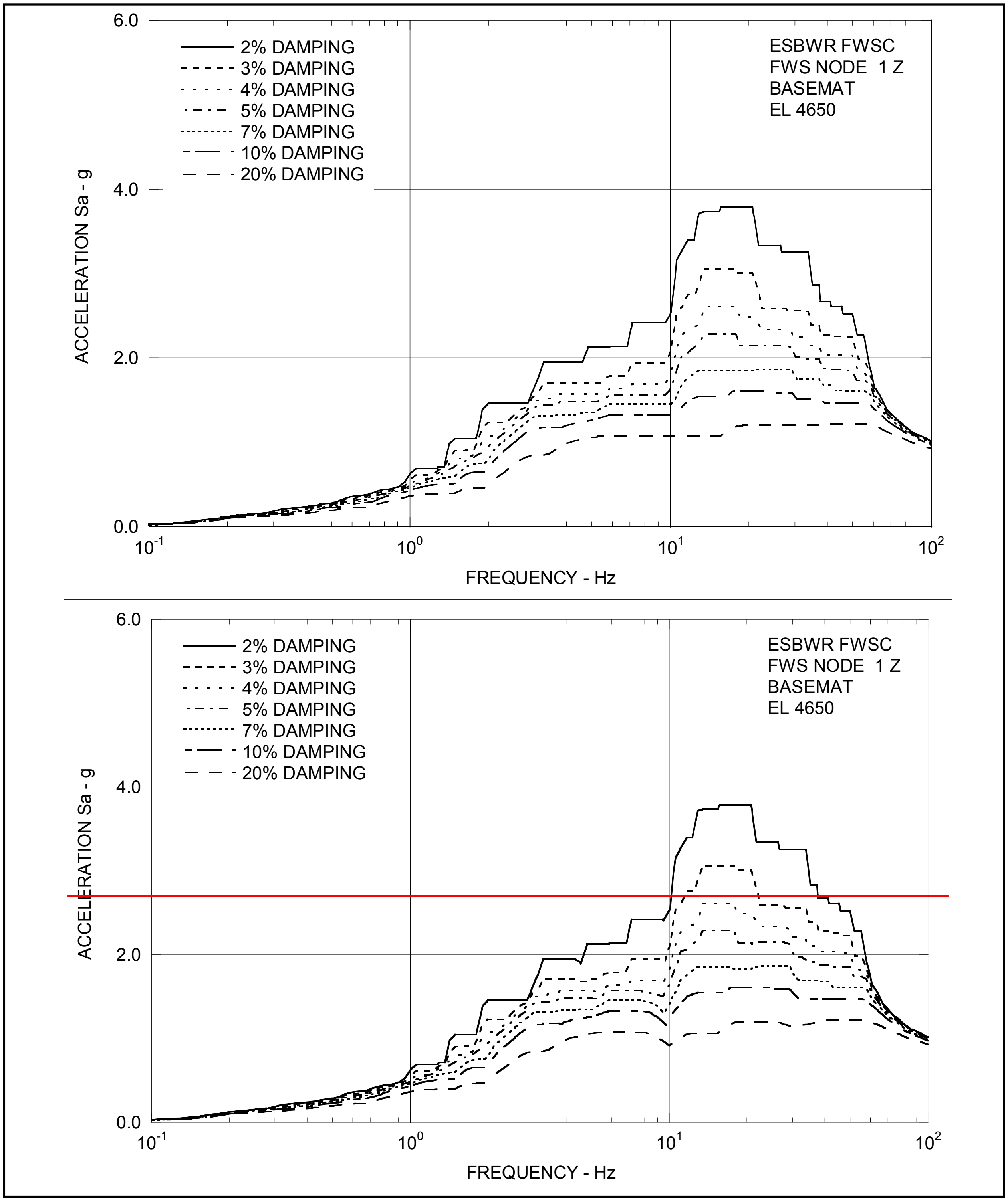
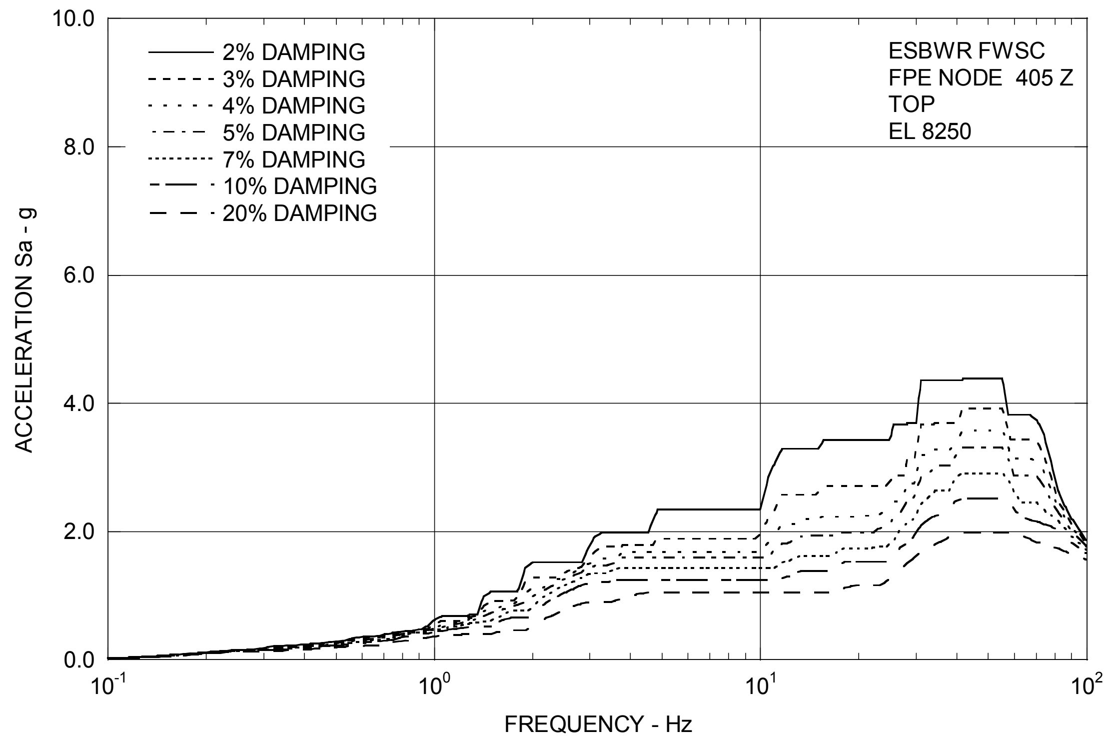


Figure 3A.9-3j. Enveloping Floor Response Spectra – FWS Basemat Z



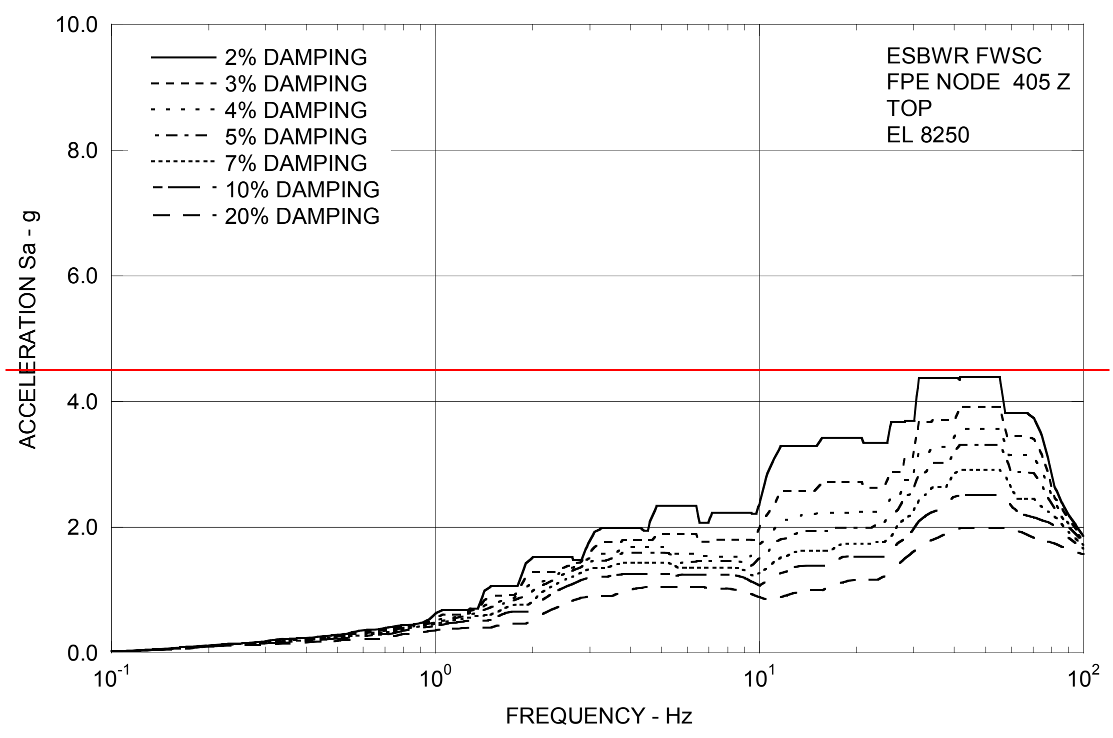


Figure 3A.9-3k. Enveloping Floor Response Spectra – FPE Top Z

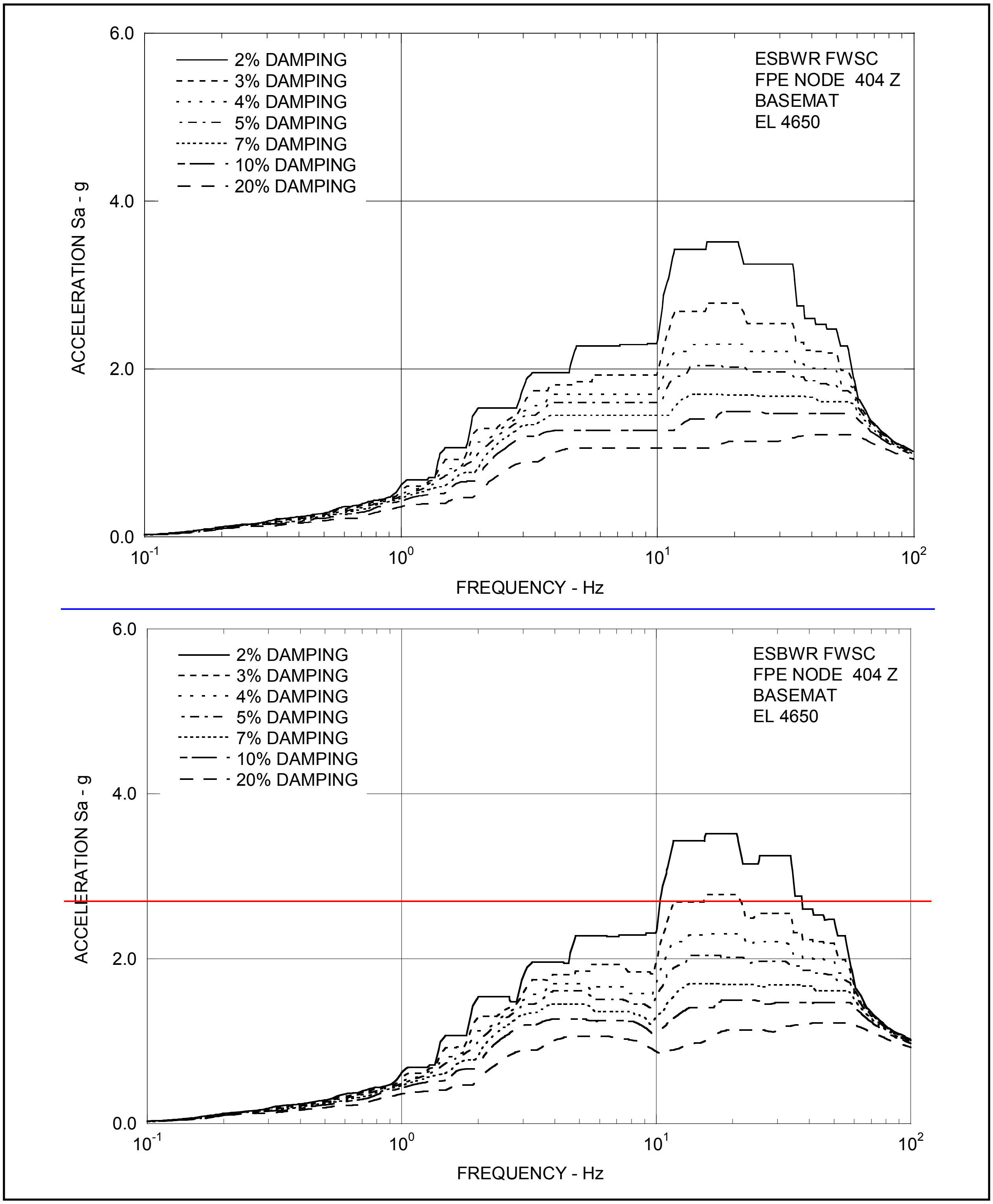


Figure 3A.9-3l. Enveloping Floor Response Spectra – FPE Basemat Z

Table 3G.1-57

Factors of Safety for Foundation Stability

Load Combination	Overturning		Sliding		Floatation	
	Required	Actual	Required	Actual	Required	Actual
D + H + E’	1.1	111.1	1.1	1.53~~41~~	--	--
D + F’	--	--	--	--	1.1	3.48

Where,

D = Dead Load

H = Lateral soil pressure

E’ = Safe Shutdown Earthquake

F’ = Buoyant forces of design basis flood

Table 3G.1-58

Maximum Dynamic Soil Bearing Pressure~~Stress~~ Involving SSE + Static

	Site Condition*		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
Bearing Stress (MPa)	1.1~~2.7~~	2.7~~7.3~~	1~~5~~.41

* See Table 3A.3-1 for site properties. For site specific application, use the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level.

Table 3G.2-26

Factors of Safety for Foundation Stability

Load Combination	Overturning		Sliding		Floatation	
	Required	Actual	Required	Actual	Required	Actual
D + H + E’	1.1	62.5	1.1	1.10~~28~~	--	--
D + F’	--	--	--	--	1.1	1.85

Where,

D = Dead Load

H = Lateral soil pressure

E’ = Safe Shutdown Earthquake

F’ = Buoyant forces of design basis flood

Table 3G.2-27

Maximum Dynamic Soil Bearing ~~Stress~~Pressure Involving SSE + Static

	Site Condition*		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
Bearing Stress (MPa)	~~2~~0.~~8~~50	~~2~~2.~~5~~2	~~2~~0.42

* See Table 3A.3-1 for site properties. For site specific application, use the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level.

Table 3G.4-22

Factors of Safety for Foundation Stability

Load Combination	**Overturning**		**Sliding**		**Floatation**	
	Required	Actual	Required	Actual	Required	Actual
D + H + E’	1.1	129.1	1.1	1.10~~3~~	--	--
D + F’	--	--	--	--	1.1	7.4

Where,

D = Dead Load

H = Lateral soil pressure

E’ = Safe Shutdown Earthquake

F’ = Buoyant forces of design basis flood

Table 3G.4-23

Maximum Dynamic Soil Bearing ~~Stress~~Pressure Involving SSE + Static

	Site Condition*		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
Bearing Stress (MPa)	0.4~~4~~6	0.69~~54~~	1~~0~~.~~67~~2

*: See Table 3A.3-1 for site properties. For site specific application, use the larger value or a linearly interpolated value of the applicable range of shear wave velocities at the foundation level.