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MFN 06-407
Supplement 13

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Subject: **Revised Response to Portion of NRC RAI Letter No. 166 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 – Seismic Category I Structures; RAI Number 3.8-94, Supplement 3**

The purpose of this letter is to submit the GE Hitachi Nuclear Energy (GEH) revised response to the U.S. Nuclear Regulatory Commission (NRC) Request for Additional Information (RAI) sent by NRC letter dated March 28, 2008 (Reference 2). Previous NRC requests and GEH responses were transmitted via references 3 through 6. RAI Number 3.8-94 Supplement 3 is addressed in Enclosure 1.

This revised response supersedes the original response to RAI 3.8-94 Supplement 3 submitted via MFN 06-407, S10 (Reference 1). The purpose of this revision is to make the RAI 3.8-94 S03 DCD Revision 6 Tier 2 Table 2.0-1, Note (17) markup consistent with the response to RAI 2.5-10, which was transmitted to the NRC on January 30, 2009 via MFN 09-083 (Reference 7). Verified DCD changes associated with this revised 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

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NRO

References:

1. MFN 06-407, Supplement 10, Letter from Richard E. Kingston to U.S. Nuclear Regulatory Commission, *Response to Portion of NRC RAI Letter No. 166 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 – Seismic Category I Structures; RAI Number 3.8-94, Supplement 3,* dated December 9, 2008

2. MFN 08-316 Letter from U.S. Nuclear Regulatory Commission to Robert E. Brown, GEH, *Request For Additional Information Letter No. 166 Related to ESBWR Design Certification Application*, dated March 28, 2008

3. MFN 06-407, Supplement 3, Letter from James C. Kinsey to U.S. Nuclear Regulatory Commission, *Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application – DCD Tier 2 Section 3.8 – Seismic Category I Structures - RAI Numbers 3.8-28 S02, 3.8-76 S02, 3.8-93 S02, 3.8-94 S02, 3.8-96 S02, 3.8-101 S02, 3.8-102 S02 and 3.8-103 S02*, dated November 28, 2007

4. MFN 06-407, Supplement 2, Letter from James C. Kinsey to U.S. Nuclear Regulatory Commission, *Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application –Structural Analysis – RAI Numbers 3.8-81 S01, 3.8-85 S01, 3.8-86 S01, 3.8-88 S01, 3.8-92 S01, 3.8-92 S01, 3.8-93 S01, 3.8-94 S01, 3.8-96 S01 and 3.8-99 S01*, dated March 26, 2007.

5. MFN 06-407, Letter from David H. Hinds to U.S. Nuclear Regulatory Commission, Response to Portion of NRC Request for Additional Information Letter No. 38 Related to ESBWR Design Certification Application –Structural Analysis – RAI Numbers 3.8-17, 3.8-24, 3.8-28, 3.8-32, 3.8-33 through 3.8-38, 3.8-44, 3.8-59, 3.8-62, 3.8-65, 3.8-69, 3.8-73, 3.8-76, 3.8-77, 3.8-79, 3.8-80, 3.8-81, 3.8-84, 3.8-85, 3.8-86, 3.8-88, 3.8-89, 3.8-92, 3.8-93 through 3.8-97, 3.8-99, 3.8-101, 3.8-102 and 3.8-103, dated November 8, 2006.

6. MFN 06-197 Letter from U.S. Nuclear Regulatory Commission to David H. Hinds, General Electric Company, *Request For Additional Information Letter No. 38 Related to ESBWR Design Certification Application*, dated July 7, 2006

7. MFN 09-083, Letter from Richard E. Kingston to U.S. Nuclear Regulatory Commission, *Response to NRC Request for Additional Information Letter No. 266 Related to ESBWR Design Certification Application –Site Characteristics – RAI Number 2.5-10,* dated January 30, 2009

Enclosure:

1. Revised Response to Portion of NRC RAI Letter No. 166 Related to ESBWR Design Certification Application - DCD Tier 2 Section 3.8 – Seismic Category I Structures; RAI Number 3.8-94 S03

cc:	AE Cubbage	USNRC (with enclosures)
	RE Brown	GEH/Wilmington (with enclosures)
	DH Hinds	GEH/Wilmington (with enclosures)
	EDRF Section	0000-0088-8879 R2 (RAI 3.8-94 S03 R1)

ENCLOSURE 1

MFN 06-407
Supplement 13

Revised Response to Portion of NRC RAI Letter No. 166

Related to ESBWR Design Certification Application[1]

DCD Tier 2 Section 3.8 – Seismic Category I Structures

RAI Number 3.8-94 S03 (Revision 1)

Please note, this is a revised response to RAI 3.8-94, Supplement 3 which was originally transmitted via MFN 06-407 S10. Revisions are denoted with ~~red strike through~~ text for deletions and <u>blue underlined text</u> for additions.

For historical purposes, the original text of RAI 3.8-94 and previous supplements and the GEH responses are included. The attachments (if any) are not included from the original response to avoid confusion.

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

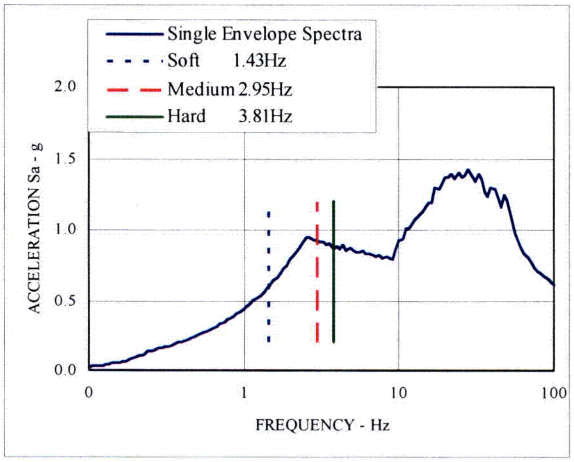
(1) Confirmation of bearing capacity is a COL item as stated in DCD Tier 2 Table 2.0-1. The higher bearing stress at the medium site condition is due to the higher spectral acceleration of the input ground motion response spectra at the SSI fundamental frequencies as shown in Figure 3.8-94(1) in comparison 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-

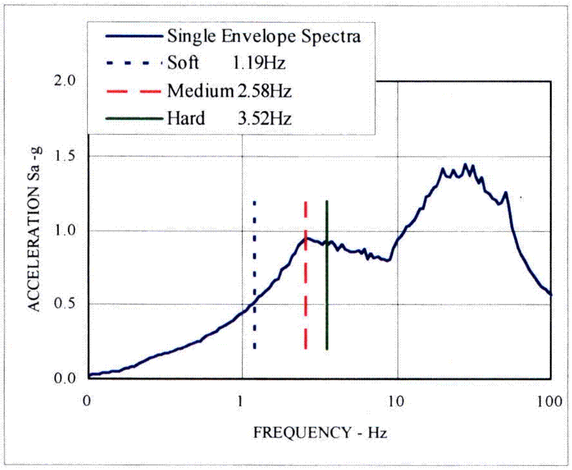
58 and 3G.2-27. The revised pages 3G-123 and 3G-228 in DCD Tier 2 Revision 4 are attached.

(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.

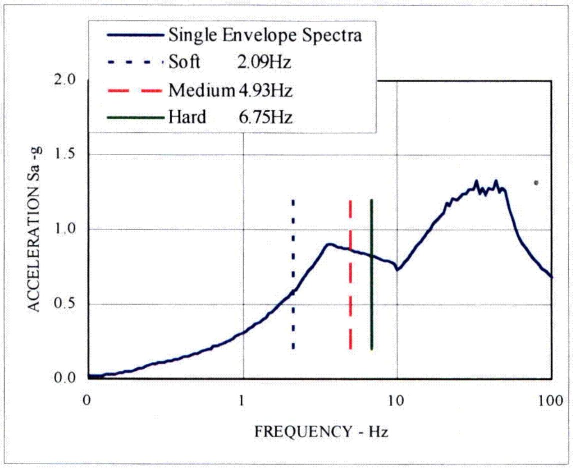
(4) Table 3.8-94(3) is a summary of the analyses results presented in the response to NRC RAI 3.8-93, Supplement 1. The comparisons of the bending moments across the basemat were provided in Figure 3.8-93(16)-c. In that figure Hard Spot case is higher than DCD condition. The allowable bending moment 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:

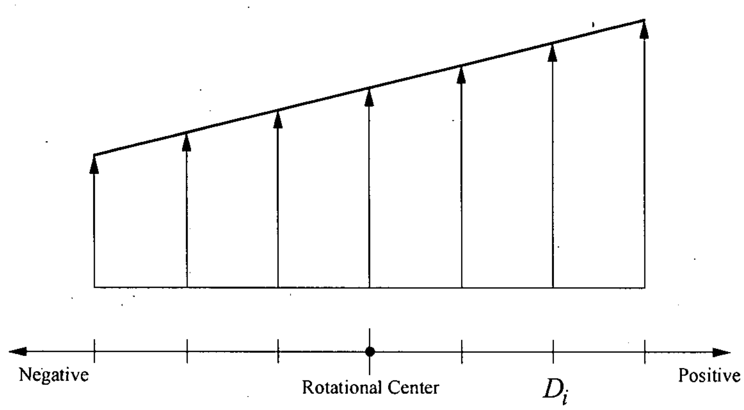
a) Although the RB/FB and the CB are deeply embedded structures, the seismic soil reactions calculated by the DAC3N soil-spring SSI analyses without the embedment effect were used for the stability analysis for these buildings. This extra conservatism is removed in the below bearing pressure demand recalculation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

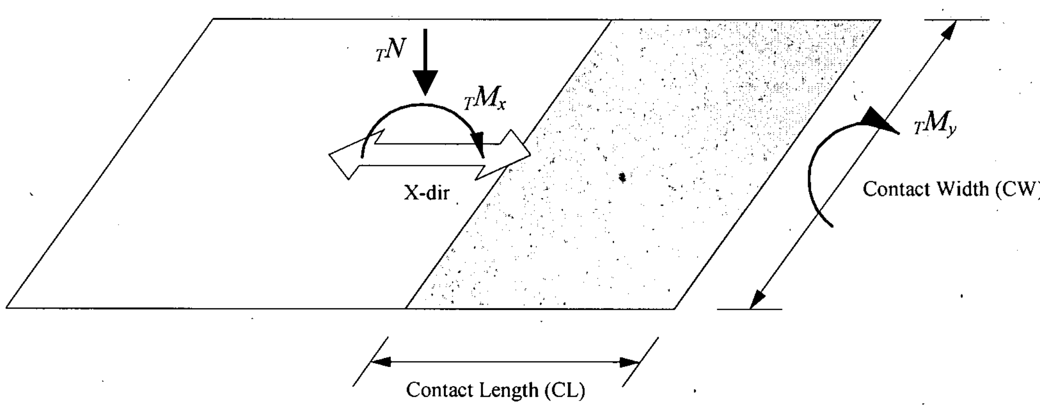
a) Calculate bearing pressure, $_TBP_x$, per the Energy Balance Method using $_TM_x$ and $_TN$

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~~equivalent uniform~~ velocity of the bottom layer divided by the average~~equivalent uniform~~ velocity of the top layer. ~~The equivalent uniform shear velocity is computed using the equation in Note (8) to DCD Tier 2 Table 2.0-1 except that 1) the depth of the soil column is the thickness of the layer under consideration and 2) either~~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. ~~If backfill material is used in any of these layers, the required minimum shear wave velocity of the backfill is determined from the Veq equation in Note (8) to this table setting Veq equal to 300 m/s (1000 ft/s) for the entire soil column with the~~

~~depth defined in Note (8).~~ 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).

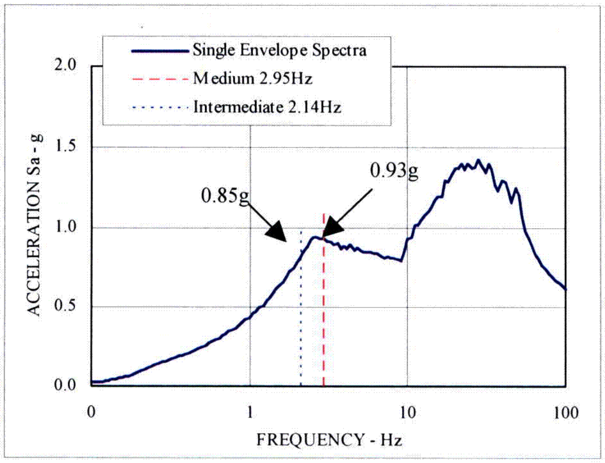
These spectral acceleration ratios are then applied to the SSE base moment and SSE vertical load of the medium site to obtain the corresponding loads for the intermediate site in the bearing pressure calculation and the maximum bearing pressure for the intermediate site is found to be 1.39 MPa.

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).

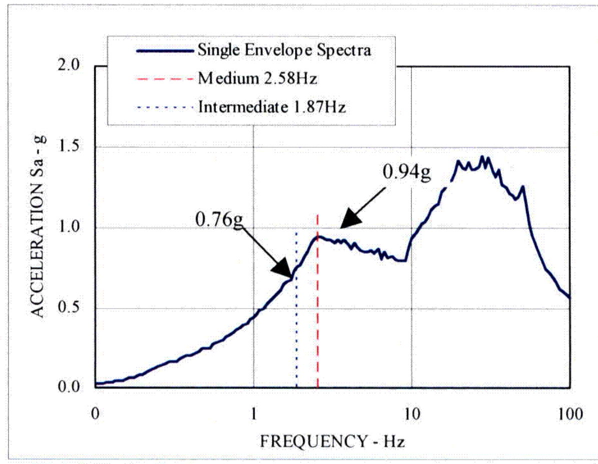
(3) DCD Tier 1 Table 5.1-1 has been revised in Revision 5 to retain only those footnotes in DCD Tier 2 Table 2.0-1 that are intrinsic to the description of the ESBWR Standard Plant site design parameter and are not background information for the parameter. Please see the GEH response to NRC RAI 2.0-1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

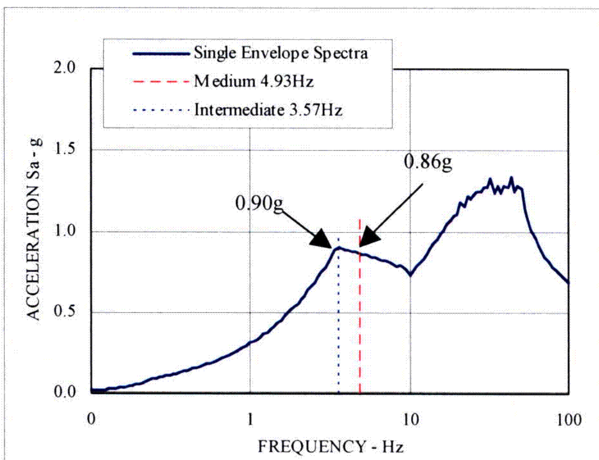
Building	Site Condition		
	Soft (V_s = 300 m/sec)	Medium (V_s = 800 m/sec)	Hard ($V_s \geq$ 1700 m/sec)
RB/FB	1.2	1.5	1.1
CB	0.44	2.2	0.42
FWSC	0.46	0.69	1.2



(a) NS Direction

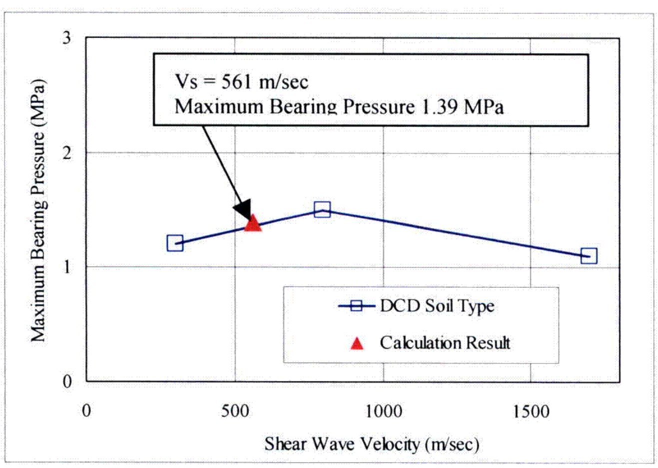


(b) EW Direction



(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

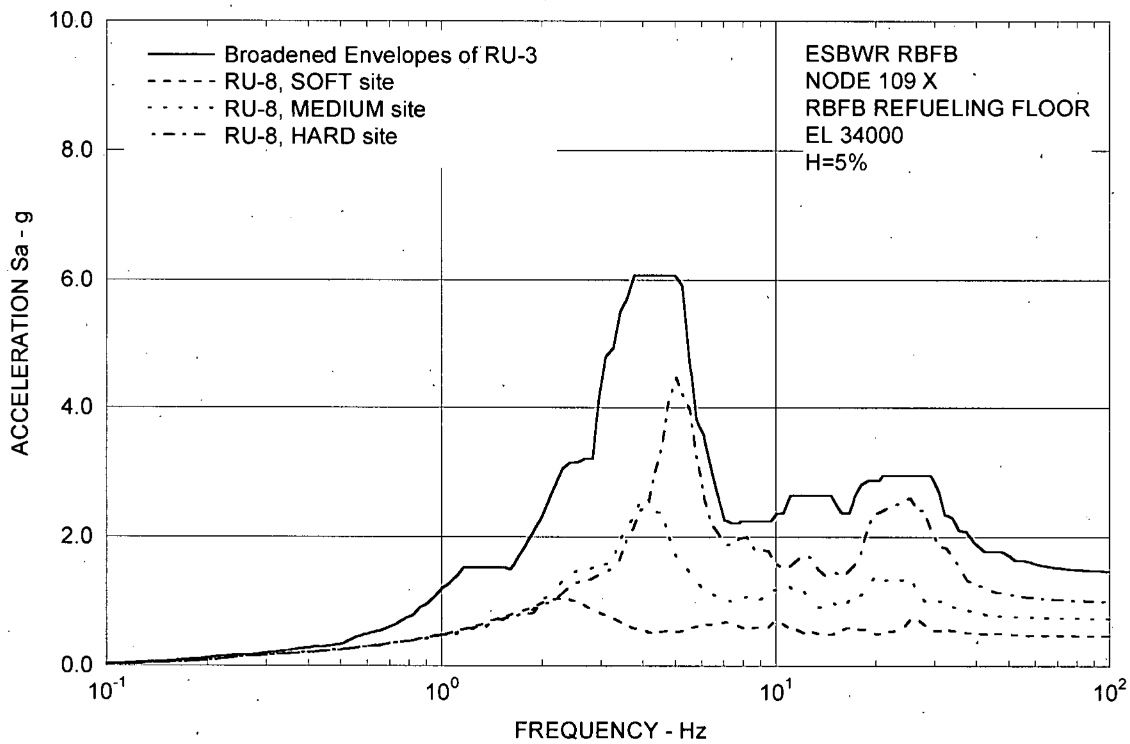


Figure 3.8-94(4) FRS (Compared with the DAC3N) – RBFB Refueling Floor X

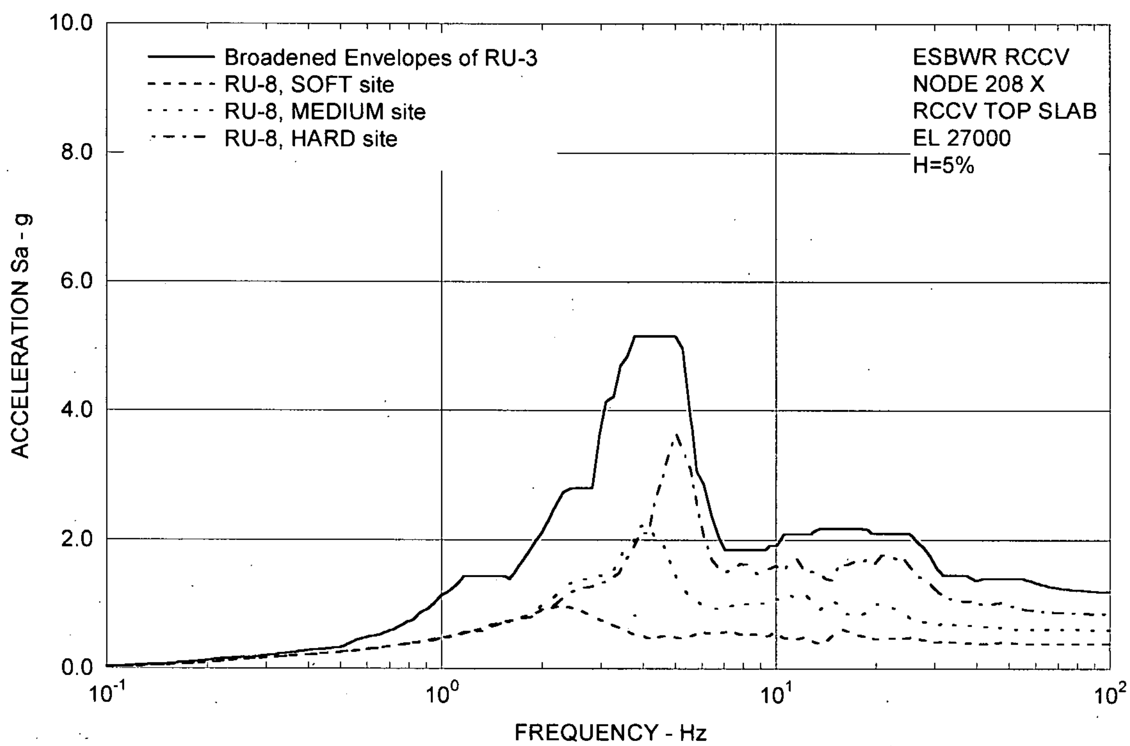


Figure 3.8-94(5) FRS (Compared with the DAC3N) – RCCV Top Slab X

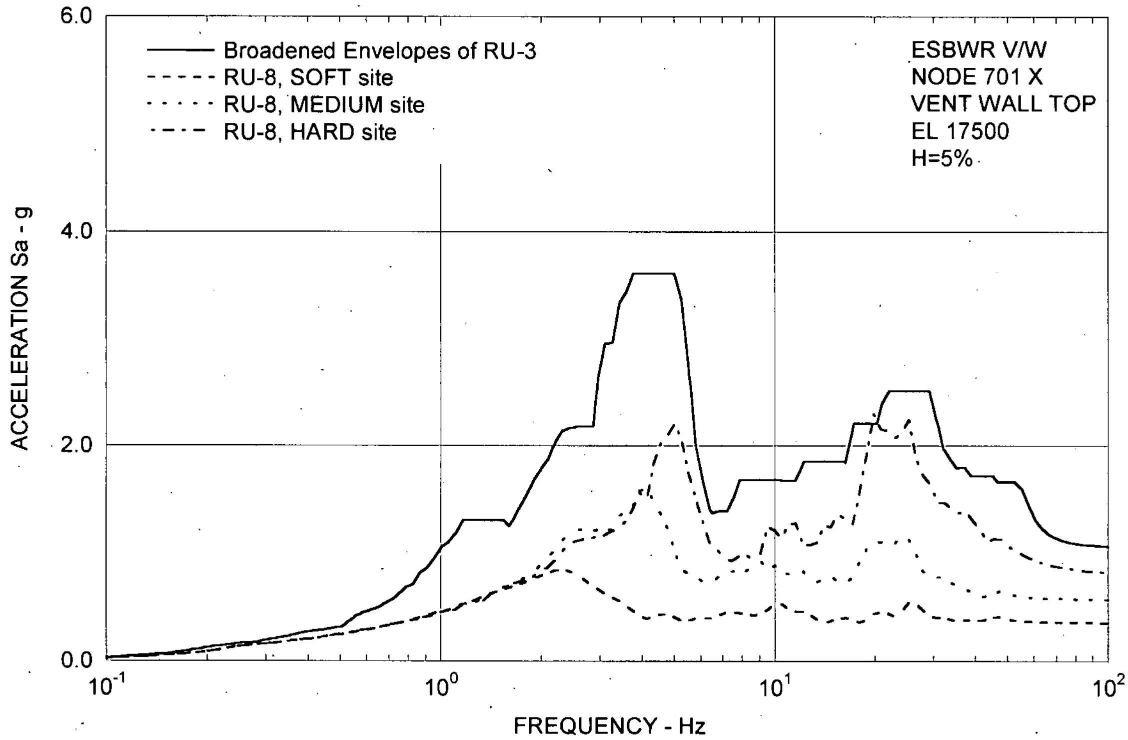


Figure 3.8-94(6) FRS (Compared with the DAC3N) – Vent Wall Top X

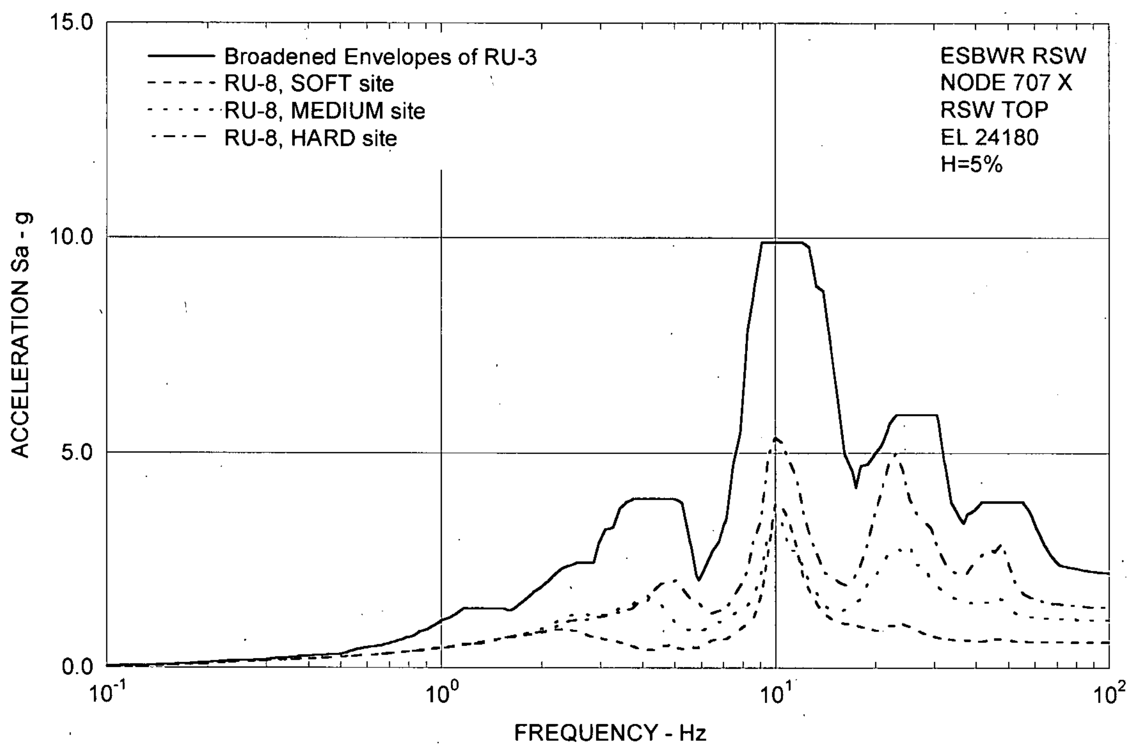


Figure 3.8-94(7) FRS (Compared with the DAC3N) – RSW Top X

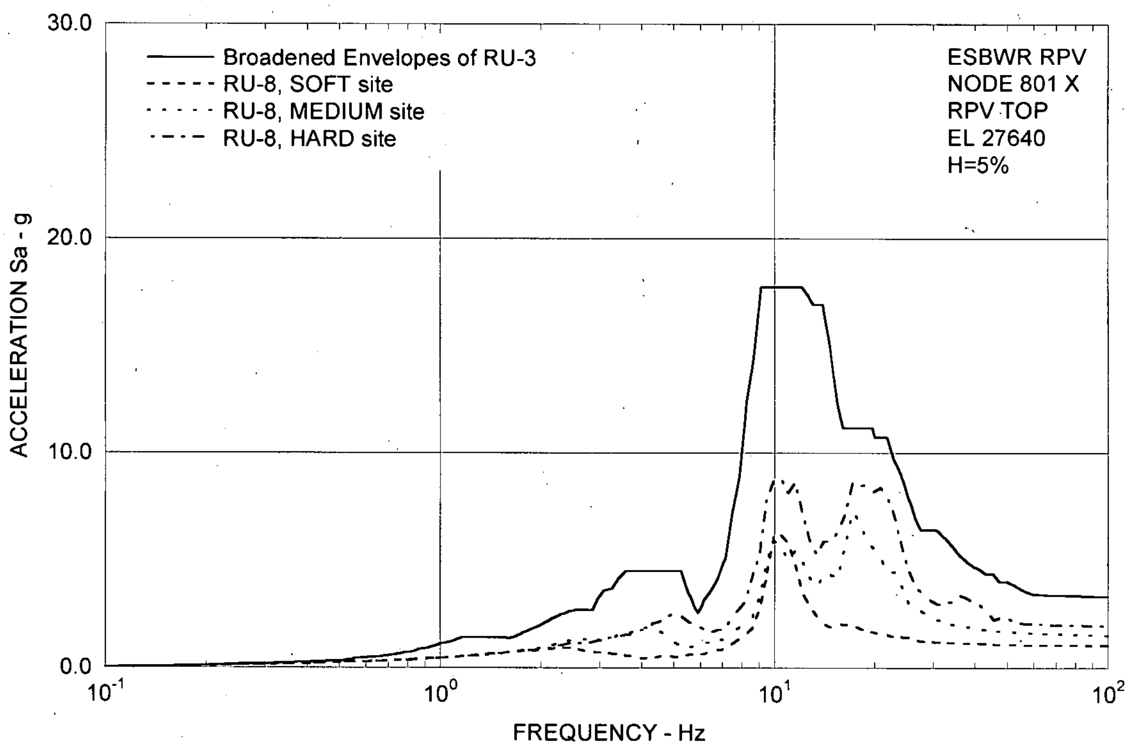


Figure 3.8-94(8) FRS (Compared with the DAC3N) – RPV Top X

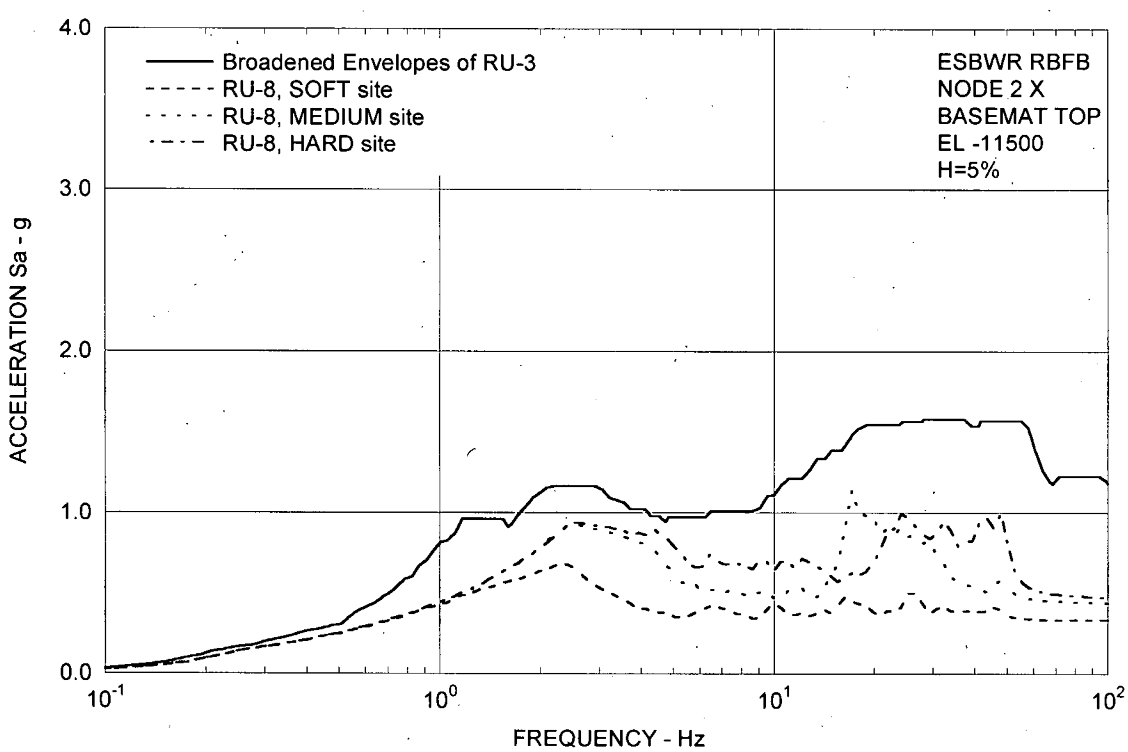


Figure 3.8-94(9) FRS (Compared with the DAC3N) – RBFB Basemat X

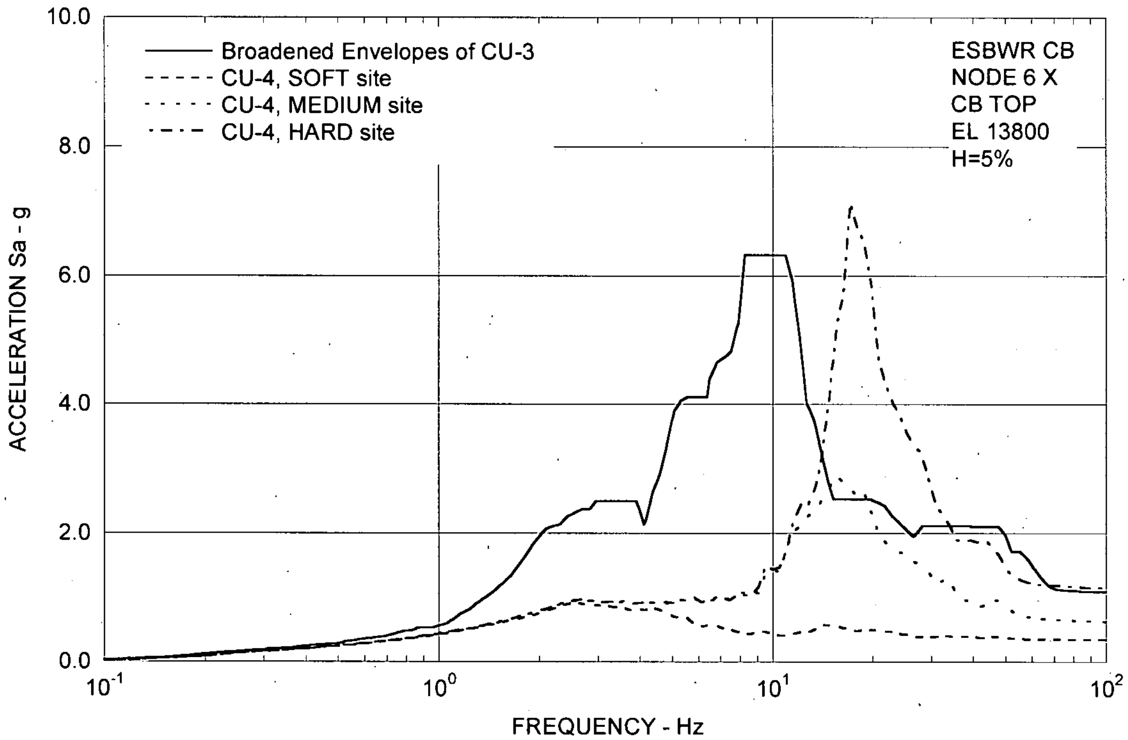


Figure 3.8-94(10) FRS (Compared with the DAC3N) – CB Top X

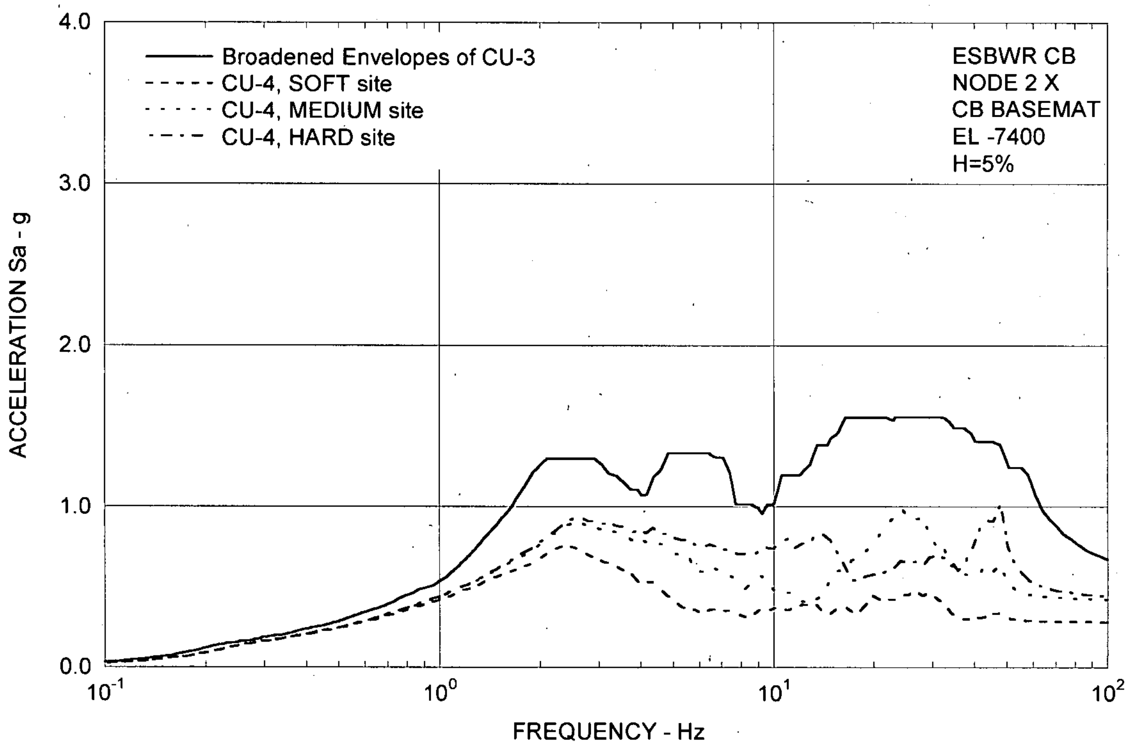


Figure 3.8-94(11) FRS (Compared with the DAC3N) – CB Basemat X

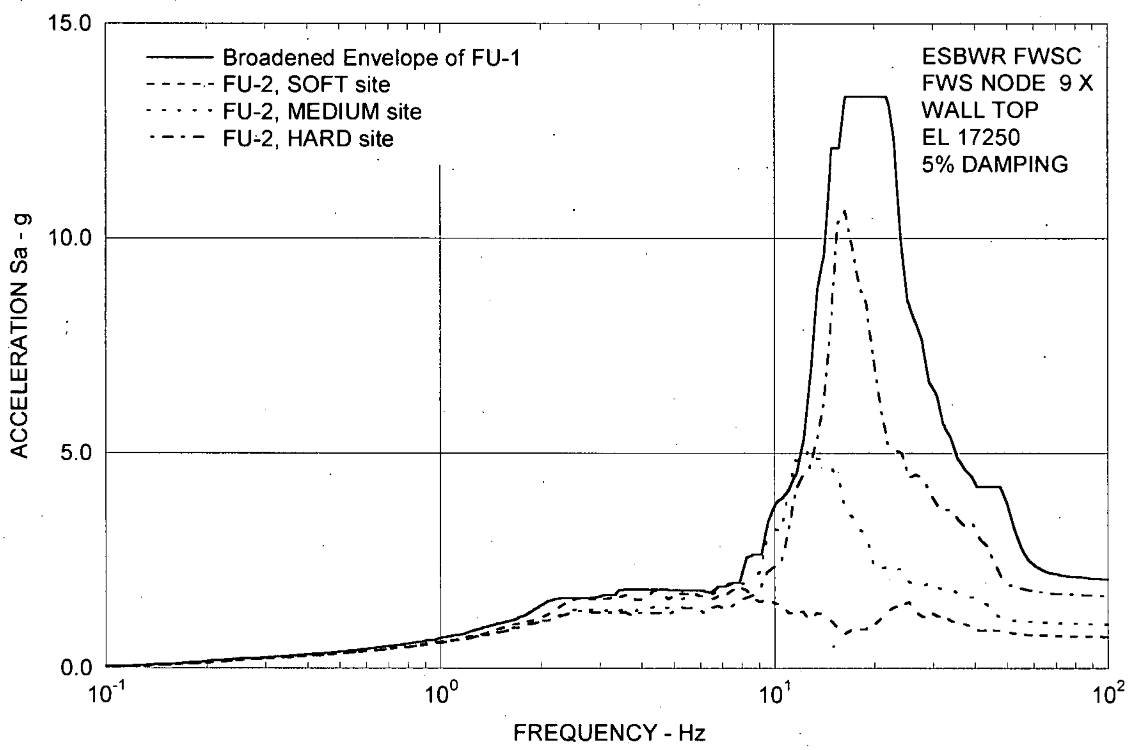


Figure 3.8-94(12) FRS (Compared with the DAC3N) – FWS Wall Top X

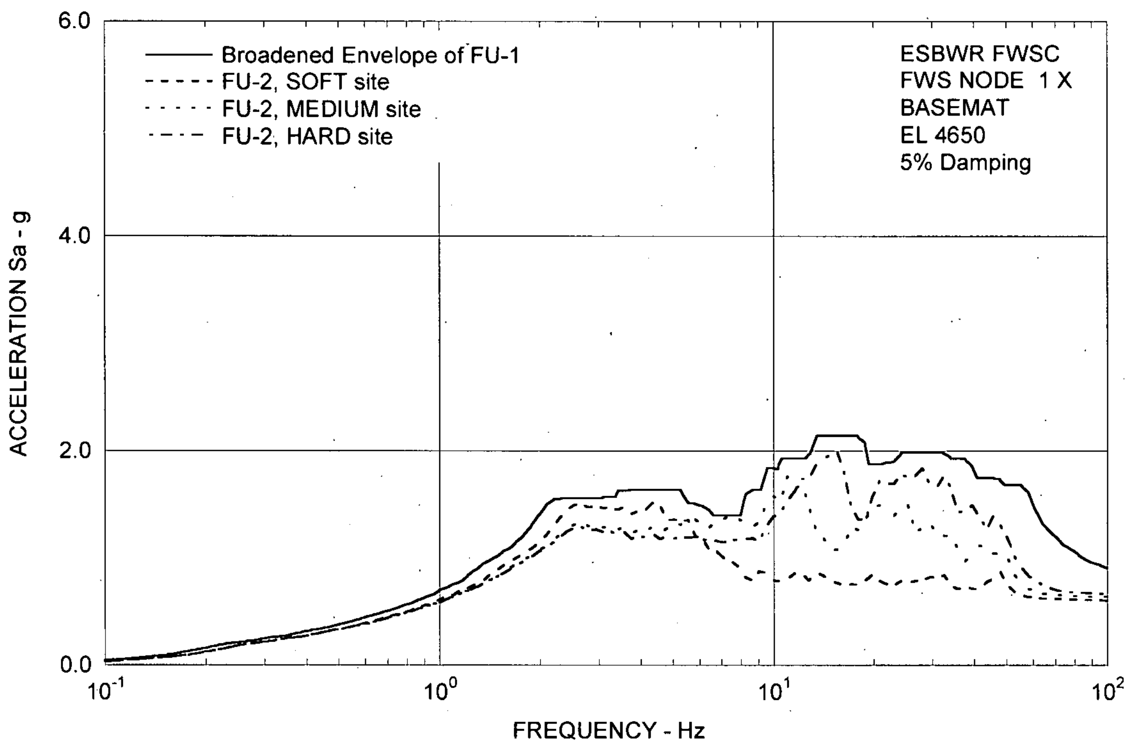


Figure 3.8-94(13) FRS (Compared with the DAC3N) – FWS Basemat X

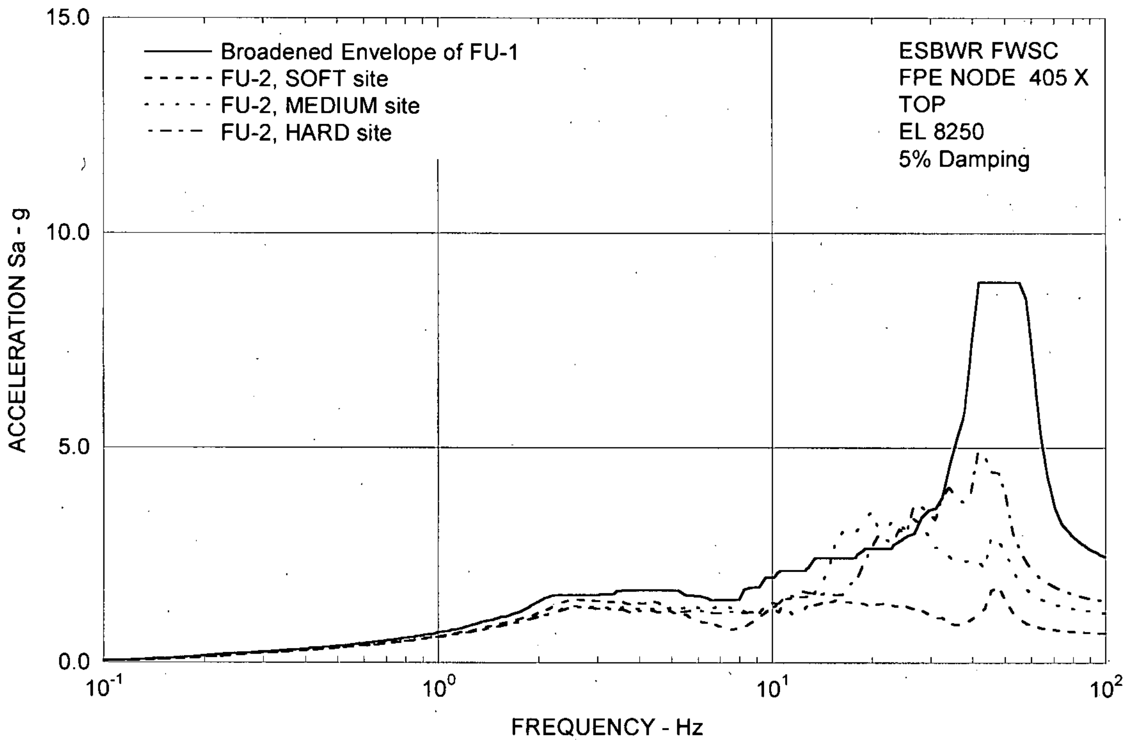


Figure 3.8-94(14) FRS (Compared with the DAC3N) – FPE Top X

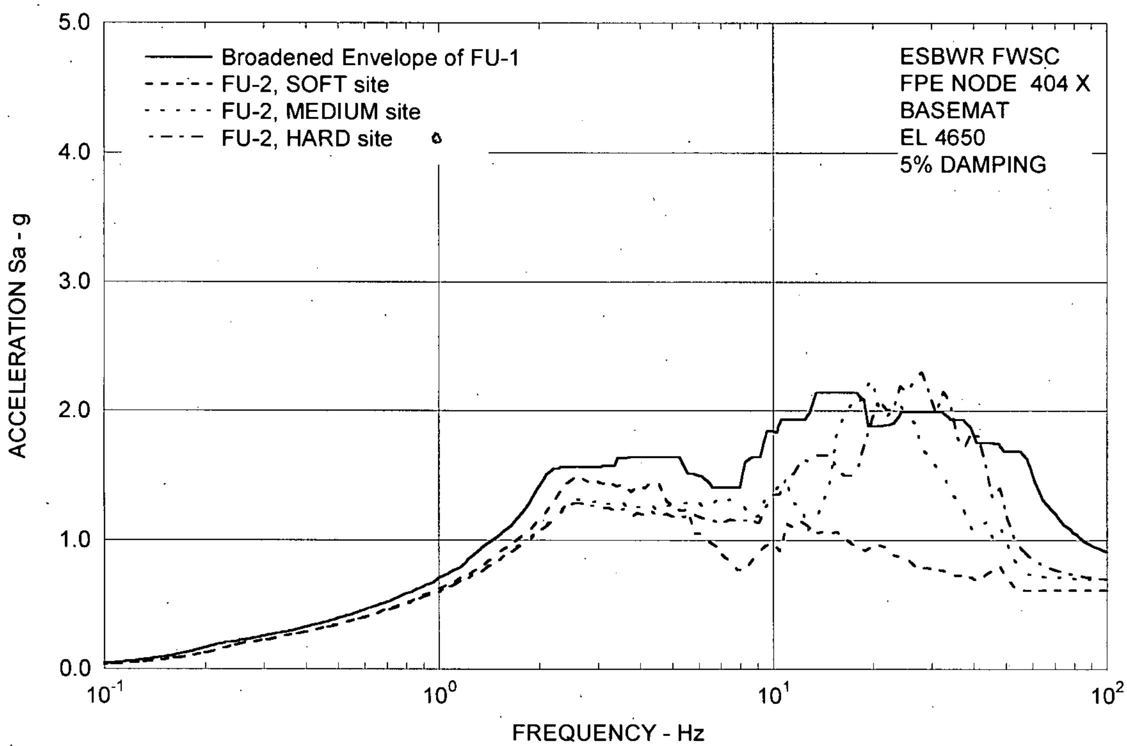


Figure 3.8-94(15) FRS (Compared with the DAC3N) – FPE Basemat X

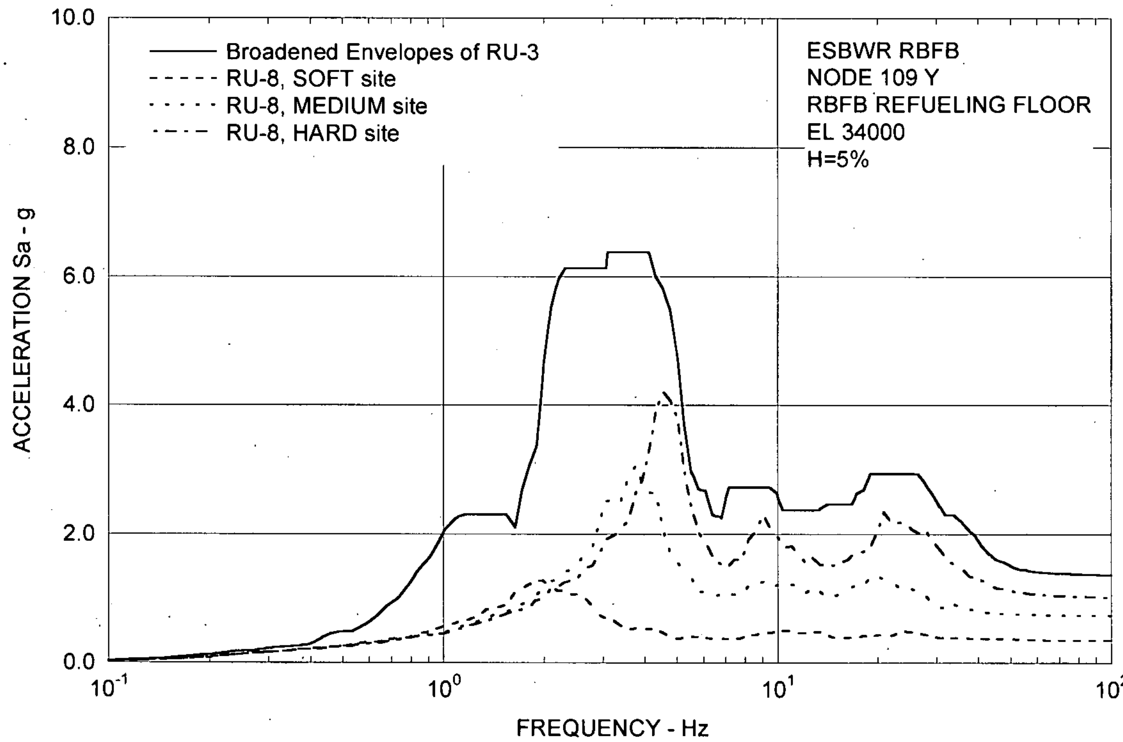


Figure 3.8-94(16) FRS (Compared with the DAC3N) – RBFB Refueling Floor Y

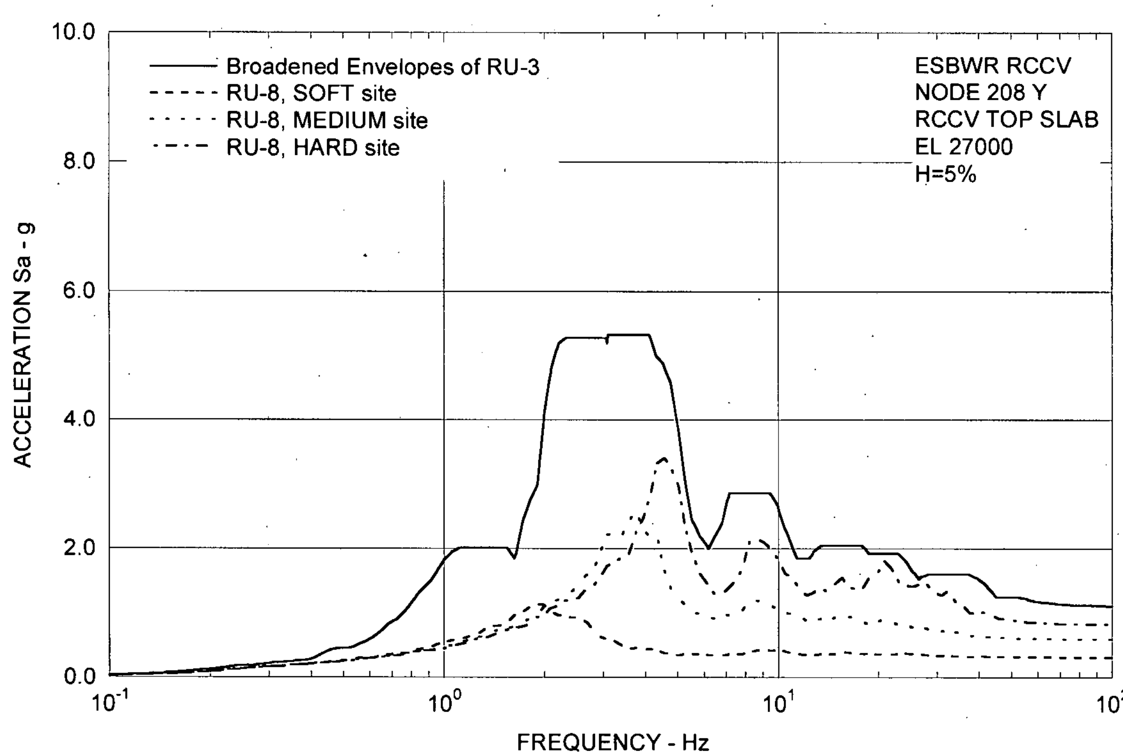


Figure 3.8-94(17) FRS (Compared with the DAC3N) – RCCV Top Slab Y

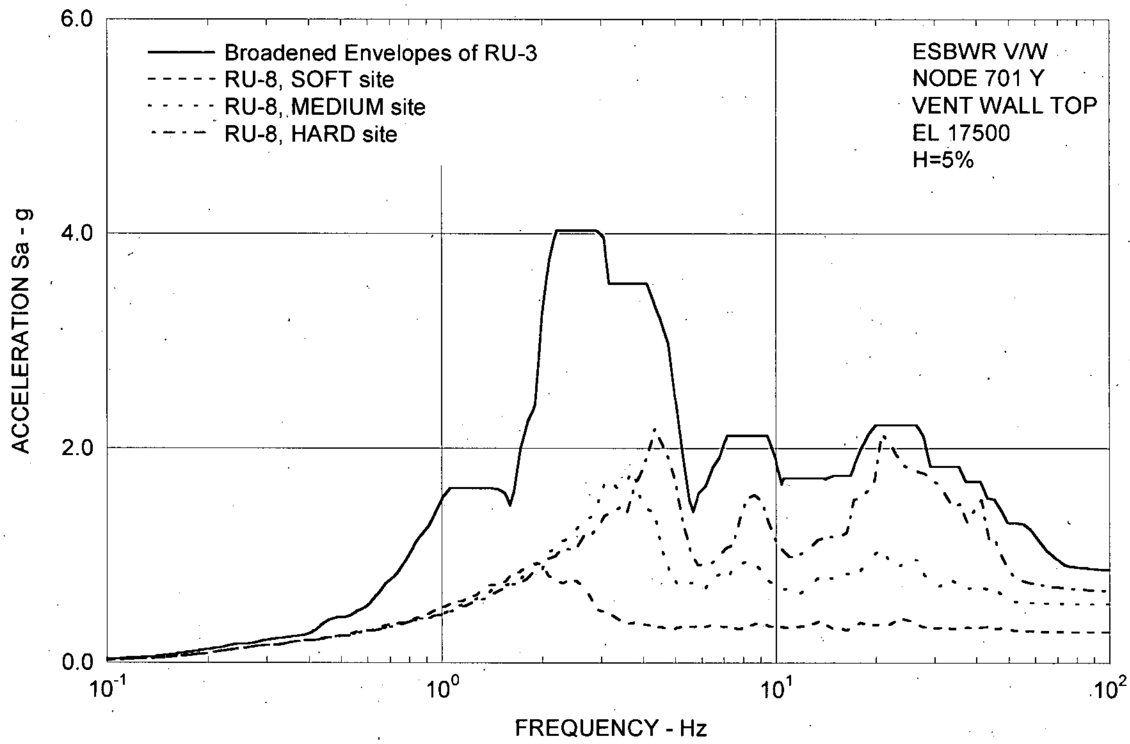


Figure 3.8-94(18) FRS (Compared with the DAC3N) – Vent Wall Top Y

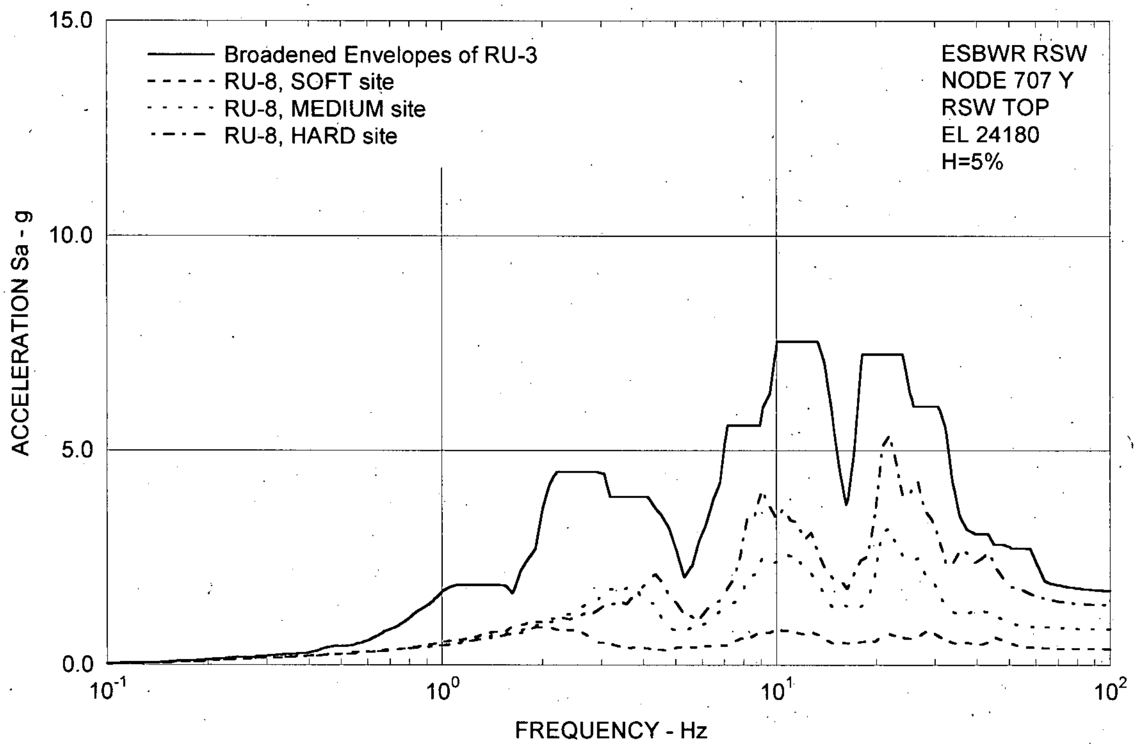


Figure 3.8-94(19) FRS (Compared with the DAC3N) – RSW Top Y

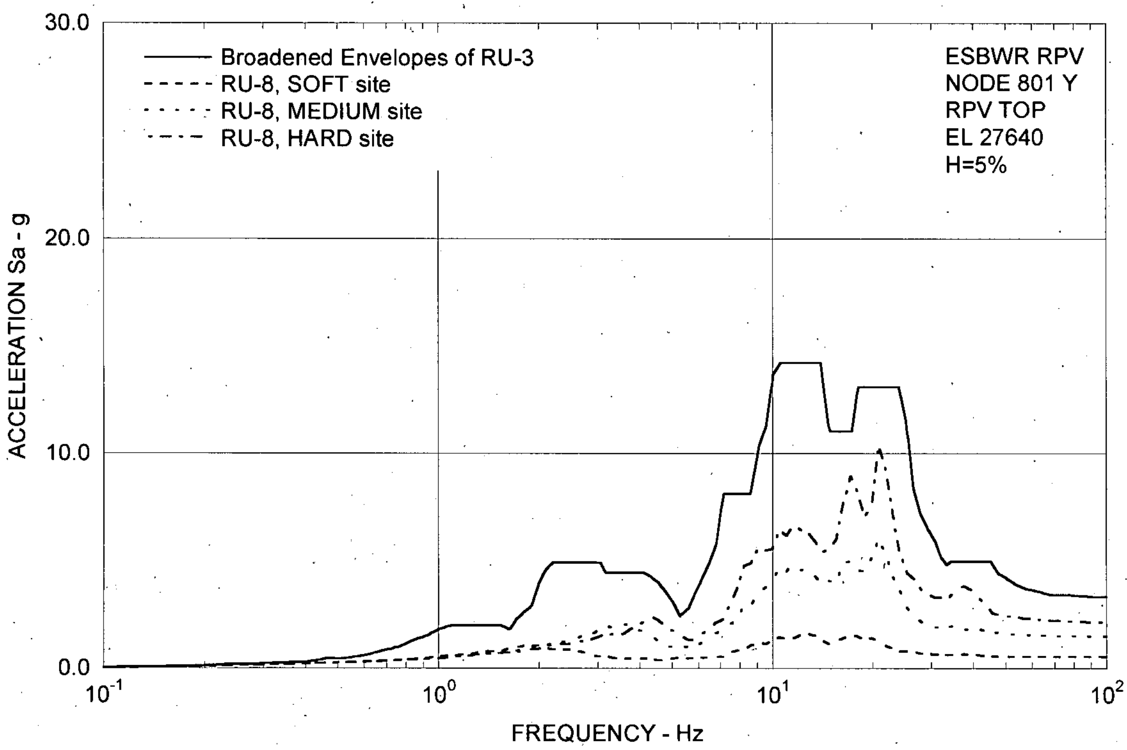


Figure 3.8-94(20) FRS (Compared with the DAC3N) – RPV Top Y

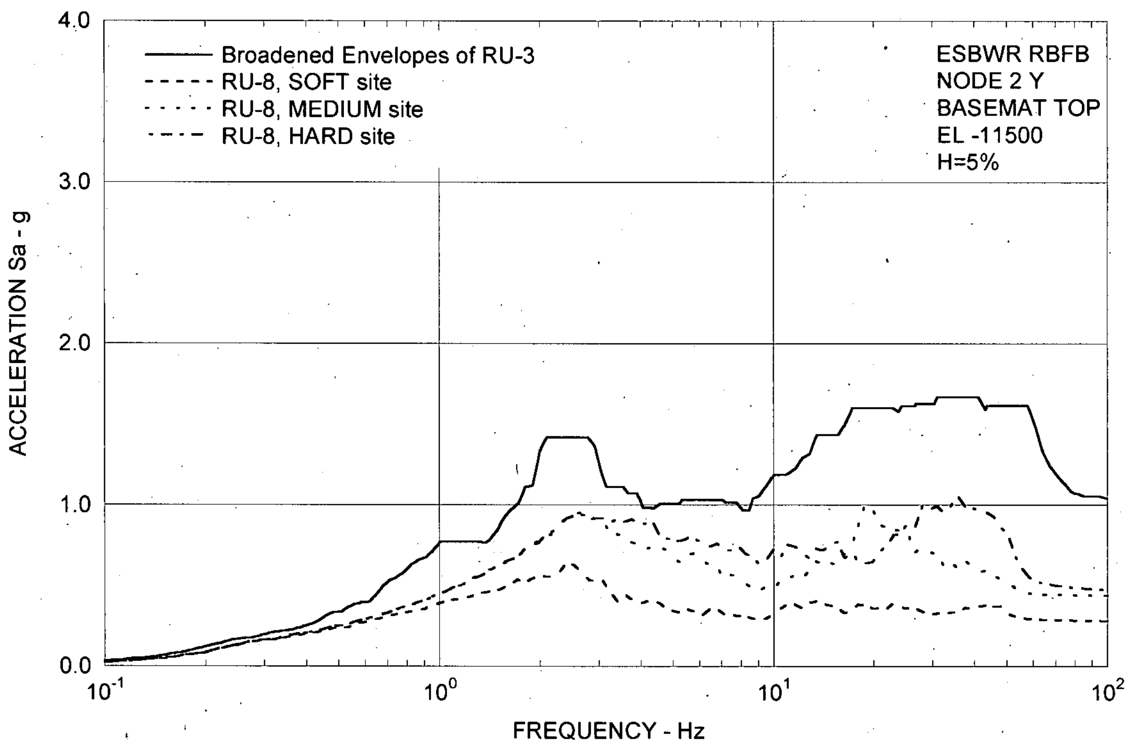


Figure 3.8-94(21) FRS (Compared with the DAC3N) – RBFB Basemat Y

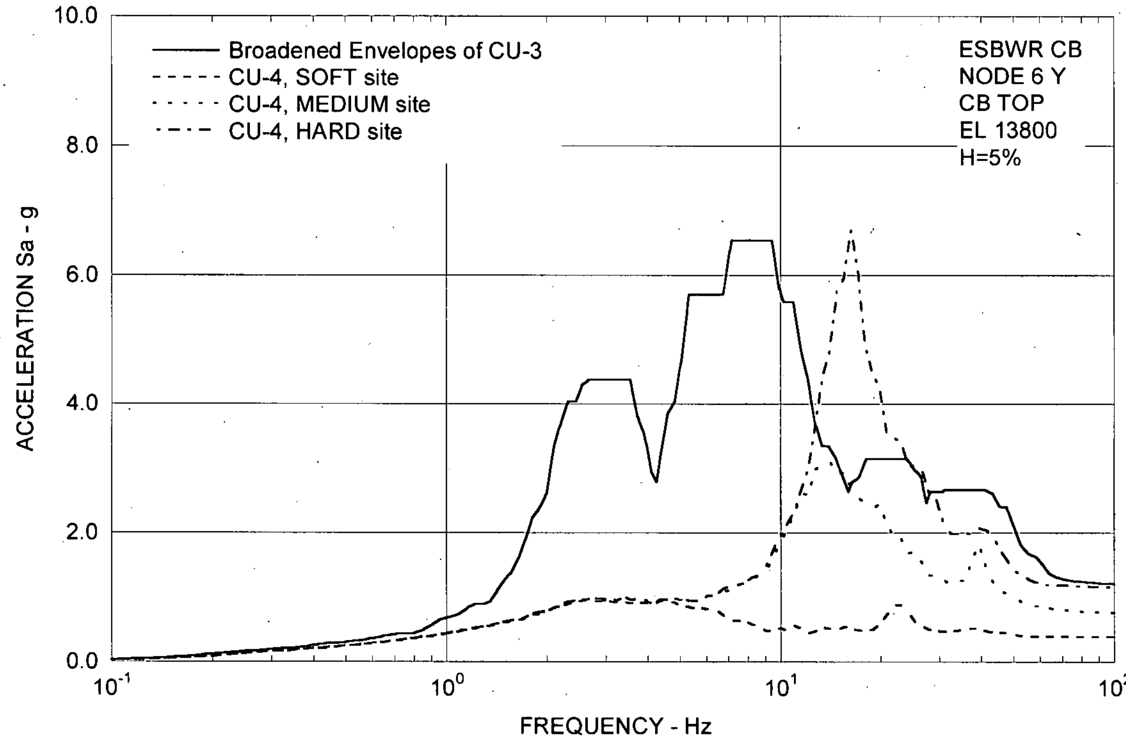


Figure 3.8-94(22) FRS (Compared with the DAC3N) – CB Top Y

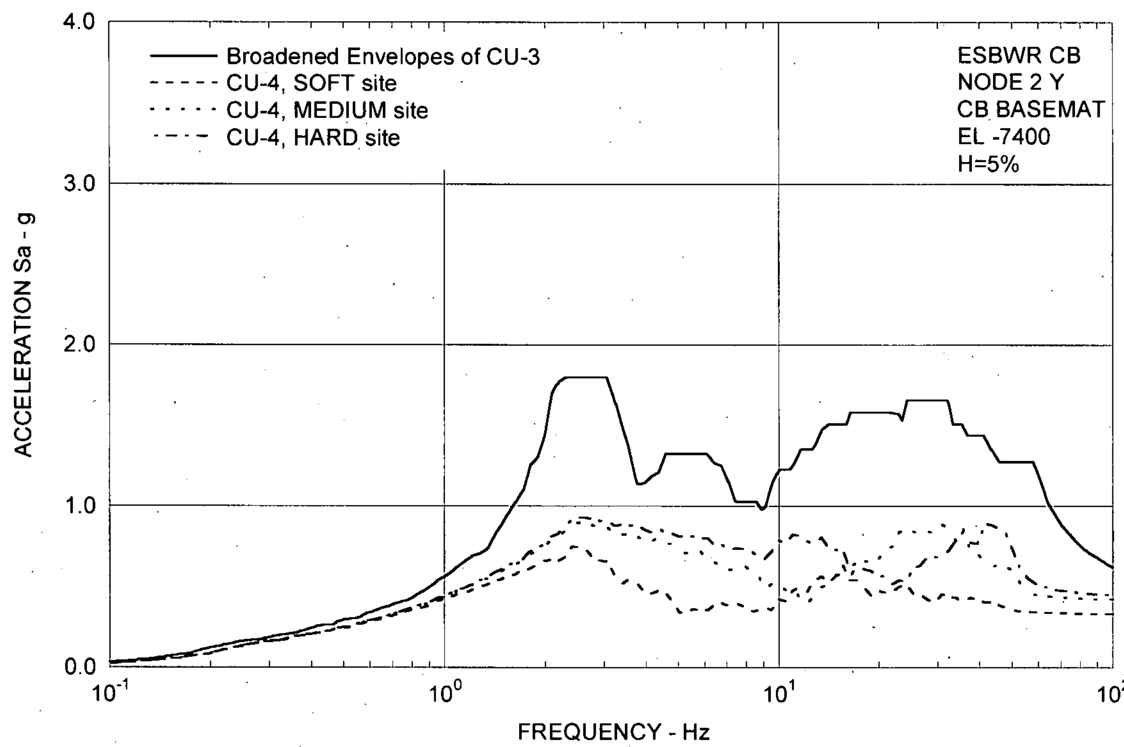


Figure 3.8-94(23) FRS (Compared with the DAC3N) – CB Basemat Y

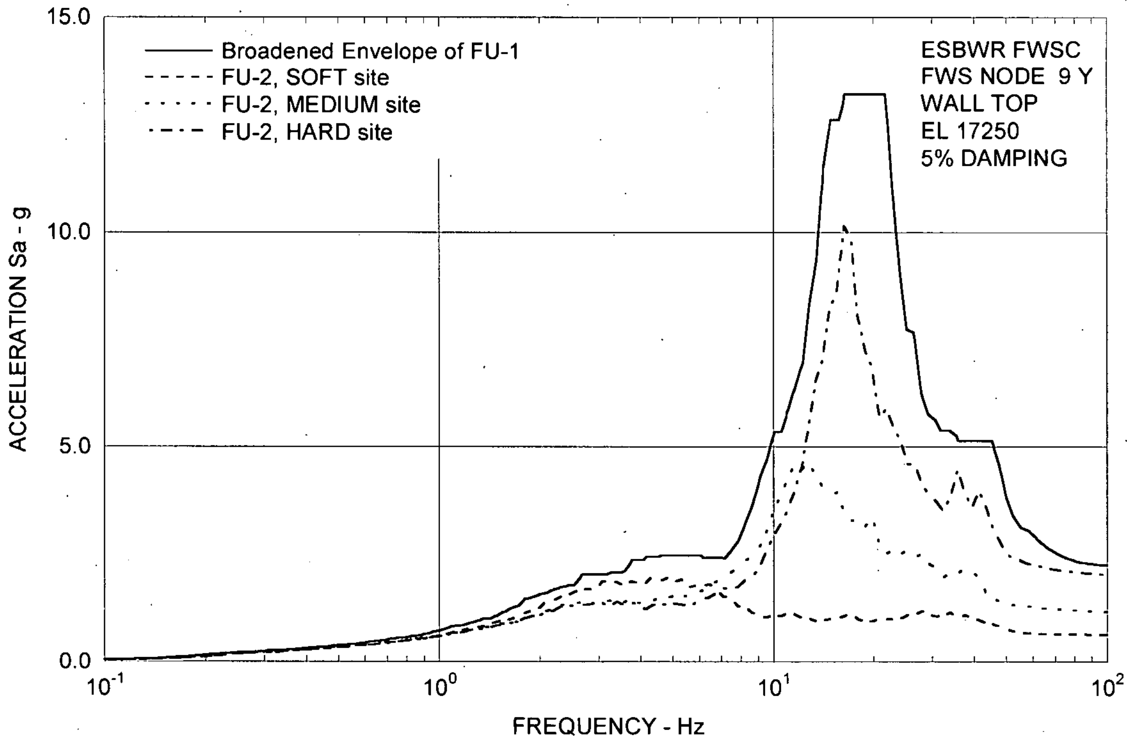


Figure 3.8-94(24) FRS (Compared with the DAC3N) – FWS Wall Top Y

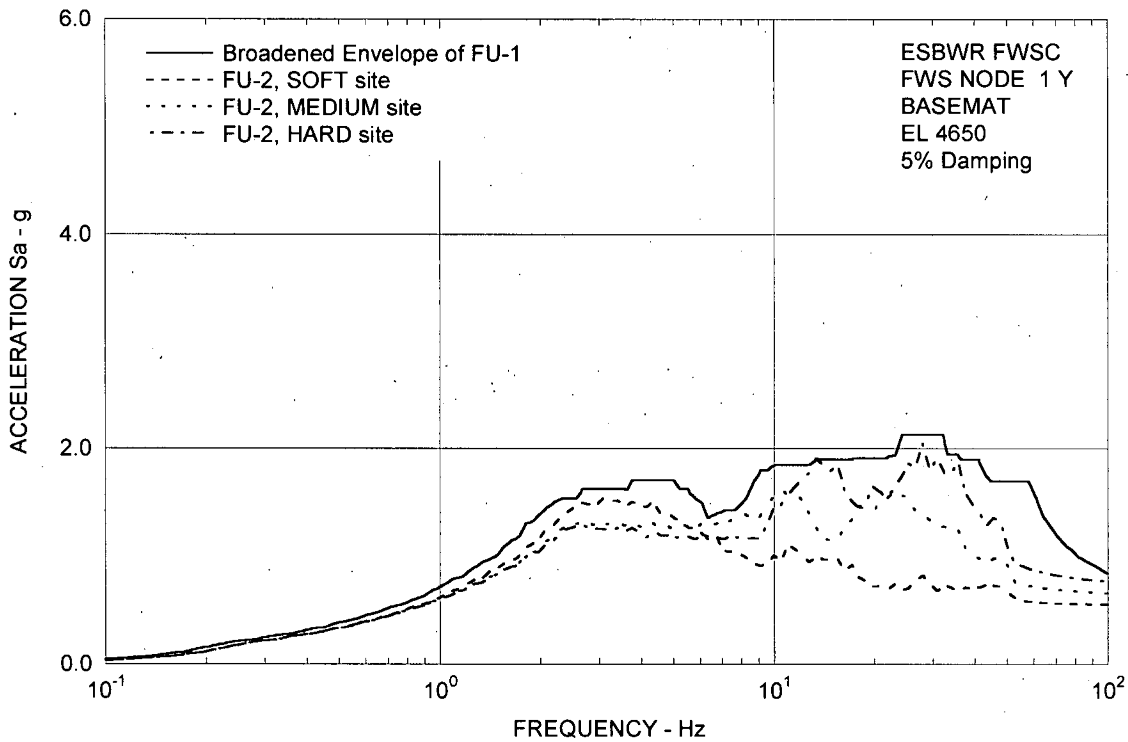


Figure 3.8-94(25) FRS (Compared with the DAC3N) – FWS Basemat Y

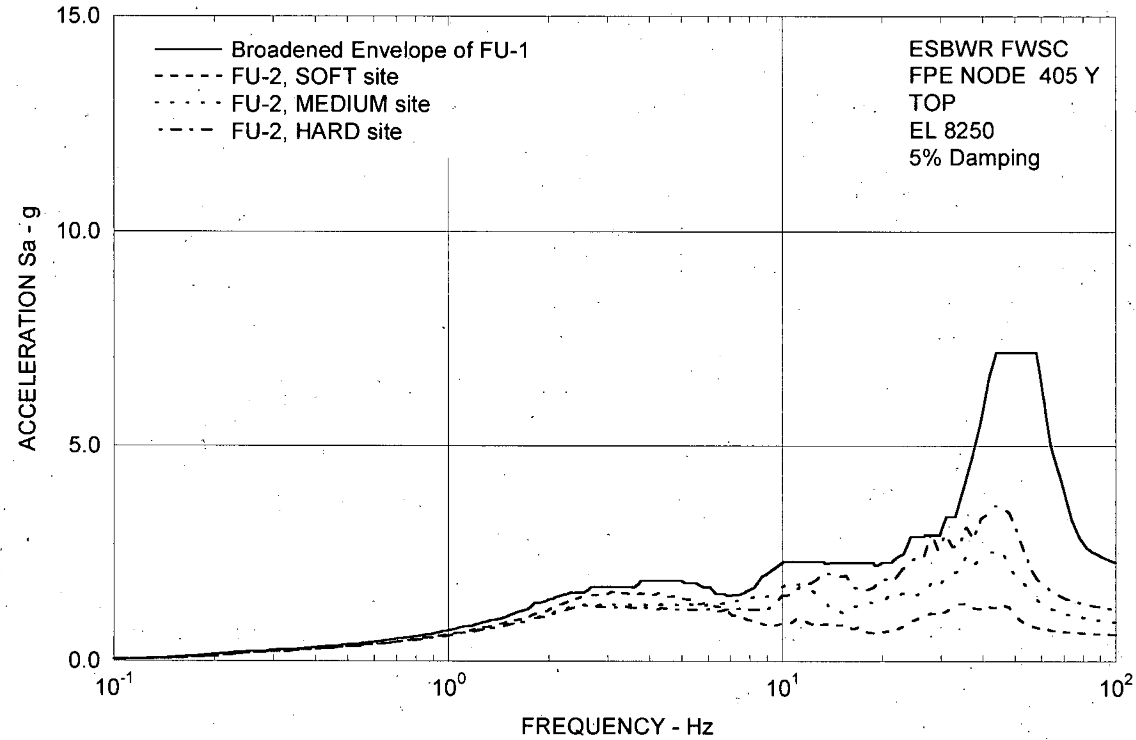


Figure 3.8-94(26) FRS (Compared with the DAC3N) – FPE Top Y

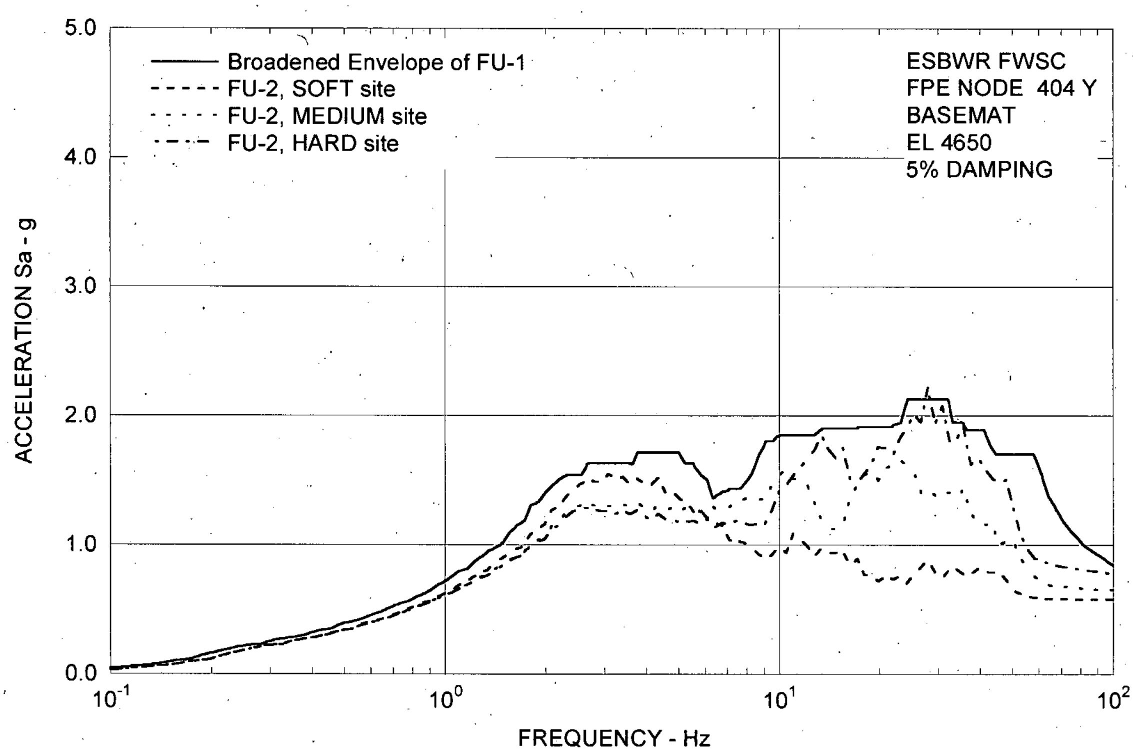


Figure 3.8-94(27) FRS (Compared with the DAC3N) – FPE Basemat Y

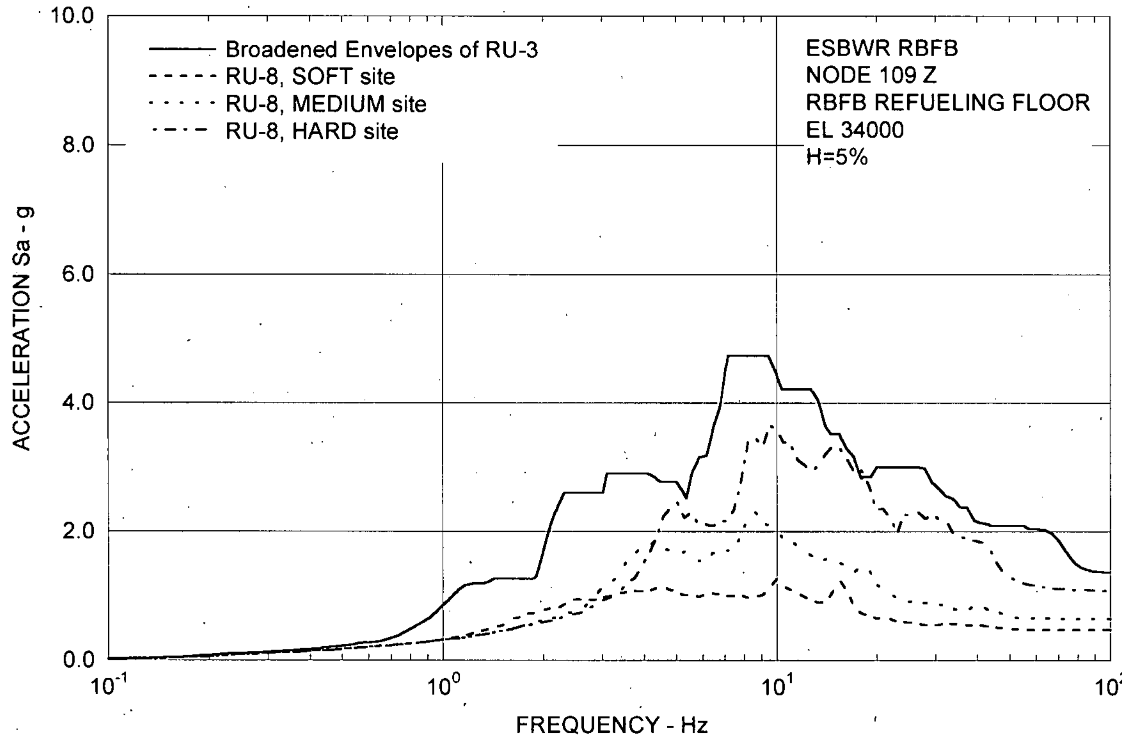


Figure 3.8-94(28) FRS (Compared with the DAC3N) – RBFB Refueling Floor Z

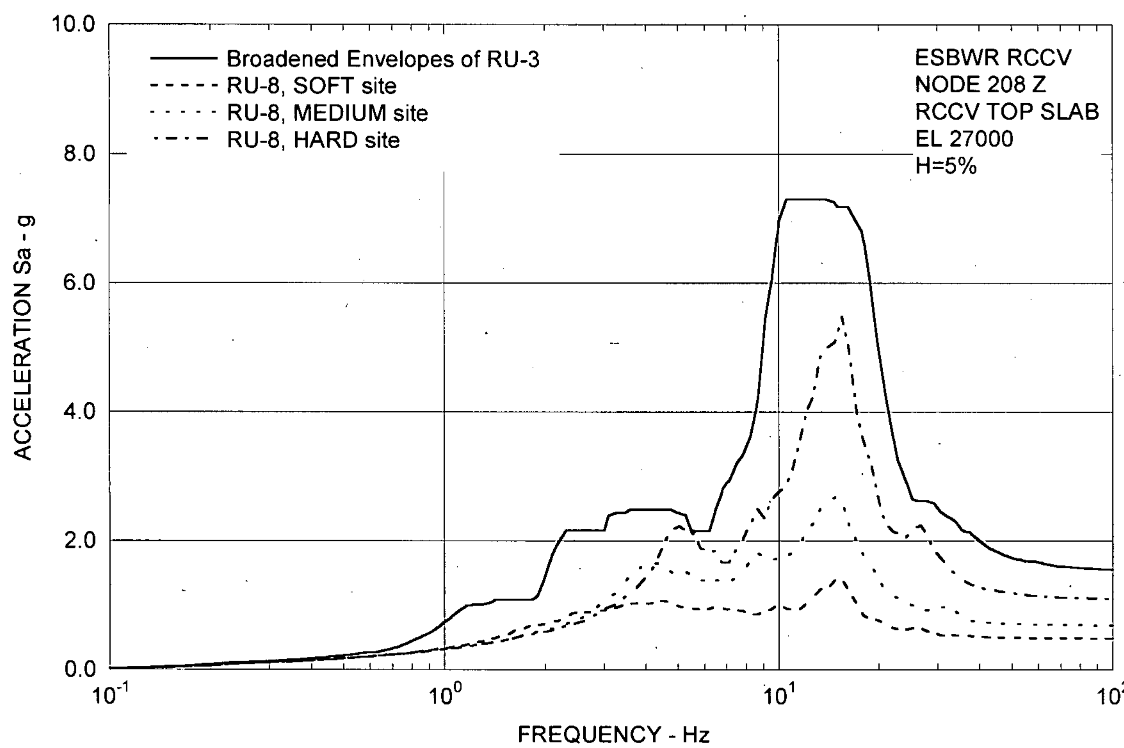


Figure 3.8-94(29) FRS (Compared with the DAC3N) – RCCV Top Slab Z

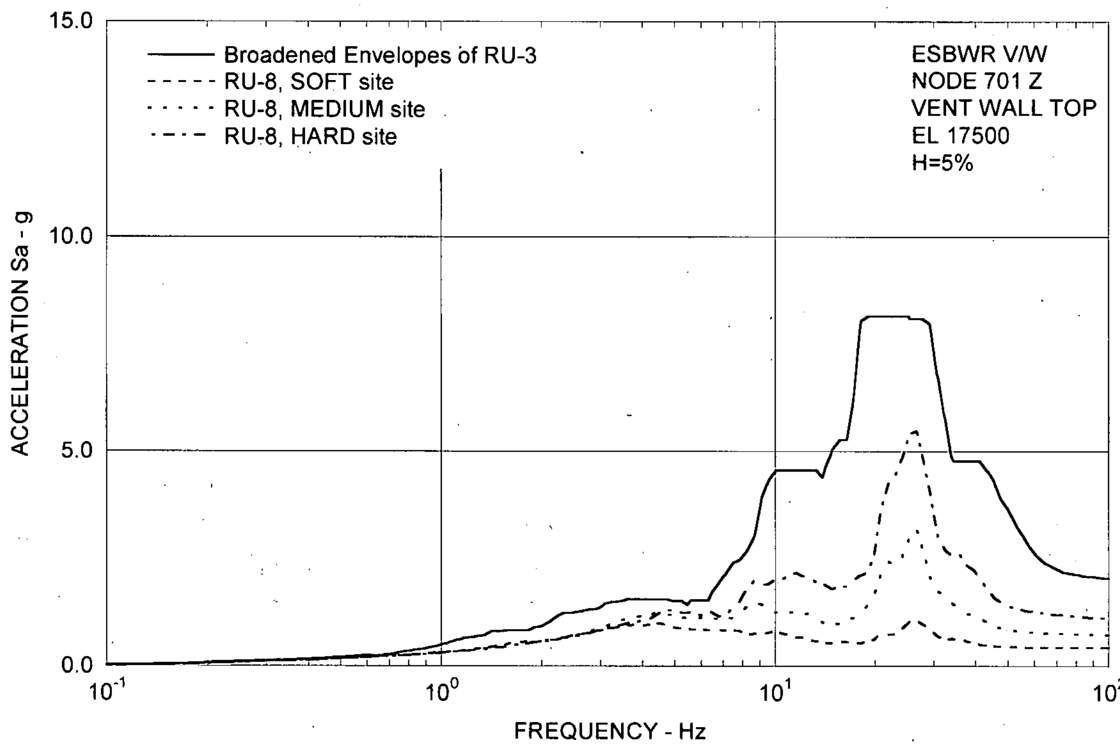


Figure 3.8-94(30) FRS (Compared with the DAC3N) – Vent Wall Top Z

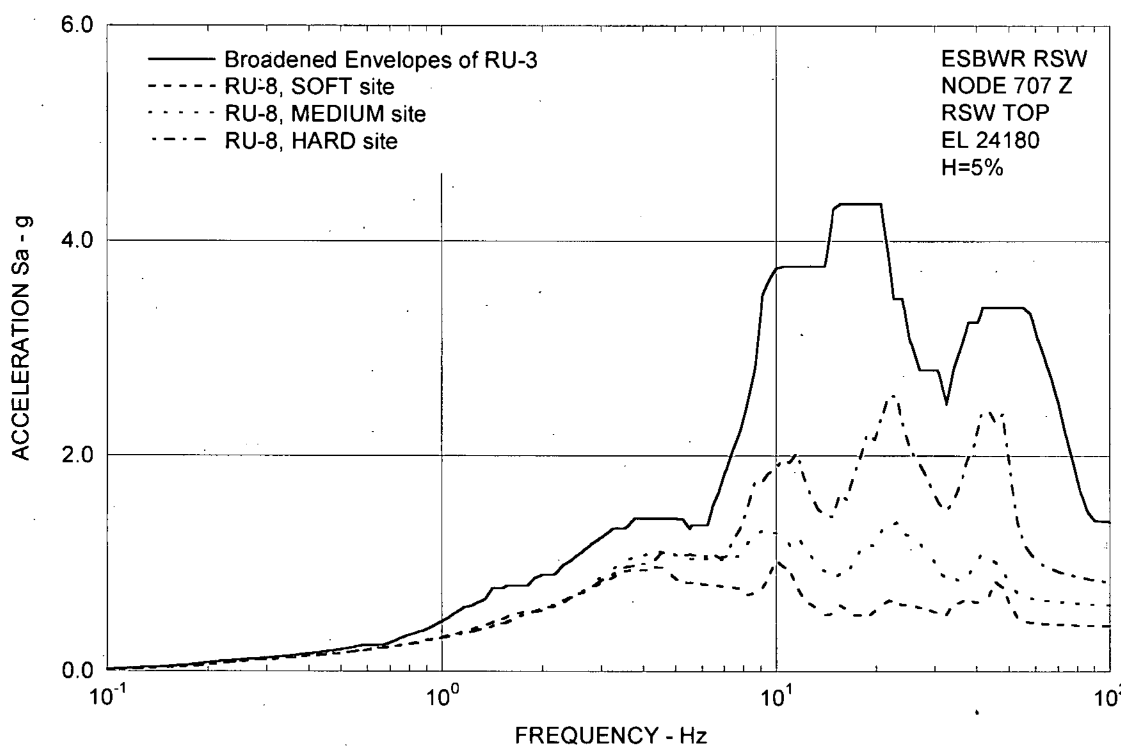


Figure 3.8-94(31) FRS (Compared with the DAC3N) – RSW Top Z

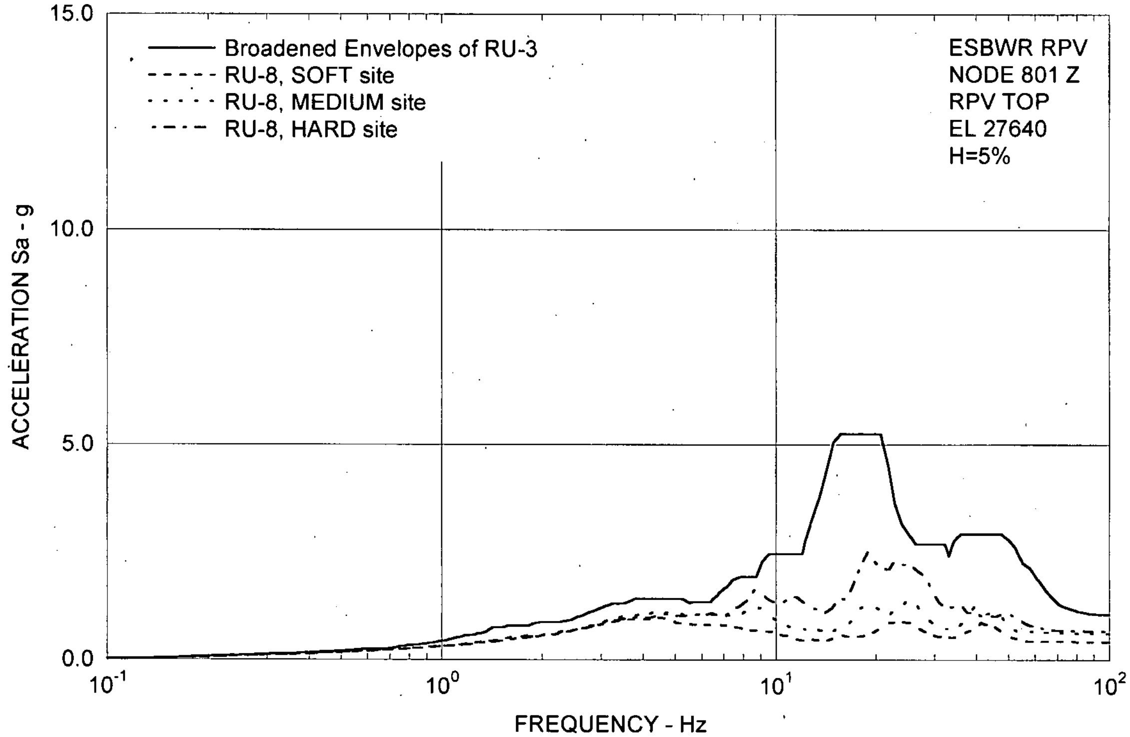


Figure 3.8-94(32) FRS (Compared with the DAC3N) – RPV Top Z

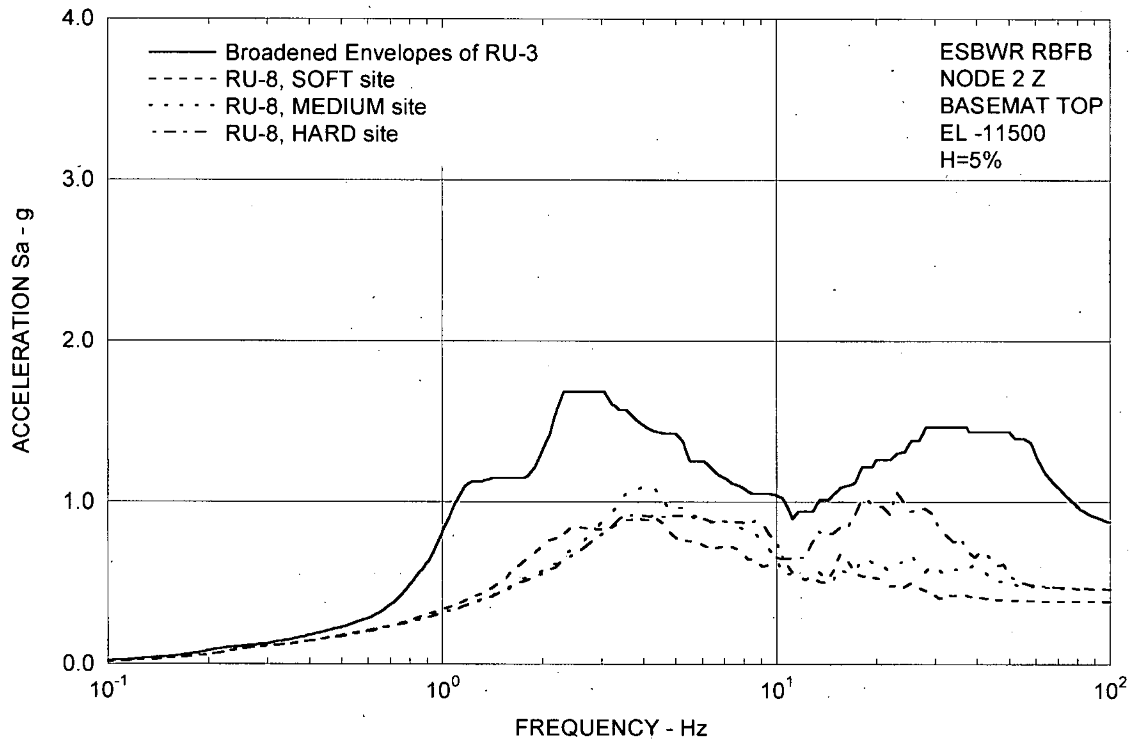


Figure 3.8-94(33) FRS (Compared with the DAC3N) – RBFB Basemat Z

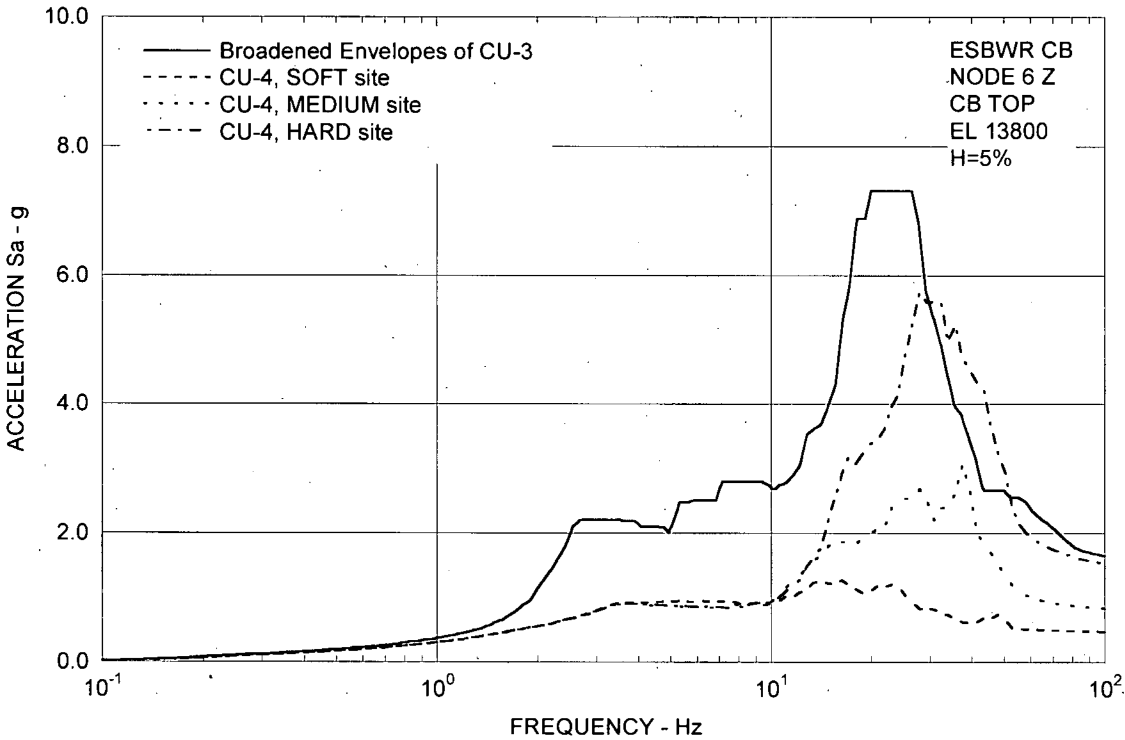


Figure 3.8-94(34) FRS (Compared with the DAC3N) – CB Top Z

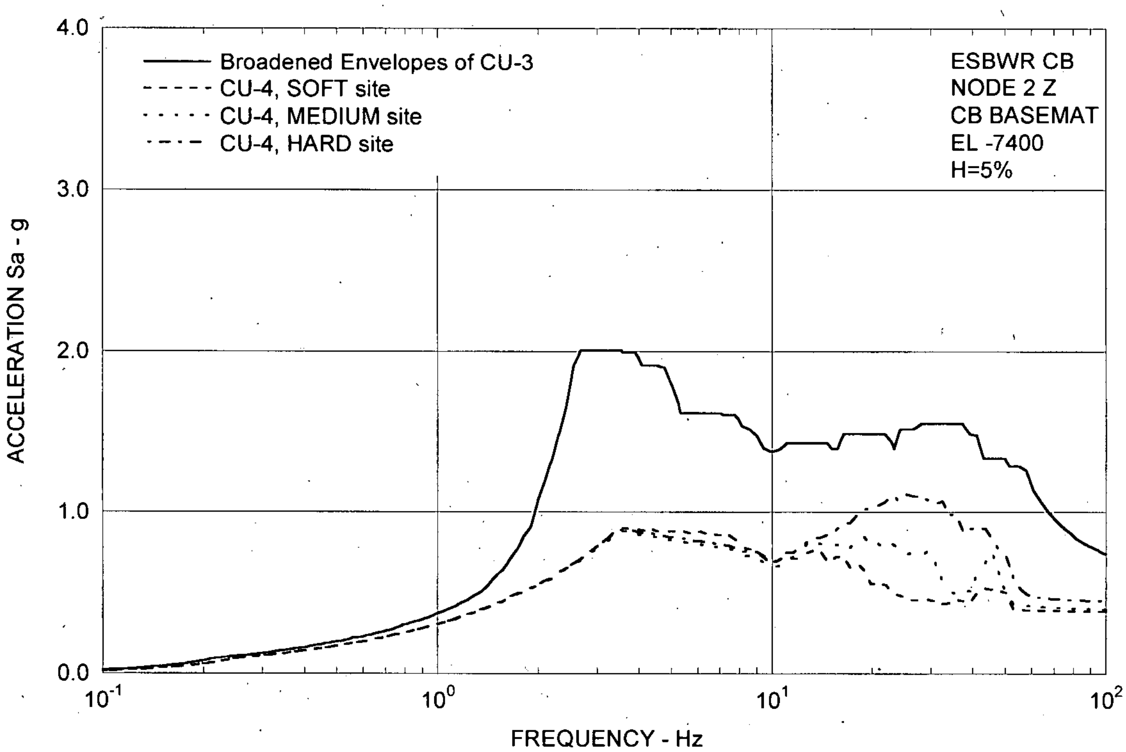


Figure 3.8-94(35) FRS (Compared with the DAC3N) – CB Basemat Z

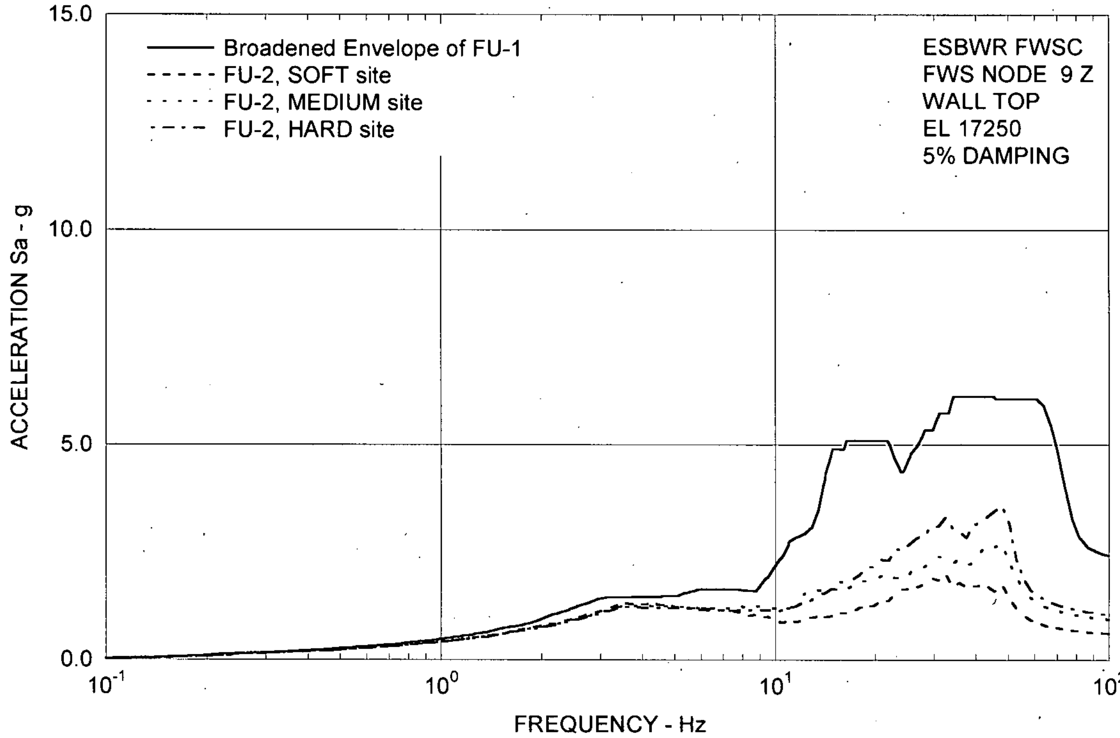


Figure 3.8-94(36) FRS (Compared with the DAC3N) – FWS Wall Top Z

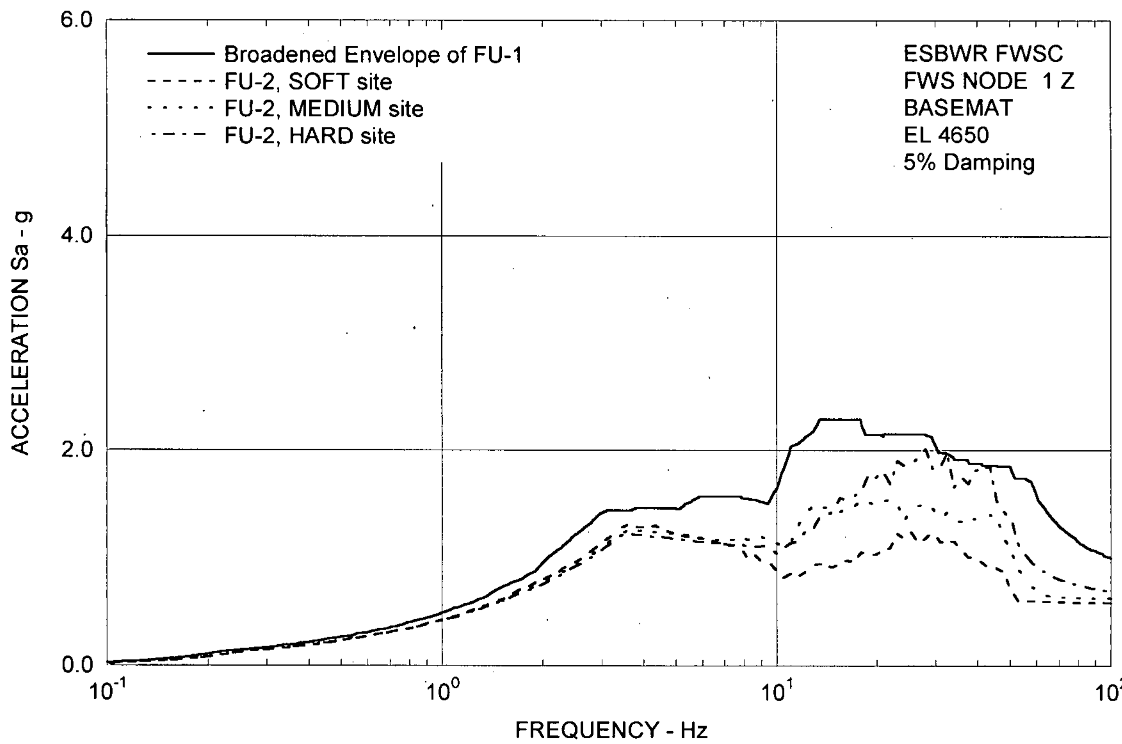


Figure 3.8-94(37) FRS (Compared with the DAC3N) – FWS Basemat Z

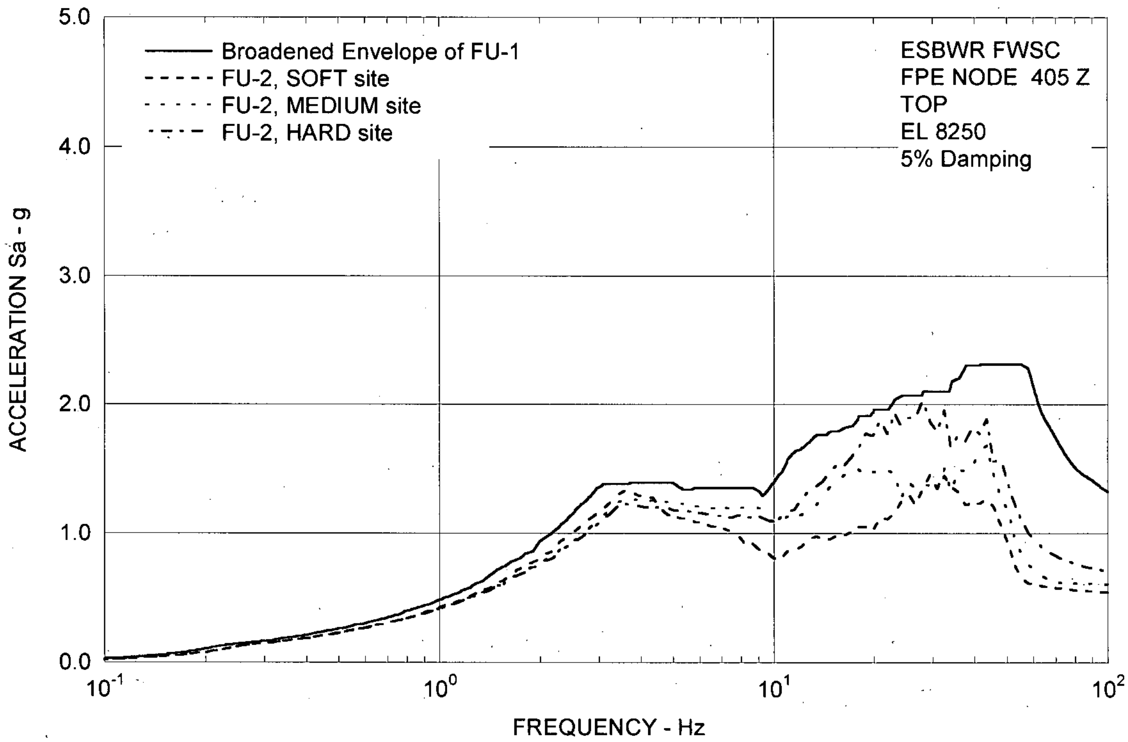


Figure 3.8-94(38) FRS (Compared with the DAC3N) – FPE Top Z

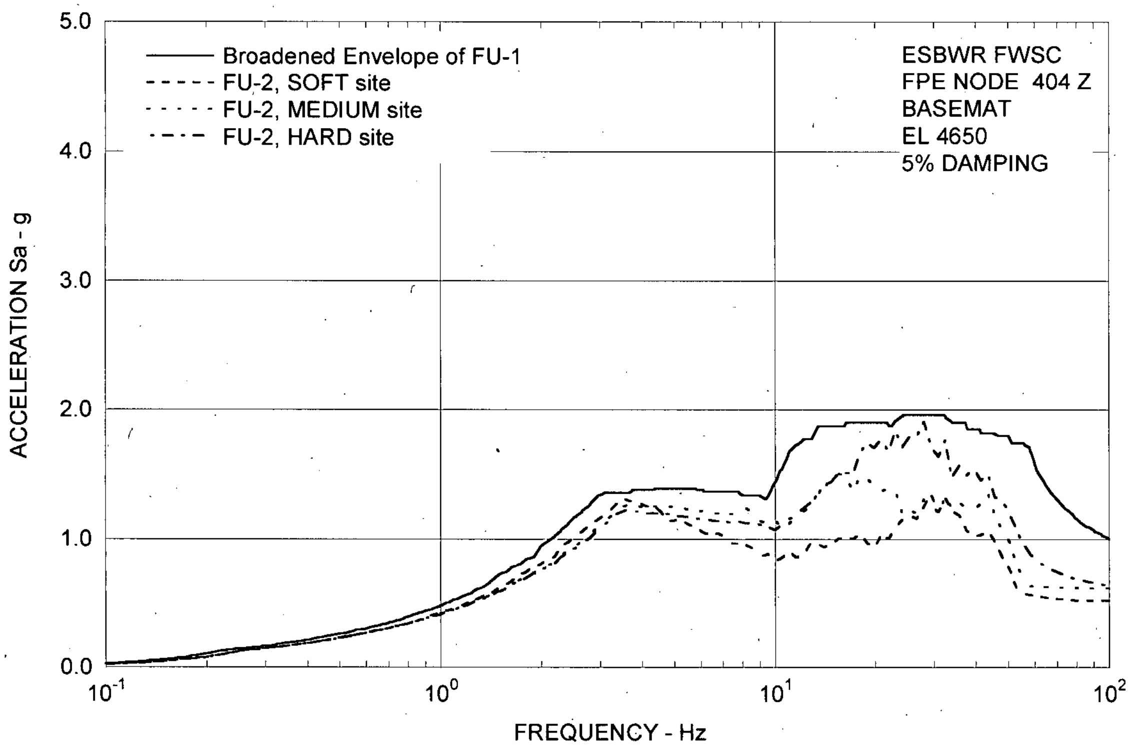


Figure 3.8-94(39) FRS (Compared with the DAC3N) – FPE Basemat Z

DCD Impact

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

DCD Tier 2 Subsections 3.7.1.1.3, 3A.5, 3A.5.2, 3A.6, 3A.8.7, 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.9-1g, 3A.9-1l, 3A.9-2g and 3A.9-3g will be revised in Revision 6 as noted in the attached markups.

DCD Tier 2 Subsection 3A.9.3 and Figures 3A.8.7-1a through 3A.8.7-1l, 3A.8.7-2a through 3A.8.7-2l and 3A.8.7-3a through 3A.8.7-3l will be added in Revision 6 as noted in the attached markups.

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

Soil Properties:	- Minimum Static Bearing Capacity:(2)	
	Reactor/Fuel Building:	699 kPa (14,600 lbf/ft²)
	Control Building:	292 kPa (6,100 lbf/ft²)
	Fire Water Service Complex:	165 kPa (3,450 lbf/ft²)
	- Minimum Dynamic Bearing Capacity (SSE + Static): (2)	
	Reactor/Fuel Building:	
	Soft:	~~27~~1200 kPa (25~~6~~,4100 lbf/ft²)
	Medium:	~~73~~1500 kPa (~~152,5~~31,400 lbf/ft²)
	Hard:	~~54~~1100 kPa (~~11~~23,0~~8~~00 lbf/ft²)
	Control Building:	
	Soft:	~~280~~440 kPa (~~58~~9,2~~5~~00 lbf/ft²)
	Medium:	22~~5~~00 kPa (45~~52~~,9~~3~~00 lbf/ft²)
	Hard:	~~24~~2~~0~~0 kPa (8~~50~~,8~~2~~00 lbf/ft²)
	Firewater Service Complex (FWSC):	
	Soft:	4460 kPa (9,6~~2~~00 lbf/ft²)
	Medium:	69~~5~~40 kPa (14~~1~~,~~3~~400 lbf/ft²)
	Hard:	120~~67~~0 kPa (25~~14~~,1~~0~~00 lbf/ft²)
	- 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	≥ 30 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
Maximum Settlement Values for Seismic Category I Buildings (5)		
Maximum Settlement at any corner of basemat	-Under Reactor/Fuel Buidling	103 mm (4.0 inches)
	-Under Control Building	18 mm (0.7 inches)
	-Under FWSC Structure	17 mm (0.7 inches)

Table 2.0-1

Envelope of ESBWR Standard Plant Site Parameters (continued)

Soil Properties: (16)	- Minimum Static Bearing Capacity: (7)	
	Reactor/Fuel Building:	699 kPa (14,600 lbf/ft²)
	Control Building:	292 kPa (6,100 lbf/ft²)
	Firewater Service Complex:	165 kPa (3,450 lbf/ft²)
	- Minimum Dynamic Bearing Capacity (SSE + Static): (7)	
	Reactor/Fuel Building:	
	Soft:	~~27~~1200 kPa (25~~6~~,4100 lbf/ft²)
	Medium:	~~73~~1500 kPa (~~152,5~~31,400 lbf/ft²)
	Hard:	~~54~~1100 kPa (~~11~~23,0~~8~~00 lbf/ft²)
	Control Building:	
	Soft:	~~280~~440 kPa (~~58~~9,2~~5~~00 lbf/ft²)
	Medium:	22~~5~~00 kPa (45~~52~~,9~~3~~00 lbf/ft²)
	Hard:	~~24~~2~~0~~0 kPa (8~~50~~,8~~2~~00 lbf/ft²)
	Firewater Service Complex (FWSC):	
	Soft:	4460 kPa (9,6~~2~~00 lbf/ft²)
	Medium:	69~~5~~40 kPa (14~~1~~,~~3~~400 lbf/ft²)
	Hard:	120~~67~~0 kPa (25~~14~~,1~~0~~00 lbf/ft²)
	- Minimum Shear Wave Velocity: (8)	300 m/s (1000 ft/s)
	- Maximum Ratio of Shear Wave Velocities in Adjacent Layers (17)	
	Bottom 20 m (66 ft) layer to top 20 m (66 ft) layer:	2.5
	Bottom 40 m (131 ft) layer to top 20 m (66 ft) layer:	2.5
	- 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	≥ 30 degrees
Seismology:	- SSE Horizontal Ground Response Spectra: (9)	See Figure 2.0-1
	- SSE Vertical Ground Response Spectra: (9)	See Figure 2.0-2

The spectrum figures are associated with 5% damping. The PGA values, corresponding to the spectral acceleration at 100 Hz of the target spectra, are 0.492 g at the CB base and 0.469 g at the RB/FB base in both horizontal and vertical directions. The time histories are generated under the spectral matching criteria given in NUREG/CR-6728 and the cross-correlations between the three individual components are all less than the 0.16 requirement. Since a more stringent matching criteria of NUREG/CR-6728 is used, a separate PSD check per SRP 3.7.1.II.1 is not required.

The high-frequency input ground motion thus defined is considered in the basic design seismic analysis for North Anna ESP site condition using the DAC3N computer code.

3.7.1.1.3 Single Envelope Ground Motion

The single envelope ground response spectra are constructed to envelope the low-frequency 0.3 g RG 1.60 spectra (Subsection 3.7.1.1.1) and the high-frequency North Anna site-specific spectra (Subsection 3.7.1.1.2). The smoothed target spectra of 5% damping are shown in Table 3.7-2 and in Figures 2.0-1 and 2.0-2. The spectral values up to and including 9 Hz and 10 Hz in the horizontal and vertical directions, respectively, are based on 0.3 g RG 1.60 spectra. At higher frequencies the spectral values closely match that of the envelope of North Anna ESP spectra at ESBWR RB/FB and CB foundations as a representative ground motion for Eastern US sites founded on rock. Note that there has never been recorded a seismic event containing simultaneously very high low-frequency excitations and very high high-frequency motions. Therefore, this envelope is very conservative in terms of energy content and is used to verify the basic design previously discussed.

A single set of three orthogonal, statistically independent time histories is generated to match the target spectra in accordance with NUREG/CR-6728 criteria. The computed response spectra are compared with the corresponding target spectra in Figures 3.7-38 through 3.7-40 for H1, H2 and vertical components, respectively. Spectral matching tests for 5% damping only is consistent with the recommendations of NUREG/CR-6728 of specifying ground-motions in terms of 5% spectra. Use of 5% only is considered sufficient because there is a strong correlation among the response-spectral ordinates at damping ratios from 1 to 20%. Thus, if a time history matches the 5% target, it is likely to match the targets at other damping ratios. Because a more stringent matching criteria of NUREG/CR-6728 is used, a separate PSD check per SRP 3.7.1.II.1 is not required. Tests performed in NUREG/CR-6728 indicate that the response-spectrum tests are sufficient.

The acceleration time histories are shown in Figures 3.7-41 through 3.7-43, together with corresponding velocity and displacement time histories. Each time history has a total duration of 40 seconds with time steps of 0.005 seconds. The strong motion duration is 7.8 seconds for H1, 12 seconds for H2 and 8.9 seconds for vertical. The cross-correlations between the three individual components are all less than the 0.16 requirement.

The single envelope ground motion is considered in the design basis seismic analysis for all generic uniform sites using DAC3N and SASSI2000 computer codes and layered sites using ~~DAC3N and~~ SASSI2000 computer codes~~, respectively~~.

Table 3.7-3

Summary of Methods of Seismic Analysis for Primary Building Structures

Building Structure	Site Condition	SSI Model	Analysis Method	Three Components Combination	Modal Combination	Computer Program	Use of Analysis Output
Reactor Building including containment and containment internal structures	Uniform Sites	3D lumped mass stick coupled with soil springs	Direct integration in the time domain	Algebraic Sum	n/a	DAC3N	~~m~~Max. forces, moments, acceleration, floor response spectra and max. relative displacements. Interface loads with foundation medium not used.
Reactor Building including containment and containment internal structures	Uniform and Layered Sites	3D lumped mass stick coupled with soil finite elements	Frequency response in the frequency domain.	Algebraic Sum	n/a	SASSI2000	~~m~~Max. forces, moments, acceleration, floor response spectra, max. relative displacements and ~~soil pressure~~interface loads with foundation medium.
Fuel Building	Uniform Sites	Integrated with the Reactor Building models	Direct integration in the time domain	Algebraic Sum	n/a	DAC3N	~~m~~Max. forces, moments, acceleration and floor response spectra. Interface loads with foundation medium not used.
Fuel Building	Uniform and Layered Sites	Integrated with the Reactor Building models	Frequency response in the frequency domain.	Algebraic Sum	n/a	SASSI2000	~~m~~Max. forces, moments, acceleration, floor response spectra and interface loads with foundation medium.~~soil pressure~~
Control Building	Uniform Sites	3D lumped mass stick coupled with soil springs	Direct integration in the time domain	Algebraic Sum	n/a	DAC3N	~~m~~Max. forces, moments, acceleration and floor response spectra. Interface loads with foundation medium not used.

Table 3.7-3

Summary of Methods of Seismic Analysis for Primary Building Structures

Building Structure	Site Condition	SSI Model	Analysis Method	Three Components Combination	Modal Combination	Computer Program	Use of Analysis Output
Control Building	Uniform and Layered Sites	3D lumped mass stick coupled with soil finite elements	Frequency response in the frequency domain.	Algebraic Sum	n/a	SASSI2000	~~m~~Max. forces, moments, acceleration, floor response spectra and interface loads with foundation medium.~~soil pressure~~
Firewater Service Complex	Uniform Sites	3D lumped mass stick coupled with soil springs	Direct integration in the time domain	Algebraic Sum	n/a	DAC3N	~~m~~Max. forces, moments, acceleration and floor response spectra. Interface loads with foundation medium not used.
Firewater Service Complex	Uniform and Layered Sites	3D lumped mass stick coupled with soil finite elements	Frequency response in the frequency domain	Algebraic Sum	n/a	SASSI2000	~~m~~Max. forces, moments, acceleration, ~~and~~ floor response spectra and interface loads with foundation medium.

3A.5 SOIL-STRUCTURE INTERACTION ANALYSIS METHOD

The seismic analysis for uniform sites is performed using the program DAC3N with the sway-rocking SSI model without embedment. The seismic analysis for layered sites is performed using the program SASSI2000 with the finite elements for modeling the SSI with embedment. SASSI2000 analysis is also performed for uniform sites with embedment for the purpose of obtaining more realistic interface loads with the foundation medium for use in the foundation stability evaluation.

3A.5.1 DAC3N Analysis Method

The analysis model is a lumped mass-beam model with soil springs. The structural models are described in Subsection 3.7.2, and in Section 3A.7 in more detail.

To account for SSI effect, sway-rocking base soil springs are attached to the structural model. The base spring is evaluated from vibration admittance theory, based on three-dimensional wave propagation theory for uniform half space soil. For this evaluation, soil material damping values are conservatively neglected. Though the spring values consist of frequency dependent real and imaginary parts, they are simplified and replaced with frequency independent soil spring K_c, and damping coefficient C_c, respectively, for the time history analysis solved in time domain. The method used to obtain the equivalent frequency-independent soil stiffness and damping is illustrated in Figure 3A.5-1. The calculated K_c and C_c values are tabulated in Tables 3A.5-1 through 3A.5-3 for the RB/FB complex, CB and FWSC, respectively.

The effect of lateral soil/backfill on embedded foundations is conservatively accounted for by applying the control motion directly at the foundation level. Dynamic lateral soil pressures are calculated separately and considered in the design of external walls, using the elastic solution procedures in Subsection 3.5.3.2 of ASCE 4-98.

Because the three component ground motion time histories are statistically independent as described in Subsections 3.7.1.1.2 and 3.7.1.1.3, they are input simultaneously in the response analysis using the time history method of analysis solved by direct integration. The numerical integration time step is 0.002 sec. for the RG 1.60 input motion and 0.001 sec. for the North Anna site input motion and the single envelope input motion. Structural responses in terms of accelerations, forces, and moments are computed directly. Floor response spectra (FRS) are obtained from the calculated response acceleration time histories (Subsection 3.7.2.5).

3A.5.2 SASSI2000 Analysis Method

For the seismic analysis of layered and uniform sites, the linear finite element computer program SASSI2000 is used. The program uses finite elements with complex moduli for modeling the structure and foundation properties and is based on the subtraction method and the frequency domain complex response method. The lumped mass-beam model described in Subsection 3A.5.1 is coupled with finite element soil model. The model details are described in Section 3A.7. Structural responses in terms of accelerations, forces and moments are computed directly. FRS are obtained from the calculated response acceleration time histories.

The SSI analyses for the three directional earthquake components are performed separately. The maximum co-directional responses to each of the three earthquake components are combined using algebraic sum in the time domain.

3A.6 SOIL-STRUCTURE INTERACTION ANALYSIS CASES

To establish design envelopes of seismic responses of the RB/FB complex, SSI analyses are performed for ~~31~~34 cases of uniform sites and 6 cases of layered sites, as summarized in Table 3A.6-1. Similarly for the CB, SSI analyses are performed for ~~11~~ 14 cases of uniform sites and 6 cases of layered sites. SSI analyses for the FWSC are performed for ~~4~~ 7 cases of uniform sites and 4 cases of layered sites.

The enveloping results are obtained from the responses of all SSI cases to cover a wide range of conditions.

Table 3A.6-1

Seismic SSI Analysis Cases

Building	No.	Model	Input Motion	Uniform Soil Condition						
				Generic Site				North Anna ESP Site		
				Soft	Medium	Hard	Fixed	NA-BE	NA-UB	NA-LB
RB/FB	RU-1	Base	RG 1.60 (0.3g)	*	*	*	*			
	RU-2	Base	NA site spectra					*	*	*
	RU-3	Base	Single envelope spectra	*	*	*	*			
	RU-4	Updated[1]	Single envelope spectra	*	*	*	*			
	RU-5	50% Infill Concrete Stiffness[2]	Single envelope spectra	*	*	*	*			
	RU-5a	100% Infill Concrete Stifness[3]	Single envelope spectra	*	*	*	*			
	RU-6	LOCA flooding[4]	Single envelope spectra	*	*	*	*			
	RU-7	Wall Out-of-plane[5]	Single envelope spectra	*	*	*	*			
	RU-8	Base[6]	Single envelope spectra	*	*	*				
CB	CU-1	Base	RG 1.60 (0.3g)	*	*	*	*			
	CU-2	Base	NA site spectra					*	*	*
	CU-3	Base	Single envelope spectra	*	*	*	*			
	CU-4	Base[6]	Single envelope spectra	*	*	*				
FWSC	FU-1	Base	Single envelope spectra	*	*	*	*			
	FU-2	Base[6]	Single envelope spectra	*	*	*				

Building	No.	Model	Input Motion	Uniform Soil Condition						
				Generic Site				North Anna ESP Site		
				Soft	Medium	Hard	Fixed	NA-BE	NA-UB	NA-LB
RB/FB	RU-1	Base	RG 1.60 (0.3g)	*	*	*	*			
	RU-2	Base	NA site spectra					*	*	*
	RU-3	Base	Single envelope spectra	*	*	*	*			
	RU-4	Updated[1]	Single envelope spectra	*	*	*	*			
	RU-5	50% Infill Concrete Stiffness[2]	Single envelope spectra	*	*	*	*			
	RU-5a	100% Infill Concrete Stifness[3]	Single envelope spectra	*	*	*	*			
	RU-6	LOCA flooding[4]	Single envelope spectra	*	*	*	*			
	RU-7	Wall Out-of-plane[5]	Single envelope spectra	*	*	*	*			
CB	CU-1	Base	RG 1.60 (0.3g)	*	*	*	*			
	CU-2	Base	NA site spectra					*	*	*
	CU-3	Base	Single envelope spectra	*	*	*	*			
FWSC	FU-1	Base	Single envelope spectra	*	*	*	*			

NA North Anna
BE best-estimate
UB upper bound
LB lower bound

Notes:

(1) Updated model for RSW, VW and D/F properties with 0% infill concrete stiffness
(2) Updated model + VW D/F 50% infill concrete stiffness
(3) Updated model + VW D/F 100% infill concrete stiffness
(4) Updated model + LOCA flooding condition
(5) Wall out-of-plane oscillator model
(6) SASSI2000 analysis with embedment for RB/FB and CB and without embedment for surface founded FWSC.

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. Note the basemat interface loads for layered site cases L-2 and L-4 are excluded in the foundation stability evaluation for RB/FB and CB. This limitation is a COL item in Table 2.0-1 for maximum ratio of shear wave velocities in adjacent layers.

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) 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 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

Table 3A.8.7-1

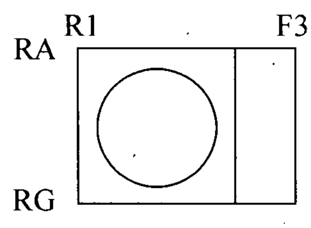
Comparisons of RB/FB Basemat Reaction Shear Force

Analysis Method	Effect of Embedment	Soft site (Vs 300m/s) (MN)			Medium site (Vs 800m/s) (MN)			Hard site (Vs 1700m/s) (MN)		
		Case	NS	EW	Case	NS	EW	Case	NS	EW
DAC3N	Neglect	RU-3	900	760	RU-3	1450	1500	RU-3	1470	1160
SASSI2000	Consider	RL-3	560	490	RL-4	1380	1310	-	-	-
		RU-8	490	480	RU-8	650	590	RU-8	590	590

Table 3A.8.8-1

Lateral Soil Pressure – RB/FB

Floor Level (m)	R1 and F3 Wall Soil Pressure (MPa)					RA and RG Wall Soil Pressure (MPa)					ASCE 4-98 (MPa)	Envelope (MPa)	
	RL-1	RL-2	RL-3	RL-4	RL-5	RL-1	RL-2	RL-3	RL-4	RL-5		R1 and F3 Wall	RA and RG Wall
4.65													
Slab													
3.65 -1.00	0.20	0.19	0.24	0.19	0.21	0.27	0.17	0.33	0.19	0.22	0.30	0.30	0.33
Slab													
-2.00 -6.40	0.15	0.21	0.20	0.21	0.26	0.17	0.19	0.21	0.19	0.20	0.29	0.29	0.29
Slab													
-7.40 -11.50	0.19	0.21	0.20	0.21	0.25	0.18	0.19	0.18	0.20	0.20	0.23	0.25	0.23
Basemat	0.29	0.24	0.28	0.25	0.23	0.25	0.24	0.25	0.26	0.20	0.16	0.29	0.26



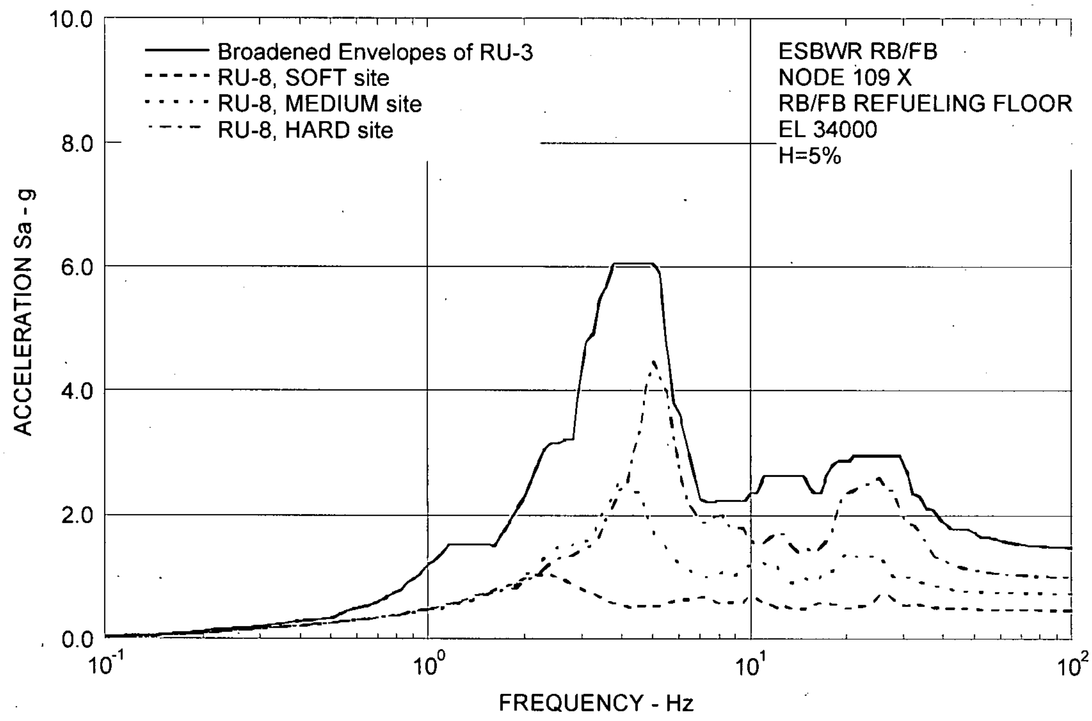


Figure 3A.8.7-1a. FRS (Effect of Embedment) – RB/FB Refueling Floor X

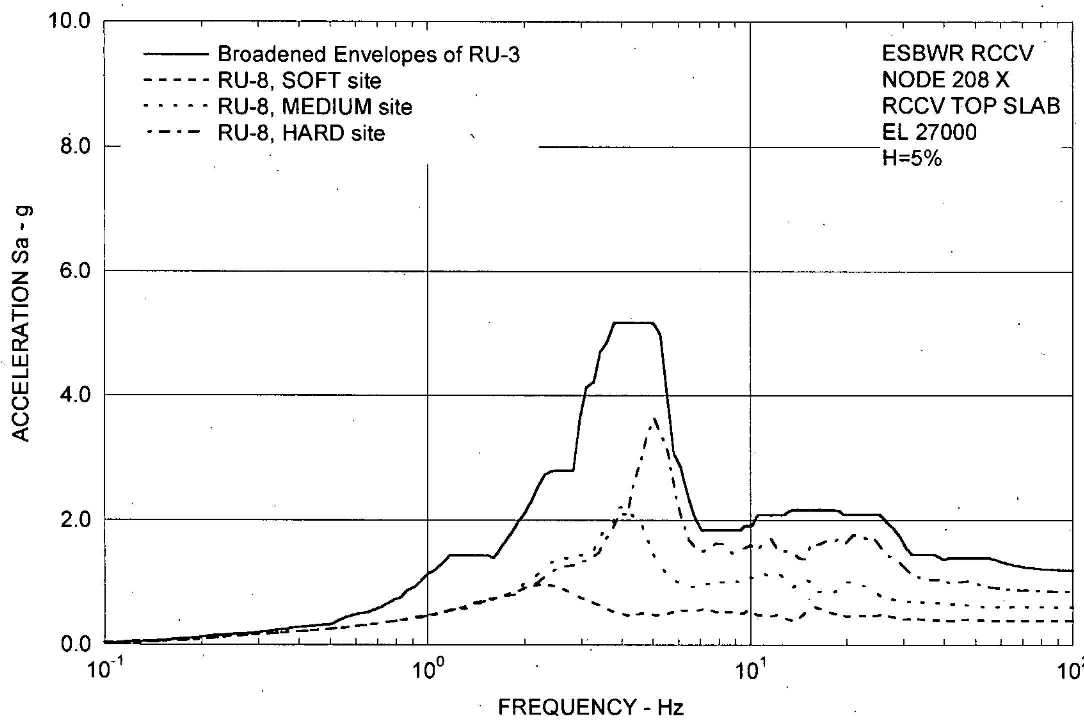


Figure 3A.8.7-1b. FRS (Effect of Embedment) – RCCV Top Slab X

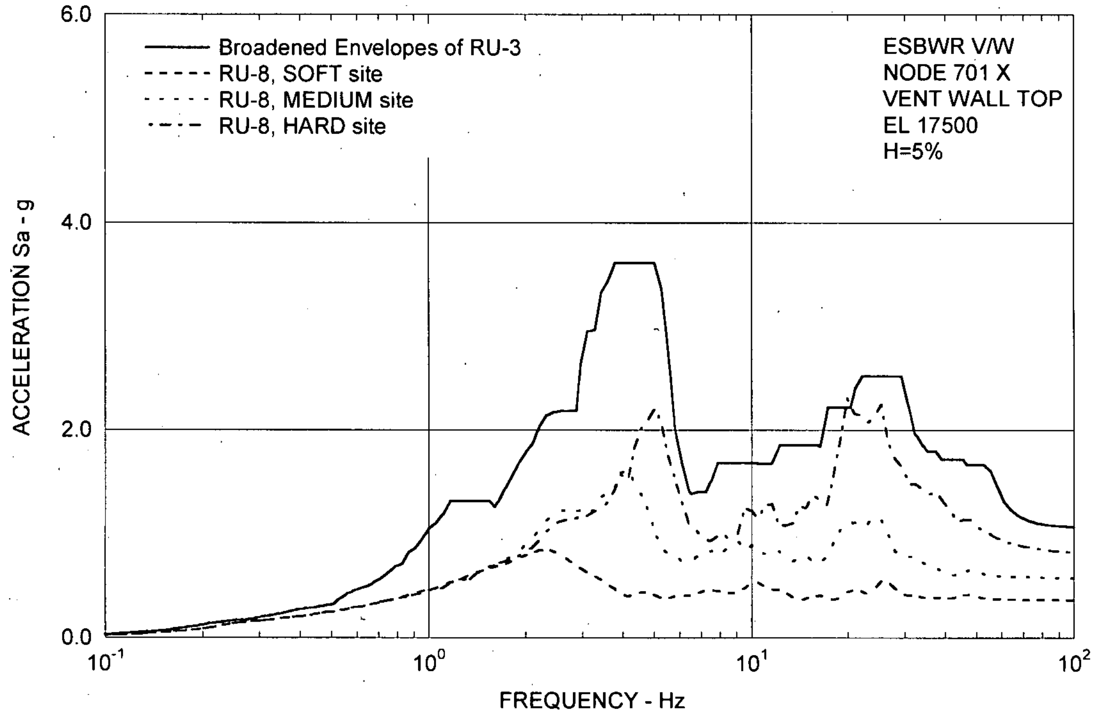


Figure 3A.8.7-1c. FRS (Effect of Embedment) – Vent Wall Top X

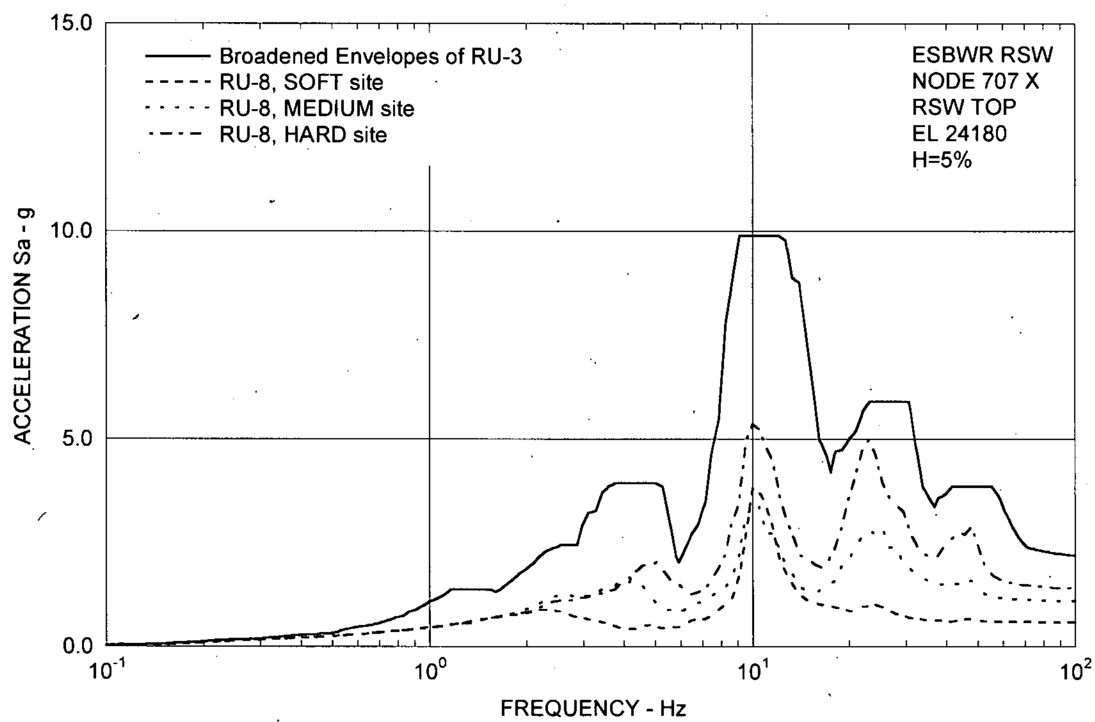


Figure 3A.8.7-1d. FRS (Effect of Embedment) – RSW Top X

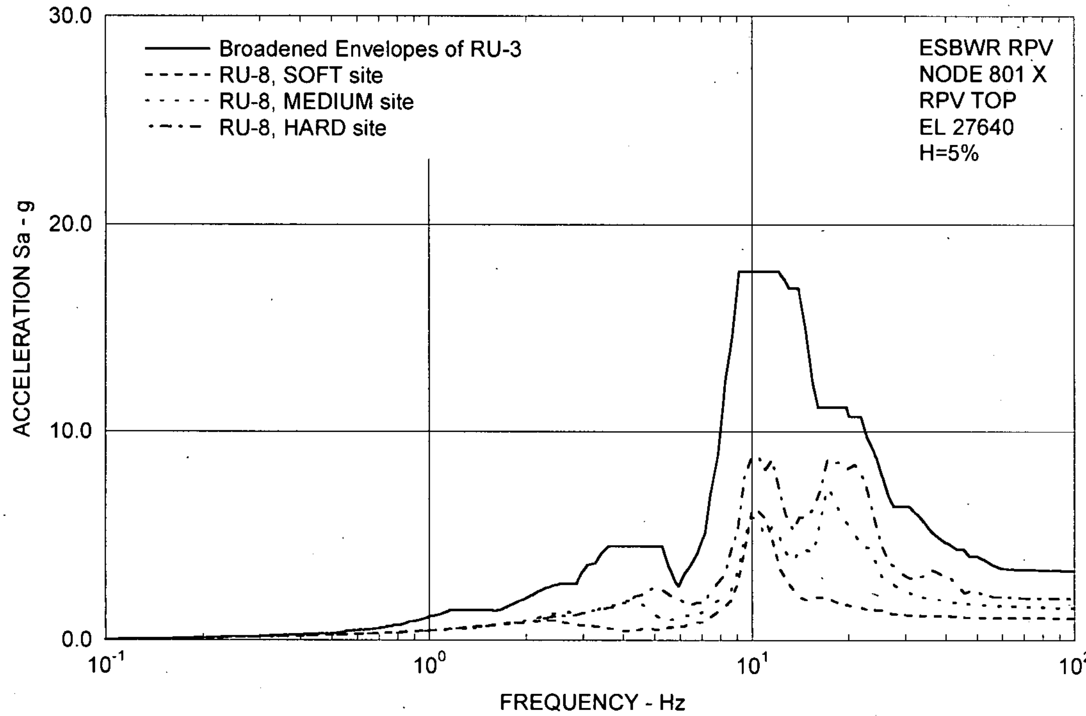


Figure 3A.8.7-1e. FRS (Effect of Embedment) – RPV Top X

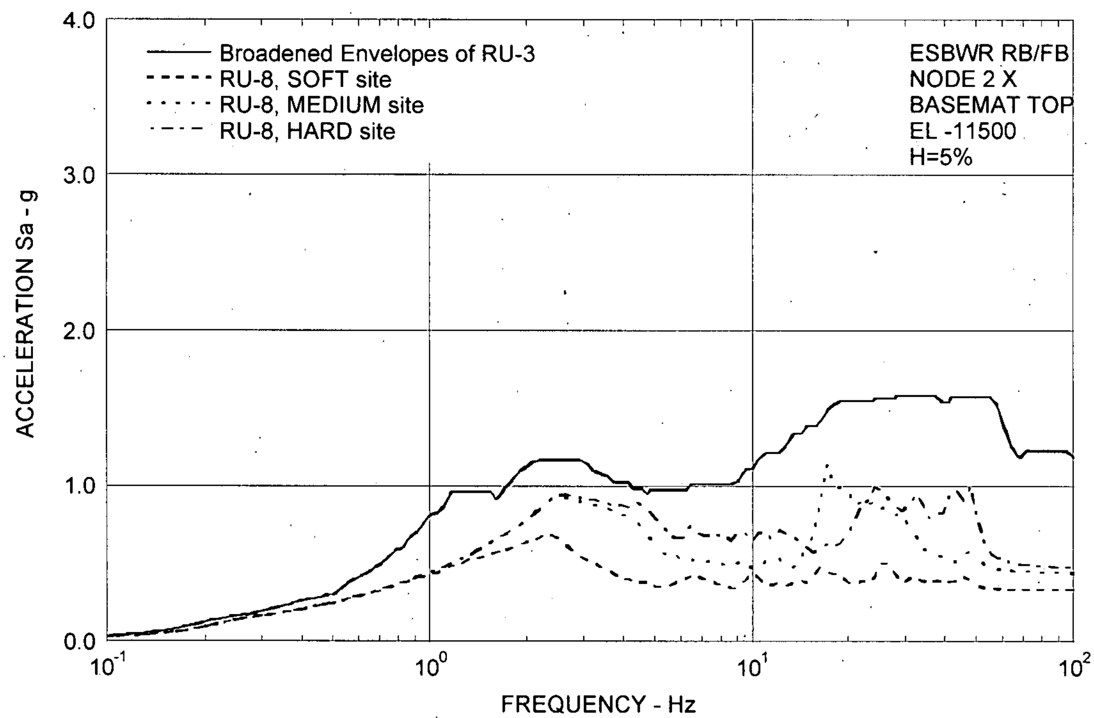


Figure 3A.8.7-1f. FRS (Effect of Embedment) – RB/FB Basemat X

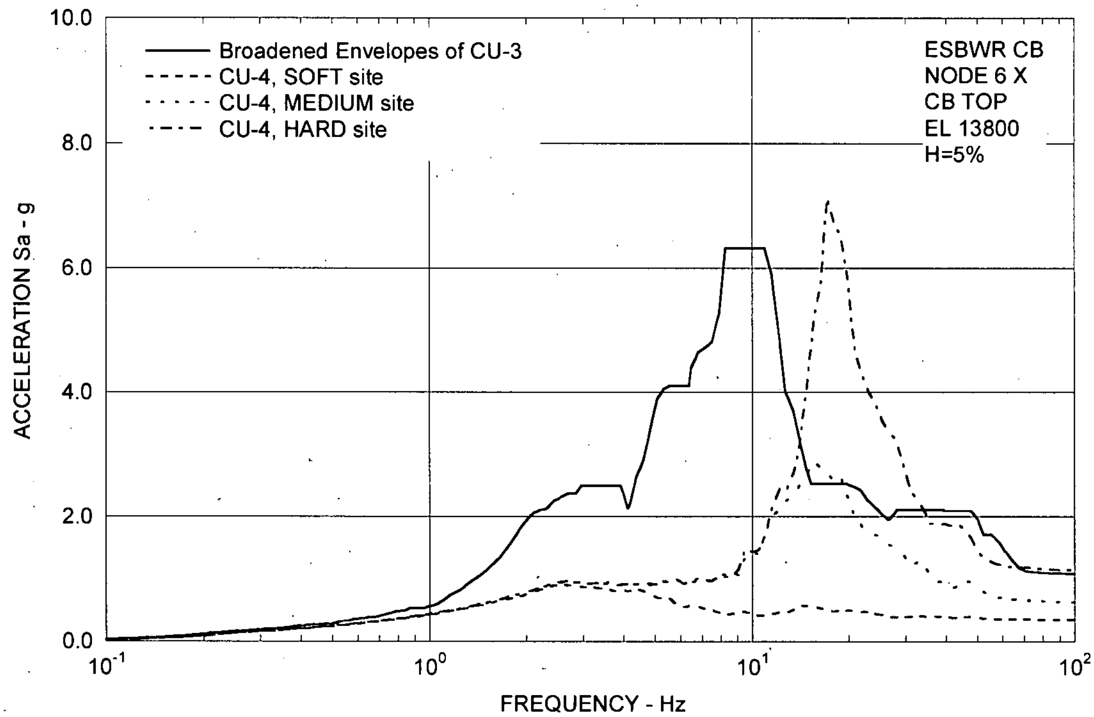


Figure 3A.8.7-1g. FRS (Effect of Embedment) – CB Top X

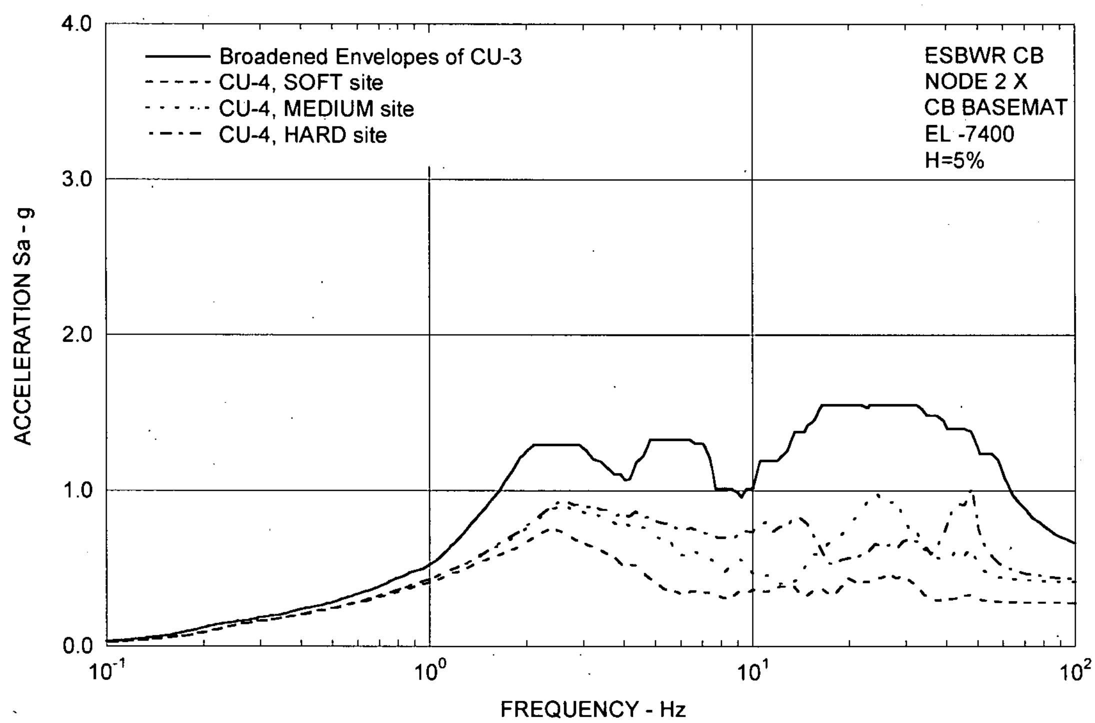


Figure 3A.8.7-1h. FRS (Effect of Embedment) – CB Basemat X

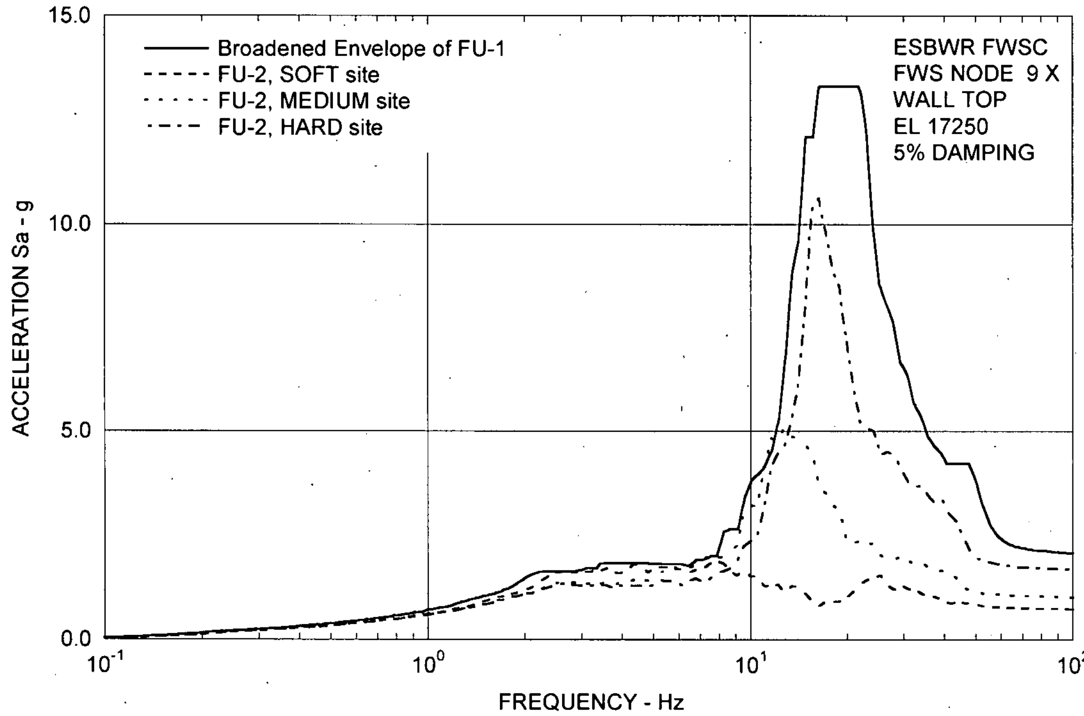


Figure 3A.8.7-1i. FRS (Effect of Embedment) – FWS Wall Top X

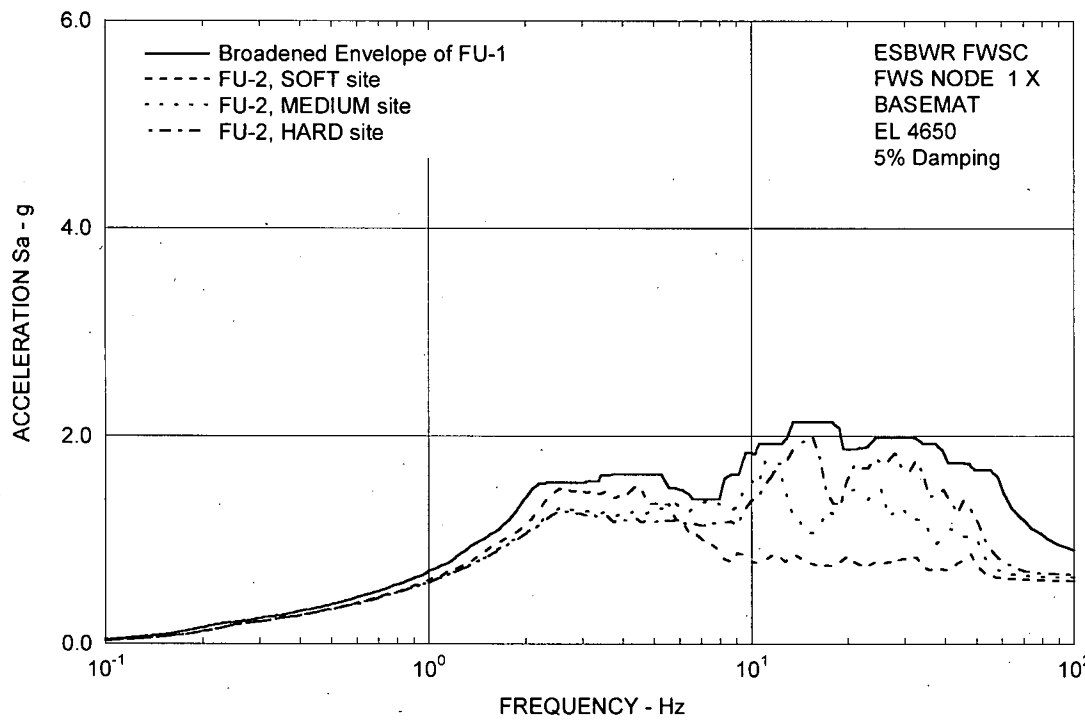


Figure 3A.8.7-1j. FRS (Effect of Embedment) – FWS Basemat X

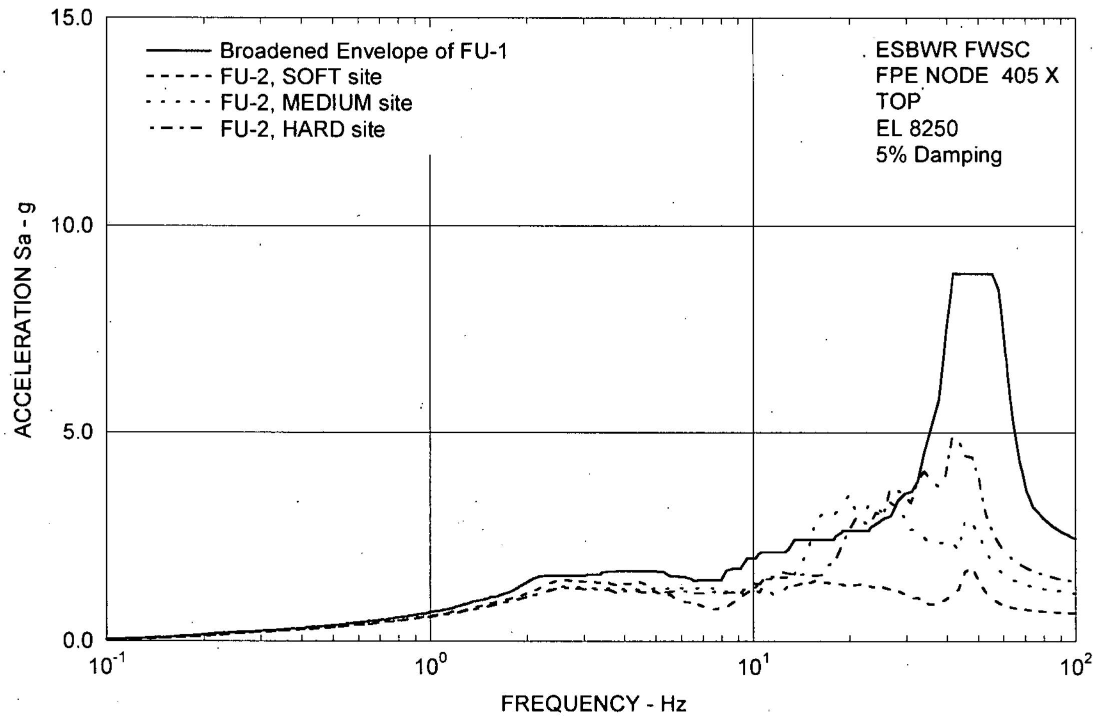


Figure 3A.8.7-1k. FRS (Effect of Embedment) – FPE Top X

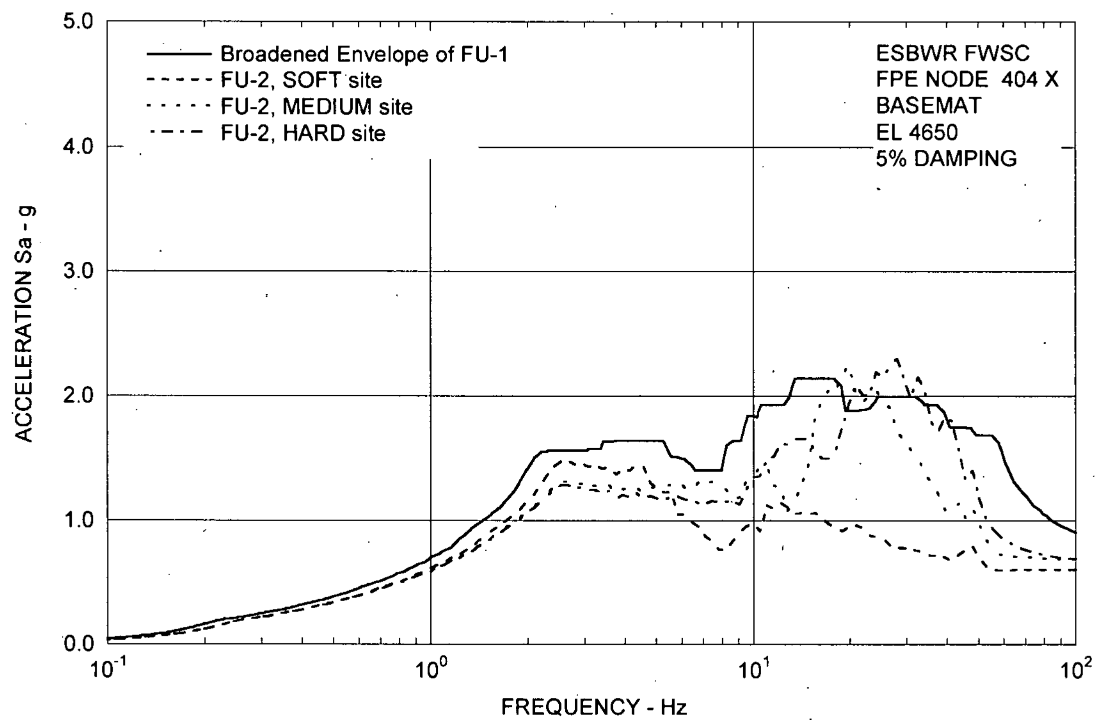


Figure 3A.8.7-1l. FRS (Effect of Embedment) – FPE Basemat X

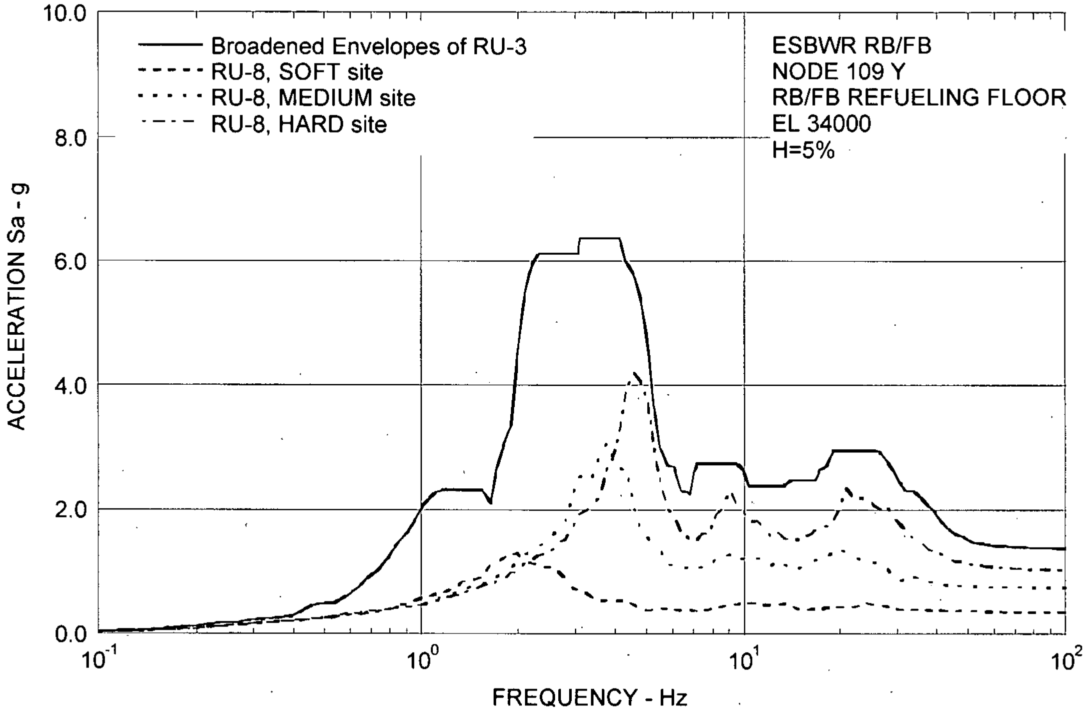


Figure 3A.8.7-2a. FRS (Effect of Embedment) – RB/FB Refueling Floor Y

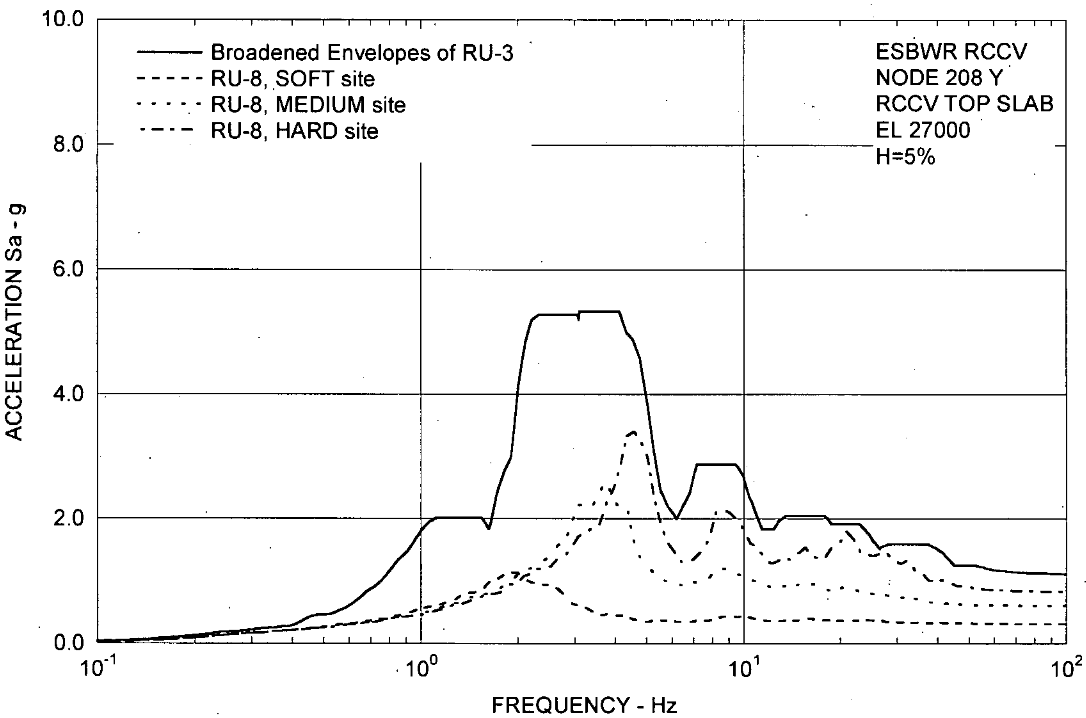


Figure 3A.8.7-2b. FRS (Effect of Embedment) – RCCV Top Slab Y

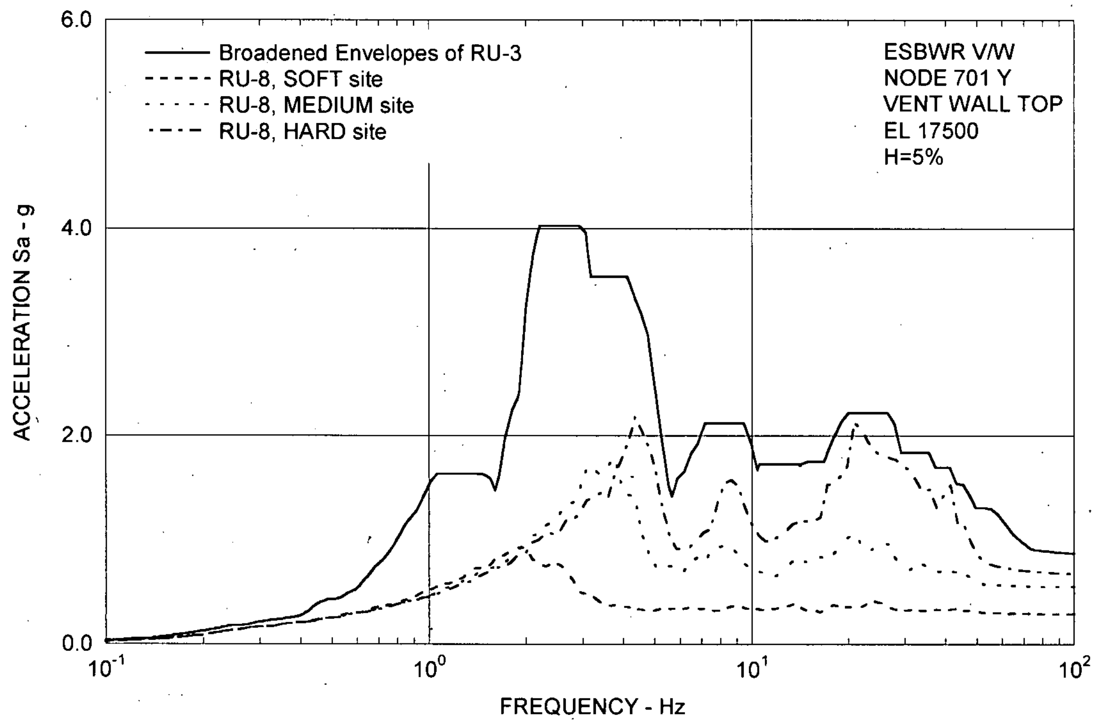


Figure 3A.8.7-2c. FRS (Effect of Embedment) – Vent Wall Top Y

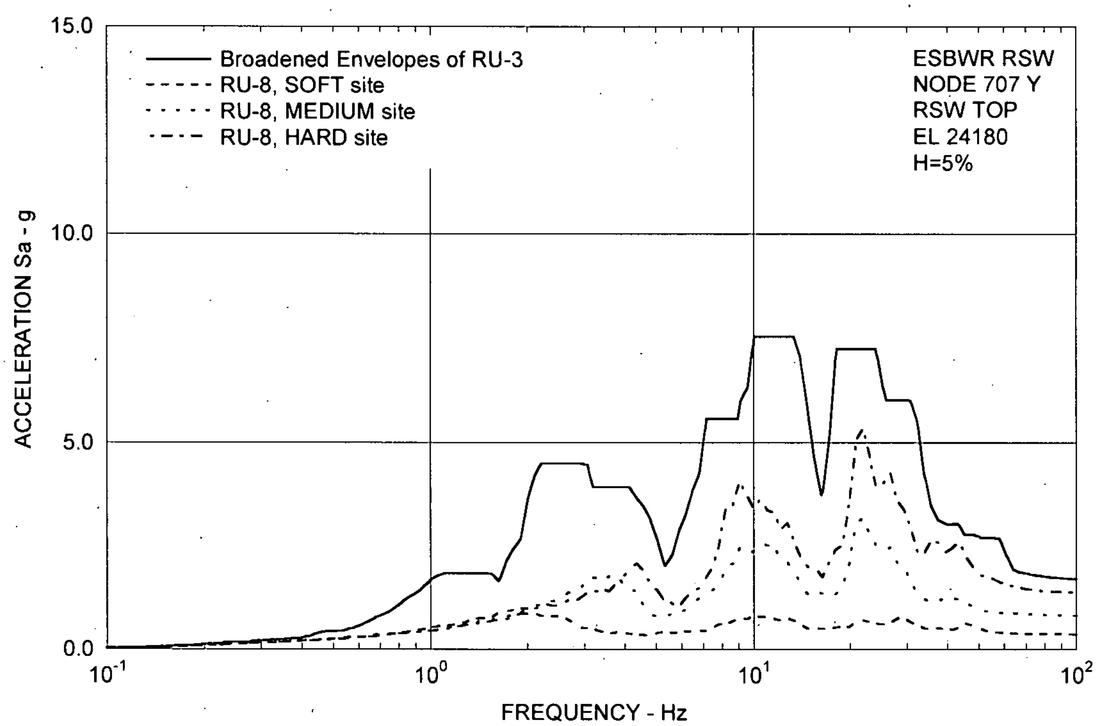


Figure 3A.8.7-2d. FRS (Effect of Embedment) – RSW Top Y

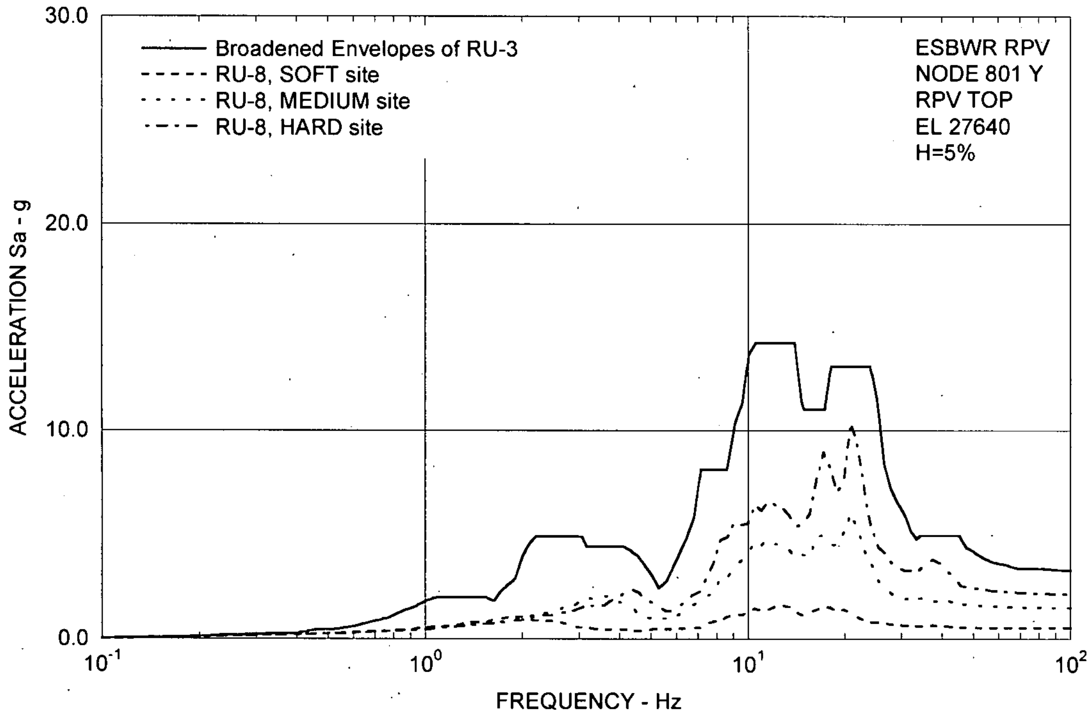


Figure 3A.8.7-2e. FRS (Effect of Embedment) – RPV Top Y

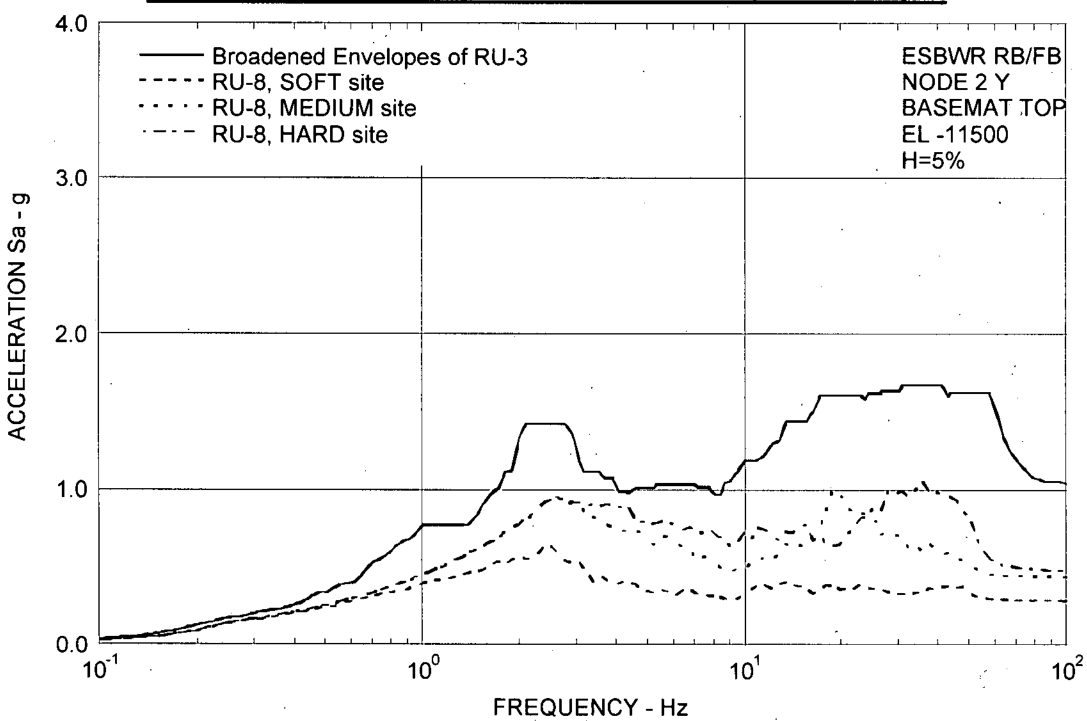


Figure 3A.8.7-2f. FRS (Effect of Embedment) – RB/FB Basemat Y

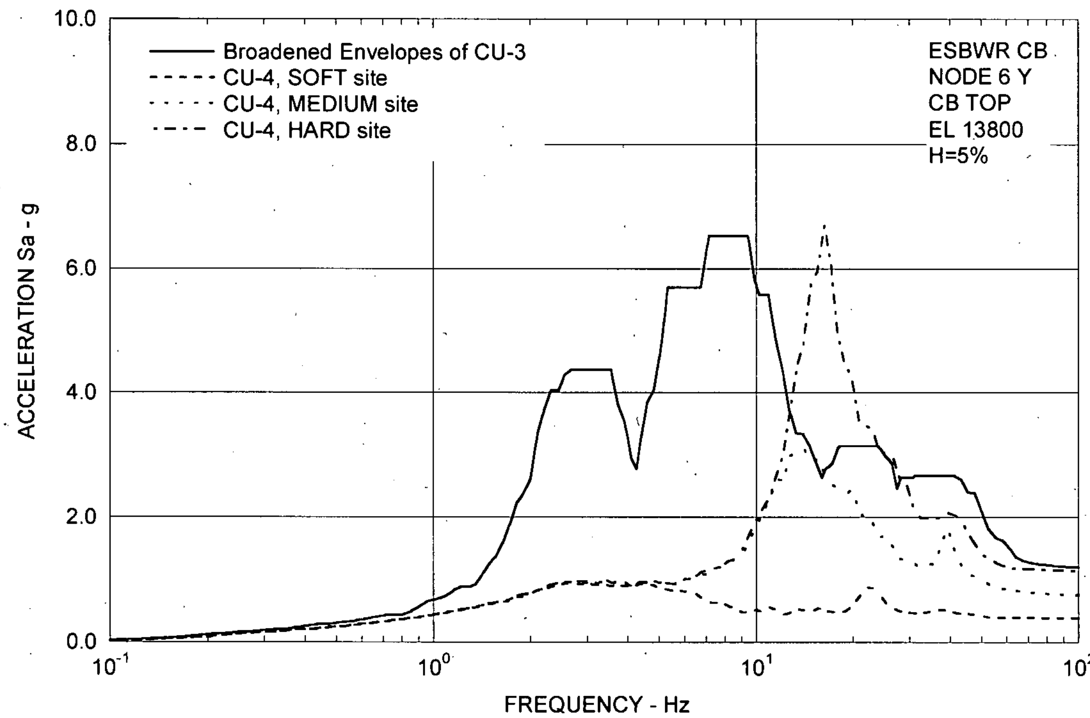


Figure 3A.8.7-2g. FRS (Effect of Embedment) – CB Top Y

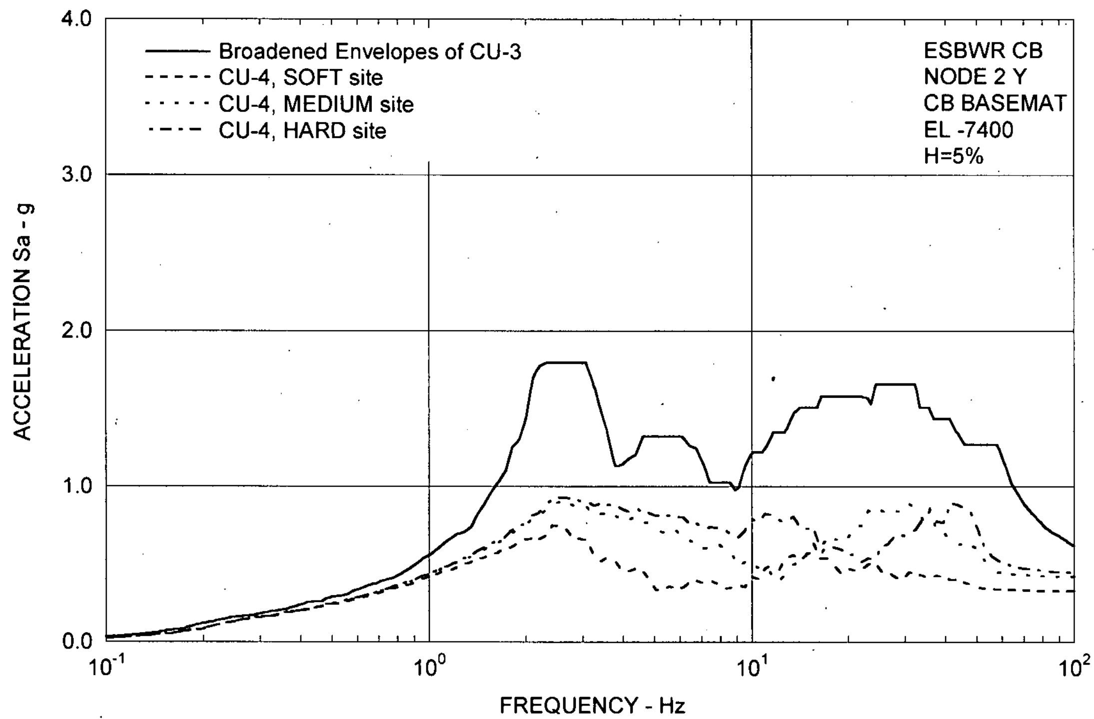


Figure 3A.8.7-2h. FRS (Effect of Embedment) – CB Basemat Y

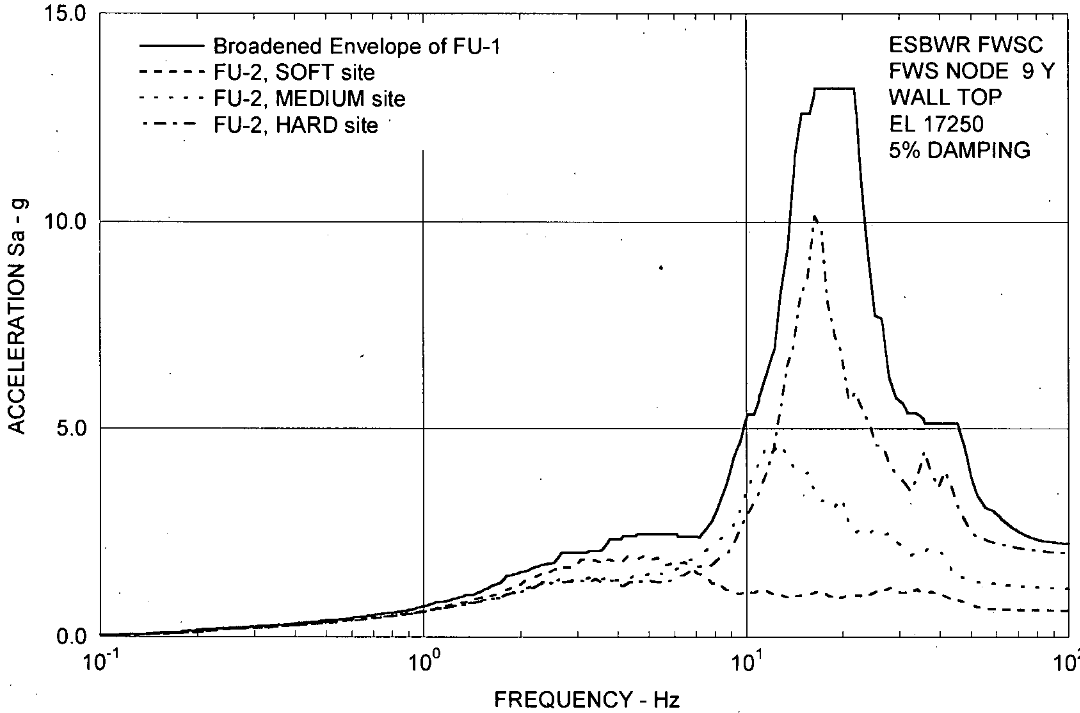


Figure 3A.8.7-2i. FRS (Effect of Embedment) – FWS Wall Top Y

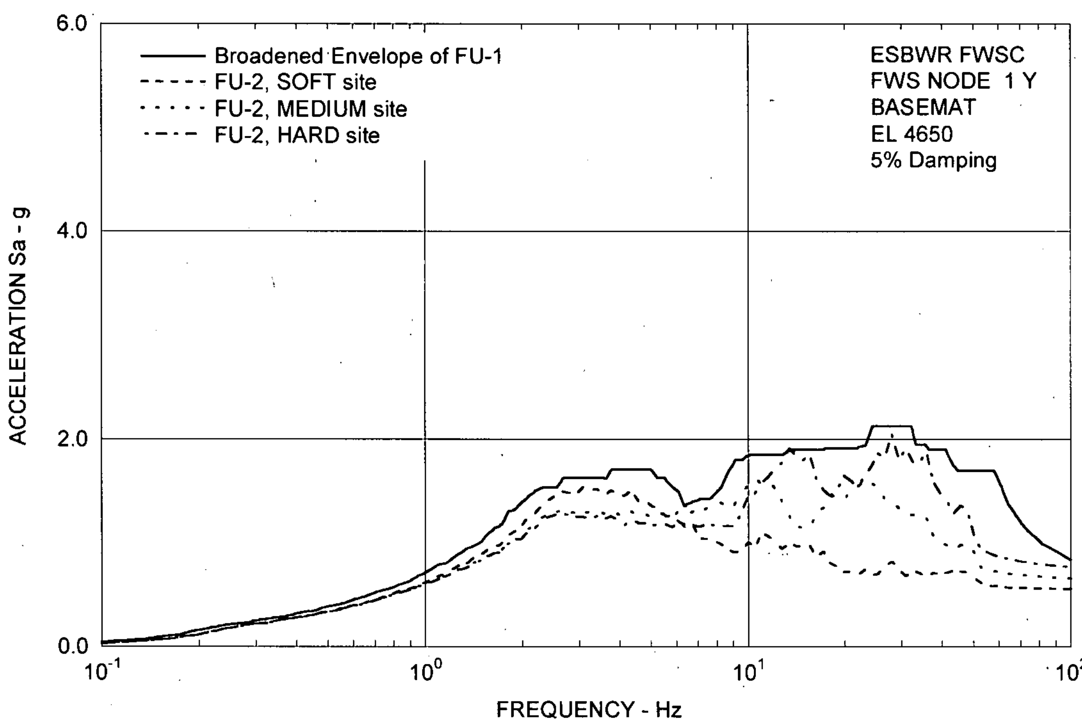


Figure 3A.8.7-2j. FRS (Effect of Embedment) – FWS Basemat Y

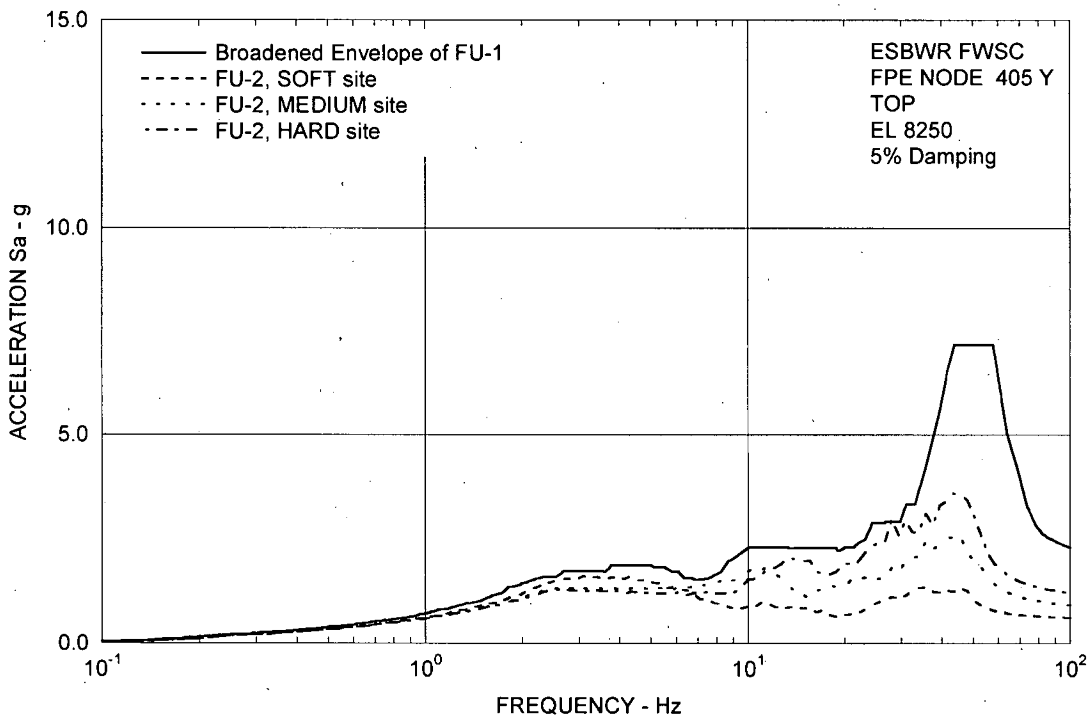


Figure 3A.8.7-2k. FRS (Effect of Embedment) – FPE Top Y

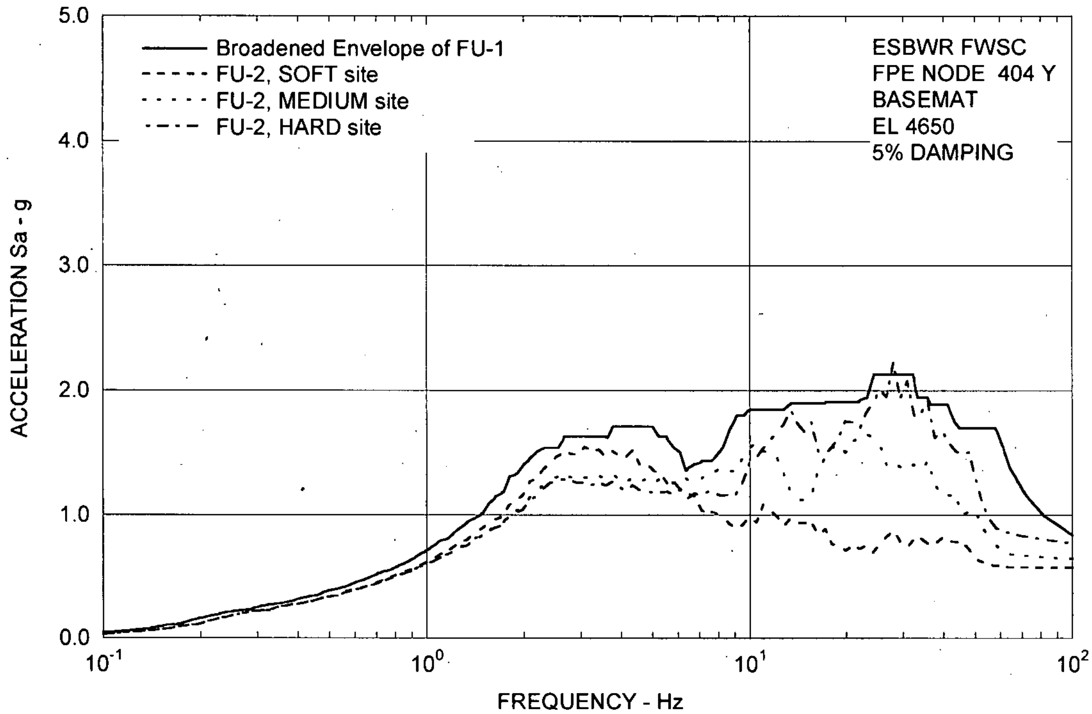


Figure 3A.8.7-2l. FRS (Effect of Embedment) – FPE Basemat Y

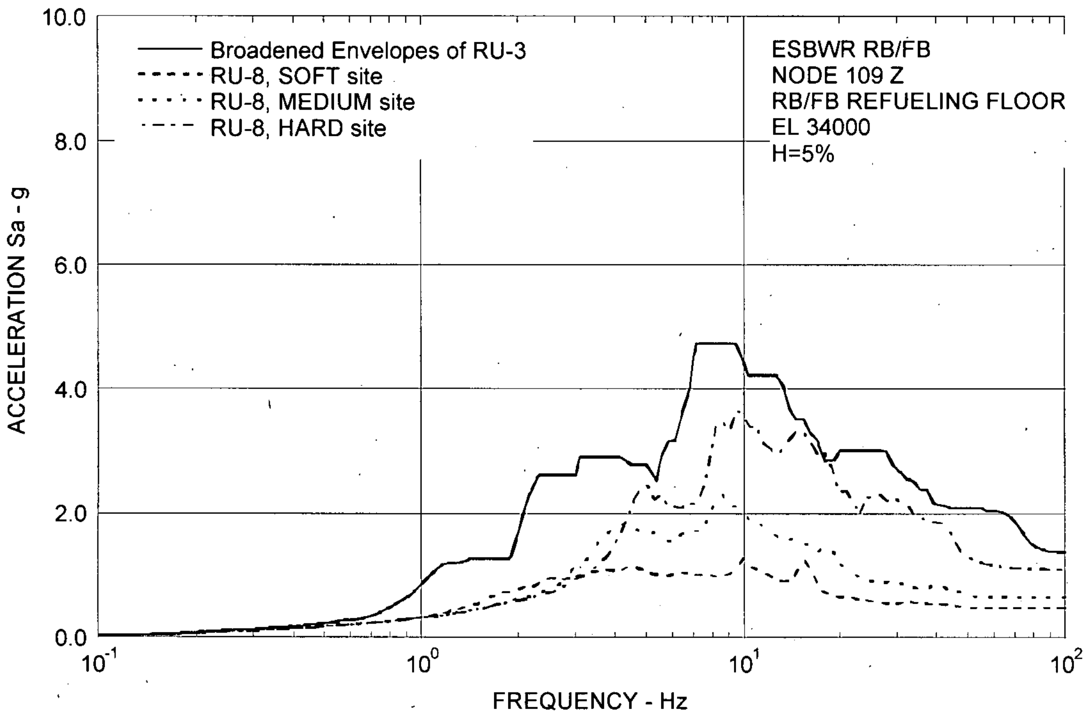


Figure 3A.8.7-3a. FRS (Effect of Embedment) – RB/FB Refueling Floor Z

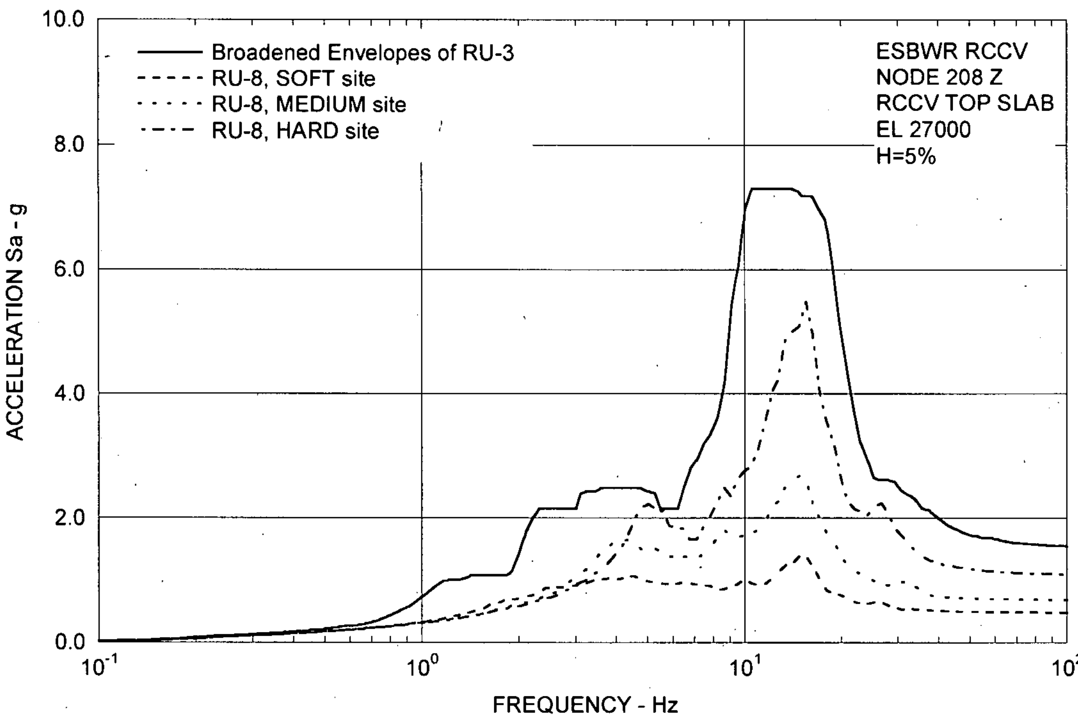


Figure 3A.8.7-3b. FRS (Effect of Embedment) – RCCV Top Slab Z

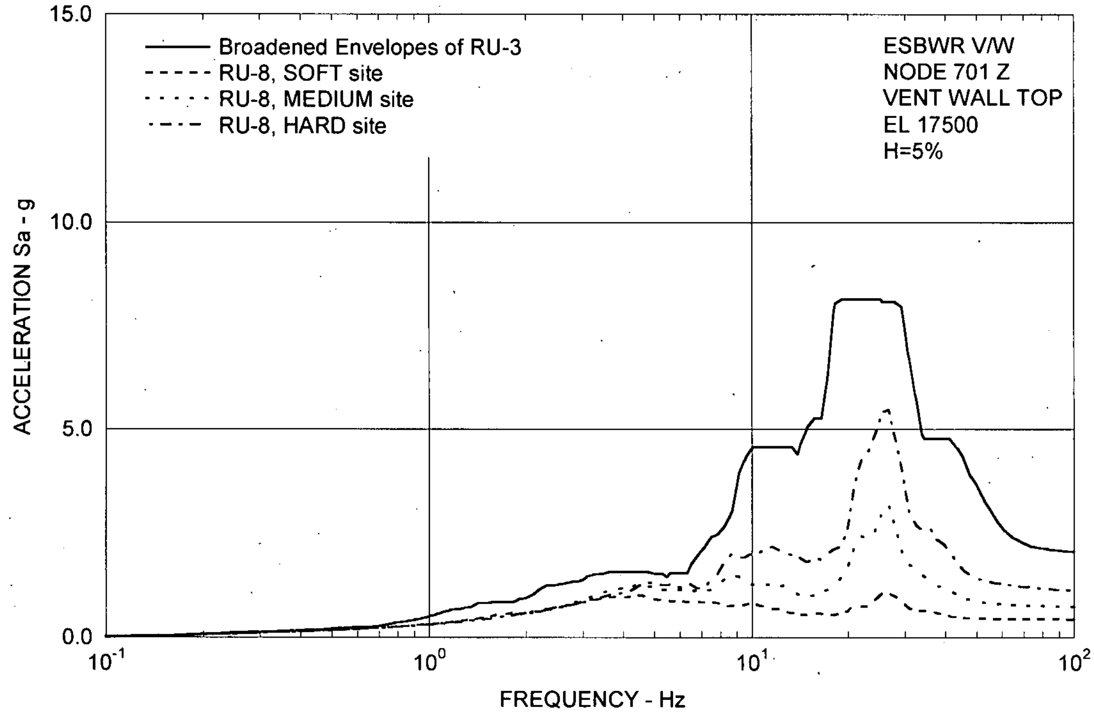


Figure 3A.8.7-3c. FRS (Effect of Embedment) – Vent Wall Top Z

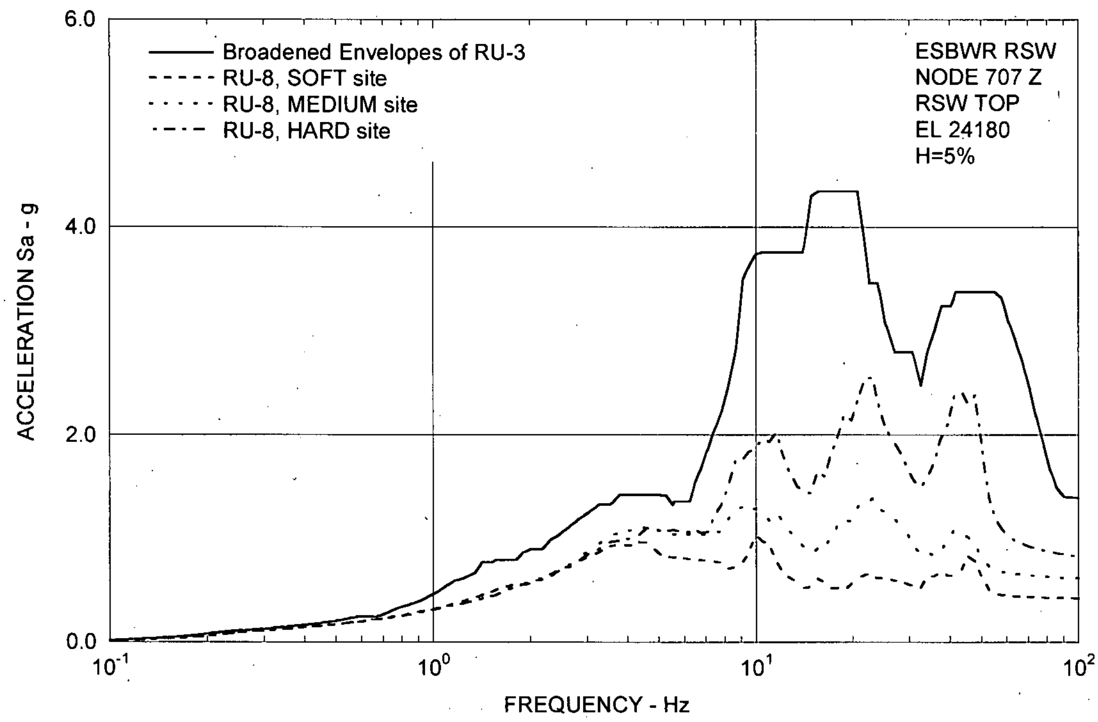


Figure 3A.8.7-3d. FRS (Effect of Embedment) – RSW Top Z

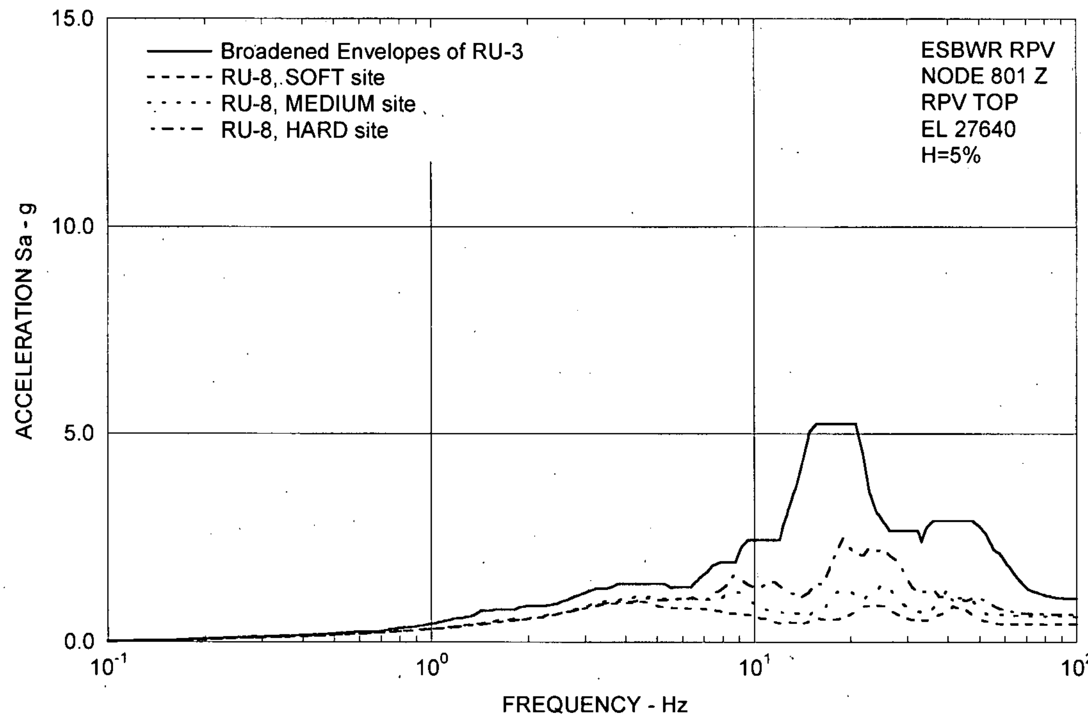


Figure 3A.8.7-3e. FRS (Effect of Embedment) – RPV Top Z

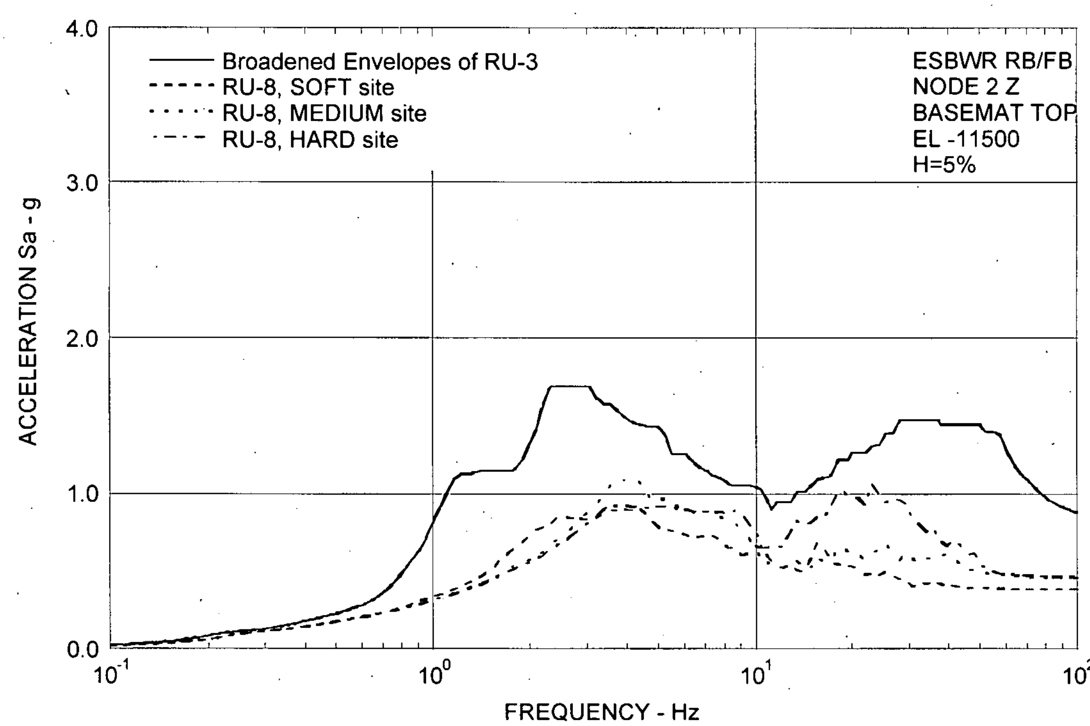


Figure 3A.8.7-3f. FRS (Effect of Embedment) – RB/FB Basemat Z

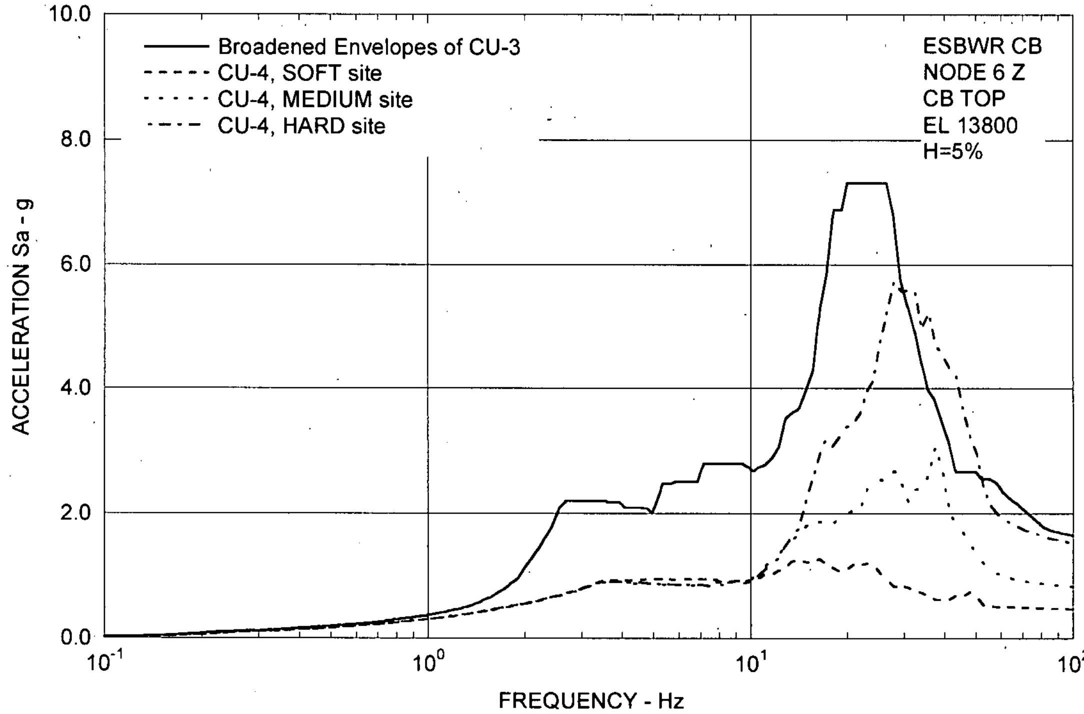


Figure 3A.8.7-3g. FRS (Effect of Embedment) – CB Top Z

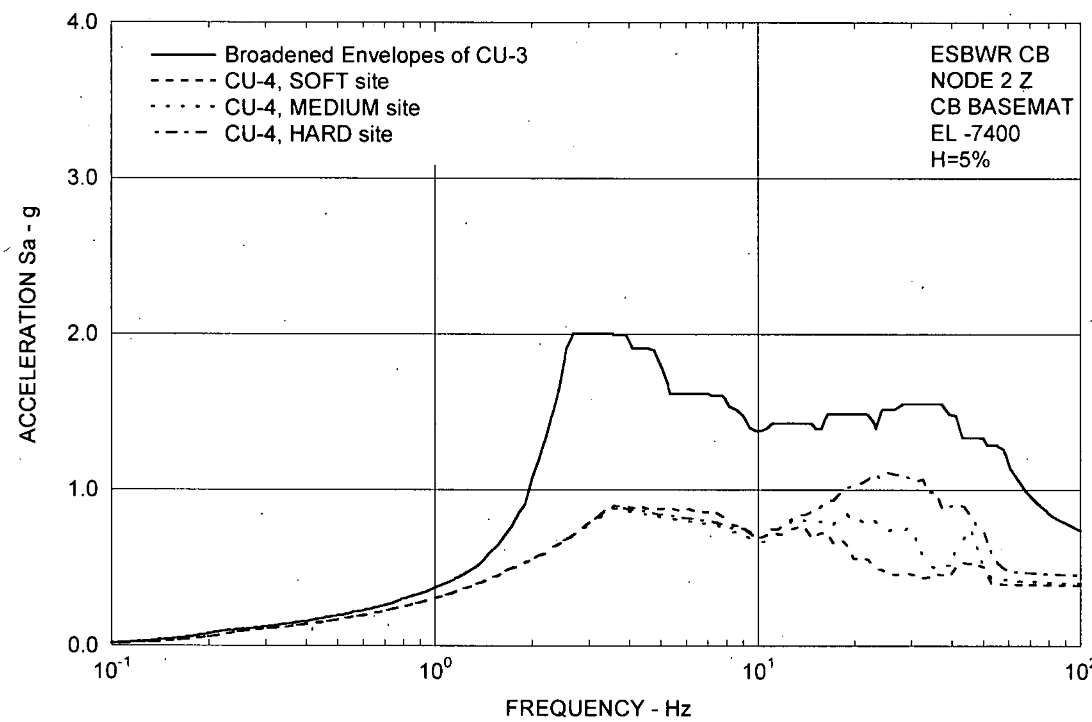


Figure 3A.8.7-3h. FRS (Effect of Embedment) – CB Basemat Z

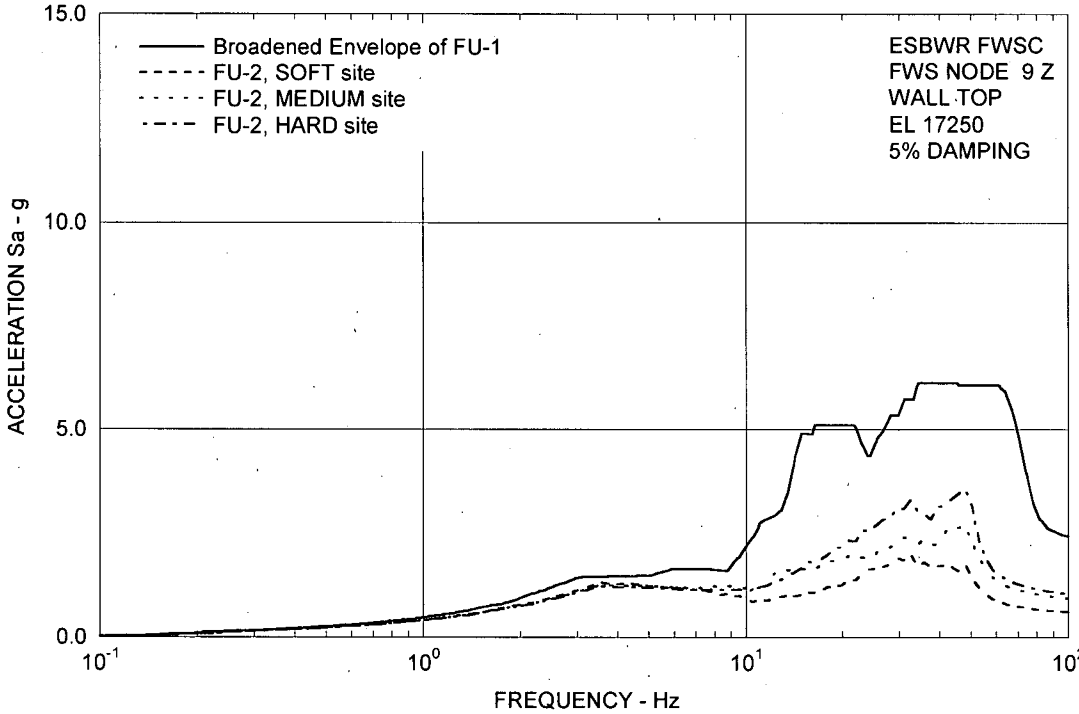


Figure 3A.8.7-3i. FRS (Effect of Embedment) – FWS Wall Top Z

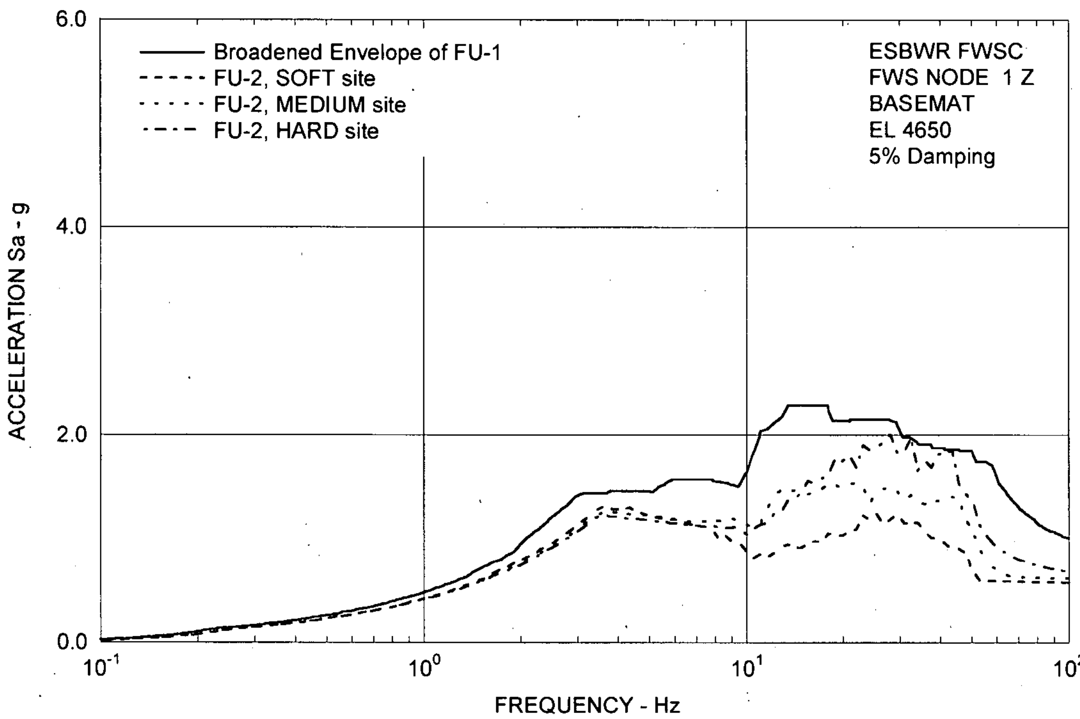


Figure 3A.8.7-3j. FRS (Effect of Embedment) – FWS Basemat Z

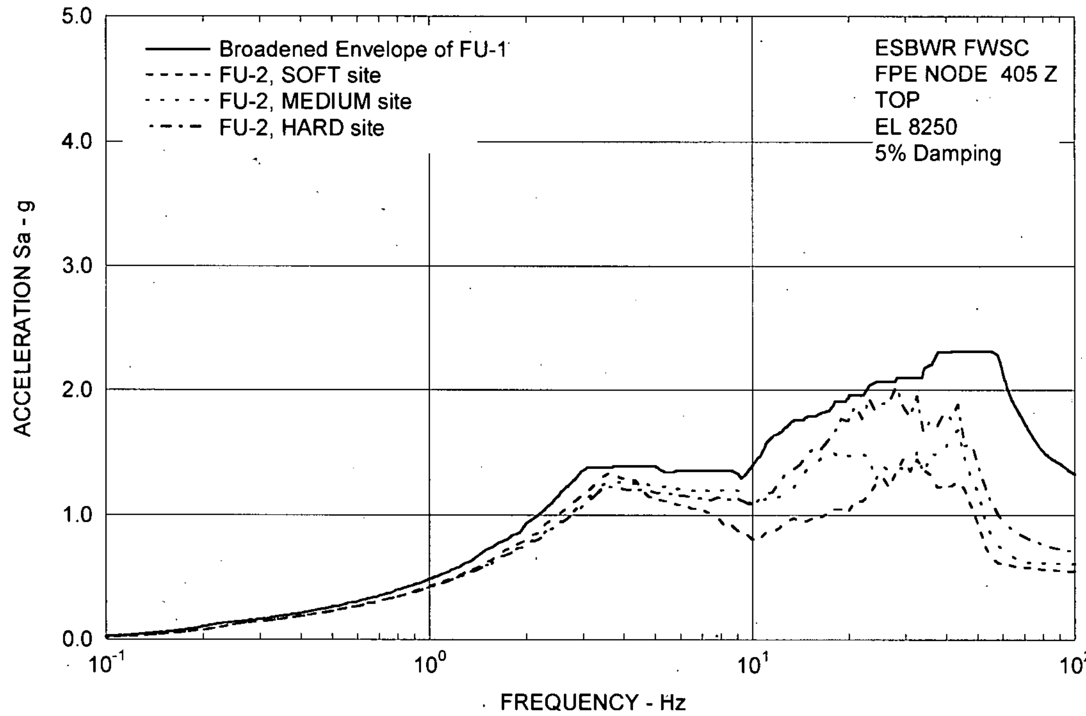


Figure 3A.8.7-3k. FRS (Effect of Embedment) – FPE Top Z

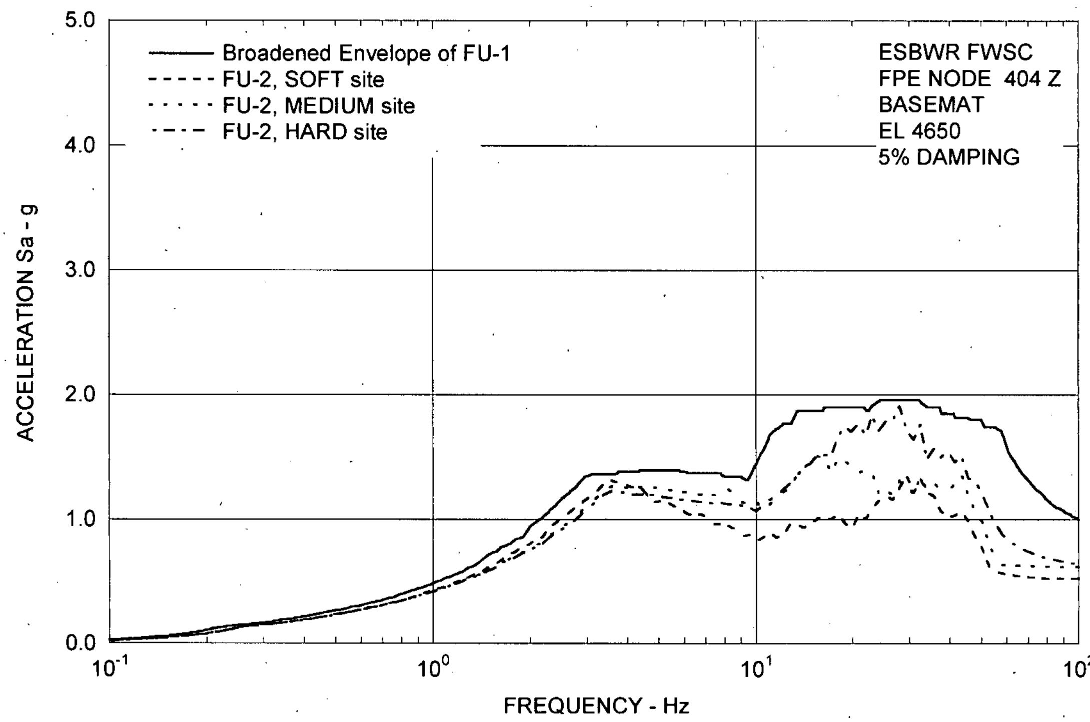


Figure 3A.8.7-3l. FRS (Effect of Embedment) – FPE Basemat Z

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 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 and SASSI2000 cases for layered sites L-2 and L-4 for the RB/FB and CB (see Subsection 3A.8.7 for details).

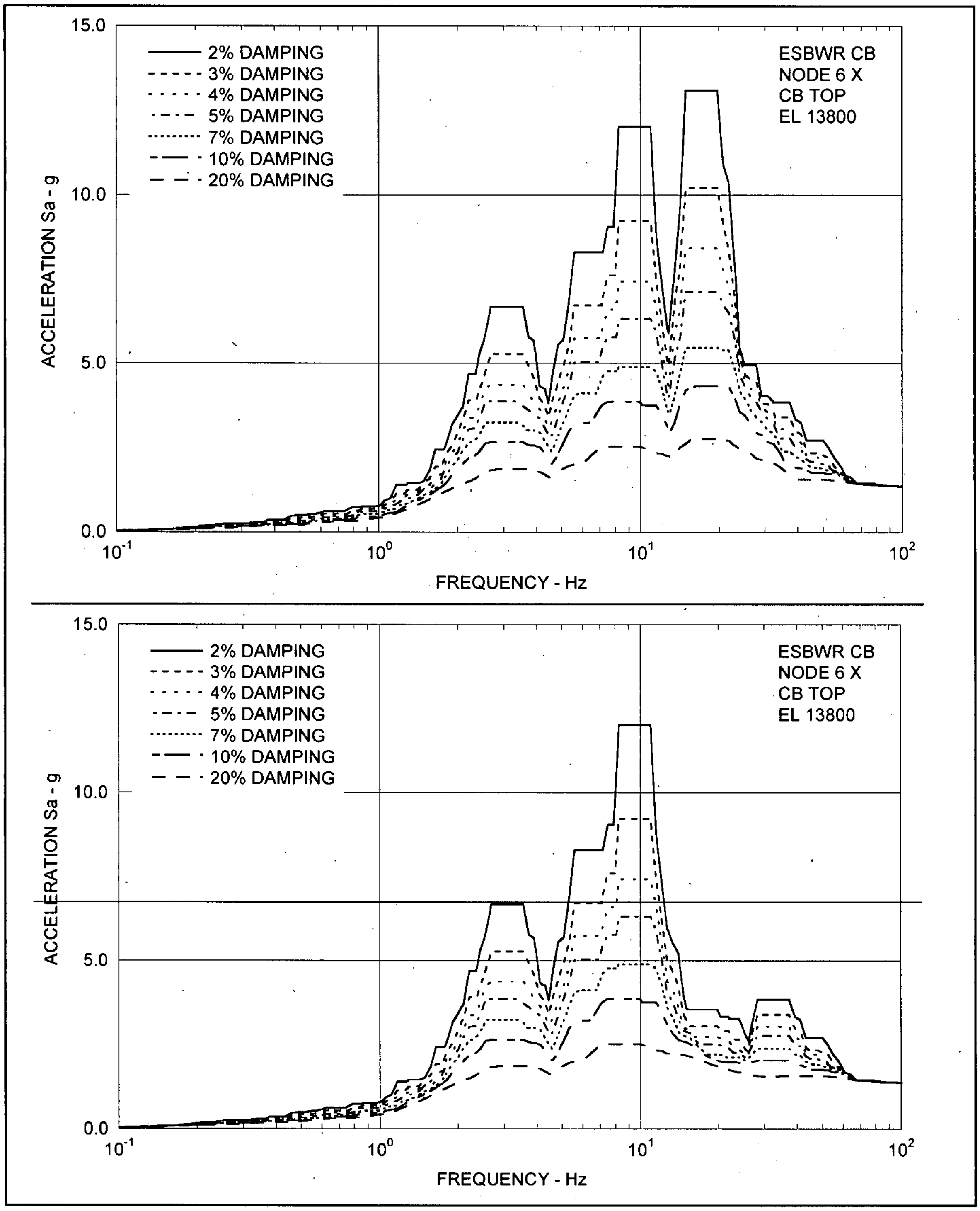
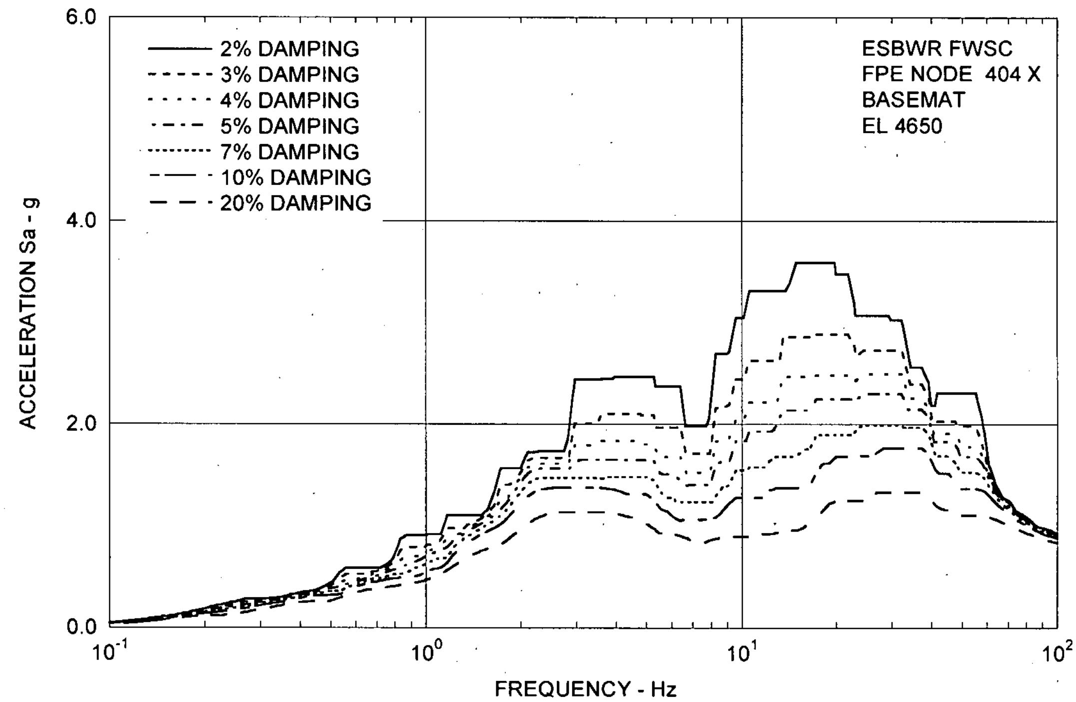


Figure 3A.9-1g. Enveloping Floor Response Spectra – CB Top X



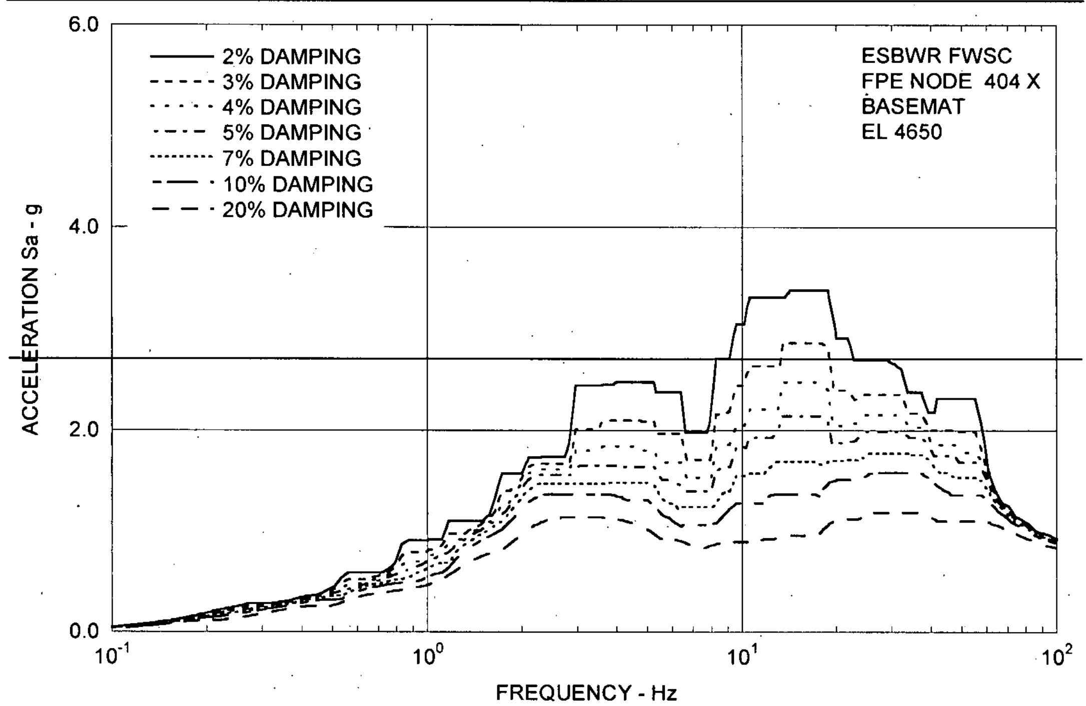


Figure 3A.9-1l. Enveloping Floor Response Spectra – FPE Basemat X

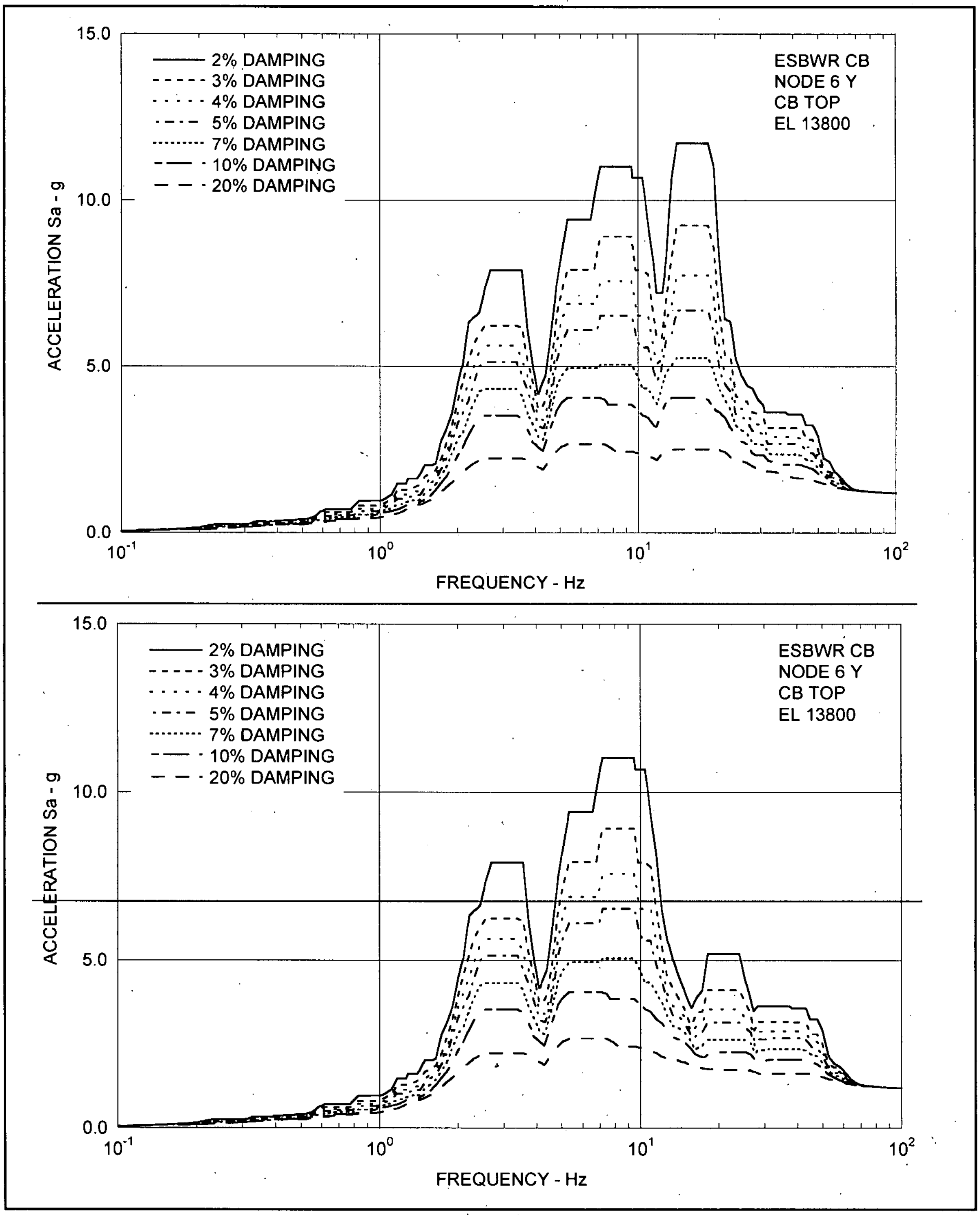


Figure 3A.9-2g. Enveloping Floor Response Spectra – CB Top Y

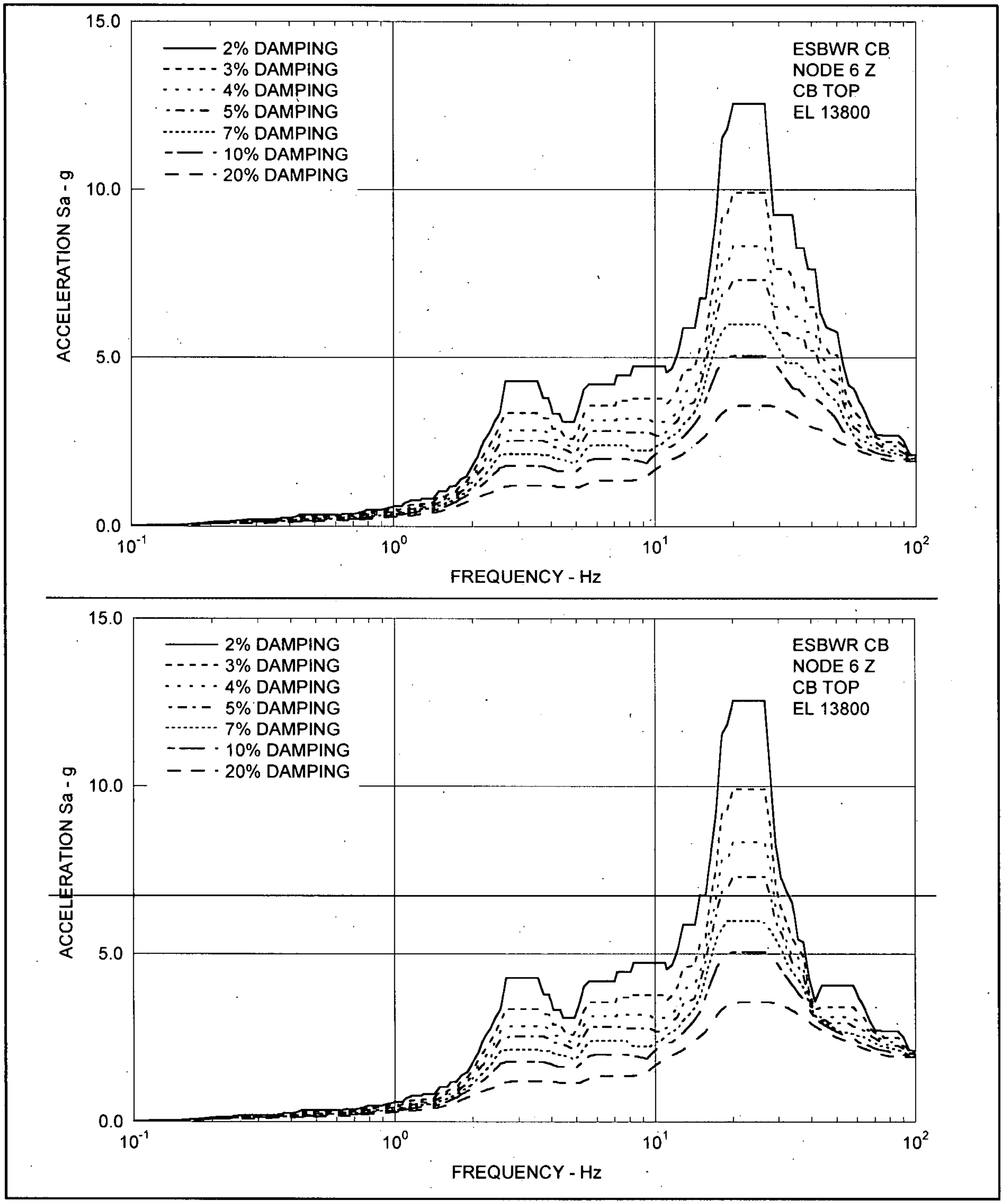


Figure 3A.9-3g. Enveloping Floor Response Spectra – CB Top Z

In the DAC3N, the SSI system is modeled by the combination of soil spring and damping coefficient. Spring and damping coefficient are determined as frequency independent values, which fit the frequency dependent real and imaginary parts of soil spring obtained by the theoretical methods, such as vibration admittance theory based on three-dimensional wave propagation theory for uniform half space soil.

As mass elements, lumped masses and consistent masses are available. Structural elements, such as beams, trusses, dampers and direct input matrices are available in this program.

This program has nonlinear analytical capability.

3C.7.1.2 Validation

DAC3N is coded and maintained by Shimizu Corporation of Tokyo, Japan. Program validation documentation is available at Shimizu Corporation.

3C.7.1.3 Extent of Application

This program is used to perform the SSI analysis ~~required to obtain enveloped seismic design loads of the concrete containment, RB, FB, CB and FWSC~~without embedment effect for Seismic Category I structures.

3C.7.2 Dynamic Soil-Structure Interaction Analysis Program – SASSI2000

3C.7.2.1 Description

SASSI2000 is used to solve a wide range of dynamic SSI problems~~, including layered soil conditions and embedment conditions,~~ in two or three dimensions. It was developed at the University of California, Berkeley in 1982 under the technical direction of John Lysmer. The program is based on the finite-element method formulated in the frequency domain using a substructuring technique.

3C.7.2.2 Validation

SASSI version 2000 was obtained from the University of California, Berkeley and implemented by Shimizu Corporation of Tokyo, Japan on the PC computer on Linux OS. Program validation documentation is available at UC Berkeley.

3C.7.2.3 Extent of Application

~~SASSI is used to obtain seismic design loads and in-structure floor response spectra for the Seismic Category I buildings accounting for the effects of SSI.~~This program is used to perform the SSI analysis for Seismic Category I structures.

3C.7.3 Free-Field Site Response Analysis – SHAKE

3C.7.3.1 Description

SHAKE is a program, which can perform the free-field site response analysis. It was developed at the University of California, Berkeley by B. Schnabel, John Lysmer and H.B. Seed in 1972. The program is based on the theory of one-dimensional propagation of shear waves in the

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	1.41	--	--
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 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)	1.2~~.7~~	~~7~~1.~~3~~5	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	1.28	--	--
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 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~~44	~~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	1.13	--	--
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 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)	0.446	0.69~~5~~4	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.

ratio of the largest to the smallest shear wave velocity over the mat foundation width at the foundation level 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.

(17) Adjacent layers are the two layers with a total depth of 40 m (131 ft) or 60 m (197 ft) below grade. They correspond to the top and middle layers shown in Table 3A.3-3 for layered site cases 2 and 4. 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.