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Revised CCNPP Unit 3 FSAR Section 2.5.2.6

converges. The final (or strain-compatible) stiffness and damping are then used to calculate the strain-compatible site transfer function. This transfer function is then multipled by the Fourier spectrum of the input rock motion to obtain the Fourier spectrum of the motion at the top of the profile or at the desired elevation (for either outcrop or in-column conditions), from which response spectra are calculated using RVT.

This process is repeated multiple times, once for each artificial profile . For sixty site profiles, sixty response spectra are calculated, from which statistics of site response are obtained.

The above calculations are repeated multiple times, once for each input rock spectrum. Thus the site response is calculated separately for the 10^{-4} HF, 10^{-4} LF, 10^{-5} HF, 10^{-5} LF, 10^{-6} HF, and 10^{-6} LF spectra.

In comparison to the SHAKE approach, the RVT approach avoids the requirement of performing spectral matching on the input time histories to match an input rock spectrum, and avoids analyzing each individual time history with a site-response program.

The site amplification factor is defined as the surface response spectral amplitude at each frequency, computed using the set of profiles that do not contain the 41 feet of fill above the nuclear island, divided by the input rock spectral amplitude. Figure 2.5-78 shows the logarithmic mean and standard deviation of site amplification factor from the 60 profiles for the 10^{-4} HF input motion. As would be expected by the large depth of sediments at the site, amplifications are largest at low frequencies, and de-amplification occurs at high frequencies because of soil damping. The maximum strains in the soil column are low for this motion, and this is shown in Figure 2.5-79, which plots the maximum strains calculated for the 60 profiles versus depth. Maximum strains are generally less than 0.01 percent, with some profiles having strains in shallow layers up to 0.03 percent.

Figure 2.5-80 and Figure 2.5-81 show similar plots of amplification factors and maximum strains for the 10^{-4} LF motion. The results are similar to those for the HF motion, with the soil column generally exhibiting maximum strains less than 0.01 percent.

Figure 2.5-82 through Figure 2.5-85 show comparable plots of amplification factors and maximum strains for the 10^{-5} input motion, both HF and LF. For this higher motion, larger maximum strains are observed, but they are still generally less than 0.03 percent. A few profiles exhibit maximum strains of about 0.1 percent at shallow depths. These strains are within the range for which the equivalent linear site response formulation has been validated.

Table 2.5-23 documents the mean amplification factors for 10^{-4}, 10^{-5}, and 10^{-6} rock input motions, and for HF and LF spectra.}

2.5.2.6 Ground Motion Response Spectra

The U.S. EPR FSAR includes the following COL Item in Section 2.5.2.6:

> A COL applicant that references the U.S. EPR design certification will compare the final site-specific soil characteristics with the U.S. EPR design generic soil parameters and verify that the site-specific seismic characteristics are enveloped by the CSDRS (anchored at 0.3 g PGA) and the 10 generic soil profiles discussed in Section 2.5.2 and Section 3.7.1 and summarized in Table 3.7.1-6.

This COL Item is addressed as follows:

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{This section and Section 3.7.1 describes the reconciliation {The purpose of the site-specific soil characteristics and site-specific seismic characteristics for CCNPP Unit 3 and demonstrates that these parameters are enveloped by the Certified Seismic Design Response Spectra (CSDRS), anchored at 0.3 g PGA, and the 10 generic soil profiles used in the design of the U.S. EPR.this section is:

Table 5.0-1 of the U.S. EPR FSAR identifies shear wave velocity as a required parameter to be enveloped, defined as "Minimum shear wave velocity of 1000 feet per second (Low strain best estimate average value at bottom of basemat)."

- to describe the development of the Ground Motion Response Spectra (GMRS) and,
- to reconcile the CCNPP3 Site Seismic Characteristics with the U.S. EPR FSAR generic seismic analysis input and results.

Figure 2.5-102 compares the 10 generic soil profile cases used for the U.S. EPR and the average shear wave velocity profile that was adopted for the CCNPP site (shown in Figure 2.5-74 and Figure 2.5-75.

The CCNPP Unit 3 Average Profile shown in the Figure 2.5-102 is for soils below El. +44 ft (bottom of the basemat is zero in the figure). Soils such as Stratum I Terrace Sand will not be used for support of foundations of Category I structures. Therefore, shear wave velocity measurements in the CCNPP site soils above El. +44 ft. regardless of value, are excluded from this evaluation as they lie above the basemat. Results from the above Figure indicate that:

1. The CCNPP Unit 3 Average Profile is bounded by the 10 generic profiles used for the U.S. EPR.
2. The CCNPP Unit 3 Average Profile offers a shear wave velocity at the bottom of the basemat (approx. El. +44 ft (or depth = 0 in Figure 2.5-102)) of 1,450 ft/sec.
3. The minimum shear wave velocity from the CCNPP Unit 3 Average Profile is 1,130 ft/sec.
4. The characteristic shear wave velocity of the soil column (weighted with respect to the 344 ft soil column) is 1,510 ft/sec.

On the basis of the above, the idealized CCNPP Unit 3 site shear wave velocity profile is bounded by the 10 generic soil profiles used for the U.S. EPR and meets the minimum 1,000 ft/sec criterion identified in the U.S. EPR FSAR.

GMRS was conducted in accordance with the performance-based approach described in Regulatory Position 5 of Regulatory Guide 1.208 (NRC, 2007a).

The GMRS was developed starting from the 10^{-4} and 10^{-5} rock Uniform Hazard Spectra. At high frequencies, the appropriate (10^{-4} or 10^{-5}) HF mean amplification factor was applied to the 10^{-4} and 10^{-5} rock spectrum, to calculate site spectral amplitudes for 10^{-4} and 10^{-5} annual frequencies of exceedance. At low frequencies, a similar technique was used with the LF mean amplification factors. At intermediate frequencies the larger of the HF and LF site spectral amplitudes was used.

Figure 2.5-86 illustrates the resulting site spectra. At high frequencies the HF spectral amplitudes are always greater, and at low frequencies the LF spectral amplitudes are always greater. The two sets of spectral amplitudes cross at 2-3 Hz.

This procedure corresponds to Approach 2A in NUREG/CR-6728 (NRC, 2001) and NUREG/CR-6769 (NRC, 2002b), wherein the rock Uniform Hazard Spectra (for example, at 10^{-4}) is multiplied by a mean amplification factor at each frequency to estimate the 10^{-4} site Uniform Hazard Spectra. Note that the amplification factors plotted in Figure 2.5-78, Figure 2.5-80, Figure 2.5-82, and Figure 2.5-84 are mean logarithmic amplification factors, which correspond approximately to the median. The amplification factors used to prepare Figure 2.5-86 are arithmetic mean amplification factors, which are slightly higher than the median.

The low-frequency character of the spectra in Figure 2.5-86 reflects the low-frequency amplification of the site, as shown in the amplification factors of Figure 2.5-78, Figure 2.5-80, Figure 2.5-82, and Figure 2.5-84. That is, there is a fundamental site resonance at about 0.22 Hz, with a dip in site response at about 0.4 Hz, and this dip occurs for all 60 of the site profiles that were used to characterize the site profile. As a result, there is a dip in the site spectra for 10^{-4} and 10^{-5} at 0.4 Hz that reflects the site characteristics.

The ASCE (ASCE, 2005) performance-based approach was used to derive a GMRS from the 10^{-4} and 10^{-5} site spectra. The spectrum is derived at each structural frequency as follows:

$$A_R = SA(10^{-5})/SA(10^{-4})$$

$$DF = 0.6\,A_R^{0.8}$$

$$GMRS = max(SA(10^{-4}) \times max(1.0, DF), 0.45 \times SA(10^{-5}))$$

The last term in the above equation was not published in this form in ASCE (ASCE, 2005) but is a supplemental modified form, as presented in NRC Regulatory Guide 1.208 (NRC, 2007a). The resulting horizontal spectrum is plotted in Figure 2.5-87. This spectrum has been smoothed slightly, particularly around 1.5 Hz, to remove slight bumps and dips in the spectrum resulting from the site amplification calculations that are not statistically significant. The average change in spectral amplitudes for the 5 frequencies that were smoothed was an increase of 1%, which is not significant.

A vertical spectrum was calculated by deriving vertical-to-horizontal (V:H) ratios and applying them to the horizontal spectrum. As background and for comparison purposes, V:H ratios were obtained by the following methods:

1. Rock V:H ratios for the central and eastern United States (CEUS) were calculated from NUREG-6728 (NRC, 2001), using the recommended ratios for PGA < 0.2g, which applies at this site (see Figure 2.5-88).

2. Soil V:H ratios for the western United States (WUS) were calculated from two publications (Abrahamson, 1997) (Campbell, 1997) that have equations estimating both horizontal and vertical motions on soil. Horizontal and vertical motions were predicted from these two references for M = 5.5 and R = 9 mi (15 km). M = 5.5 was selected because earthquakes around this magnitude dominate the high frequency motions, and R = 9 mi (15 km) was selected because this distance resulted in a horitontal PGA of approximately 0.1 g at the site, which is close to the PGA associated

with the horizontal SSE. For each reference, the V:H ratio was formed, and the average ratio (average from the two references) was then calculated.

3. The WUS V/H ratios for soil were modified in an approximate way for CEUS conditions by shifting the frequency axis of the V:H ratios so that they more closely resemble what might be expected at a soil site. This shifted the WUS peak V/H ratio from about 15 Hz to about 45 Hz.

Figure 2.5-88 shows these three V:H ratios plotted vs. structural frequency. As a conservative choice, the envelope V/H ratio shown as a thick dashed line was selected because this envelops all three approaches. The recommended V:H ratio is 1.0 for frequencies greater than 25 Hz, 0.75 for frequencies less than 5 Hz, and is interpolated (log-linear) between 5 and 25 Hz. Figure 2.5-87 plots the resulting vertical spectrum, calculated in this manner from the horizontal spectrum. Table 2.5-22 lists the horizontal and vertical GMRS amplitudes.

2.5.2.6.1 Ground Motion Response Spectra Development

This section and Section 3.7.1 describes the reconciliation of the site-specific soil characteristics and site-specific seismic characteristics for CCNPP Unit 3 and demonstrates that these parameters are enveloped by the Certified Seismic Design Response Spectra (CSDRS), anchored at 0.3 g PGA, and the 10 generic soil profiles used in the design of the U.S. EPR.

Table 5.0-1 of the U.S. EPR FSAR identifies shear wave velocity as a required parameter to be enveloped, defined as "Minimum shear wave velocity of 1000 feet per second (Low strain best estimate average value at bottom of basemat)."

Figure 2.5-102 compares the 10 generic soil profile cases used for the U.S. EPR and the average shear wave velocity profile that was adopted for the CCNPP site (shown in Figure 2.5-74 and Figure 2.5-75.

The CCNPP Unit 3 Average Profile shown in the Figure 2.5-102 is for soils below El. +44 ft (bottom of the basemat is zero in the figure). Soils such as Stratum I Terrace Sand will not be used for support of foundations of Category I structures. Therefore, shear wave velocity measurements in the CCNPP site soils above El. +44 ft. regardless of value, are excluded from this evaluation as they lie above the basemat. Results from the above Figure indicate that:

1. The CCNPP Unit 3 Average Profile is bounded by the 10 generic profiles used for the U.S. EPR.

2. The CCNPP Unit 3 Average Profile offers a shear wave velocity at the bottom of the basemat (approx. El. +44 ft (or depth = 0 in Figure 2.5-102)) of 1,450 ft/sec.

3. The minimum shear wave velocity from the CCNPP Unit 3 Average Profile is 1,130 ft/sec.

4. The characteristic shear wave velocity of the soil column (weighted with respect to the 344 ft soil column) is 1,510 ft/sec.

On the basis of the above, the idealized CCNPP Unit 3 site shear wave velocity profile is bounded by the 10 generic soil profiles used for the U.S. EPR and meets the minimum 1,000 ft/sec criterion identified in the U.S. EPR FSAR.

GMRS was conducted in accordance with the performance-based approach described in Regulatory Position 5 of Regulatory Guide 1.208 (NRC, 2007a).

The GMRS was developed starting from the 10^{-4} and 10^{-5} rock Uniform Hazard Spectra. At high frequencies, the appropriate (10^{-4} or 10^{-5}) HF mean amplification factor was applied to the 10^{-4} and 10^{-5} rock spectrum, to calculate site spectral amplitudes for 10^{-4} and 10^{-5} annual frequencies of exceedance. At low frequencies, a similar technique was used with the LF mean amplification factors. At intermediate frequencies the larger of the HF and LF site spectral amplitudes was used.

Figure 2.5-86 illustrates the resulting site spectra. At high frequencies the HF spectral amplitudes are always greater, and at low frequencies the LF spectral amplitudes are always greater. The two sets of spectral amplitudes cross at 2-3 Hz.

This procedure corresponds to Approach 2A in NUREG/CR-6728 (NRC, 2001) and NUREG/CR-6769 (NRC, 2002b), wherein the rock Uniform Hazard Spectra (for example, at 10^{-4}) is multiplied by a mean amplification factor at each frequency to estimate the 10^{-4} site Uniform Hazard Spectra. Note that the amplification factors plotted in Figure 2.5-78, Figure 2.5-80, Figure 2.5-82, and Figure 2.5-84 are mean logarithmic amplification factors, which correspond approximately to the median. The amplification factors used to prepare Figure 2.5-86 are arithmetic mean amplification factors, which are slightly higher than the median.

The low-frequency character of the spectra in Figure 2.5-86 reflects the low-frequency amplification of the site, as shown in the amplification factors of Figure 2.5-78, Figure 2.5-80, Figure 2.5-82, and Figure 2.5-84. That is, there is a fundamental site resonance at about 0.22 Hz, with a dip in site response at about 0.4 Hz, and this dip occurs for all 60 of the site profiles that were used to characterize the site profile. As a result, there is a dip in the site spectra for 10^{-4} and 10^{-5} at 0.4 Hz that reflects the site characteristics.

The ASCE (ASCE, 2005) performance-based approach was used to derive a GMRS from the 10^{-4} and 10^{-5} site spectra. The spectrum is derived at each structural frequency as follows:

$$A_R = SA(10^{-5})/SA(10^{-4})$$

$$DF = 0.6\ A_R^{\sout{0.8}}$$

$$GMRS = max(SA(10^{-4}) \times max(1.0, DF), 0.45 \times SA(10^{-5}))$$

The last term in the above equation was not published in this form in ASCE (ASCE, 2005) but is a supplemental modified form, as presented in NRC Regulatory Guide 1.208 (NRC, 2007a). The resulting horizontal spectrum is plotted in Figure 2.5-87. This spectrum has been smoothed slightly, particularly around 1.5 Hz, to remove slight bumps and dips in the spectrum resulting from the site amplification calculations that are not statistically significant. The average change in spectral amplitudes for the 5 frequencies that were smoothed was an increase of 1%, which is not significant.

A vertical spectrum was calculated by deriving vertical-to-horizontal (V:H) ratios and applying them to the horizontal spectrum. As background and for comparison purposes, V:H ratios were obtained by the following methods:

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1. Rock V:H ratios for the central and eastern United States (CEUS) were calculated from NUREG-6728 (NRC, 2001), using the recommended ratios for PGA < 0.2g, which applies at this site (see Figure 2.5-88).

2. Soil V:H ratios for the western United States (WUS) were calculated from two publications (Abrahamson, 1997) (Campbell, 1997) that have equations estimating both horizontal and vertical motions on soil. Horizontal and vertical motions were predicted from these two references for M = 5.5 and R = 9 mi (15 km). M = 5.5 was selected because earthquakes around this magnitude dominate the high frequency motions, and R = 9 mi (15 km) was selected because this distance resulted in a horitontal PGA of approximately 0.1 g at the site, which is close to the PGA associated with the horizontal SSE. For each reference, the V:H ratio was formed, and the average ratio (average from the two references) was then calculated.

3. The WUS V/H ratios for soil were modified in an approximate way for CEUS conditions by shifting the frequency axis of the V:H ratios so that they more closely resemble what might be expected at a soil site. This shifted the WUS peak V/H ratio from about 15 Hz to about 45 Hz.

Figure 2.5-88 shows these three V:H ratios plotted vs. structural frequency. As a conservative choice, the envelope V/H ratio shown as a thick dashed line was selected because this envelops all three approaches. The recommended V:H ratio is 1.0 for frequencies greater than 25 Hz, 0.75 for frequencies less than 5 Hz, and is interpolated (log-linear) between 5 and 25 Hz. Figure 2.5-87 plots the resulting vertical spectrum, calculated in this manner from the horizontal spectrum. Table 2.5-22 lists the horizontal and vertical GMRS amplitudes.

2.5.2.6.2 CCNPP3 Seismic Site Characteristics Reconciliation

The CCNPP3 Site Seismic Characteristics are reconciled with the U.S. EPR FSAR generic seismic analysis input and output thus assuring that the generic design of the U.S. EPR Nuclear Island (NI), Emergency Power Generation Building (EPGB), and the Essential Service Water Building (ESWB) bounds the CCNPP3 site requirements for these structures and the associated equipment. This reconciliation follows the nine-step methodology and guidelines defined in U.S. EPR FSAR Section 2.5.2.6. The overall conclusion of the reconciliation is that the CCNPP3 Site Seismic Characteristics are well bounded by the U.S. EPR FSAR generic analyses and resulting design.

The U.S. EPR FSAR states:

"A COL applicant that references the U.S. EPR design certification will compare the final site-specific soil characteristics with the U.S. EPR design generic soil parameters and verify that the site-specific seismic characteristics are enveloped by the CSDRS (anchored at 0.3g PGA) and the 10 generic soil profiles discussed in Section 2.5.2 and Section 3.7.1 and summarized in Table 3.7.1-6. The applicant develops site-specific ground motion response spectra (GMRS) and foundation input response spectra (FIRS). The applicant will also describe site-specific soil conditions and evaluate the acceptability of the U.S. EPR standard design described in Section 3.7.1 for the particular site. In making this comparison, the applicant will refer to Sections 3.7.1 and 3.7.2 for a description of the soil-structure interaction analyses performed for the U.S. EPR in addressing the following evaluation guidelines."

This COL Item is addressed as follows:

The reconciliation of the CCNPP3 Seismic Site Characteristics consists of two parts:

- A comparison of the CCNPP3 seismic analysis inputs to those used for the U.S. EPR generic design and,
- A comparison of the CCNPP3 site-specific confirmatory seismic analysis results to the U.S. EPR FSAR generic analysis results.

Summaries of these comparisons are presented below. Then, subsections 1 through 9 discuss each of the nine reconciliation steps included in the U.S. EPR FSAR guidelines. Table 2.5-75 highlights the primary CCNPP3 responses to each of the nine steps. The nine-step reconciliation sections include appropriate references to various supporting tables and figures contained in this and other sections of the COLA and the U.S. EPR FSAR.

Summary of the Comparison of Seismic Analysis Inputs

The key site characteristics used as input to the seismic analysis are the GMRS and the shear wave velocity (SWV) profiles. The most significant input is the GMRS.

The U.S. EPR FSAR design is based on the EUR spectra with a Zero Period Acceleration (ZPA) of 0.3g and a peak spectral acceleration of 0.9g (Figure 3.7-5). The corresponding values for the CCNPP3 site, as determined using the performance-based approach described in Section 2.5.2.6.1, are 0.076g and 0.18g (Table 2.5-22). The CCNPP3 Safe Shutdown Earthquake (SSE) has been defined as response spectra with a zero period acceleration of 0.15g and a peak spectral acceleration of 0.45g (Figure 3.7-1). The shape of the CCNPP3 SSE is an envelope of the shapes defined by Regulatory Guide 1.60 and the EUR spectra. 10CFR50, Appendix S requires an "appropriate" SSE spectra shape with a ZPA of at least 0.10g. Therefore:

- The defined CCNPP3 SSE exceeds the Appendix S requirement by 50% and,
- The U.S. EPR FSAR exceeds this defined SSE by a factor of two.

The reason the site SSE was developed in this manner is to assure that the analysis and design of the site-specific buildings and equipment are performed in a conservative manner. For simplicity and conservatism the site SSE which bounds the FIRS for the NI, EPGB, and ESWB, is also used for the confirmatory analysis comparison to the U.S. EPR FSAR generic design.

The U.S. EPR FSAR generic design is based on a broad range of SWV profiles with a minimum value of 700 feet per second and a maximum value of 13,123 feet per second. The U.S. EPR FSAR also analyzes cases with shear wave velocities that vary by depth. The U.S. EPR FSAR seismic analysis results show that the design of the U.S. EPR is generally controlled by the maximum (13,123 feet per second) SWV analysis. As discussed in the reconciliation below, the CCNPP3 SWV varies by structure and with depth and is within, or less than, the low end of the range of SWV profiles used by the U.S. EPR FSAR for the generic design of the plant. Foundation Input Response Spectra (FIRS) have been developed using the CCNPP3 SWV profiles. These FIRS are shown to be bounded by the CCNPP3 site SSE.

Summary of the Comparison of Seismic Analysis Results

CCNPP3 confirmatory seismic analyses were performed, as described in Section 3.7, for the NI, EPGB, and ESWB. The confirmatory analyses inputs consist of the CCNPP3 defined SSE response spectra and associated strain-compatible site-specific SWV profiles. For the NI confirmatory analysis, the NI is modeled as a surface mounted structure and uses the SWV profile without backfill. As described in Section 3.8.4.6.1, the placement of a sand layer and backfill is being used to accommodate the waterproofing system provided to protect the reinforced concrete NI common basemat. Supporting NI FIRS have been developed for an embedded NI using a SWV profile that includes backfill. These FIRS are shown to be enveloped by the site SSE.

The purpose of the U.S. EPR FSAR seismic analyses is to obtain seismic results to develop the generic design of these buildings and associated equipment. This generic design is used for the CCNPP3 site. The purpose of the CCNPP3 confirmatory seismic analyses is to confirm that the seismic results used for the generic design of the U.S. EPR bound the CCNPP3 requirements. The CCNPP3 confirmatory seismic analysis results are not used for design.

The results of the CCNPP3 confirmatory analyses are presented in Section 3.7. Figures 3.7-25 through 3.7-51 show comparisons of the U.S. EPR FSAR design In-Structure Response Spectra (ISRS) with the results of the CCNPP3 confirmatory seismic analysis for the NI. Figures 3.7-54 through 3.7-72 provide the same comparisons for the EPGB and the ESWB. In all cases, except for EPGB and ESWB accelerations in the very low frequency range (0.3 Hz and below), the U.S. EPR design ISRS exceed the CCNPP3 confirmatory analysis results by a large margin. This large margin is quantified in Reconciliation Step 8 and an assessment of the acceleration results below 0.3 Hz is presented in Reconciliation Step 9.

The U.S. EPR FSAR nine-step reconciliation process is presented below in a standard format consisting of the quote from the U.S. EPR FSAR step statement followed by the CCNPP3 response to this statement.

1. **Reconciliation Step 1**

   U. S. EPR FSAR Statement: The applicant will confirm that the peak ground acceleration (PGA) for the GMRS is less than 0.3g.

   CCNPP3 Response: The PGA for the CCNPP3 GMRS is 0.076g. However, a site SSE with a PGA of 0.15g has been defined for CCNPP3 and, the site SSE is used as the input to the CCNPP3 confirmatory analysis. A discussion of the development of this input is included in Section 3.7.1.1.1.1 for the NI and Section 3.7.1.1.1.2 for the EPGB and ESWB.

2. **Reconciliation Step 2**

   U. S. EPR FSAR Statement: The applicant will confirm that the low-strain, best-estimate, value of SWV at the bottom of the foundation basemat of the NI Common Basemat Structures and other Seismic Category I structures is 1000 fps, or greater. This comparison will confirm that the NI Common Basemat Structures and other Seismic Category I structures are founded on competent material.

   CCNPP3 Response: The CCNPP3 low-strain best-estimate SWV profile for the NI, EPGB, and ESWB are discussed in Section 2.5.2.6.1 and are reconciled to the 1,000 fps requirement. However, backfill is used below each of these structures and this backfill is expected to have a SWV of less than 1,000 fps.

CCNPP3 has identified a departure related to the SWV of the backfill. The in-situ material after backfill placement below these buildings meets the 1,000 fps SWV.

For the backfill, the comparison of the site characteristics to a SWV of 1,000 fps is not the only method used to assure that the structures are founded on competent material. Industry has found that competent backfill is not necessarily expected to meet a SWV of 1000 fps. To assure competent material is used for backfill, CCNPP3 has performed a backfill investigation as described in Section 2.5.4.2.3. This investigation has resulted in assuring that a competent source of backfill is available and the properties of this competent backfill can be defined. The impact of the backfill properties (SWV) on the seismic analysis is evaluated as described in the following subsections and the SWV of less than 1,000 fps is determined to be acceptable.

3. **Reconciliation Step 3**

U. S. EPR FSAR Statement: The applicant will demonstrate that the FIRS for the NI Common Basemat Structures is enveloped by the CSDRS. In addition, the applicant will demonstrate that the input motion, which considers the difference in elevation between each structure and the NI Common Basemat Structures, the embedment of the ESWB, and SSSI effect of the NI Common Basemat Structures is less than the modified CSDRS used for the design of the EPGB and the ESWB (see Section 3.7.1.1.1).

CCNPP3 Response: Figures 3.7-2 and Figure 3.7-3 show comparisons of the FIRS to the site SSE for the NI Common Basemat Structures, without considering the NI backfill. Figure 2.5-241 shows a comparison of the CCNPP3 FIRS to the site SSE for the NI Common Basemat Structures, considering backfill. These figures show that the FIRS are bounded by the site SSE as well as the Regulatory Guide 1.60 spectra. Figure 3.7-6 shows a comparison of the site SSE, which is used as input to the confirmatory analysis, with the CSDRS. For most frequencies, the SSE is bounded by the CSDRS by a factor of 2.

Two sets of NI FIRS are shown on Figure 2.5-241 to account for the varying depth of backfill between the planned excavation and the varying bottom contour of the NI basemat. The two sets of strain-compatible SWV's associated with the CCNPP3 SSE response spectra are shown in Figures 2.5-242 and 2.5-243 for the upper 200 feet of the soil. Tables 2.5-76 and 2.5-77 show the strain-compatible values for the entire soil depth.

Figure 3F-27 shows a comparison of the EPGB and ESWB CCNPP3 FIRS and the site SSE as well as the Regulatory Guide 1.60 spectra. The FIRS are bounded by the site SSE. Figure 3.7-6 shows a comparison of the CCNPP3 site SSE to the unmodified CSDRS. For most frequencies, the site SSE is bounded by the unmodified CSDRS by a factor of 2. The modified CSDRS used for the design of the EPGB and ESWB is greater than the unmodified CSDRS. The modified CSDRS are shown in U.S. EPR FSAR Figures 3.7.1-33 and 3.7.1-34. The ZPA for the modified CSDRS is 26% greater and the peak spectral acceleration is 33% greater than the unmodified CSDRS.

4. **Reconciliation Step 4**

U. S. EPR FSAR Statement: The applicant will demonstrate that the site-specific profile is laterally uniform by confirming that individual layers with the profile have an angle of dip no greater than 20 degrees.

CCNPP3 Response: As discussed in Section 2.5.4.10.3, Item 2, the CCNPP3 individual layers dip up to about 10 degrees.

In addition, Sections 2.5.4.2.2.2 and 2.5.4.10.3 summarizes the results of extensive geotechnical studies and field surveys of the CCNPP3 site that have been performed to confirm that soil layers are laterally uniform.

5. **Reconciliation Step 5**

U. S. EPR FSAR Statement: The applicant will compare the final site-specific soil characteristics including backfill with the U.S. EPR design generic soil parameters and demonstrate that the idealized strain-compatible site soil profile is similar to or bounded by the 10 generic soil profiles used for the U.S. EPR. The 10 generic profiles include a range of uniform and layered site conditions. The applicant also considers the assumptions used in the SSI analyses including backfill, as described in Section 3.7.1 and Section 3.7.2. Site soil properties of soil columns beneath Category I structures must be bounded by design soil properties listed in Tables 3.7.1-6 and 3.7.2-9. The soil column beneath the embedded NI Common Basemat and the soil column, starting at grade, for the EPGB and ESWB must meet this requirement.

CCNPP3 Response: The comparison between site soil properties and design soil properties is performed in two steps. First, the SWV site profile is compared to the design SWV's, and then the influence of unit weight is evaluated.

As far as the NI SWV site profile is concerned, a departure has been identified because the backfill portion of the Best Estimate SWV profile is less than the minimum analyzed in the U.S. EPR FSAR (700 fps). The CCNPP3 data included in Tables 2.5-76 and 2.5-77 results in weighted average backfill SWV's of

620 fps and 688 fps. This departure can be justified for the following reasons.

The departure addresses a SWV that is on average less than 12% lower than the minimum used in the U.S. EPR FSAR (700 fps).

The strain-compatible SWV's decrease from the low-strain values as the seismic demand increases. The CCNPP3 values of 620 fps and 688 fps are associated with the site-specific SSE which is used in the confirmatory analyses. Considering the CCNPP3 site-specific FIRS rather than the SSE, the Best Estimate strain-compatible SWV values would be equal to or larger than the minimum SWV value considered in the U.S. EPR FSAR. Refer to Figures 2.5-244 and 2.5-245. This means that the departure is a result of the use of a conservative SSE input to the confirmatory analyses. These two facts demonstrate that the CCNPP3 site characteristics are very close to the generic design conditions.

For the EPGB and ESWB, the CCNPP3 Best Estimate, Lower Bound, Upper Bound SWV profiles are included in Tables 3F-3, 3F-4, and 3F-5. Similar to the NI, these tables show a departure from the U.S. EPR FSAR minimum SWV of 700 fps.

In order to quantify the impact of these departures, two approaches are taken.

For the EPGB and ESWB, the confirmatory analysis was performed with CCNPP3 values reflecting the backfill. The CCNPP3 SWV profiles are in the low end of the range of SWV's analyzed in the U.S. EPR FSAR. The results of these analyses are presented in Section 3.7 and compared with the U.S. EPR FSAR results. As discussed in Reconciliation Step 8 below, the comparison shows that the CCNPP3 ISRS are well bounded.

For the NI, because the backfill was introduced after the completion of the confirmatory analysis, a different approach is used. This approach compares the FIRS with and without backfill. The data for this comparison are shown on Figure 2.5-241 and Figures 3.7-2 and 3.7-3. The effect of the backfill is to increase the ZPA and peak spectral accelerations of the FIRS by 11% and 16% respectively. The NI FIRS with backfill remain bounded by the site SSE which is the basis for the confirmatory analysis.

Another reason which makes the departure acceptable is that the departure is associated with low, not high SWV's. Figure 3.7-20 shows a comparison of the NI Lower Bound, Best Estimate, and Upper Bound CCNPP3 SWV profiles without backfill being considered. Tables 3.7-2, 3.7-3, and 3.7-4 provide

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the associated data. This figure also shows the U.S. EPR FSAR SWV profiles. As can be seen from the figure, even without considering the backfill, the NI SWV profile is in the low end of the range of SWV's analyzed in the U.S. EPR FSAR. When considering backfill, the SWV profile is even lower. This is not critical because hard rock SWV profiles, not low SWV profiles, generally control the design of the U.S. EPR.

The basis for stating that the hard rock or U.S. EPR SWV of 13,123 fps generally controls the generic design is contained in U.S. EPR FSAR Tables 3.7.2-10 through 3.7.2-17 for the NI, Table 3.7.2-27 for the EPGB, and Table 3.7.2-28 for the ESWB. These tables list the ZPA values for each of the SWV's analyzed. The ZPA's are provided at various elevations for each of the buildings. As an example, Figure 2.5-246 shows a plot of the Containment Building horizontal ZPA's in the x-direction at each elevation for three of the SWV's analyzed in the U.S. EPR FSAR. Since the design of the structure and the development of the ISRS are based on these ZPA's, it can be seen that the seismic analysis results from the SWV of 13,123 fps generally controls the generic design. For comparison purposes, the figure also includes the ZPA's resulting from the CCNPP3 confirmatory analysis which is based on the site SSE input and a strain-compatible SWV profile without backfill.

Based on the logic that the high SWV's generally control the generic design, the low values that are the basis for the departure do not impact the conclusion that the U.S. EPR FSAR seismic response bounds the CCNPP3 site-specific response. This conclusion has been confirmed by the results of the CCNPP3 confirmatory analysis which are discussed in Reconciliation Step 8 below.

The overall conclusion is that the CCNPP3 SWV's profile is similar to and bounded by the 10 generic soil profiles used for the U.S. EPR. The CCNPP3 SWV profile leads to seismic analysis results which are bounded by the results from the U.S. EPR FSAR range of profiles because high rather than low SWV profiles generally control the generic design of the U. S. EPR.

The departure has also been written to address the fact that the U.S. EPR FSAR seismic analyses are based on a soft soil unit weight of 110 pcf. The CCNPP3 unit weight for the in-situ soil in the NI, EPGB, and ESWB area ranges from 105 pcf to 125 pcf. The unit weight of the backfill is 145 pcf partially a result of the high compaction requirements. The confirmatory analysis for the EPGB and ESWB and the development of the FIRS for the NI used the site-specific unit weights. Therefore, the influence of this departure has been taken into account in the supporting analyses.

6. **Reconciliation Step 6**

U. S. EPR FSAR Statement: If the conditions of steps one through five are met, the characteristics of the site fall within the site parameters for the U.S. EPR and the site is acceptable.

CCNPP3 Response: The conditions of steps one through five have been met or the departures have been shown to be acceptable because:

- The primary input influencing the seismic analysis results is the earthquake magnitude as defined by the GMRS and, the regulatory required earthquake for the CCNPP3 site is one-third of that used for the generic design of the U.S. EPR.
- The secondary input, the CCNPP3 SWV values are similar to the U.S. EPR FSAR values and the SWV profiles are relatively low when compared to the range of SWV profiles used for the generic design of the U.S. EPR. In general, the high SWV profiles (a rock site) control the design of the U.S. EPR.
- The FIRS which include the influence of the CCNPP3 SWV profiles are bounded by the defined site SSE.

However to conservatively confirm the above assessment, CCNPP3 site-specific confirmatory seismic analyses have been performed. The results of these analyses are discussed in the following sections.

7. **Reconciliation Step 7**

U. S. EPR FSAR Statement: If the conditions of steps one through five are not met, the applicant will demonstrate by other appropriate means that the U.S. EPR is acceptable at the proposed site. The applicant may perform intermediate-level additional studies to demonstrate that the particular site is bounded by the design of the U.S. EPR. An example of such studies is to show that the site-specific motion at top-of-basemat level, with consideration of the range of structural frequencies involved, is bounded by the U.S. EPR design.

CCNPP3 Response: The CCNPP3 confirmatory analysis seismic modeling and methodology are consistent with the modeling and methodology described in the U.S. EPR FSAR. Therefore, the confirmatory analyses are "detailed site-specific SSI analyses" as defined in Step 8.

The development of the NI FIRS to assess the impact of the backfill layers on the NI confirmatory analysis is an "intermediate study" which supports the conclusion that the FIRS are enveloped by the CCNPP3 SSE and the confirmatory analysis remains conservative. Refer to Reconciliation Step 5.

Future changes, in particular those anticipated in Revision 3 of the U.S. EPR FSAR, may require further reconciliation of the CCNPP3 Site Seismic Characteristics. Some of these changes are expected to affect the specific modeling and methodology used in the U.S. EPR FSAR without changing the broad features of the generic analysis and design. Considering the expected limited impact, these changes could be reconciled through the use of the CCNPP3 confirmatory analyses and other "intermediate level" studies in accordance with this Step 7.

8. **Reconciliation Step 8**

U. S. EPR FSAR Statement: If the evaluations of step 7 are not sufficient, the applicant will perform detailed site-specific SSI analyses for the particular site. This site-specific evaluation will include dynamic seismic analyses and development of ISRS for comparison with ISRS for the U.S. EPR. These analyses will be performed in accordance with the methodologies described in Section 3.7.1 and Section 3.7.2. Results from this comparison will be acceptable if the amplitude of the site-specific ISRS do not exceed the ISRS for the U.S. EPR by greater than 10 percent on a location-by-location basis. Comparisons will be made at the following key locations, defined in Section 3.7.2:

(For brevity, the defined Locations A though G contained in U.S. EPR FSAR Section 2.5.2.6 are not repeated here.)

CCNPP3 Response: CCNPP3 site-specific confirmatory analyses have been performed. These confirmatory analyses are performed in accordance with the methodologies described in U.S. EPR FSAR Section 3.7.1 and Section 3.7.2.

ISRS are developed for the Lower Bound, Best Estimate, and Upper Bound SWV profiles shown in Tables 3.7-2, 3.7-3, and 3.7-4 for the NI and Tables 3F-3, 3F-4, and 3F-5 for the EPGB and ESWB. The resulting CCNPP3 ISRS are compared to the ISRS for the U.S. EPR in Figures 3.7-25 through 3.7-51 for the NI and Figures 3.7-64 through 3.7-72 for the EPGB and ESWB.

The comparison figures show:

- For the designated locations of the NI, the U.S. EPR FSAR ISRS bound the CCNPP3 results. The margin between the generic design and confirmatory analysis results is large in the range of frequencies affecting the design of structures and equipment. The multiplication factor between the peak spectral acceleration of the U.S. EPR FSAR ISRS and the CCNPP3 confirmatory analysis ISRS for the same building location ranges from a minimum of 1.99 to a maximum of 5.45. The corresponding ZPA factor ranges from 2.06 to 3.78.

- For the designated locations of the EPGB and ESWB, the U.S. EPR FSAR ISRS bound the CCNPP3 results above a frequency of 0.3Hz. The margin between the generic design and confirmatory analysis results is large in the range of frequencies affecting the design of structures and equipment. The multiplication factor between the peak spectral acceleration of the U.S. EPR FSAR ISRS and the CCNPP3 confirmatory analysis ISRS for the same building location ranges from a minimum of 2.48 to a maximum of 5.40. The corresponding ZPA factor ranges from 2.49 to 7.73.

9. **Reconciliation Step 9**

U. S. EPR FSAR Statement: Exceedances in excess of the limits discussed in step 8 will require additional evaluation to determine if safety-related structures, systems, and components of the U.S. EPR at the location(s) in question will be affected.

CCNPP3 Response: As noted in the Step 8 response, in EPGB and ESWB building locations the CCNPP3 results exceed the U.S. EPR FSAR ISRS below a frequency of 0.3 Hz. This is caused by the fact that the confirmatory analyses use the site SSE as input. And, the site SSE is conservative when compared to the FIRS or the U.S. EPR FSAR response spectra shape. This can be seen from Figure 3F-27. It is well known that structures and equipment are not affected by accelerations in this frequency range. Data supporting this fact can be obtained from the modal frequency and mass participation information contained in U.S. EPR FSAR Tables 3.7.2-1 through 3.7.2-5 for the NI. The lowest frequency affecting the response of the structure and included in the table is 3.75 Hz. The lowest frequency affecting the response of the structure and included in U.S. EPR FSAR Table 3.7.2-7 for the EPGB is 10.72 Hz. The lowest frequency affecting the response of the structure and included in U.S. EPR FSAR Table 3.7.2-8 for the ESWB is 6.67 Hz.

Sloshing associated with water storage containers could be affected by very low frequency accelerations. However, the associated maximum acceleration below the frequency of 0.3 Hz is 0.06g and the exceedance is less than 0.007g. Taking into account the cause (the CCNPP3 confirmatory analysis use of an enveloping SSE response spectrum that is more conservative than the U.S. EPR response spectra in this low frequency range) and the magnitude of the exceedances (0.007g), the exceedances have no impact on the application of the U.S. EPR FSAR generic design to the CCNPP3 site.

Based on the above reconciliation process which includes a comparison of the CCNPP3 site seismic characteristics inputs and the results of confirmatory seismic analyses with the U.S. EPR FSAR inputs and results; the CCNPP3 Seismic Site Characteristics are bounded by the U.S. EPR FSAR. Therefore, the CCNPP3 site is acceptable.

2.5.2.7 Conclusions

This section is added as a supplement to the U.S. EPR FSAR.

Calvert Cliffs 3 Nuclear Project, LLC and UniStar Nuclear Operating Services, LLC used the seismic source and ground motion models published by the Electric Power Research Institute (EPRI) for the central and eastern United States (CEUS), Seismic Hazard Methodology for the Central and Eastern United States, (EPRI, 1986). As such, FSAR Section 2.5.2 focuses on those data developed since publication of this 1986 EPRI report. Regulatory Guide 1.165, Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion, (NRC, 1997), indicates that applicants may use the seismic source interpretations developed by Lawrence Livermore National Laboratory (LLNL) in the "Eastern Seismic Hazard Characterization Update," published in 1993, or the EPRI document as inputs for a site-specific analysis.

Calvert Cliffs 3 Nuclear Project, LLC and UniStar Nuclear Operating Services, LLC also used the guidance of Regulatory Guide 1.208, A Performance–Based Approach to Define the Site-Specific Earthquake Ground Motion, (NRC, 2007a) to develop the Ground Motion Response Spectrum (GMRS).

Calvert Cliffs 3 Nuclear Project, LLC and UniStar Nuclear Operating Services, LLC has provided a characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. Calvert Cliffs 3 Nuclear Project, LLC and UniStar Nuclear Operating Services, LLC has adequately addressed the uncertainties inherent in the characterization of these seismic sources through a PSHA, and that this PSHA followed the guidance provided in Regulatory Guide 1.208 (NRC, 2007a).

The GMRS developed by UniStar Nuclear Operating Services, LLC uses the performance-based approach described in Regulatory Guide 1.208 (NRC, 2007a), adequately representing the regional and local seismic hazards and accurately includes the effects of the local CCNPP Unit 3 subsurface properties.

The performance-based approach outlined in Regulatory Guide 1.208 (NRC, 2007a) is an advancement over the solely hazard-based reference probability approach recommended in Regulatory Guide 1.165 (NRC, 1997) and it was used where appropriate in the determination of the GMRS. The performance-based approach uses not only the seismic hazard characterization of the site from the PSHA but also basic seismic fragility SSC modeling in order to obtain an SSE that directly targets a structural performance frequency value. Calvert Cliffs 3 Nuclear Project, LLC and UniStar Nuclear Operating Services, LLC conclude that the application for the CCNPP Unit 3 site is acceptable from a geologic and seismologic standpoint and meets the requirements of 10 CFR 100.23(d) (CFR, 2007). However, because the site specific SSE is smaller

Table 2.5-75— Summary of CCNPP3 Response to U.S. EPR FSAR Site Seismic Characteristics Reconciliation

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U.S. EPR FSAR Reconciliation Nine-Step Process	Guideline	CCNPP3 Response	CCNPP3 Comments
1	PGA for the GMRS less than 0.3g	PGA for the GMRS equals 0.076g	Reference COLA Section 2.5.2.6
2	Low Strain Best Estimate SWV 1,000 fps	Low Strain Best Estimate SWV: Remaining in-situ soil after backfill placement greater than 1,000 fps Backfill expected to be less than 1,000 fps	Reference COLA Part 7 Departure Reference Section 2.5.2.6.1 Purpose of Guideline to assure competent foundation material. CCNPP3 COLA Section 2.5.4.2.3 backfill investigations assure competent foundation material.
3	FIRS enveloped by CSDRS (ZPA 0.3g and Peak Spectral Acceleration of 0.9g)	NI, EPGB, and ESWB with backfill, (ZPA 0.08g and Peak Spectral Acceleration of 0.22g)	Reference Figures 2.5-242, 2.5-243, and 2.5-244, Tables 2.5-76 and 2.5-77, and COLA Appendix 3F
4	Soil layer angle of dip no greater than 20 degrees	CCNPP3 angle of dip up to 10 degrees	Reference COLA Sections 2.5.4.2.2.2 and 2.5.4.10.3
5	Strain-compatible soil profile similar to or bounded by 10 generic EPR profiles (SWV range from 700 fps to 13,123 fps)	In-situ soil profiles bounded by EPR profiles. Backfill layers are lower than minimum EPR value. Departure identified and justified - - -	Reference Figures 2.5-245, 2.5-246, and 3.7-20 and Tables 3F-3, 3F-4, and 3F-5 CCNPP3 SWV values similar to the U.S. EPR FSAR High SWV profiles generally control the generic design, refer to Figure 2.5-247 EPGB and ESWB Confirmatory Analyses performed with backfillNI FIRS developed with backfill Comparison of NI FIRS with and without backfill show relatively minor increase in acceleration NI FIRS bounded by SSE used in CCNPP3 confirmatory analysis
6	Steps 1 through 5 met, site characteristics bounded	Essentially shown to be bounded but CCNPP3 confirmatory analysis performed	-
7	If Steps 1 thru 5 not met, perform "intermediate studies"	None performed	CCNPP3 confirmatory analysis meets more stringent requirements of Step 8
8	Compare U.S. EPR In-Structure Response Spectra with site-specific spectra	CCNPP3 In-Structure Response Spectra well bounded within frequency range of interest (above 0.3 Hz)	Reference NI Figures 3.7-25 thru -51 and EPGB & ESWB Figures 3.7-64 thru -72

Table 2.5-75— Summary of CCNPP3 Response to U.S. EPR FSAR Site Seismic Characteristics Reconciliation

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U.S. EPR FSAR Reconciliation Nine-Step Process	**Guideline**	**CCNPP3 Response**	**CCNPP3 Comments**
9	Reconcile any exceedances	Exceedances below 0.3 Hz reconciled	Exceedance caused by use of SSE for the CCNPP3 confirmatory analysis. SSE developed using a conservative enveloping approach (RG 1.60 and EUR shapes)Exceedances very small

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Table 2.5-76— {CCNPP Unit 3 Soil Profiles for SSE - NI Common Basemat Structure – RB36 Soil Column}

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Layer No.	Thick [ft]	Top Depth [ft]	Unit Weight [kcf]	Lower Bound (LB)			Best Estimate (BE)			Upper Bound (UB)		
				S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]
1	3.0	0.0	0.145	549.4	1143.7	2.89	672.9	1400.7	1.97	824.1	1715.5	1.34
2	3.0	3.0	0.145	491.8	1023.8	4.92	633.9	1319.6	3.05	817.1	1700.9	1.89
3	3.5	6.0	0.145	454.1	945.2	7.51	609.7	1269.3	4.40	818.8	1704.5	2.58
4	4.0	9.5	0.145	409.7	853.0	9.32	585.8	1219.5	5.46	837.6	1743.7	3.20
5	4.0	13.5	0.145	400.7	834.2	10.41	586.6	1221.2	6.15	858.8	1787.7	3.63
6	4.5	17.5	0.145	406.0	845.1	10.97	601.0	1251.2	6.56	889.8	1852.4	3.92
7	4.0	22.0	0.145	393.4	819.0	11.00	598.3	1245.6	6.49	910.0	1894.3	3.83
8	4.0	26.0	0.145	426.8	888.4	10.84	633.5	1318.8	6.54	940.5	1957.8	3.95
9	3.0	30.0	0.145	411.7	2099.4	11.28	628.4	3204.4	6.83	959.2	4800.0	4.14
10	3.0	33.0	0.145	409.2	2086.5	11.32	629.8	3211.2	6.99	969.2	4800.0	4.32
11	3.0	36.0	0.145	401.1	2045.0	11.62	630.7	3216.1	7.12	991.9	4800.0	4.36
12	3.0	39.0	0.145	395.5	2016.6	11.78	633.7	3231.1	7.21	1015.3	4800.0	4.41
13	3.0	42.0	0.145	390.2	1989.8	11.86	629.8	3211.4	7.36	1016.5	4800.0	4.57
14	4.0	45.0	0.145	383.7	1956.5	12.00	632.8	3226.5	7.42	1043.5	4800.0	4.59
15	6.0	49.0	0.120	1016.6	4800.0	2.92	1410.6	4800.0	2.08	1957.2	6491.2	1.48
16	5.0	55.0	0.120	1229.1	4800.0	2.50	1709.5	5669.7	1.89	2377.6	7885.7	1.43
17	5.0	60.0	0.120	1226.8	4800.0	2.56	1707.8	5664.0	1.92	2377.2	7884.4	1.44
18	5.0	65.0	0.120	1224.6	4800.0	2.60	1706.1	5658.4	1.94	2376.8	7883.0	1.45
19	5.0	70.0	0.120	769.8	3925.4	3.67	1094.1	5579.1	2.55	1555.1	7929.4	1.77
20	5.0	75.0	0.120	766.9	3910.5	3.71	1091.3	5564.7	2.57	1553.0	7918.7	1.78
21	5.0	80.0	0.120	765.9	3905.1	3.74	1116.6	5693.8	2.57	1628.1	8301.8	1.76
22	5.0	85.0	0.120	1196.1	4800.0	2.68	1686.1	5592.0	1.95	2376.6	7882.4	1.42
23	5.0	90.0	0.120	1234.7	4800.0	2.60	1705.6	5656.8	1.94	2356.2	7814.5	1.45
24	5.0	95.0	0.118	1172.7	4800.0	2.51	1639.8	5438.5	1.86	2292.8	7604.4	1.38
25	5.0	100.0	0.106	1039.0	4800.0	1.62	1289.9	5421.8	1.33	1601.6	6731.6	1.09
26	5.0	105.0	0.105	1040.1	4800.0	1.46	1273.9	5354.3	1.27	1560.2	6557.7	1.10
27	7.0	110.0	0.105	1039.7	4800.0	1.46	1273.4	5352.2	1.27	1559.6	6555.1	1.10
28	8.0	117.0	0.105	1039.3	4800.0	1.48	1272.8	5350.0	1.28	1558.9	6552.4	1.10
29	10.0	125.0	0.105	1038.8	4800.0	1.49	1272.3	5347.6	1.28	1558.2	6549.4	1.10
30	10.0	135.0	0.105	1037.5	4800.0	1.43	1270.6	5340.7	1.26	1556.2	6541.0	1.11
31	10.0	145.0	0.105	1038.1	4800.0	1.49	1271.4	5344.0	1.28	1557.2	6545.1	1.10
32	10.0	155.0	0.105	1036.0	4800.0	1.53	1268.8	5333.2	1.30	1554.0	6531.8	1.11
33	10.0	165.0	0.105	1035.7	4800.0	1.53	1268.5	5331.7	1.30	1553.6	6530.0	1.11
34	10.0	175.0	0.105	1035.1	4800.0	1.54	1267.8	5328.7	1.31	1552.7	6526.3	1.11
35	10.0	185.0	0.105	1041.1	4800.0	1.60	1275.1	5359.5	1.35	1561.7	6564.0	1.14
36	10.0	195.0	0.105	1036.4	4800.0	1.59	1269.4	5335.4	1.35	1554.6	6534.5	1.15
37	10.0	205.0	0.105	1034.4	4800.0	1.60	1266.9	5325.1	1.36	1551.7	6521.9	1.16
38	10.0	215.0	0.105	1033.9	4800.0	1.62	1266.3	5322.5	1.37	1550.9	6518.7	1.16
39	10.0	225.0	0.105	1033.5	4800.0	1.64	1265.8	5320.2	1.38	1550.2	6515.9	1.16
40	10.0	235.0	0.105	1035.6	4800.0	1.58	1268.4	5331.3	1.36	1553.5	6529.4	1.17
41	8.0	245.0	0.105	1032.4	4800.0	1.63	1264.4	5314.5	1.37	1548.6	6508.9	1.15
42	7.0	253.0	0.105	1031.8	4800.0	1.64	1263.7	5311.5	1.36	1547.7	6505.2	1.13
43	5.0	260.0	0.105	1033.9	4800.0	1.62	1266.3	5322.5	1.35	1550.9	6518.7	1.13

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Table 2.5-76— {CCNPP Unit 3 Soil Profiles for SSE - NI Common Basemat Structure – RB36 Soil Column}

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Layer No.	Thick [ft]	Top Depth [ft]	Unit Weight [kcf]	Lower Bound (LB)			Best Estimate (BE)			Upper Bound (UB)		
				S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]
44	5.0	265.0	0.106	1060.5	4800.0	1.74	1298.9	5459.4	1.39	1590.8	6686.3	1.11
45	5.0	270.0	0.107	1077.9	4800.0	1.80	1320.2	5549.0	1.41	1616.9	6796.1	1.11
46	5.0	275.0	0.110	1101.0	4800.0	2.08	1348.4	5667.5	1.53	1651.4	6941.3	1.12
47	5.0	280.0	0.114	1167.7	4908.1	2.37	1430.2	6011.2	1.68	1751.6	7362.2	1.19
48	5.0	285.0	0.118	1215.0	4800.0	2.53	1553.1	5151.2	1.84	1985.5	6585.0	1.34
49	5.0	290.0	0.120	1296.4	4800.0	2.57	1633.2	5416.7	1.92	2057.5	6824.1	1.43
50	5.0	295.0	0.122	1320.8	4800.0	2.66	1667.6	5530.7	2.00	2105.4	6982.9	1.51
51	5.0	300.0	0.123	1462.9	4851.7	2.50	1827.5	6061.0	1.99	2282.9	7571.7	1.58
52	5.0	305.0	0.124	1584.6	4800.0	2.35	1975.6	5319.6	1.95	2463.2	6632.3	1.62
53	5.0	310.0	0.125	1636.6	4800.0	2.41	2004.4	5397.1	2.01	2454.9	6610.0	1.68
54	5.0	315.0	0.125	1642.9	4800.0	2.38	2012.1	5741.6	2.02	2464.3	7032.0	1.72
55	5.0	320.0	0.125	1635.5	4800.0	2.36	2003.1	5716.0	2.03	2453.3	7000.7	1.75
56	5.0	325.0	0.125	1648.5	4800.0	2.38	2019.0	5761.2	2.02	2472.7	7056.0	1.72
57	5.0	330.0	0.125	1643.6	4800.0	2.40	2013.0	5744.3	2.04	2465.4	7035.3	1.74
58	5.0	335.0	0.125	1617.7	4800.0	2.43	1981.3	5653.8	2.07	2426.6	6924.5	1.76
59	5.0	340.0	0.125	1607.1	4800.0	2.45	1968.2	5616.5	2.08	2410.6	6878.8	1.76
60	5.0	345.0	0.125	1590.6	4800.0	2.49	1948.1	5559.0	2.12	2385.9	6808.4	1.81
61	5.0	350.0	0.125	1580.6	4800.0	2.54	1935.9	5524.1	2.16	2370.9	6765.6	1.84
62	5.0	355.0	0.125	1560.2	4800.0	2.58	1910.9	5837.8	2.18	2340.3	7149.8	1.84
63	5.0	360.0	0.125	1530.9	4800.0	2.61	1875.0	5728.2	2.20	2296.4	7015.6	1.85
64	5.0	365.0	0.125	1519.2	4800.0	2.63	1860.6	5684.2	2.23	2278.8	6961.7	1.89
65	6.0	370.0	0.125	1510.2	4800.0	2.65	1849.7	5650.8	2.26	2265.4	6920.8	1.93
66	6.0	376.0	0.125	1511.1	4800.0	2.66	1850.8	5654.2	2.27	2266.7	6924.9	1.94
67	5.0	382.0	0.124	1522.2	4800.0	2.65	1864.3	5695.6	2.25	2283.3	6975.7	1.91
68	5.0	387.0	0.124	1538.2	4800.0	2.64	1883.9	5755.5	2.24	2307.3	7049.0	1.90
69	5.0	392.0	0.122	1573.0	4805.7	2.56	1926.6	5885.7	2.19	2359.5	7208.5	1.87
70	5.0	397.0	0.120	1609.8	4918.1	2.51	1971.6	6023.4	2.17	2414.7	7377.1	1.87
71	5.0	402.0	0.119	1638.8	4800.0	2.49	2007.1	4916.5	2.15	2458.2	6021.4	1.86
72	8.0	407.0	0.117	1676.7	4800.0	2.45	2053.5	5030.1	2.11	2515.1	6160.6	1.82
73	10.0	415.0	0.116	1709.7	4800.0	2.42	2093.9	5129.0	2.11	2564.5	6281.7	1.84
74	10.0	425.0	0.115	1724.9	4800.0	2.41	2112.5	5174.6	2.09	2587.3	6337.6	1.81
75	10.0	435.0	0.115	1728.6	4800.0	2.43	2117.1	5185.9	2.09	2593.0	6351.4	1.80
76	20.0	445.0	0.115	1728.1	4800.0	2.45	2116.5	5184.2	2.10	2592.1	6349.4	1.80
77	20.0	465.0	0.115	1727.3	4800.0	2.45	2115.6	5182.0	2.10	2591.0	6346.7	1.80
78	30.0	485.0	0.115	1726.4	4800.0	2.47	2114.4	5179.3	2.11	2589.6	6343.3	1.80
79	30.0	515.0	0.115	1725.4	4800.0	2.47	2113.2	5176.1	2.12	2588.1	6339.5	1.82
80	40.0	545.0	0.115	1724.1	4800.0	2.50	2111.6	5172.3	2.14	2586.2	6334.8	1.83
81	40.0	585.0	0.115	1722.6	4800.0	2.54	2109.7	5167.8	2.17	2583.9	6329.2	1.85

Table 2.5-77— {CCNPP Unit 3 Soil Profiles for SSE - NI Common Basemat Structure – RB26 Soil Column}

(Page 1 of 2)

Layer No.	Thick [ft]	Top Depth [ft]	Unit Weight [kcf]	Lower Bound (LB)			Best Estimate (BE)			Upper Bound (UB)		
				S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]
1	3.0	0.0	0.145	583.7	1215.2	2.92	719.0	1496.7	1.96	885.6	1843.5	1.31
2	3.0	3.0	0.145	521.4	1085.4	4.83	680.6	1416.9	2.90	888.5	1849.6	1.74
3	3.5	6.0	0.145	482.5	1004.4	6.47	663.7	1381.6	3.72	912.9	1900.4	2.14
4	4.0	9.5	0.145	452.8	942.6	7.94	659.2	1372.2	4.47	959.6	1997.6	2.52
5	4.0	13.5	0.145	434.8	905.2	9.01	656.3	1366.3	5.11	990.7	2062.2	2.90
6	4.5	17.5	0.145	437.5	910.7	9.69	663.9	1382.1	5.60	1007.6	2097.5	3.24
7	4.0	22.0	0.145	436.9	909.4	10.11	660.0	1373.8	5.89	997.0	2075.4	3.43
8	4.0	26.0	0.145	459.0	955.5	10.21	688.0	1432.1	6.00	1031.1	2146.4	3.53
9	3.0	30.0	0.145	472.2	2408.0	10.33	698.2	3560.1	6.17	1032.3	4800.0	3.69
10	3.0	33.0	0.145	461.2	2351.6	10.45	702.5	3582.3	6.28	1070.2	4800.0	3.77
11	3.0	36.0	0.145	459.4	2342.3	10.63	701.1	3575.1	6.44	1070.2	4800.0	3.90
12	3.0	39.0	0.145	448.9	2289.2	10.95	698.7	3562.5	6.56	1087.3	4800.0	3.93
13	3.0	42.0	0.145	453.2	2310.8	10.92	705.2	3595.7	6.62	1097.3	4800.0	4.01
14	4.0	45.0	0.145	436.7	2226.8	11.34	696.8	3553.0	6.82	1111.8	4800.0	4.10
15	3.0	49.0	0.145	450.6	2297.6	11.30	714.8	3644.7	6.81	1133.9	4800.0	4.10
16	3.0	52.0	0.145	442.0	2253.9	11.57	710.8	3624.3	6.92	1142.9	4800.0	4.14
17	4.0	55.0	0.145	459.3	2342.2	11.60	720.7	3675.1	7.01	1130.9	4800.0	4.23
18	5.0	59.0	0.120	1228.2	4800.0	2.48	1714.4	5686.0	1.85	2393.1	7937.0	1.38
19	6.0	64.0	0.120	1225.4	4800.0	2.51	1712.4	5679.4	1.87	2392.9	7936.3	1.39
20	5.0	70.0	0.120	711.6	3628.4	3.78	1035.6	5280.4	2.56	1507.1	7684.6	1.73
21	5.0	75.0	0.120	708.7	3613.5	3.83	1032.8	5266.4	2.59	1505.2	7675.2	1.75
22	5.0	80.0	0.120	711.8	3629.3	3.89	1056.7	5388.2	2.62	1568.8	7999.5	1.76
23	5.0	85.0	0.120	1101.7	4800.0	2.72	1579.6	5239.0	2.02	2264.9	7511.7	1.50
24	5.0	90.0	0.120	1159.2	4800.0	2.65	1610.2	5340.4	2.01	2236.7	7418.3	1.52
25	5.0	95.0	0.118	1119.7	4800.0	2.55	1555.2	5157.9	1.89	2160.0	7163.9	1.40
26	5.0	100.0	0.105	997.6	4800.0	1.52	1221.8	5135.4	1.32	1496.4	6289.5	1.14
27	5.0	105.0	0.105	988.7	4800.0	1.54	1211.0	5089.9	1.32	1483.1	6233.8	1.13
28	7.0	110.0	0.105	988.2	4800.0	1.55	1210.3	5087.0	1.33	1482.3	6230.3	1.14
29	8.0	117.0	0.105	987.6	4800.0	1.56	1209.6	5084.0	1.33	1481.4	6226.6	1.14
30	10.0	125.0	0.105	987.0	4800.0	1.59	1208.8	5080.7	1.35	1480.5	6222.6	1.15
31	10.0	135.0	0.105	982.4	4800.0	1.58	1203.2	5057.3	1.35	1473.6	6193.9	1.15
32	10.0	145.0	0.105	980.0	4800.0	1.51	1200.3	5045.0	1.31	1470.0	6178.9	1.14
33	10.0	155.0	0.105	979.8	4800.0	1.55	1200.0	5043.7	1.33	1469.7	6177.3	1.14
34	10.0	165.0	0.105	979.2	4800.0	1.55	1199.3	5040.7	1.33	1468.8	6173.6	1.14
35	10.0	175.0	0.105	978.6	4800.0	1.57	1198.6	5037.7	1.34	1467.9	6169.9	1.14
36	10.0	185.0	0.105	977.3	4800.0	1.58	1196.9	5030.9	1.34	1465.9	6161.5	1.13
37	10.0	195.0	0.105	979.1	4800.0	1.67	1199.1	5040.1	1.39	1468.6	6172.8	1.16
38	10.0	205.0	0.105	982.4	4800.0	1.65	1203.2	5057.3	1.39	1473.6	6193.9	1.17
39	10.0	215.0	0.105	981.9	4800.0	1.67	1202.5	5054.5	1.40	1472.8	6190.5	1.17
40	10.0	225.0	0.105	981.3	4800.0	1.67	1201.9	5051.7	1.40	1472.0	6187.1	1.17
41	10.0	235.0	0.105	982.1	4800.0	1.68	1202.8	5055.5	1.41	1473.1	6191.6	1.18
42	8.0	245.0	0.105	979.9	4800.0	1.65	1200.2	5044.5	1.40	1469.9	6178.2	1.19
43	7.0	253.0	0.105	981.3	4800.0	1.65	1201.8	5051.6	1.39	1472.0	6186.9	1.17

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Table 2.5-77— {CCNPP Unit 3 Soil Profiles for SSE - NI Common Basemat Structure – RB26 Soil Column}

(Page 2 of 2)

Layer No.	Thick [ft]	Top Depth [ft]	Unit Weight [kcf]	Lower Bound (LB)			Best Estimate (BE)			Upper Bound (UB)		
				S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]	S-Wave Vel. [ft/sec]	P-Wave Vel. [ft/sec]	Damp. [%]
44	5.0	260.0	0.105	979.9	4800.0	1.66	1200.1	5044.3	1.40	1469.8	6178.0	1.18
45	5.0	265.0	0.105	979.6	4800.0	1.66	1199.8	5043.0	1.40	1469.5	6176.4	1.18
46	5.0	270.0	0.107	1007.8	4800.0	1.79	1234.3	5188.0	1.45	1511.7	6354.0	1.17
47	5.0	275.0	0.109	1052.7	4800.0	1.93	1289.3	5419.0	1.52	1579.0	6636.8	1.19
48	5.0	280.0	0.113	1088.1	4800.0	2.22	1363.2	5730.0	1.67	1707.9	7178.6	1.26
49	5.0	285.0	0.116	1139.5	4800.0	2.40	1444.9	4800.0	1.82	1832.2	6076.6	1.38
50	5.0	290.0	0.120	1230.7	4800.0	2.56	1578.8	5236.4	1.98	2025.5	6717.9	1.53
51	5.0	295.0	0.123	1338.1	4800.0	2.61	1716.4	5692.8	2.10	2201.8	7302.5	1.69
52	5.0	300.0	0.125	1480.7	4910.9	2.55	1836.4	6090.8	2.10	2277.6	7554.1	1.73
53	5.0	305.0	0.125	1569.9	4800.0	2.50	1961.6	5281.9	2.05	2451.2	6600.0	1.68
54	5.0	310.0	0.125	1628.2	4800.0	2.44	2029.5	5464.6	2.02	2529.6	6811.2	1.67
55	5.0	315.0	0.125	1664.3	4800.0	2.39	2038.4	5816.7	2.02	2496.5	7124.0	1.71
56	5.0	320.0	0.125	1646.4	4800.0	2.39	2016.5	5754.1	2.02	2469.6	7047.3	1.71
57	5.0	325.0	0.125	1625.0	4800.0	2.44	1990.2	5679.0	2.04	2437.4	6955.4	1.71
58	5.0	330.0	0.125	1614.0	4800.0	2.44	1976.7	5640.7	2.05	2421.0	6908.5	1.72
59	5.0	335.0	0.125	1595.4	4800.0	2.45	1953.9	5575.6	2.06	2393.0	6828.7	1.73
60	5.0	340.0	0.125	1591.3	4800.0	2.46	1948.9	5561.3	2.07	2386.9	6811.1	1.74
61	5.0	345.0	0.125	1590.8	4800.0	2.43	1948.3	5559.6	2.06	2386.2	6809.1	1.75
62	5.0	350.0	0.125	1567.4	4800.0	2.45	1919.6	5477.7	2.07	2351.0	6708.8	1.75
63	5.0	355.0	0.125	1547.6	4800.0	2.48	1895.5	5790.7	2.10	2321.5	7092.2	1.78
64	5.0	360.0	0.125	1525.9	4800.0	2.49	1868.8	5709.3	2.14	2288.8	6992.4	1.84
65	5.0	365.0	0.125	1515.1	4800.0	2.50	1855.6	5668.9	2.14	2272.6	6942.9	1.83
66	6.0	370.0	0.125	1515.4	4800.0	2.51	1856.0	5670.1	2.15	2273.1	6944.4	1.84
67	6.0	376.0	0.125	1510.9	4800.0	2.49	1850.5	5653.4	2.14	2266.4	6924.0	1.84
68	5.0	382.0	0.125	1511.4	4800.0	2.49	1851.1	5655.3	2.14	2267.2	6926.3	1.84
69	5.0	387.0	0.124	1535.4	4800.0	2.48	1880.4	5744.8	2.12	2303.1	7035.9	1.81
70	5.0	392.0	0.123	1553.0	4800.0	2.49	1902.0	5810.6	2.12	2329.4	7116.5	1.81
71	5.0	397.0	0.121	1597.7	4880.9	2.44	1956.7	5977.9	2.11	2396.5	7321.4	1.82
72	5.0	402.0	0.119	1624.6	4800.0	2.49	1989.7	4873.8	2.13	2436.9	5969.1	1.82
73	8.0	407.0	0.116	1673.1	4800.0	2.48	2049.1	5019.3	2.12	2509.6	6147.4	1.81
74	10.0	415.0	0.115	1691.5	4800.0	2.49	2071.6	5074.4	2.13	2537.2	6214.9	1.82
75	10.0	425.0	0.115	1697.9	4800.0	2.50	2079.5	5093.8	2.13	2546.9	6238.6	1.82
76	10.0	435.0	0.115	1697.5	4800.0	2.50	2079.0	5092.5	2.13	2546.3	6237.0	1.82
77	20.0	445.0	0.115	1696.9	4800.0	2.50	2078.2	5090.6	2.14	2545.3	6234.7	1.83
78	20.0	465.0	0.115	1696.1	4800.0	2.52	2077.3	5088.2	2.15	2544.1	6231.8	1.84
79	30.0	485.0	0.115	1695.1	4800.0	2.53	2076.1	5085.4	2.16	2542.7	6228.4	1.85
80	30.0	515.0	0.115	1694.1	4800.0	2.55	2074.8	5082.2	2.17	2541.1	6224.4	1.85
81	40.0	545.0	0.115	1692.9	4800.0	2.57	2073.4	5078.7	2.19	2539.4	6220.2	1.87
82	40.0	585.0	0.115	1691.6	4800.0	2.60	2071.8	5074.8	2.21	2537.4	6215.4	1.88

Figure 2.5-241— {Site SSE Comparison with Adjusted FIRS and RG 1.60 - NI Common Basemat Structures}

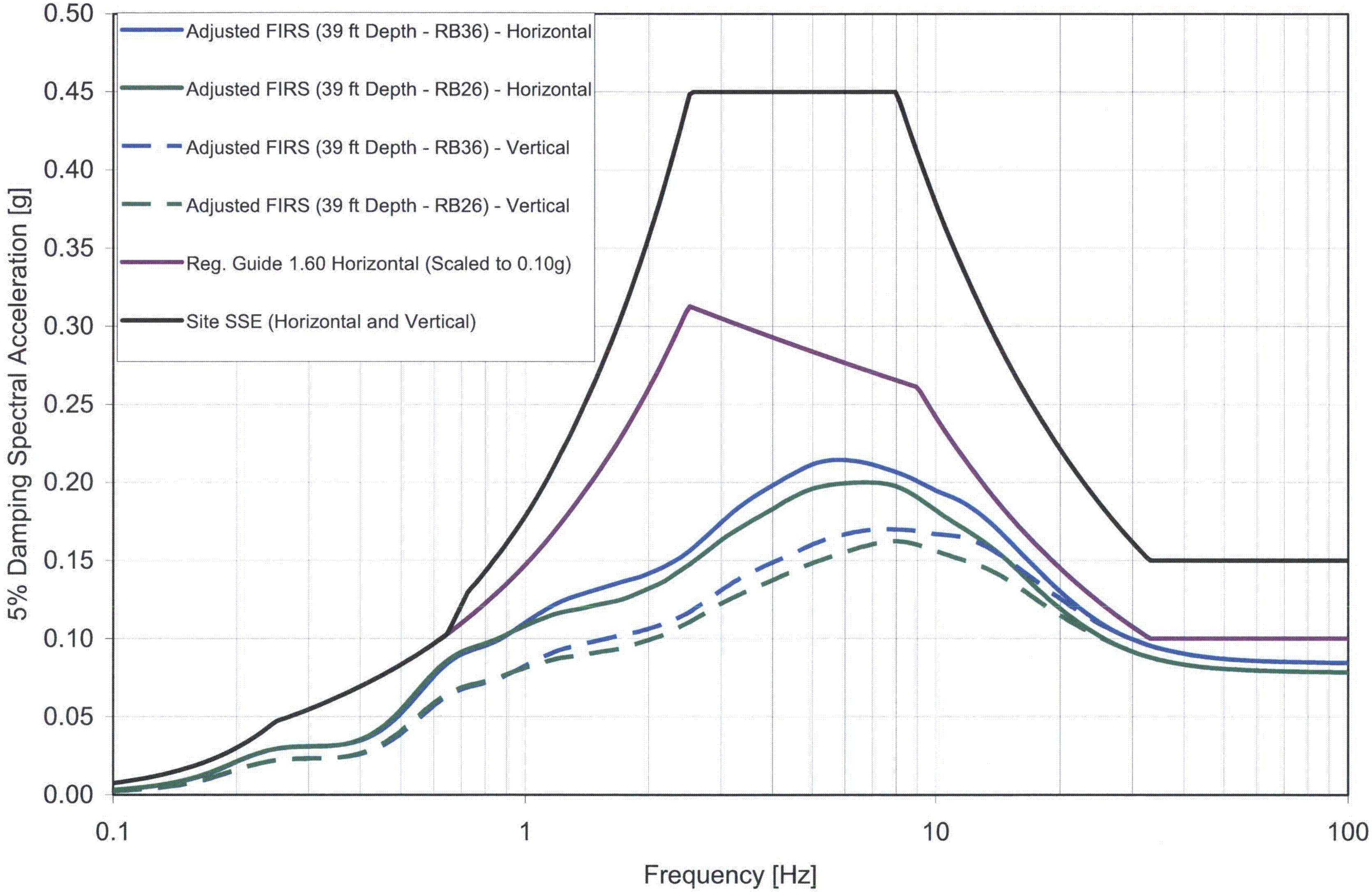

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Figure 2.5-242— {CCNPP Unit 3 Strain - Compatible Profiles for NI Common Basemat Structures - RB36 Soil Column}

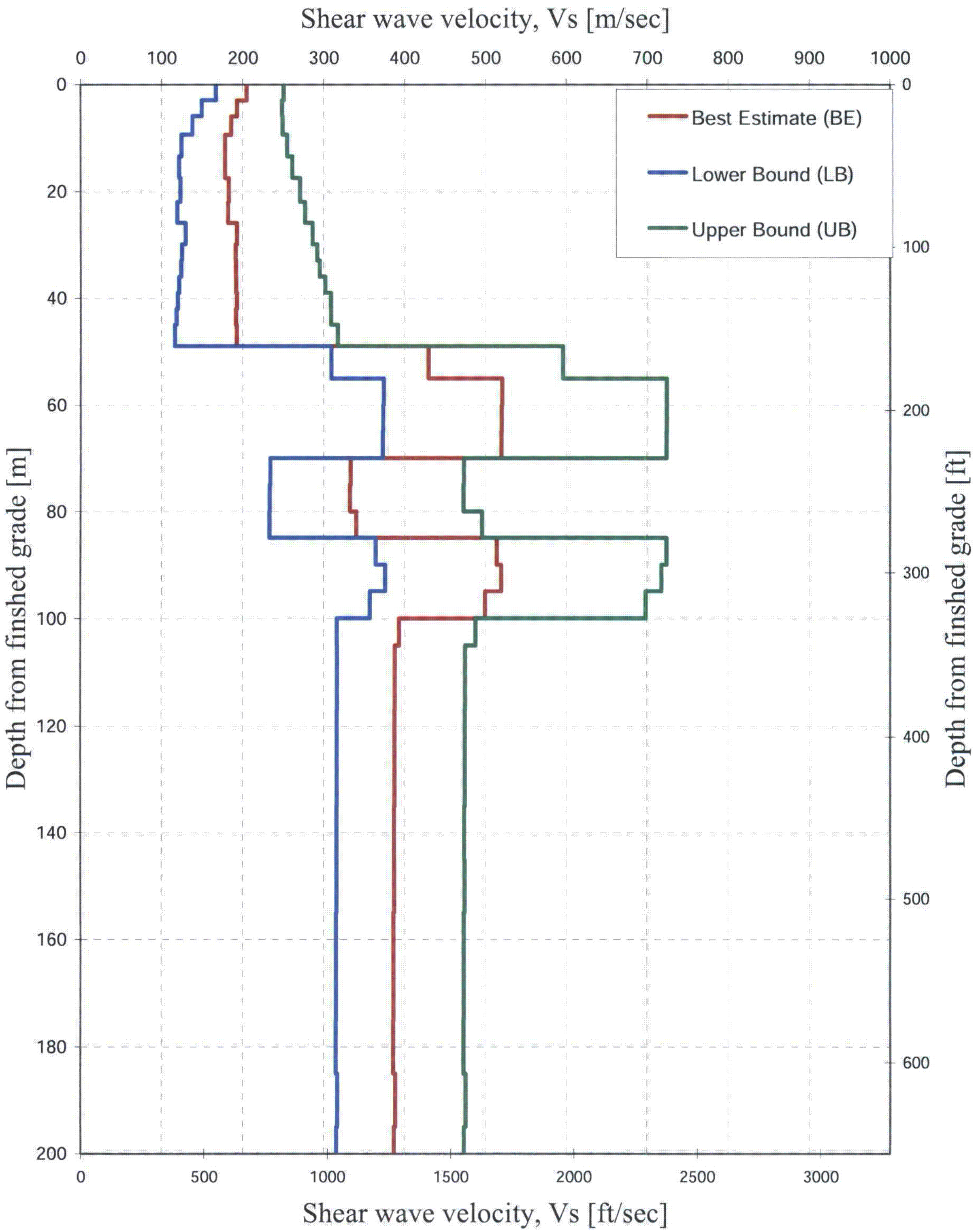

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Figure 2.5-243— {CCNPP Unit 3 Strain - Compatible Profiles for NI Common Basemat Structures - RB26 Soil Column}

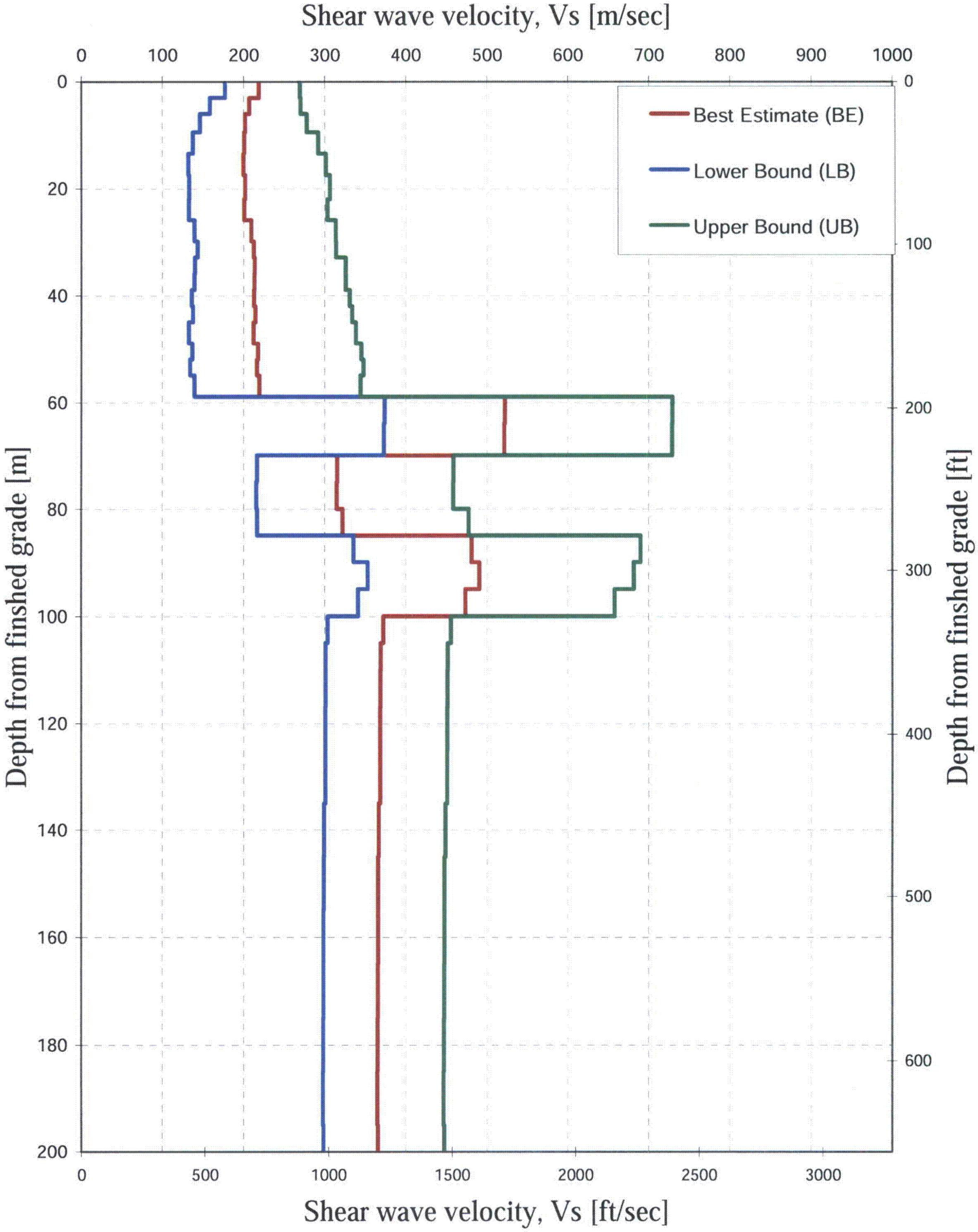

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Figure 2.5-244— {Shear-Wave Velocity Profiles Strain-Compatible with FIRS for the RB36 Soil Column}

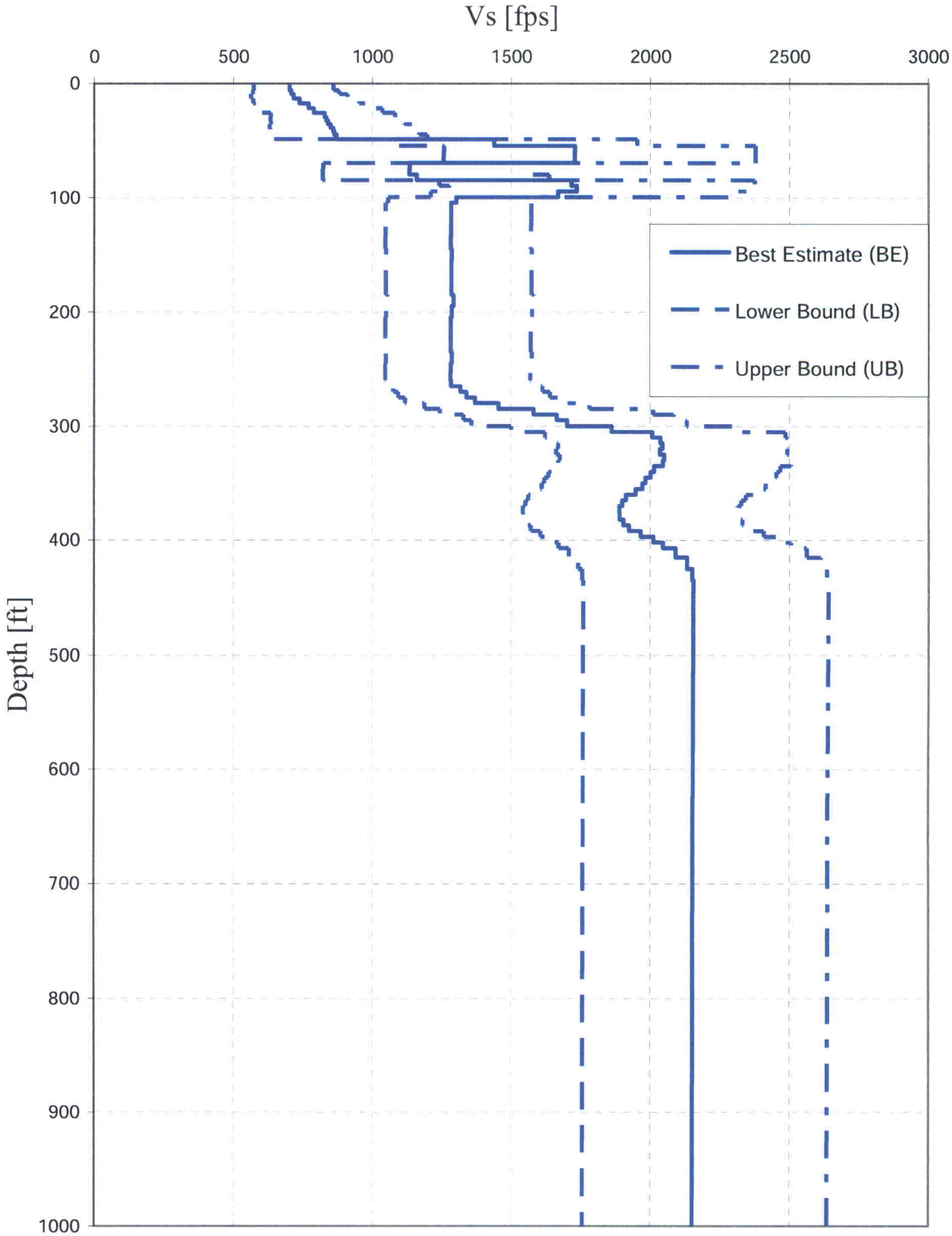


Figure 2.5-245— {Shear-Wave Velocity Profiles Strain-Compatible with FIRS for the RB26 Soil Column}

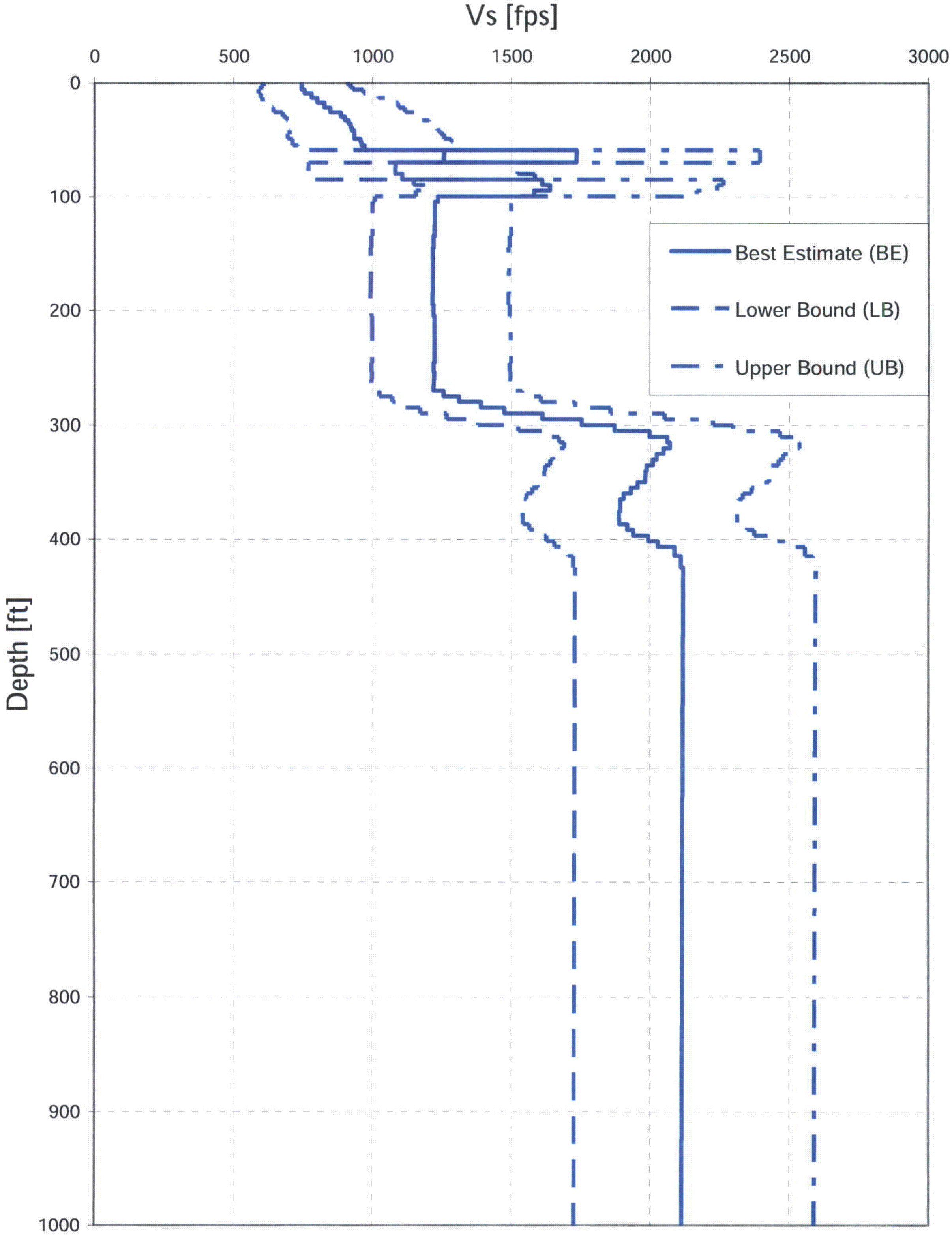
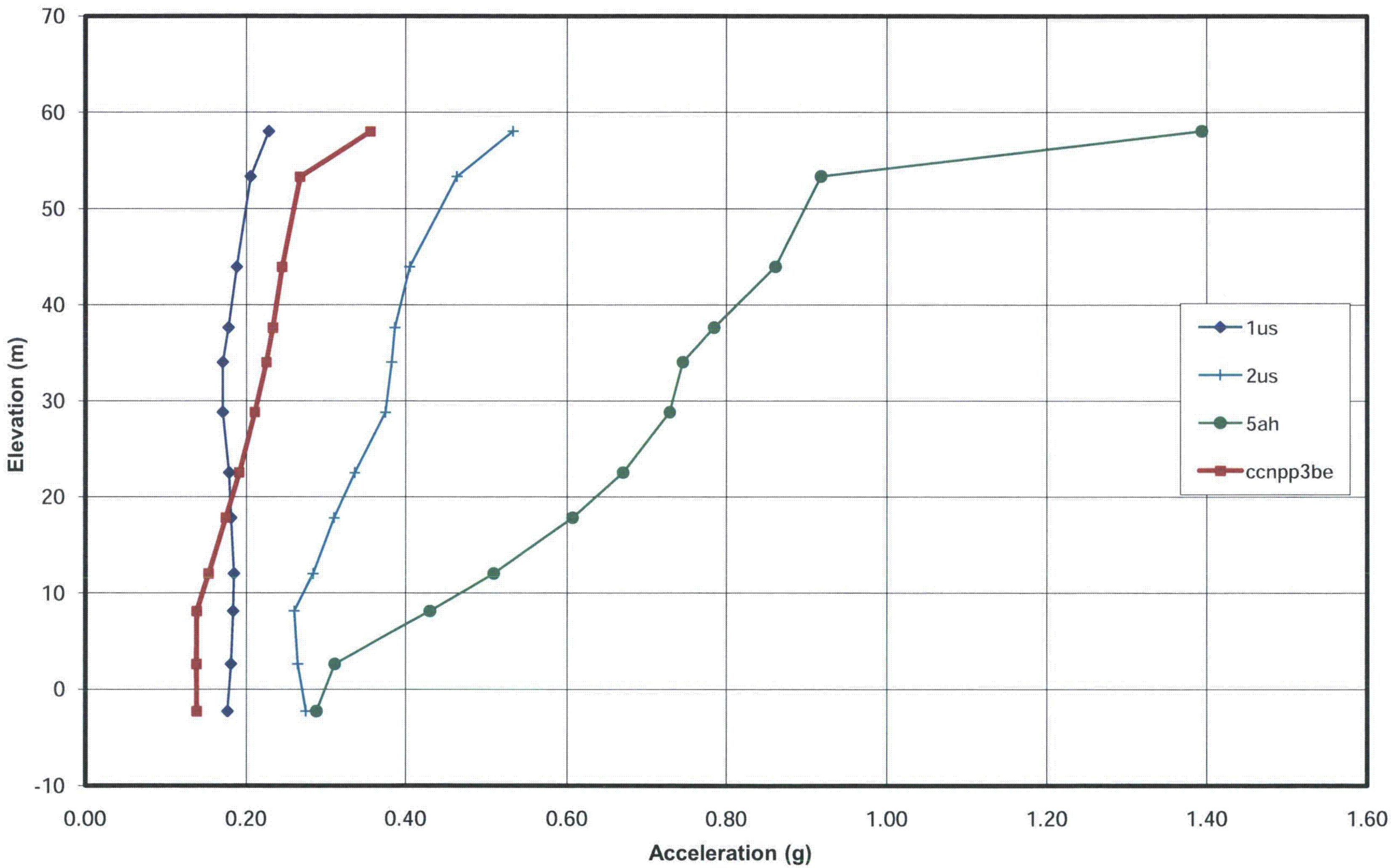


Figure 2.5-246— {EPR Project, Acceleration Profile for Containment Building, X-Direction}



Enclosure 3

Revised CCNPP Unit 3 FSAR Section 3.7

3.7 SEISMIC DESIGN

This section of the U.S. EPR FSAR is incorporated by reference with the supplements as described in the following sections.

3.7.1 Seismic Design Parameters

{Section 3.7.1 and Appendix 3F describe the site-specific seismic design characteristics for CCNPP Unit 3. Section 3.7.2 provides the methodology and results of the confirmatory site-specific Soil-Structure Interaction (SSI) analysis. The confirmatory analysis of the Nuclear Island (NI) is based on:

- a surface mounted structure

- the in-situ soil SWV profile, without the backfill as described in Section 3.8.4.6.1, and

- the site-specific SSE response spectra.

~~{Section 3.7.1~~ The results of this confirmatory analysis are not used for design because the US EPR Design of the NI, EPGB, and ~~Appendix 3F describe~~ ESWB are adopted for CCNPP3. Section 2.5.2.6 compares the site-specific seismic ~~design~~ characteristics ~~for CCNPP Unit 3. Section 3.7.2 demonstrates, through~~ and the results of the confirmatory ~~site-specific Soil-Structure Interaction (SSI) analysis,~~ analysis with the US EPR Analysis and Design. This comparison confirms that the ~~U.S.~~ US EPR seismic design ~~is applicable.~~ envelopes the CCNPP Unit 3 site by a large margin. In addition, the SSI analysis of the site-specific Seismic Category I structures, listed below, is presented in Section 3.7.2.

Throughout this section, three groups of structures are considered:

- Nuclear Island (NI) Common Basemat Structures
- Emergency Power Generating Buildings (EPGB) and Essential Service Water Buildings (ESWB) located in the NI area
- Site-specific Seismic Category I structures

The site-specific Seismic Category I structures at CCNPP Unit 3 are:

- Ultimate Heat Sink (UHS) Makeup Water Intake Structure
- Forebay
- Buried Electrical Duct Banks and Pipes

Two site-specific Seismic Category I structures: the UHS Makeup Water Intake Structure and the UHS Forebay, as well as the Seismic Category II Circulating Water Makeup Water Intake Structure share the same basemat; they are referred to as Common Basemat Intake Structures (CBIS). The CBIS are situated at the CCNPP Unit 3 site along the west bank of the Chesapeake Bay. Figures 9.2-4, 9.2-5 and 9.2-6 provide plan views of the Seismic Category I UHS structures, along with associated sections. Figures 10.4-4 and 10.4-5 provide the plan and section views of the Seismic Category II Circulating Water Makeup Intake Structure. The bottom of the CBIS basemat is situated approximately 37.5 ft (11.4 m) below a nominal grade elevation of 10 ft (3.0 m) NGVD 29. The layout of the Seismic Category I buried electrical duct banks and Seismic

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Category I buried piping is defined in Figures 3.8-1 and 3.8-2, and Figures 3.8-3 and 3.8-4, respectively.

3.7.1.1 Design Ground Motion

The site-specific Foundation Input Response Spectra (FIRS) for CCNPP Unit 3 are developed using Regulatory Guide 1.208 (NRC, 2007a). The FIRS for confirmatory analysis purposes are developed for the NI common basemat structures and the Seismic Category I ESWB and EPGB in the NI area, as well as area. The FIRS for design purposes are developed for the site-specific Seismic Category I CBIS in the Intake area. The development of the Site Safe Shutdown Earthquake (Site SSE) is discussed in Section 3.7.1.1.1. All FIRS are shown to be enveloped by the Site SSE. Therefore, the Site SSE is conservatively used as the input motion for both the confirmatory analysis of the structures in Section 3.7.2.US EPR FSAR structures; the NI, EPGB, and ESWB; and the design of the site-specific structures.

3.7.1.1.1 Design Ground Motion Response Spectra

3.7.1.1.1.1 Design Ground Motion Response Spectra for Nuclear Island Common Basemat Structures

Development of FIRS

As described, in For confirmatory analysis purposes, the US EPR FSAR Section 3.7.2.4, the NI Common Basemat Structures are analyzed as surface-founded structures and structural embedment is ignored in the Soil-Structure Interaction (SSI) analysis. The Foundation Input Response Spectra (FIRS) for the NI Common Basemat Structures is defined at the bottom of the basemat at approximately 40 ft (12 m) below grade. The GMRS are also defined at this depth. The FIRS for the NI common basemat is therefore taken as the GMRS for CCNPP Unit 3. The GMRS are developed, in Section 2.5.2.6, using Regulatory Guide 1.208 (NRC, 2007a). Computer programs SOILSIM (version 1.3) and RVTSITE (version 1.2) were used to perform site response analysis for the NI Common Basemat Structures and develop GMRS.

Development of Site SSE

Appendix S of 10 CFR Part 50 (CFR, 2008) requires that the horizontal component of the SSE ground motion in the free-field at the foundation level of the structures must be an appropriate response spectrum with a peak ground acceleration of at least 0.1 g. The FIRS for the horizontal direction in the free-field at the foundation level of the NI Common Basemat Structures has a peak ground acceleration of 0.076 g. Therefore an appropriate Site SSE for CCNPP Unit 3 is defined as follows.

The Site SSE ground motion for CCNPP Unit 3 is the envelope of the U.S. EPR FSAR European Utility Requirements (EUR) Soft Soil spectrum anchored at 0.15 g and the horizontal RG 1.60 spectrum anchored at 0.1 g, therefore satisfying the requirements of Appendix S of 10 CFR Part 50. The Site SSE ground motion, which is specified for both horizontal and vertical directions, is presented in Figure 3.7-1 and Table 3.7-1.

Comparison of FIRS, CSDRS and Site SSE

A comparison of the horizontal and vertical GMRS (or FIRS for NI Common Basemat Structures) versus the Site SSE is shown in 3.7-2 and 3.7-3, respectively. The horizontal and vertical GMRS are enveloped by the Site SSE. A comparison of the GMRS and Site SSE to the CSDRS is outlined below:

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1. The PGA for the GMRS (FIRS for the NI Common Basemat Structures) and Site SSE are less than 0.3 g, the PGA for the CSDRS.

2. A comparison of the FIRS for the NI Common Basemat Structures (i.e., GMRS) with the CSDRS is shown in 3.7-4 and 3.7-5 for the horizontal and vertical directions, respectively. This comparison shows that the CSDRS envelops the GMRS (FIRS for the NI Common Basemat Structures).

3. A comparison of the Site SSE with the CSDRS is shown in Figure 3.7-6. This comparison shows that the CSDRS does not envelop the Site SSE in the low frequency range. This very small exceedence is shown to be acceptable in the site seismic characteristics reconciliation documented in Section 2.5.2.6.

~~In conclusion, while the CCNPP Unit 3 GMRS are enveloped by the CSDRS, the Site SSE is not enveloped by the CSDRS. Therefore, a confirmatory SSI analysis is conducted, as described in Section 3.7.2.~~

Development of Site OBE

RG 1.166 states that the operating basis earthquake (OBE) response spectrum check is performed using the lower of: 1) The spectrum used in the certified design, or 2) A spectrum other than (1) used in the design of any Seismic Category I structure.

Section 3.7.4.4 of the U.S. EPR FSAR states that the application of OBE Exceedance Criteria is based on the following:

i. For the certified design portion of the plant, the OBE ground motion is one-third of the certified seismic design response spectra (CSDRS).

ii. For the safety-related noncertified design portion of the plant, the OBE ground motion is one-third of the site-specific SSE design motion response spectra, as described in Section 3.7.1.

iii. The threshold response spectrum ordinate criterion to be used in conjunction with RG 1.166 is the lowest of (i) and (ii).

The EUR soft soil spectrum is lower than the Site SSE below approximately 0.36 Hz. Therefore, the Site OBE for CCNPP Unit 3 is the composite earthquake which consists of one-third of the site SSE (i.e. the Site SSE anchored at 0.05g vs. 0.15g) in the high frequency, and one-third of the EUR Soft Soil spectrum (i.e. the EUR Soft Soil Spectrum anchored at 0.10g vs. 0.30g) in the low frequency (approximately 0.36Hz and below). The Site OBE is shown in Figure 3.7-6.

3.7.1.1.1.2 Design Ground Motion Response Spectra for EPGB and ESWB

Development of FIRS

The FIRS for Seismic Category I Emergency Power Generating Buildings (EPGB) and the Seismic Category I Essential Service Water Buildings (ESWB) are developed in accordance with Regulatory Guide 1.208 (NRC, 2007a). The FIRS are developed through seismic site response analysis using the rock motion spectra, presented in Section 2.5.2.5.1.4, and the soil profile properties representing the NI area site conditions, presented in Section 2.5.4.2 (including properties for structural backfill that supports both the EPGB and ESWB). Appendix 3F discusses in detail the development of FIRS as well as the site response analysis methodology and the computer codes.

Comparison of FIRS, CSDRS and Site SSE

The FIRS are checked for adequacy as SSI input according to the applicable requirements (NEI, 2009 and NRC, 2009), and amplified to account for the structure-soil-structure Interaction (SSSI) effects at the NI area (see Appendix 3F for details). The modified and amplified FIRS are referred to as Adjusted FIRS in the following discussion. Figure 3.7-7 compares the Site SSE with the following spectra:

- Site-specific horizontal and vertical Adjusted FIRS for the EPGB and ESWB. The FIRS for the EPGB and ESWB are calculated as the envelope of the FIRS at ground surface (the EPGB in the SSI analysis is surface founded) and the FIRS at 22 ft (6.7 m) below grade (corresponding to the bottom of foundation elevation of the ESWB).
- Regulatory Guide 1.60 (NRC, 1973) horizontal spectrum scaled to a PGA of 0.10 g.
- The CSDRS based on the EUR soft, medium and hard soil spectra.

The comparison shows that the CSDRS envelops the Adjusted FIRS at all frequencies except for small exceedance at the low frequency range (around 0.2 Hz). The comparison also shows, as presented more clearly in Figure 3.7-8, that in addition to satisfying the requirements of Appendix S of 10 CFR Part 50 (CFR, 2008), the Site SSE envelops the Adjusted FIRS. As such, confirmatory SSI analyses are performed for the EPGB and ESWB using the Site SSE as the design response spectrum and a set of site-specific LB, BE and UB soil profiles strain-compatible with Site SSE, presented in Section 3.7.1.3.2.

The site-specific confirmatory SSI analysis is presented in Section 3.7.2 and demonstrates that the U.S. EPR design is applicable to the EPGB and ESWB.

3.7.1.1.1.3 Design Ground Motion Response Spectra for Common Basemat Intake Structures

Development of FIRS

The FIRS for the site-specific structures (CBIS) are developed in accordance with Regulatory Guide 1.208 (NRC, 2007a). The FIRS are developed through seismic site response analysis using the rock motion spectra, presented in Section 2.5.2.5.1.4, and the soil profile properties representing the Intake area site conditions, presented in Section 2.5.4.2 (including properties for structural backfill surrounding the CBIS). Appendix 3F discusses in detail the development of FIRS as well as the site response analysis methodology and the computer codes used.

Comparison of FIRS and Site SSE

The FIRS are checked for adequacy as SSI input according to the applicable requirements (NEI, 2009 and NRC, 2009), see Appendix 3F for details. The modified FIRS are referred to as Adjusted FIRS in the following discussion. Figure 3.7-9 compares the Site SSE with the following spectra:

- Site-specific horizontal and vertical Adjusted FIRS for the Intake area at 37.5 ft (11.4 m) below grade (corresponding to the bottom of foundation elevation of the CBIS).

- Regulatory Guide 1.60 (NRC, 1973) horizontal spectrum scaled to a PGA of 0.10 g.

Figure 3.7-9 demonstrates that, in addition to satisfying the requirements of Appendix S of 10 CFR Part 50 (CFR, 2008), there is significant margin between the Site SSE and the horizontal and vertical Adjusted FIRS.

The SSI analysis for the CBIS is described in detail in Section 3.7.2.4. The analysis uses the Site SSE as the design response spectrum and a set of site-specific LB, BE and UB profiles (presented in Section 3.7.1.3.3) that are strain-compatible with the Site SSE.

3.7.1.1.1.4 Design Ground Motion Response Spectra for Seismic Category I Buried Utilities

A separate site response analysis can not be performed for the utility corridor between the NI and Intake areas until detailed design. However, the FIRS developed for the NI area (Section 3.7.1.1.1.1 and Section 3.7.1.1.1.2) and Intake area (Section 3.7.1.1.1.3) are shown to be comfortably enveloped by the Site SSE. The Site SSE is therefore considered as the design ground motion for the seismic analysis of the buried utilities.

3.7.1.1.2 Design Ground Motion Time History

A three component set of spectrum compatible acceleration time histories is developed for use as input time histories for SSI analysis. The two horizontal and one vertical components are modified to be spectrum compatible with the Site SSE. The spectral matching criteria given in NUREG CR-6728 (McGuire et al., 2001) and NUREG-0800, Section 3.7.1, Approach 2, Option 1 (NRC, 2007b) are followed for the spectral matching procedure, including the cross-correlation between the three components of less than 0.16. The starting seed input time histories are selected as the EUR soft soil three component acceleration time histories, presented in U.S. EPR FSAR Section 3.7.1.1.2. These time histories are spectrum compatible with the EUR soft target spectra scaled to a PGA of 0.3g. Figure 3.7-10 presents the acceleration, velocity and displacement time histories for the first horizontal component (S1) spectrally matched to Site SSE. Figure 3.7-11 presents the time histories for the second horizontal component (S2) and Figure 3.7-12 presents the time histories for the vertical component (S3). Bechtel proprietary computer programs RSPM (version 1.0) and SETARGET (version 1.0) were used to develop these spectrally matched time histories.

3.7.1.1.2.1 Design Ground Motion Time History for Nuclear Island Common Basemat

As described in the US EPR FSAR Section 3.7.2.4, the NI Common Basemat Structures are analyzed as surface-founded structures and structural embedment is ignored in the SSI analysis. The three component set of Site SSE spectrum compatible acceleration time histories presented in Figure 3.7-10 through Figure 3.7-12 are used as the input ground motion for the confirmatory SSI analysis of the NI Common Basemat Structures.

3.7.1.1.2.2 Design Ground Motion Time History for EPGB and ESWB

As described in the US EPR FSAR Section 3.7.2.4, the EPGB is analyzed as a surface-founded structure. The three component set of Site SSE spectrum compatible acceleration time histories presented in Figure 3.7-10 through Figure 3.7-12 are used as the input ground motion for the confirmatory SSI analysis of the EPGB.

In the case of the ESWB, which is analyzed as an embedded structure, the "within" acceleration time histories at the FIRS horizon are calculated using the computer program SHAKE2000 (described in Appendix 3F). In this analysis, the Site SSE spectrally matched time histories are used as input "outcrop" motions at the foundation level in conjunction with the strain-compatible profiles for the NI area, presented in Section 3.7.1.3.2. No further iterations on soil properties are performed as the acceleration time history is converted from "outcrop" to "within." The analysis results in a set of three "within" motions (two horizontal and one vertical) at the same FIRS horizon. Three sets are developed corresponding to the LB, BE and UB profiles for the ESWB, as presented in Figure 3.7-13 through Figure 3.7-15. The development of the "within" acceleration time histories is discussed in detail in Appendix 3F. In the SSI analysis, the time histories are applied at the FIRS horizon as "within" motions and are used in conjunction with the respective SSI soil profiles, described in Section 3.7.1.3.2.

3.7.1.1.2.3 Design Ground Motion Time History for Common Basemat Intake Structures

In the case of the CBIS, which are analyzed as embedded structures, the "within" acceleration time histories at each FIRS horizon are calculated using the computer program SHAKE2000 (described in Appendix 3F). In this analysis, the Site SSE spectrally matched time histories are used as input "outcrop" motions at the foundation level in conjunction with the strain-compatible profiles for the Intake area, presented in Section 3.7.1.3.3. No further iterations on soil properties are performed as the acceleration time history is converted from "outcrop" to "within." The analysis results in a set of three "within" motions (two horizontal and one vertical) at the same FIRS horizon. Three sets are developed corresponding to the LB, BE and UB profiles for the CBIS, as presented in Figure 3.7-16 through Figure 3.7-18. The development of the within acceleration time histories is discussed in detail in Appendix 3F. The time histories are applied at the FIRS horizon as "within" motions and are used in conjunction with the corresponding SSI soil profiles, described in Section 3.7.1.3.3.

3.7.1.2 Percentage of Critical Damping Values

Operating Basis Earthquake (OBE) structural damping values, defined in Table 2 of RG 1.61, Rev 1 (NRC, 2007c), are used for the dynamic analysis of site-specific Seismic Category I SSCs and confirmatory SSI analysis of the NI Common Basemat Structures as well as for the EPGB and ESWB. In-structure response spectra (ISRS) for site-specific Seismic Category I structures are also based on OBE structural damping values.

The damping values for site-specific Seismic Category II-SSE and Seismic Category II structures are in accordance with RG 1.61, Rev. 1 (NRC, 2007c).

3.7.1.3 Supporting Media for Seismic Category I Structures

3.7.1.3.1 Nuclear Island Common Basemat

The supporting media for the seismic analysis of the NI Common Basemat Structures is shown in Figure 3.7-19 and Table 3.7-2 through Table 3.7-4. The presented soil profiles are site-specific and are strain-compatible with the Site SSE. Lower bound and upper bound profiles are calculated maintaining a minimum variation of 0.5 on the shear modulus. An evaluation of the CCNPP Unit 3 site-specific soil profiles with respect to the criteria provided in U.S. EPR FSAR Section 2.5.2.6 is described ~~below:~~in Section 2.5.2.6.

~~1. The NI Common basemat is founded on top of Chesapeake Cemented Sand with a low-strain, best-estimate shear wave velocity of approximately 1,450 ft/s (440 m/s) (see Figure 2.5-167). Since this shear wave velocity is greater than 1,000 ft/s (300 m/s), the CCNPP Unit 3 NI is founded on competent material as defined in NUREG-0800 Section 3.7.1 (NRC, 2007b).~~

~~2. The lateral uniformity of site-specific profile (using the criterion of a soil layer with an angle of dip less than 20 degrees) is addressed in Section 2.5.4.10.3.~~

3. The range of shear wave velocities of the CCNPP Unit 3 strain-compatible soil profiles is shown in Figure 3.7-19, and is bounded by that of the generic strain-compatible soil profiles used in the U.S. EPR FSAR as shown in Figure 3.7-20. However, there are variations in the soil layering and shear wave velocities from the generic soil profiles considered in the U.S. EPR FSAR.

In view of such variations, confirmatory Confirmatory site-specific SSI analyses are performed, as described in Section 3.7.2. The resulting in-structure response spectra (ISRS) at representative locations of the NI structures, as reported in Section 3.7.2.5.1, are found to be bounded by the corresponding U.S. EPR FSAR ISRS. Therefore, the U.S. EPR design is applicable to CCNPP Unit 3 NI Common Basemat Structures.

3.7.1.3.2 EPGB and ESWB

The supporting media for the seismic analysis of the EPGB and ESWB in the NI area are presented in Figure 3.7-21. The presented soil profiles are site-specific and are strain-compatible with the Site SSE. The development of the Site SSE strain-compatible soil profiles is described in detail in Appendix 3F.

Note that in contrast to Figure 3.7-19, where the top layer is located at the bottom of the NI common basemat foundation at approximately 40 ft (12 m) below grade, Figure 3.7-21 presents the profiles for the upper 656 ft (200m) with the top layer at grade, including the structural backfill layers, therefore consistent with the confirmatory SSI analyses of the EPGB and ESWB, described in Section 3.7.2.

3.7.1.3.3 Common Basemat Intake Structures

The supporting media for the seismic analysis of the CBIS in the Intake area are presented in Figure 3.7-22 for the upper 656 ft (200m). The presented soil profiles are site-specific and are strain-compatible with the Site SSE. The development of the Site SSE strain-compatible soil profiles is described in detail in Appendix 3F. The dimensions of the CBIS, including the structural height, are described in Section 3.7.2.3.2.

3.7.1.4 References

CFR, 2008. Domestic Licensing of Production and Utilization Facilities, 10 CFR Part 50, U.S. Nuclear Regulatory Commission, February 2008.

McGuire, R.K., W.J. Silva, and C.J. Constantino, 2001. Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk-Consistent Ground Motion Spectra Guidelines, NUREG CR-6728, October, 2001.

Nuclear Energy Institute [NEI], 2009. Consistent Site-Response/Soil Structure Interaction Analysis and Evaluation. NEI White Paper, June 12, 2009 (ADAMS Accession No. ML091680715).

NRC, 1973. Design Response Spectra for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.60, Revision 1, U.S. Nuclear Regulatory Commission, December 1973.

NRC, 2007a. A Performance-Based Approach to Define the Site Specific Earthquake Ground Motion, Regulatory Guide 1.208, Revision 0, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2007b. Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants, NUREG-0800, Revision 3, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2007c. Damping Values for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.61, Revision 1, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2009. Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses, DC/COL-ISG-017 Draft Issued for Comments.}

3.7.2 Seismic System Analysis

The U.S. EPR FSAR includes the following COL Item in Section 3.7.2:

> A COL applicant that references the U.S. EPR design certification will confirm that the site-specific seismic response is within the parameters of Section 3.7 of the U.S. EPR standard design.

This COL Item is addressed as follows:

{The confirmatory soil-structure interaction (SSI) ~~analysis~~analyses of Nuclear Island (NI) Common Basemat Structures, Emergency Power Generating Buildings (EPGBs) and Essential Service Water Buildings (ESWBs) for Site SSE and site-specific strain-compatible soil properties is addressed in Section 3.7.2.4.~~The confirmatory SSI analysis is performed since:~~

- ~~the U.S. EPR FSAR certified seismic design response spectra (CSDRS) does not envelop the Site SSE in the low frequency range, as shown in Figure 3.7-6, and~~
- ~~the site-specific strain-compatible best estimate (BE), lower bound (LB) and upper bound (UB) soil profiles are bounded, but exhibit variations in the upper layers when compared with the ten generic soil profiles used in U.S. EPR FSAR, as described in FSAR Section 3.7.1.3.1.~~

Site-specific Seismic Category I structures at CCNPP Unit 3 include:

- Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS)
- Forebay

The Seismic Category I UHS Makeup Water Intake Structure and Seismic Category I Forebay are situated at the CCNPP Unit 3 site along the west bank of the Chesapeake Bay. These structures are part of the UHS Makeup Water System, which provides makeup water to the Essential Service Water Buildings for maintaining the safe shutdown of the plant 72 hours after a design basis accident. The UHS Makeup Water Intake Structure and Forebay are supported on a common basemat, which also supports the Seismic Category II Circulating Water Makeup Intake Structure. The UHS Makeup Water Intake Structure, Forebay, and Circulating Water Makeup Intake Structure, henceforth referred to as the Common Basemat Intake Structures (CBIS) in Section 3.7.2, are integrally connected. The Circulating Water Makeup Intake Structure and the UHS Makeup Water Intake Structure, respectively, are located on the north and south end of the Forebay. Figure 2.1-1 depicts the CCNPP Unit 3 site plan, which shows the position of the UHS Makeup Water Intake Structure and Forebay relative to the NI.

The bottom of the CBIS common basemat is situated approximately 37.5 ft (11.4 m) below a nominal grade elevation of 10 ft (3.0 m). 9.2-4, 9.2-5, and 9.2-6 provide plan views of the Seismic Category I structures, along with associated sections and details. 10.4-4 and 10.4-5 provide the plan and section views of the Seismic Category II Circulating Water Makeup Intake Structure.

3.7.2.1 Seismic Analysis Methods

No departures or supplements.

3.7.2.1.1 Time History Analysis Method

No departures or supplements.

3.7.2.1.2 Response Spectrum Method

No departures or supplements.

3.7.2.1.3 Complex Frequency Response Analysis Method

As described in Section 3.7.2.3.2, an integrated finite element model is developed for the CBIS. The complex frequency response analysis method is used for the seismic SSI analysis of these structures, with earthquake motion considered in three orthogonal directions (two horizontal and one vertical) as described in Section 3.7.2.6. The SSI analysis of site-specific structures is performed, as described in Section 3.7.2.4, using RIZZO computer code SASSI, Version 1.3a. The hydrodynamic load effects are considered as described in Section 3.7.2.3.2.

3.7.2.1.4 Equivalent Static Load Method of Analysis

No departures or supplements.

3.7.2.2 Natural Frequencies and Response Loads

3.7.2.2.1 Nuclear Island Common Basemat Structures

Section 3.7.2.5.1 provides the in-structure response spectra (ISRS) for NI Common Basemat Structures for site-specific strain-compatible soil properties and Site SSE.

3.7.2.2.2 EPGB and ESWB

Section 3.7.2.5.2 provides the ISRS for EPGB and ESWB at the locations defined in U.S. EPR FSAR Section 3.7.2.5 for site-specific strain-compatible soil properties and Site SSE. Section 3.7.2.4.6.2 provides the combined average maximum nodal accelerations for the site-specific confirmatory SSI analysis.

3.7.2.2.3 Common Basemat Intake Structures

The SSI analysis of site-specific Seismic Category I structures is performed using the complex frequency response analysis method described in Section 3.7.2.1.3, where the equation of motion is solved in the frequency domain. The natural frequencies and associated modal analysis results are not obtained from this analysis. However, fixed base undamped eigenvalue analyses have been performed separately for the Common Basemat Intake Structures. The analysis results are tabulated in 3.7-5 and 3.7-6 for reference purposes only.

Section 3.7.2.5.3 provides the ISRS at the locations of safety-related UHS Makeup Water pumps and facilities in the UHS Makeup Water Intake Structure at El. 11.5 ft and El. -22.5 ft, and at the location of safety-related electrical equipment at El. 26.5 ft. Section 3.7.2.4.6.3 provides the combined maximum nodal accelerations for the CBIS.

3.7.2.3 Procedures Used for Analytical Modeling

No departures or supplements.

3.7.2.3.1 Seismic Category I Structures – Nuclear Island Common Basemat

No departures or supplements.

3.7.2.3.2 Seismic Category I Structures – Not on Nuclear Island Common Basemat

As described in Section 3.7.2.4.2.2, the confirmatory SSI analysis of EPGB and ESWB is performed using ~~the same structural model defined in U.S. EPR FSAR.~~finite element models.

The UHS Makeup Water Intake Structure and Forebay are the site-specific Seismic Category I structures situated away from the NI in the intake area.

The CBIS, i.e., the UHS Makeup Water Intake Structure, Forebay, and Circulating Water Makeup Intake Structure are reinforced concrete shear wall structures, and are supported on a 5 ft (1.5 m) thick reinforced concrete basemat. The Common Basemat Intake Structures extend approximately 260 ft (79.3 m) along the North-South direction and 89 ft (27.1 m) along the East-West direction, with respect to CCNPP Unit 3 coordinate system. The maximum height of the structures from the bottom of common basemat to the top of the UHS Makeup Water Intake Structure roof is approximately 69 ft (21.0 m).

Figures 9.2-4 through 9.2-6 and 10.4-4 and 10.4-5 are used as the bases for the development of the analytical model of the aforementioned structures.

A 3D finite element model of the CBIS is developed in STAAD Pro, Version 8i, as shown in Figures 3.7-23 and 3.7-24. The model is used to generate the finite element model for seismic SSI analysis using RIZZO computer code SASSI, Version 1.3a, and to perform static analysis for non-seismic loads.

The CBIS are symmetric about the North-South axis, as depicted in Figures 9.2-4 through 9.2-6 and 10.4-4 and 10.4-5. A sensitivity analysis was performed to consider the effects of the non-symmetric features such as door openings and equipment masses. Based on the sensitivity analysis, only one-half (western half) of the CBIS is modeled for the SSI analysis. Figure 3.7-23 depicts the finite element mesh for the half model.

The reinforced concrete basemat, floor slabs, and walls of the Common Basemat Intake Structures are modeled using plate/shell elements to accurately represent the structural geometry and to capture both in-plane and out-of-plane effects from applied loads. The finite element mesh is sufficiently refined to accurately represent the global and local modes of vibration. The skimmer walls, at the entrance of the UHS Makeup Water Intake Structure and Circulating Water Makeup Intake Structure into the Forebay Structure, have an inclination of approximately 10 degrees with the vertical, which is neglected in the finite element model. This simplification has an insignificant effect on the global mass and stiffness distribution, and is conservative for the local response of structural panels. The finite element model in SASSI uses a thin shell element formulation that represents the in-plane and out-of-plane bending effects. In-plane shear deformation are accurately reproduced by the finite element mesh, while out-of-plane shear deformations are considered negligible due to the low thickness/ height ratio of these walls.

The reinforced concrete basemat, floor slabs, and walls of the CBIS are modeled using thin shell elements in RIZZO computer code SASSI, Version 1.3a, to accurately represent the structural geometry and to capture in-plane membrane and out-of-plane bending. The average mesh size used in the finite element model below ground level and along the vertical direction is approximately 1.6 ft (0.5 m), based on one-fifth of the wave length at the highest frequency of the SASSI analysis. The average mesh size in the plan direction is approximately 5 ft (1.5 m), abased on an aspect ratio of approximately 3.0.

The skimmer walls, at the entrance of the UHS Makeup Water Intake Structure and the Circulating Water Makeup Intake Structure into the Forebay, have an inclination of approximately 10 degrees with the vertical. However, these walls are modeled vertically for simplification of the finite element model. This simplification has an insignificant effect on the global mass and stiffness distribution, and on the local responses of the structural panels.

The east and west bottom walls of the Forebay, to the top portion of the forebay wall corners, and the basemat below the backfill inside the UHS MWIS are the only structural panels that will crack during any of the applicable loading conditions. These walls crack since they retain approximately 37.5 ft (11.5 m) of soil and exhibit cantilever behavior. The out-of-plane bending stiffness of these walls is reduced by one-half to simulate cracked behavior in accordance with ASCE 43-05 (ASCE, 2005). For the walls located in the plane of symmetry, the modulus of elasticity and density are reduced by one-half to accurately represent mass and stiffness in the half model.

As shown in 10.4-4 and 10.4-5, the pump house enclosure and the electrical room for the Circulating Water Makeup Intake Structure are steel enclosures founded on grade slabs. The grade slabs are separated from the CBIS by providing an expansion joint, and are not included in the finite element model. The south end of the pump house enclosure is partially supported on the operating deck slab of the Circulating Water Makeup Intake Structure. The masses corresponding to the applicable dead loads and snow loads for the pump house enclosure are appropriately included in the finite element model.

The finite element model used for the seismic SSI analysis includes masses corresponding to 25 percent of floor design live load and 75 percent of roof design snow load, as applicable, and 50 pounds per square feet of miscellaneous dead load in addition to the self weight of the structure. The weights of equipment are included in the dynamic analysis.

The hydrodynamic effects of water contained in the CBIS are considered in accordance with ACI 350.3-06 (ACI, 2006). The impulsive and convective water masses due to horizontal earthquake excitation are calculated using the clear dimensions between the walls perpendicular to the direction of motion and the minimum height of water during a hurricane (Elev. -4.0 ft NGVD 29). The impulsive water masses are rigidly attached to the walls, and the convective water masses are connected to the walls using springs with appropriate stiffness. The entire water mass is lumped at the basemat nodes for earthquake ground motion in the vertical direction. The hydrodynamic loads are included for walls both in the Forebay and basement of the UHS Makeup Water Intake Structure.

The maximum sloshing heights in both directions for the UHS Makeup Water Intake Structure and the Forebay are approximately 0.6 ft (0.2 m) and 0.5 ft (0.15 m), respectively. The minimum available freeboard for the UHS Makeup Water Intake Structure and the minimum clearance for the Forebay are significantly higher than the maximum sloshing heights.

The earthquake excitation along the North-South and vertical directions cause symmetric loading on the structure, whereas the earthquake excitation along the East-West direction causes anti-symmetric loading on the structure. The seismic SSI analysis is performed by applying appropriate symmetric and anti-symmetric boundary conditions in the plane of symmetry of the half model shown in Figure 3.7-23, as indicated in Table 3.7-7.

3.7.2.3.3 Seismic Category II Structures

Site-specific Seismic Category II-SSE structures, systems, and components (SSCs) are analyzed and designed to meet the same requirements as the Seismic Category I SSCs. Seismic Category II Circulating Water Makeup Intake Structure is analyzed along-with the Seismic Category I Forebay and Seismic Category I UHS Makeup Water Intake Structure, as described in Section 3.7.2.3.2. Other site-specific Seismic Category II structures are designed using conventional codes and standards, but are also analyzed for Site SSE.

3.7.2.3.4 Conventional Seismic (CS) Structures

No departures or supplements.

3.7.2.4 Soil-Structure Interaction

This section describes the confirmatory soil-structure interaction (SSI) analyses for the Nuclear Island Common Basemat Structures, EPGB, and ESWB. In addition the SSI analysis of the CBIS are also described.

The complex frequency response analysis method is used for the SSI analyses, in accordance with the requirements of NUREG-0800 Section 3.7.2, Acceptance Criteria 1.A and 4 and Section 3.7.1, Acceptance Criteria 4.A.vii (NRC, 2007a). During the SSI analyses, the effects of foundation embedment (for ESWB and CBIS), soil layering, soil nonlinearity, ground water table, and variability of soil and rock properties on the seismic response of the structures are accounted for, as described in the following sections. In particular, Sections 3.7.2.4.1 through 3.7.2.4.6 provide the steps followed to perform the SSI analyses. Section 3.7.2.4.7 describes the computer codes used in the analyses.

3.7.2.4.1 Step 1 - SSE Strain Compatible Soil Properties

3.7.2.4.1.1 Nuclear Island Common Basemat Structures

For the Nuclear Island Common Basemat Structures, confirmatory SSI analyses are performed for the lower bound, best estimate and upper bound soil profiles established in Section 3.7.1.3.1 and shown in 3.7-2, 3.7-3 and 3.7-4. Soil properties used in the SSI analysis are strain-compatible with the Site SSE, and account for the range of variation of shear-wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.1.2 EPGB and ESWB

For the EPGB and ESWB, confirmatory SSI analyses are performed for the lower bound, best estimate and upper bound soil profiles established in Section 3.7.1.3.2. 3F-3, 3F-4, and 3F-5 show the properties for the top fifty layers of each soil profile (approximately 300 ft), while 3F-29, 3F-30 and 3F-31, respectively, show the shear wave velocity, damping ratio and P-wave velocity for the top six hundred feet in this area. Soil properties used in the SSI analysis are strain-compatible with the Site SSE, and account for the range of variation of shear-wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.1.3 Common Basemat Intake Structures

SSI analyses for the CBIS are performed for the lower bound, best estimate and upper bound soil profiles established in Section 3.7.1.3.3. Tables 3F-6, 3F-7 and 3F-8 show the properties for the top fifty layers of each soil profile (approximately 380 ft), while 3F-32, 3F-33 and 3F-34, respectively, show the shear wave velocity, damping ratio and P-wave velocity for the top six hundred feet in the intake area. Soil properties used in the SSI analysis are strain-compatible with the Site SSE, and account for the range of variation of shear-wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.2 Step 2 - Development of Structural Model

3.7.2.4.2.1 Nuclear Island Common Basemat Structures

Confirmatory SSI analyses of the Nuclear Island Common Basemat Structures ~~use the same structural model as used in U.S. EPR FSAR, except that~~ uses a surface founded stick model. 4 percent structural damping for reinforced concrete is used and 3 percent structural damping for pre-stressed concrete, NSSS components and vent stack is applied. ~~In particular, the NI Common Basemat Structures are analyzed as surface-founded structures on a rigid foundation.~~

3.7.2.4.2.2 EPGB and ESWB

Confirmatory SSI analyses for the EPGB and ESWB use ~~the same structural model and structural~~ finite element models. 4% structural damping ~~(i.e., 4 percent structural damping) as described in U.S. EPR FSAR Sections 3.7.2.3.2 and 3.7.2.4.2 for these structures.~~ is used.

3.7.2.4.2.3 Common Basemat Intake Structures

Section 3.7.2.3.2 describes the development of the integrated finite element model of the CBIS in STAAD Pro, and translation of the model into SASSI. The thin plate element in SASSI is used to model all the structural panels.

The Common Basemat Intake Structures are reinforced concrete structures. A structural damping of 4 percent is used in the SSI analysis to obtain the ISRS, while 5 percent is used to obtain internal forces for the design of the CBIS using STAAD Pro.

3.7.2.4.3 Step 3 - Development of Soil Model

3.7.2.4.3.1 Nuclear Island Common Basemat Structures

SSI analyses are conducted for the three soil profiles discussed in Section 3.7.2.4.1.1, namely CCNPP Unit 3 strain-compatible BE, CCNPP Unit 3 strain-compatible LB and CCNPP Unit 3 strain-compatible UB. Each soil profile is discretized in a sufficient number of horizontal sub-layers, followed by a uniform half space beneath the lowest sub-layer.

The effect of ground water table on the seismic soil-structure-interaction (SSI) analysis of NI Common Basemat Structures is considered through modification of the P-Wave velocity profiles and by using the saturated weight for the soil below the ground water table.

3.7.2.4.3.2 EPGB and ESWB

The soil model is developed using the SSE strain-compatible lower bound, best estimate and upper bound soil profiles discussed in Section 3.7.2.4.1.2. Each soil profile is discretized in a sufficient number of horizontal sub-layers, followed by a uniform half space beneath the lowest sub-layer, which is located at a depth of 435 ft. The material soil or rock damping does not exceed 15 percent. P-wave damping is set to be equal to S-wave damping for all soil layers.

The effect of ground water table on the seismic soil-structure-interaction (SSI) analysis of the structure is considered through modification of the P-Wave velocity profiles as discussed in Section 3.7.1.3.2 and by using the saturated weight for the soil below the ground water table.

3.7.2.4.3.3 Common Basemat Intake Structures

The soil model is developed using the SSE strain-compatible lower bound, best estimate and upper bound soil profiles discussed in Section 3.7.2.4.1.3. Each soil profile is discretized in a number of horizontal sub-layers, based on shear propagation requirement, and a uniform half space is introduced beneath the lowest sub-layer, which is located at a depth of 350 ft. The material soil or rock damping does not exceed 15 percent. P-wave damping is set to be equal to S-wave damping for all soil layers.

The effect of ground water table on the seismic SSI analysis of the integrated CBIS is considered through modification of the P-Wave velocity profiles as discussed in Section 3.7.1.3.3, and by using the saturated weight for the soil below the ground water table.

3.7.2.4.4 Step 4 – Development of SSI Analysis Soil Model

3.7.2.4.4.1 Nuclear Island Common Basemat Structures

~~The same SSI~~ A surface founded stick model ~~and methodology~~ is used ~~in U.S. EPR FSAR~~ for the Nuclear Island Common Basemat Structures ~~is used for the~~ confirmatory SSI ~~analyses, with~~ analyses. The analysis uses the following ~~exceptions:~~ inputs:

- Site-specific soil profiles strain-compatible with the Site SSE are used, as described in Section 3.7.2.4.1.1.
- The free-field control input motion to the SSI analysis of the NI Common Basemat Structures is the Site SSE previously described in Section 3.7.1.1.2.1. The Site SSE is applied at NI foundation level, which is the horizon used for development of the NI FIRS (i.e., CCNPP Unit 3 GMRS described in Section 2.5.2.6). In particular, the surface outcrop motions (acceleration time histories) shown in 3.7-10, 3.7-11 and 3.7-12 are used for the SSI analysis.
- Four percent structural damping is applied.

3.7.2.4.4.2 EPGB and ESWB

~~The same~~ An SSI model and methodology ~~used in U.S. EPR FSAR for~~ of the EPGB and ESWB is used for the confirmatory SSI ~~analyses, with~~ analyses. The analysis uses the following ~~exceptions:~~ inputs:

- ~~Interaction forces are obtained at the basemat nodes at the soil-structure interface, and subsequently used in the stability analyses described in Section 3.7.2.14.2.~~
- Site-specific soil profiles strain-compatible with the Site SSE are used, as described in Section 3.7.2.4.1.2.
- The control input motion for the SSI analysis of the EPGB and ESWB is the Site SSE described in Section 3.7.1.1.2.2. The control motion is applied at the foundation level (i.e., at the same horizon used for development of FIRS). In particular, for the EPGB, the surface outcrop motions (acceleration time histories) shown in 3.7-10, 3.7-11 and 3.7-12 are used, while for the ESWB the within soil-column motions (acceleration time histories) shown in 3.7-13, 3.7-14 and 3.7-15 are used.

Interaction forces are obtained at the basemat nodes at the soil-structure interface, and subsequently used in the stability analyses described in Section 3.7.2.14.2.

3.7.2.4.4.3 Common Basemat Intake Structures

The SSI model includes the CBIS, the surrounding layers of structural fill and the existing soil media as shown in Figure 3.7-24. Interaction forces are obtained at the basemat nodes at the soil-structure interface, and subsequently used in the stability analyses described in Section 3.7.2.14.2.

The control input motion for the SSI analysis of the CBIS is the within soil-column motion corresponding to the outcrop Site SSE for each soil profile, shown in Figures 3.7-16, 3.7-17 and 3.7-18 and described in Section 3.7.1.1.2.3. Consistent with the development of the within soil-column motion, the control motion is applied at the foundation level of the CBIS (i.e., at the same horizon used for development of FIRS for the CBIS).

3.7.2.4.5 Step 5 - Performing SSI Analysis

3.7.2.4.5.1 Nuclear Island Common Basemat Structures

Confirmatory SSI analyses for the Nuclear Island Common Basemat Structures are performed following the ~~same methodology used in U.S. EPR FSAR for this structure.~~previously described methodology.

3.7.2.4.5.2 EPGB and ESWB

Confirmatory SSI analyses for the EPGB and ESWB are performed following the ~~same methodology used in U.S. EPR FSAR for these structures.~~previously described methodology.

3.7.2.4.5.3 Common Basemat Intake Structures

The SSI analysis of the model for the CBIS is performed using RIZZO computer code SASSI. SSI analysis is performed for each direction of the Site SSE (i.e., X (N-S), Y (E-W), Z (Vertical)) and for each of the three soil profiles described in Section 3.7.2.4.1.3.

3.7.2.4.6 Step 6 - Extracting Seismic SSI Responses

3.7.2.4.6.1 Nuclear Island Common Basemat Structures

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. In particular in-structure response spectra for 5 percent damping are generated at the key locations as described in Section 3.7.2.5.1.

3.7.2.4.6.2 EPGB and ESWB

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. Accelerations, in-structure response spectra, and interaction forces at the soil-basemat interface are calculated.

3.7-8 and 3.7-9 provide the combined average maximum nodal accelerations at various elevations of EPGB and ESWB, respectively.~~These accelerations have been obtained using the same methodology outlined in U.S. EPR FSAR Section 3.7.2.4.6.~~Comparison of the structural accelerations provided in 3.7-8 and 3.7-9 with the corresponding structural accelerations reported in U.S. EPR FSAR Tables 3.7.2-27 and 3.7.2-28, respectively, show that the site-specific accelerations for EPGB and ESWB are bounded by the certified design.

Output response time histories of nodal interaction forces for each of the basemat nodes of the EPGB and ESWB are used to calculate response time histories of resultant sliding forces and overturning moments, which are used to evaluate the overall stability of each structure as described in Section 3.7.2.14.2.

In-structure response spectra are reported at selected locations of the EPGB and ESWB as detailed in Section 3.7.2.5.2.

3.7.2.4.6.3 Common Basemat Intake Structures

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. Accelerations, relative displacements, element forces, in-structure response spectra, resultant sliding force and total overturning moments are calculated.

Table 3.7-10 provides the combined maximum nodal accelerations at various elevations of UHS Makeup Water Intake Structure. These accelerations have been obtained using the methodology outlined in U.S. EPR FSAR Section 3.7.2.4.6.

Absolute peak element forces and moments (i.e., membrane and out-of-plane bending and shear resultants) are calculated for each soil profile and direction of the input motion. These forces and moments are used for the design of critical walls and slabs, as detailed in Appendix 3E.

For determination of seismic stability of the CBIS, the seismically induced normal and shear stresses at the base of the CBIS foundation are computed and compared with the restoring stresses from the self weight of the structure as described in Section 3.7.2.14.3.

In-structure response spectra (ISRS) are reported at selected locations of the CBIS as detailed in Section 3.7.2.5.3.

3.7.2.4.7 Computer Codes

The confirmatory SSI analysis of the NI Common Basemat Structures is performed using AREVA computer code SASSI, Version 4.2; which has been verified and validated in accordance with the AREVA 10 CFR 50 Appendix B QA program.

Bechtel computer code SASSI2000, Version 3.1, is used to perform the seismic confirmatory SSI analysis of the EPGB and ESWB. This program is developed and maintained in accordance with Bechtel's engineering department and QA procedures. Validation manuals are maintained in the Bechtel Computer Services Library. The program is in compliance with the requirements of ASME NQA-1-1994.

RIZZO computer code SASSI, Version 1.3a, is used to perform the seismic confirmatory SSI analysis of the CBIS. This program is developed and maintained in accordance with RIZZO's engineering department and QA procedures. Validation manuals are maintained in the RIZZO Computer Services Library. The program is in compliance with the requirements of ASME NQA-1-1994.

3.7.2.5 Development of Floor Response Spectra

A structural damping of 4 percent is used for the development of ISRS for the site-specific reconciliation of NI Common Basemat Structures, EPGB and ESWB; this is in compliance with RG 1.61, Revision 1 (NRC, 2007b). This damping value is also used for the development of ISRS for the Common Basemat Intake Structures.

As described in Sections 3.7.2.5.1 and 3.7.2.5.2, the ISRS for NI Common Basemat Structures, EPGB and ESWB are bounded by the corresponding U.S. EPR FSAR ISRS. Therefore, the U.S. EPR FSAR ISRS are applicable to CCNPP Unit 3 NI Common Basemat Structures, EPGB and ESWB.

3.7.2.5.1 Nuclear Island Common Basemat Structures

U.S. EPR FSAR Section 3.7.2.5 describes the development of floor response spectra for the NI Common Basemat Structures. The soil cases are described in U.S. EPR FSAR Table 3.7.1-6 and the ground design response spectra are shown in U.S. EPR FSAR Figure 3.7.1-1 for the NI. The ISRS used to design the piping, cable trays and commodity supports for the NI are the spectrum envelopes shown in U.S. EPR FSAR, Tier 2, Figures 3.7.2-74 through 3.7.2-100 and Figures 3.7.2-110 through 3.7.2-112.

For site-specific confirmatory analysis, response spectra for 5 percent damping in the three directions are generated, using methodology consistent with the U.S. EPR FSAR Section 3.7.2.5, at the following key locations:

- Reactor Building Internal Structure at Elev. 16.9 ft (5.15 m) and 64.0 ft (19.5 m).
- Safeguard Building 1 at Elev. 27 ft (8.1 m) and 69.9 ft (21.0 m).
- Safeguard Building 2/3 at Elev. 27 ft (8.1 m) and 50.5 ft (15.4 m).
- Safeguard Building 4 at Elev. 69.9 ft (21.0 m).
- Containment Building at Elev. 123 ft (37.6 m) and 190 ft (58.0 m).

A comparison of the 5 percent damped ISRS for the CCNPP Unit 3 BE, LB and UB soil profiles with the corresponding peak-broadened Design Certification ISRS show that the certified design bounds the CCNPP Unit 3 seismic demands by a large margin (3.7-25 through 3.7-51). Therefore, the CCNPP Unit 3 site-specific seismic responses are bounded by the U.S. EPR FSAR results. The Seismic Category II vent stack structure is part of the NI common basemat structures. Consequently, the site-specific seismic response of the vent stack is confirmed as well.

The site-specific seismic responses for the Nuclear Auxiliary Building (NAB) and Radioactive Waste Processing Building (RWPB) are within the parameters of Section 3.7 of the U.S. EPR standard design. The seismic responses at the center of basemats of the NAB and RWPB structures were computed from the site-specific SSI analysis for the Nuclear Island common basemat structures described in Section 3.7.2.4. The site-specific response for the NAB is enveloped by U.S. EPR standard design response as shown by comparing the site-specific ISRS (3.7-52 through 3.7-54) at the basemat for NAB to the corresponding U.S. EPR standard design ISRS (3.7-55 through 3.7-57). Similarly, the site-specific response for the RWPB is enveloped by U.S. EPR standard design response as shown by comparing the site-specific ISRS (3.7-58 through 3.7-60) at the basemat for RWPB to the corresponding U.S. EPR standard design ISRS (3.7-61 through 3.7-63).

3.7.2.5.2 EPGB and ESWB

U.S. EPR FSAR Section 3.7.2.5 describes the development of floor response spectra for the EPGB and ESWB. The soil cases are described in U.S. EPR FSAR Table 3.7.1-6 and the ground design response spectra are shown in U.S. EPR FSAR Figures 3.7.1-33 and 3.7.1-34 for the EPGB and ESWB.

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For site-specific confirmatory analysis, ISRS are generated for EPGB and ESWB at locations identified in U.S. EPR FSAR Section 3.7.2.5, using the guidelines described in U.S. EPR FSAR Section 3.7.2.5. The ISRS are however, calculated from 0.2 to 100 Hz, and correspond to the envelope of the ISRS for the site-specific strain-compatible BE, LB and UB soil profiles. For the purposes of confirmatory analyses, 3.7-64 to 3.7-72 show the comparison of 5 percent damped ISRS, which are representative of the response at all damping values, with the corresponding ISRS from U.S. EPR FSAR. The site-specific ISRS for these structures are enveloped by the corresponding design certification ISRS by a large margin, except for frequencies less than approximately 0.3 Hz. Though the maximum site-specific spectral acceleration in this frequency range is 0.07g, the ISRS exceed the certified design ISRS by more than 10 percent in this frequency range. This represents a departure from the U.S. EPR FSAR based on the guidelines specified in U.S. EPR FSAR Section 2.5.2.6. The effects Reconciliation of the accelerations at these low frequency exceedances on EPGB and ESWB are addressed as follows: frequencies is discussed in Section 2.5.2.6.

- The structural reconciliation is addressed in Sections 3.8.4 and 3.8.5.
- The ISRS used to design the systems and components housed within these structures are the envelop of the ISRS shown in U.S. EPR FSAR Figures 3.7.2-101 through 3.7.2-109 and the corresponding site-specific ISRS shown in 3.7-72 through 3.7-64.

3.7.2.5.3 Common Basemat Intake Structures

ISRS at the location of safety-related equipment within the UHS Makeup Water Intake Structure are generated using the SSI model described in Section 3.7.2.4. The ISRS are calculated from 0.1 to 50 Hz, which meets the guidelines provided in RG 1.122, Revision 1 (NRC, 1978). For the UHS Makeup Water Intake Structure, the ISRS are calculated at 0.5 percent, 2 percent, 3 percent, 4 percent, 5 percent, 7 percent and 10 percent damping. The ISRS are enveloped for the site-specific strain-compatible BE, LB and UB soil profiles.

For the UHS Makeup Water Intake Structure, the ISRS are developed at the location of safety-related makeup pumps and facilities, as shown in 3.7-73 through 3.7-78 and at the location of safety-related electrical equipment supported at EL +26.5 ft in the CBIS, and are shown in 3.7-79 through Figure 3.7-81. ISRS will be generated at the support locations of additional safety-related equipment, as required.

3.7.2.6 Three Components of Earthquake Motion

As indicated in Section 3.7.2.4, the SSI analysis of the site-specific Seismic Category I structures is performed using the integrated finite element model, with the input ground motion applied separately in the three directions. Following the methodology described in U.S. EPR FSAR Section 3.7.2.5 for EPGB and ESWB, the ISRS in the UHS Makeup Water Intake Structure are determined by using the Square Root of Sum of Squares (SRSS) of the calculated response spectra in a given direction, due to earthquake motion in the three directions.

The maximum member forces and moments due to the three earthquake motion components are combined using the ASCE 4-98 (ASCE, 2000) "100-40-40" combination rule to obtain the maximum total member forces and moments. The 100-40-40 rule used is consistent with the requirements of RG 1.92, Revision 2 (NRC, 2006).

3.7.2.7 Combination of Modal Responses

No departures or supplements.}

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

The U.S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8:

> A COL applicant that references the U.S. EPR design certification will provide the site-specific separation distances for the Access Building and Turbine Building.

The COL Item is addressed as follows:

The conceptual design information in U.S. EPR FSAR, Tier 2 , Figure 3B-1 provides the separation gaps between the AB and SBs 3 and 4 and between the TB and the NI Common Basemat Structures. This information is incorporated by reference.

The U. S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8 - Access Building:

> A COL applicant that references the U.S. EPR design certification will demonstrate that the response of the Access Building to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions.
>
> [[The Access Building is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure with the exception of sliding and overturning criteria. Because the Access Building does not have a safety function, it may slide or uplift provided that the gap between the Access Building and any Category I structure is adequate to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the Access Building and adjacent Category I structures. The separation gaps between the Access Building and SBs 3 and 4 are 0.98 ft and 1.31 ft, respectively (see Figure 3B-1).]]
>
> For COL applicants that incorporate the conceptual design for the Access Building presented in the U.S. EPR FSAR (i.e ., [[the Access Building is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure with the exception of sliding and overturning criteria]]), this COL item is addressed by demonstrating that the gap between the Access Building and adjacent Category I structures is sufficient to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the Access Building and adjacent Category I structures.

This COL Item is addressed as follows:

{The Access Building is classified as Seismic Category II structure and will be designed to satisfy SRP 3.7.2 Acceptance Criterion 8.C.}

The U. S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8 - Turbine Building:

> A COL applicant that references the U.S. EPR design certification will demonstrate that the response of the TB (including Switchgear Building on the common basemat) to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions.
>
> [[The TB is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equiv alent to that of a Category I structure with the exception of sliding and overturning criteria. Because the TB does not have a safety function, it may slide or uplift provided that the gap between the TB and any Category I structure is adequate to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the TB and adjacent Category I structures. The separation between the TB and N I Common Basemat Structures is approximately 30 ft (see Figure 3B-1).]]
>
> For COL applicants that incorporate the conceptual design for the TB presented in the U.S. EPR FSAR (i .e., [[the TB is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure with the exception of sliding and overturning criteria]]), this COL item is addressed by demonstrating that the gap between the TB and adjacent Category I structures is sufficient to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the TB and adjacent Category I structures.

This COL Item is addressed as follows:

{The Turbine Building and Switchgear Building (also referred to as the Turbine Island (TI) structure) are classified as Seismic Category II structures. These structures were analyzed and designed to the same requirements as other Seismic Category I structures for site-specific SSE loads. This design methodology meets the NUREG 0800 SRP 3.7.2 Acceptance Criterion 8.C.}

The U.S. EPR FSAR includes the following COL Item in Section 3.7.2.8:

> A COL applicant that references the U.S. EPR design certification will provide the seismic design basis for the sources of fire protection water supply for safe plant shutdown in the event of a SSE.

The COL Item is addressed as follows:

The U.S EPR FSAR Section 3.7.2.8 states that the Fire Protection Storage Tanks and Buildings are classified as Conventional Seismic Structures and that RG 1.189 (NRC, 2007) requires that a water supply be provided for manual firefighting in areas containing equipment for safe plant shutdown in the event of a SSE. The U.S. EPR FSAR Section 3.7.2.8 also states the fire protection storage tanks and building are designed to provide system pressure integrity under SSE loading conditions.

In addition to the Seismic Classifications defined in U.S. EPR FSAR Section 3.2.1, a seismic classification of Seismic Category II-SSE is utilized. This designation is utilized to ensure the design basis requirement that Fire Protection SSC are required to remain functional during and following a seismic event to support equipment required to achieve safe shutdown.

Refer to Section 3.2.1 and U.S. EPR FSAR Section 3.2.1 for further discussion of seismic classifications. In addition, Section 3.2.1 categorizes Fire Protection SSC into two categories:

1. SSC that must remain functional during and after an SSE (i.e., Seismic Category II-SSE); and
2. SSC that must remain intact after an SSE without deleterious interaction with Seismic Category I or Seismic Category II-SSE (i.e., Seismic Category II).

Fire Protection SSC required to remain functional during and following a safe shutdown earthquake to support safe shutdown of the plant following a design basis seismic event are designated as Seismic Category II-SSE. The following Fire Protection structures, systems, and components are required to remain functional during and after a seismic event:

1. Fire Water Storage Tanks;
2. Fire Protection Building;
3. Diesel driven fire pumps and their associated sub systems and components, including the diesel fuel oil system;
4. Critical support systems for the Fire Protection Building, i.e., ventilation; and
5. The portions of the fire water piping system and components (including isolation valves) which supply water to the stand pipes in buildings that house the equipment required for safe shutdown of the plant following an SSE.

Manual actions may be required to isolate the portion of the Fire Protection piping system that is not qualified as Seismic Category II-SSE.

U.S. EPR FSAR Section 3.7.2.8 addresses the interaction of the following Non-Seismic Category I structures with Seismic Category I structures:

- Vent Stack
- Nuclear Auxiliary Building
- Access Building
- Turbine Building
- Radioactive Waste Processing Building

{The following CCNPP Unit 3 Non-Seismic Category I structures identified in Table 3.2-1 could also potentially interact with Seismic Category I SSC:

- Buried and above ground Seismic Category II and Seismic Category II-SSE Fire Protection SSC, including Fire Water Storage Tanks and Fire Protection Building.
- Seismic Category II Turbine Building (U.S. EPR FSAR Section 3.7.2.8 also provides conceptual information to address seismic interaction of Turbine Building with the Seismic Category I SSCs)

- Seismic Category II Switchgear Building
- Conventional Seismic Grid Systems Control Building
- Seismic Category II Circulating Water Makeup Intake Structure
- Conventional Seismic Sheet Pile Wall.
- Existing Baffle Wall.

The buried Seismic Category II-SSE Fire Protection SSC identified in Table 3.2-1 are seismically analyzed using the design response spectra identified in Section 3.7.1.1.1.4 for use in the analysis of the Seismic Category I site-specific buried utilities. The analysis of the buried Seismic Category II-SSE fire protection SSC will confirm they remain functional during and following an SSE in accordance with NRC Regulatory Guide 1.189 (NRC, 2007). Section 3.7.3.12 further defines the methodology for the analysis of buried Fire Protection piping. Seismic Category II-SSE buried piping is an embedded commodity that by its nature does not significantly interact with above ground Seismic Category I SSC. The buried Seismic Category II-SSE Fire Protection SSCs are designed to the same requirements as the buried Seismic Category I SSCs.

The above ground Seismic Category II and Seismic Category II-SSE Fire Protection SSC, including Fire Water Storage Tanks and Fire Protection Building, identified in Table 3.2-1 are seismically analyzed utilizing the appropriate design response spectra. Seismic load combinations are developed in accordance with the requirements of ASCE 43-05 (ASCE, 2005) using a limiting acceptance condition for the structure characterized as essentially elastic behavior with no damage (i.e., Limit State D) as specified in the Standard. The analysis of the above ground Seismic Category II-SSE fire protection SSC will confirm they remain functional during and following an SSE in accordance with NRC Regulatory Guide 1.189 (NRC, 2007). The analysis of the above ground Seismic Category II fire protection SSCs will confirm they maintain a pressure boundary after an SSE event.

Table 3.7-11 provides the criteria used to prevent seismic interaction of Turbine Building, Switchgear Building, Circulating Water Makeup Intake Structure and Grid Systems Control Building with other Seismic Category I structures, systems and components (SSCs).

The Seismic Category II Turbine Building and Seismic Category II Switchgear Building together comprise a common Turbine Island (TI) structure and are situated approximately 30 ft (9.1 m) from the NI Common Basemat structures. The Switchgear Building is a steel framed structure. The Turbine Building and Switchgear Building are designed using conventional seismic codes and standards presented in Table 3.7-11, but are also analyzed and designed using Site SSE to prevent seismic interaction with the Seismic Category I SSCs. An evaluation of the site-specific SSE responses will confirm that the separation distance between the TI structure and the Seismic Category I SSCs exceeds the sum of the maximum relative seismic displacement between the structures, construction tolerances and settlement effects by an appropriate factor of safety.

The Conventional Seismic Grid Systems Control Building is located in the Switchyard area, and has a minimum separation distance of approximately 700 ft (213.4 m) from the nearest Seismic Category I SSCs (see Figure 2.1-5). Therefore, potential collapse of this building has no adverse impact on the function of Seismic Category I SSCs. This meets NUREG-0800 Section 3.7.2, Acceptance Criterion 8.A (NRC, 2007a).

The Seismic Category II Circulating Water Makeup Intake Structure is situated between the Seismic Category I Buried Intake Pipes and is comprised of a reinforced concrete embedded structure and an above ground steel structure. The reinforced concrete embedded structure is integrally connected to the Seismic Category I Forebay and is designed to the same requirements as a Seismic Category I structure. The Seismic Category I Buried Intake Pipes are approximately 15 ft (4.6 m) away from the embedded walls of the Circulating Water Makeup Intake Structure. Therefore, there is no possibility of any seismic interaction between the Buried Intake Pipes and the Circulating Water Makeup Intake Structure. Therefore, the design methodology for the reinforced concrete embedded structure meets NUREG-0800 Section 3.7.2, Acceptance Criterion 8.C (NRC, 2007a).

The above ground steel structure is located such that it cannot directly strike any Seismic Category I SSCs. Since the reinforced concrete embedded structure supporting the steel structure is integrally connected to the Seismic Category I Forebay, the reinforced concrete embedded structure is analyzed to demonstrate that the collapse of the steel superstructure does not impair the integrity of Seismic Category I SSCs, nor result in incapacitating injury to control room occupants.

The Conventional Seismic Unit 3 Sheet Pile Wall is located approximately 30 ft (9.1 m) from the north end of the Seismic Category I Buried Intake Pipes. The Sheet Pile Wall will be analyzed and designed using conventional seismic codes and standards but will also be analyzed using Site SSE to prevent any adverse interaction with the Seismic Category I Buried Intake Pipes. The existing Baffle Wall is approximately 46 ft (14.0 m) above the bed of the intake area and is located approximately 50 ft (15.2 m) from the north end of the Seismic Category I Buried Intake Pipes. Therefore, the interaction of the Baffle Wall with the Buried Intake Pipes is not possible.

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

In-structure response spectra are smoothed and the peaks associated with each of the structural frequencies are broadened according to procedure described in RG 1.122 (NRC, 1978). This accounts for uncertainties in the structural frequencies owing to uncertainties in the material properties of the structure and soil, approximation in the modeling techniques used in the seismic analysis and the effect of potential concrete cracking.

3.7.2.10 Use of Constant Vertical Static Factors

No departures or supplements.

3.7.2.11 Method Used to Account for Torsional Effects

For the CBIS, both inherent and accidental torsional effects are accounted for in the seismic design. The inherent torsion effects are built into the 3D finite element model used for the SSI analysis.

The seismic inertia force at each story level is calculated using the maximum absolute structural accelerations in each horizontal direction, provided in Table 3.7-10, and the horizontal mass at that level. The accidental torsional moment is determined as the story inertia force times a moment arm equal to ±5 percent of the building plan dimension in the perpendicular direction, in accordance with NUREG-0800 Section 3.7.2, Acceptance Criterion 11 (NRC, 2007a). These moments are then used to calculate the in-plane shear forces in the walls, which are used for structural design. The responses from earthquakes in three orthogonal directions are combined in accordance with the co-directional response combination provisions of FSAR Section 3.7.2.6.

3.7.2.12 Comparison of Responses

As multiple seismic analysis methods are not employed for the site-specific Seismic Category I structures, a comparison of responses is not applicable.

3.7.2.13 Methods for Seismic Analysis of Category I Dams

No departures or supplements.

3.7.2.14 Determination of Dynamic Stability of Seismic Category I Structures

3.7.2.14.1 Nuclear Island Common Basemat Structures

The methodology to perform dynamic stability evaluation of the Nuclear Island Common Basemat Structures is incorporated by reference to U.S. EPR Section 3.7.2.14.

3.7.2.14.2 EPGB and ESWB

The stability of the EPGB and ESWB for seismic loading is determined using the stability load combinations provided in NUREG-0800 Section 3.8.5, Acceptance Criteria 3 (NRC, 2007a).

For determination of seismic stability, the overturning moments about each of the four edges of the basemat and sliding forces at the bottom of the basemat are computed by using the response time histories of reactions at the basemat nodes. These responses include the effects of seismic forces, static and dynamic lateral earth pressures, and hydrostatic and hydrodynamic forces. The following steps are used to assess the seismic stability:

i. The response time histories of reaction forces for each basemat node are obtained for each Site SSE direction and soil profile (i.e., BE, LB and UB as described in section 3.7.2.4.3). Three reaction forces are obtained for each earthquake direction; therefore nine response time histories of reaction forces are reported per soil profile at each basemat node.

ii. The response time histories of total force are calculated in the vertical and two horizontal directions for each soil profile. The total force in a particular direction is calculated by algebraic addition of nodal reactions in that direction due to earthquake in each direction.

iii. The response time history of total sliding force is calculated for each soil profile. The sliding force is calculated as the magnitude of the vector sum of the total forces in the two horizontal directions.

iv. The response time histories of seismic overturning moment are calculated about each of the four edges of the basemat for each soil profile. The overturning moment about a particular edge is calculated by algebraic sum of the overturning moments about that edge from each nodal reaction due to earthquake in each direction.

v. Evaluation of the sliding, overturning and bearing seismic stability of each structure is performed for each soil profile and each point in time.

The loads considered in the calculation of structural mass in the seismic SSI analysis, which includes the self weight of the structure, weight of the permanent equipment and contained water during normal operation, 25% of the design live load and 75% of the design snow load are consistently used to determine the restoring moments. The vertical force calculated in

Step ii is accounted for during the calculation of sliding resistance. Results of dynamic stability are reported in Appendix 3E.

3.7.2.14.3 Seismic Stability of Common Basemat Intake Structures (CBIS)

The stability of the CBIS Building for seismic loading is determined using the stability load combinations provided in NUREG-0800 Section 3.8.5, Acceptance Criteria 3 (NRC, 2007a), listed as Load Combination 7 in FSAR Table 3E-1.

For determination of seismic stability of the CBIS, the seismically induced normal and shear stresses at the base of the CBIS foundation are computed and compared with the restoring stresses from the self weight of the structure.

The seismic reaction stresses at the CBIS foundation-soil interface are computed at selected locations using 3D brick elements modeled at the base of the CBIS foundation. The seismic normal and shear stresses at the bottom of the basemat are computed by using the response time histories of reaction stresses at the selected basemat locations. These responses include the effects of seismic forces, dynamic lateral earth pressures, and hydrodynamic forces.

The stabilizing forces for the CBIS are considered from the self weight of the intake structure and static earth pressure. The resultant stabilizing stresses are obtained from PLAXIS 3D analysis of the CBIS. PLAXIS 3D analysis considered the self weight of the intake structure, static earth pressures, and the uplift effect of the ground water at the base of the basemat. The effective shear resistance of the soil is computed using PLAXIS 3D output and the vertical seismic load on the CBIS basemat.

The following steps are used to assess the seismic stability of the CBIS:

i. The response time histories of stresses at selected locations of the basemat are obtained for each site SSE direction and soil profile (i.e., BE, LB and UB) from the seismic SSI analysis. Three reaction stresses are obtained for each earthquake direction; therefore nine response time histories of reaction stresses are reported per soil profile.

ii. The response time histories of normal and shear stresses are calculated in the vertical and two horizontal directions for each soil profile. The total stress in a particular direction is calculated by algebraic addition of the stresses in that direction due to earthquake in each direction.

iii. The response time history of total sliding shear stress is calculated for each soil profile. The sliding shear stress is calculated as the magnitude of the vector sum of the shear stresses in the two horizontal directions.

iv. Evaluation of the seismic stability for sliding and uplifting/overturning of the CBIS is performed for each soil profile (BE, LB and UB) at each point in time by computing the factors of safety as the ratio of the restoring stresses of the CBIS to the corresponding seismically induced stresses.

The factors of safety evaluated for the seismic stability are compared with the minimum required factors of safety specified in U.S. EPR FSAR Table 3.8-11. According to this reference, the minimum required factors of safety for sliding and overturning associated with Safe Shutdown Earthquake (E', Seismic Category I foundations) loading combination is 1.1. As a result the CBIS are evaluated to be safe against sliding and overturning due to seismic loads. Results of dynamic stability are reported in Appendix 3E.

3.7.2.15 Analysis Procedure for Damping

The structure and soil damping used in SSI analyses of site-specific Seismic Category I structures are described in Sections 3.7.2.4.2.3 and 3.7.2.4.3.3.

3.7.2.16 References

{**ACI, 2006.** Seismic Design of Liquid-Containing Concrete Structures, ACI 350.3-06, American Concrete Institute, 2006.

ASCE, 2000. Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE Standard 4-98, American Society of Civil Engineers, 2000.

ASCE, 2005. Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, ASCE 43-05, American Society of Civil Engineers, January 2005.

NRC, 1973. Design Response Spectra for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.60, U.S. Nuclear Regulatory Commission, December 1973.

NRC, 1978. Development of Floor Design Response Spectra for Seismic Design of Floor-Supported equipment or Components, Regulatory Guide 1.122, U.S. Nuclear Regulatory commission, February, 1978.

NRC, 2006. Combining Modal Responses and Spatial Components in Seismic Response Analysis, Regulatory Guide 1.92 Revision 2, U.S. Nuclear Regulatory Commission, July 2006.

NRC, 2007. Fire Protection for Nuclear Power Plants, Regulatory Guide 1.189, Revision 1, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2007a. Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants, NUREG-0800, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2008. Earthquake Engineering Criteria for Nuclear Power Plants, Title 10, Code of Federal Regulations, Part 50, Appendix S, U. S. Nuclear Regulatory Commission, February 2008.}

3.7.3 Seismic Subsystem Analysis

No departures or supplements.

3.7.3.1 Seismic Analysis Methods

No departures or supplements.

3.7.3.2 Determination of Number of Earthquake Cycles

No departures or supplements.

3.7.3.3 Procedures Used for Analytical Modeling

{No departures or supplements.}

3.7.3.4 Basis for Selection of Frequencies

{No departures or supplements.}

3.7.3.5 Analysis Procedure for Damping

{No departures or supplements.}

3.7.3.6 Three Components of Earthquake Motion

No departures or supplements.

3.7.3.7 Combination of Modal Responses

No departures or supplements.

3.7.3.8 Interaction of Non-Seismic Category I Subsystems

No departures or supplements.

3.7.3.9 Multiply-Supported Equipment and Components with Distinct Inputs

No departures or supplements.

3.7.3.10 Use of Equivalent Vertical Static Factors

No departures or supplements.

3.7.3.11 Torsional Effects of Eccentric Masses

No departures or supplements.

3.7.3.12 Buried Seismic Category I Piping and Conduits

{For CCNPP Unit 3, a buried duct bank refers to multiple PVC electrical conduits encased in reinforced concrete.

The seismic analysis and design of Seismic Category I buried reinforced concrete electrical duct banks is in accordance with IEEE 628-2001 (R2006) (IEEE, 2001), ASCE 4-98 (ASCE, 2000) and ACI 349-01(ACI, 2001), including supplemental guidance of Regulatory Guide 1.142 (NRC, 2001).

Side walls of electrical manholes are analyzed for seismic waves traveling through the surrounding soil in accordance with the requirements of ASCE 4-98 (ASCE, 2000), including dynamic soil pressures.

Seismic Category I buried Essential Service Water Pipes, Seismic Category I buried Intake Pipes and Seismic Category II and Seismic Category II-SSE buried Fire Protection pipe are analyzed for the effects of seismic waves traveling through the surrounding soil in accordance with the specific requirements of ASCE 4-98 (ASCE, 2000):

- Long, straight buried pipe sections, remote from bends or anchor points, are designed assuming no relative motion between the flexible structure and the ground (i.e. the structure conforms to the ground motion).
- The effects of bends and differential displacement at connections to buildings are evaluated using equations for beams on elastic foundations, and subsequently combined with the buried pipe axial stress.

For long straight sections of buried pipe, maximum axial strain and curvature are calculated per equations contained in ASCE 4-98 (ASCE, 2000). These equations reflect seismic wave propagation and incorporate the material's modulus of elasticity to determine the corresponding maximum axial and bending stresses. The procedure combines stresses from compression, shear and surface waves by the square root of the sum of the squares (SRSS) method. Maximum stresses for each wave type are then combined using the SRSS method. Subsequently, seismic stresses are combined with stresses from other loading conditions, e.g., long-term surcharge loading.

For straight sections of buried pipe, the transfer of axial strain from the soil to the buried structure is limited by the frictional resistance developed. Consequently, axial stresses may be reduced by consideration of such slippage effects, as appropriate.

The seismic analysis of bends of buried pipe is based on the equations developed for beams on elastic foundations. Specifically, the transverse leg is assumed to deform as a beam on an elastic foundation due to the axial force in the longitudinal leg. The spring constant at the bend depends on the stiffness of the longitudinal and transverse legs as well as the degree of fixity at the bend and ends of the legs.

Seismic analysis of restrained segments of buried pipe utilizes guidance provided in Appendix VII, Procedures for the Design of Restrained Underground Piping, of ASME B31.1-2004 (ASME, 2004).}

3.7.3.13 Methods for Seismic Analysis of Category I Concrete Dams

The U.S. EPR FSAR includes the following COL Item in Section 3.7.3.13:

> A COL applicant that references the U.S. EPR design certification will provide a description of methods for seismic analysis of site-specific Category I concrete dams, if applicable.

This COL Item is addressed as follows:

{No Seismic Category I dams will be used at CCNPP Unit 3.}

3.7.3.14 Methods for Seismic Analysis of Aboveground Tanks

No departures or supplements.

3.7.3.15 References

{**ACI, 2001.** Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures, ACI 349-01/349-R01, American Concrete Institute, 2001.

ASCE, 2000. Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE 4-98, American Society of Civil Engineers, 2000.

ASME, 2004. Procedures for the Design of Restrained Underground Piping, Appendix VII, Power Piping, ASME B31.1-2004, American Society of Mechanical Engineers, 2004.

IEEE, 2001. IEEE Standard Criteria for the Design, Installation, and Qualification of Raceway Systems for Class 1E Circuits for Nuclear Power Generating Stations, IEEE 628-2001, IEEE, 2001.

NRC, 2001. Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments), Regulatory Guide 1.142, U.S. Nuclear Regulatory Commission, November 2001.}

3.7.4 Seismic Instrumentation

No departures or supplements.

3.7.4.1 Comparison with NRC Regulatory Guide 1.12

No departures or supplements.

3.7.4.2 Location and Description of Instrumentation

The U.S. EPR FSAR includes the following COL Item in Section 3.7.4.2:

> A COL applicant that references the U.S. EPR design certification will determine whether essentially the same seismic response from a given earthquake is expected at each of the units in a multi-unit site or instrument each unit. In the event that only one unit is instrumented, annunciation shall be provided to each control room.

This COL Item is addressed as follows:

{CCNPP Unit 3 is a single unit, U.S. EPR facility. Annunciation of the seismic instrumentation for CCNPP Unit 3 will be provided in the CCNPP Unit 3 main control room.}

3.7.4.2.1 Field Mounted Sensors

The U.S. EPR FSAR includes the following COL Item in Section 3.7.4.2.1:

> A COL applicant that references the U.S. EPR design certification will determine a location for the free-field acceleration sensor such that the effects associated with surface features, buildings, and components on the recordings of ground motion are insignificant. The acceleration sensor must be based on material representative of that upon which the Nuclear Island (NI) and other Seismic Category I structures are founded.

This COL Item is addressed as follows:

{Calvert Cliffs 3 Nuclear Project, LLC and UniStar Nuclear Operating Services, LLC} shall determine the location for the free-field acceleration sensor in accordance with the guidance provided in Regulatory Guide 1.12 prior to fuel load. The location will be sufficiently distant from nearby structures that may have significant influence on the recorded free-field seismic motion. The free-field acceleration sensor will be located on a base mat that is founded on material that is representative of that upon which the NI and other Seismic Category I structures are founded. The sensor will be protected from accidental impact, and will be readily accessible for surveillance, maintenance, and repair activities. The sensor will be rigidly mounted in alignment with the orthogonal axes assumed for seismic analysis. To maintain occupational radiation exposures ALARA, the free-field acceleration sensor location will be sufficiently distant from radiation sources such that there is minimal occupational exposure expected during normal operating modes.

3.7.4.2.2 System Equipment Cabinet

No departures or supplements.

3.7.4.2.3 Seismic Recorder(s)

No departures or supplements.

3.7.4.2.4 Central Controller

No departures or supplements.

3.7.4.2.5 Power Supplies

No departures of supplements.

3.7.4.3 Control Room Operator Notification

No departures or supplements.

3.7.4.4 Comparison with Regulatory Guide 1.166

Post-earthquake actions and an assessment of the damage potential of the event using the EPRI-developed OBE Exceedance Criteria follow the guidance of EPRI reports NP-5930 (EPRI, 1988) and NP-6695 (EPRI, 1989), as endorsed by the U.S. Nuclear Regulatory Commission in Regulatory Guide 1.166 (NRC, 1997a) and Regulatory Guide 1.167 (NRC, 1997b). OBE Exceedance Criteria is based on a threshold response spectrum ordinate check and a CAV check using recorded motions from the free-field acceleration sensor. If the respective OBE ground motion is exceeded in a potentially damaging frequency range or significant plant damage occurs, the plant must be shutdown following plant procedures. The shutdown OBE for CCNPP Unit 3, which is described in Section 3.7.1.1, is the composite earthquake which consists of one-third site-specific SSE (anchored at 0.05g) and EUR Soft Soil spectrum anchored at 0.10g in the low frequency (approximately 0.36Hz and below).

3.7.4.5 Instrument Surveillance

No departures or supplements.

3.7.4.6 Program Implementation

No departures or supplements.

3.7.4.7 References

{**ASCE, 2005.** Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, ASCE 43-05, American Society of Civil Engineers, January 2005.

EPRI, 1988. A Criterion for Determining Exceedance of the Operating Basis Earthquake, NP-5930, Electric Power Research Institute, July 1988.

EPRI, 1989. Guidelines for Nuclear Plant Response to an Earthquake, NP-6695, Electric Power Research Institute, December 1989.

NRC, 1997a. Pre-Earthquake Planning and Immediate Nuclear Power Plant Operator Post-Earthquake Actions, Regulatory Guide 1.166, Revision 0, U. S. Nuclear Regulatory Commission, March 1997.

NRC, 1997b. Restart of a Nuclear Power Plant Shut Down by a Seismic Event, Regulatory Guide 1.167, Revision 0, U. S. Nuclear Regulatory Commission, March 1997.}

Table 3.7-1— {Site SSE (Horizontal and Vertical) Spectral Accelerations at 5% Damping}

Freq (Hz)	Spectral Acceleration (g)
0.1	7.53E-03
0.125	1.18E-02
0.15	1.69E-02
0.2	3.01E-02
0.3	5.47E-02
0.4	6.93E-02
0.5	8.33E-02
0.6	9.67E-02
0.7	1.21E-01
0.8	1.43E-01
0.9	1.61E-01
1	1.79E-01
1.25	2.23E-01
1.5	2.68E-01
2	3.57E-01
2.5	4.46E-01
3	4.50E-01
4	4.50E-01
5	4.50E-01
6	4.50E-01
7	4.50E-01
8	4.48E-01
9	4.11E-01
10	3.79E-01
12.5	3.18E-01
15	2.76E-01
20	2.21E-01
25	1.86E-01
30	1.62E-01
35	1.50E-01
40	1.50E-01
45	1.50E-01
50	1.50E-01
60	1.50E-01
70	1.50E-01
80	1.50E-01
90	1.50E-01
100	1.50E-01

Table 3.7-2— {CCNPP Unit 3 Best Estimate Soil for SSI Analysis of NI Common Basemat Structure}

(Page 1 of 2)

Layer No.	Layer Thickness (m)	Weight Density (kN/m³)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
1	1.22	18.85	451.9	1524.0	0.0147	0.0147	74	1.22
2	1.52	18.85	451.4	1524.0	0.0148	0.0148	59	2.74
3	1.52	18.85	450.6	1524.0	0.0150	0.0150	59	4.27
4	2.29	18.85	516.3	1712.5	0.0147	0.0147	45	6.55
5	2.29	18.85	515.6	1709.9	0.0148	0.0148	45	8.84
6	1.14	18.85	333.0	1697.9	0.0172	0.0172	58	9.98
7	1.14	18.85	333.0	1697.9	0.0172	0.0172	58	11.13
8	1.14	18.85	331.9	1692.5	0.0174	0.0174	58	12.27
9	1.14	18.85	331.9	1692.5	0.0174	0.0174	58	13.41
10	2.29	18.85	497.4	1649.8	0.0151	0.0151	44	15.70
11	2.29	18.85	497.0	1648.3	0.0152	0.0152	43	17.98
12	1.07	17.28	364.8	1533.5	0.0171	0.0171	68	19.05
13	1.07	17.28	364.8	1533.5	0.0171	0.0171	68	20.12
14	1.07	17.28	363.6	1528.4	0.0173	0.0173	68	21.18
15	1.07	17.28	363.6	1528.4	0.0173	0.0173	68	22.25
16	1.07	17.28	363.0	1525.8	0.0174	0.0174	68	23.32
17	1.07	17.28	363.0	1525.8	0.0174	0.0174	68	24.38
18	1.07	17.28	362.6	1524.0	0.0175	0.0175	68	25.45
19	1.07	17.28	362.6	1524.0	0.0175	0.0175	68	26.52
20	1.07	17.28	361.9	1524.0	0.0176	0.0176	68	27.58
21	1.07	17.28	361.9	1524.0	0.0176	0.0176	68	28.65
22	1.52	17.28	374.8	1575.1	0.0118	0.0118	49	30.18
23	1.52	17.28	374.8	1575.1	0.0118	0.0118	49	31.70
24	1.52	17.28	374.3	1573.2	0.0118	0.0118	49	33.22
25	1.52	17.28	374.3	1573.2	0.0118	0.0118	49	34.75
26	1.52	17.28	372.0	1563.6	0.0118	0.0118	49	36.27
27	1.52	17.28	372.0	1563.6	0.0118	0.0118	49	37.80
28	1.52	17.28	371.7	1562.3	0.0118	0.0118	49	39.32
29	1.52	17.28	371.7	1562.3	0.0118	0.0118	49	40.84
30	1.52	17.28	371.5	1561.7	0.0118	0.0118	49	42.37
31	1.52	17.28	371.5	1561.7	0.0118	0.0118	49	43.89
32	1.52	17.28	371.4	1561.0	0.0118	0.0118	49	45.42
33	1.52	17.28	371.4	1561.0	0.0118	0.0118	49	46.94
34	1.52	17.28	371.2	1560.4	0.0119	0.0119	49	48.46
35	1.52	17.28	371.2	1560.4	0.0119	0.0119	49	49.99
36	1.52	17.28	371.1	1559.8	0.0119	0.0119	49	51.51
37	1.52	17.28	371.1	1559.8	0.0119	0.0119	49	53.04

Table 3.7-2— {CCNPP Unit 3 Best Estimate Soil for SSI Analysis of NI Common Basemat Structure}
(Page 2 of 2)

Layer No.	Layer Thickness (m)	Weight Density (kN/m^3)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
38	1.52	17.28	370.9	1559.1	0.0119	0.0119	49	54.56
39	1.52	17.28	370.9	1559.1	0.0119	0.0119	49	56.08
40	1.52	17.28	370.8	1558.5	0.0119	0.0119	49	57.61
41	1.52	17.28	370.8	1558.5	0.0119	0.0119	49	59.13
42	1.52	17.28	370.6	1557.8	0.0119	0.0119	49	60.66
43	1.52	17.28	370.6	1557.8	0.0119	0.0119	49	62.18
44	1.52	17.28	370.6	1557.8	0.0119	0.0119	49	63.70
45	1.52	17.28	370.6	1557.8	0.0119	0.0119	49	65.23
46	1.52	17.28	370.3	1556.6	0.0119	0.0119	49	66.75
47	1.52	17.28	370.3	1556.6	0.0119	0.0119	49	68.28
48	1.52	17.28	370.0	1555.3	0.0119	0.0119	49	69.80
49	1.52	17.28	370.0	1555.3	0.0119	0.0119	49	71.32
50	1.52	17.28	370.0	1555.3	0.0119	0.0119	49	72.85
51	1.52	17.28	370.0	1555.3	0.0119	0.0119	49	74.37
52	1.52	18.85	533.9	1770.6	0.0157	0.0157	70	75.90
53	1.52	18.85	533.9	1770.6	0.0157	0.0157	70	77.42
54	1.52	18.85	533.7	1770.1	0.0157	0.0157	70	78.94
55	1.52	18.85	533.7	1770.1	0.0157	0.0157	70	80.47
56	3.05	18.85	651.2	1753.4	0.0150	0.0150	43	83.52
57	3.05	18.85	622.4	1776.1	0.0152	0.0152	41	86.56
58	3.05	18.85	622.2	1775.6	0.0152	0.0152	41	89.61
59	3.05	18.85	621.9	1774.7	0.0153	0.0153	41	92.66
60	3.05	18.85	621.6	1773.9	0.0153	0.0153	41	95.71
61	2.74	18.85	630.6	1926.6	0.0153	0.0153	46	98.45
62	2.74	18.85	630.6	1926.6	0.0153	0.0153	46	101.19
63	2.74	18.85	630.3	1925.7	0.0153	0.0153	46	103.94
64	2.74	18.85	630.9	1927.5	0.0153	0.0153	46	106.68
65	2.74	18.85	630.8	1927.1	0.0153	0.0153	46	109.42
66	3.05	18.85	673.8	2058.4	0.0151	0.0151	44	112.47
67	3.05	18.85	673.8	2058.4	0.0151	0.0151	44	115.52
68	3.05	18.85	673.6	1813.7	0.0151	0.0151	44	118.57
69	3.05	18.85	674.5	1652.2	0.0151	0.0151	44	121.62
70	3.05	18.85	674.2	1651.5	0.0151	0.0151	44	124.66
71	3.05	18.85	674.2	1651.5	0.0152	0.0152	44	127.71
72	3.05	18.85	674.1	1651.1	0.0152	0.0152	44	130.76
73	3.05	18.85	673.8	1650.4	0.0152	0.0152	44	133.81
	Halfspace	18.07	673.7	1650.1	0.0155	0.0155		

Table 3.7-3— {CCNPP Unit 3 Lower Bound Soil for SSI Analysis of NI Common Basemat Structure}

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Layer No.	Layer Thickness (m)	Weight Density (kN/m^3)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
1	1.22	18.85	343.0	1524.0	0.0211	0.0211	56	1.22
2	1.52	18.85	342.4	1524.0	0.0212	0.0212	45	2.74
3	1.52	18.85	341.6	1524.0	0.0214	0.0214	45	4.27
4	1.14	18.85	383.6	1524.0	0.0207	0.0207	67	5.41
5	1.14	18.85	383.6	1524.0	0.0207	0.0207	67	6.55
6	1.14	18.85	382.6	1524.0	0.0209	0.0209	67	7.70
7	1.14	18.85	382.6	1524.0	0.0209	0.0209	67	8.84
8	1.14	18.85	243.9	1243.6	0.0243	0.0243	43	9.98
9	1.14	18.85	243.9	1243.6	0.0243	0.0243	43	11.13
10	1.14	18.85	242.8	1238.1	0.0246	0.0246	42	12.27
11	1.14	18.85	242.8	1238.1	0.0246	0.0246	42	13.41
12	1.14	18.85	402.5	1524.0	0.0215	0.0215	70	14.55
13	1.14	18.85	402.5	1524.0	0.0215	0.0215	70	15.70
14	1.14	18.85	402.0	1524.0	0.0217	0.0217	70	16.84
15	1.14	18.85	402.0	1524.0	0.0217	0.0217	70	17.98
16	1.07	17.28	297.9	1519.0	0.0240	0.0240	56	19.05
17	1.07	17.28	297.9	1519.0	0.0240	0.0240	56	20.12
18	1.07	17.28	296.9	1513.9	0.0242	0.0242	56	21.18
19	1.07	17.28	296.9	1513.9	0.0242	0.0242	56	22.25
20	1.07	17.28	296.4	1511.3	0.0244	0.0244	56	23.32
21	1.07	17.28	296.4	1511.3	0.0244	0.0244	56	24.38
22	1.07	17.28	296.0	1509.4	0.0245	0.0245	55	25.45
23	1.07	17.28	296.0	1509.4	0.0245	0.0245	55	26.52
24	1.07	17.28	295.5	1506.9	0.0247	0.0247	55	27.58
25	1.07	17.28	295.5	1506.9	0.0247	0.0247	55	28.65
26	1.52	17.28	306.0	1524.0	0.0174	0.0174	40	30.18
27	1.52	17.28	306.0	1524.0	0.0174	0.0174	40	31.70
28	1.52	17.28	305.6	1524.0	0.0174	0.0174	40	33.22
29	1.52	17.28	305.6	1524.0	0.0174	0.0174	40	34.75
30	1.52	17.28	303.7	1524.0	0.0174	0.0174	40	36.27
31	1.52	17.28	303.7	1524.0	0.0174	0.0174	40	37.80
32	1.52	17.28	303.5	1524.0	0.0174	0.0174	40	39.32
33	1.52	17.28	303.5	1524.0	0.0174	0.0174	40	40.84
34	1.52	17.28	303.4	1524.0	0.0175	0.0175	40	42.37
35	1.52	17.28	303.4	1524.0	0.0175	0.0175	40	43.89
36	1.52	17.28	303.2	1524.0	0.0175	0.0175	40	45.42
37	1.52	17.28	303.2	1524.0	0.0175	0.0175	40	46.94

Table 3.7-3— {CCNPP Unit 3 Lower Bound Soil for SSI Analysis of NI Common Basemat Structure}

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Layer No.	Layer Thickness (m)	Weight Density (kN/m^3)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
38	1.52	17.28	303.1	1524.0	0.0175	0.0175	40	48.46
39	1.52	17.28	303.1	1524.0	0.0175	0.0175	40	49.99
40	1.52	17.28	303.0	1524.0	0.0175	0.0175	40	51.51
41	1.52	17.28	303.0	1524.0	0.0175	0.0175	40	53.04
42	1.52	17.28	302.9	1524.0	0.0175	0.0175	40	54.56
43	1.52	17.28	302.9	1524.0	0.0175	0.0175	40	56.08
44	1.52	17.28	302.7	1524.0	0.0175	0.0175	40	57.61
45	1.52	17.28	302.7	1524.0	0.0175	0.0175	40	59.13
46	1.52	17.28	302.6	1524.0	0.0175	0.0175	40	60.66
47	1.52	17.28	302.6	1524.0	0.0175	0.0175	40	62.18
48	1.52	17.28	302.6	1524.0	0.0175	0.0175	40	63.70
49	1.52	17.28	302.6	1524.0	0.0175	0.0175	40	65.23
50	1.52	17.28	302.4	1524.0	0.0176	0.0176	40	66.75
51	1.52	17.28	302.4	1524.0	0.0176	0.0176	40	68.28
52	1.52	17.28	302.1	1524.0	0.0176	0.0176	40	69.80
53	1.52	17.28	302.1	1524.0	0.0176	0.0176	40	71.32
54	1.52	17.28	302.1	1524.0	0.0176	0.0176	40	72.85
55	1.52	17.28	302.1	1524.0	0.0176	0.0176	40	74.37
56	1.52	18.85	435.9	1524.0	0.0221	0.0221	57	75.90
57	1.52	18.85	435.9	1524.0	0.0221	0.0221	57	77.42
58	1.52	18.85	435.8	1524.0	0.0221	0.0221	57	78.94
59	1.52	18.85	435.8	1524.0	0.0221	0.0221	57	80.47
60	1.52	18.85	531.7	1524.0	0.0212	0.0212	70	81.99
61	1.52	18.85	531.7	1524.0	0.0212	0.0212	70	83.52
62	1.52	18.85	508.2	1524.0	0.0212	0.0212	67	85.04
63	1.52	18.85	508.2	1524.0	0.0212	0.0212	67	86.56
64	1.52	18.85	508.1	1524.0	0.0212	0.0212	67	88.09
65	1.52	18.85	508.1	1524.0	0.0212	0.0212	67	89.61
66	1.52	18.85	507.8	1524.0	0.0213	0.0213	67	91.14
67	1.52	18.85	507.8	1524.0	0.0213	0.0213	67	92.66
68	1.52	18.85	507.6	1524.0	0.0213	0.0213	67	94.18
69	1.52	18.85	507.6	1524.0	0.0213	0.0213	67	95.71
70	2.74	18.85	514.9	1573.1	0.0213	0.0213	38	98.45
71	2.74	18.85	514.9	1573.1	0.0213	0.0213	38	101.19
72	2.74	18.85	514.7	1572.3	0.0214	0.0214	38	103.94
73	2.74	18.85	515.2	1573.8	0.0214	0.0214	38	106.68
74	2.74	18.85	515.0	1573.4	0.0214	0.0214	38	109.42

Table 3.7-3— {CCNPP Unit 3 Lower Bound Soil for SSI Analysis of NI Common Basemat Structure}
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Layer No.	Layer Thickness (m)	Weight Density (kN/m^3)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
75	3.05	18.85	550.1	1680.6	0.0213	0.0213	36	112.47
76	3.05	18.85	550.1	1680.6	0.0213	0.0213	36	115.52
77	3.05	18.85	550.0	1524.0	0.0213	0.0213	36	118.57
78	3.05	18.85	550.7	1524.0	0.0213	0.0213	36	121.62
79	3.05	18.85	550.5	1524.0	0.0213	0.0213	36	124.66
80	3.05	18.85	550.5	1524.0	0.0214	0.0214	36	127.71
81	3.05	18.85	550.4	1524.0	0.0214	0.0214	36	130.76
82	3.05	18.85	550.1	1524.0	0.0214	0.0214	36	133.81
	Halfspace	18.07	550.0	1524.0	0.0218	0.0218		

Table 3.7-4— {CCNPP Unit 3 Upper Bound Soil for SSI Analysis of NI Common Basemat Structure}

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Layer No.	Layer Thickness (m)	Weight Density (kN/m^3)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
1	1.22	18.85	595.3	1698.7	0.0103	0.0103	98	1.22
2	1.52	18.85	595.1	1698.1	0.0104	0.0104	78	2.74
3	1.52	18.85	594.5	1696.5	0.0105	0.0105	78	4.27
4	2.29	18.85	695.0	2305.0	0.0104	0.0104	61	6.55
5	2.29	18.85	694.7	2304.2	0.0105	0.0105	61	8.84
6	1.14	18.85	454.7	2318.3	0.0121	0.0121	80	9.98
7	1.14	18.85	454.7	2318.3	0.0121	0.0121	80	11.13
8	1.14	18.85	453.7	2313.6	0.0123	0.0123	79	12.27
9	1.14	18.85	453.7	2313.6	0.0123	0.0123	79	13.41
10	2.29	18.85	614.8	2039.0	0.0106	0.0106	54	15.70
11	2.29	18.85	614.5	2037.9	0.0106	0.0106	54	17.98
12	1.07	17.28	446.8	1878.1	0.0122	0.0122	84	19.05
13	1.07	17.28	446.8	1878.1	0.0122	0.0122	84	20.12
14	1.07	17.28	445.3	1871.9	0.0123	0.0123	83	21.18
15	1.07	17.28	445.3	1871.9	0.0123	0.0123	83	22.25
16	1.07	17.28	444.6	1868.7	0.0124	0.0124	83	23.32
17	1.07	17.28	444.6	1868.7	0.0124	0.0124	83	24.38
18	1.07	17.28	444.0	1866.4	0.0125	0.0125	83	25.45
19	1.07	17.28	444.0	1866.4	0.0125	0.0125	83	26.52
20	1.07	17.28	443.3	1863.2	0.0126	0.0126	83	27.58
21	1.07	17.28	443.3	1863.2	0.0126	0.0126	83	28.65
22	1.52	17.28	459.0	1929.1	0.0080	0.0080	60	30.18
23	1.52	17.28	459.0	1929.1	0.0080	0.0080	60	31.70
24	1.52	17.28	458.4	1926.8	0.0080	0.0080	60	33.22
25	1.52	17.28	458.4	1926.8	0.0080	0.0080	60	34.75
26	1.52	17.28	455.6	1915.0	0.0080	0.0080	60	36.27
27	1.52	17.28	455.6	1915.0	0.0080	0.0080	60	37.80
28	1.52	17.28	455.2	1913.5	0.0080	0.0080	60	39.32
29	1.52	17.28	455.2	1913.5	0.0080	0.0080	60	40.84
30	1.52	17.28	455.1	1912.7	0.0080	0.0080	60	42.37
31	1.52	17.28	455.1	1912.7	0.0080	0.0080	60	43.89
32	1.52	17.28	454.9	1911.9	0.0080	0.0080	60	45.42
33	1.52	17.28	454.9	1911.9	0.0080	0.0080	60	46.94
34	1.52	17.28	454.7	1911.1	0.0080	0.0080	60	48.46
35	1.52	17.28	454.7	1911.1	0.0080	0.0080	60	49.99
36	1.52	17.28	454.5	1910.3	0.0081	0.0081	60	51.51
37	1.52	17.28	454.5	1910.3	0.0081	0.0081	60	53.04

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Table 3.7-4— {CCNPP Unit 3 Upper Bound Soil for SSI Analysis of NI Common Basemat Structure}
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Layer No.	Layer Thickness (m)	Weight Density (kN/m³)	S-Wave Velocity (m/s)	P-Wave Velocity (m/s)	S-Damp Ratio	P-Damp Ratio	Passing Frequency (Hz)	Depth (m)
38	1.52	17.28	454.3	1909.5	0.0081	0.0081	60	54.56
39	1.52	17.28	454.3	1909.5	0.0081	0.0081	60	56.08
40	1.52	17.28	454.1	1908.7	0.0081	0.0081	60	57.61
41	1.52	17.28	454.1	1908.7	0.0081	0.0081	60	59.13
42	1.52	17.28	453.9	1908.0	0.0081	0.0081	60	60.66
43	1.52	17.28	453.9	1908.0	0.0081	0.0081	60	62.18
44	1.52	17.28	453.9	1908.0	0.0081	0.0081	60	63.70
45	1.52	17.28	453.9	1908.0	0.0081	0.0081	60	65.23
46	1.52	17.28	453.6	1906.4	0.0081	0.0081	60	66.75
47	1.52	17.28	453.6	1906.4	0.0081	0.0081	60	68.28
48	1.52	17.28	453.2	1904.8	0.0081	0.0081	59	69.80
49	1.52	17.28	453.2	1904.8	0.0081	0.0081	59	71.32
50	1.52	17.28	453.2	1904.8	0.0081	0.0081	59	72.85
51	1.52	17.28	453.2	1904.8	0.0081	0.0081	59	74.37
52	1.52	18.85	653.8	2168.5	0.0112	0.0112	86	75.90
53	1.52	18.85	653.8	2168.5	0.0112	0.0112	86	77.42
54	1.52	18.85	653.6	2167.9	0.0112	0.0112	86	78.94
55	1.52	18.85	653.6	2167.9	0.0112	0.0112	86	80.47
56	3.05	18.85	797.6	2147.5	0.0106	0.0106	52	83.52
57	3.05	18.85	762.3	2175.2	0.0109	0.0109	50	86.56
58	3.05	18.85	762.1	2174.7	0.0109	0.0109	50	89.61
59	3.05	18.85	761.7	2173.6	0.0109	0.0109	50	92.66
60	3.05	18.85	761.3	2172.5	0.0110	0.0110	50	95.71
61	2.74	18.85	772.4	2359.6	0.0109	0.0109	56	98.45
62	2.74	18.85	772.4	2359.6	0.0109	0.0109	56	101.19
63	2.74	18.85	772.0	2358.4	0.0110	0.0110	56	103.94
64	2.74	18.85	772.7	2360.7	0.0110	0.0110	56	106.68
65	2.74	18.85	772.5	2360.2	0.0110	0.0110	56	109.42
66	3.05	18.85	825.2	2521.0	0.0107	0.0107	54	112.47
67	3.05	18.85	825.2	2521.0	0.0107	0.0107	54	115.52
68	3.05	18.85	825.0	2221.4	0.0107	0.0107	54	118.57
69	3.05	18.85	826.1	2023.5	0.0107	0.0107	54	121.62
70	3.05	18.85	825.7	2022.6	0.0107	0.0107	54	124.66
71	3.05	18.85	825.7	2022.6	0.0107	0.0107	54	127.71
72	3.05	18.85	825.5	2022.2	0.0108	0.0108	54	130.76
73	3.05	18.85	825.2	2021.3	0.0108	0.0108	54	133.81
	Halfspace	18.07	825.1	2021.0	0.0110	0.0110		

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Table 3.7-5— {Frequencies and Mass Participation Factors for Common Basemat Intake Structures with Symmetric Boundary Conditions - Fixed Base Analysis}
(Coordinates based on CCNPP Unit 3)
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Mode #	Frequency (Hz)	Mass Participation Factors (%)			Mode #	Frequency (Hz)	Mass Participation Factors (%)		
		N-S	Vertical	E-W			N-S	Vertical	E-W
1	8.32	0	6.16	0	51	46.48	0	0.24	0.36
2	11.72	21.29	0.07	1.30	52	47.31	0.01	0.01	0.81
3	11.97	1.30	0.02	7.85	53	47.94	0.01	0.43	0.11
4	13.30	1.34	0.01	0.08	54	49.39	1.11	0.14	0.50
5	13.62	0.50	0.01	4.06	55	49.64	1.55	0	0.02
6	13.70	4.45	0.74	0.25	56	49.68	2.11	0	0.06
7	13.95	1.38	7.63	0.03	57	50.24	0.25	0.07	0.28
8	15.56	2.94	0	0.02	58	50.35	0.46	0.15	0.66
9	15.83	6.10	0	0.06	59	52.34	0.28	0.64	0.11
10	16.09	6.87	0	0.07	60	53.49	0	0.49	0
11	17.59	1.11	0.02	0.25	61	53.63	0.01	0.69	0
12	17.83	0.23	0	1.68	62	56.38	0.04	0.65	0.06
13	17.99	0	0.06	0.33	63	56.60	0.02	0.64	1.00
14	18.22	1.33	1.17	0.18	64	56.75	0.01	0.03	0.55
15	18.40	0.59	2.33	0.02	65	57.03	0.03	0.04	1.31
16	18.69	0.21	0.04	0.18	66	57.11	0.06	0	1.36
17	19.24	0	0.69	0	67	57.13	0.01	0	1.36
18	25.12	0.59	1.79	0	68	57.31	0.00	0.08	0.46
19	27.23	13.33	0.07	0	69	57.75	0.18	1.21	0.42
20	29.26	0.60	0	1.53	70	58.87	0.10	0.27	0.19
21	29.31	0.12	0	0.28	71	58.94	0.01	0.69	0.67
22	29.35	0.51	0	0.79	72	58.99	0.09	0.20	0.56
23	29.42	0.23	0	0.29	73	59.32	0.03	1.09	0.89
24	29.92	0.06	0	0.69	74	59.96	0	0.52	0
25	30.06	0.02	0	0.47	75	60.48	0	0.37	0.16
26	30.12	0	0	0.39	76	61.40	0.03	0.16	0.29
27	31.13	0.02	0	1.13	77	61.65	0.39	0	0.50
28	32.85	0	0.02	0.38	78	64.02	0.01	0.65	0.02
29	33.00	0.02	0	0.70	79	67.40	0.08	0	0.72
30	33.08	0.16	0	0.41	80	68.03	0.09	0	0.30
31	33.92	0	3.41	0.03	81	68.49	0	0.32	0.15
32	34.37	0.03	0.02	0.40	82	68.68	0.21	0.06	0.25
33	34.40	0.07	0.02	0.66	83	69.07	0.54	0.19	0.05
34	34.44	0.02	0	0.67	84	70.75	0.03	0.36	0.04
35	34.82	0.06	0.01	0.43	85	71.90	0.18	0	0.57
36	34.84	0.03	0.02	0.40	86	71.98	0.03	0.02	0.35

Table 3.7-5— {Frequencies and Mass Participation Factors for Common Basemat Intake Structures with Symmetric Boundary Conditions - Fixed Base Analysis}
(Coordinates based on CCNPP Unit 3)
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Mode #	Frequency (Hz)	Mass Participation Factors (%)			Mode #	Frequency (Hz)	Mass Participation Factors (%)		
		N-S	Vertical	E-W			N-S	Vertical	E-W
37	35.72	1.27	0	0	87	73.69	0.01	0.29	0.15
38	36.64	0	2.05	0	88	75.11	0.27	0.58	0.67
39	36.84	0.05	4.11	0.01	89	75.50	0.06	0.25	0.14
40	37.86	0.74	0.97	0	90	76.64	0.01	0.02	3.80
41	39.27	0.01	0.40	0	91	76.96	0	0.26	0.41
42	42.89	0.53	1.68	0.09	92	77.93	0.09	0.33	6.55
43	42.93	0.25	0.69	0.01	93	78.64	0.05	0.59	0.53
44	44.11	0.77	0.11	0.06	94	79.46	0.20	0.08	0.15
45	44.36	0.01	1.01	0.30	95	80.21	0.12	0.75	0.02
46	44.61	0	0.13	0.30	96	80.44	0.02	1.52	0.03
47	44.95	0.01	0.26	0.95	97	81.36	0.03	0.50	0.20
48	45.32	0.01	0.04	0.45	98	84.48	0.01	0.14	0.36
49	45.62	0.20	0.02	0.18	99	84.95	0.04	0	0.48
50	45.72	0	0.05	0.56	100	87.70	0.07	0.26	0.18

Table 3.7-6— {Frequencies and Mass Participation Factors for Common Basemat Intake Structures with Anti-Symmetric Boundary Conditions - Fixed Base Analysis}

(Coordinates based on CCNPP Unit 3)
(Page 1 of 2)

Mode #	Frequency (Hz)	Mass Participation Factors (%)			Mode #	Frequency (Hz)	Mass Participation Factors (%)		
		N-S	Vertical	E-W			N-S	Vertical	E-W
1	8.27	0.01	9.83	0	51	36.89	1.34	0	0.20
2	9.61	0.02	33.28	0.12	52	36.90	0.47	0	0.20
3	11.31	0.58	12.13	2.20	53	36.93	0.49	0	0
4	12.17	1.47	4.12	4.01	54	37.13	0.40	0.01	0.27
5	12.43	6.38	1.27	0.29	55	37.50	0.81	0.06	0.01
6	14.13	0.17	7.14	0.26	56	37.58	0.74	0.11	0
7	14.26	0.06	4.54	1.66	57	37.71	0.14	0.04	0.35
8	14.33	0.01	1.18	0.85	58	37.73	0.33	0.08	0.03
9	14.49	2.29	0.02	0.02	59	38.79	0.03	0.00	0.39
10	15.03	0.29	0.88	0.11	60	38.87	0.93	0.05	0.16
11	15.53	0.55	0.07	0	61	39.13	0.01	0	0.39
12	15.70	0.07	0.01	1.07	62	39.35	0.22	0.01	0.30
13	15.80	2.53	0.04	0.05	63	40.43	0.03	0	0.81
14	16.63	1.62	0.16	0.09	64	41.00	0.07	0	0.46
15	17.21	1.59	0.07	0.06	65	42.68	1.14	0	0.27
16	17.28	0.28	0.11	0.05	66	42.71	0.43	0.01	0.08
17	17.80	0.93	0.61	0.13	67	42.73	0.75	0.01	0.15
18	18.16	0.05	0.02	1.17	68	43.96	0.61	0	0.10
19	18.21	0.42	0.10	0.01	69	46.69	0.34	0.01	0.04
20	19.08	0.60	0.07	0.02	70	46.72	1.60	0.02	0.03
21	19.37	0.43	0.02	0	71	46.74	2.55	0.02	0.04
22	19.38	0	0	0.76	72	46.79	0.40	0	0.02
23	19.67	0.99	0.03	0.01	73	47.76	0.29	0.04	0.33
24	19.83	0	0.01	0.77	74	50.32	0.64	0.01	0.09
25	22.78	0.32	0.12	0	75	50.94	0.01	0.33	0.19
26	22.79	0.36	0.27	0.01	76	51.33	0.43	0.01	0.95
27	22.82	0.24	0.18	00	77	52.44	1.81	0.19	0.02
28	22.94	0.33	0.19	0.01	78	53.43	0.72	0.04	0.15
29	23.02	0.29	0.17	0.01	79	53.87	0.16	0.01	0.47
30	23.11	0.32	0.18	0.01	80	54.72	0.43	0.05	0.14
31	24.44	0.35	0.16	0	81	54.87	0.24	0.06	0.43
32	26.93	1.23	0.28	0.13	82	55.20	0.09	0.03	0.95
33	26.94	0.53	0.02	0.03	83	56.80	0	0.01	0.76
34	28.33	0.13	0.32	0.07	84	60.46	0.29	0.04	0.23
35	28.81	0.13	0.16	0.13	85	61.85	0.19	0.12	1.87
36	29.15	0.89	0.19	0.10	86	62.91	0.02	0	0.38

Table 3.7-6— {Frequencies and Mass Participation Factors for Common Basemat Intake Structures with Anti-Symmetric Boundary Conditions – Fixed Base Analysis}

(Coordinates based on CCNPP Unit 3)
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Mode #	Frequency (Hz)	Mass Participation Factors (%)			Mode #	Frequency (Hz)	Mass Participation Factors (%)		
		N-S	Vertical	E-W			N-S	Vertical	E-W
37	29.24	0.14	0.17	0.29	87	64.85	0.06	0.01	0.40
38	29.44	0.28	0.11	0.13	88	65.73	0.11	0.19	0.29
39	31.60	0.12	0.19	0.11	89	66.02	0.04	0.01	0.97
40	31.63	0.37	0.48	0	90	66.63	0.04	0.08	1.05
41	31.66	0.30	0.22	0.13	91	67.86	0.04	0.22	0.27
42	34.07	0.50	0.02	0.02	92	68.46	0.13	0	0.32
43	34.09	0.38	0.02	0	93	70.72	0.01	0.01	0.41
44	34.33	0.55	0.03	0	94	72.15	0	0.03	1.12
45	35.17	0.46	0.01	0.05	95	72.34	0.03	0.01	0.47
46	35.48	1.80	0.10	0.17	96	75.13	0	0.01	0.40
47	36.43	0.44	0.04	0	97	75.15	0.06	0.14	3.74
48	36.51	0.64	0.13	0	98	76.09	0.03	0.01	0.74
49	36.66	0	0.63	0.02	99	76.69	0.27	0.03	1.94
50	36.67	0.01	0.91	0	100	77.32	0.03	0	0.65

Table 3.7-7— {Boundary Conditions for Nodes in Plane of Symmetry of the CBIS Finite Element Model}

Direction of Seismic Loading	Condition of Plane of symmetry	Degree of Freedom of nodes on symmetric plane					
		U_x	U_y	U_z	φ_x	φ_y	φ_z
North-South	Symmetric	Free	Fix	Free	Fix	Free	Fix
East-West	Anti-Symmetric	Fix	Free	Fix	Free	Fix	Free
Vertical	Symmetric	Free	Fix	Free	Fix	Free	Fix

Notes:
U_x, U_y and U_z are the displacements, and φ_x, φ_y and φ_z are the rotations.

Table 3.7-8— {Worst Case Accelerations in Emergency Power Generating Building}

Slab Elevation	X (E-W) Direction	Y (N-S) Direction	Z (Vert) Direction
+68'-0"	0.31g	0.30g	0.29g
+51'-6"	0.27g	0.29g	0.29g
+19'-3"	0.22g	0.24g	0.23g
0'-0"	0.20g	0.21g	0.24g

Note:
Elevations and plant coordinate system refer to U.S EPR FSAR

Table 3.7-9— {Worst Case Accelerations in Essential Services Water Building}

Slab Elevation	X (N-S) Direction	Y (E-W) Direction	Z (Vert) Direction
+114'-0"	0.28g	0.28g	0.32g
+80'-9"	0.24g	0.22g	0.33g
+61'-10"	0.22g	0.26g	0.22g
+33'-0"	0.20g	0.18g	0.21g
0'-0"	0.16g	0.16g	0.20g

Note:
Elevations and plant coordinate system refer to U.S EPR FSAR.

Table 3.7-10— {Worst Case Accelerations in Common Basemat Intake Structures}

UHS Makeup Water Intake Structure			
Floor Elevation	**X (N-S) Direction**	**Y (E-W) Direction**	**Z (Vert) Direction**
22.5	0.225g	0.147g	0.233g
11.5	0.315g	0.199g	0.238g
26.5	0.342g	0.236g	0.240g
Forebay			
Floor Elevation	**X (N-S) Direction**	**Y (E-W) Direction**	**Z (Vert) Direction**
-22.5	0.227g	0.153g	0.215g

Note:
Elevations and plant coordinate system refer to U.S EPR FSAR.

Table 3.7-11— {Criteria for Seismic Interaction of Site-Specific Non-Seismic Category I Structures with Seismic Category I Structures}

Basis: Control Interaction through Prevention of Structure-to-Structure Impact[1]

Structure	Seismic Category	Design Code	Seismic Interaction Criteria	Seismic Interaction Evaluation
Turbine Building and Switchgear Building	SC-II[2b]	IBC Steel – AISC 341, AISC 360 & AISC N690[3] Concrete – ACI 318 & ACI 349[3]	SSE	No Interaction
Grid Systems Control Building	CS[2a]	IBC Steel – AISC 360 Concrete – ACI 318	None	No Interaction
Circulating Water Intake Structure	SC-II[2b]	IBC Steel – AISC 341 & ACI 360 Concrete – ACI 349	SSE	No Interaction

Notes:

1. This table is not applicable to equipment and subsystems qualification criteria.
2. Seismic Classification
   a. Conventional Seismic
   b. Seismic Category II
3. AISC N690 and ACI 349, as applicable, will be used for SSE and tornado load combinations in the design of the Lateral Force Resisting System (LFRS).

Figure 3.7-1— {CCNPP Unit 3 Site SSE Spectrum (0.15g PGA), 5% damping}

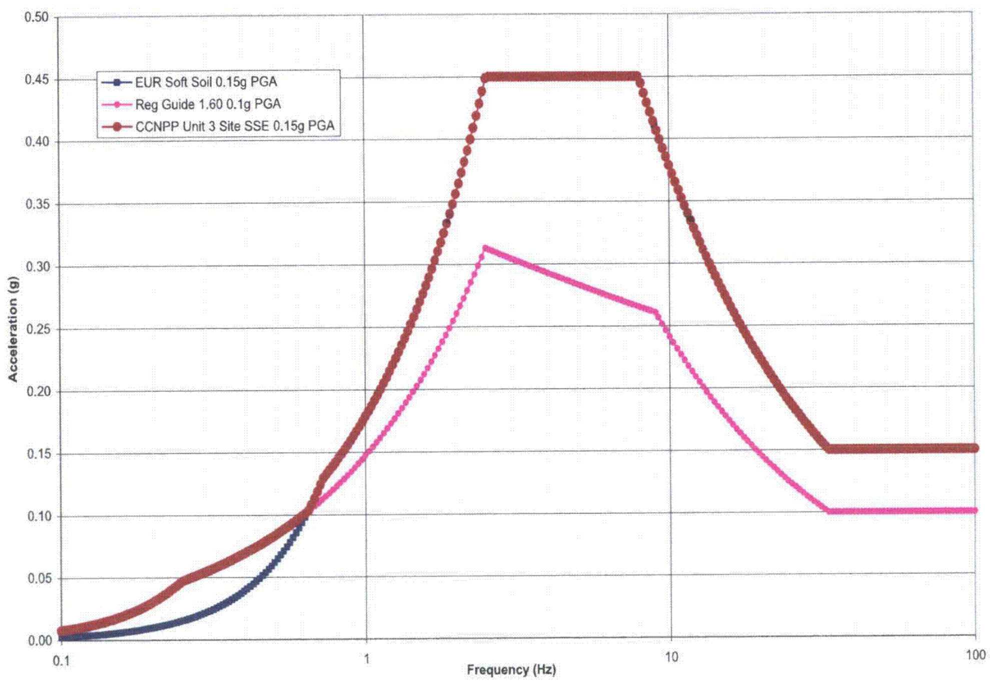

Figure 3.7-2— {CCNPP Unit 3 GMRS (Horizontal) and CCNPP Unit 3 Site SSE Spectrum, 5% damping}

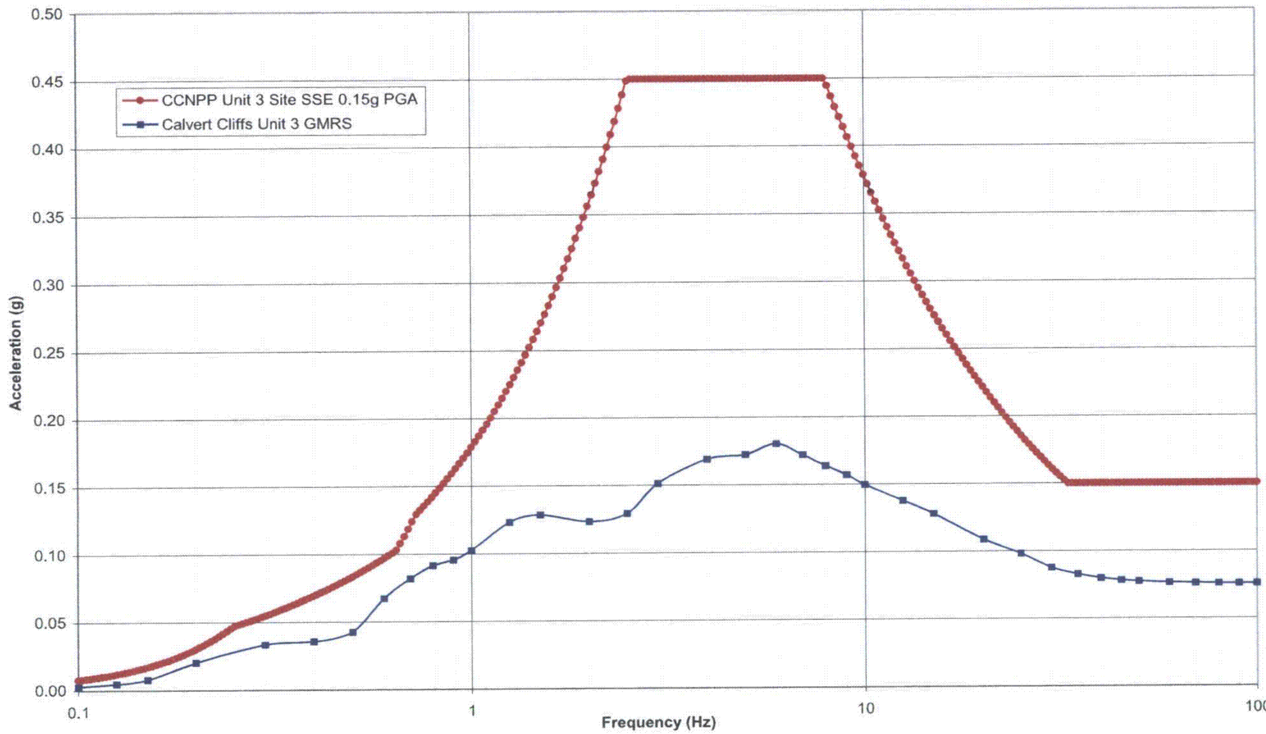

Figure 3.7-3— {CCNPP Unit 3 GMRS (Vertical) and CCNPP Unit 3 Site SSE Spectrum, 5% damping}

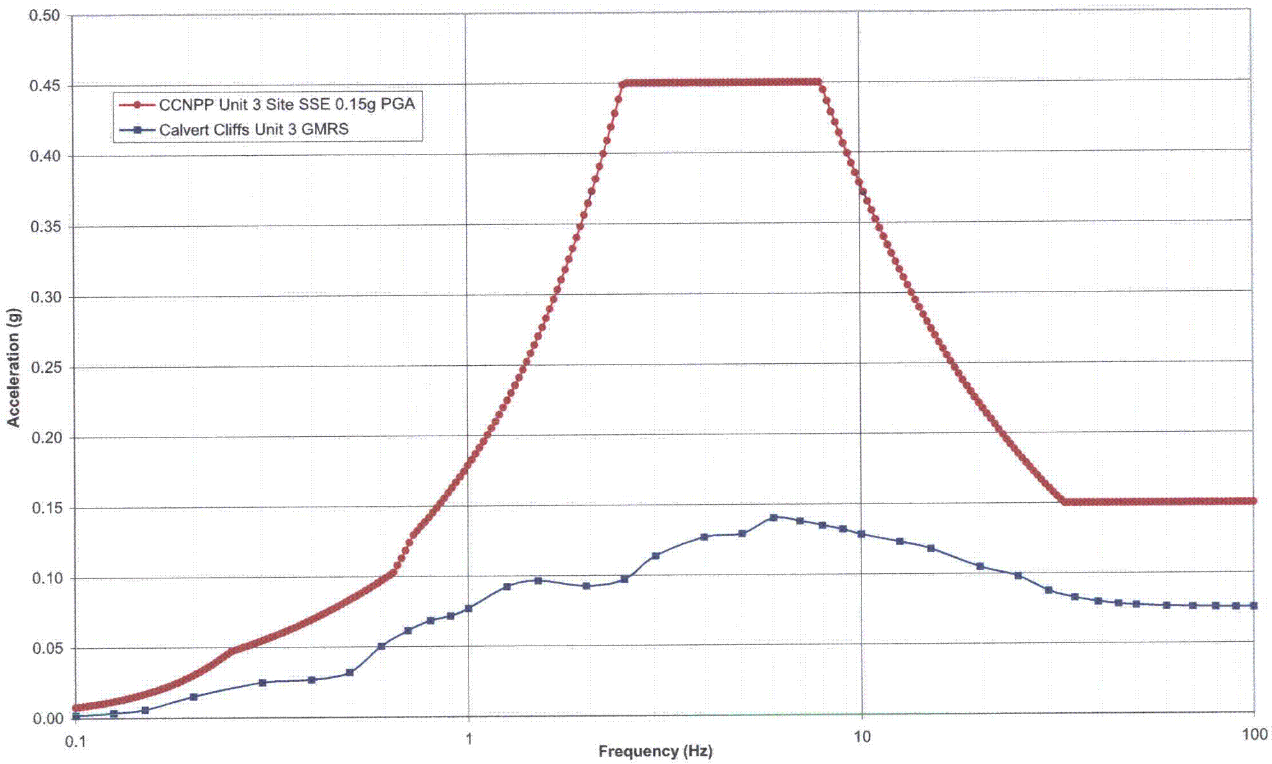

Figure 3.7-4— {CCNPP Unit 3 GMRS and EUR (Horizontal) for the Nuclear Island Common Basemat Structures}

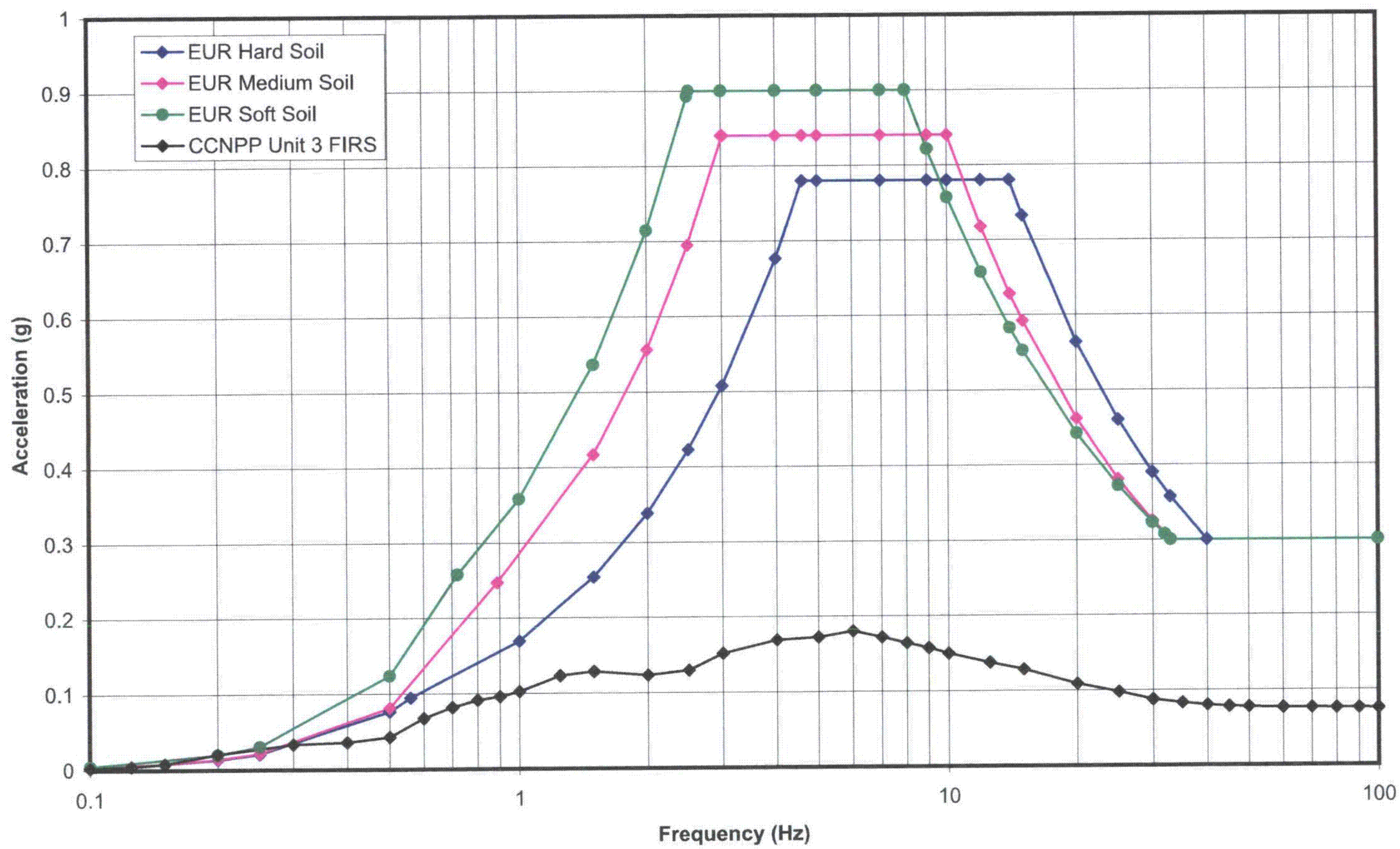

Figure 3.7-5— {CCNPP Unit 3 GMRS and EUR CSDRS (Vertical) for the Nuclear Island Common Basemat Structures}

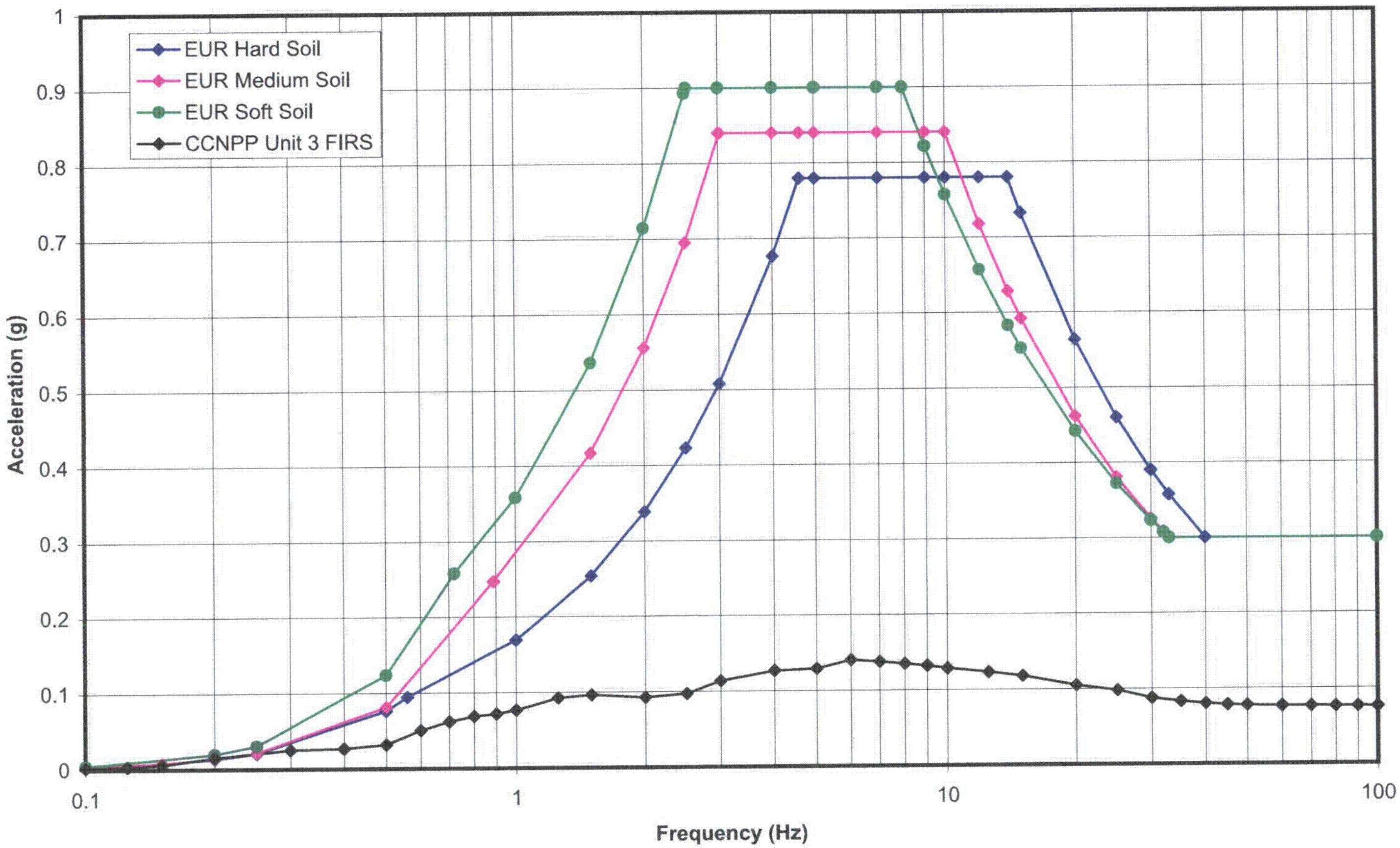

Figure 3.7-6— {CCNPP Unit 3 Site SSE, Site OBE and EUR CSDRS}

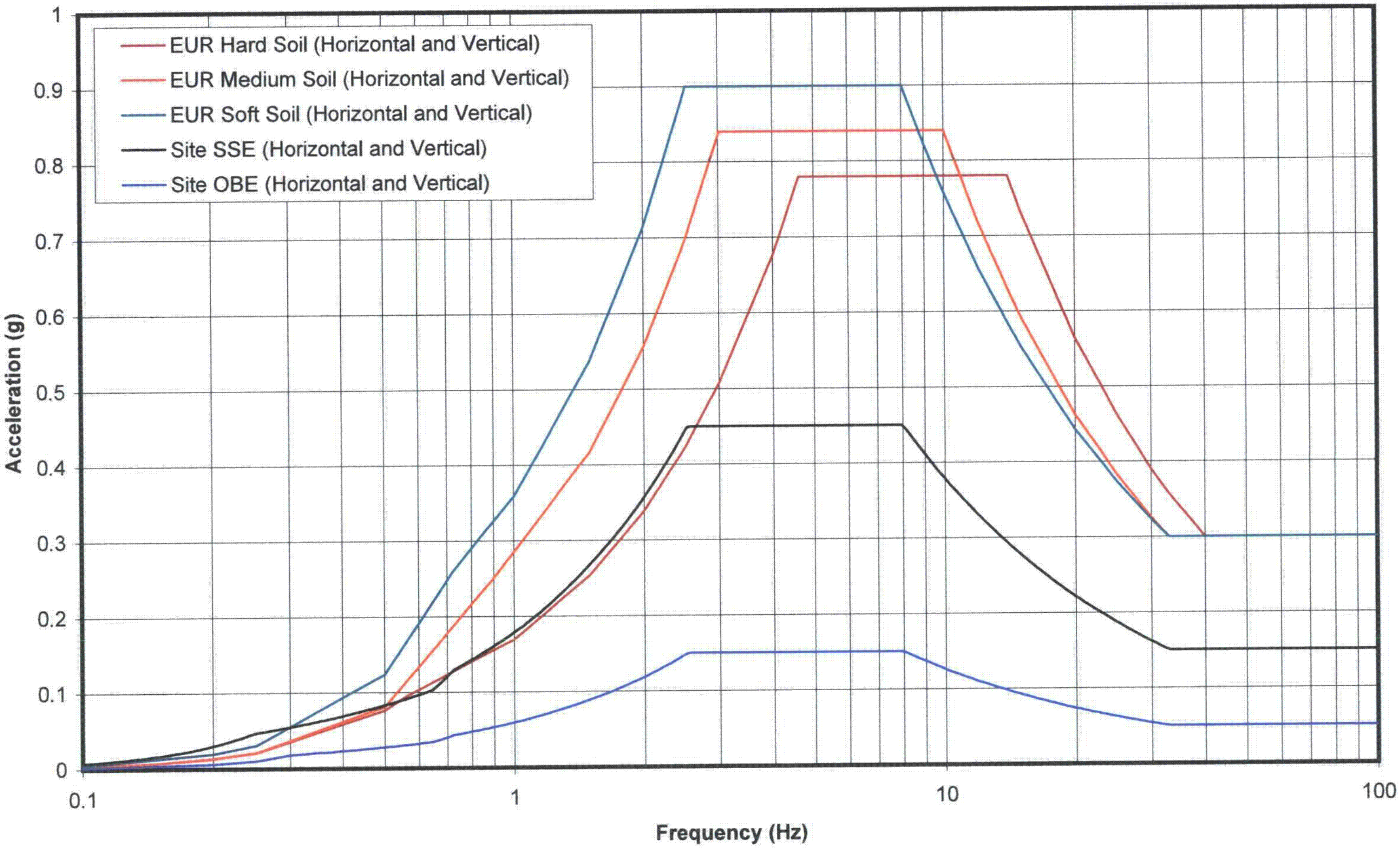

Figure 3.7-7— {Comparison of CSDRS, Site SSE and Horizontal RG 1.60 scaled to 0.10 g to Adjusted FIRS for ESWB and EPGB}

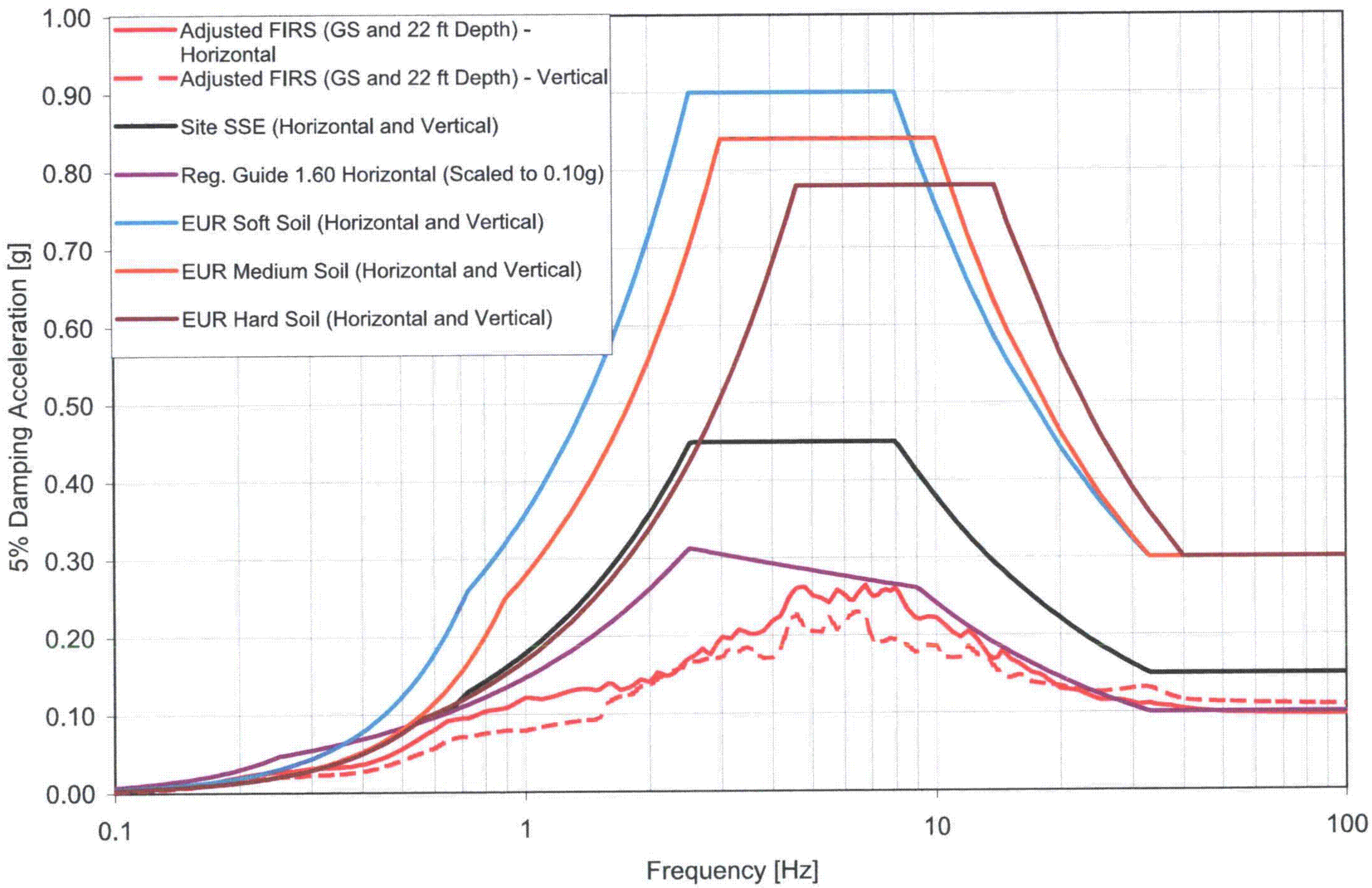

Figure 3.7-8— {Comparison of Site SSE and Horizontal RG 1.60 scaled to 0.10 g to Adjusted FIRS for ESWB and EPGB}

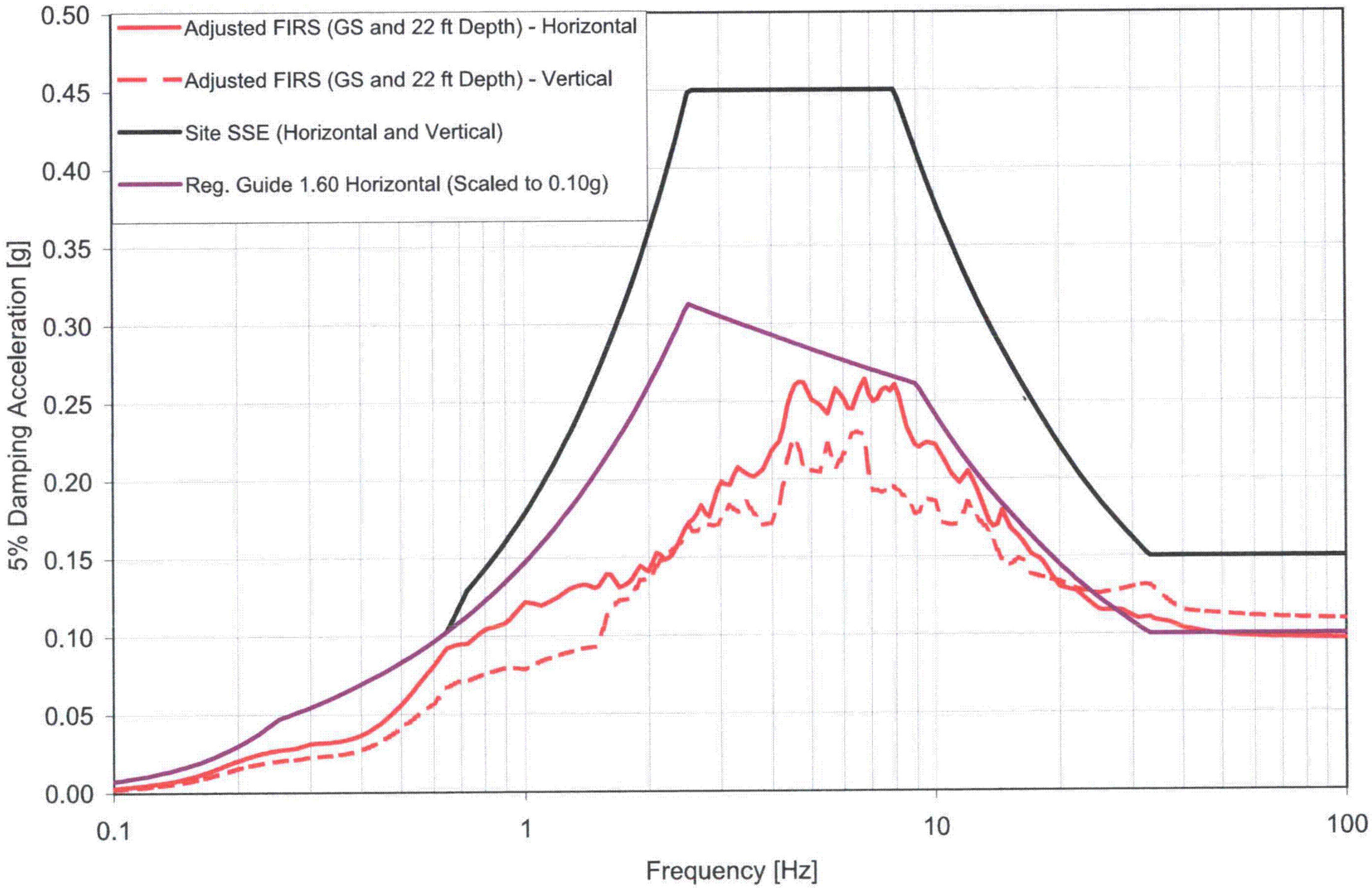

Figure 3.7-9— {Comparison of Site SSE and Horizontal RG 1.60 scaled to 0.10 g to Adjusted FIRS for CBIS}

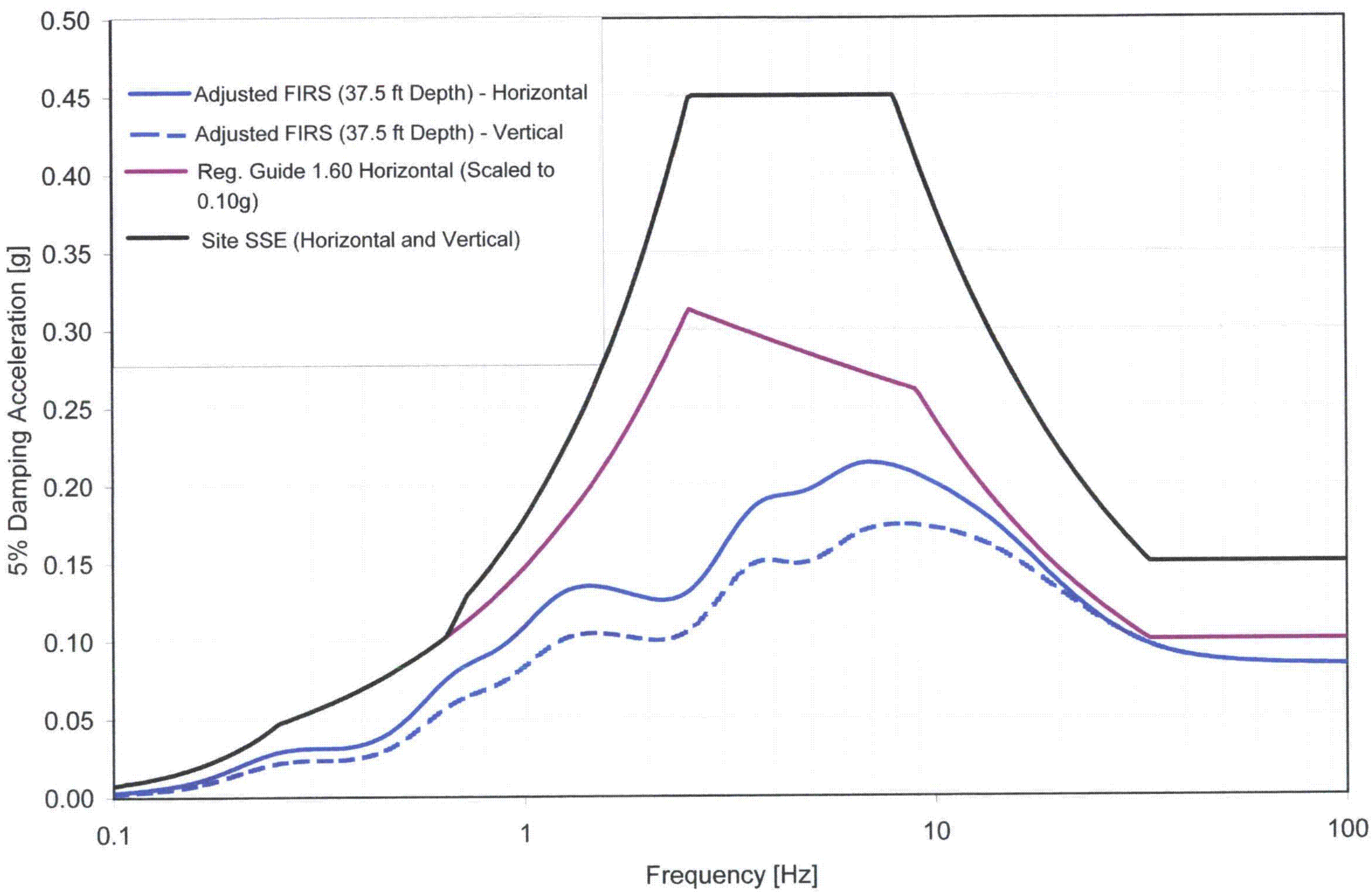

Figure 3.7-10— {Site SSE Spectrum Compatible Acceleration, Velocity, and Displacement Time Histories for Horizontal Component S1}

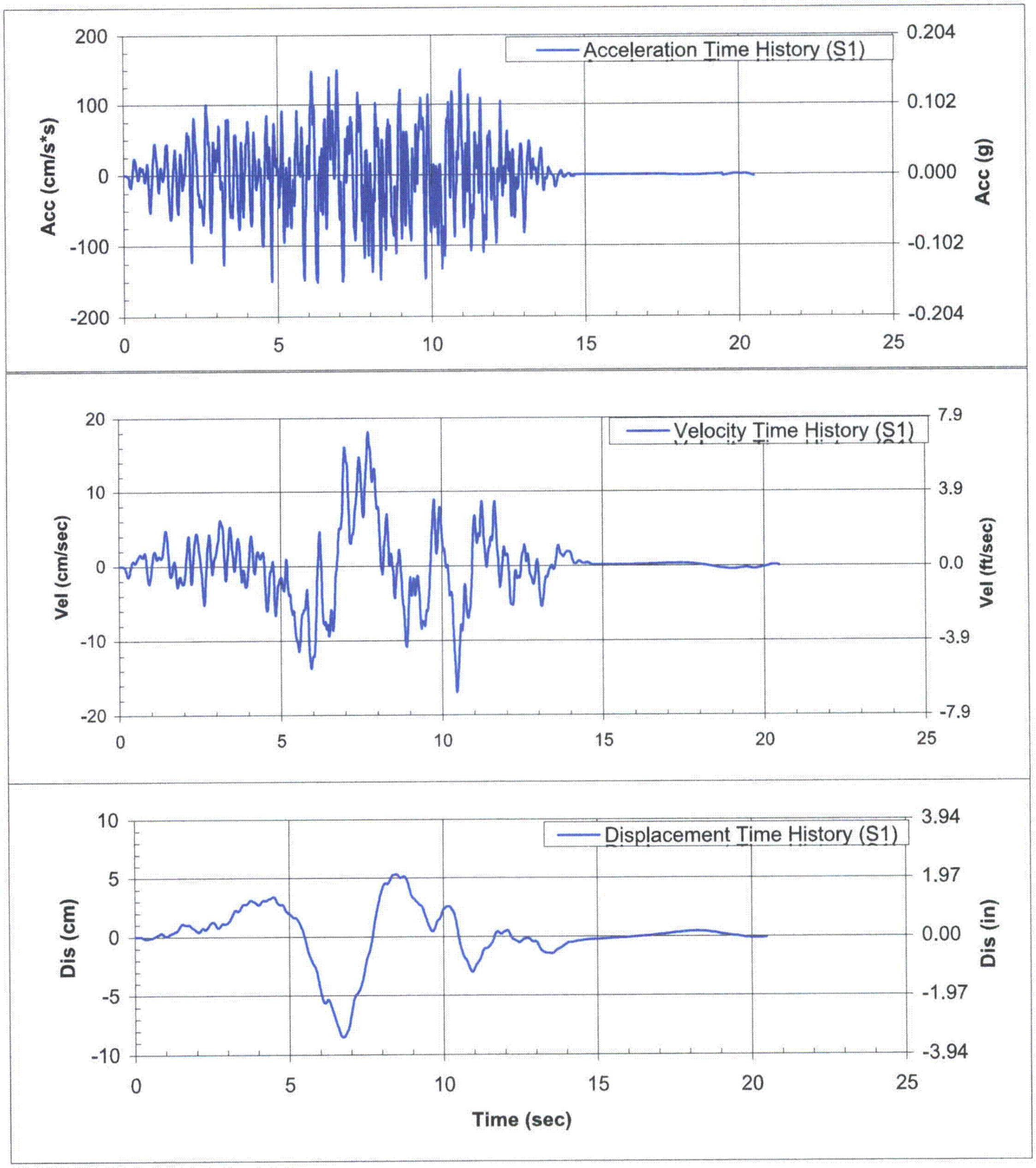

Figure 3.7-11— {Site SSE Spectrum Compatible Acceleration, Velocity, and Displacement Time Histories for Horizontal Component S2}

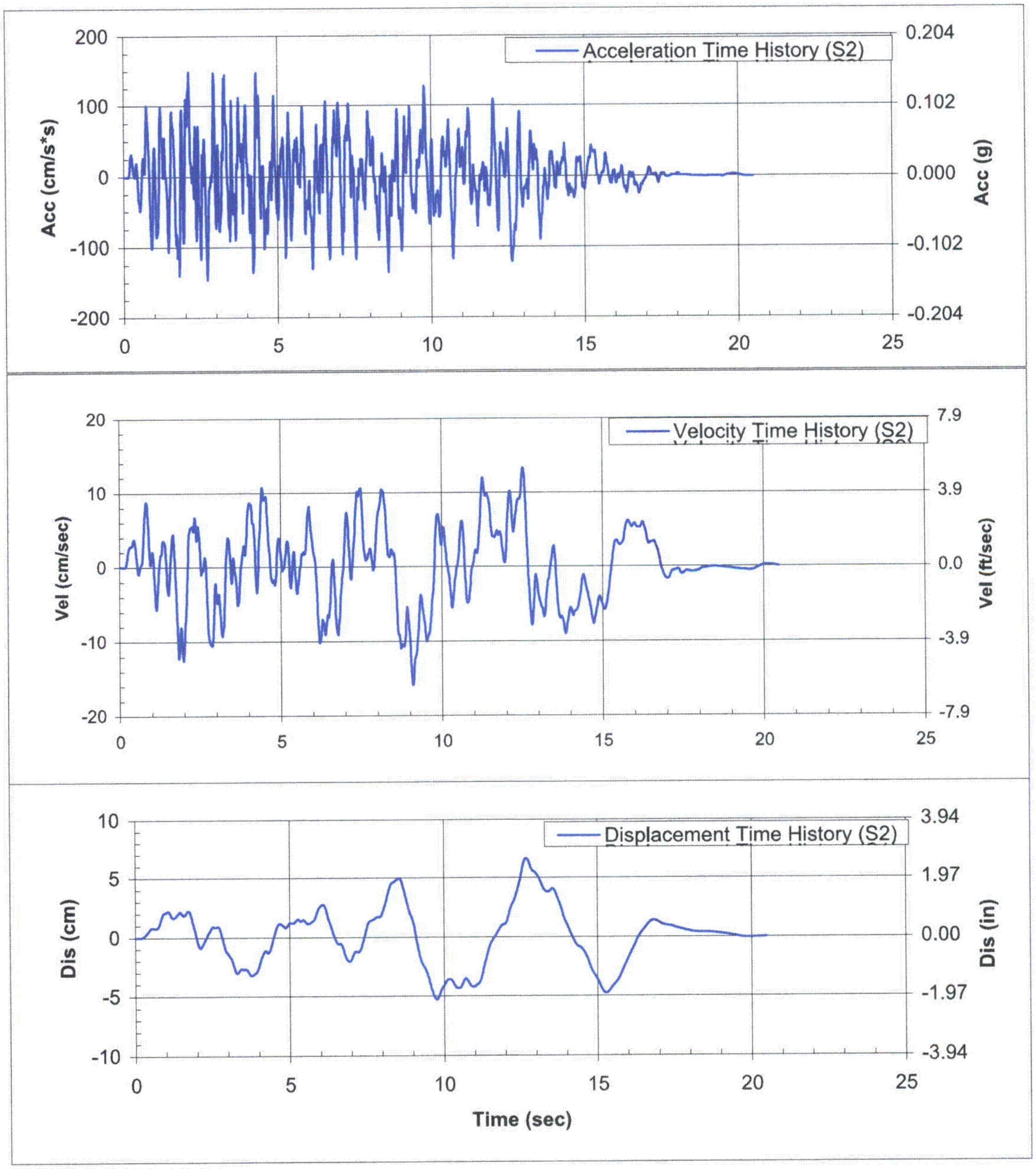

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Figure 3.7-12— {Site SSE Spectrum Compatible Acceleration, Velocity, and Displacement Time Histories for Vertical Component S3}

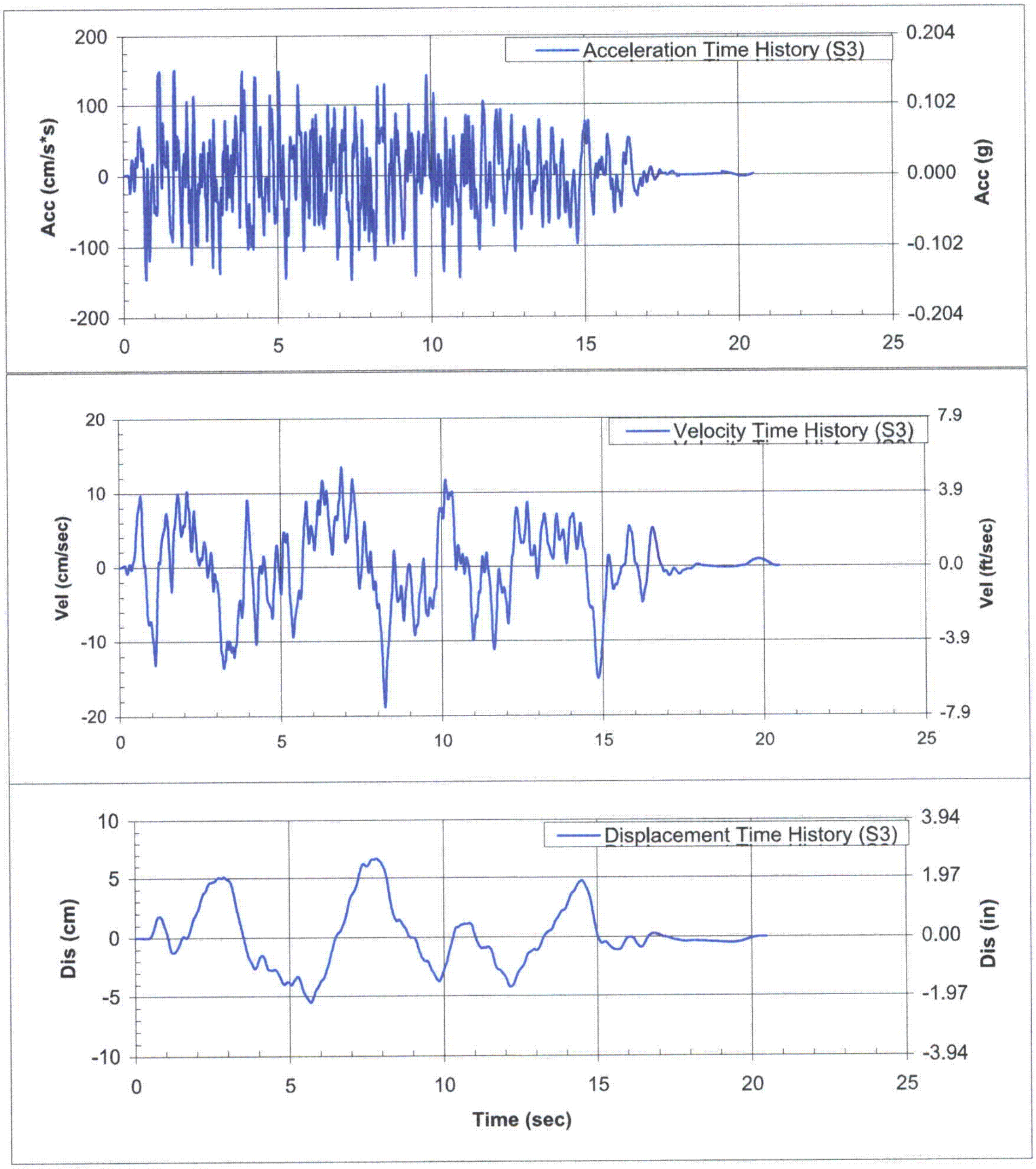
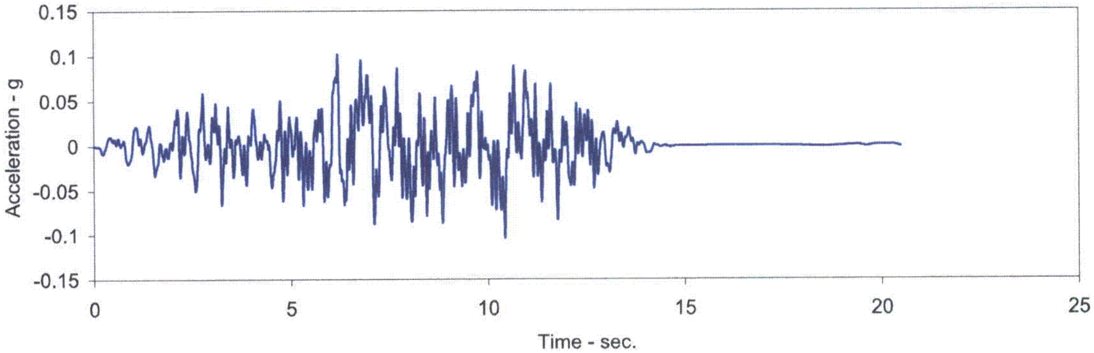
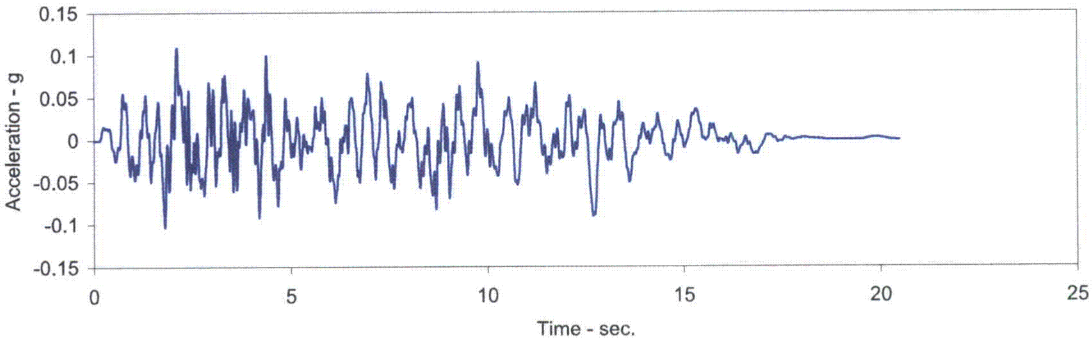


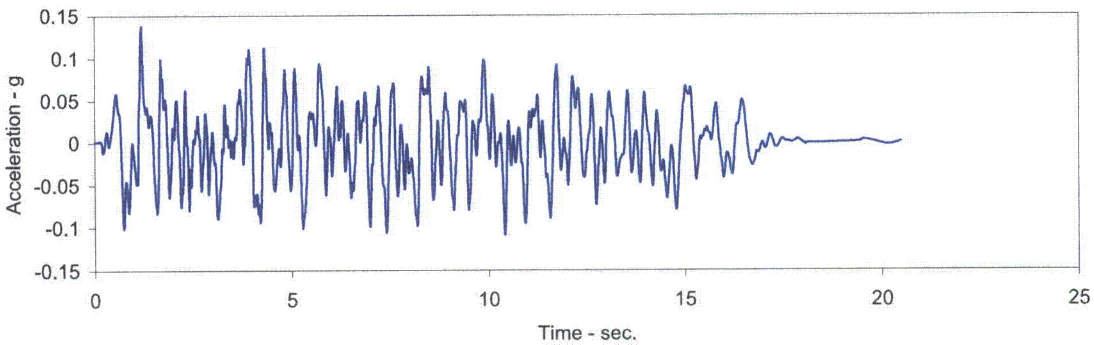
Figure 3.7-13— {SSI "Within" Acceleration Time Histories for Input at ESWB Foundation (LB Soil Case) NI Area (22 ft Depth)}



a) Horizontal Direction S1

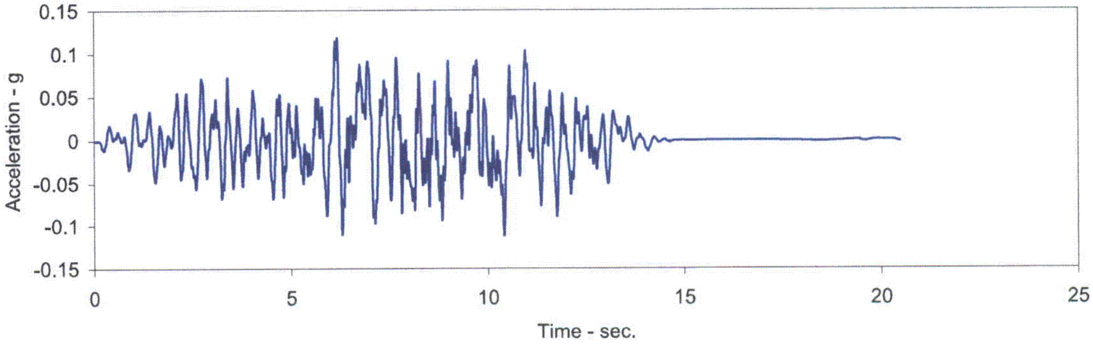


b) Horizontal Direction S2

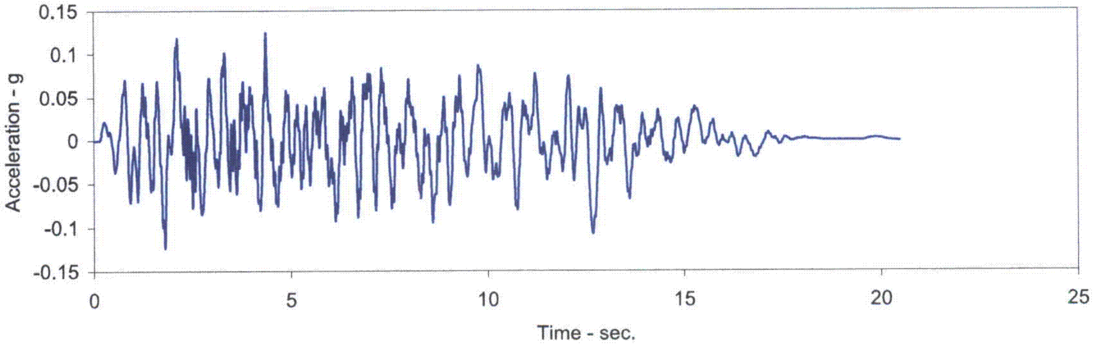


c) Vertical Direction S3

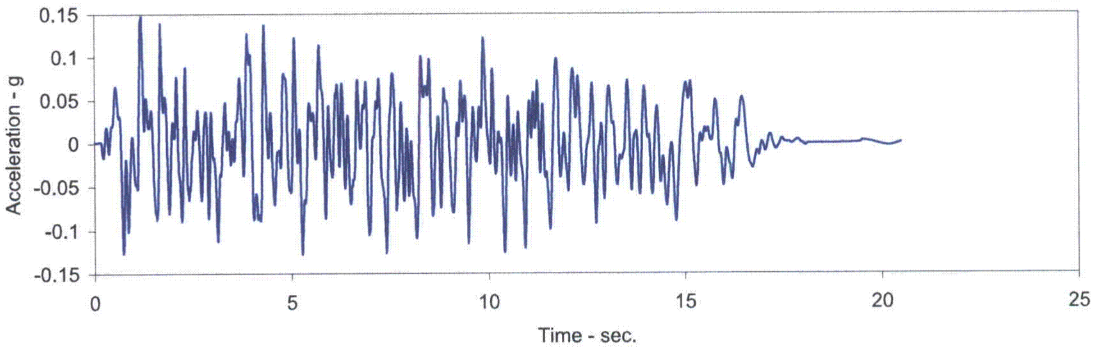
Figure 3.7-14— {SSI "Within" Acceleration Time Histories for Input at ESWB Foundation (BE Soil Case)- NI Area (22 ft Depth)}



a) Horizontal Direction S1



b) Horizontal Direction S2



c) Vertical Direction S3

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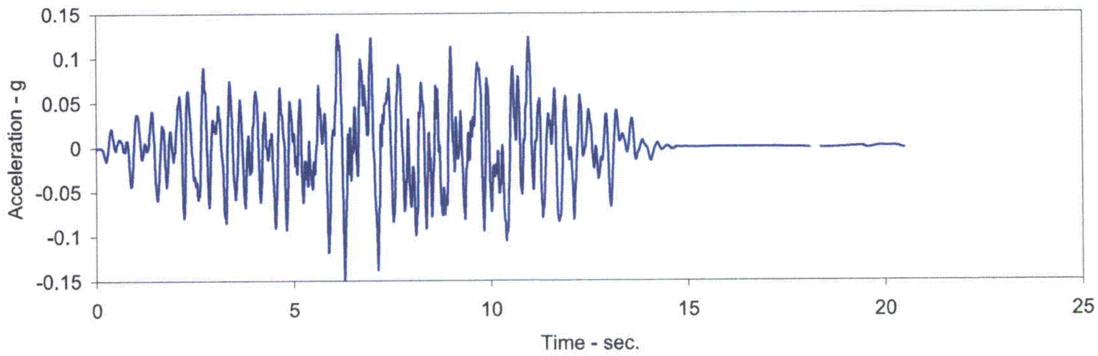
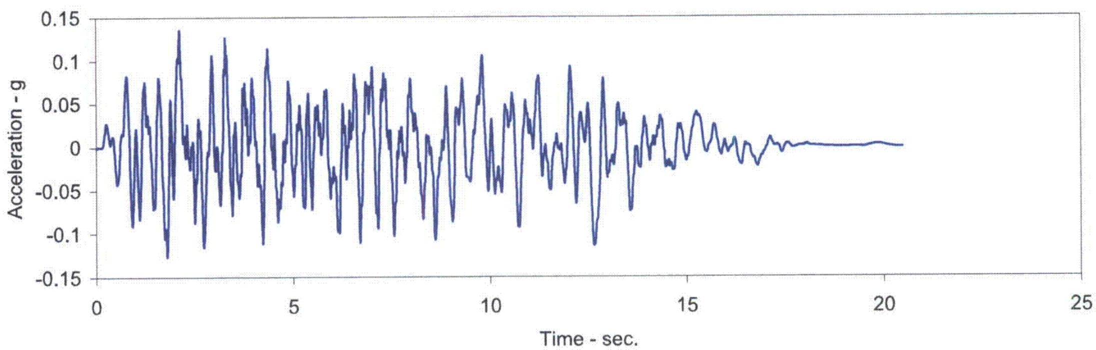
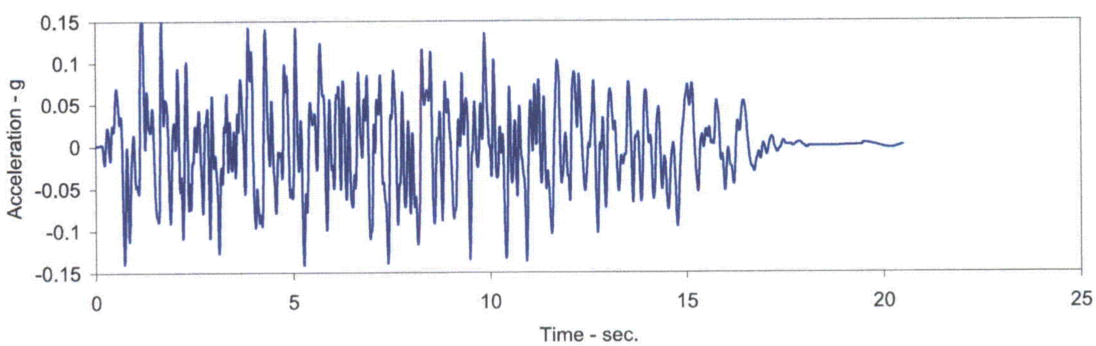
Figure 3.7-15— {SSI "Within" Acceleration Time Histories for Input at ESWB Foundation (UB Soil Case)- NI Area (22 ft Depth)}



a) Horizontal Direction S1

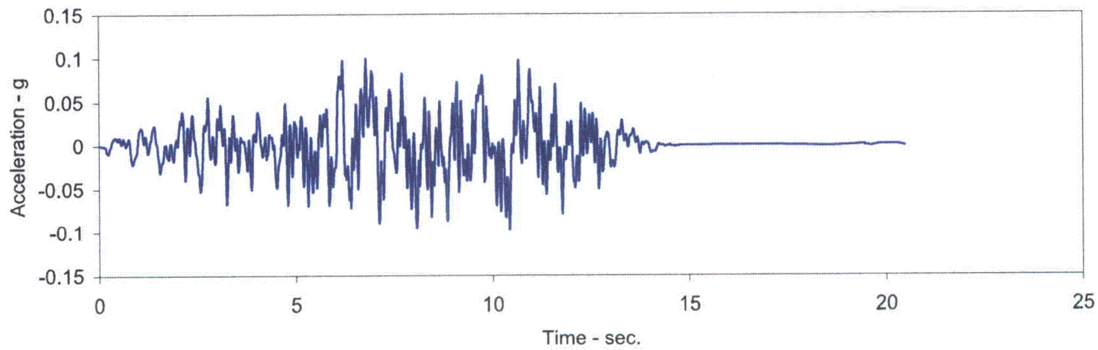


b) Horizontal Direction S2

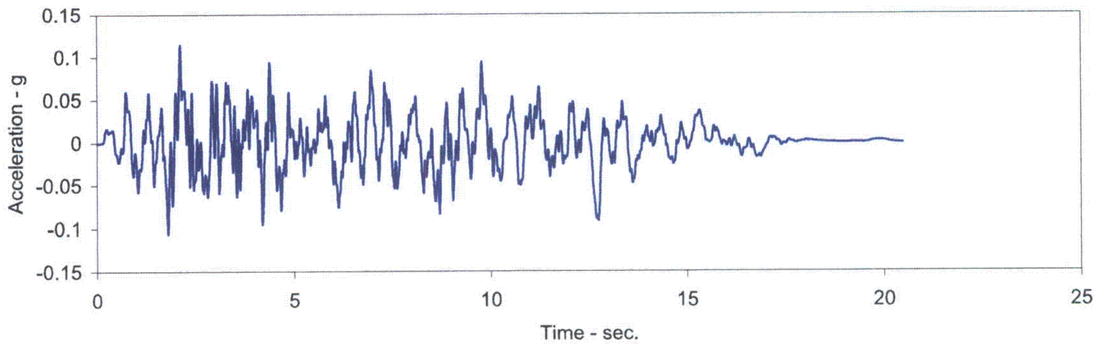


c) Vertical Direction S3

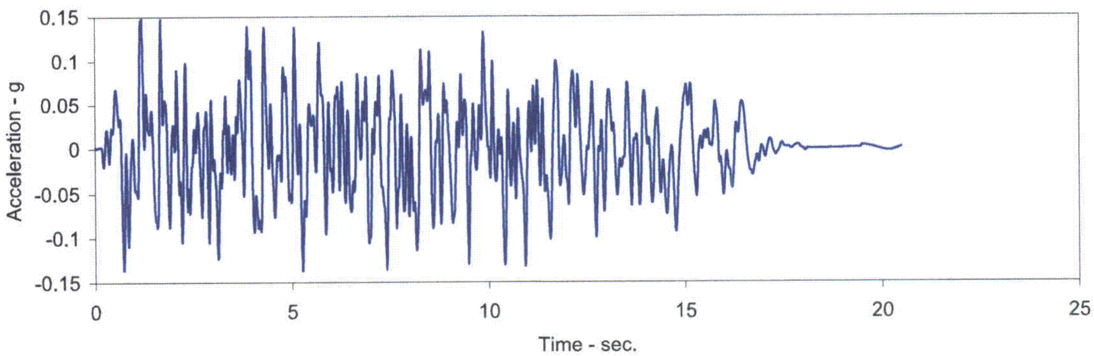
Figure 3.7-16— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (LB Soil Case)- Intake Area (37.5 ft Depth)}



a) Horizontal Direction S1



b) Horizontal Direction S2



c) Vertical Direction S3

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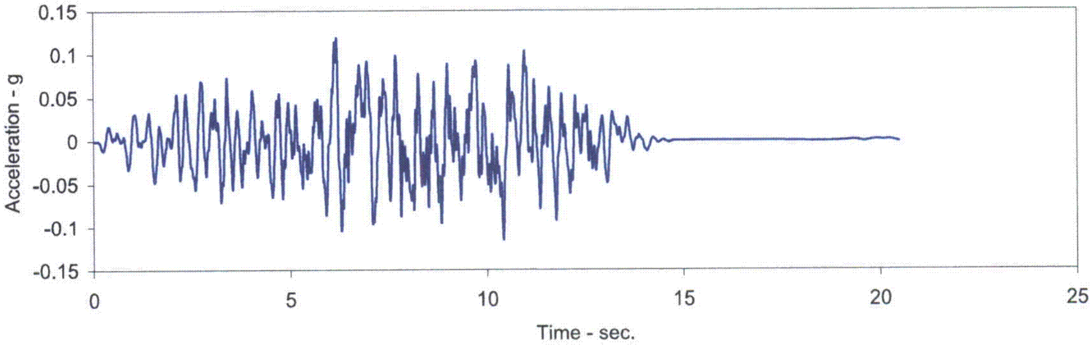
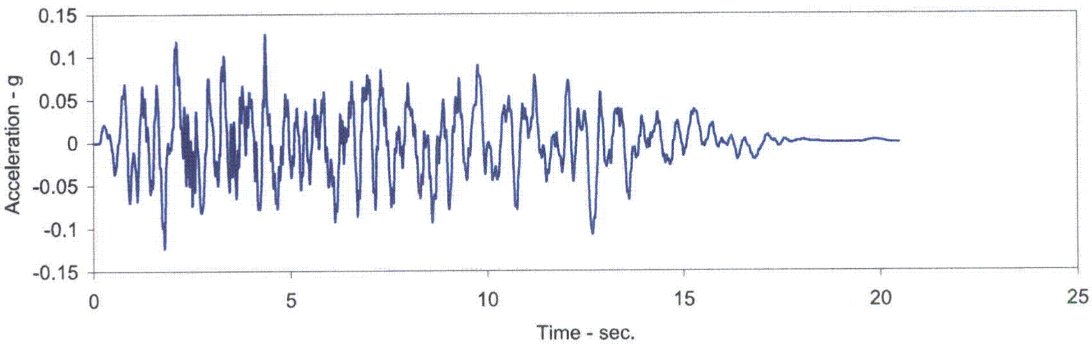
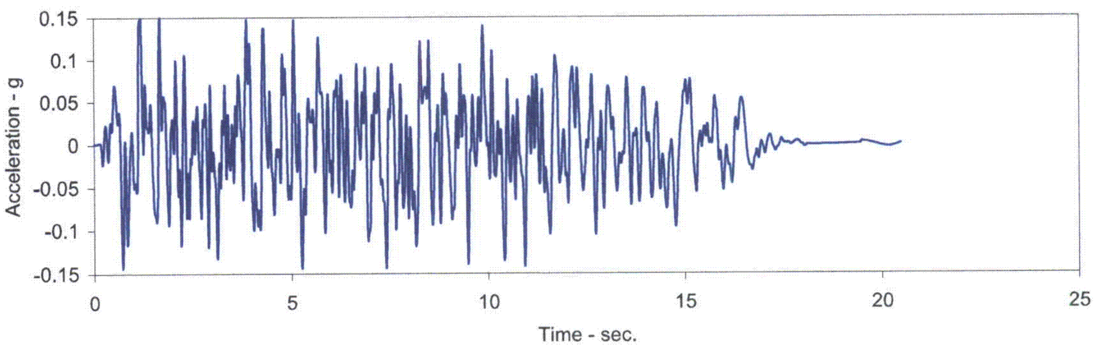
Figure 3.7-17— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (BE Soil Case)- Intake Area (37.5 ft Depth)}



a) Horizontal Direction S1

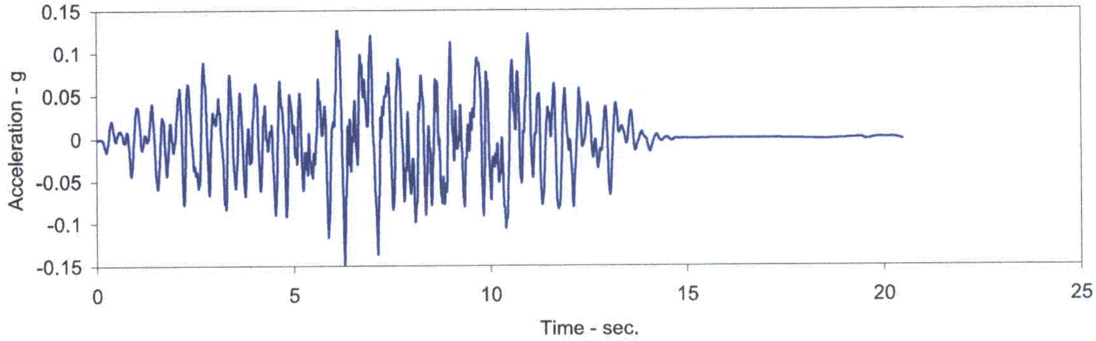


b) Horizontal Direction S2

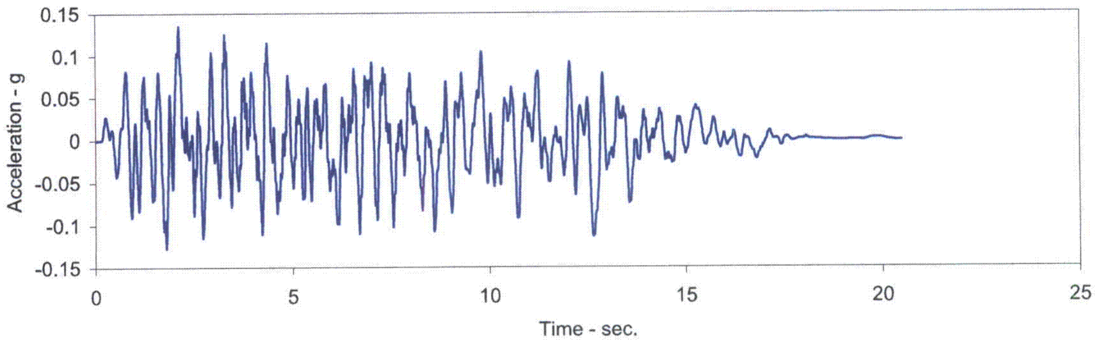


c) Vertical Direction S3

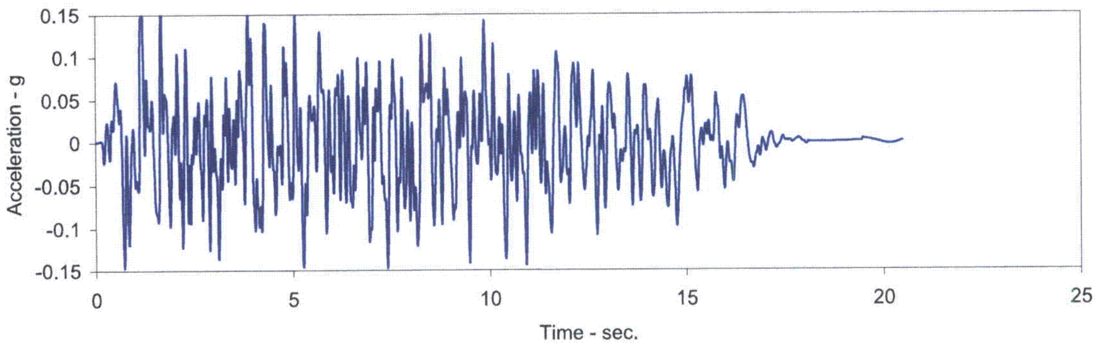
Figure 3.7-18— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (UB Soil Case)- Intake Area (37.5 ft Depth)}



a) Horizontal Direction S1



b) Horizontal Direction S2



c) Vertical Direction S3

Figure 3.7-19— {CCNPP Unit 3 Strain-Compatible Soil Profiles for NI Common Basemat Structures}

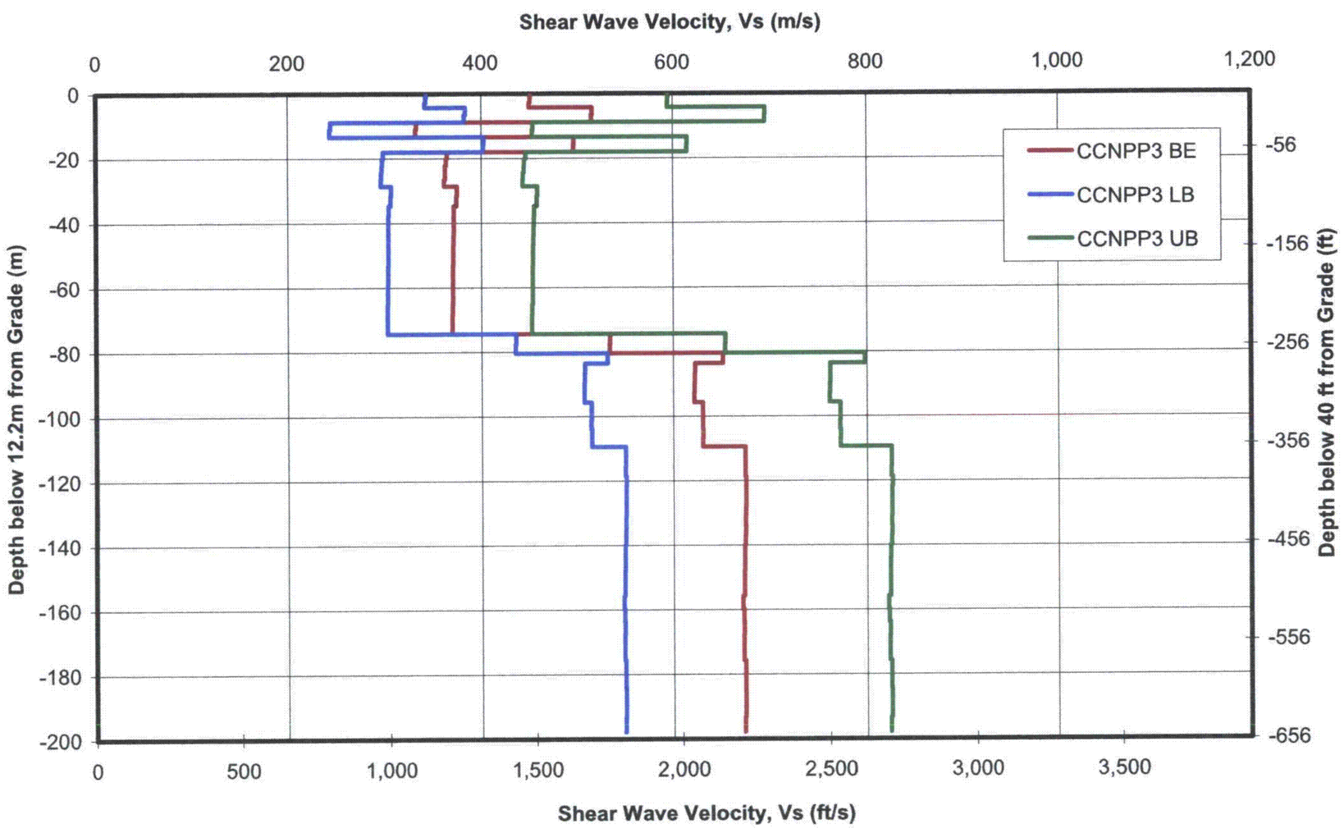


Figure 3.7-20— {EPR DC Soil Cases vs. CCNPP Unit 3 Soil Cases for SSI Analysis}

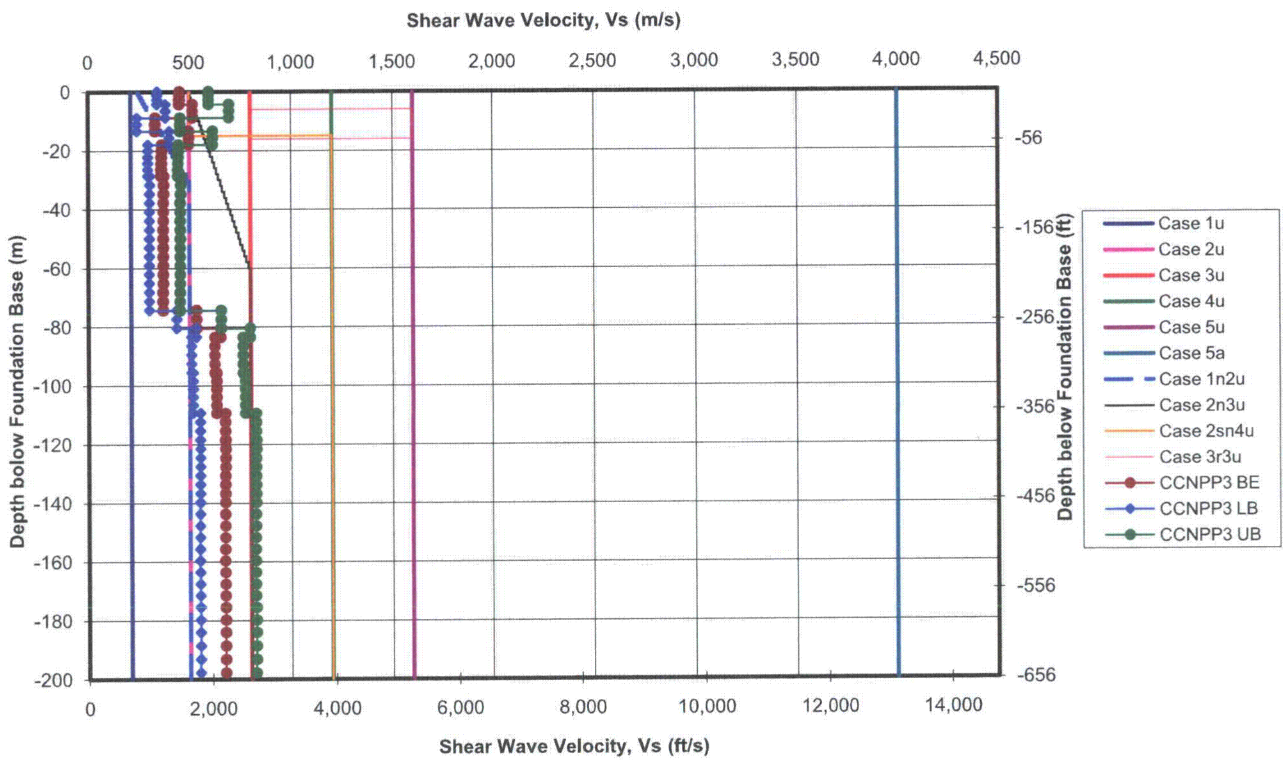


Figure 3.7-21— {CCNPP Unit 3 Strain-Compatible profiles at the NI Area for EPGB and ESWB}

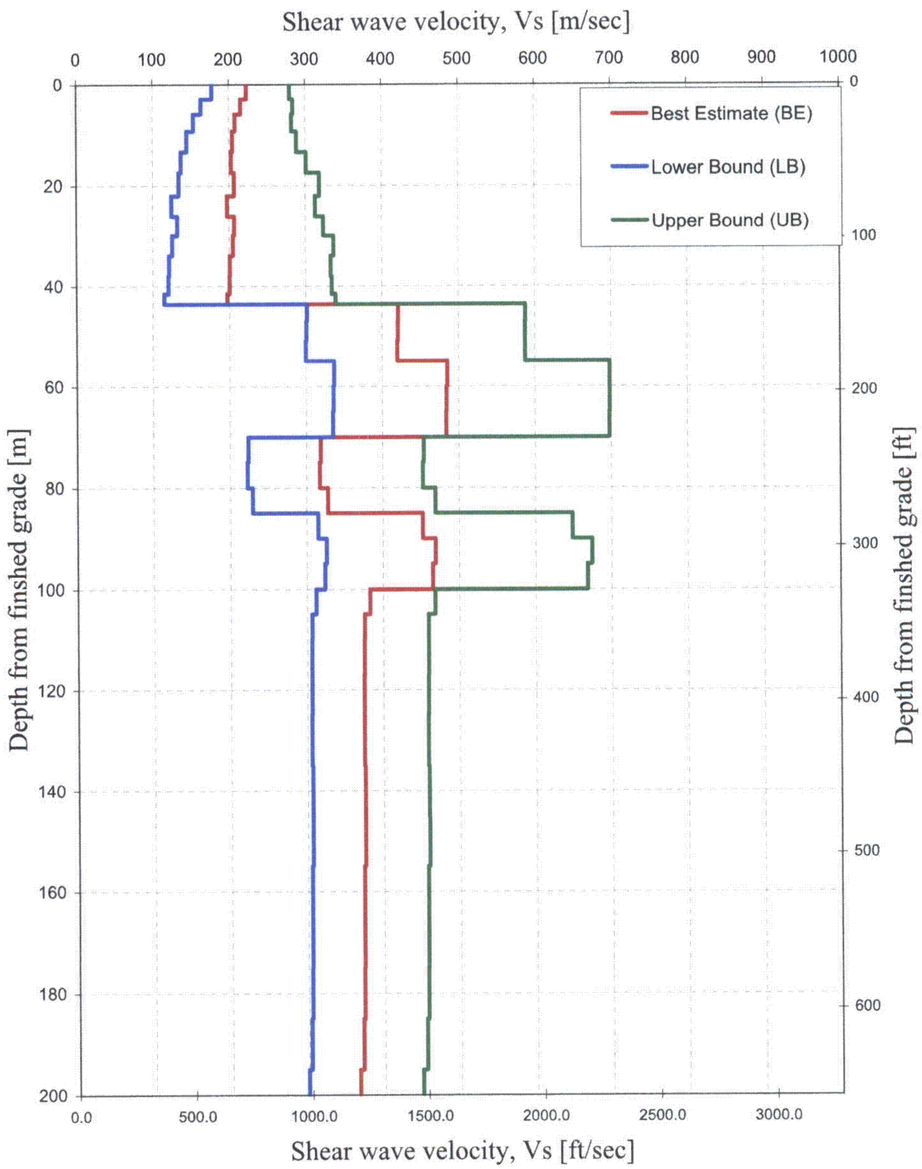


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Figure 3.7-22— {CCNPP Unit 3 Strain-Compatible profiles at the Intake Area for CBIS}

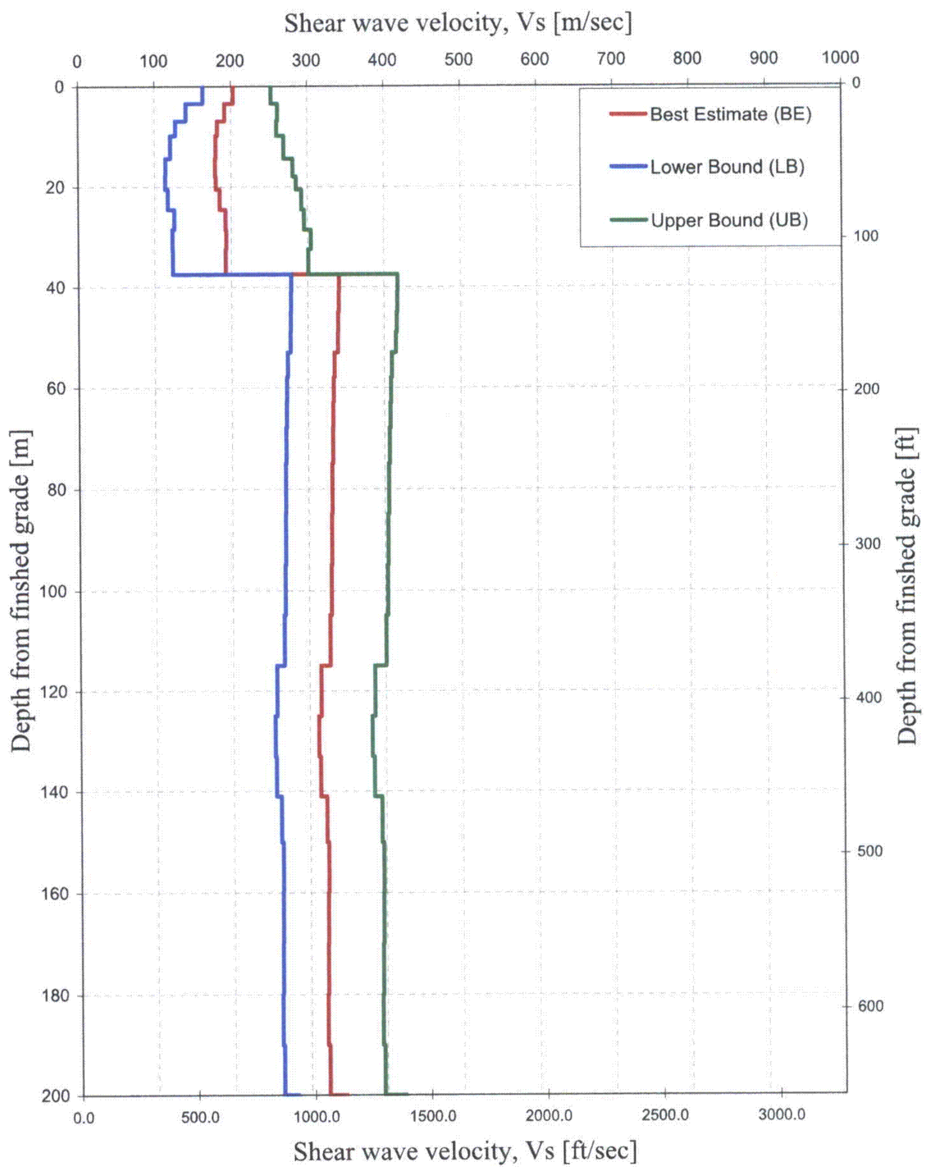


Figure 3.7-23— {Isometric View of the Common Basemat Intake Structures}

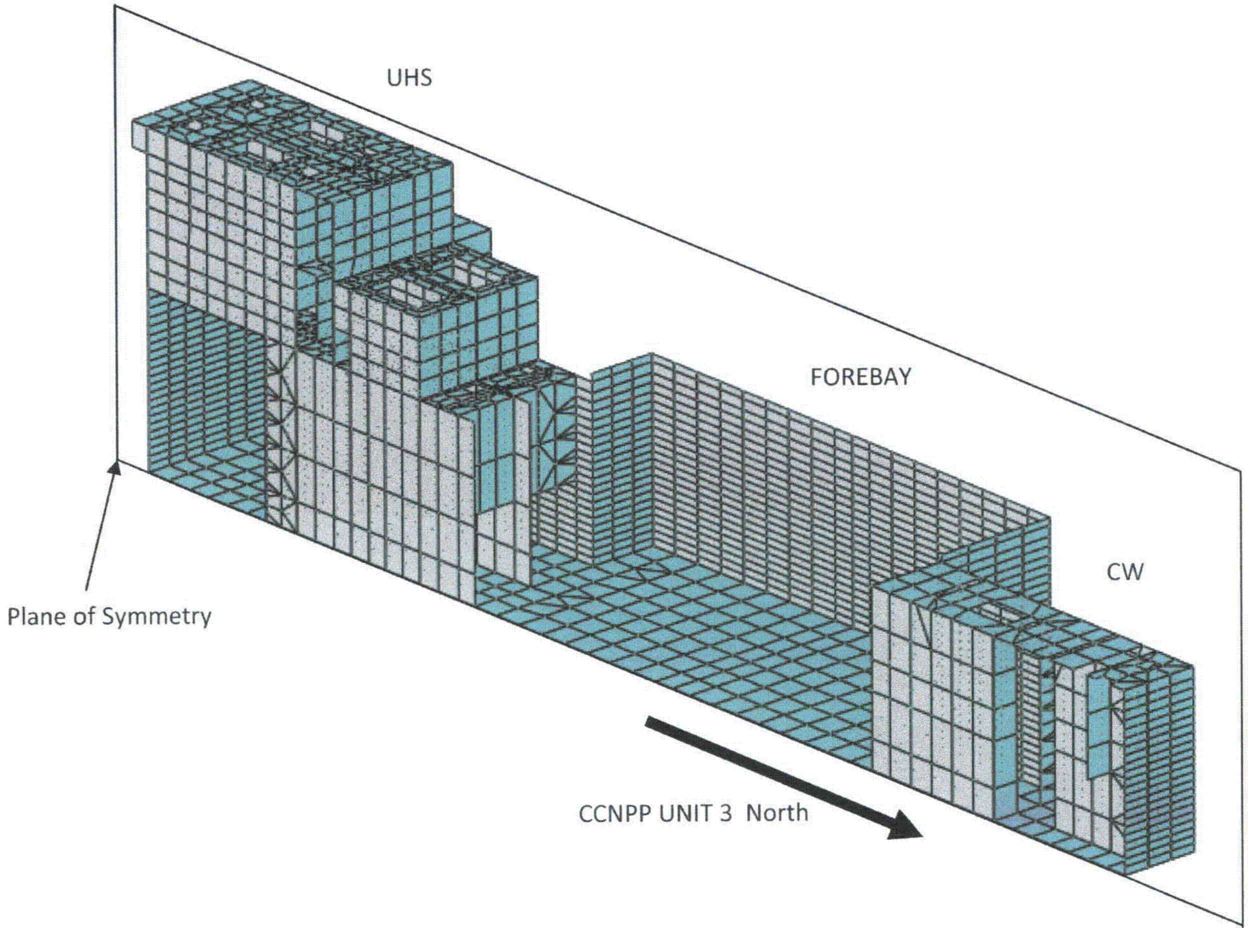
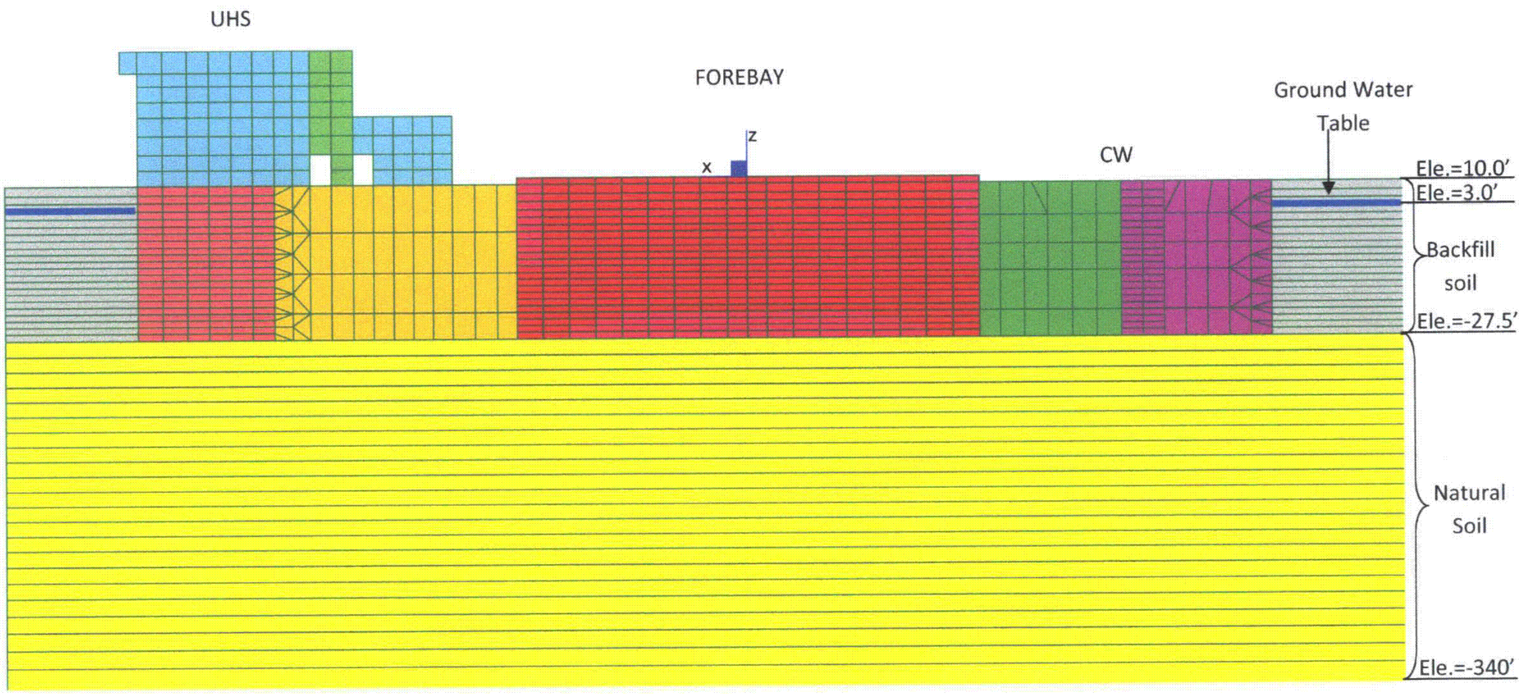


Figure 3.7-24— {Soil-Structure Interaction (SSI) model for the Common Basemat Intake Structures (Elevations and plant coordinate system refer to CCNPP Unit 3)}



The soil layering system shown is schematic.

Figure 3.7-25— {Reactor Bldg Internal Structure, Elev. 5.15m, X(E-W) Direction, 5% Damping}

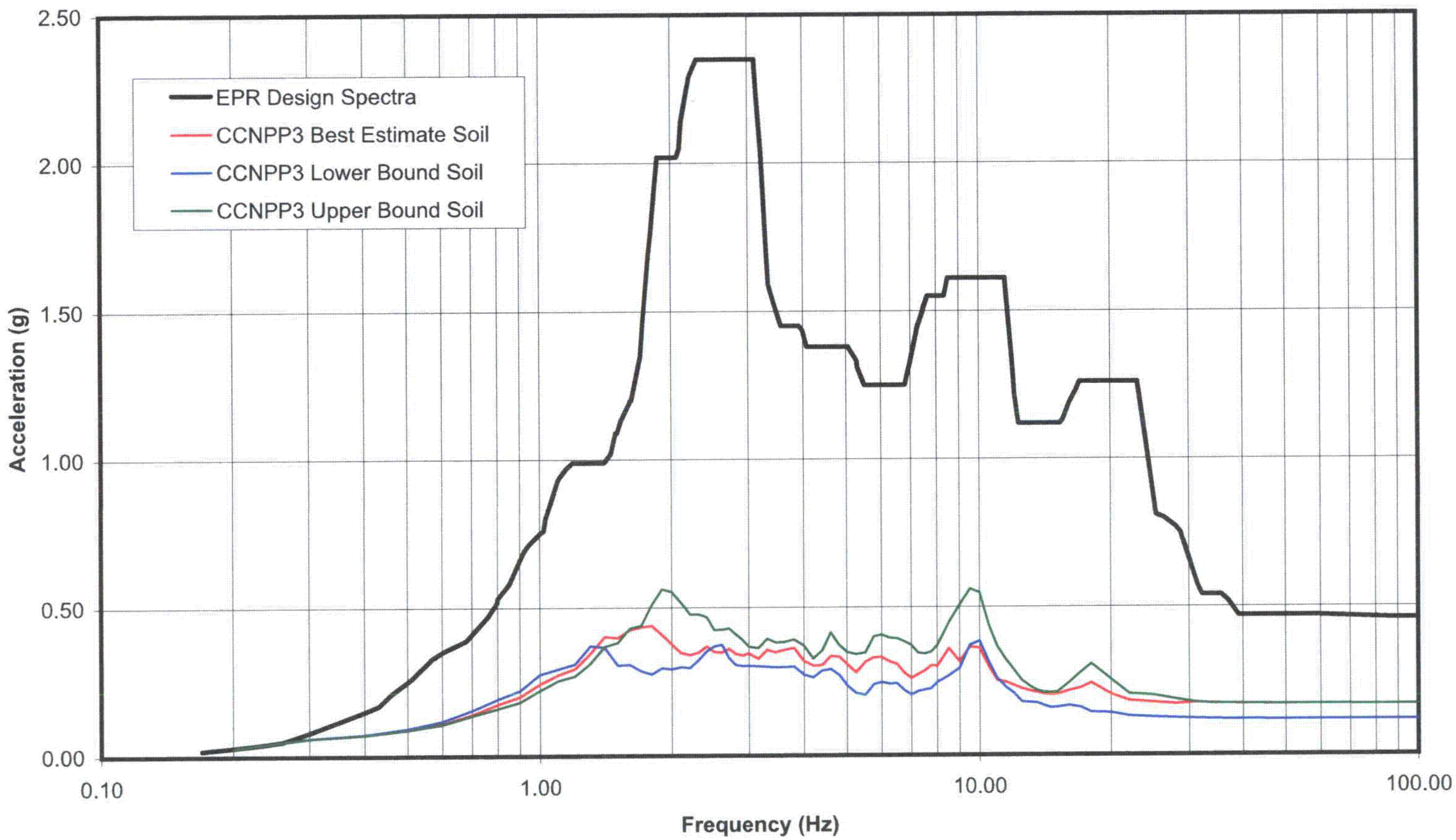


Figure 3.7-26— {Reactor Bldg Internal Structure, Elev. 5.15m, Y(N-S) Direction, 5% Damping}

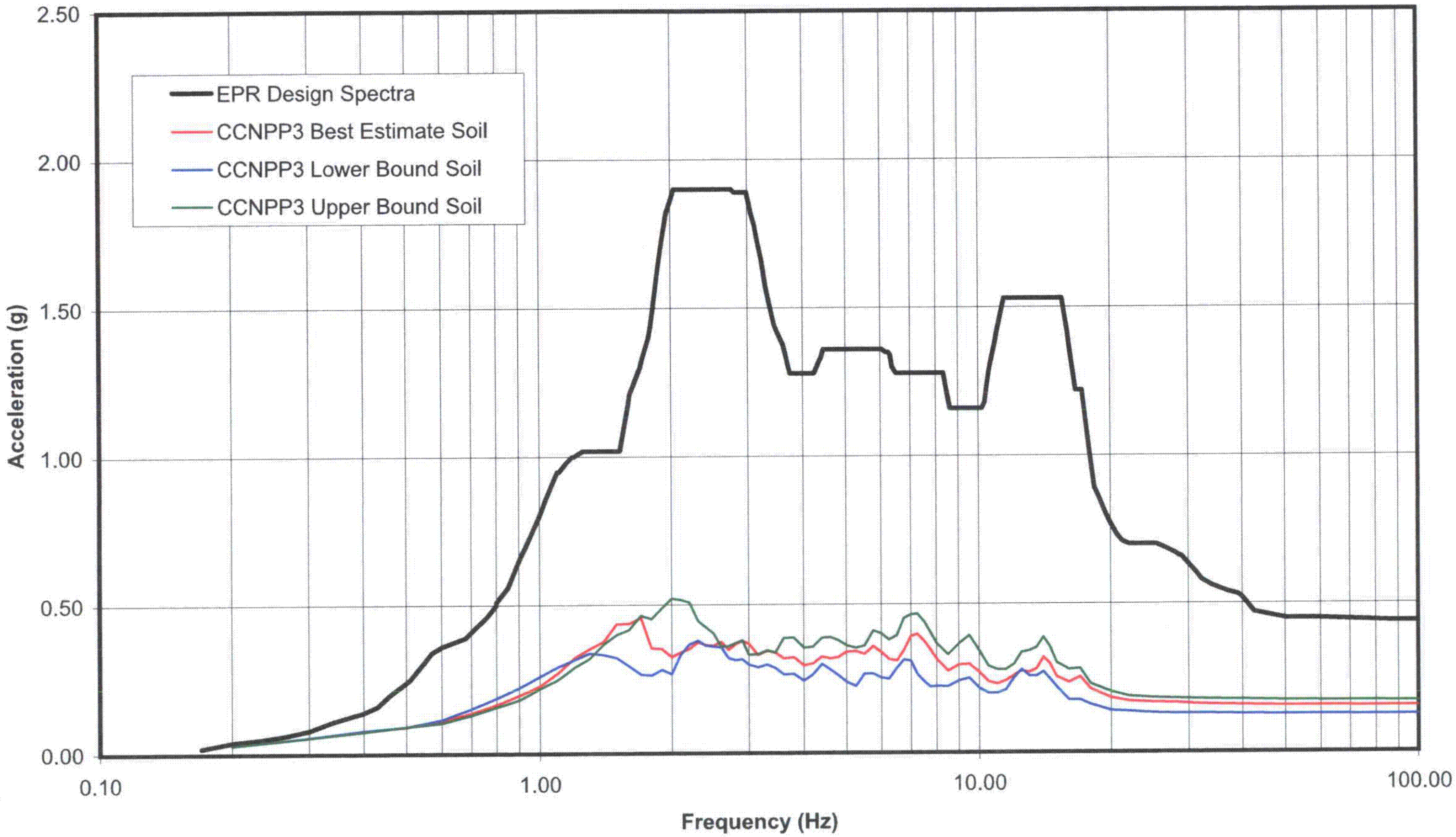


Figure 3.7-27— {Reactor Bldg Internal Structure, Elev. 5.15m, Z(Vert) Direction, 5% Damping}

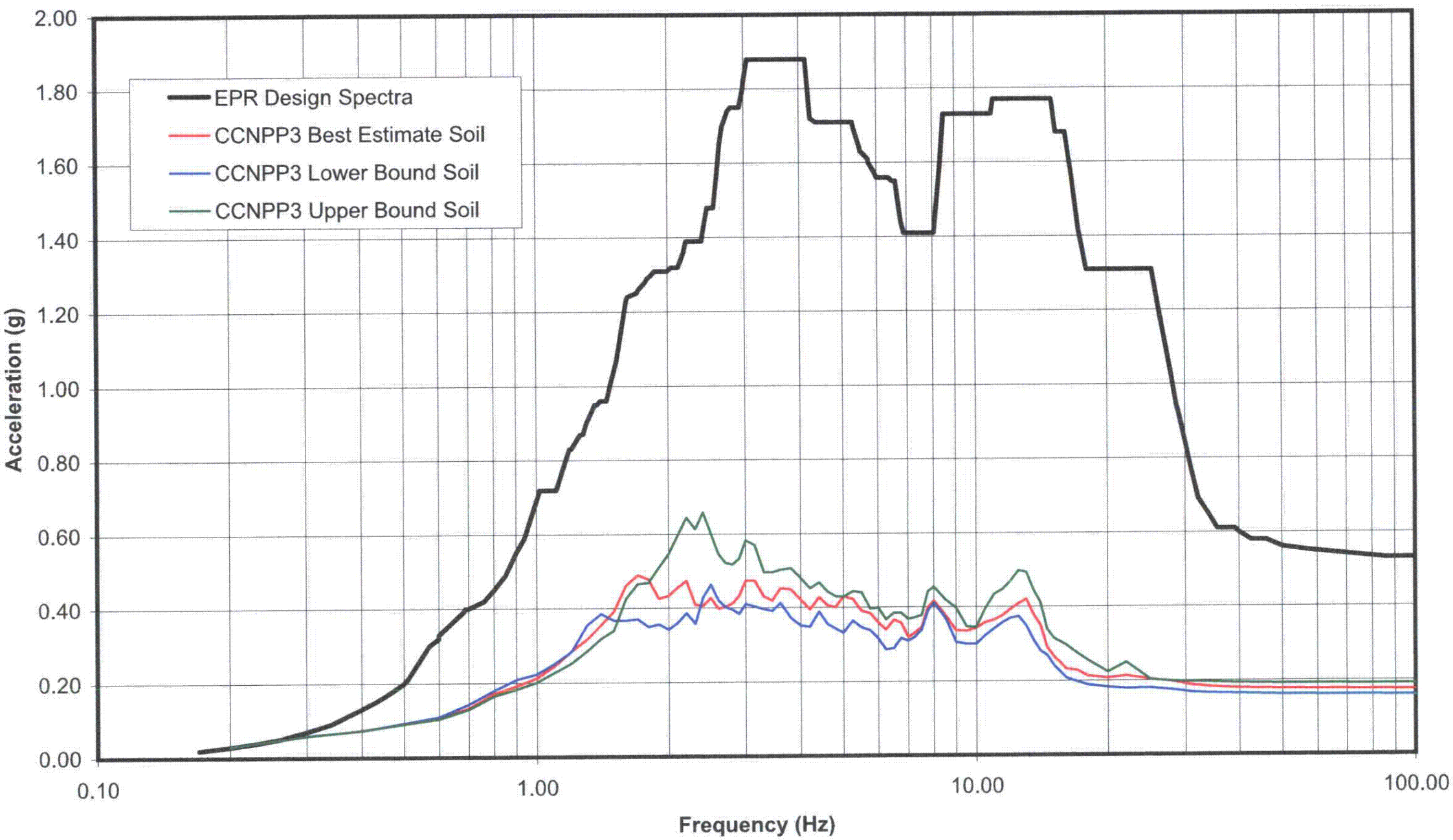


Figure 3.7-28— {Reactor Bldg Internal Structure, Elev. 19.5m, X(E-W) Direction, 5% Damping}

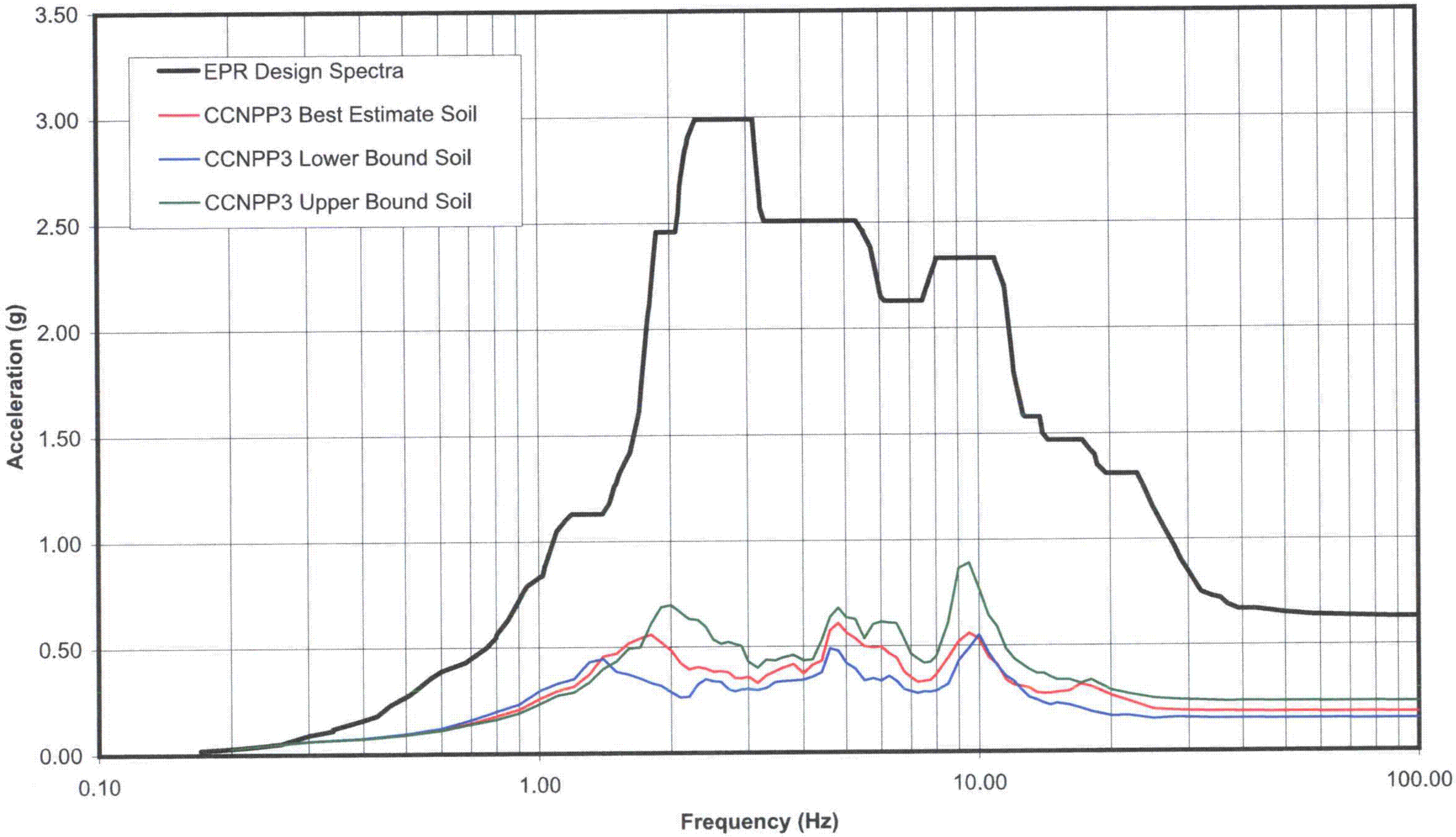


Figure 3.7-29— {Reactor Bldg Internal Structure, Elev. 19.5 m, Y(N-S) Direction, 5% Damping}

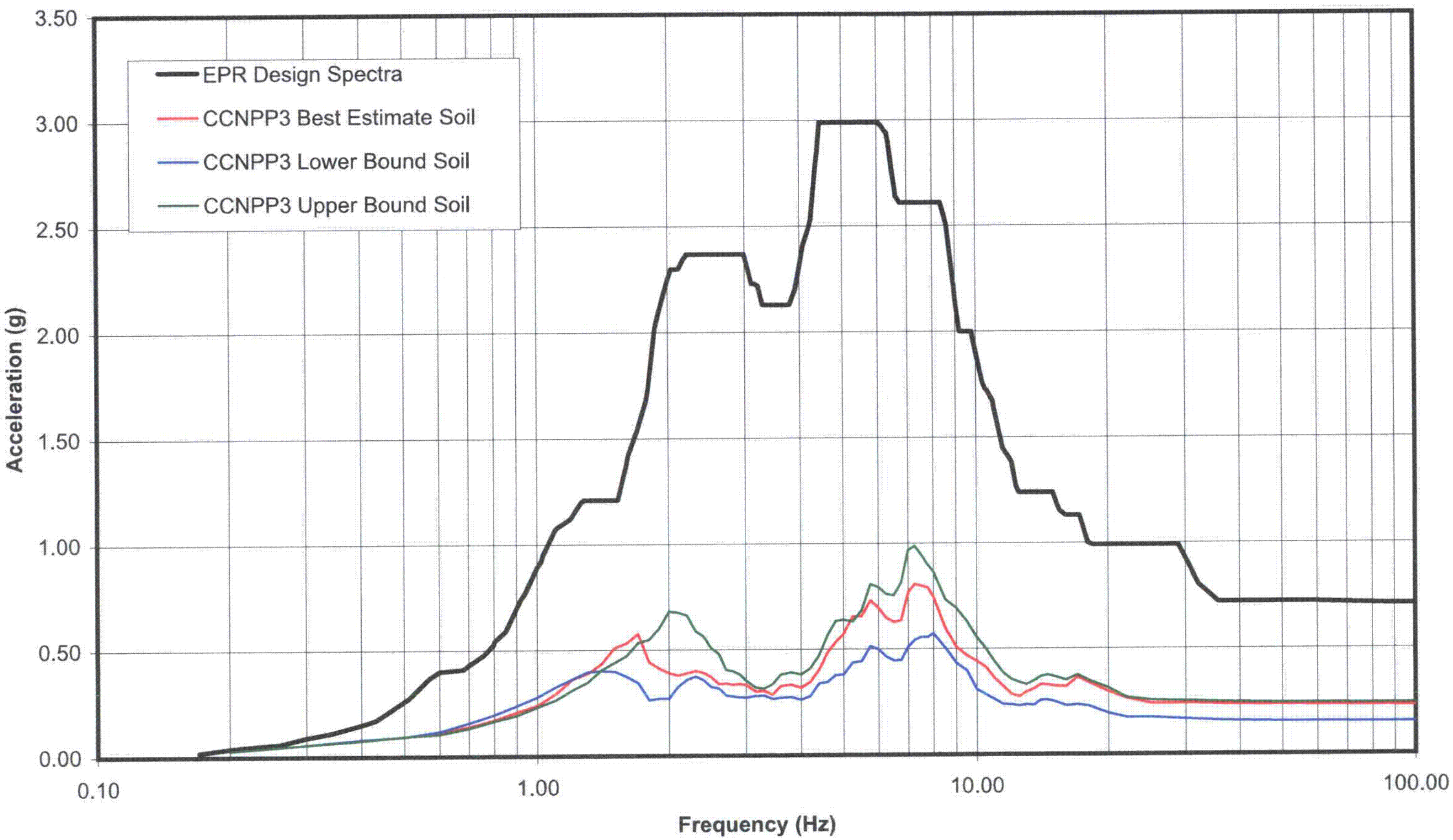


Figure 3.7-30— {Reactor Bldg Internal Structure, Elev. 19.5 m, Z(Vert) Direction, 5% Damping}

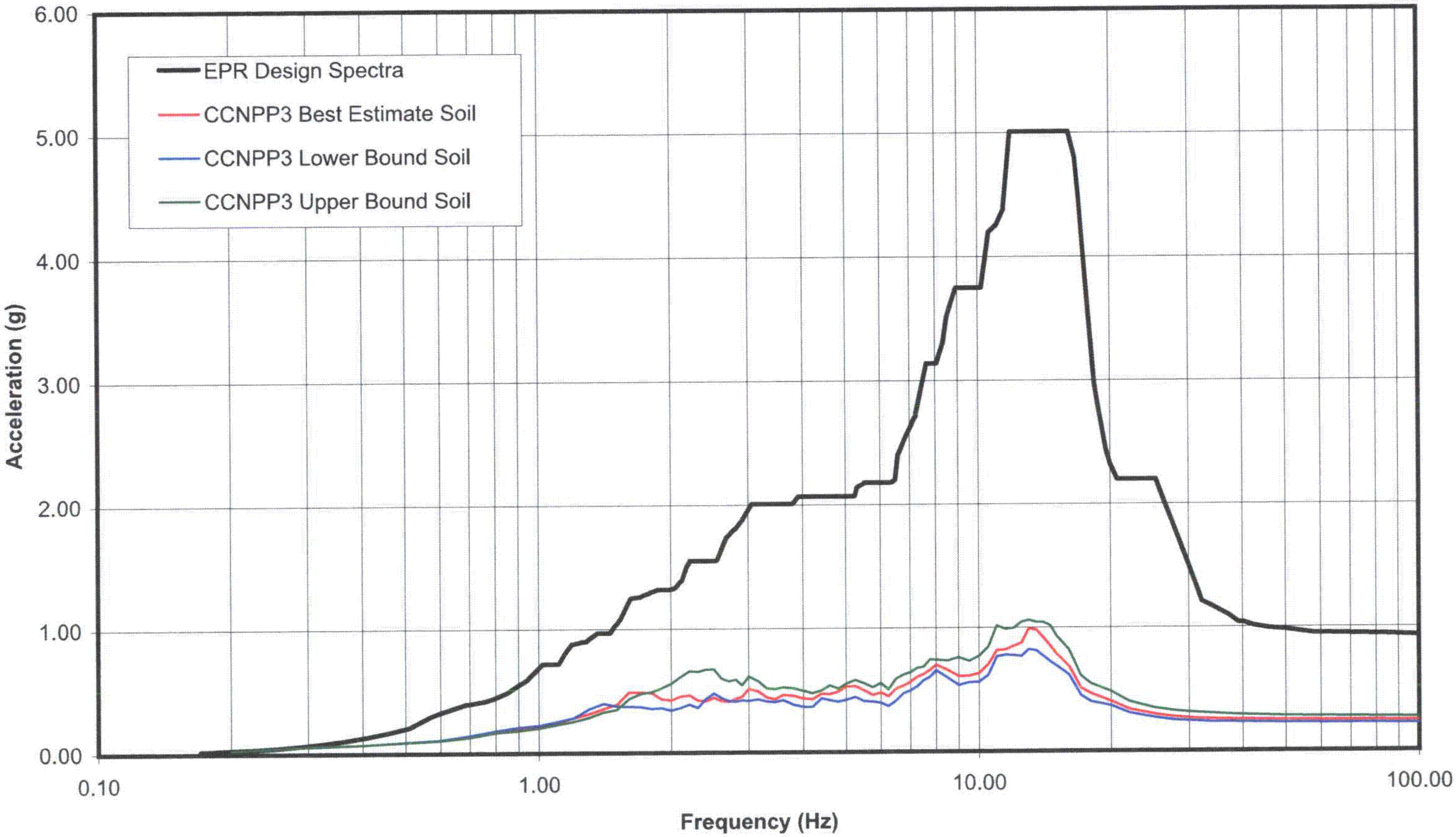


Figure 3.7-31— {Safeguard Building 1, Elev. 8.1m, X(E-W) Direction, 5% Damping}

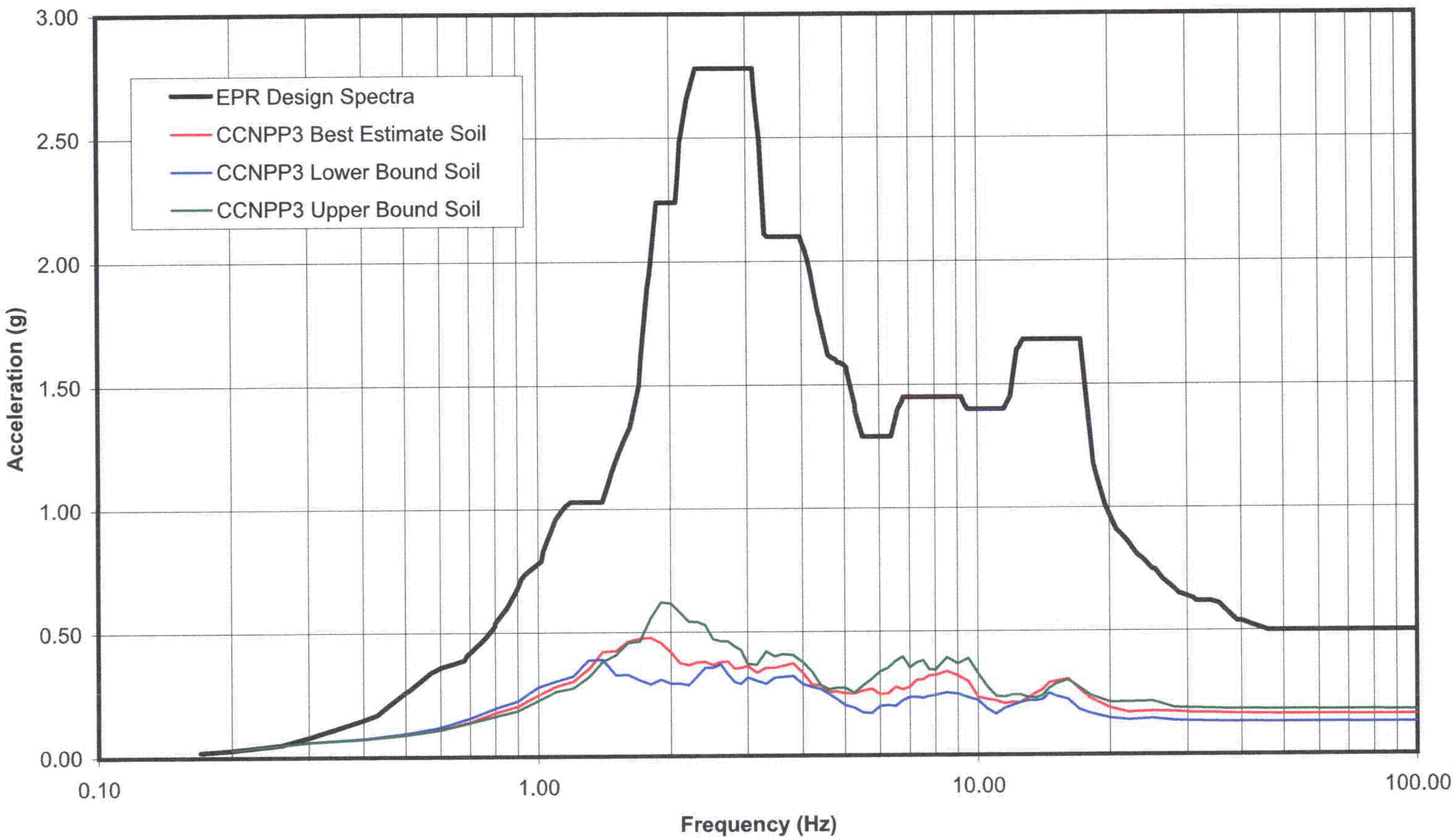


Figure 3.7-32— {Safeguard Building 1, Elev. 8.1m, Y(N-S) Direction, 5% Damping}

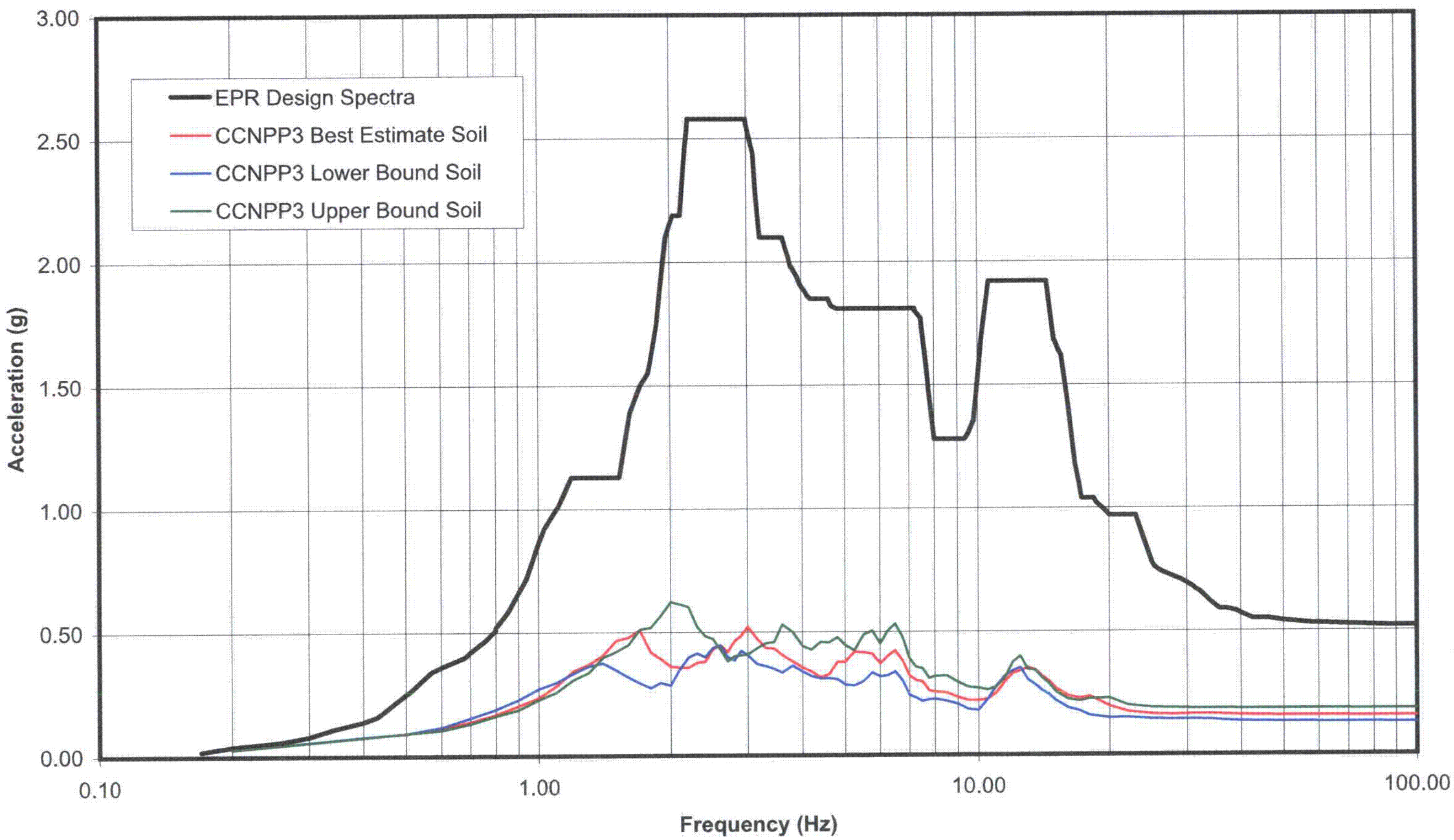


Figure 3.7-33— {Safeguard Building 1, Elev. 8.1m, Z(Vert) Direction, 5% Damping}

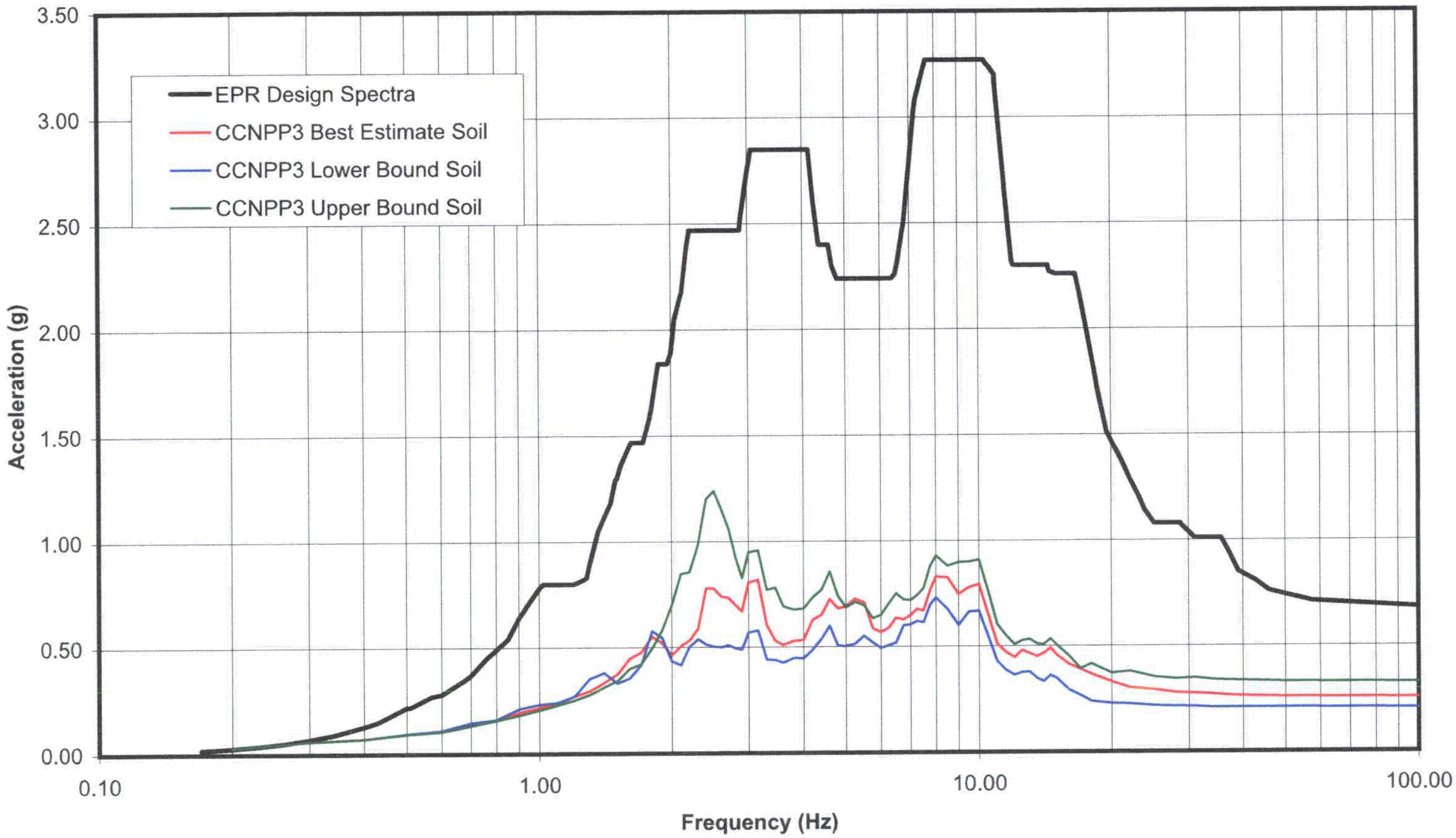


Figure 3.7-34— {Safeguard Building 1, Elev. 21.0 m, X(E-W) Direction, 5% Damping}

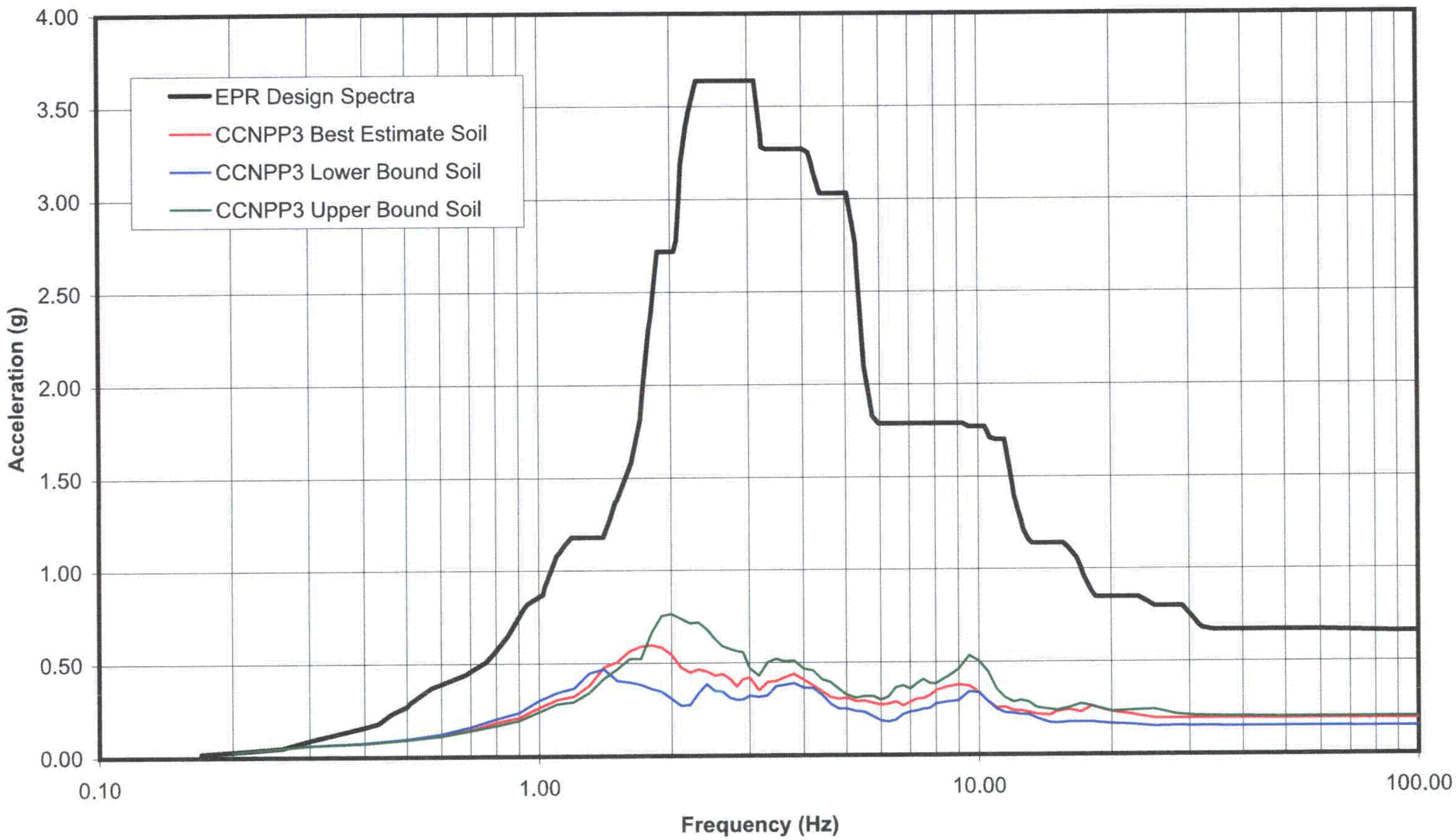


Figure 3.7-35— {Safeguard Building 1, Elev. 21.0 m, Y(N-S) Direction, 5% Damping}

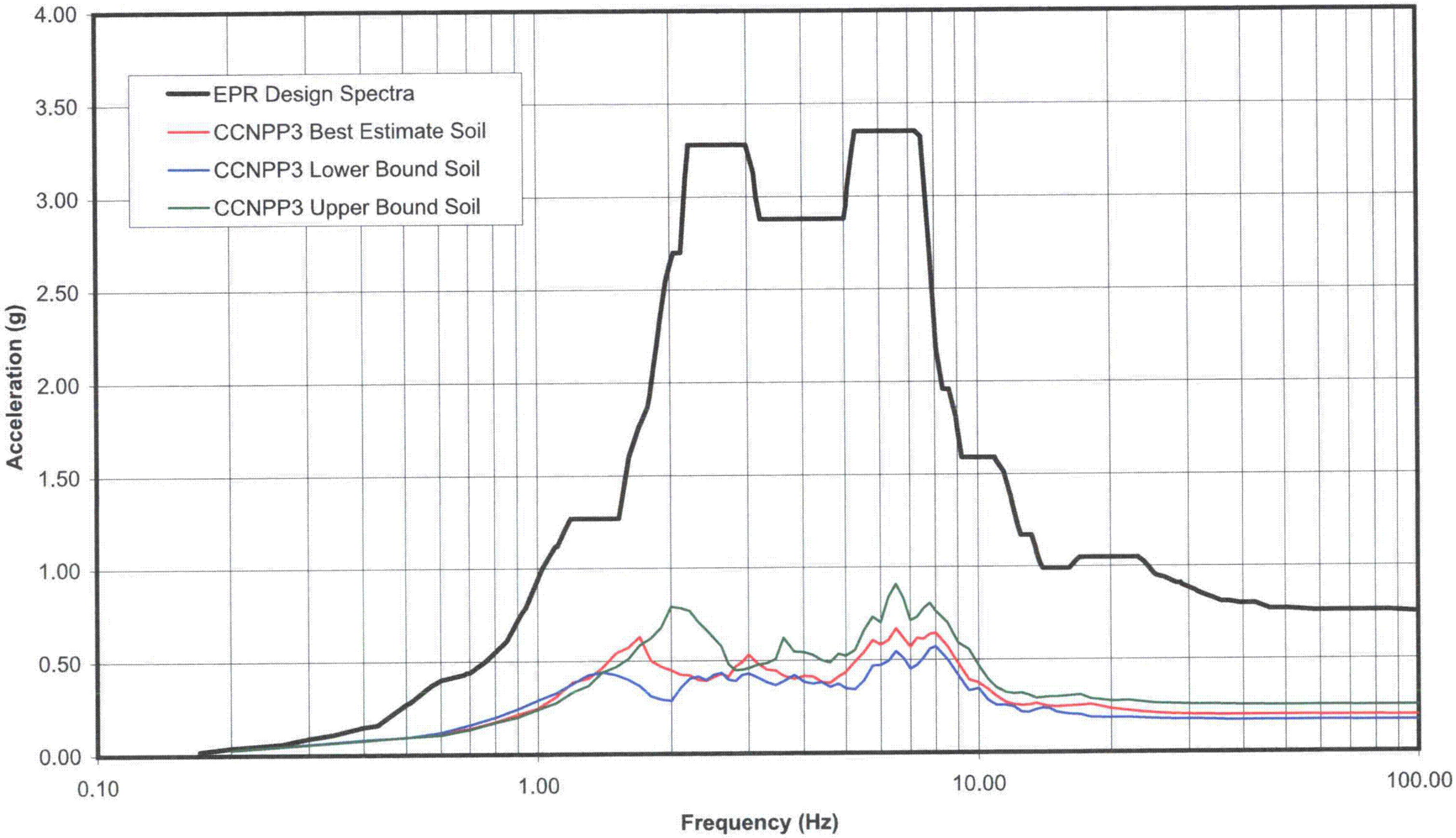


Figure 3.7-36— {Safeguard Building 1, Elev. 21.0 m, Z(Vert) Direction, 5% Damping}

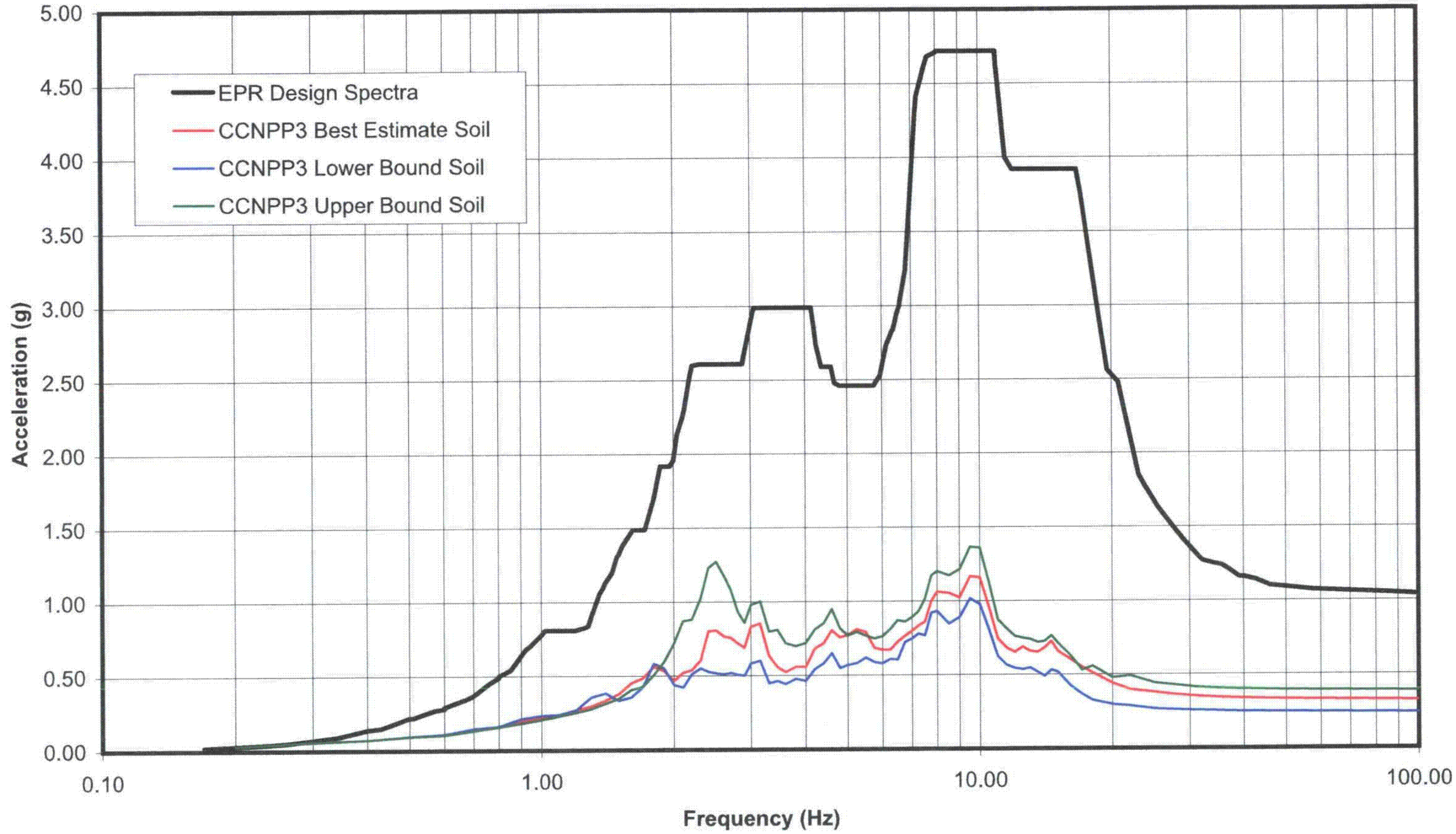


Figure 3.7-37— {Safeguard Building 2/3, Elev. 8.1m, X(E-W) Direction, 5% Damping}

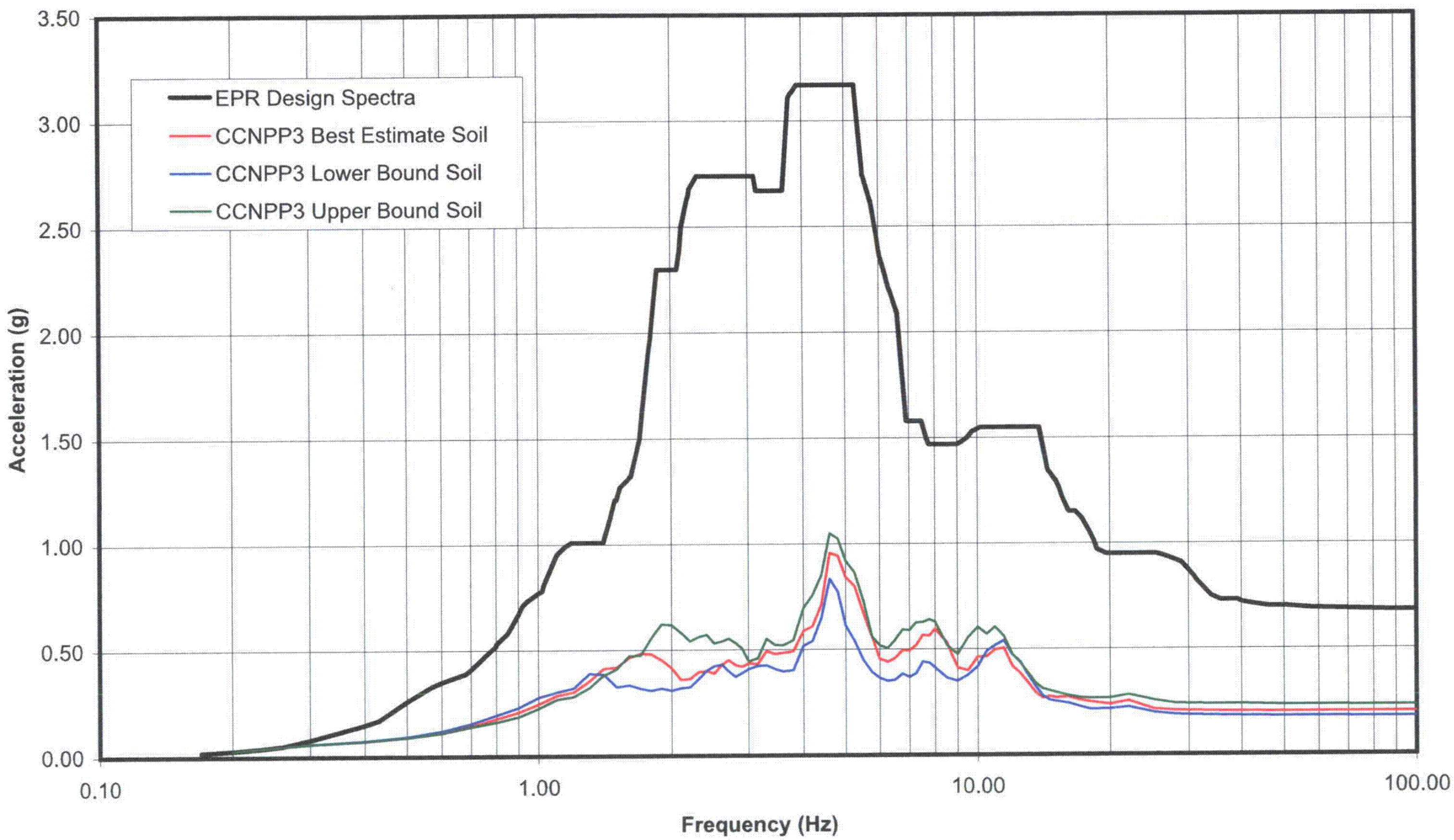


Figure 3.7-38— {Safegurd Building 2/3, Elev. 8.1m, Y(N-S) Direction, 5% Damping}

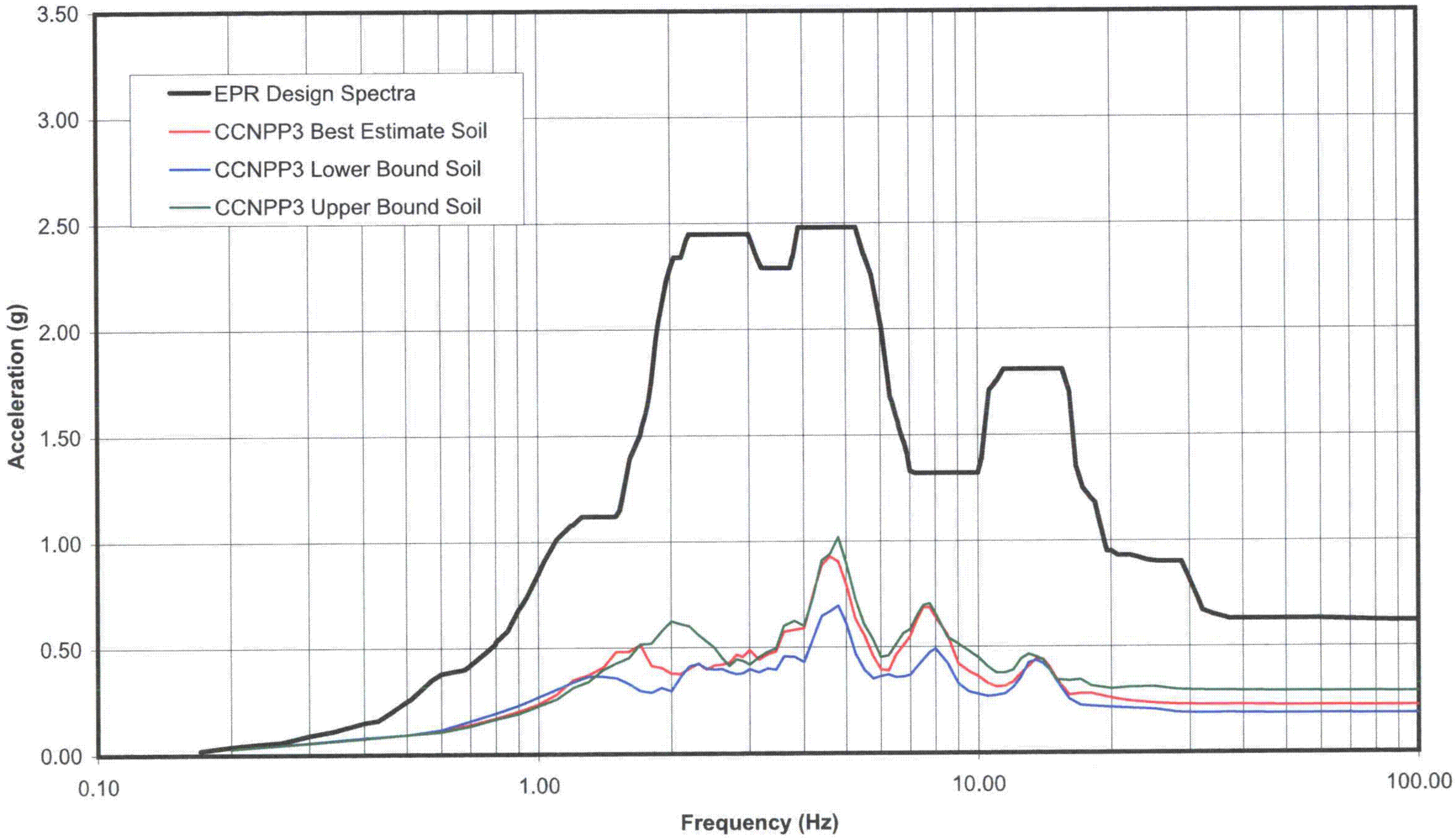


Figure 3.7-39— {Safeguard Building 2/3, Elev. 8.1m, Z(Vert) Direction, 5% Damping}

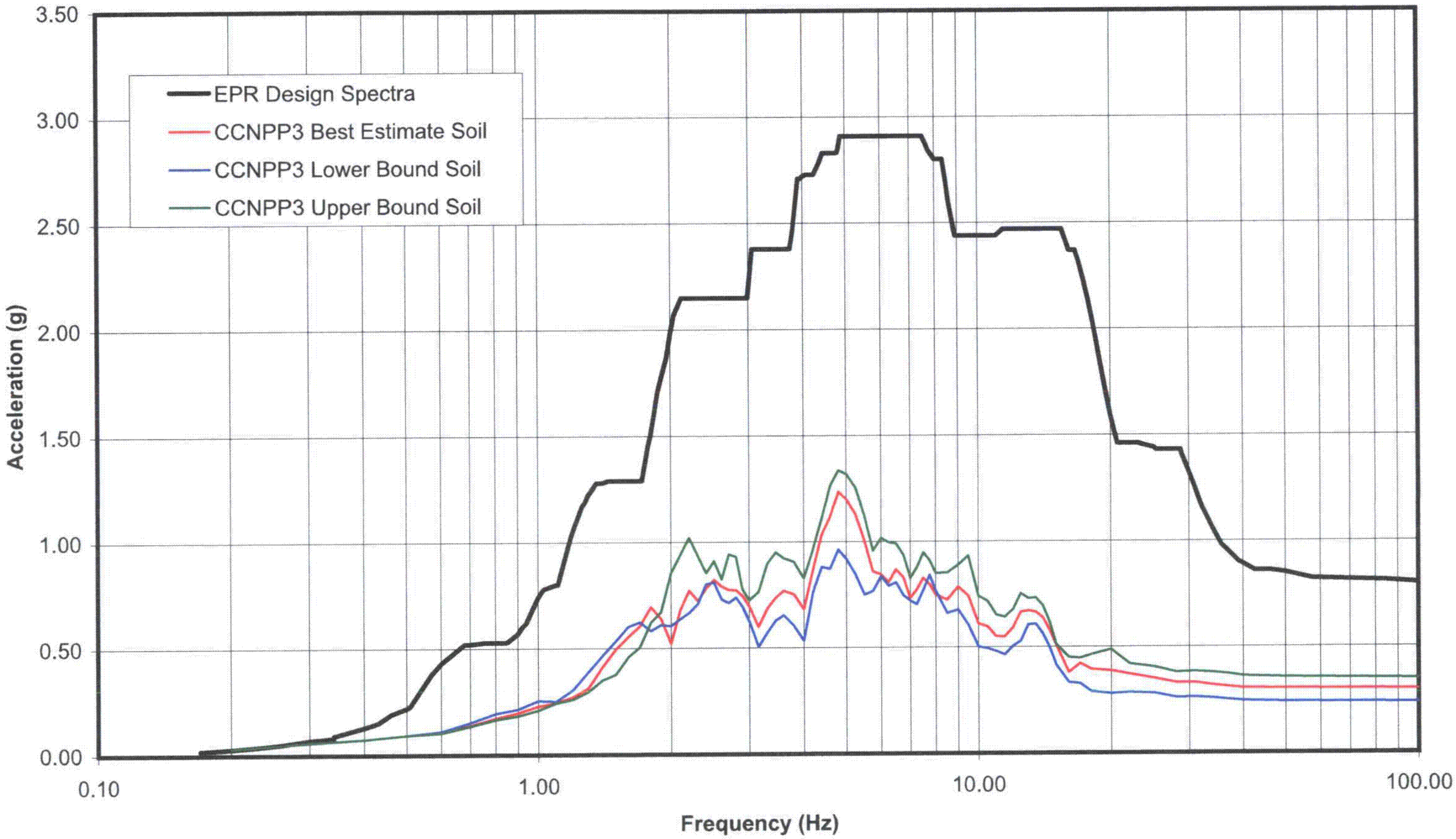


Figure 3.7-40— {Safeguard Building 2/3, Elev. 15.4 m, X(E-W) Direction, 5% Damping}

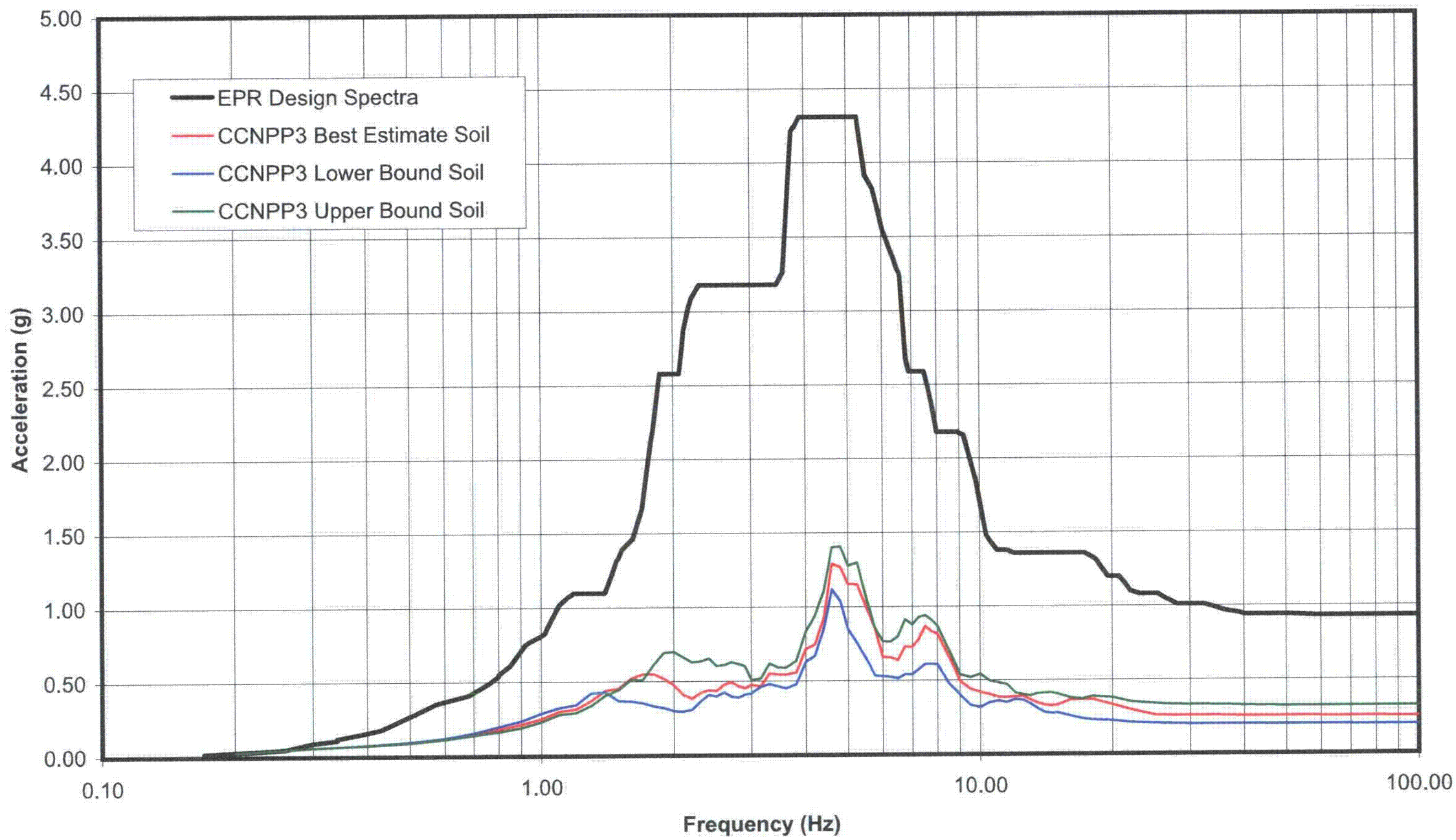


Figure 3.7-41— {Safeguard Building 2/3, Elev. 15.4 m, Y(N-S) Direction, 5% Damping}

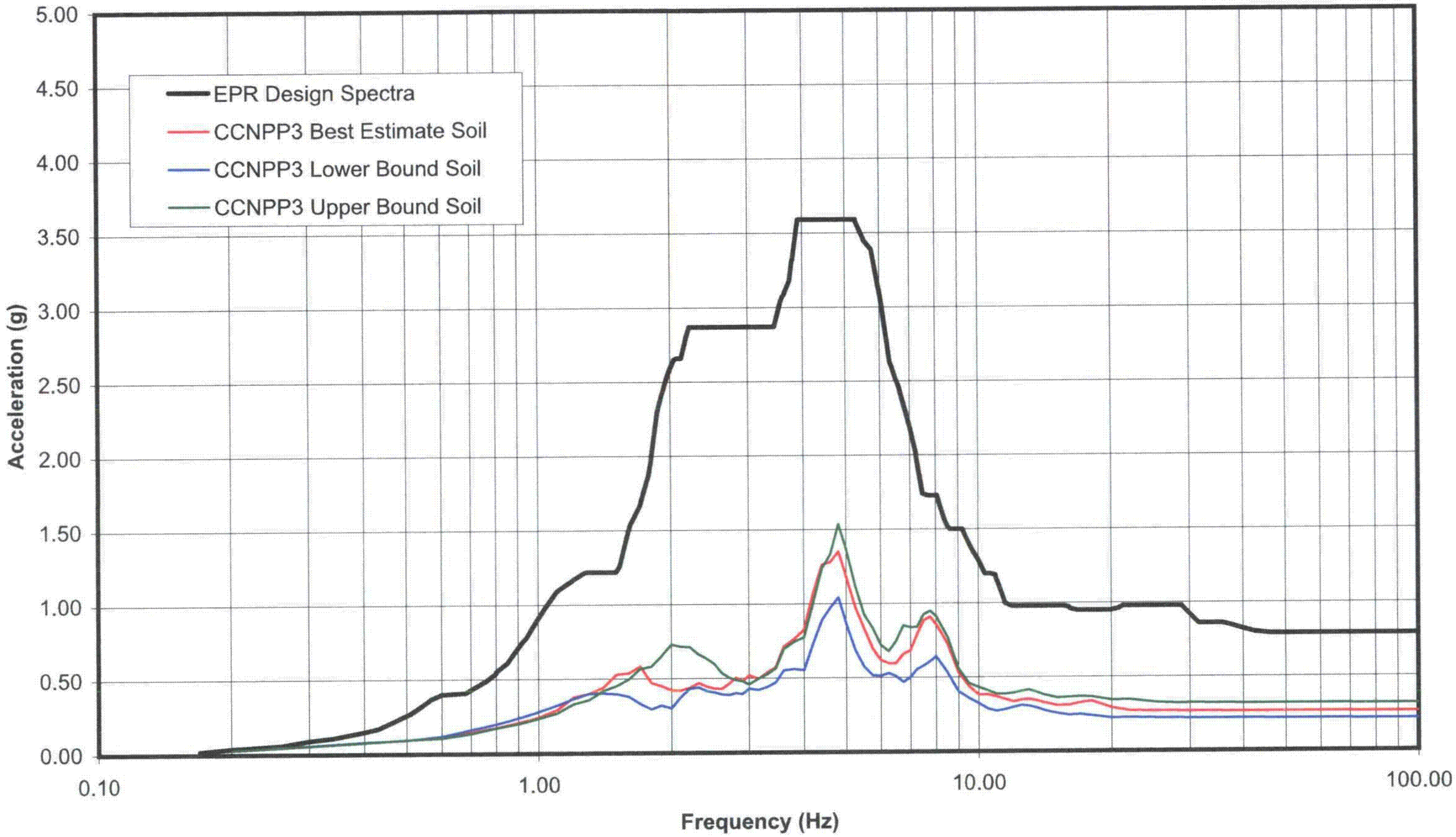


Figure 3.7-42— {Safeguard Building 2/3, Elev. 15.4 m, Z(Vert) Direction, 5% Damping}

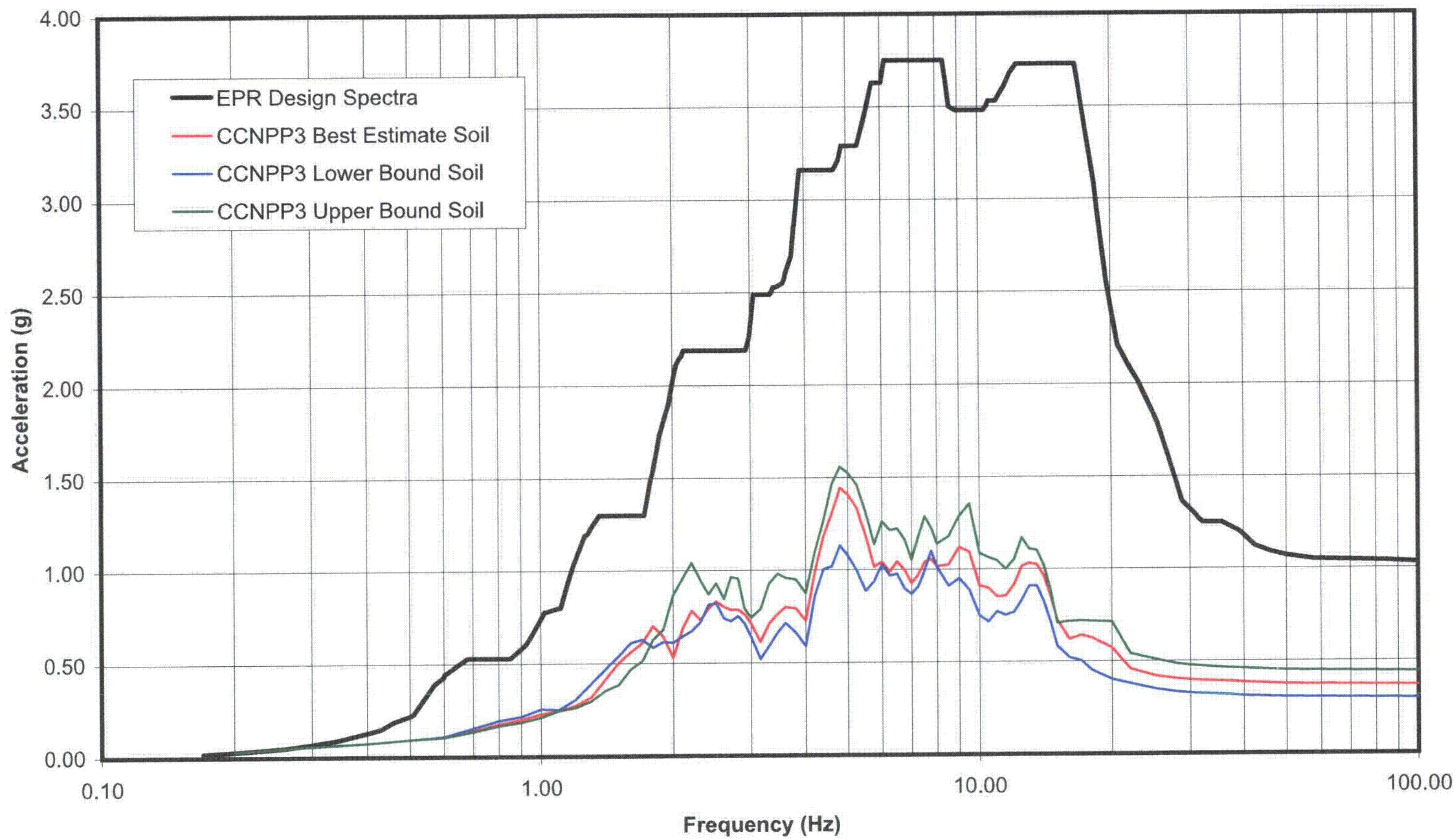


Figure 3.7-43— {Safeguard Building 4, Elev. 21.0 m, X(E-W) Direction, 5% Damping}

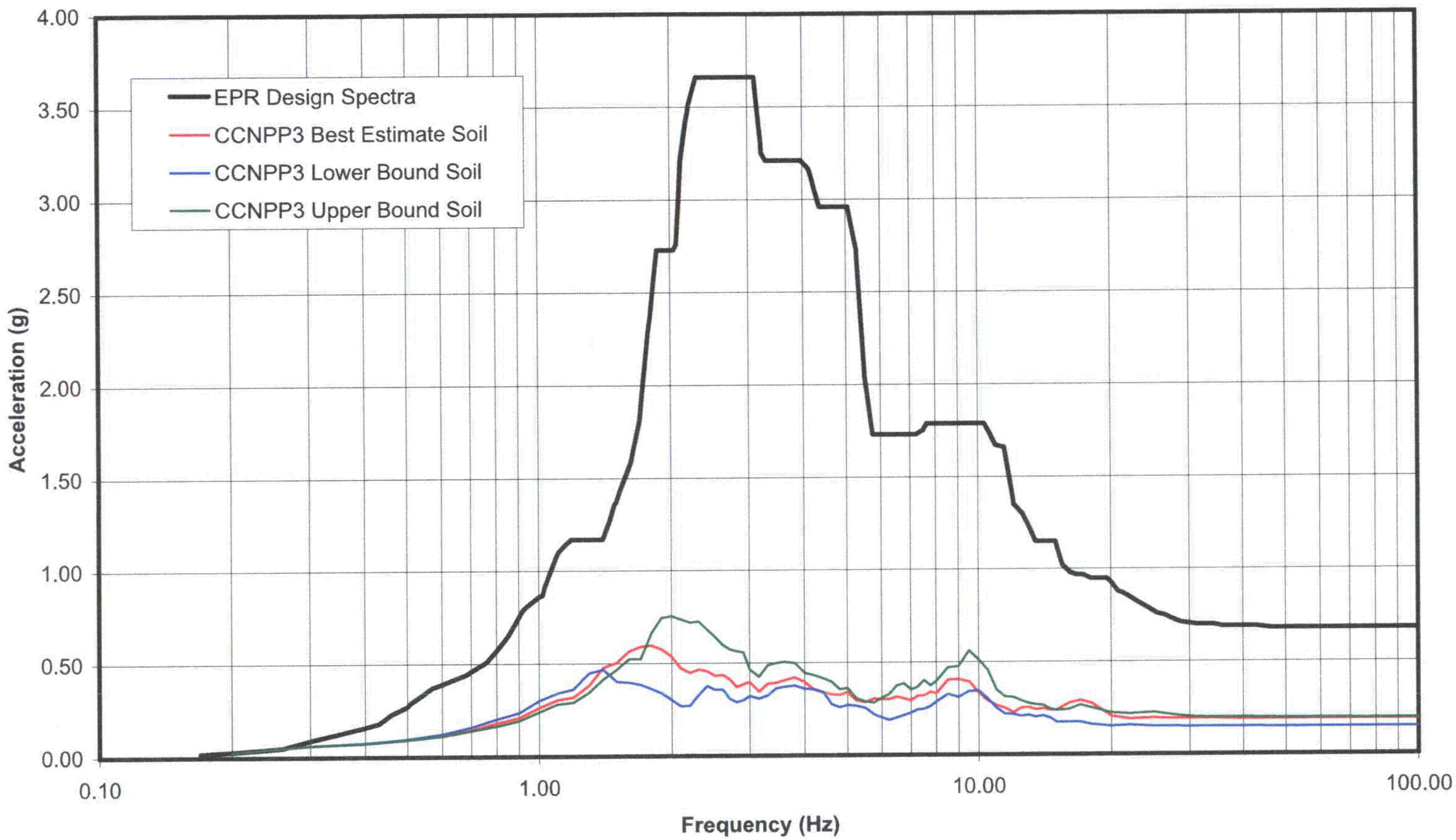


Figure 3.7-44— {Safeguard Building 4, Elev. 21.0m, Y(N-S) Direction, 5% Damping}

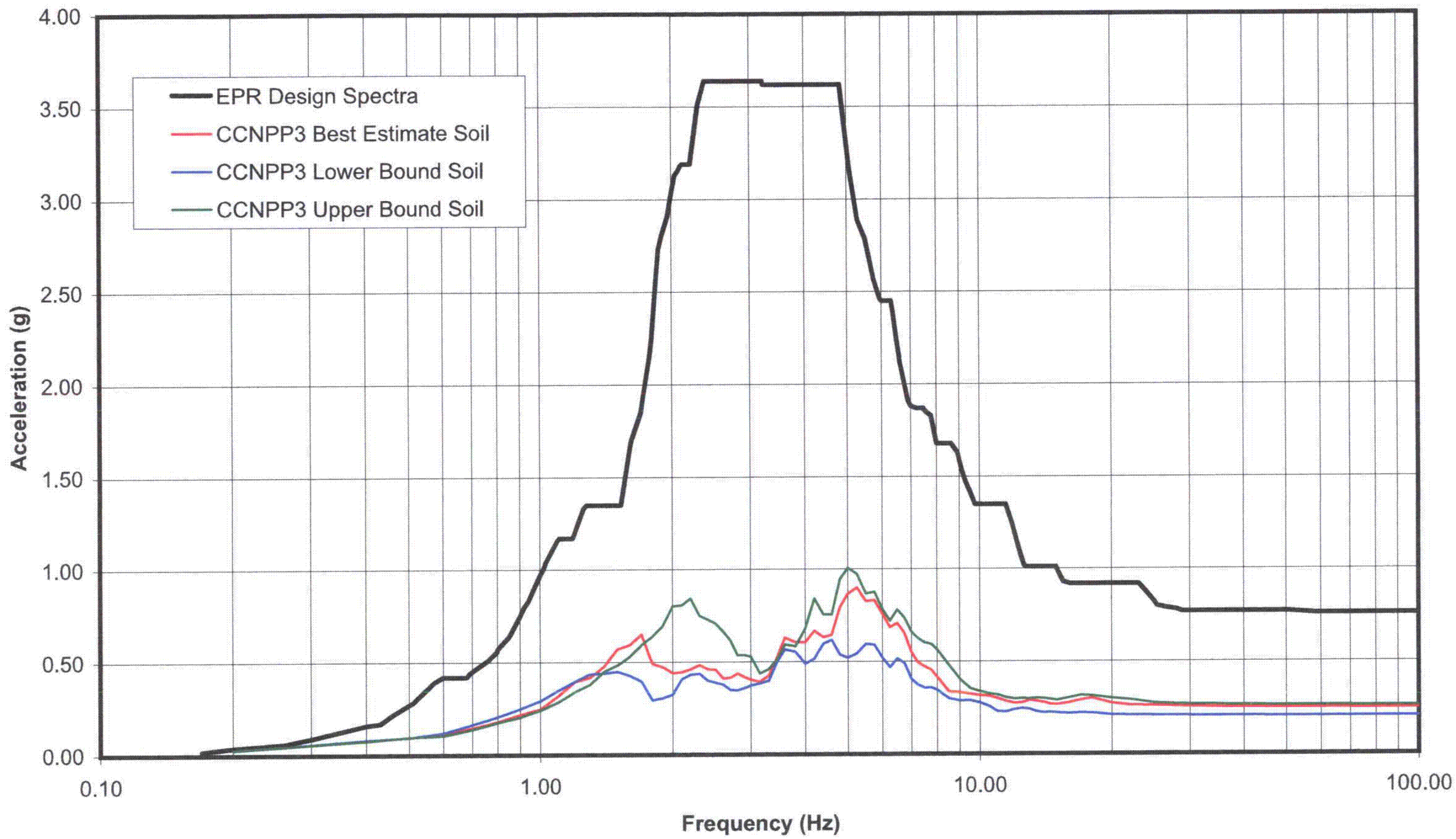


Figure 3.7-45— {Safeguard Building 4, Elev. 21.0m, Z(Vert) Direction, 5% Damping}

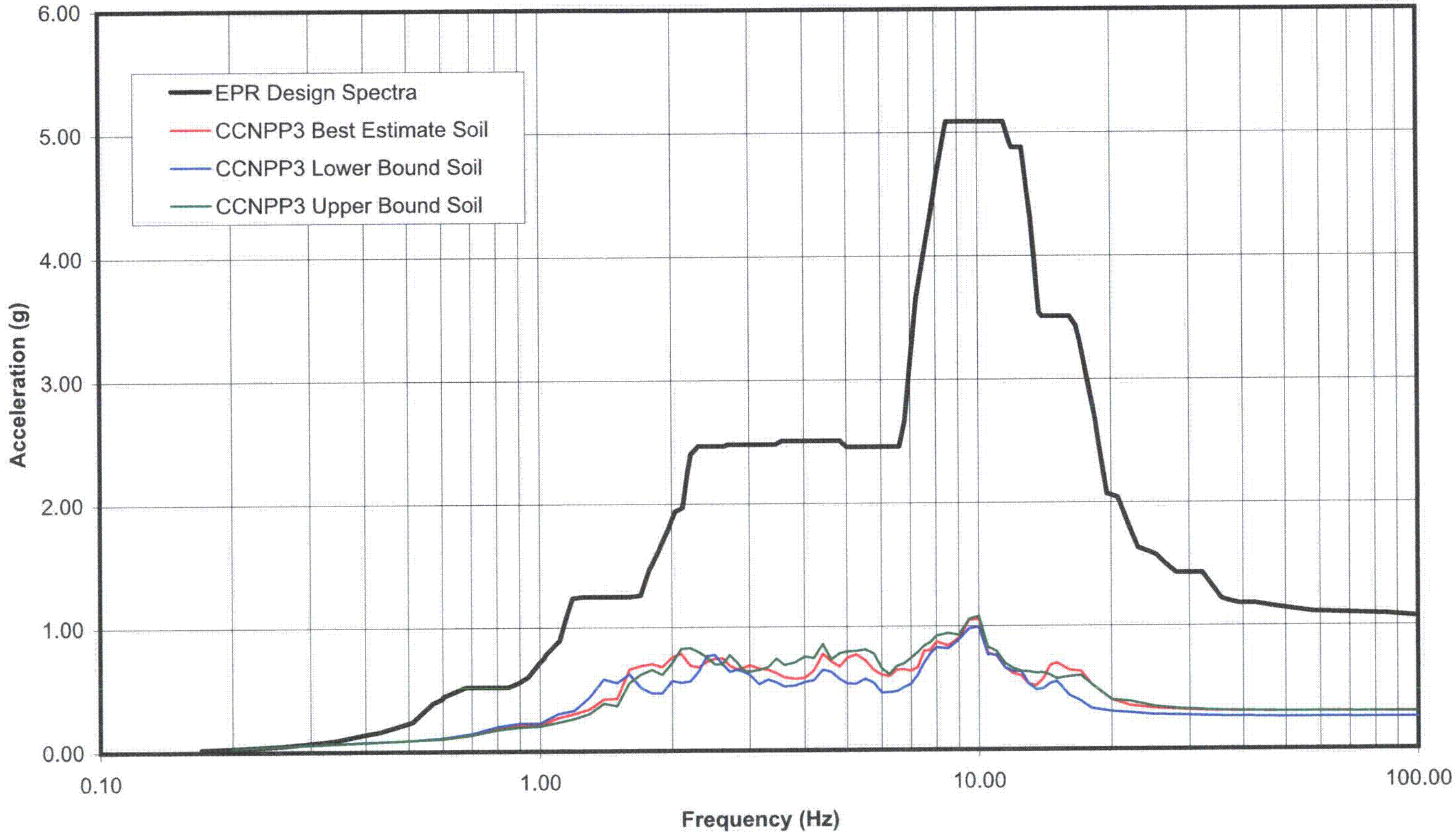


Figure 3.7-46— {Containment Building, Elev. 37.6 m, X(E-W) Direction, 5% Damping}

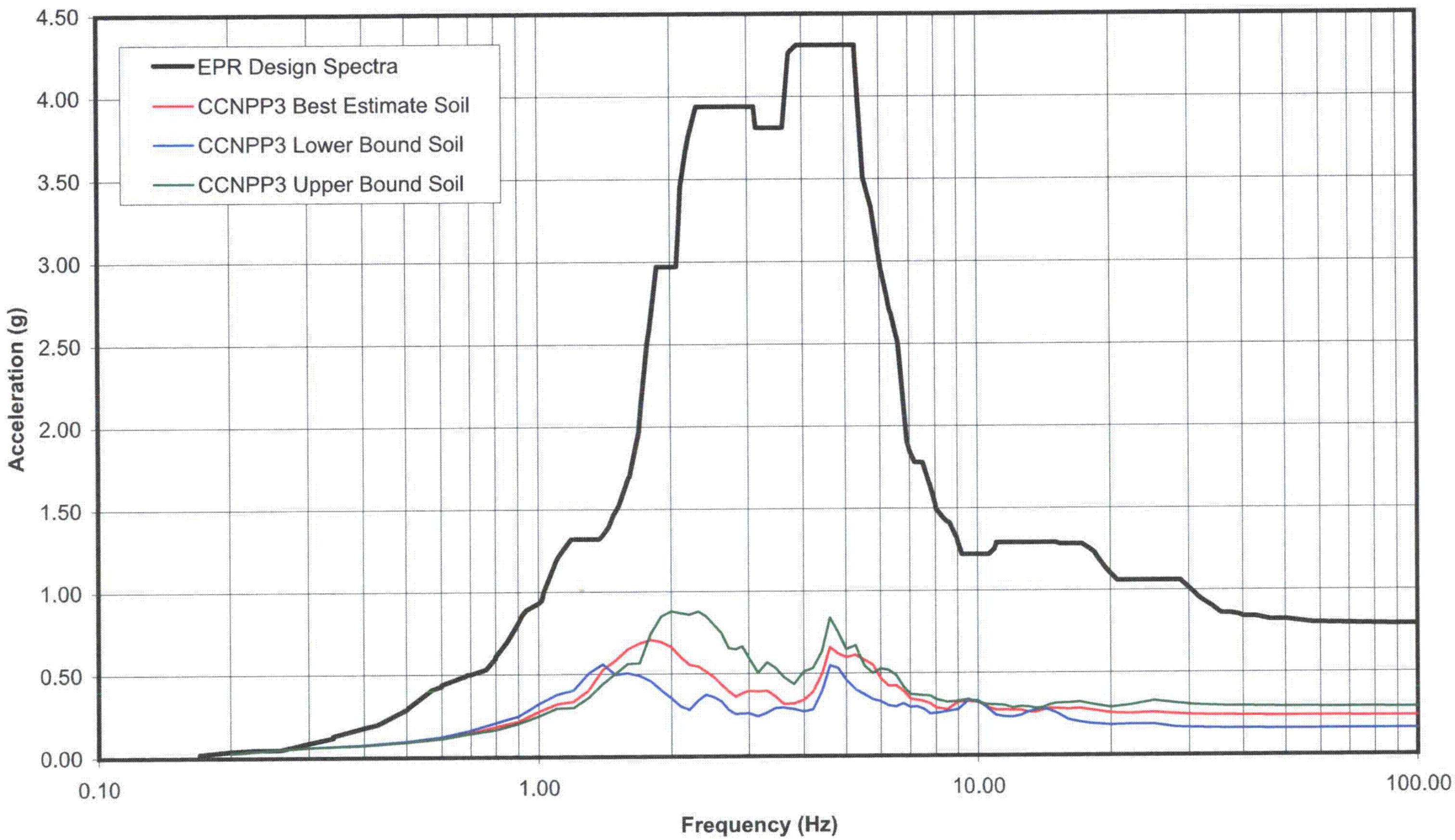


Figure 3.7-47— {Containment Building, Elev. 37.6m, Y(N-S) Direction, 5% Damping}

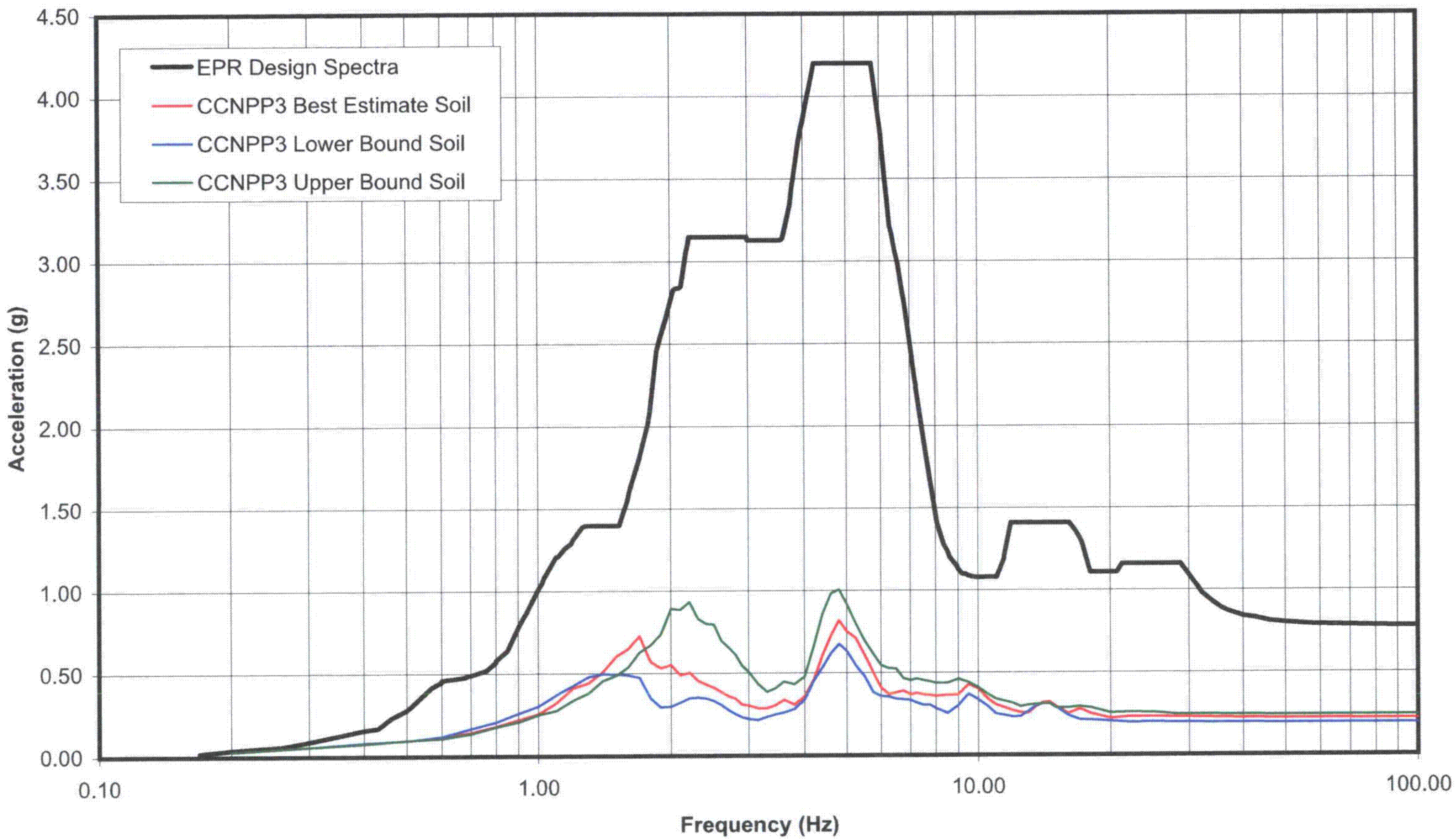


Figure 3.7-48— {Containment Building, Elev. 37.6 m, Z(Vert) Direction, 5% Damping}

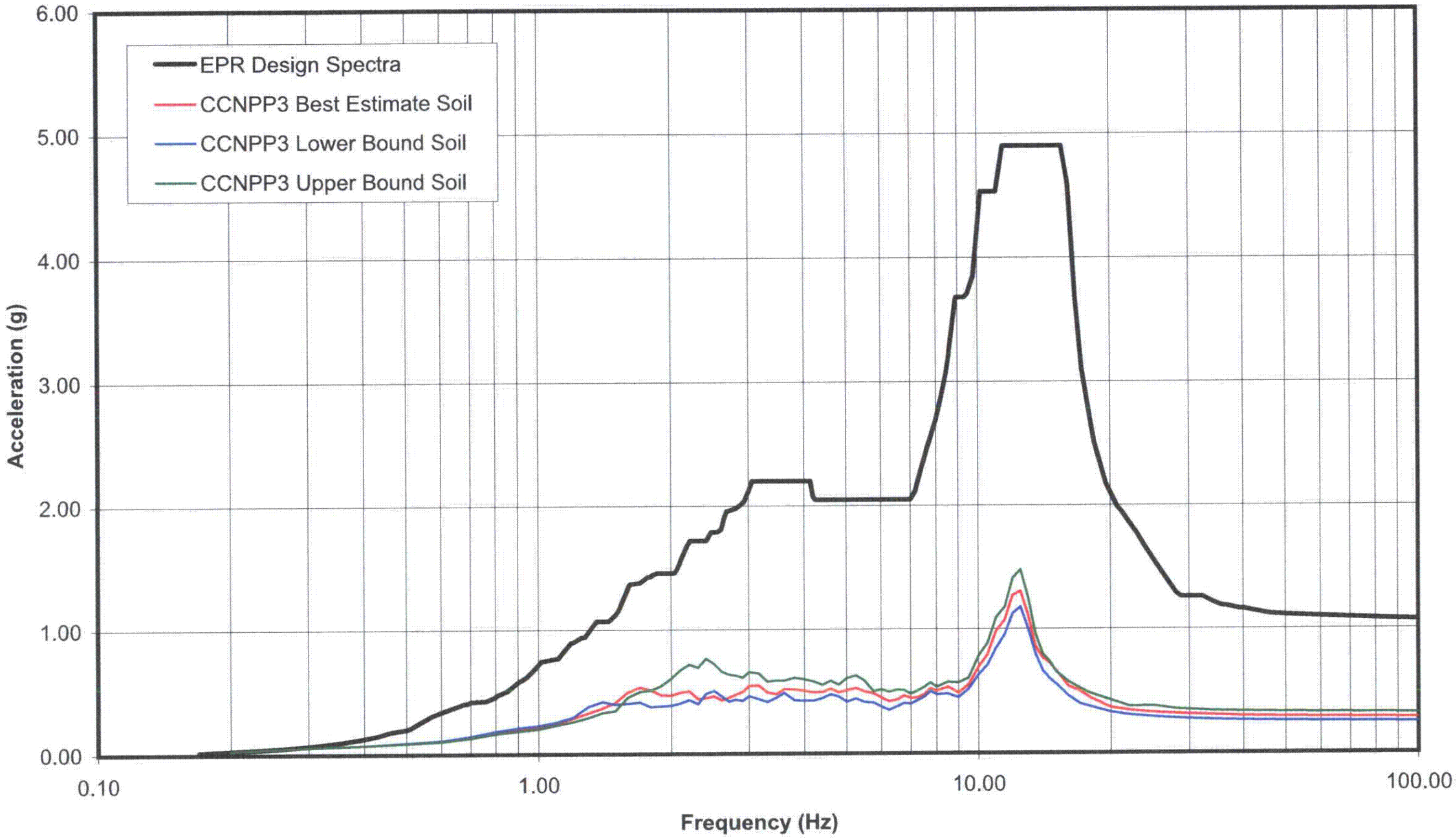


Figure 3.7-49— {Containment Building, Elev. 58.0 m, X(E-W) Direction, 5% Damping}

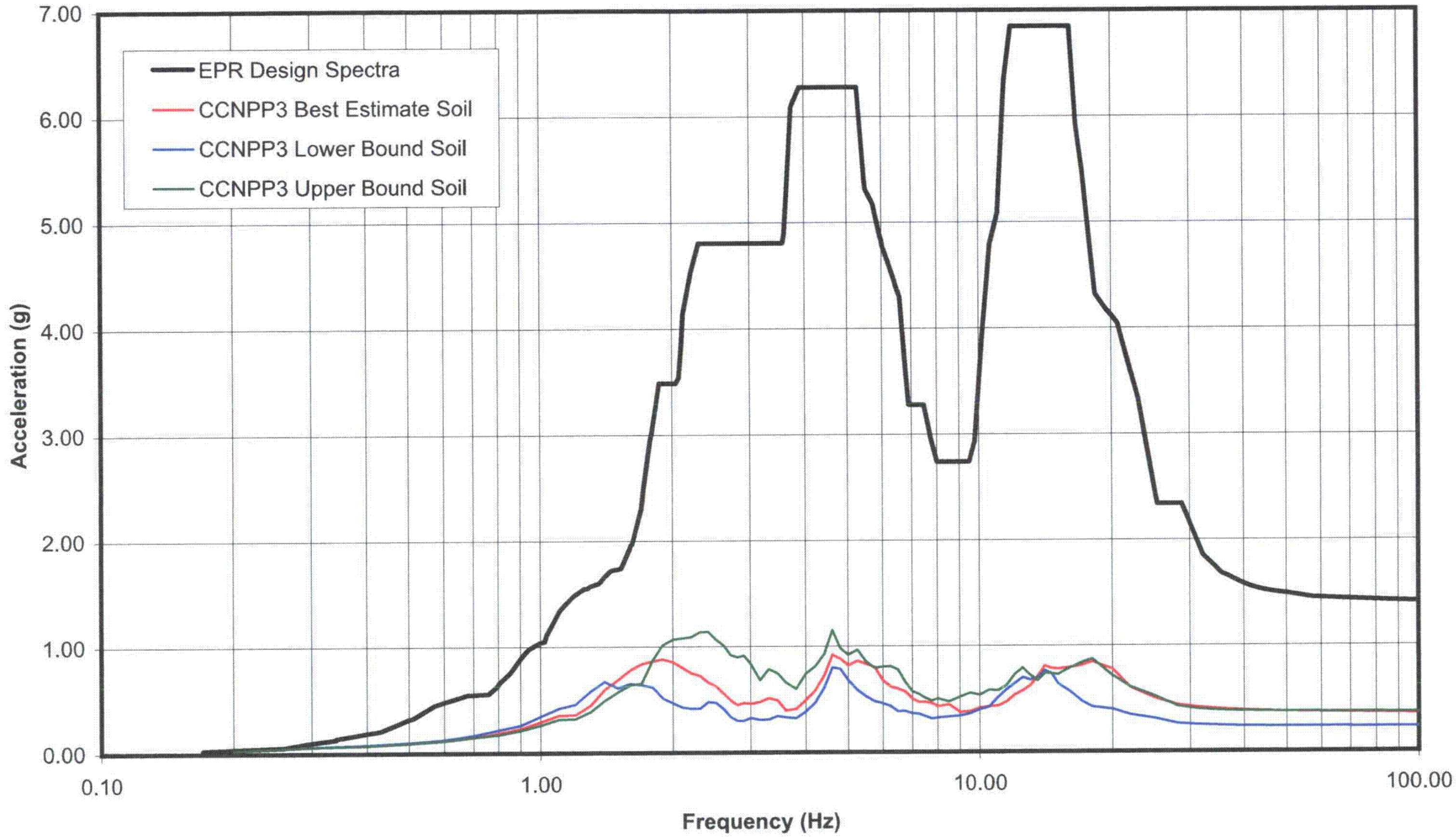


Figure 3.7-50— {Containment Building, Elev. 58.0 m, Y(N-S) Direction, 5% Damping}

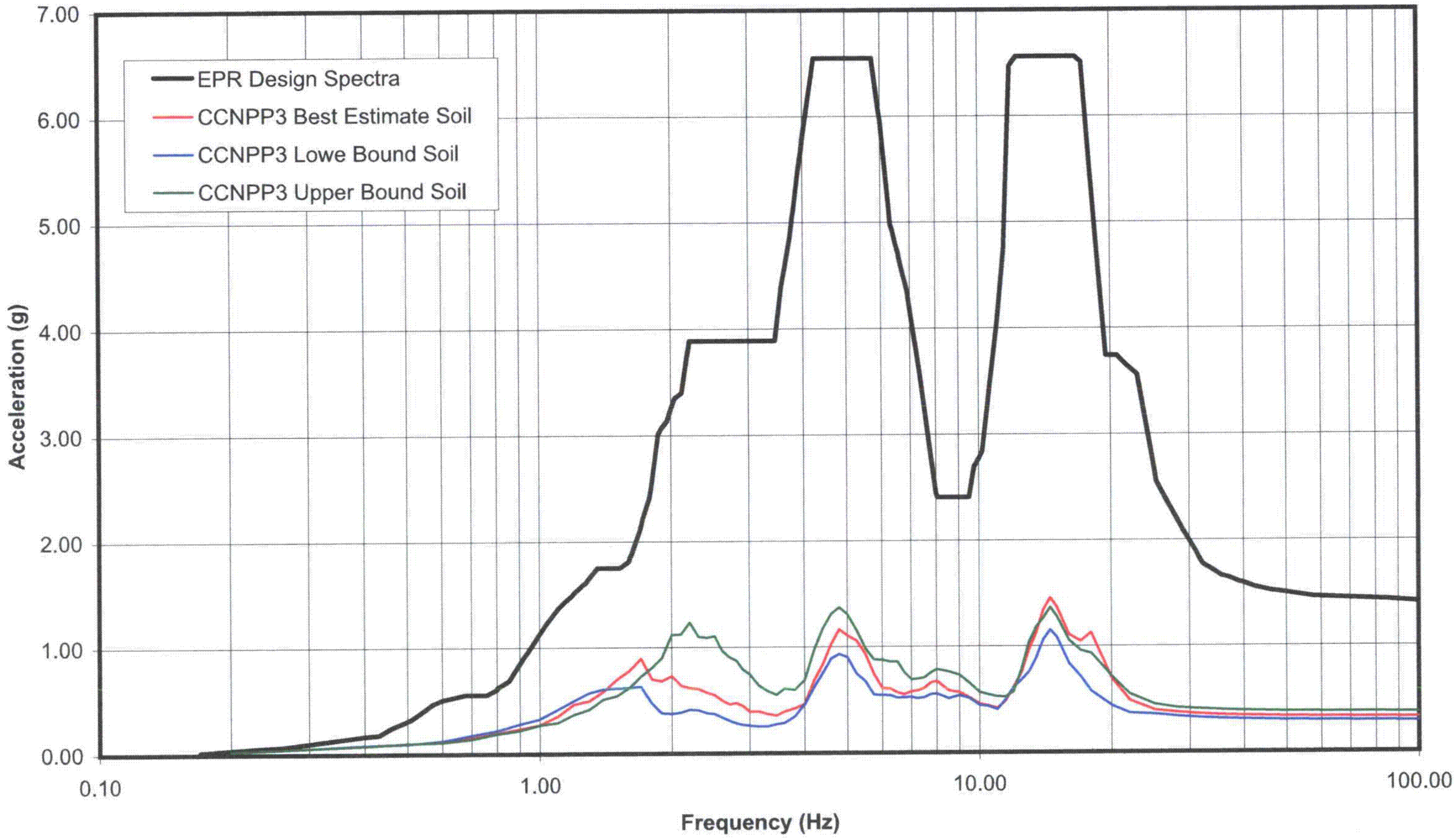


Figure 3.7-51— {Containment Building, Elev. 58.0 m, Z(Vert) Direction, 5% Damping}

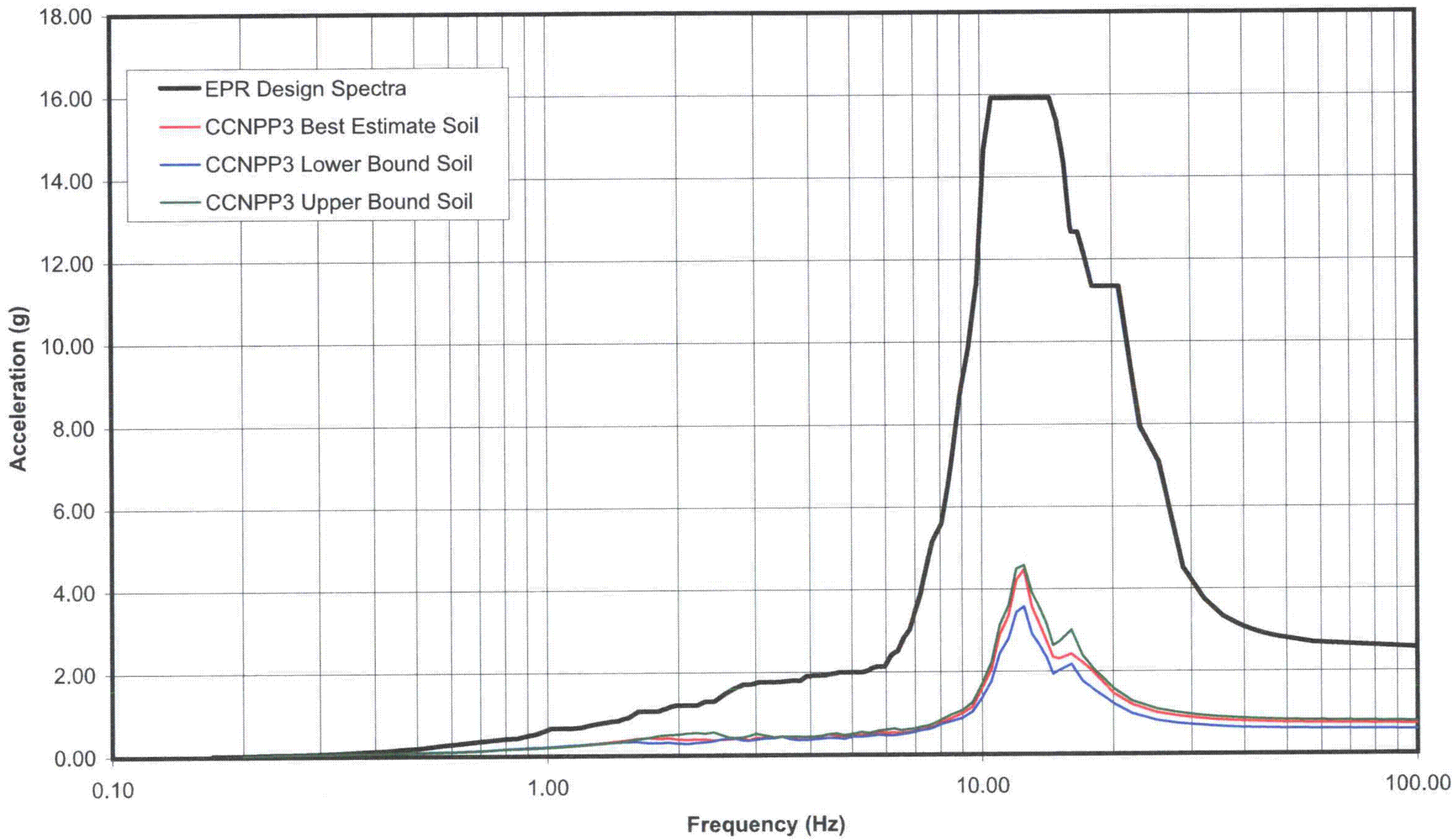


Figure 3.7-52— {CCNPP Unit 3 NAB Basemat X(E-W) Direction Spectra (5% Damping)}

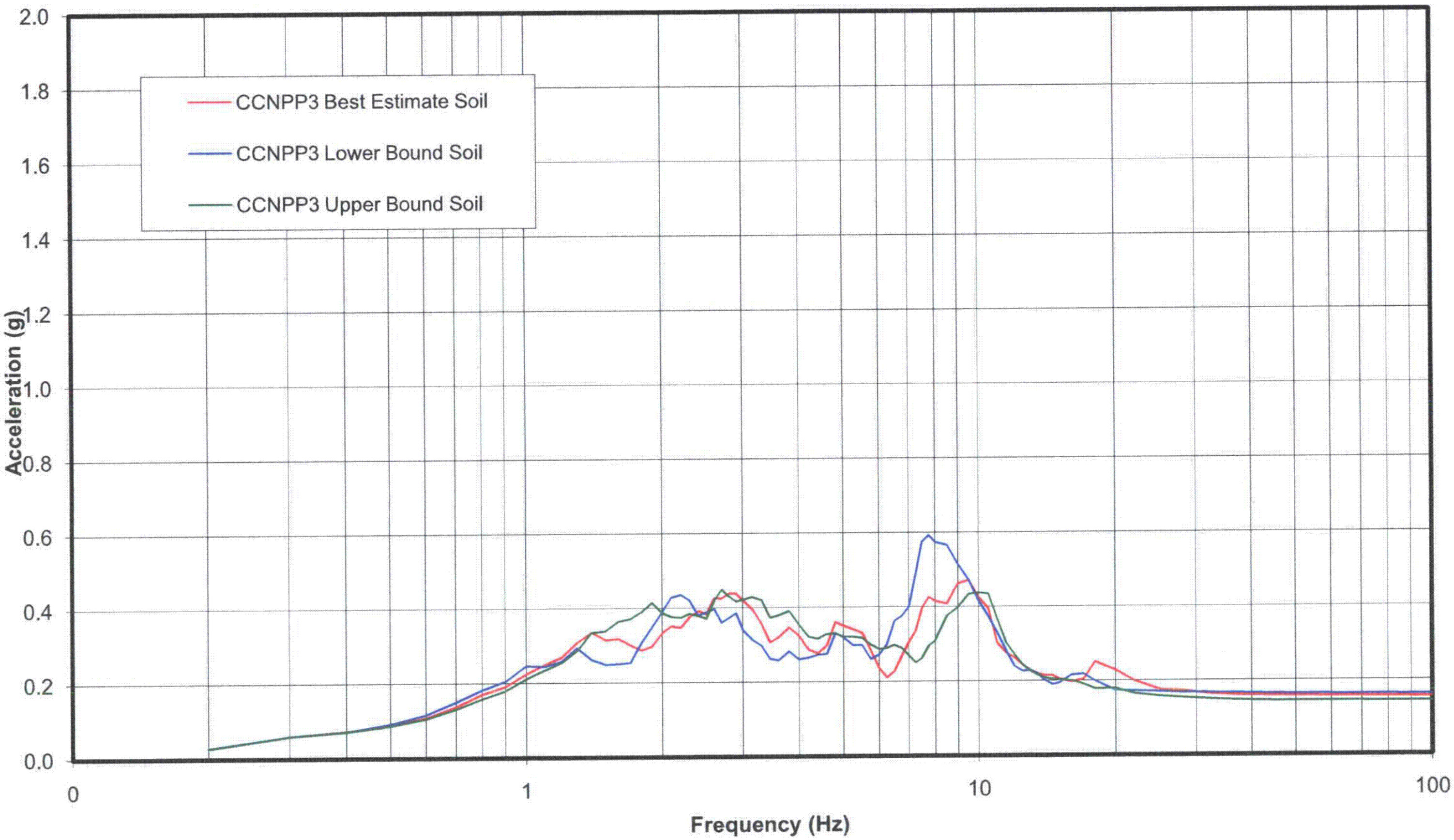


Figure 3.7-53— {CCNPP Unit 3 NAB Basemat Y(N-S) Direction Spectra (5% Damping)}

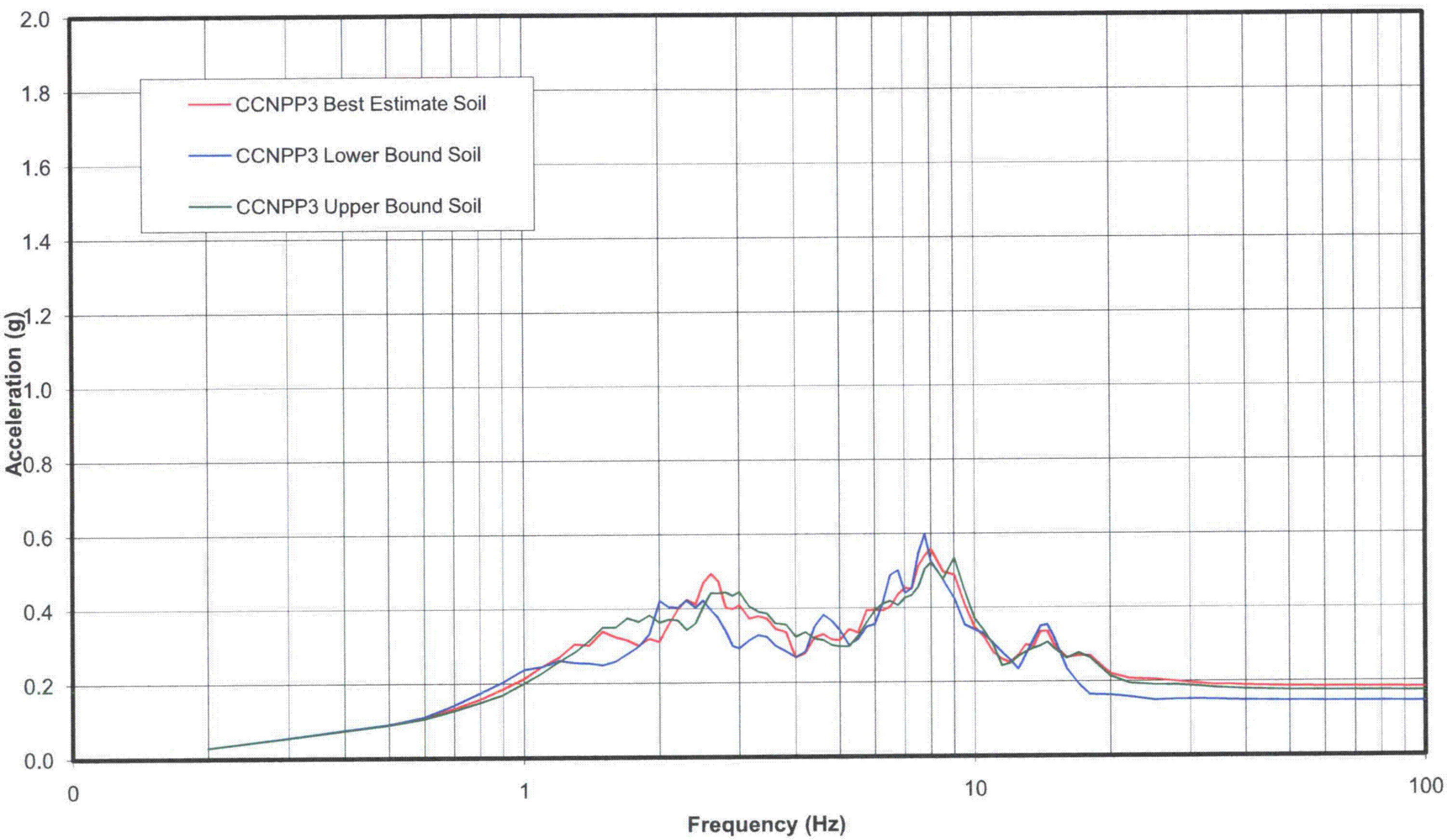


Figure 3.7-54— {CCNPP Unit 3 NAB Basemat Z(Vert) Direction Spectra (5% Damping)}

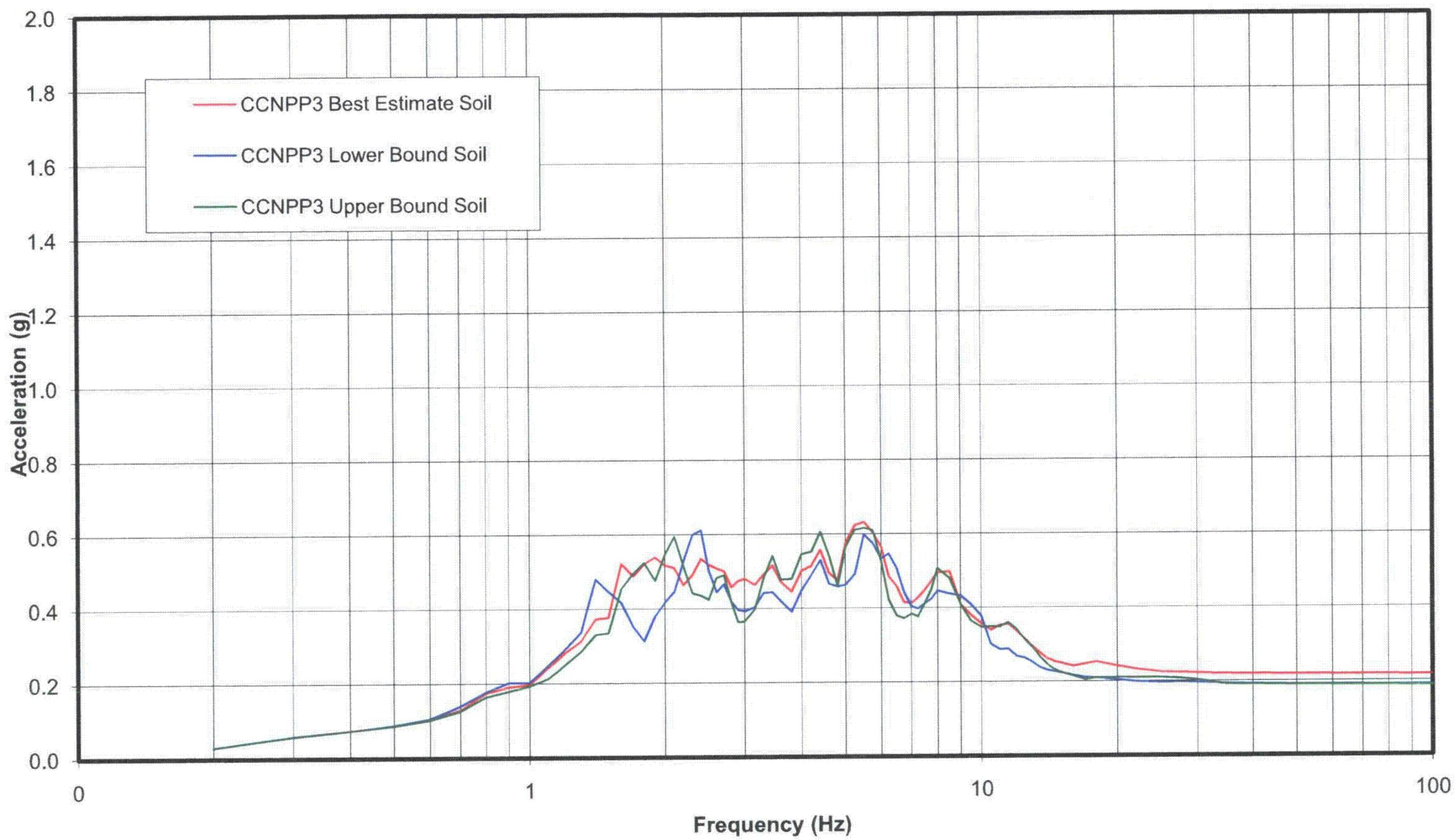


Figure 3.7-55— {Design Certification NAB Basemat X(E-W) Direction Spectra (5% Damping)}

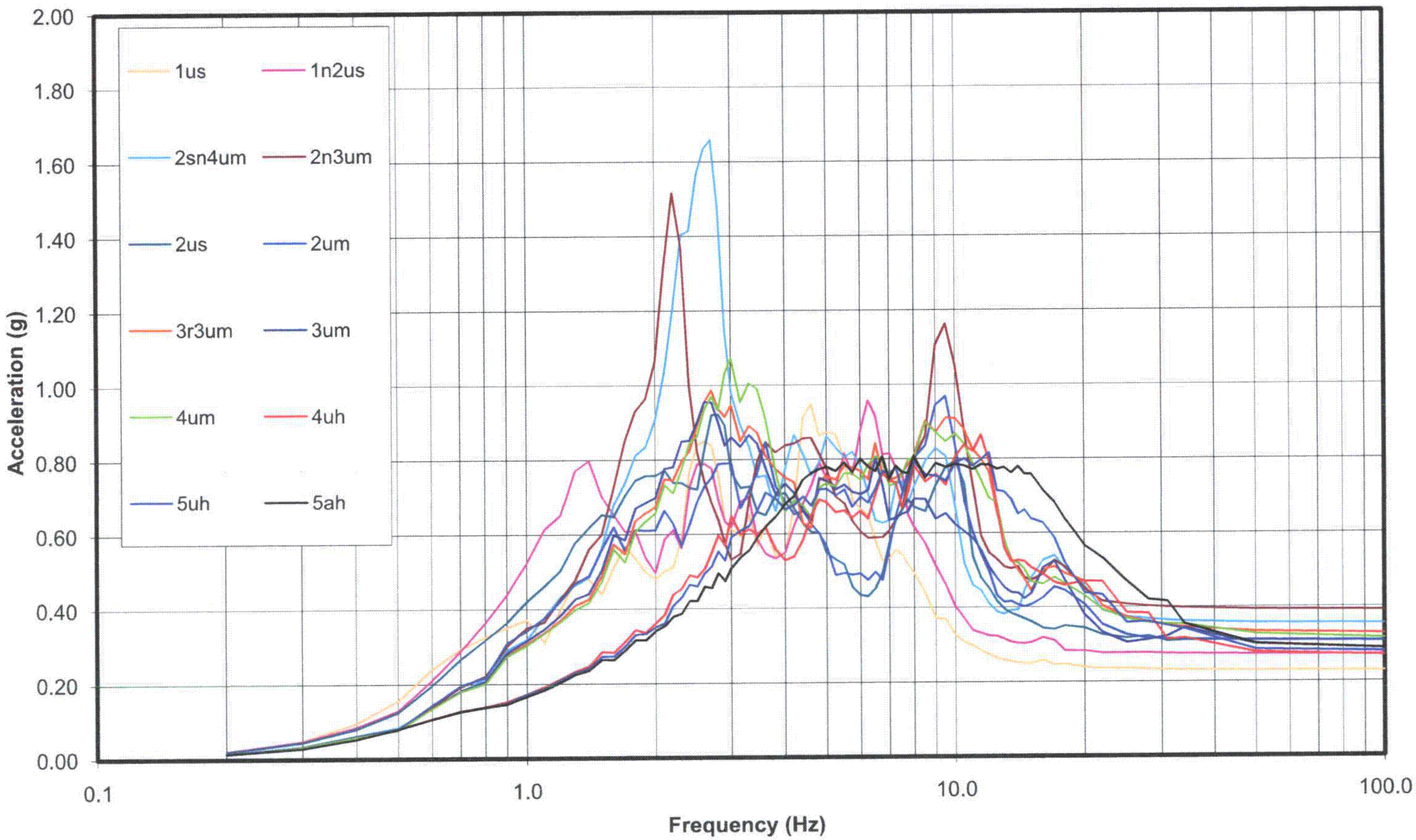


Figure 3.7-56— {Design Certification NAB Basemat Y(N-S) Direction Spectra (5% Damping)}

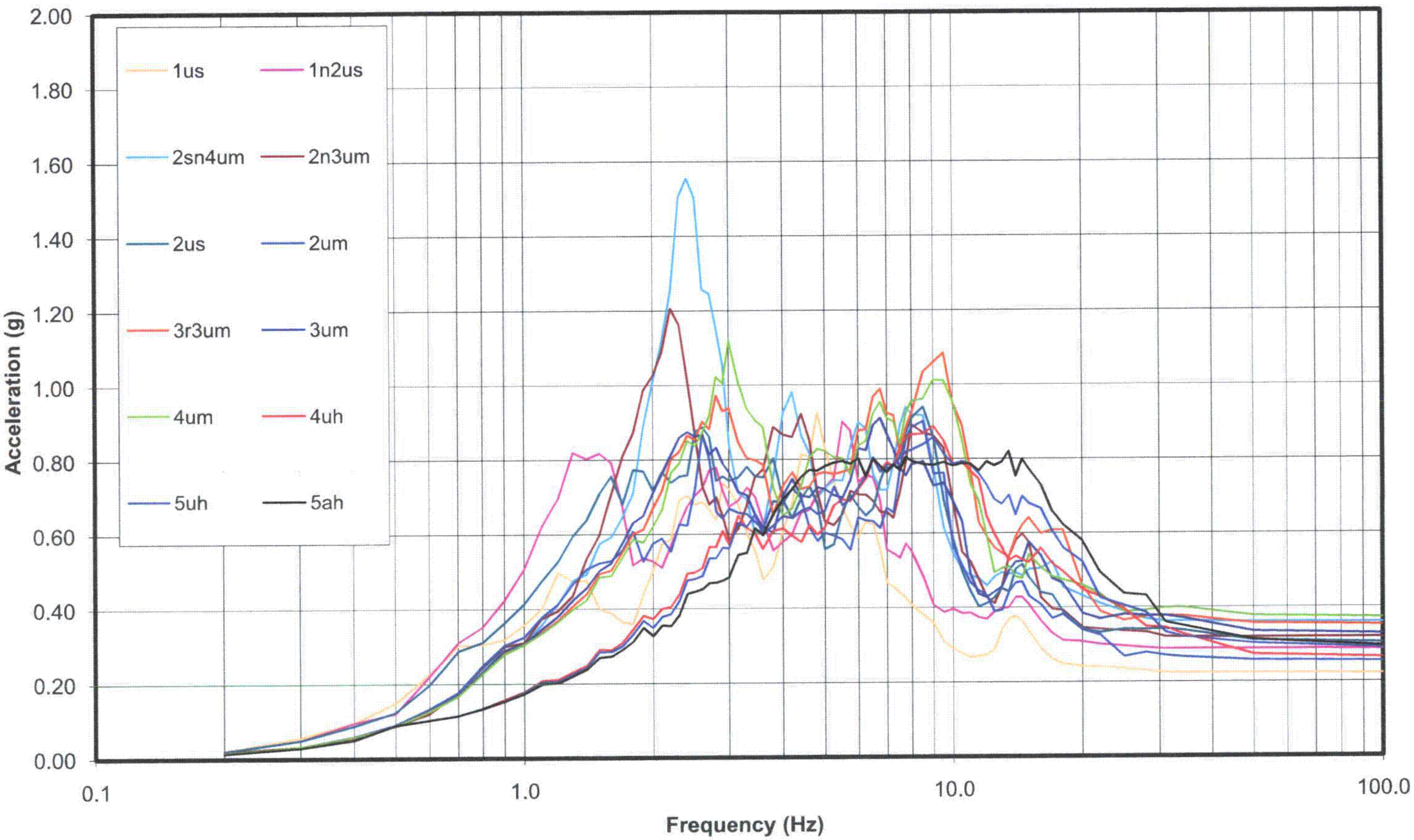


Figure 3.7-57— {Design Certification NAB Basemat Z(Vert) Direction Spectra (5% Damping)}

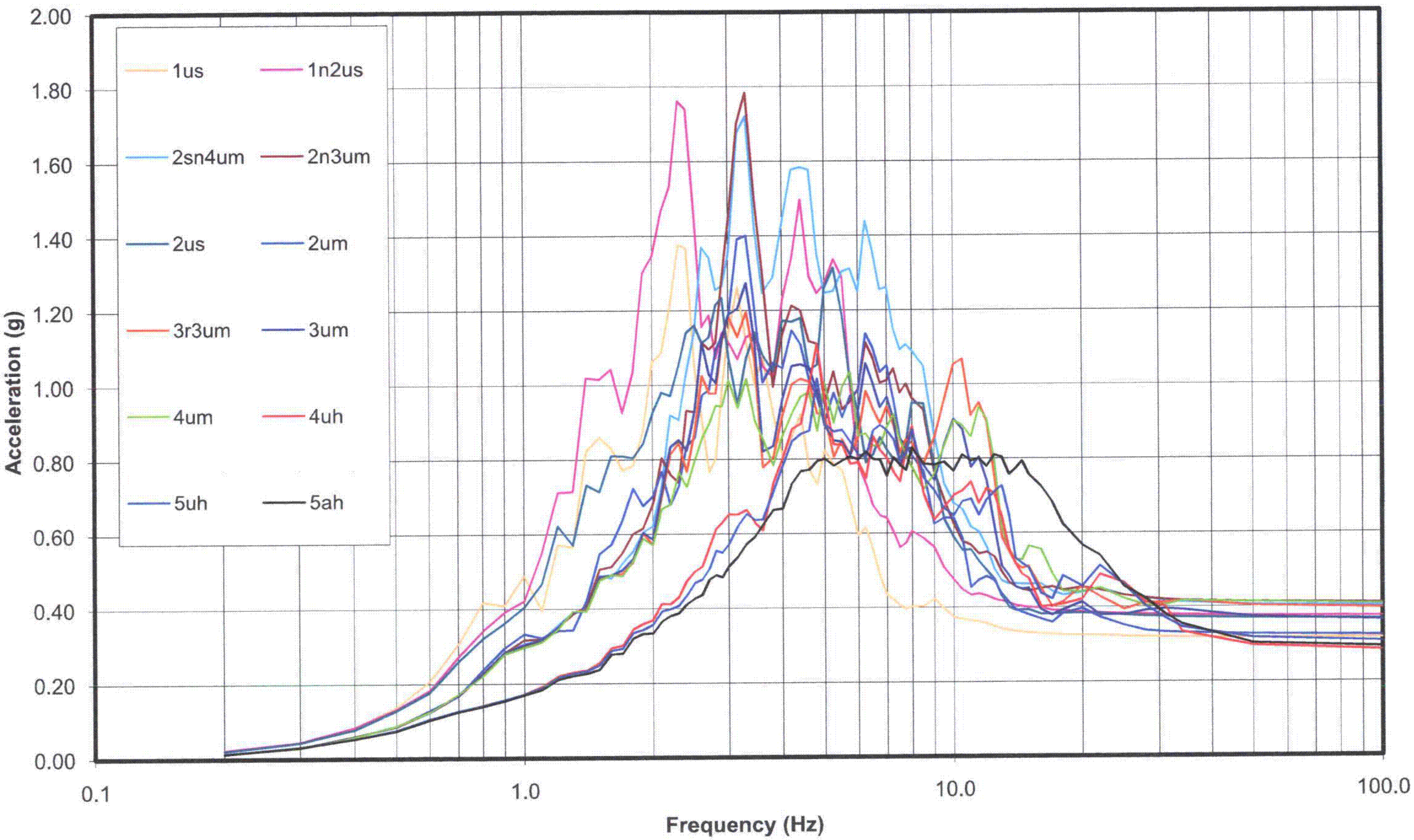


Figure 3.7-58— {CCNPP Unit 3 Radioactive Waste Processing Building Basemat X-Direction Spectra (5% Damping)}

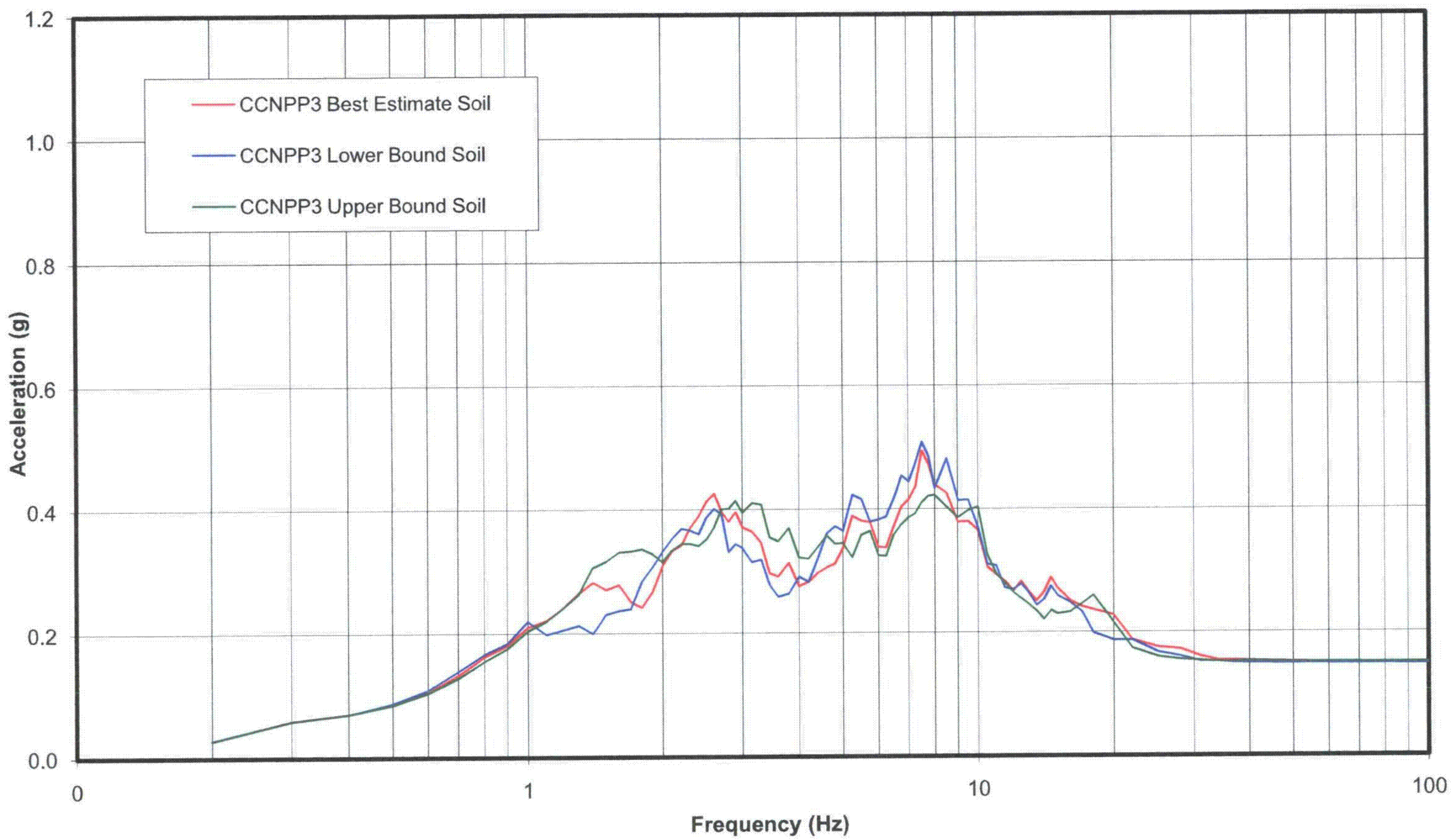


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Figure 3.7-59— {CCNPP Unit 3 Radioactive Waste Processing Building Basemat Y-Direction Spectra (5% Damping)}

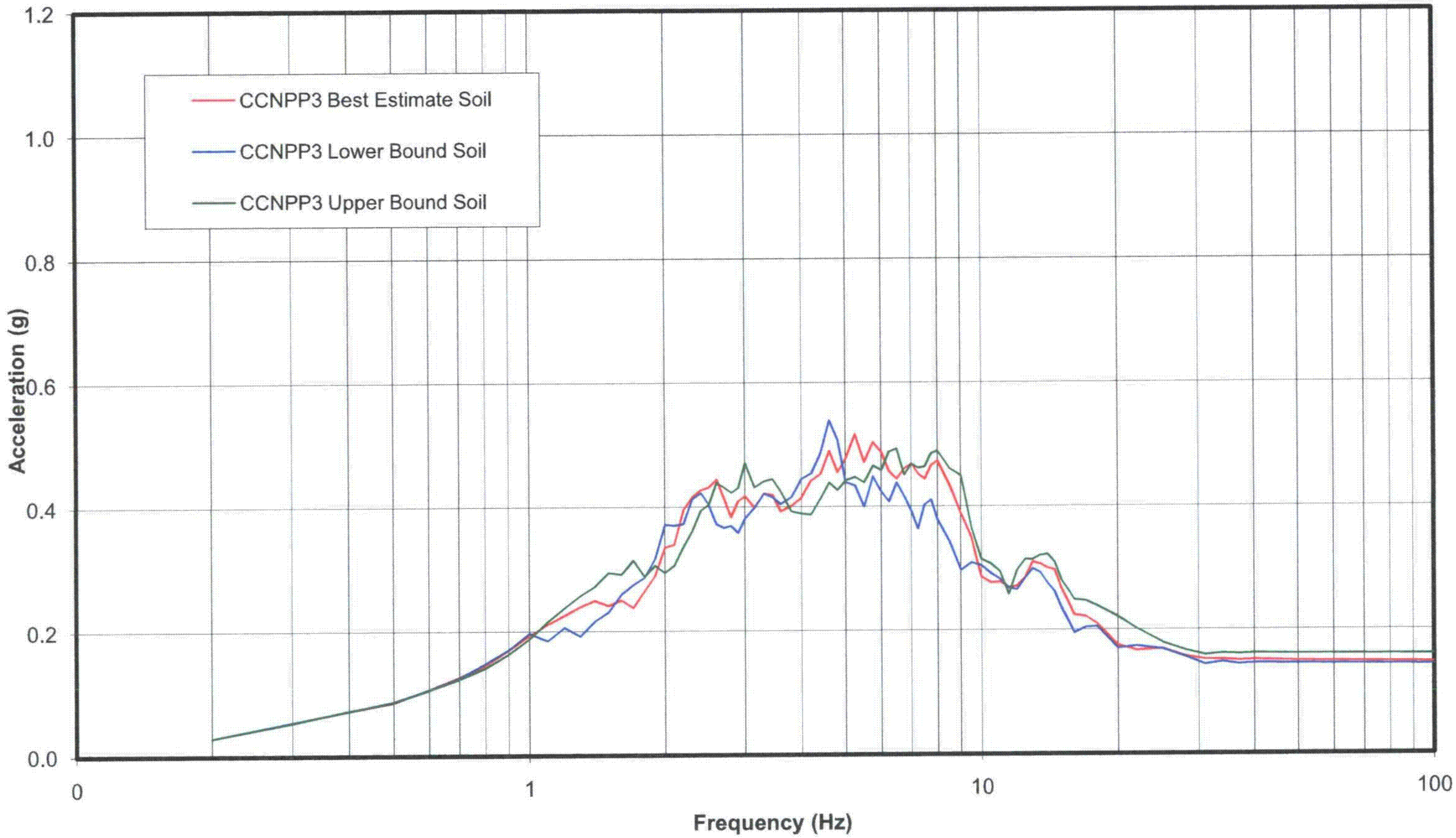


Figure 3.7-60— {CCNPP Unit 3 Radioactive Waste Processing Building Basemat Z-Direction Spectra (5% Damping)}

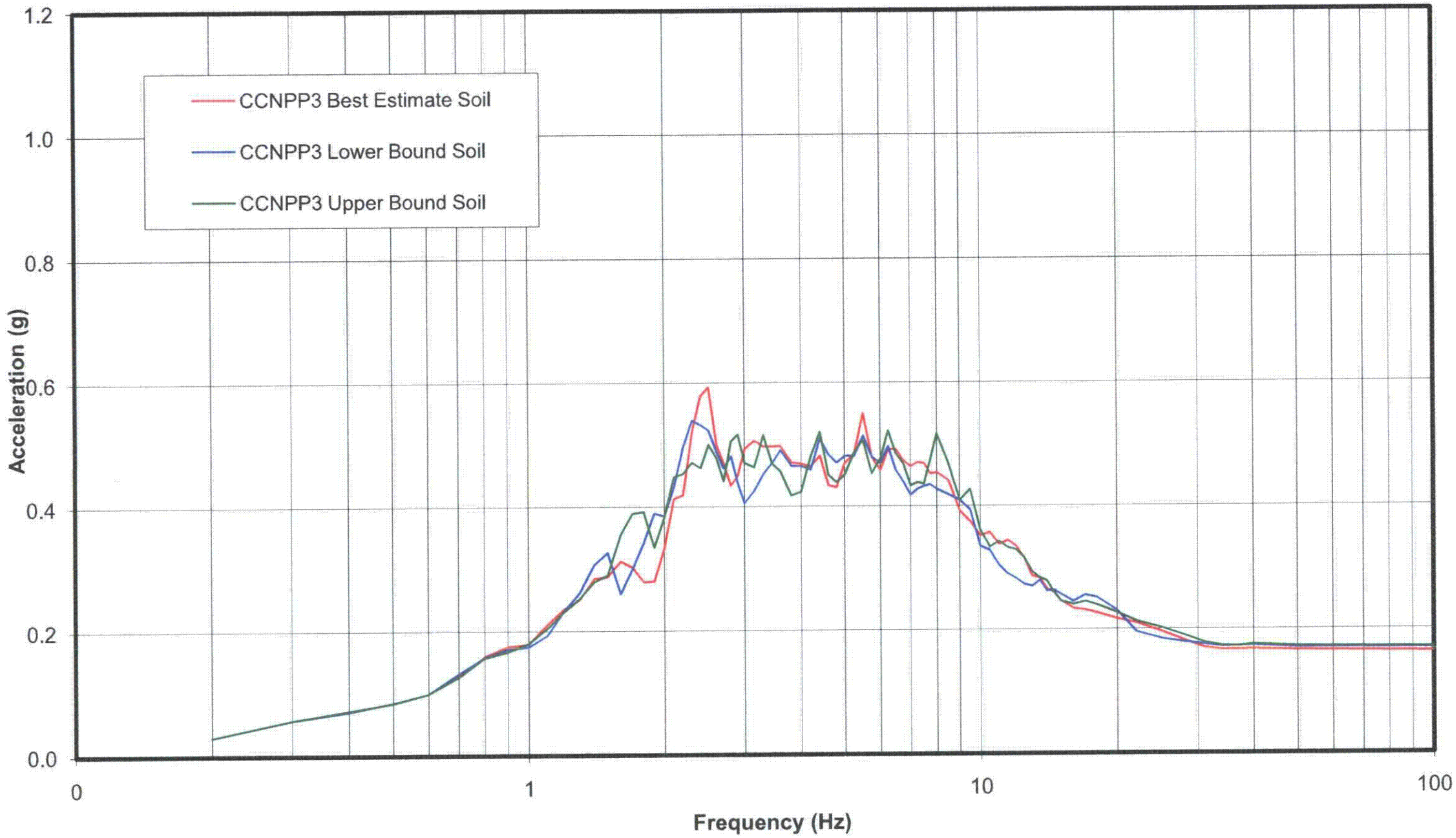


Figure 3.7-61— {Design Certification Radioactive Waste Processing Building Basemat X-Direction Spectra (5% Damping)}

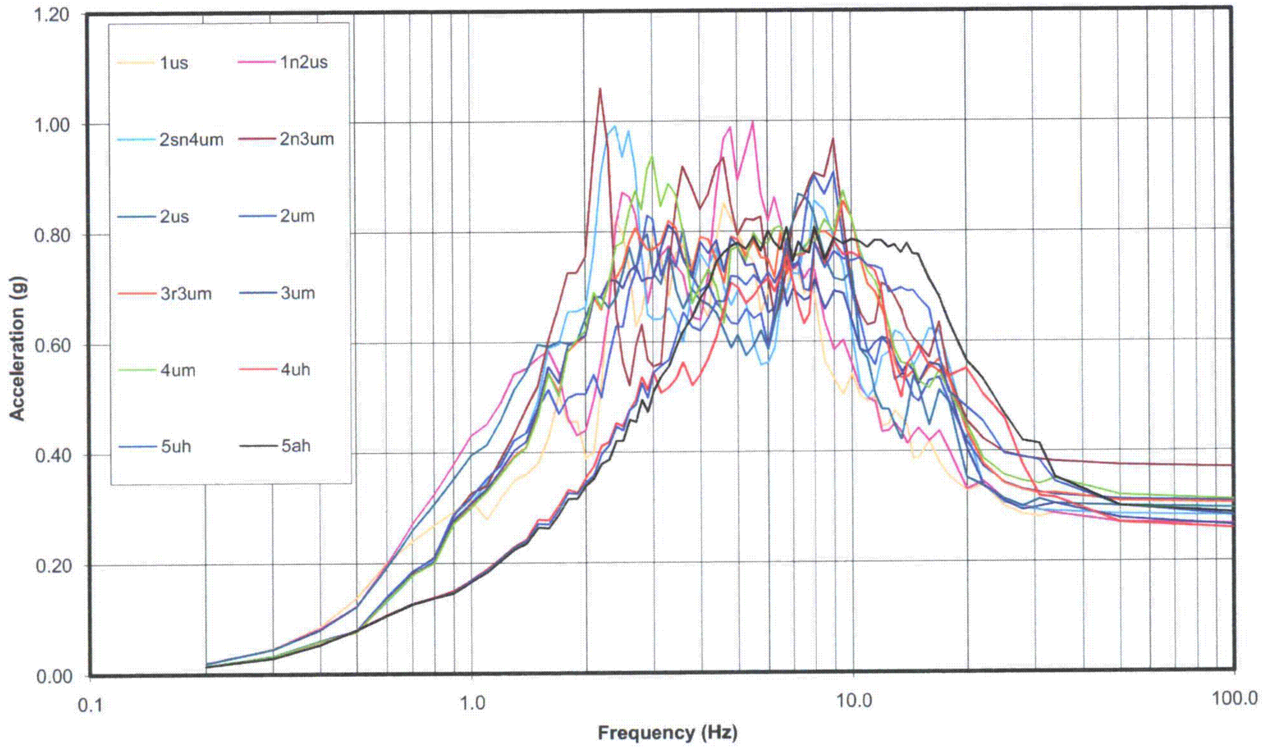


Figure 3.7-62— {Design Certification Radioactive Waste Processing Building Basemat Y-Direction Spectra (5% Damping)}

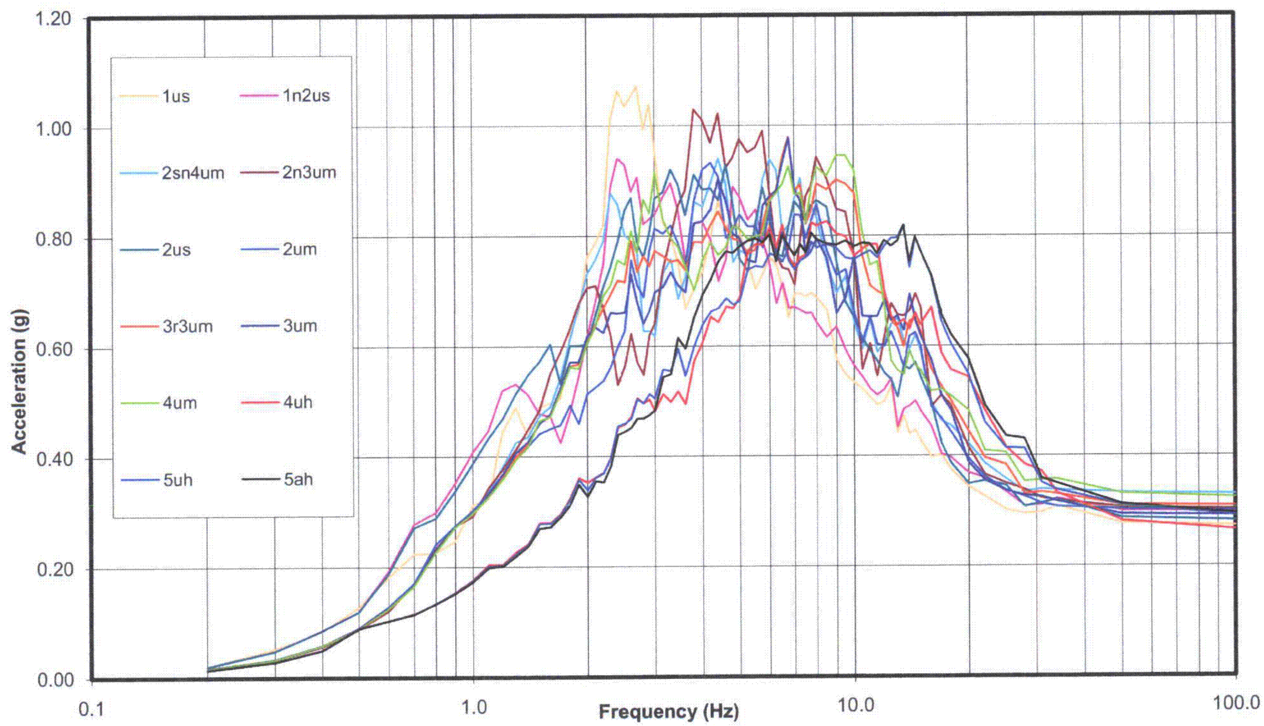


Figure 3.7-63— {Design Certification Radioactive Waste Processing Building Basemat Z-Direction Spectra (5% Damping)}

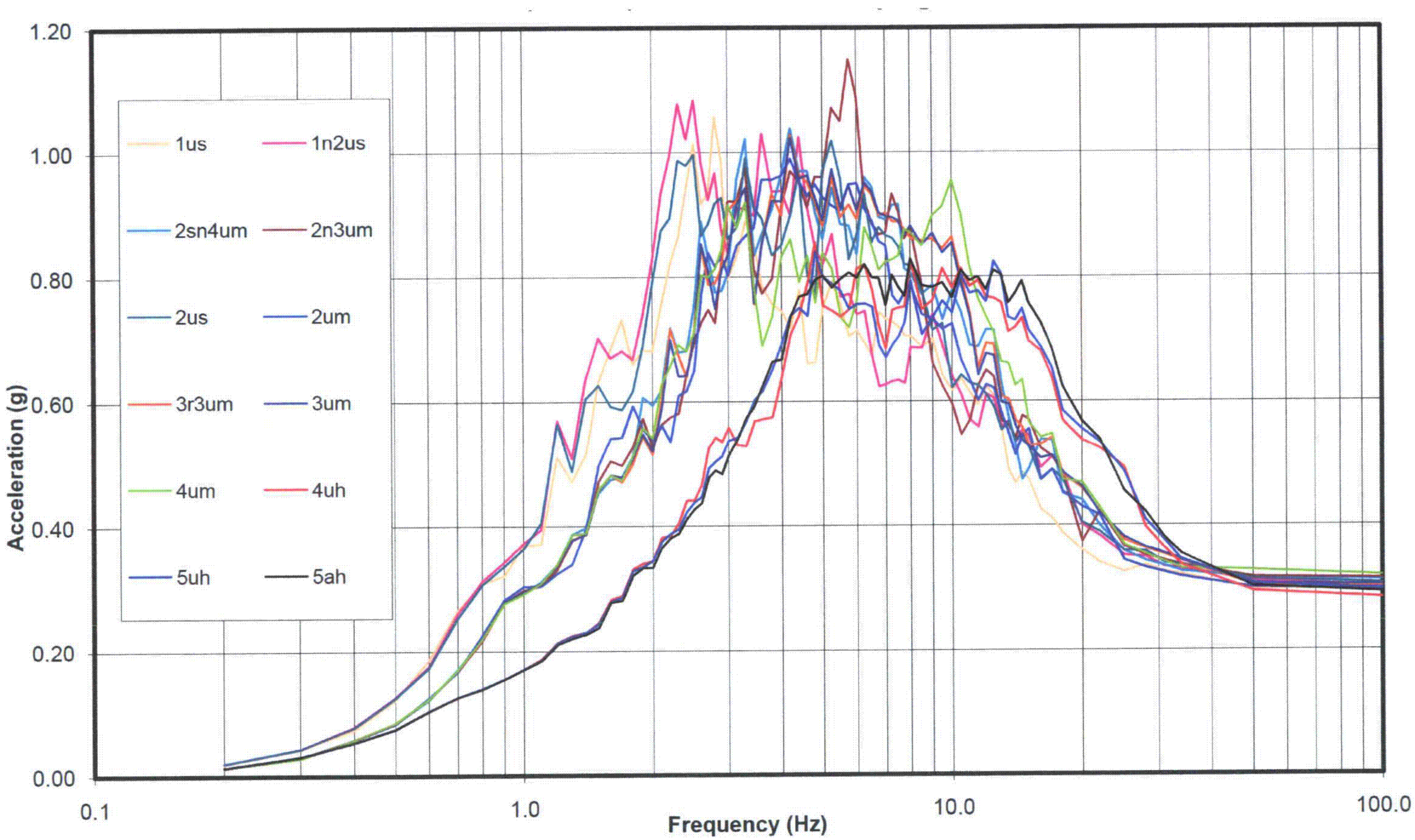


Figure 3.7-64— {Emergency Power Generating Building (EPGB), Elev. 0.0 ft (0.0 m), X (E-W) Direction ISRS, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

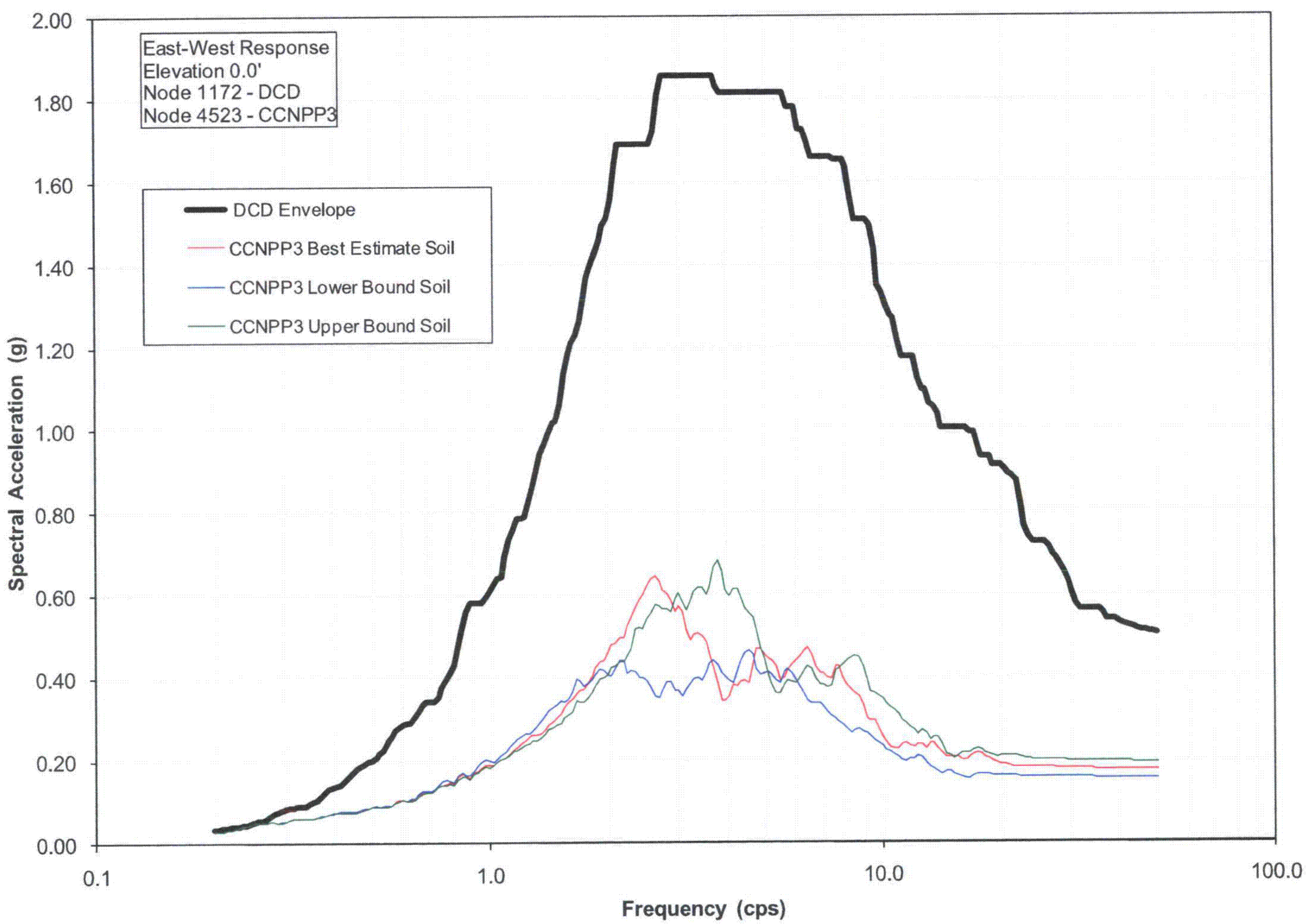


Figure 3.7-65— {Emergency Power Generating Building (EPGB), Elev. 0.0 ft (0.0 m), Y (N-S) Direction ISRS, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

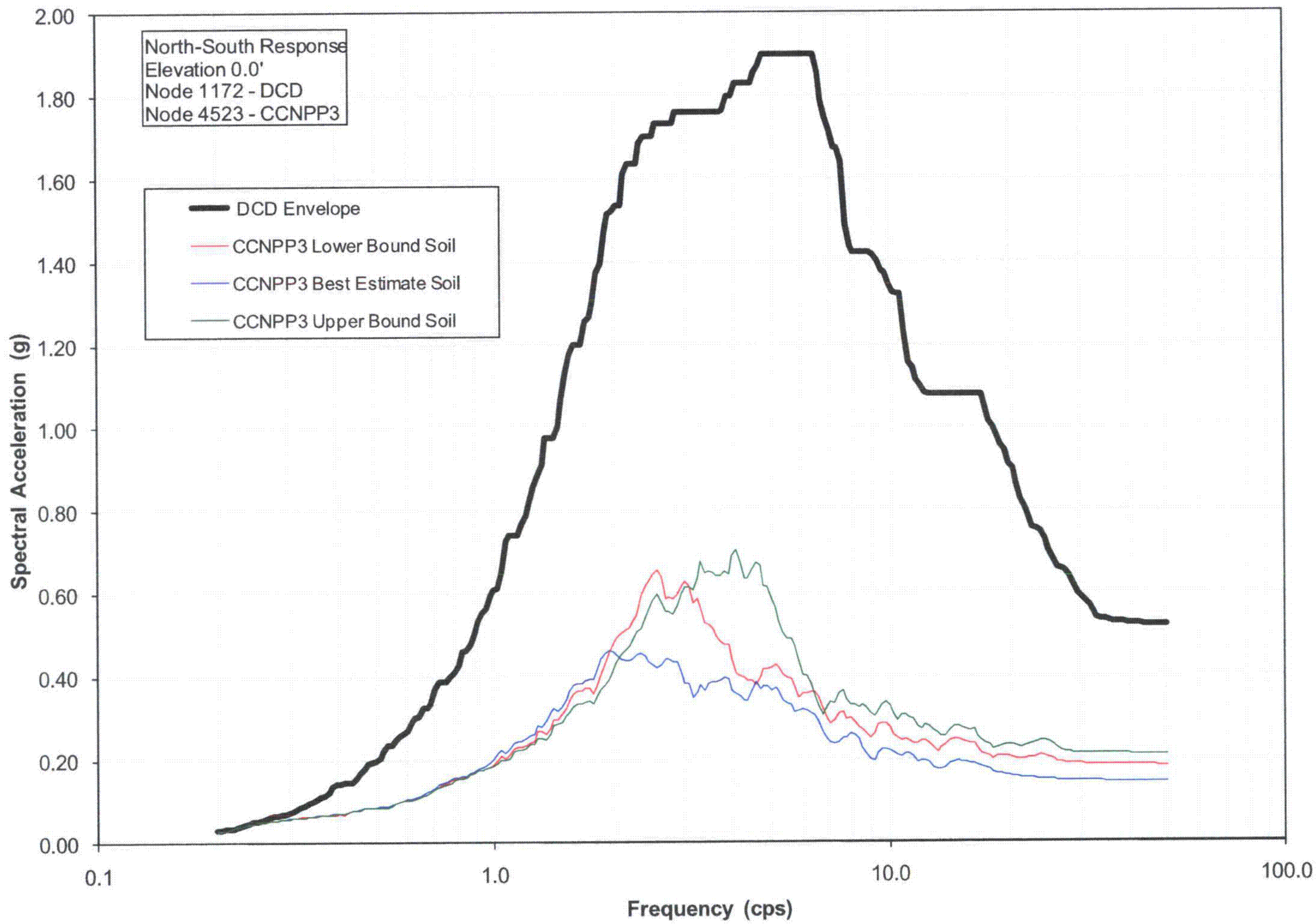


Figure 3.7-66— {Emergency Power Generating Building (EPGB), Elev. 0.0 ft (0.0 m), Z (Vert) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

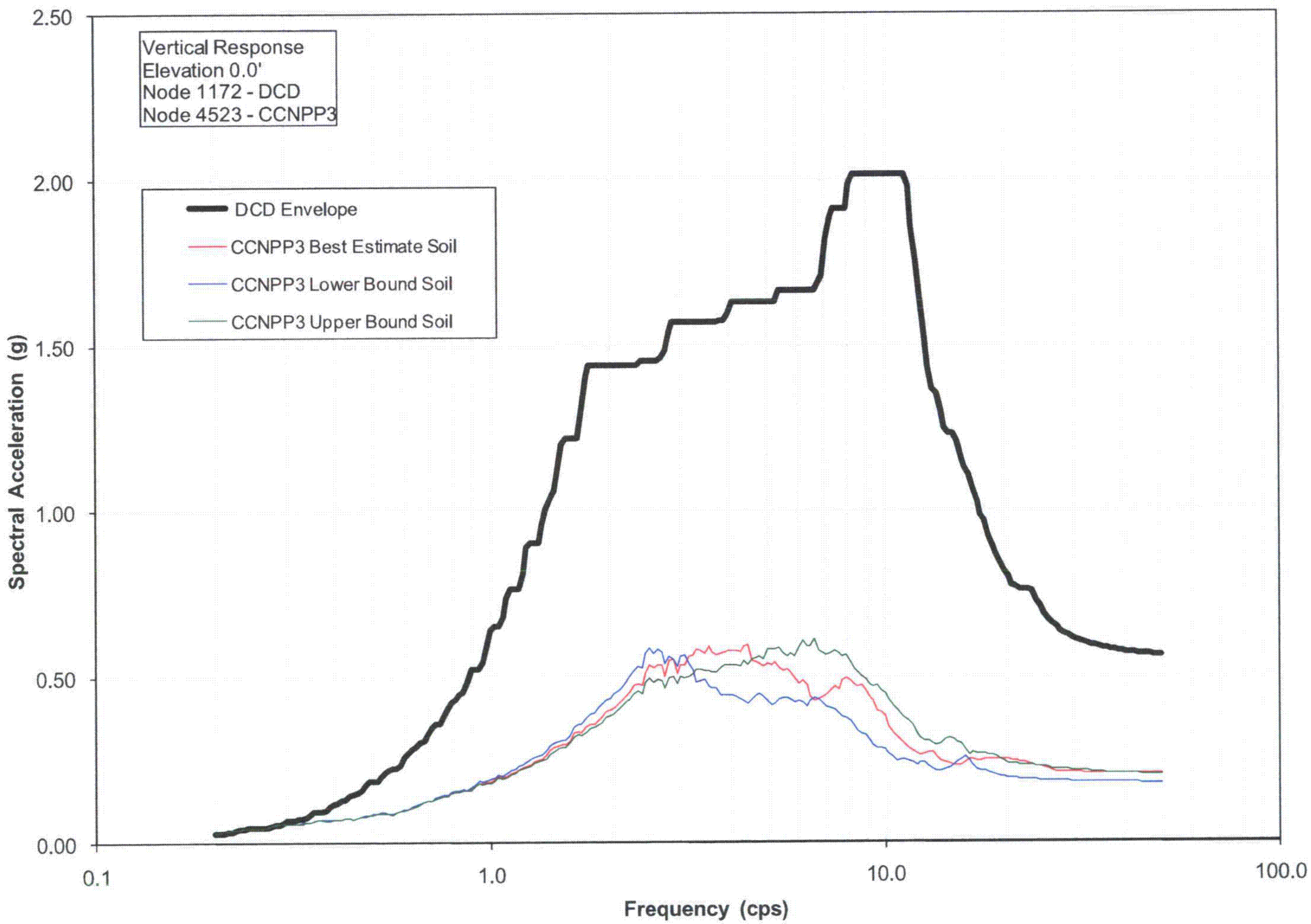


Figure 3.7-67— {Essential Service Water Building (ESWB), Elev. 63.0 ft (19.2 m), X (N-S) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

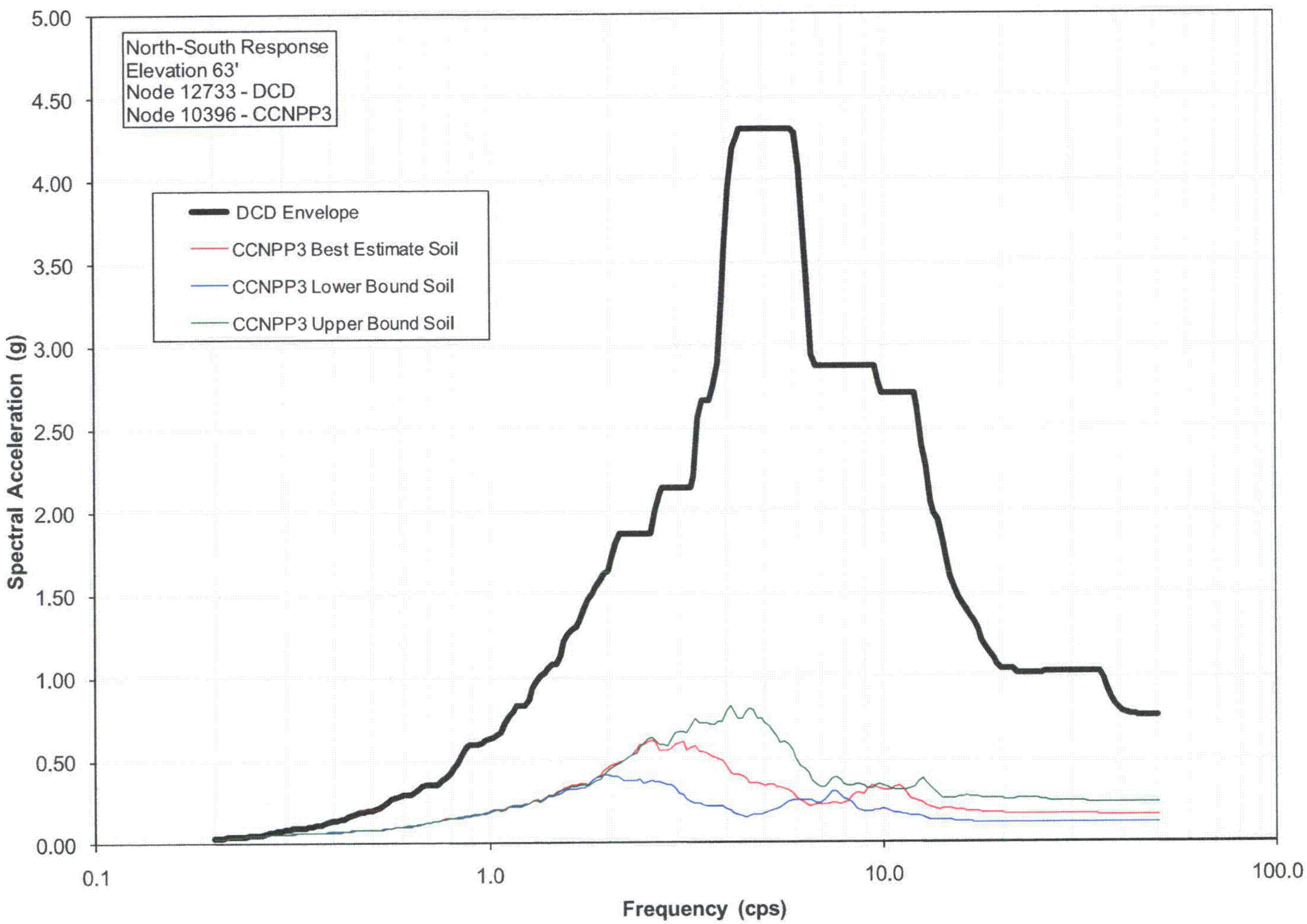


Figure 3.7-68— {Essential Service Water Building (ESWB), Elev. 63.0 ft (19.2 m), Y (E-W) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

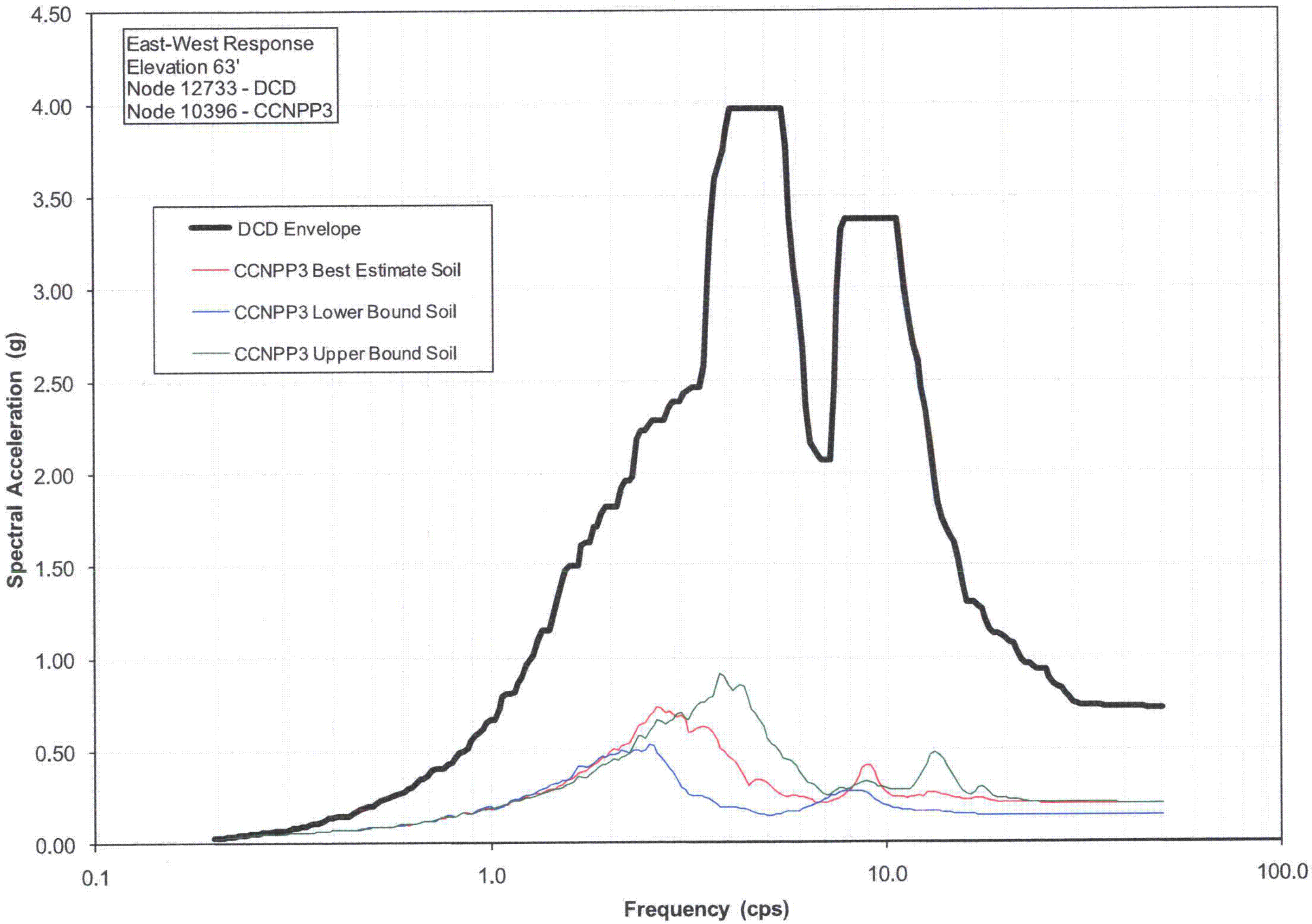


Figure 3.7-69— {Essential Service Water Building (ESWB), Elev. 63.0 ft (19.2 m), Z (Vert) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

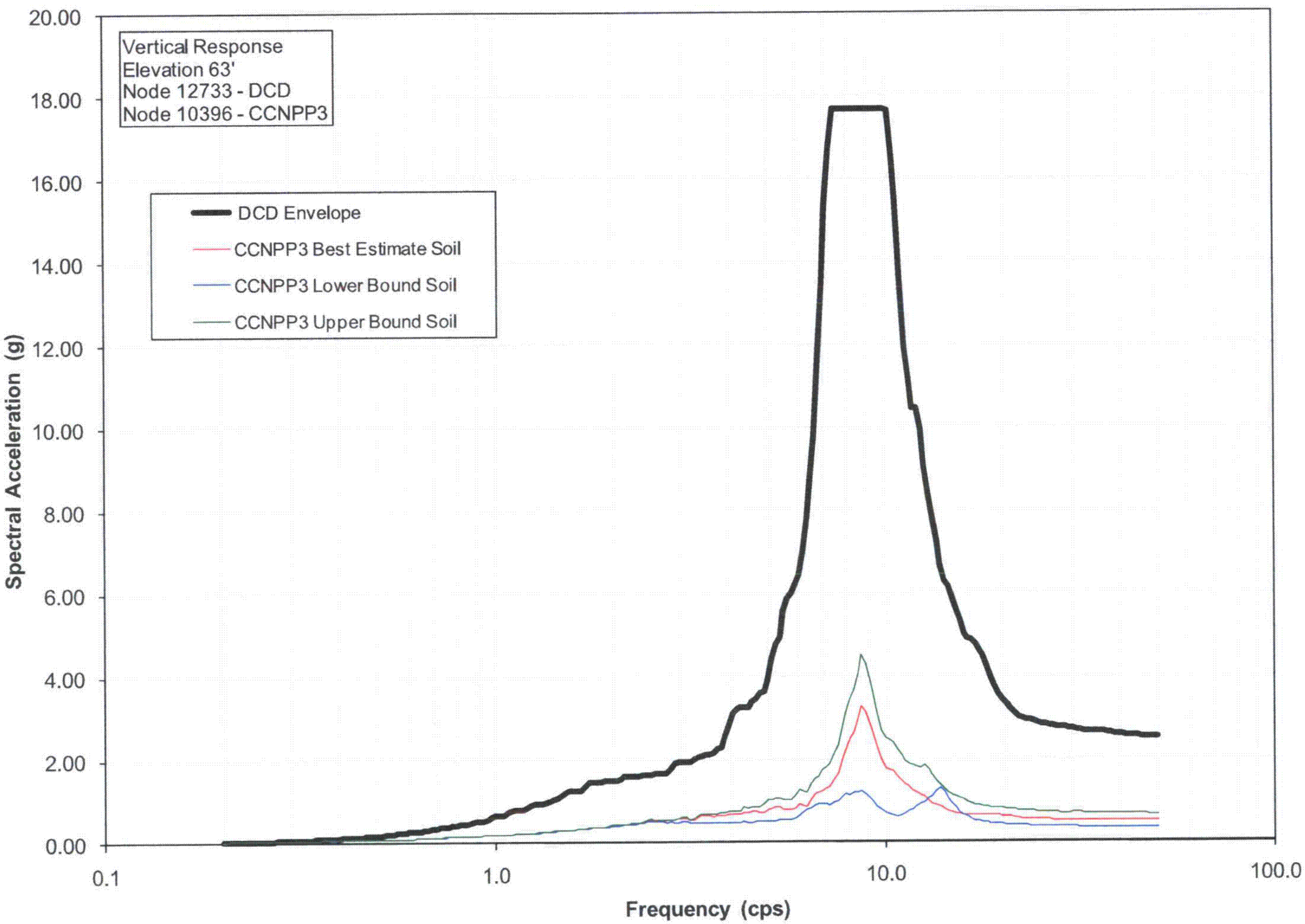


Figure 3.7-70— {Essential Service Water Building (ESWB), Elev. 14.0 ft (4.3 m), X (N-S) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

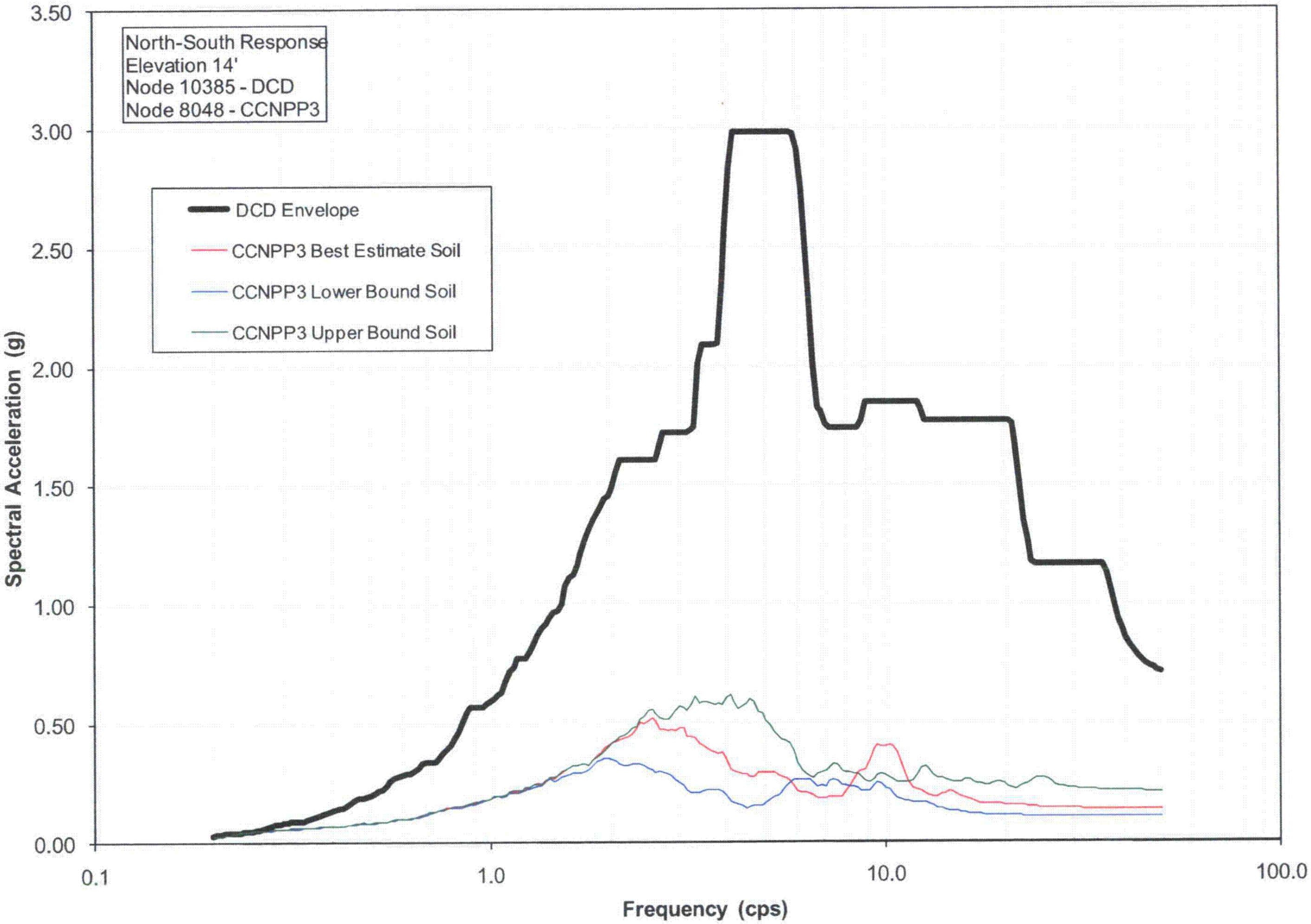


Figure 3.7-71— {Essential Service Water Building (ESWB), Elev. 14.0 ft (4.3 m), Y (E-W) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

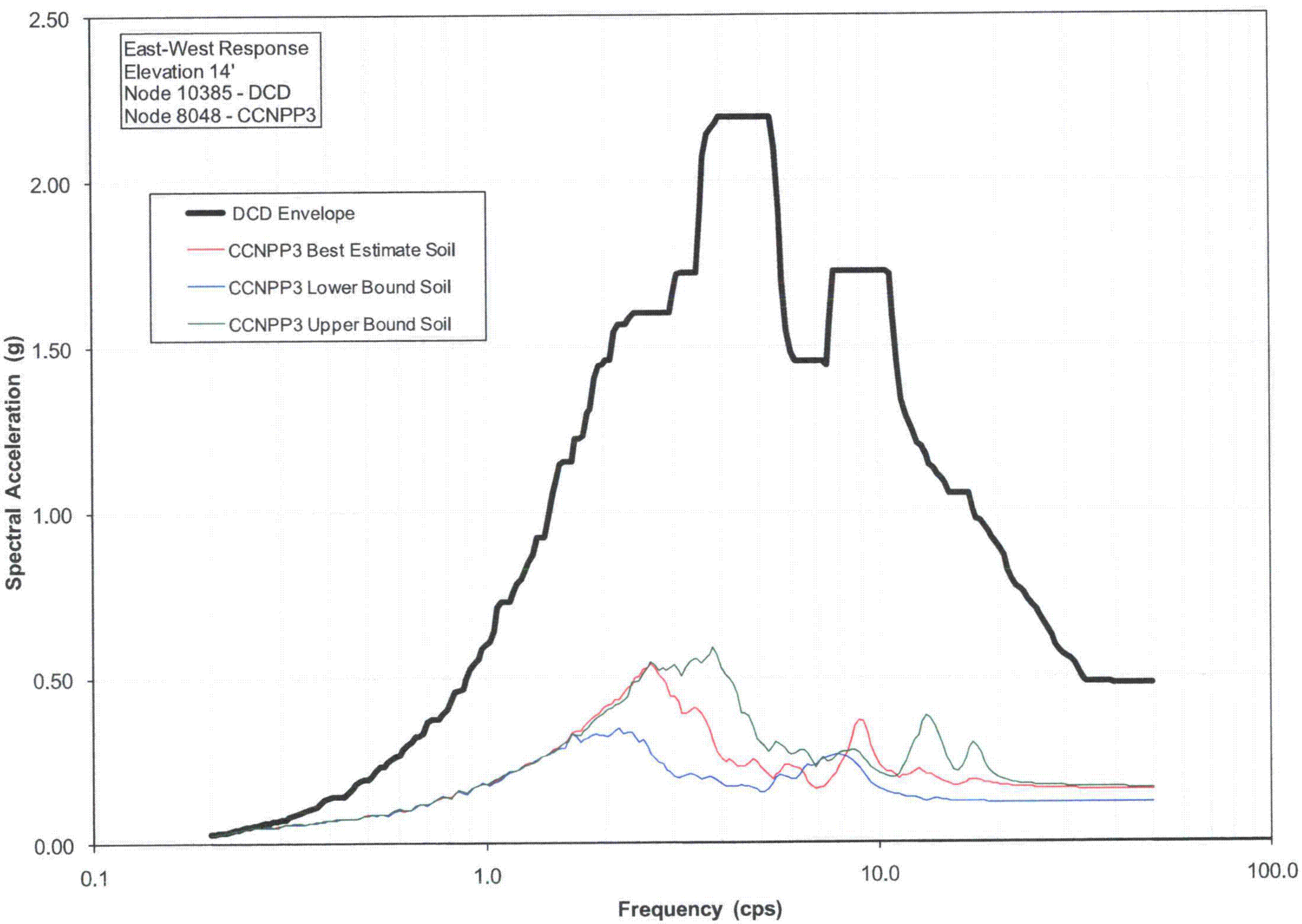


Figure 3.7-72— {Essential Service Water Building (ESWB), Elev. 14.0 ft (4.3 m), Z (Vert) Direction Spectra, 5% Damping. Elevations and plant coordinate system refer to U.S EPR FSAR.}

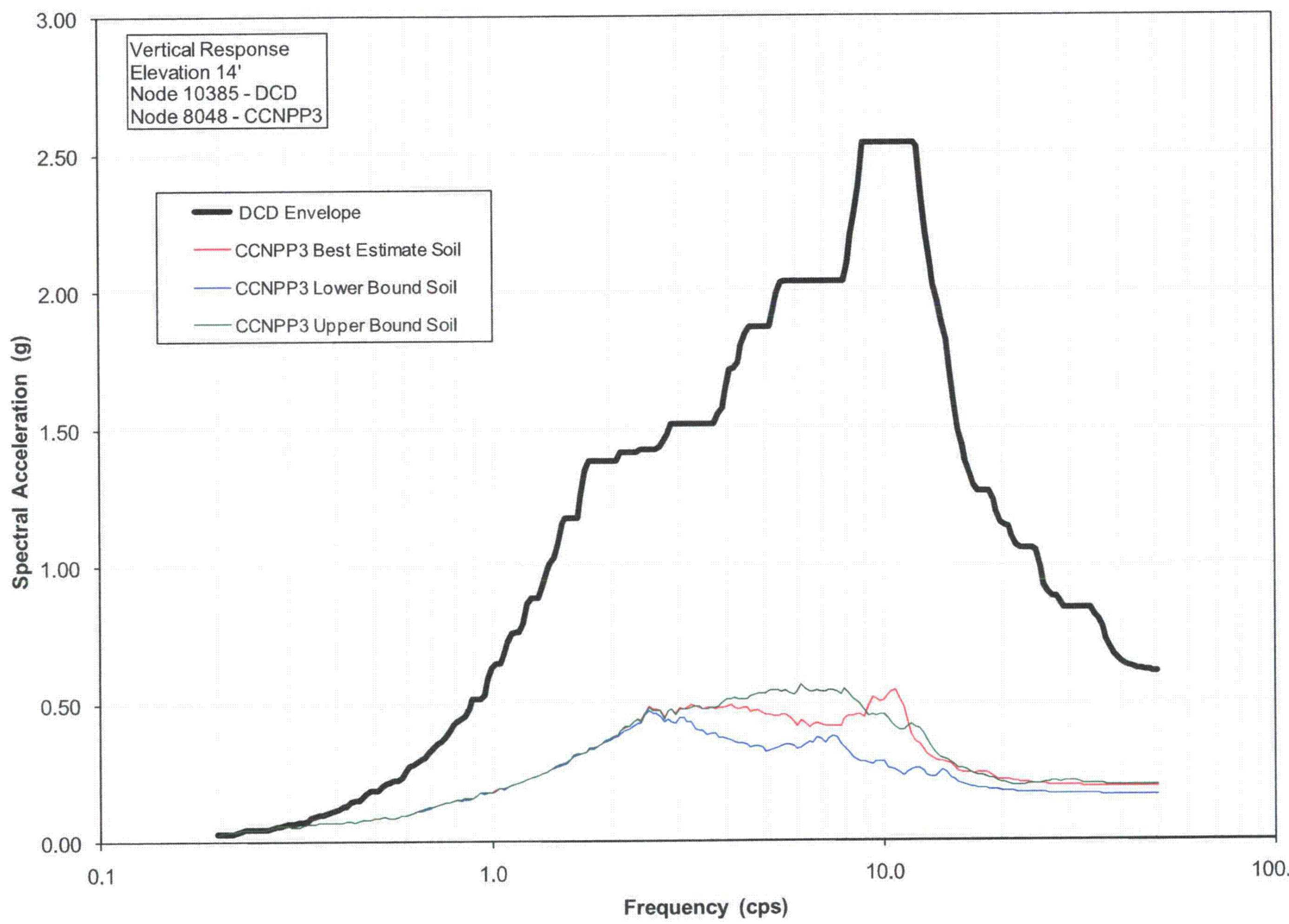


Figure 3.7-73— {ISRS for UHS Makeup Water Intake Structure at location at Elev. -22.5 ft (-6.86 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}

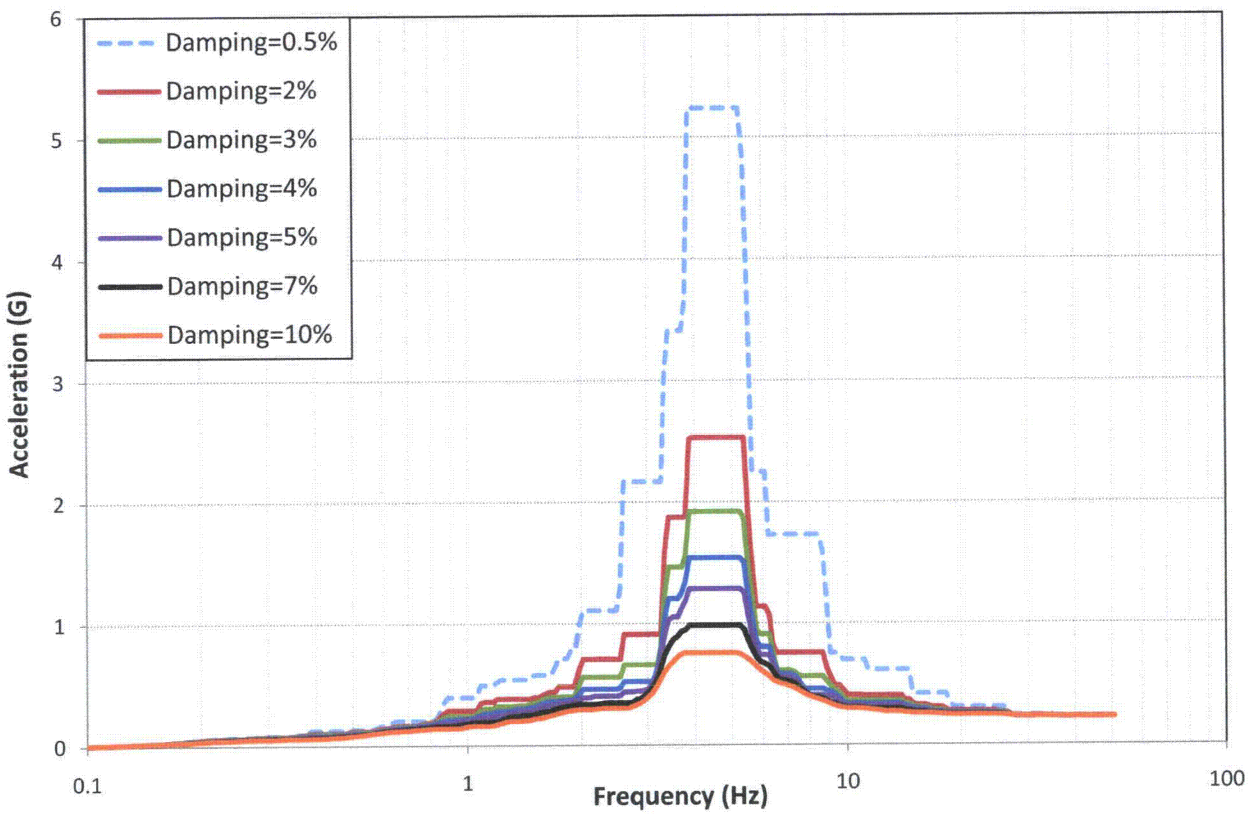
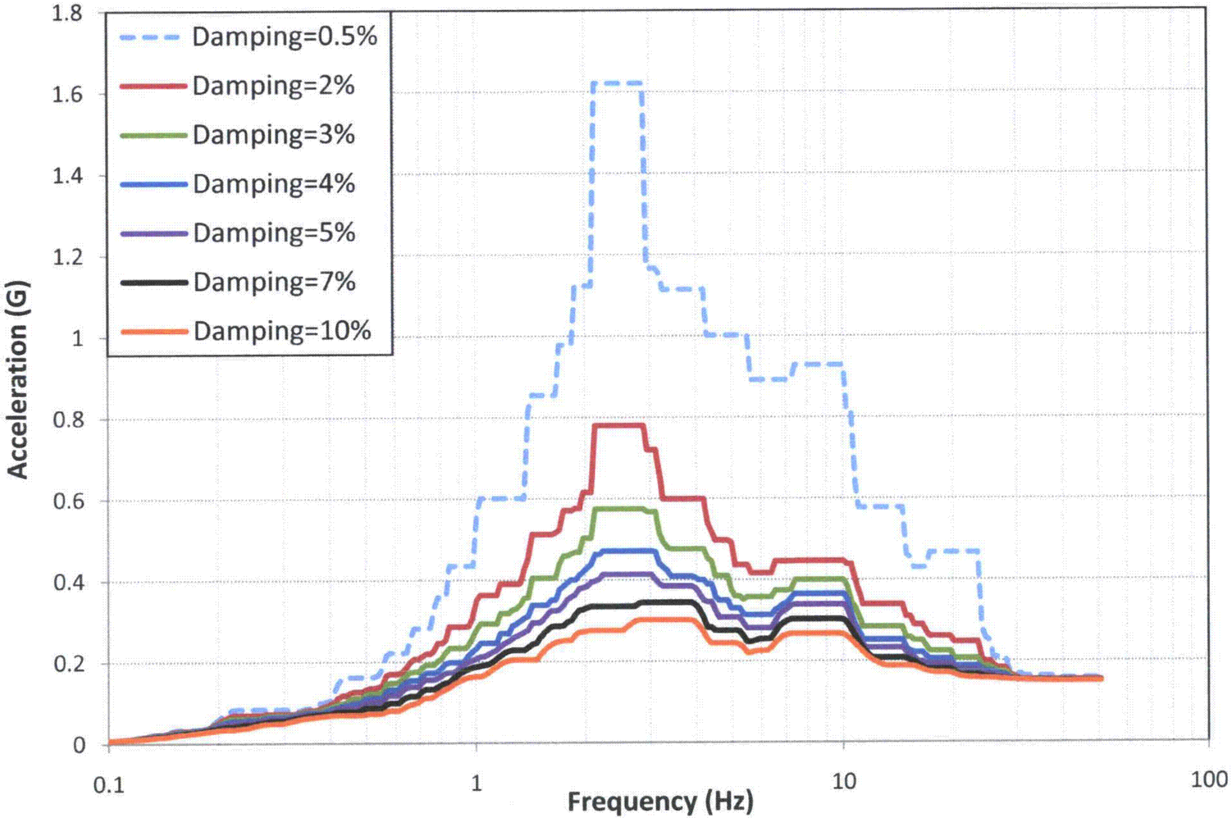


Figure 3.7-74— {ISRS for UHS Makeup Water Intake Structure at Elev. -22.5 ft (-6.86 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3.}



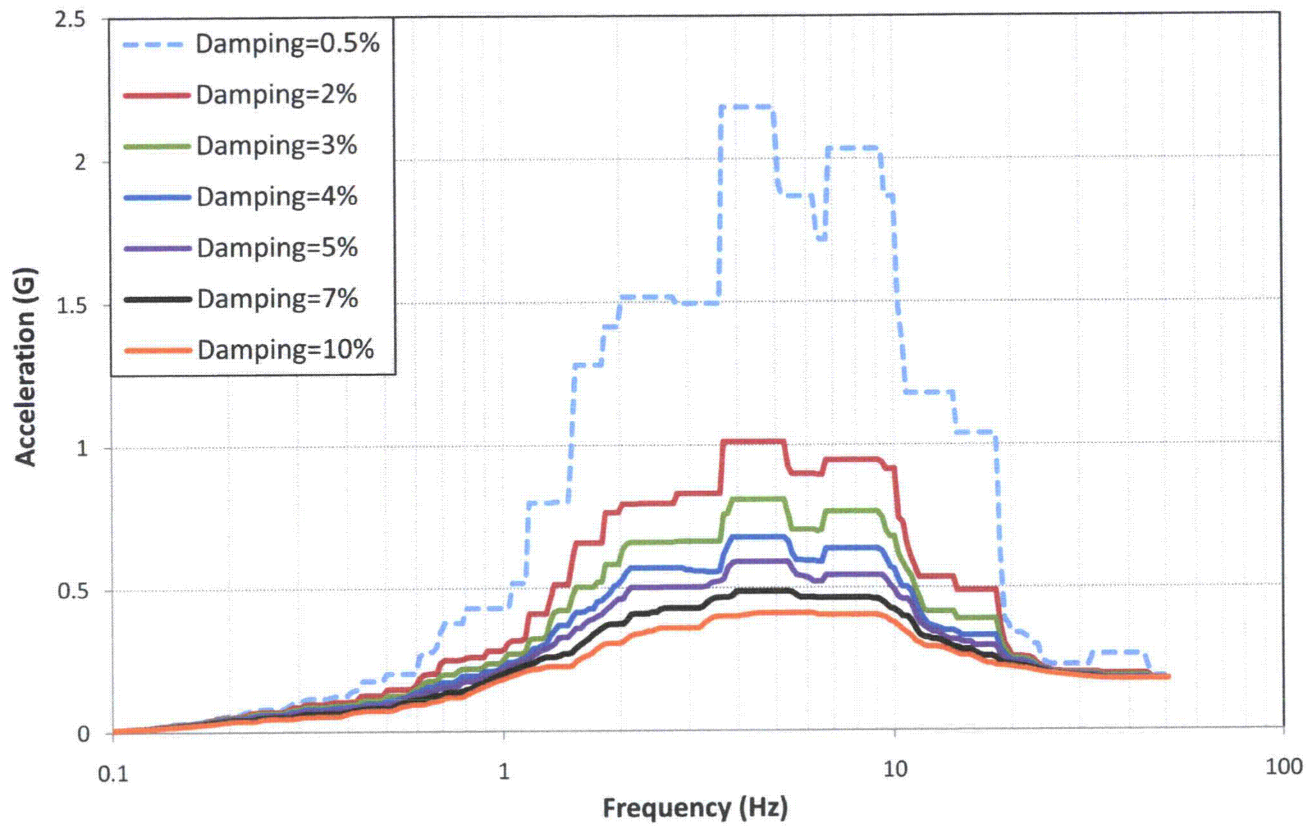


Figure 3.7-75— {ISRS for UHS Makeup Water Intake Structure at Elev. -22.5 ft (-6.86 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}

Figure 3.7-76— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3.}

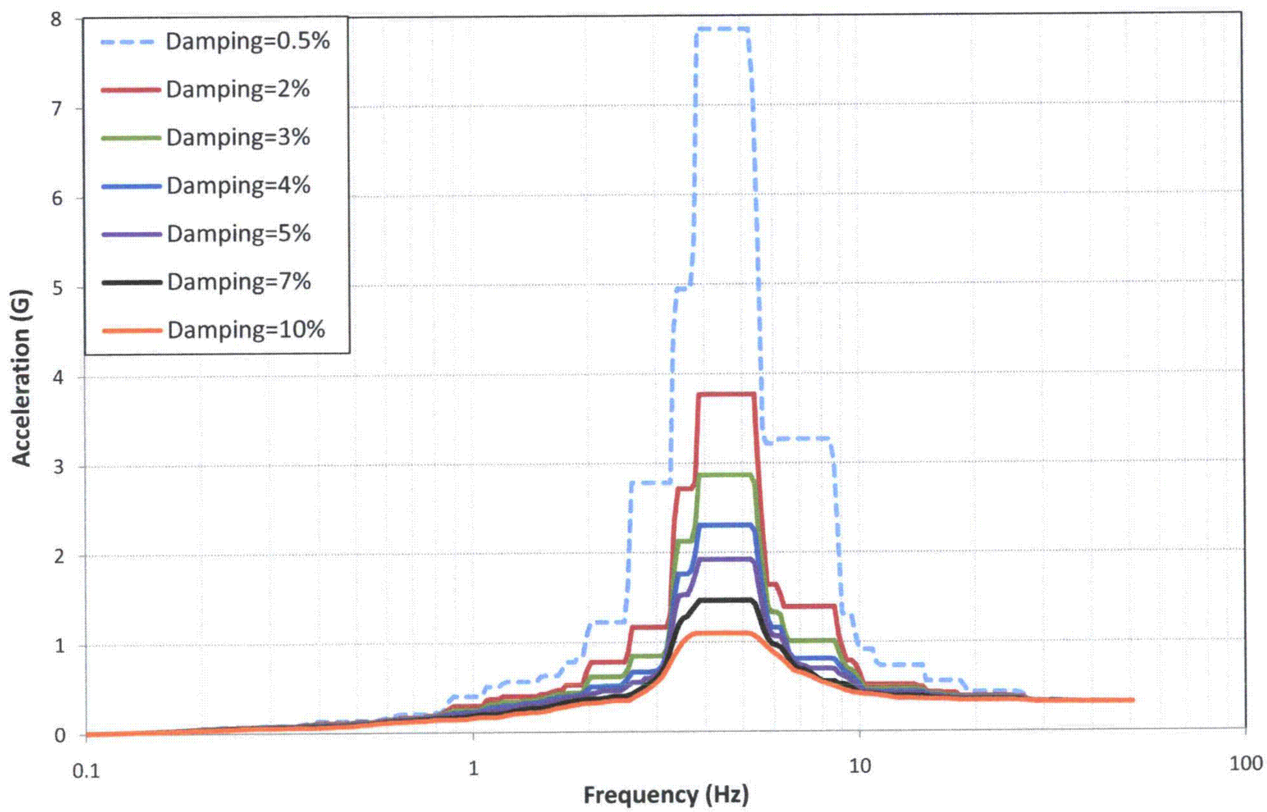


Figure 3.7-77— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}

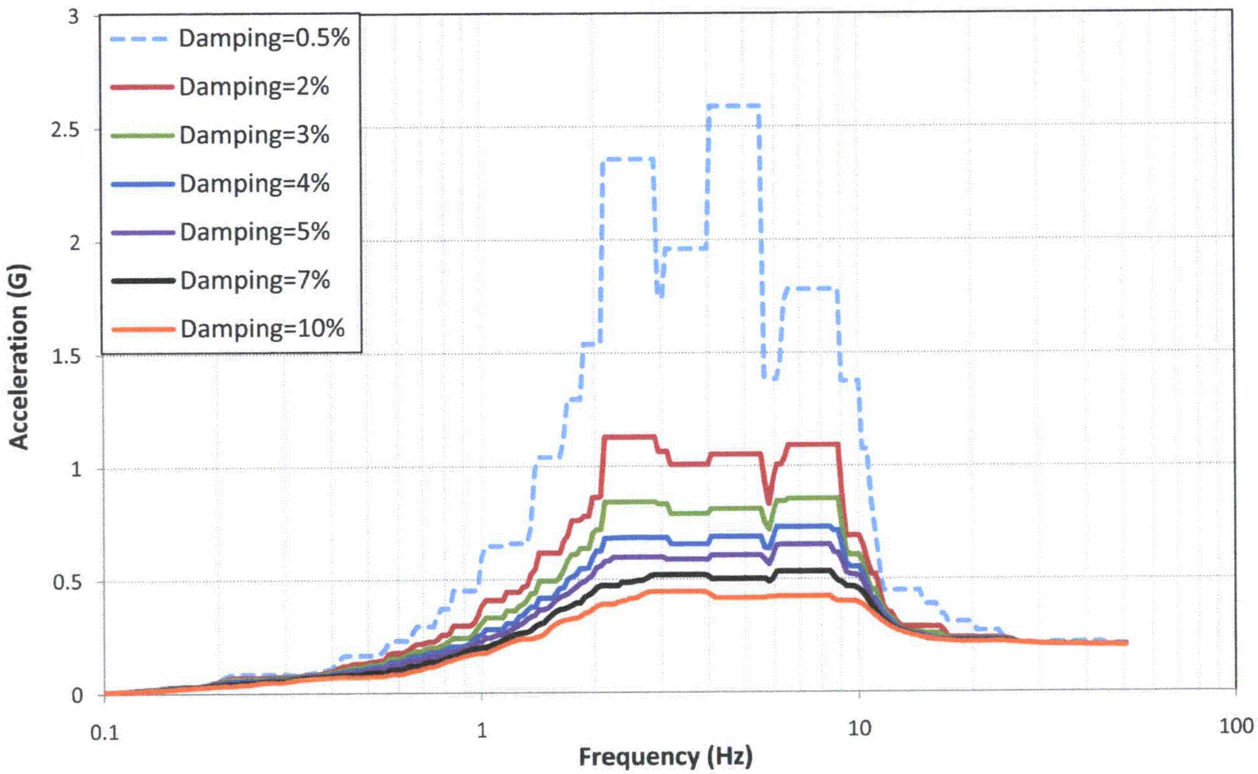


Figure 3.7-78— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}

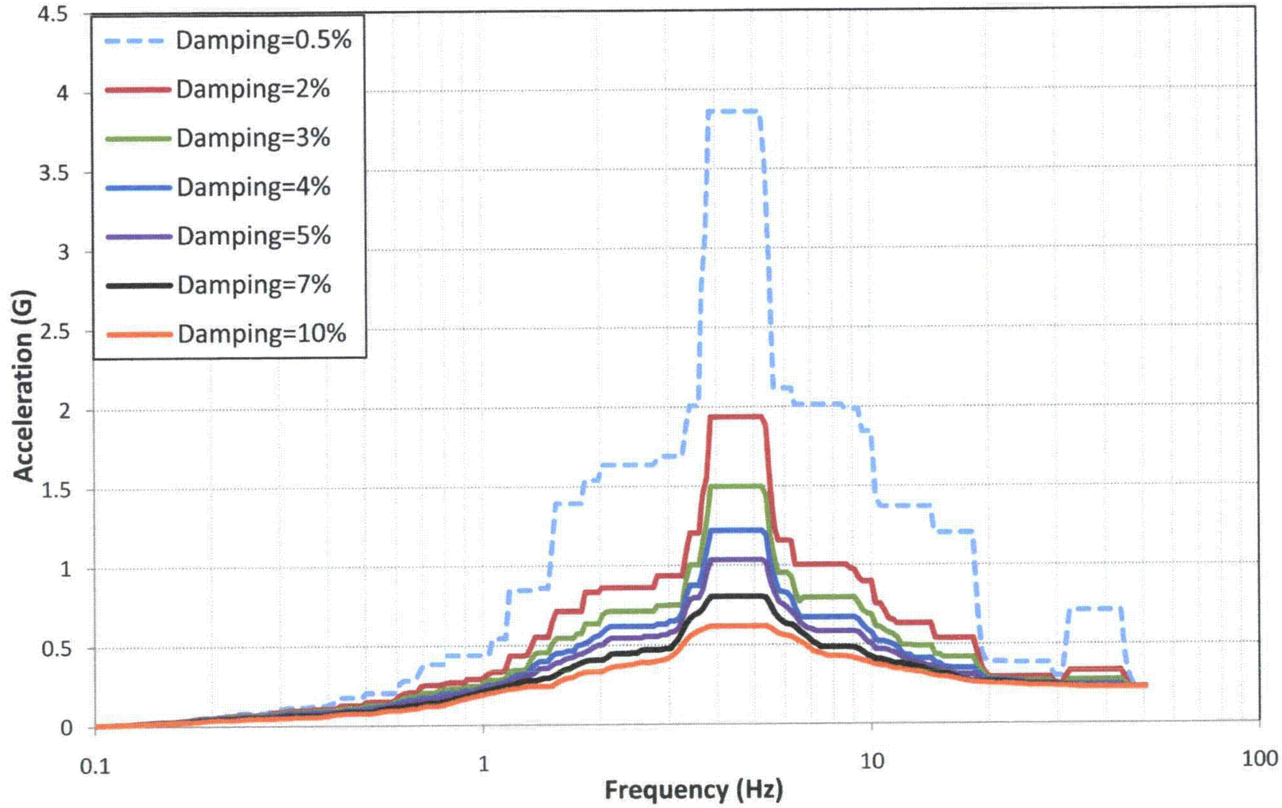


Figure 3.7-79— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}

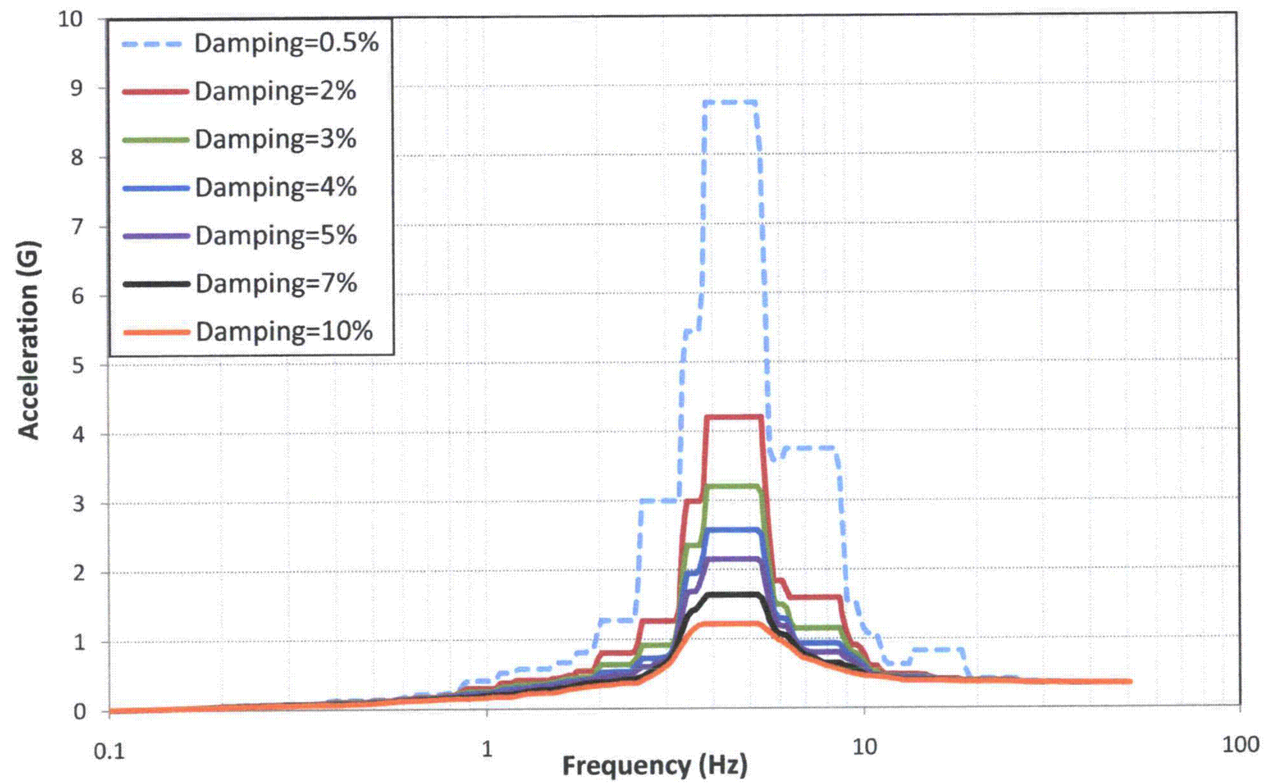


Figure 3.7-80— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}

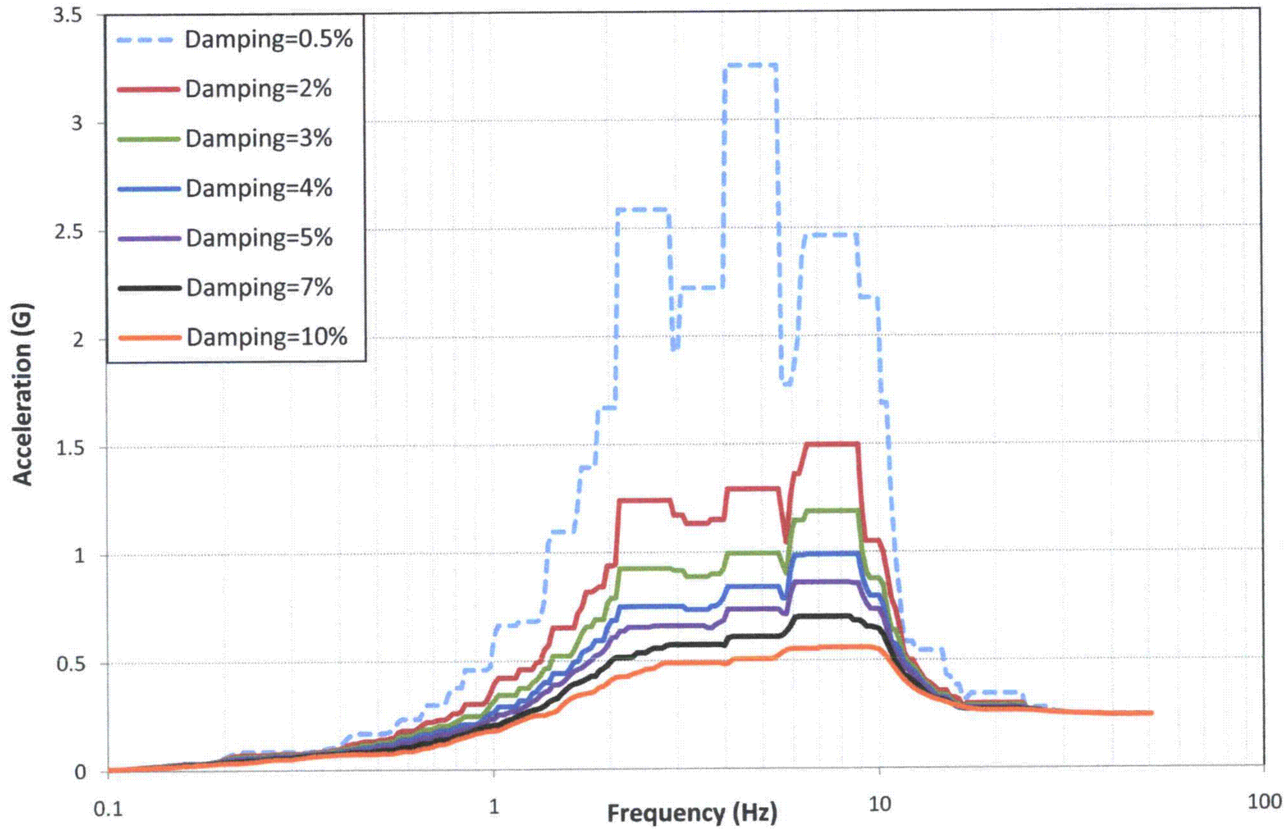
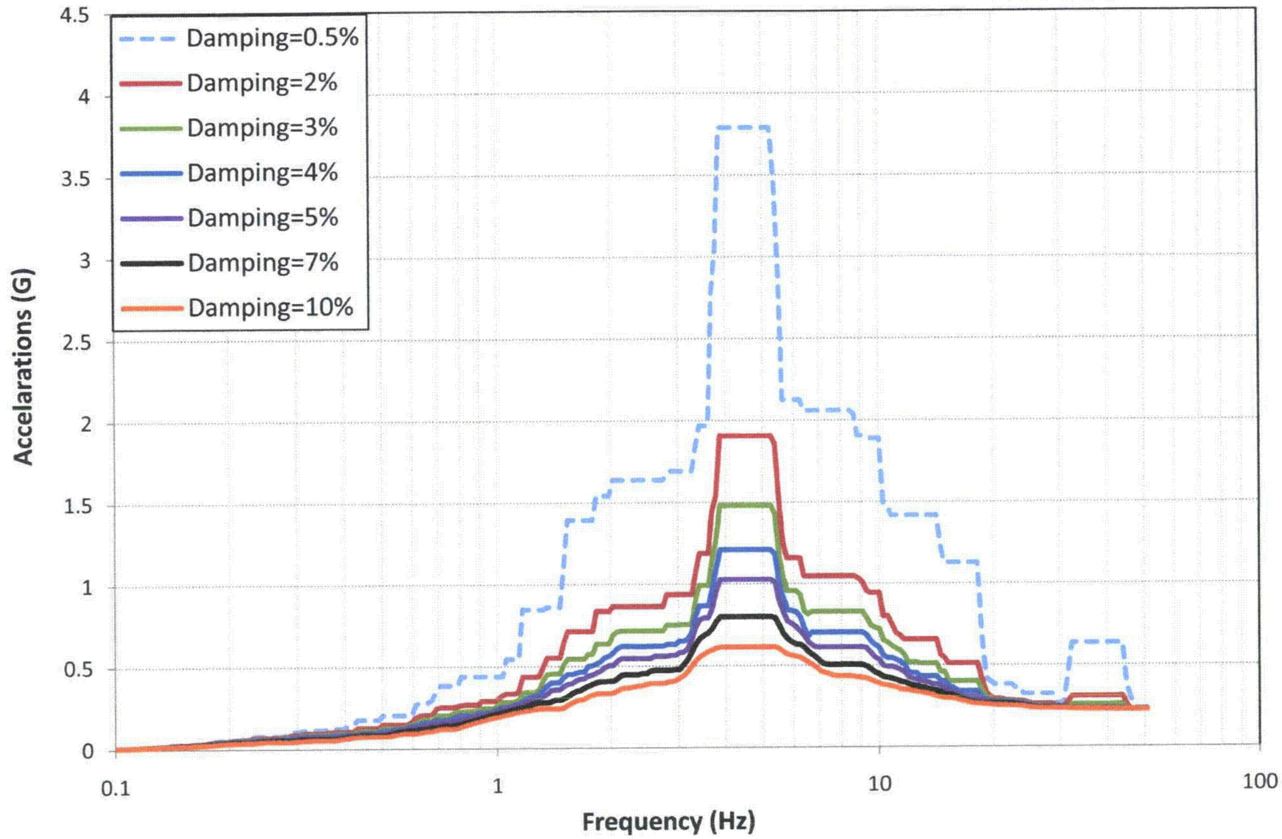


Figure 3.7-81— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}



Enclosure 4

Revised CCNPP Unit 3 FSAR COLA Part 7

1.1 DEPARTURES

This Departure Report includes deviations in the CCNPP Unit 3 COL application FSAR from the information in the U.S. EPR FSAR, pursuant to 10 CFR Part 52. The U.S. EPR Design Certification Application is currently under review with the NRC. However, for the purposes of evaluating these deviations from the information in the U.S. EPR FSAR, the guidance provided in Regulatory Guide 1.206, Section C.IV.3.3, has been utilized.

The following Departures are described and evaluated in detail in this report:

1. Maximum Differential Settlement (across the basemat)
2. Maximum Annual Average Atmospheric Dispersion Factor (limiting sector),
3. Accident Atmospheric Dispersion Factor (0-2 hour, Low Population Zone)
4. Toxic Gas Detection and Isolation
5. Shear Wave Velocity
6. Coefficient of Static Friction
7. Maximum Non-Coincident Wet Bulb Temperature Value at 0% Exceedance (85°F)
8. ~~In-Structure Response Spectra~~Soil Column Beneath the Nuclear Island, ESWB and EPGB
9. Generic Technical Specifications and Bases - Setpoint Control Program

1.1.1 MAXIMUM DIFFERENTIAL SETTLEMENT (ACROSS THE BASEMAT)

Affected U.S. EPR FSAR Sections: Tier 1 Table 5.0-1, Tier 2 Table 2.1-1, Tier 2 Section 2.5.4.10.2

Summary of Departure:

The U.S. EPR FSAR identifies a maximum differential settlement of 1/2 inch in 50 feet (i.e., 1/1200) in any direction across the basemat. The estimated settlement values for the Nuclear Island common basemat, Emergency Generating Building foundations, and Essential Service Water System Cooling Tower foundations exceed the U.S. EPR FSAR value.

Extent/Scope of Departure:

This Departure is identified in CCNPP Unit 3 FSAR Table 2.0-1 and Section 2.5.4.10.2.

Departure Justification:

The estimated site-specific values for settlement of the CCNPP Unit 3 Nuclear Island common basemat foundation are in the range of 1/600 (1 inch in 50 feet) to 1/1200 (1/2 inch in 50 feet) as stated in FSAR Section 2.5.4.10.2.

As described in FSAR Section 3.8.5.5.1, to account for the Calvert Cliffs site-specific expected differential settlement values, an evaluation of differential settlements up to 1/600 (1 inch in 50 feet) was performed. The evaluation consisted of a static finite element analysis of the foundation structures which considered the effects of the higher expected displacement (tilt) on the foundation bearing pressures and basemat stress due to structural eccentricities resulting from a uniform rotation of the foundation mat along the axis of the nuclear island common basemat. The evaluation assumed no changes in the soil stiffness or increased flexure due to differential settlement consistent with the design analysis for the standard U.S.

8. Result in a departure from a method of evaluation described in the plantspecific FSAR used in establishing the design bases or in the safety analyses.

This Departure does not affect resolution of a severe accident issue identified in the plant-specific FSAR.

Therefore, this Departure has no safety significance.

~~1.1.8 In-Structure Response Spectra (ISRS)~~

~~Affected U.S. EPR FSAR Sections: Tier 2 Section 3.7.2~~

~~Summary of Departure:~~

~~The U.S. EPR FSAR identifies ISRS at representative locations of the EPGB, and ESWB. The corresponding CCNPP Unit 3 ISRS are identified in the CCNPP3 FSAR Section 3.7.2.5 and represent a departure from the U.S. EPR FSAR.~~

~~Scope/Extent of Departure:~~

~~This Departure is identified in Part 2 FSAR, Section 3.7.2.5.2.~~

~~Departure Justification:~~

~~This departure is justified using the U.S. EPR FSAR Section 2.5.2.6 seismic reconciliation guidelines. The CCNPP Unit3 site-specific in-structure response spectra (ISRS) for the EPGB and ESWB are developed from the CCNPP Unit 3 Site SSE spectrum and soil profiles and are compared with the U.S. EPR design certification ISRS.~~

~~For critical locations in EPGB and ESWB at frequencies greater than approximately 0.3 Hz, the CCNPP Unit 3 site-specific ISRS do not exceed the ISRS for the U.S. EPR.~~

~~For frequencies less than approximately 0.3 Hz where the site-specific ISRS exceed the design ISRS by more than 10 %, evaluations of safety-related structures, systems, and components (SSC) were performed in accordance with Step 9 of U.S. EPR FSAR 2.5.2.6. These evaluations are discussed in Sections 3.7.2.5.2 and 3.10 and confirm the SSCs will perform their safety related functions following an SSE.~~

~~Departure Evaluation:~~

~~This Departure, associated with ISRS, has been evaluated in accordance with the U.S. EPR FSAR Section 2.5.2.6 seismic reconciliation guidelines and determined to not affect the safety function of the safety-related SSCs of the U.S. EPR at the building locations where CCNPP Unit 3 site-specific ISRS exceed the ISRS for the U.S. EPR design certification by more than 10%. Accordingly, this Departure does not:~~

1. ~~Result in more than a minimal increase in the frequency of occurrence of an accident previously evaluated in the plant-specific FSAR;~~

2. ~~Result in more than a minimal increase in the likelihood of occurrence of malfunction of a structure, system, or component (SSC) important to safety and previously evaluated in the plant-specific FSAR;~~

3. ~~Result in more than a minimal increase in the consequences of an accident previously Result in more than a minimal increase in the consequences of a malfunction of an SSC important to safety previously evaluated in the plant-specific FSAR;~~

4. Create a possibility for an accident of a different type than any evaluated previously in the plant-specific FSAR;

5. Create a possibility for a malfunction of an SSC important to safety with a different result than any evaluated previously in the plant-specific FSAR;

6. Result in a design basis limit for a fission product barrier as described in the plant-specific FSAR being exceeded or altered; or

7. Result in a departure from a method of evaluation described in the plant-specific FSAR used in establishing the design bases or in the safety analyses.

This Departure does not affect resolution of a severe accident issue identified in the plant-specific FSAR.

Therefore, this Departure has no safety significance.

1.1.9 Soil Column Beneath the Nuclear Island, ESWB and EPGB

Affected U.S. EPR FSAR Sections: Tier 2 Section 2.5.2.6, 3.7.1, and 3.7.2

Summary of Departure:

The soil column for the NI discussed in section 2.5.2.6 and presented in Table 2.5 – 76 and 2.5-77 and Figures 2.5-242 and 2.5-243 have a minimum strain compatible shear wave velocity, less than the 700 fps specified in U.S. EPR FSAR Tables 3.7.1-6 and 3.7.2-9. In addition the soil weight density is greater than the value specified in Table 3.7.2-9.

Scope/Extent of Departure:

This Departure is identified in Part 2 FSAR, Section 2.5.2.6.

Departure Justification:

This departure is justified in two parts as follows:

a. The soil column for the NI discussed in section 2.5.2.6 and presented in Table 2.5 – 76 and 2.5-77 and Figures 2.5-243 and 2.5-244 have a minimum strain compatible shear wave velocity, less than the 700 fps specified in U.S. EPR FSAR Tables 3.7.1-6 and 3.7.2-9.

   This portion of the departure has been identified because the NI Best Estimate SWV profile consists of weighted average backfill SWV's of 620 fps and 688 fps for the backfill layer. This departure can be justified for the following reasons.

   The departure addresses a SWV that is on average less than 12% lower than the minimum used in the U.S. EPR FSAR (700 fps).

   The average backfill SWV's of 620 fps and 688 fps is associated with the site-specific SSE which is used in the confirmatory analyses. Considering the CCNPP3 site-specific FIRS rather than the SSE, the strain-compatible SWV values would be equal to or larger than the minimum SWV value analyzed in the U.S. EPR FSAR. This means that the departure is a result of the use of a conservative SSE input to the confirmatory analyses.

For the EPGB and ESWB, the CCNPP3 Best Estimate, Lower Bound, Upper Bound SWV profiles are included in Tables 3F-3, 3F-4, and 3F-5. Similar to the NI, these tables show a departure from the U.S. EPR FSAR minimum SWV of 700 fps.

In order to quantify the impact of these departures, two approaches are taken.

For the EPGB and ESWB, the confirmatory analysis was performed with the CCNPP3 values reflecting the backfill. As discussed in Section 2.5.2.6.2, Reconciliation Step 8, the comparison shows that the CCNPP3 ISRS are well bounded.

For the, NI because the backfill was introduced after the completion of the confirmatory analysis, a different approach is used. This approach compares the FIRS with and without backfill. The effect of the backfill is to increase the ZPA and peak accelerations of the FIRS by 11% and 16% respectively. The NI FIRS with backfill remain bounded by the site SSE which is the basis for the confirmatory analysis.

Another reason which makes the departure acceptable is that the departure is associated with low, not high SWV's. This is not critical because hard rock SWV profiles, not low SWV profiles, generally control the design of the U.S. EPR. Based on the logic that the high SWV's generally control the generic design, the low values that are the basis for the departure do not impact the conclusion that the U.S. EPR FSAR seismic response bounds the CCNPP3 site-specific response. This conclusion has been confirmed by the results of the CCNPP3 confirmatory analysis which are discussed in Reconciliation Step 8.

The overall conclusion is that the CCNPP3 SWV's profile is similar to and bounded by the 10 generic soil profiles used for the U.S. EPR. The CCNPP3 SWV profile is bounded by the U.S. EPR FSAR range of profiles because high rather than low SWV profiles generally control the generic design of the U. S. EPR.

b. In addition the soil weight density is greater than the value specified in Table 3.7.2-9.

This portion of the departure has been written to address the fact that the U.S. EPR FSAR seismic analyses are based on a soft soil unit weight of 110 pcf. The CCNPP3 unit weight for the in-situ soil in the NI, EPGB, and ESWB area ranges from 105 pcf to 125 pcf. The unit weight of the backfill is 145 pcf partially a result of the high compaction requirements. The confirmatory analysis for the EPGB and ESWB and the development of the FIRS for the NI used the site-specific unit weights. Therefore, the influence of this departure has been taken into account in the supporting analyses.

Departure Evaluation:

This Departure, associated with strain compatible shear wave velocities beneath the NI, EPGB, and ESWB has been evaluated in accordance with the U.S. EPR FSAR Section 2.5.2.6 seismic reconciliation guidelines and determined to not affect the conclusion that the NI, EPGB, and ESWB safety-related structures may be used at the CCNPP Unit 3 as designed in the U.S. EPR FSAR.

Accordingly, this Departure does not:

1. Result in more than a minimal increase in the frequency of occurrence of an accident previously evaluated in the plant-specific FSAR;
2. Result in more than a minimal increase in the likelihood of occurrence of malfunction of a structure, system, or component (SSC) important to safety and previously evaluated in the plant-specific FSAR;
3. Result in more than a minimal increase in the consequences of an accident previously Result in more than a minimal increase in the consequences of a malfunction of an SSC important to safety previously evaluated in the plant-specific FSAR;
4. Create a possibility for an accident of a different type than any evaluated previously in the plant-specific FSAR;
5. Create a possibility for a malfunction of an SSC important to safety with a different result than any evaluated previously in the plant-specific FSAR;
6. Result in a design basis limit for a fission product barrier as described in the plant-specific FSAR being exceeded or altered; or
7. Result in a departure from a method of evaluation described in the plant-specific FSAR used in establishing the design bases or in the safety analyses.

This Departure does not affect resolution of a severe accident issue identified in the plant-specific FSAR.

Therefore, this Departure has no safety significance.

1.1.10 GENERIC TECHNICAL SPECIFICATIONS AND BASES - SETPOINT CONTROL PROGRAM

Affected U.S. EPR FSAR Sections: Tier 2, Section 16 - Technical Specifications (TS) 3.3.1 and 5.5, and Bases 3.3.1

Summary of Departure:

A Setpoint Control Program is adopted in the CCNPP Unit 3 Technical Specifications (TS). TS 3.3.1 is revised to delete the associated Reviewer's Notes and bracketed information. Applicable Surveillance Requirements and footnotes are revised to reference the Setpoint Control Program. Numerical setpoints are removed and replaced with a reference to the Setpoint Control Program. TS 5.5 is revised to add a Setpoint Control Program description to the Administrative Controls - Programs and Manuals Section (5.5). The Setpoint Control Program description references the NRC approved setpoint methodology documents that shall be used for the development of required numerical setpoints. The TS Bases 3.3.1 are revised to incorporate additional background information and clarify the applicability of the program to specific functions.

Enclosure 5

Revised CCNPP Unit 3 FSAR Section 3.10

UN#11-107
Enclosure 5
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3.10 SEISMIC AND DYNAMIC QUALIFICATION OF MECHANICAL AND ELECTRICAL EQUIPMENT

{This section of the U.S. EPR FSAR is incorporated by reference with the supplements and departures as described in the following sections.

~~For CCNPP Unit 3, seismic qualification of equipment located in the EPGB and ESWB is performed using the In-Structure Response Spectra (ISRS) provided in Section 3.7.2.5 instead of the U.S. EPR FSAR ISRS.~~

For CCNPP Unit 3, seismic and dynamic qualification of mechanical and electrical equipment (identified in Table 3.10-1) includes equipment associated with the:

- UHS Makeup Water System, including the UHS Makeup Water Intake Structure; and
- Fire Protection System components that are required to protect equipment required to achieve safe shutdown following an earthquake, including the Fire Protection Building and Fire Water Storage Tanks.

Results of seismic and dynamic qualification of equipment by testing and/or analysis were not available at the time of submittal of the original COL application. Thus, in conformance with NRC Regulatory Guide 1.206 (NRC, 2007), a seismic qualification implementation program is provided. As depicted in Table 3.10-2, the qualification program will be implemented in five major phases.

Phase I (Seismic Qualification Methodology) involves the development of a summary table for equipment. This summary table shall:

- List equipment, along with the associated equipment identification number.
- Define the building in which each equipment is located, along with the equipment mounting elevation.
- Clarify whether the equipment is wall mounted, floor mounted, or line mounted.
- For mechanical equipment, identify if the equipment is active or passive.
- Provide a description of the intended mounting (e.g., skid mounted versus mounted directly on the floor, welded versus bolted, etc.).
- List the applicable In-Structure Response Spectra or, for line mounted equipment, the required input motion.
- Define operability and functionality requirements.
- Identify the acceptable qualification methods (i.e., analysis, testing, and/or a combination of both).
- Provide a requirement for environmental testing prior to seismic testing, when applicable.

The basis and criteria established in Phase I shall be used as technical input to the Phase II (Specification Development) technical requirements that will be provided to bidders. In addition, the specification will include the applicable seismic qualification requirements of the