# Enclosure 1

## Response to NRC Request for Additional Information
## RAI 339, Questions 03.08.04-33 and -34, Other Seismic Category I Structures, Calvert Cliffs Nuclear Power Plant, Unit 3

**RAI No. 339**

**Question 03.08.04-33**

**<u>Follow Up to RAI 301, Question 03.08.04-18</u>:**

The staff reviewed the response to RAI 301, Question 03.08.04-18 provided in UniStar Letter UN#11-227 dated October 31, 2011 (ML11307A243). Based on this response and CCNPP Unit 3 FSAR Revision 7, the staff understands that, for the design and analysis method for the Forebay and the UHS MWIS, as discussed in FSAR Section 3.8.4.4.7: (1) the complex frequency response analysis method is used for the seismic SSI analysis to obtain accelerations, and the SRSS method is used to combine the accelerations due to three components of earthquake motion to obtain maximum nodal accelerations, (2) the maximum nodal accelerations are applied to the static FE model to obtain member forces due to seismic loads, and the design member forces due to seismic loads are calculated using the SRSS method. Based on a review of U.S. EPR FSAR Revision 3, the major difference between EPR and CCNPP methods to determine the design member forces due to seismic loads is that the EPR DC application uses algebraic summation to combine the accelerations due to three components of earthquake motion for all seismic Category I structures. Furthermore, in the EPR DC application, confirmatory analyses were performed to verify the conservatism of the EPR method to determine the design member forces due to seismic loads.

Since the CCNPP method is different than the EPR method, which has been accepted by the staff, provide the technical basis to demonstrate that the CCNPP method to determine the design member forces due to seismic loads for the design and analysis for the Forebay and the UHS MWIS is at least as conservative as the EPR method, or more detailed methods which utilize time history analysis or response spectrum analysis. Guidance on the need for justification for the use of equivalent static load methods for seismic analysis is discussed in SRP Acceptance Criteria 3.7.2.II.1.B.i.

In addition, since CCNPP Unit 3 no longer follows the EPR method to combine accelerations, any CCNPP Unit 3 FSAR reference to the EPR FSAR regarding use of the the EPR methods should be updated, and, a detailed description of the methodology utilized by CCNPP Unit 3 should be included in the applicable section of the CCNPP Unit 3 FSAR.

The staff needs the above information to be able to determine whether FSAR Section 3.8.4.4.7 is consistent with SRP Acceptance Criteria 3.7.2.II.1.B.i and 3.8.4.II.4.

**Response to RAI 339 Question 03.08.04-33**

Due to the new 2012 Central and Eastern United States (CEUS) Seismic Source Characterization, the seismic soil structure interaction (SSI) analysis and design of the common basemat intake structure (CBIS) has been updated.

In the new analysis, the accelerations due to three components of earthquake motion are combined using algebraic summation.

**COLA Impact**

Enclosure 2 provides the COLA impact of the response to RAI 339, Question 03.08.04-33.

**RAI No. 339**

**Question 03.08.04-34**

**Follow Up to RAI 301, Question 03.08.04-20:**

In RAI 301, Question 03.08.04-20, the staff requested that the applicant provide additional information on (1) the exclusion of soil load/lateral earth pressure (H) in the load combinations listed in FSAR Section 3.8.5.3 for bearing pressure evaluation, and (2) the large differences in maximum bearing pressure and bearing capacities between Revision 6 and Revision 7 of the CCNPP Unit 3 FSAR, for the design of the UHS Makeup Water Intake Structure (MWIS).

The staff reviewed the response to RAI 301, Question 03.08.04-20 provided in UniStar Letter UN#11-278 dated November 4, 2011 (ML11314A040). Regarding the bearing pressure calculation, the staff determined that additional information is needed to resolve this RAI item. Since the RAI response indicates that the maximum bearing pressures (static and dynamic) of the CBIS are the maximum of the average pressures under each of the three intake structures (the UHS MWIS, the Forebay and the Circulating Water Makeup Intake Structure), they are not the localized pressures such as the pressures at the toe of the CBIS basemat. The staff requests that the applicant provide the values of the maximum pressures considering all locations of the CBIS basemat design (e.g., maximum pressures that may occur at the toe/edge/corner of the basemat) under worst-case static and dynamic loads. This would represent the localized maximum soil bearing pressure, not the maximum of the average of pressures under each of the three intake structures. Furthermore, explain how these presures are obtained, e.g., how the soil springs are developed, what the differences are between soil springs for static and dynamic pressures, and how the dynamic pressures from the STAAD Pro FE model analysis compare with the pressures from the SSI analysis, etc. In addition, explain whether the CCNPP Unit 3 bearing capacities provided in Table 3.8-3, as well as those in Table 2.5-65, of the FSAR Revision 7, are the bearing capacities for localized pressure. If they are bearing capacities for localized pressure, explain why the calculated average pressures of the CBIS are compared with the bearing capacities for localized pressure. If they are bearing capacities for average pressure, explain why in Table 2.0-1 of the CCNPP Unit 3 FSAR Revision 7 the CCNPP Unit 3 bearing capacities are compared with the U.S. EPR bearing capacities for localized pressure.

The staff needs the information to determine whether the foundation design of the seismic Category I structures is consistent with SRP Acceptance Criteria 3.8.5.II.2 and 4 and has been adequately addressed in the CCNPP Unit 3 FSAR.

**Response to RAI 339 Question 03.08.04-34**

Static Load Combinations:

Bearing pressures for the following static load combinations are obtained from the STAAD model.

-D+L+F

-D+L+F+W

-D+L+F+Wt

-D+L+F+SPH

-D+L+F+PMH

(D: Dead Load; L: Live Load; F: Fluid; W: Wind; Wt: Tornado; PMH: Probable Maximum Hurricane; SPH: Standard Project Hurricane)

Contour plots of bearing pressure for these load combinations are provided in ***Figure 1***. Average bearing pressures across the CBIS basemat and maximum localized pressures for each load combination are provided in ***Table 1***.

For the static load combinations, the STAAD model maximum bearing pressures at each node are obtained by dividing the nodal reaction (spring force) by the nodal tributary area.

The springs for the STAAD model are developed based on the geotechnical PLAXIS 3D model. Springs are developed in such a way that the settlement from the STAAD model is within 5% of the settlement from the PLAXIS 3D model for a given foundation location. These springs are used for all static load combinations.

The contour plots in ***Figure 1*** indicate that maximum local pressures are concentrated around the Southern edge of the CBIS.

Bearing capacity as reported in Table 2.5-65 of FSAR Rev. 7 (now Table 2.5-67 of FSAR Rev. 9) is associated with the global soil failure underneath the foundation (general shear failure) rather than a local failure such as the failure of a soil element at a corner of the foundation. Therefore, the local maximum bearing pressure is not comparable to the bearing capacity reported in Table 2.5-67 of FSAR Rev. 9.

In order to make a relevant comparison, the following three steps are implemented:

1. Calculation of the resultant foundation load and its corresponding eccentricity that is equivalent to the bearing pressure distribution shown in ***Figure 1***.

2. Determination of the reduced area (effective area) due to eccentricity.

3. Computation of the increased average bearing pressure as the ratio of the total vertical load to the reduced area.

***Table 1*** shows that the average foundation pressures for the entire CBIS area (not the reduced area) are lower than the static bearing capacity, 11.7 ksf, reported in Table 2.5-67 of Rev. 9. The reduced area or effective area calculated based on the eccentricity is at least 65 percent of the overall area. To be conservative, a reduction of 50% in the area of the CBIS is considered in the calculation of the average bearing pressure. The increased average bearing pressures corresponding to the 50 percent reduction in the area are shown in ***Table 1*** and these are lower than the bearing capacity.

The information provided in Table 2.0-1 is only meant for the design certification structures. Therefore, bearing capacities for the site-specific CBIS are not compared to the values of Table 2.0-1.

Seismic Bearing Pressures:

For the seismic load combination (D+L+F+E'), the static bearing pressures are summed with the seismic bearing pressures. In the revised SSI analysis, the STAAD model is not used to evaluate seismic bearing pressures, since it is overly conservative to assume maximum accelerations for all nodes to occur simultaneously. Instead, results from the SSI SASSI analysis are used to evaluate the seismic bearing pressures.

The seismic bearing pressures are obtained from the SSI SASSI analysis at the CBIS foundation-soil interface using 3D brick elements throughout the entire basemat area. From the SSI simulations, the time histories of vertical and horizontal stresses throughout the entire basemat area are obtained. The seismic bearing pressures are computed for the SSE and OBE conditions considering three soil cases (Lower Bound (LB), Best Estimate (BE), and Upper Bound (UB)).

The time history of the total vertical seismic bearing pressure at the bottom of the basemat is obtained using the algebraic summation of the bearing pressures due to three components of the earthquake motion.

Static bearing pressures associated with the dead loads and soil loads (including buoyancy) are obtained from the PLAXIS 3D model of the CBIS.

For the evaluation of seismic bearing pressures, average bearing pressures are obtained for the part of the foundation that is not subjected to uplift as follows:

1. For a given time step, the nodal net vertical pressure (seismic vertical pressure from SASSI+static vertical pressure from PLAXIS 3D) is obtained.

2. If the nodal net pressure is compressive, the pressure is multiplied with the nodal tributary area to get the nodal compressive force; negative nodal pressures are not accounted for.

3. The total compressive forces from all nodes that are in compression are summed, and divided by the area that is under compression.

The seismic bearing capacity check is conducted for the following time steps:

1. The time step of maximum uplift, which represents the smallest area subjected to compression

2. The time step at which the compressive pressure as defined above is maximum.

3. The time step at which the overturning factor of safety is minimum

4. The time step at which the sliding factor of safety is minimum.

These time steps are the critical time steps in terms of bearing capacity check.

In addition to checking for average seismic bearing pressures, local seismic bearing pressures are also checked at all time steps at all locations.

The SASSI simulations for all three soil cases are conducted for the operational water level and for both SSE and OBE conditions. In addition, seismic stability is checked for the maintenance and the maximum water level cases with the BE soil profile and SSE conditions.

The maximum average seismic bearing pressure is less than 4.0 ksf based on the area that is in compression. Similar to the static case, a 50 percent reduction to the area in compression (not the entire CBIS area) is applied to account for eccentricity, resulting in an average pressure of 8.0 ksf, which is lower than the seismic bearing capacity.

The maximum local bearing pressure, when all time steps and all cases are considered, is 18.6 ksf. For the 558 CBIS basemat solid elements checked and for more than 8,000 time steps, the local bearing pressures are below 17.6 ksf except on one corner element at two time steps (***Figure 2***).

***Figure 2*** shows the contour plots at the two time steps in which the maximum bearing pressures are obtained. Average seismic bearing pressures the CBIS basemat (***Table 1***) are below the seismic bearing capacity.

**TABLE 1**
**BEARING PRESSURE EVALUATION**

| | Load Combination | Maximum Local Pressure (KSF) | Average Bearing Pressure (KSF) | Increased Average Bearing Pressure (1) (KSF) | Bearing Capacity (KSF) |
|---|---|---|---|---|---|
| Static | D +L +F | 14.8 | 2.5 | 5.1 | 11.7 |
| | D +L +F+W | 14.9 | 2.5 | 5.0 | |
| | D +L +F+Wt | 15.6 | 2.5 | 4.9 | |
| | D +L +F+SPH | 14.9 | 2.4 | 4.9 | |
| | D +L +F+PMH | 16.9 | 2.3 | 4.7 | |
| Seismic | D+L+F+E' | 18.6 | 4.0 | 8.0 (2) | 17.6 |

(1) Effective area of the foundation resisting the load is assumed as the 50% of the CBIS basemat area.

(2) Effective area of the foundation resisting the load is assumed as the 50% of the CBIS area that is in compression.

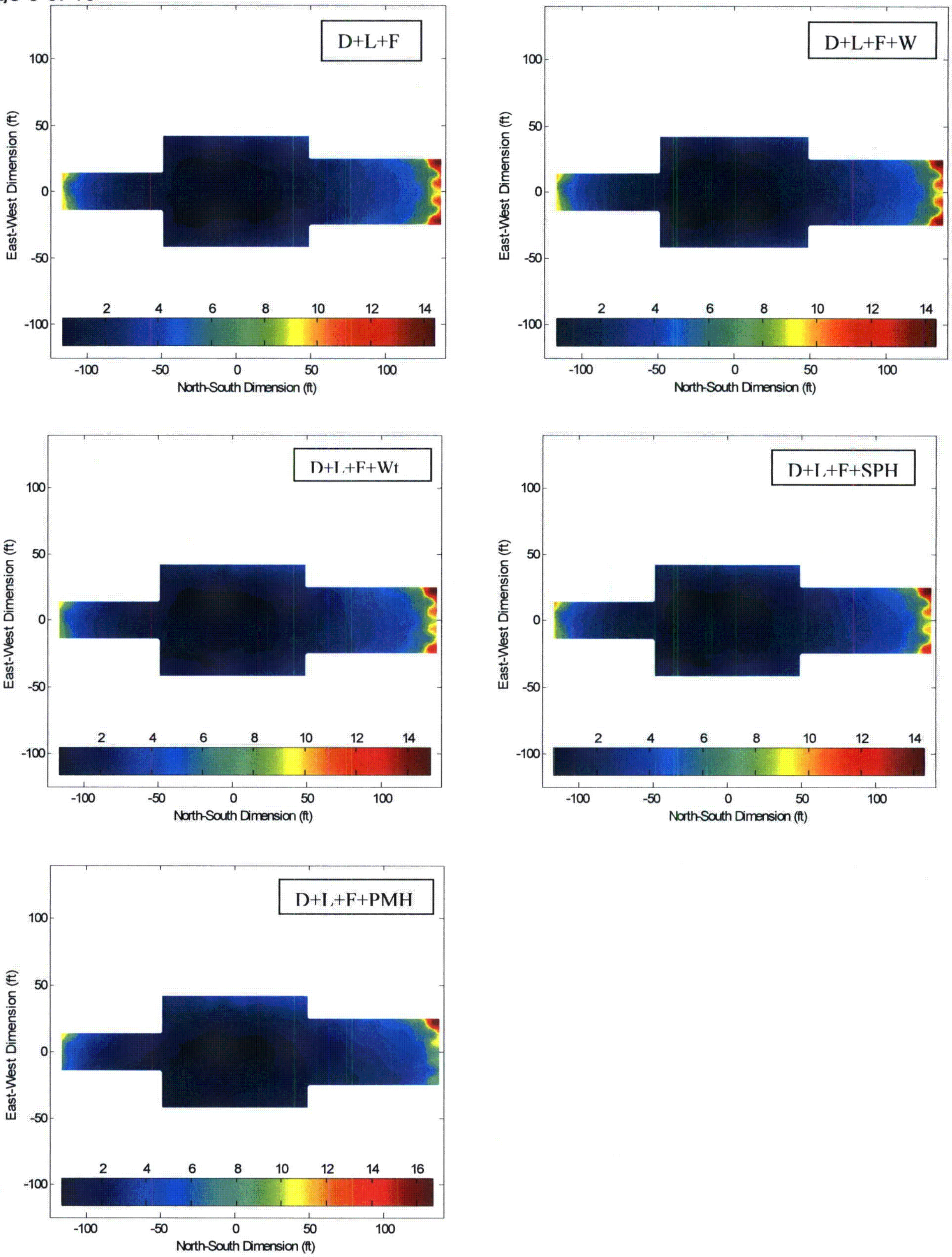
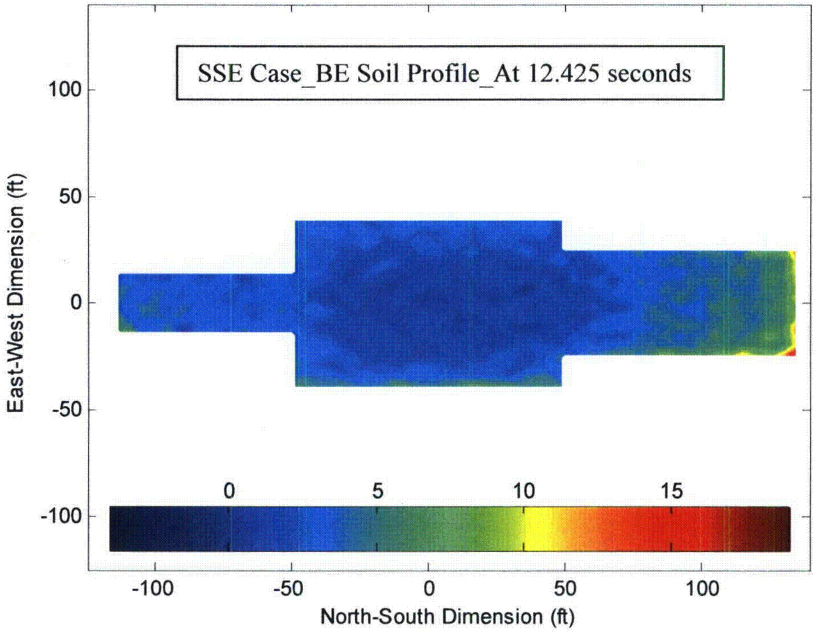



**FIGURE 1**
**CONTOUR PLOTS OF BEARING PRESSURE FOR STATIC LOAD COMBINATIONS**



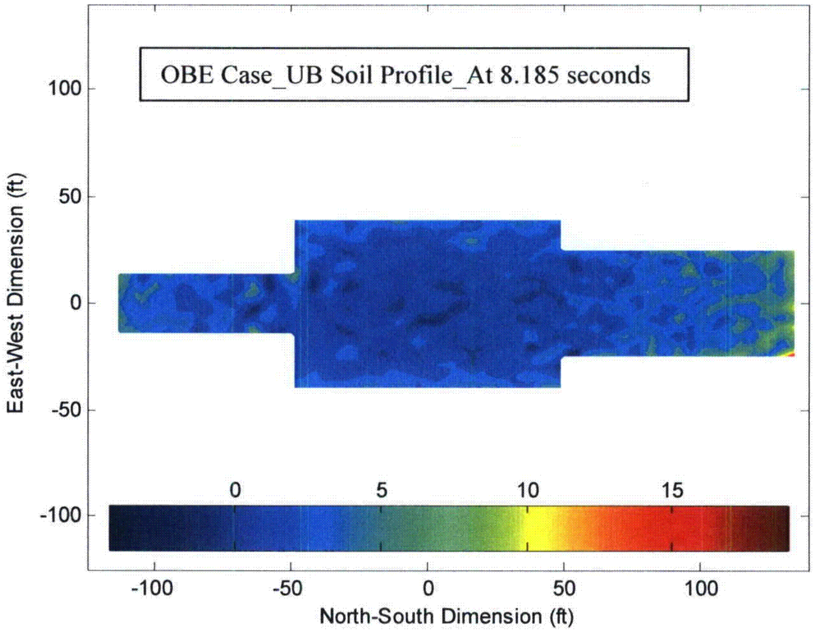




**FIGURE 2**
**CONTOUR PLOTS OF BEARING PRESSURE FOR SEISMIC LOAD COMBINATIONS**

**COLA Impact**

Enclosure 2 provides the COLA impact of the response to RAI 339, Questions 03.08.04-34.

# Enclosure 2

## Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 339, Questions 03.08.04-33 and -34, Calvert Cliffs Nuclear Power Plant, Unit 3

# FINAL SAFETY ANALYSIS REPORT

# CHAPTER 3

# DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

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.

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03.08.04-33

### 3.7.1.3.3 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~~ 1000 ft (~~200m~~304.8m). 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) 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.

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**

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03.07.02-72

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~~ACS SASSI, Version ~~1.3a~~2.3.0. 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.

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03.07.02-72

### 3.7.2.2.3 Common Basemat Intake Structures

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03.07.02-72

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 Table 3.7-5 ~~and Table 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~~, at the locations of safety-related traveling screens at EL. 21.0 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 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.

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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~~ACS SASSI, Version ~~1.3a~~2.3.0, and to perform static analysis for non-seismic loads.

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03.07.02-72

~~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.~~

RAI 343
03.07.02-74

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 in all three directions of motion except for one local mode of vibration of the slabs in the UHS along the vertical direction, with a frequency of around 30 Hz (secondary peak). The maximum difference observed, when comparing the response to a very refined model, is limited to 15 percent around the peak. This small difference is accounted for by increasing the peaks around 30 Hz of the vertical ISRS at the center of slabs of the UHS, by a factor of 1.2 after performing the smoothing and broadening per RG. 1.122.

RAI 343
03.07.02-72

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 is 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.~~ in the X (north-south) and Z (vertical) directions. In the Y (east-west) direction, a small shift in the main frequencies (less than 1 Hz, or 6%), is observed when comparing the fixed base SASSI model with a fixed base STAAD model that includes a thick shell element formulation. There is also a maximum difference of 10% for the amplitudes of the main and secondary peaks. These differences are accounted for by increasing the peaks of ISRS in the Y (east-west) direction by a factor of 1.15 after performing the smoothing and broadening. Per RG 1.122, ISRS main frequencies are broadened by a factor of 15%, which covers the differences on the frequency shift observed in Y (E-W) direction.

RAI 343
03.07.02-74

The reinforced concrete basemat, floor slabs, and walls of the CBIS are modeled using thin shell elements in ~~RIZZO computer code~~ACS SASSI, Version ~~1.3a~~2.3.0, 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~~1.9 ft (~~0.5~~0.6 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~~ 5.3 ft (~~1.5~~1.6 m), ~~a~~based on an aspect ratio of approximately ~~3.0~~2.8.

RAI 339
03.08.04-34

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.

Two sets of models of the CBIS are considered to represent the effect of cracking in the concrete: one fully uncracked with OBE damping (RG 1.92) and the other one fully cracked with SSE damping. The cracked model considers that the out-of-plane bending of the walls and floors as 50 percent of the gross bending stiffness ASCE 43-05 (ASCE, 2005).The envelope of the response of both models is used for the calculation of stresses in the soil for stability analysis, ISRS and accelerations on the structure.

RAI 343
03.07.02-72

~~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.~~

RAI 343
03.07.02-72

As shown in Figures 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~~ Therefore, the stiffness and 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 for normal water level, corresponding to MSL, at El. 0.64 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 CW, Forebay, and basement of the UHS Makeup Water Intake Structure.

RAI 343
03.07.02-71

The maximum sloshing heights in both directions for ~~the UHS Makeup Water Intake Structure and~~ the Forebay are approximately 0.82 ft (0.25 m) and 0.95 ft (0.29 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.

RAI 343
03.07.02-72

~~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**

The 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 ????, Table 3.7-3 and Table 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. Table 3F-3, Table 3F-4, and Table 3F-5 show the properties for the top fifty layers of each soil profile (approximately 300 ft), while Figure 3F-29, Figure 3F-30 and Figure 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. Table 3F-6, Table 3F-7 and Table 3F-8 show the properties for the top fifty layers of each soil profile (approximately 380 ft), while Figure 3F-32, Figure 3F-33 and Figure 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

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#### 3.7.2.4.2.1 Nuclear Island Common Basemat Structures

Confirmatory SSI analyses of the Nuclear Island Common Basemat Structures 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.

#### 3.7.2.4.2.2 EPGB and ESWB

Confirmatory SSI analyses for the EPGB and ESWB use finite element models. 4% structural damping 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 t~~Thin plate elements are used in SASSI ~~is used~~ to model all of the structural panels.

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The Common Basemat Intake Structures are primarily reinforced concrete structures with steel structures in the Steel Enclosure building and in the Forebay area. Structural damping corresponding to the OBE case (4 percent for concrete and 3 percent for steel) is used for the fully uncracked concrete model; and SSE damping (7 percent for concrete and 4 percent for steel) is used for the fully ~~un~~cracked concrete model.

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~~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

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space is introduced beneath the lowest sub-layer, which is located at a depth of ~~350~~ 365 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

A surface founded stick model is used for the Nuclear Island Common Basemat Structures confirmatory SSI analyses. The analysis uses the following 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 Figure 3.7-10, Figure 3.7-11 and Figure 3.7-12 are used for the SSI analysis.
- Four percent structural damping is applied.

#### 3.7.2.4.4.2 EPGB and ESWB

An SSI model and methodology of the EPGB and ESWB is used for the confirmatory SSI analyses. The analysis uses the following inputs:

- 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 Figure 3.7-10, Figure 3.7-11 and Figure 3.7-12 are used, while for the ESWB the within soil-column motions (acceleration time histories) shown in Figure 3.7-13, Figure 3.7-14 and Figure 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.

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#### 3.7.2.4.4.3 Common Basemat Intake Structures

The SSI model includes the CBIS, the Steel Enclosure Building, ~~-~~the surrounding layers of structural fill, and the existing soil media as shown in Figure 3.7-24. Three-dimensional brick elements are used for the entire basemat area in order to obtain seismic stresses for ~~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.

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 previously described methodology.

**3.7.2.4.5.2 EPGB and ESWB**

Confirmatory SSI analyses for the EPGB and ESWB are performed following the previously described methodology.

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**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~~ACS SASSI version 2.3.0. 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 and for two set of properties for the concrete: one considering all the elements uncracked with OBE damping (4 percent for concrete and 3 percent for steel) and the other with all the elements cracked with SSE damping (7 percent for concrete and 4 percent for steel).

**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.

Table 3.7-8 and Table 3.7-9 provide the combined average maximum nodal accelerations at various elevations of EPGB and ESWB, respectively. Comparison of the structural accelerations provided in Table 3.7-8 and Table 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.

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**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, soil stresses, ~~relative displacements, element forces,~~ in-structure response spectra, resultant sliding force and total overturning moments are calculated using the CBIS model in ACS SASSI.

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.

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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 using the CBIS model in STAAD Pro. 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.

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~~RIZZO computer code~~ACS SASSI, Version ~~1.3a~~2.3.0, is used to perform the seismic confirmatory SSI analysis of the CBIS. This program is ~~developed and maintained~~verified and validated 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~~the RIZZO 10 CFR 50 Appendix B QA program.

**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 (Figure 3.7-25 through Figure 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 (Figure 3.7-52 through Figure 3.7-54) at the basemat for NAB to the corresponding U.S. EPR standard design ISRS (Figure 3.7-55 through Figure 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 (Figure 3.7-58 through Figure 3.7-60) at the basemat for RWPB to the corresponding U.S. EPR standard design ISRS (Figure 3.7-61 through Figure 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.

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, Figure 3.7-64 to Figure 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. Reconciliation of the accelerations at these low frequencies is discussed in Section 2.5.2.6.

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### 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~~0.01 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 Figure 3.7-73 through Figure 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 Figure 3.7-79 through Figure 3.7-81. ISRS will be generated at the support locations of additional safety-related equipment, as required.

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### 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. The ISRS in the UHS Makeup Water Intake Structure are determined using the time history equal to the algebraic summation of the ~~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 Square Root of the Sum of the Squares (SRSS) combination rule to obtain the maximum total member forces and moments. The SRSS method 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 Structures

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:

### 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, 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.

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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.

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~~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~~ considers the self weight of the intake structure, static ~~earth pressures~~backfill loads within the structure, 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~~ summation of the stresses in that direction due to earthquake in each direction.

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iii. The response time history of total sliding shear stress is calculated at all nodes for each soil profile for both horizontal (X and Y) directions. The sliding shear stress in each horizontal direction is multiplied by the nodal tributary area to get the nodal sliding shear force. The sliding shear forces from all nodes are summed to get the total sliding shear in the X and Y directions. The total sliding shear force is then obtained as the square root of the sum of the squares of the sliding shear forces in the X and Y directions.

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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~~ forces/moments of the CBIS to the corresponding seismically induced ~~stresses~~forces/moments.

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v. For each soil profile, seismic stability is assessed for for two set of properties for the concrete: one considering all the elements uncracked with OBE damping (4 percent for concrete and 3 percent for steel) and the other with all the elements cracked with SSE damping (7 percent for concrete and 4 percent for steel).

vi. Two sets of friction coefficients are checked during the stability analysis.

Basemat-Mudmat Interface: tanφ = 0.6 and adhesion = 0;

Mudmat-Chesapeake Clay/Silt Layer Interface: tan φ = 0.21 and adhesion = 1.2 ksf

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~~iv.~~vii. The resisting shear stress $\tau$ at each node at each time step is obtained by calculating the net restoring vertical stress $\sigma_v$ (total vertical stress including water weight inside the structure and buoyancy under the CBIS) at each node at each time step and using $\tau = \sigma_v \tan\varphi + c$, where $\varphi$ = friction angle, and c=adhesion component. The resisting shear stress at each node is multiplied with the nodal tributary area to get the resisting nodal shear force. Finally, all resisting shear forces from all nodes are summed across the CBIS basemat. If the vertical stress at a given node is tensile, no contribution is considered to the resisting shear force from that node.

Only seismic active earth pressures are considered in the seismic stability analysis. Seismic active earth pressures are calculated according to the Mononobe-Okabe method (Kramer 1996). Not considering the passive earth pressures is conservative in the seismic stability analysis. Also not considered is the side friction for all cases except for the maintenance condition stability check. The stability analysis of the maintenance condition is conducted assuming no water within the CBIS. This is somehow conservative, since even during such maintenance condition, there will be water in some portions of the CBIS that still contribute to the weight and the overall sliding stability . To avoid the incorporation of excessive conservatism, a minor fraction of the side friction is introduced into the stability analysis for the maintenance condition. Static active earth pressures are considered as the normal forces and the friction coefficient is considered as 0.58 to calculate the side friction. Only 5% of the overall side friction is considered in the seismic sliding analysis of the CBIS for maintenance condition.

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

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## 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

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## Table 3.7.5-{Frequencies and Mass Participation Factors for Common Basemat Intake Structures-Fixed Base Analysis} (Coordinates based on CCNPP Unit 3)

| | Frequency | Mass Participation Factors (%) | | | | Frequency | Mass Participation Factors (%) | | |
|---|---|---|---|---|---|---|---|---|---|
| Mode # | (Hz) | N-S | Vertical | E-W | Mode # | (Hz) | N-S | Vertical | E-W |
| 1 | 1.69 | 0.71 | 0.00 | 0.82 | 51 | 31.68 | 72.61 | 25.85 | 86.96 |
| 2 | 2.23 | 0.80 | 0.00 | 1.15 | 52 | 31.98 | 72.65 | 26.75 | 87.00 |
| 3 | 2.24 | 1.05 | 0.00 | 1.15 | 53 | 32.07 | 72.65 | 26.78 | 87.89 |
| 4 | 2.74 | 1.07 | 0.02 | 1.42 | 54 | 32.22 | 73.42 | 26.88 | 87.90 |
| 5 | 3.10 | 1.59 | 0.02 | 1.46 | 55 | 32.94 | 73.98 | 27.00 | 88.50 |
| 6 | 7.60 | 1.63 | 0.27 | 14.14 | 56 | 33.40 | 74.53 | 27.85 | 88.58 |
| 7 | 10.79 | 1.68 | 0.53 | 55.99 | 57 | 33.47 | 74.64 | 28.25 | 88.58 |
| 8 | 11.79 | 1.71 | 1.02 | 56.00 | 58 | 33.77 | 74.88 | 28.88 | 88.59 |
| 9 | 13.83 | 46.21 | 1.51 | 56.01 | 59 | 34.01 | 74.89 | 29.67 | 88.59 |
| 10 | 14.19 | 46.22 | 1.51 | 56.51 | 60 | 34.26 | 74.94 | 30.20 | 88.60 |
| 11 | 14.46 | 46.24 | 1.51 | 60.18 | 61 | 34.38 | 74.94 | 30.20 | 88.92 |
| 12 | 15.12 | 46.25 | 1.51 | 61.56 | 62 | 34.55 | 74.96 | 31.56 | 88.92 |
| 13 | 15.87 | 46.48 | 1.52 | 70.26 | 63 | 34.72 | 75.01 | 32.56 | 88.94 |
| 14 | 16.41 | 46.48 | 1.52 | 73.10 | 64 | 35.06 | 75.04 | 34.54 | 88.95 |
| 15 | 16.80 | 46.54 | 1.52 | 76.80 | 65 | 35.42 | 75.10 | 35.53 | 89.04 |
| 16 | 17.78 | 46.54 | 1.54 | 77.09 | 66 | 35.44 | 75.10 | 36.00 | 89.04 |
| 17 | 18.22 | 46.57 | 1.60 | 77.43 | 67 | 35.45 | 75.13 | 37.20 | 89.04 |
| 18 | 18.85 | 46.77 | 2.82 | 77.43 | 68 | 35.64 | 75.14 | 37.84 | 89.05 |
| 19 | 19.52 | 47.91 | 2.82 | 77.44 | 69 | 36.02 | 75.71 | 38.38 | 89.07 |
| 20 | 20.09 | 48.42 | 2.96 | 77.55 | 70 | 36.20 | 76.82 | 38.47 | 89.08 |
| 21 | 20.57 | 48.42 | 2.96 | 77.86 | 71 | 36.33 | 77.47 | 38.50 | 89.12 |
| 22 | 21.10 | 48.42 | 2.98 | 78.51 | 72 | 36.94 | 78.25 | 38.99 | 89.17 |
| 23 | 21.34 | 48.42 | 3.04 | 79.23 | 73 | 37.04 | 78.25 | 39.34 | 89.17 |
| 24 | 21.47 | 49.62 | 15.51 | 79.24 | 74 | 37.22 | 78.34 | 40.86 | 89.19 |
| 25 | 22.56 | 49.79 | 15.60 | 79.73 | 75 | 37.40 | 78.36 | 41.55 | 89.21 |
| 26 | 23.23 | 49.79 | 15.66 | 81.35 | 76 | 37.50 | 78.43 | 42.74 | 89.21 |
| 27 | 23.24 | 49.79 | 15.71 | 83.35 | 77 | 37.76 | 78.51 | 43.66 | 89.22 |
| 28 | 23.96 | 49.82 | 15.71 | 84.05 | 78 | 38.34 | 78.86 | 45.42 | 89.27 |
| 29 | 24.55 | 51.15 | 16.17 | 84.26 | 79 | 38.47 | 78.90 | 45.84 | 89.27 |
| 30 | 24.59 | 51.17 | 16.18 | 84.89 | 80 | 38.62 | 78.97 | 46.13 | 89.28 |
| 31 | 24.78 | 51.20 | 16.73 | 84.89 | 81 | 38.69 | 79.07 | 46.64 | 89.29 |
| 32 | 24.83 | 51.20 | 17.02 | 84.89 | 82 | 38.74 | 79.08 | 46.90 | 89.29 |
| 33 | 24.84 | 51.21 | 17.39 | 84.90 | 83 | 39.85 | 79.34 | 48.09 | 89.35 |
| 34 | 25.21 | 51.94 | 17.74 | 84.92 | 84 | 43.06 | 80.39 | 51.19 | 89.72 |
| 35 | 25.45 | 51.95 | 17.74 | 85.74 | 85 | 44.13 | 81.62 | 51.56 | 89.75 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 36 | 25.56 | 52.10 | 19.79 | 85.74 | 86 | 45.11 | 82.37 | 51.71 | 90.25 |
| 37 | 26.31 | 52.12 | 21.06 | 85.87 | 87 | 46.05 | 83.59 | 51.88 | 90.65 |
| 38 | 26.38 | 52.13 | 21.41 | 85.87 | 88 | 46.17 | 84.43 | 51.89 | 90.66 |
| 39 | 26.51 | 55.02 | 21.42 | 85.96 | 89 | 46.22 | 84.98 | 51.95 | 90.70 |
| 40 | 26.57 | 55.03 | 21.76 | 85.96 | 90 | 55.48 | 88.08 | 53.74 | 91.58 |
| 41 | 26.67 | 55.05 | 21.77 | 86.20 | 91 | 60.05 | 89.29 | 57.09 | 92.13 |
| 42 | 26.71 | 55.83 | 21.77 | 86.21 | 92 | 60.07 | 89.32 | 57.47 | 92.13 |
| 43 | 26.84 | 60.36 | 21.87 | 86.21 | 93 | 60.22 | 89.36 | 58.31 | 92.13 |
| 44 | 27.31 | 60.72 | 21.98 | 86.35 | 94 | 60.23 | 89.37 | 58.80 | 92.13 |
| 45 | 27.33 | 68.33 | 21.98 | 86.35 | 95 | 60.24 | 89.40 | 59.16 | 92.13 |
| 46 | 28.15 | 68.60 | 22.64 | 86.42 | 96 | 60.43 | 89.42 | 60.07 | 92.14 |
| 47 | 28.50 | 70.33 | 22.65 | 86.59 | 97 | 60.91 | 89.51 | 61.46 | 92.16 |
| 48 | 28.56 | 70.66 | 23.50 | 86.70 | 98 | 65.77 | 90.43 | 65.06 | 92.37 |
| 49 | 28.78 | 71.88 | 24.25 | 86.71 | 99 | 66.55 | 90.66 | 66.60 | 92.45 |
| 50 | 31.22 | 72.48 | 25.38 | 86.92 | 100 | 70.75 | 91.14 | 68.56 | 92.90 |

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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)

| Mode # | Frequency (Hz) | Mass Participation Factors (%) | | | Mode # | Frequency (Hz) | Mass Participation Factors (%) | | |
|---|---|---|---|---|---|---|---|---|---|
| | | N-S | Vertical | E-W | | | N-S | Vertical | E-W |
| 1 | 8.[illegible]2 | 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 | [illegible].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 | 4.4[illegible] | 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.[illegible]5 | 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 | 60.4[illegible] | 0 | 0.37 | 0.16 |
| 26 | 30.12 | 0 | 0 | 0.39 | 76 | 61.40 | 0.03 | 0.16 | 0.29 |
| 27 | 31.13 | 0.0[illegible] | 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.[illegible]7 | 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.[illegible]9 | 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 |

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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 |
| 36 | 34.[illegible]4 | 0.03 | 0.02 | 0.40 | 86 | 71.98 | 0.03 | 0.02 | 0.35 |
| 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 | 0.2[illegible] | 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.[illegible]4 | 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 |

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**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)

| 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.[illegible]7 | 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 | 0.1[illegible] | 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 | 39.1[illegible] | 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.[illegible]2 | 0 | 75 | 50.[illegible]4 | 0.01 | 0.33 | 0.19 |
| 26 | 22.79 | 0.36 | 0.27 | 0.01 | 76 | 51.3[illegible] | 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 | 0.4[illegible] | 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.[illegible]4 | 0.53 | 0.02 | 0.03 | 83 | 56.80 | 0 | 0.01 | 0.76 |
| 34 | [illegible]8.33 | 0.13 | 0.32 | 0.07 | 84 | 60.46 | 0.29 | 0.04 | 0.23 |

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**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)

| Mode # | Frequency (Hz) | Mass Participation Factors (%) | | | Mode # | Frequency (Hz) | Mass Participation Factors (%) | | |
|---|---|---|---|---|---|---|---|---|---|
| | | N-S | Vertical | E-W | | | N-S | Vertical | E-W |
| 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 |
| 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 | [illegible] | 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 |

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**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 | | | | | |
|---|---|---|---|---|---|---|---|
| | | $U_x$ | $U_y$ | $U_z$ | $\Phi_x$ | $\Phi_y$ | $\Phi_z$ |
| 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 $\varphi_x$, $\varphi_y$ and $\varphi_z$ are the rotations.

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**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~~22.5~~ | 0.179g~~0.225g~~ | 0.16g~~0.147g~~ | 0.196g~~0.233g~~ |
| 11.5~~11.5~~ | 0.22g~~0.315g~~ | 0.201g~~0.199g~~ | 0.208g~~0.238g~~ |
| 26.5~~26.5~~ | 0.247g~~0.342g~~ | 0.225g~~0.236g~~ | 0.211g~~0.240g~~ |
| **Forebay** | | | |
| **Floor Elevation** | **X (N-S) Direction** | **Y (E-W) Direction** | **Z (Vert) Direction** |
| -22.5~~-22.5~~ | 0.199g~~0.227g~~ | 0.173g~~0.153g~~ | 0.249g~~0.215g~~ |

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

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

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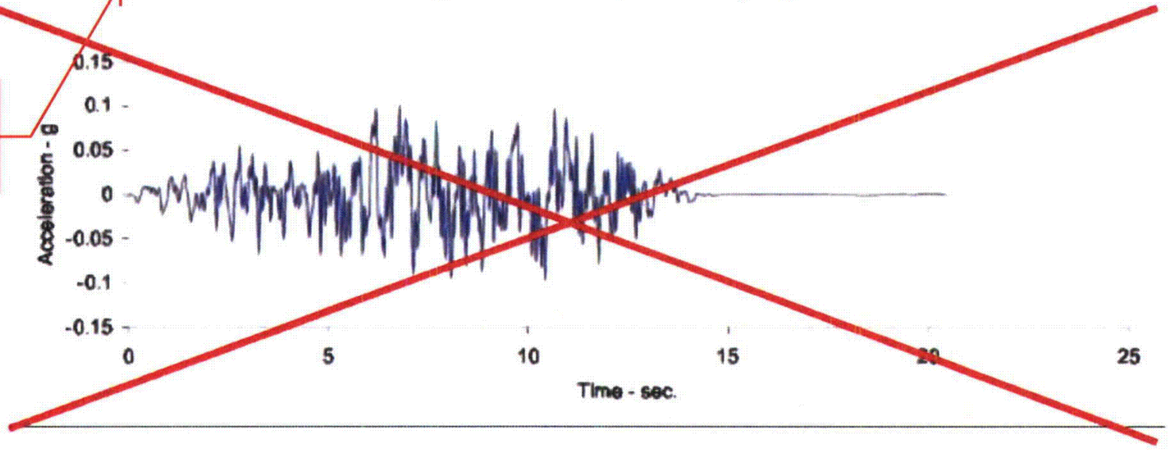


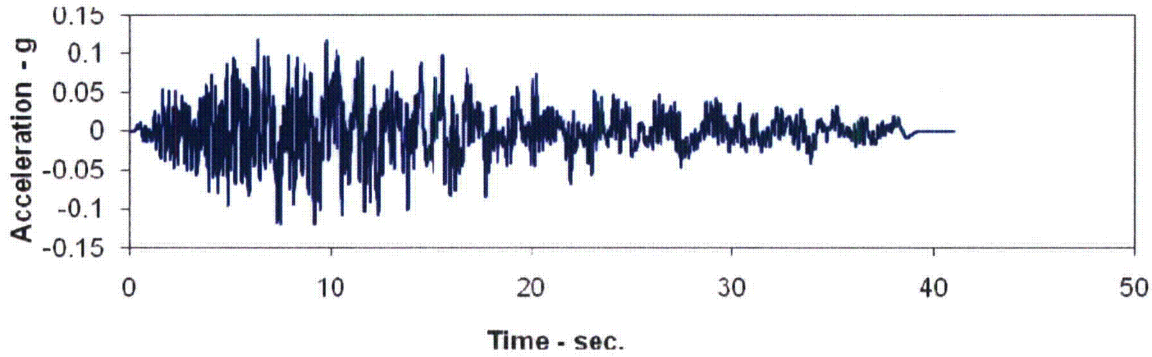


a) Horizontal Direction S1

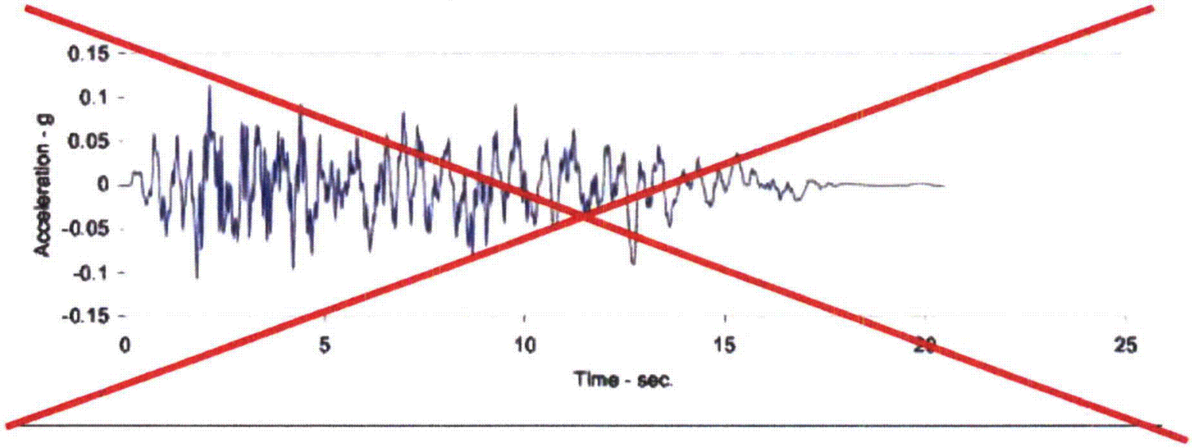

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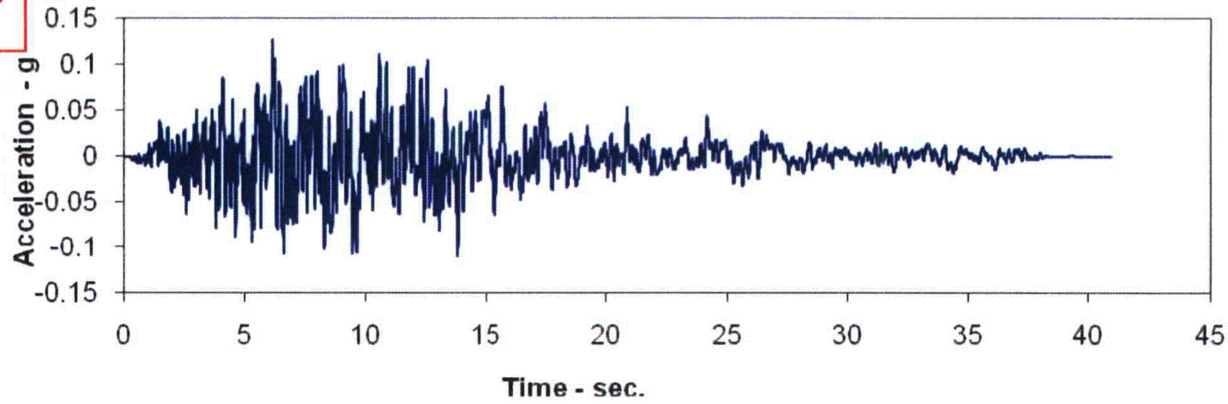
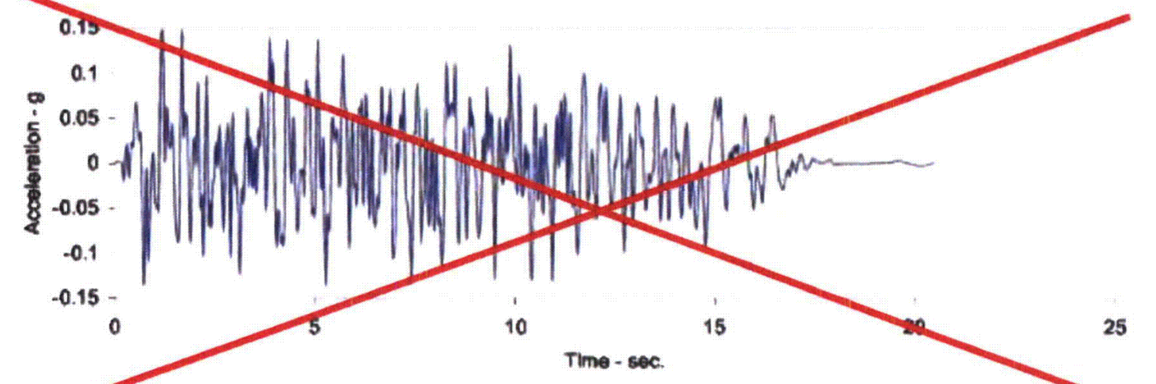
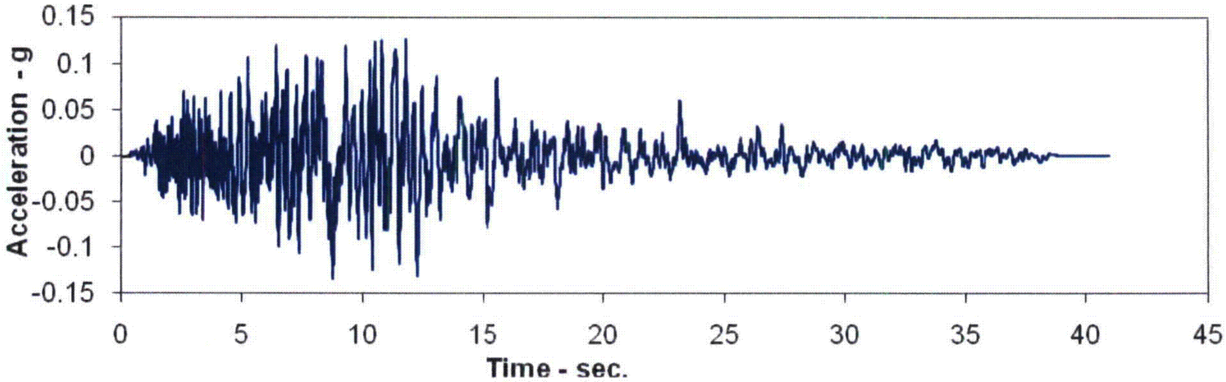
**Figure 3.7-16— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (LB Soil Case)- Intake Area (37.5 ft Depth)}**




b) Horizontal Direction S2




c) Vertical Direction S3

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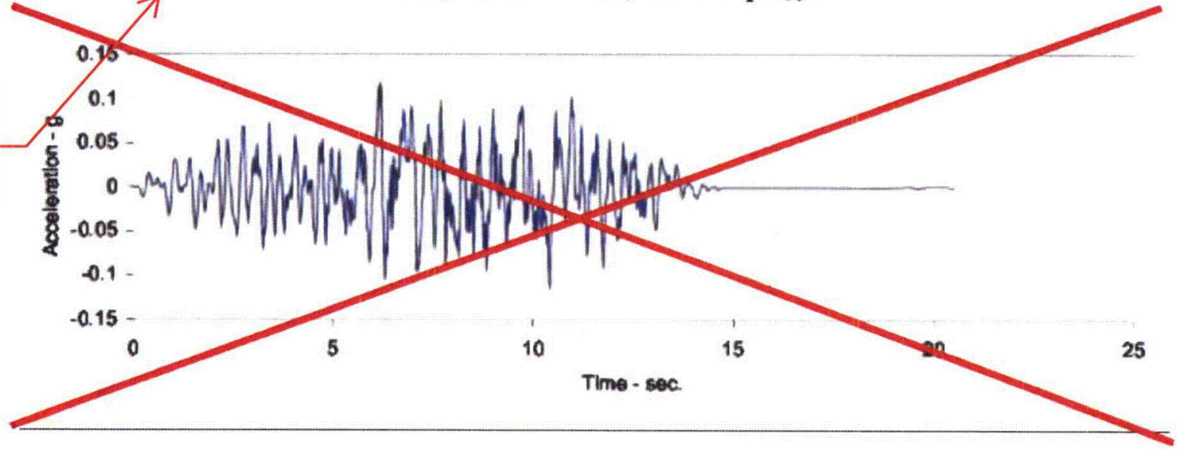
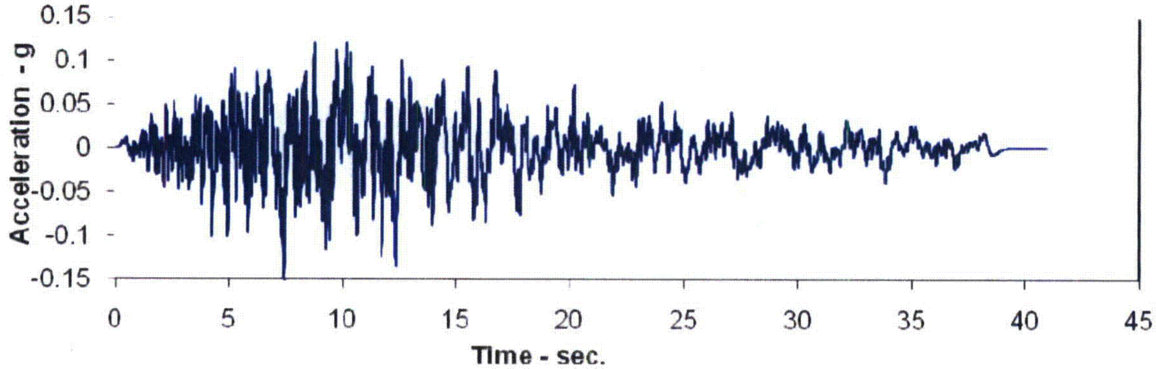
**Figure 3.7-17— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (BE Soil Case)- Intake Area (37.5 ft Depth)}**




a) Horizontal Direction S1

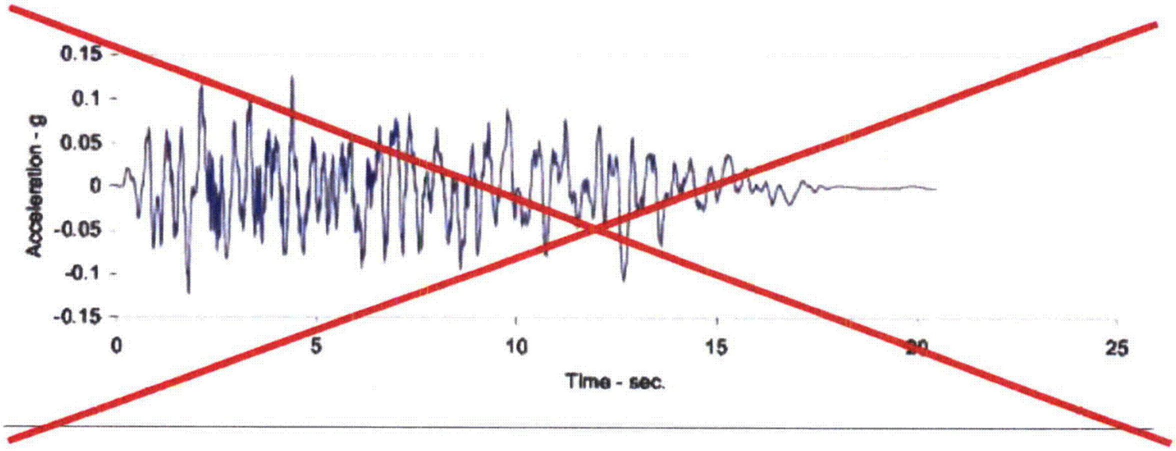


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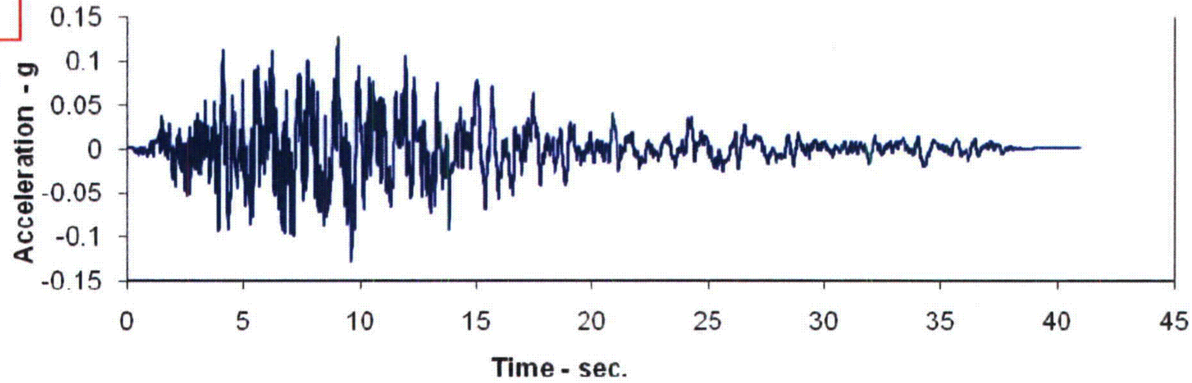
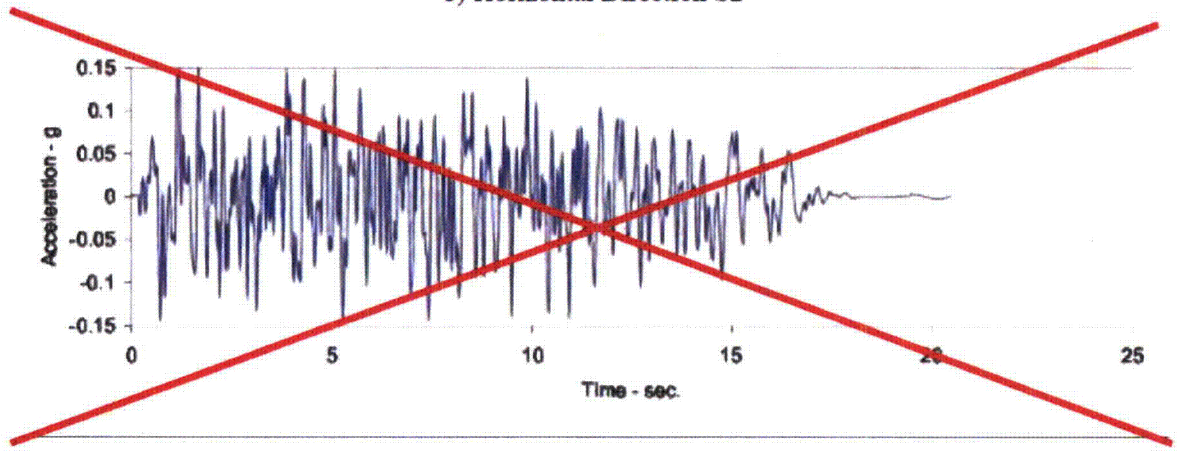
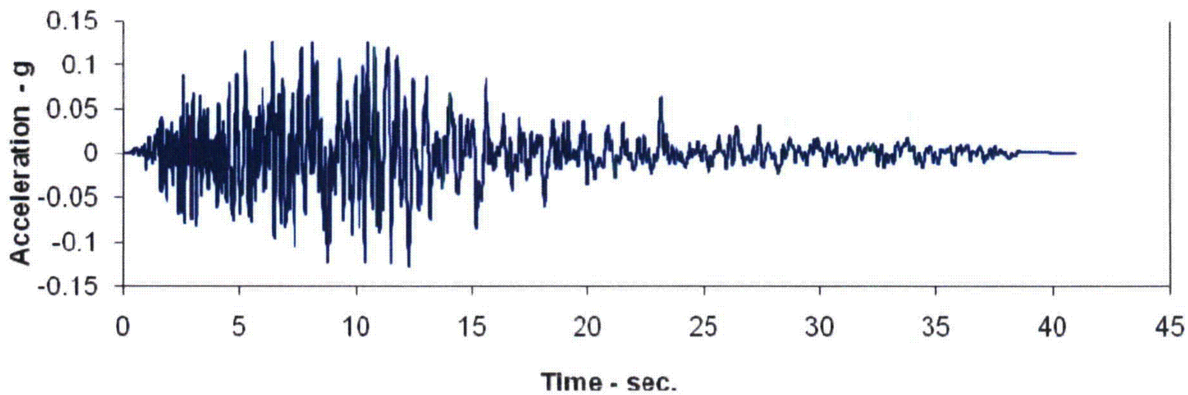
**Figure 3.7-17— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (BE Soil Case)- Intake Area (37.5 ft Depth)}**




b) Horizontal Direction S2




c) Vertical Direction S3

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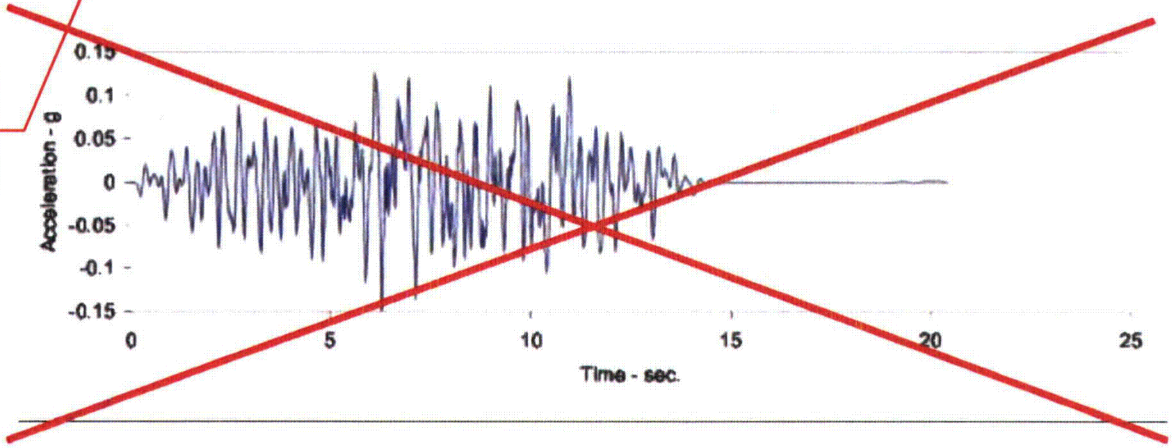
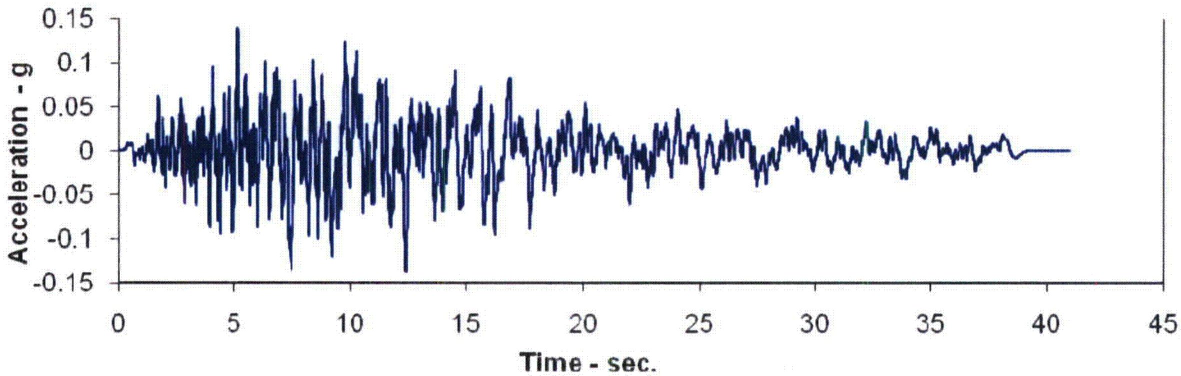
**Figure 3.7-18— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (UB Soil Case)- Intake Area (37.5 ft Depth)}**




a) Horizontal Direction S1

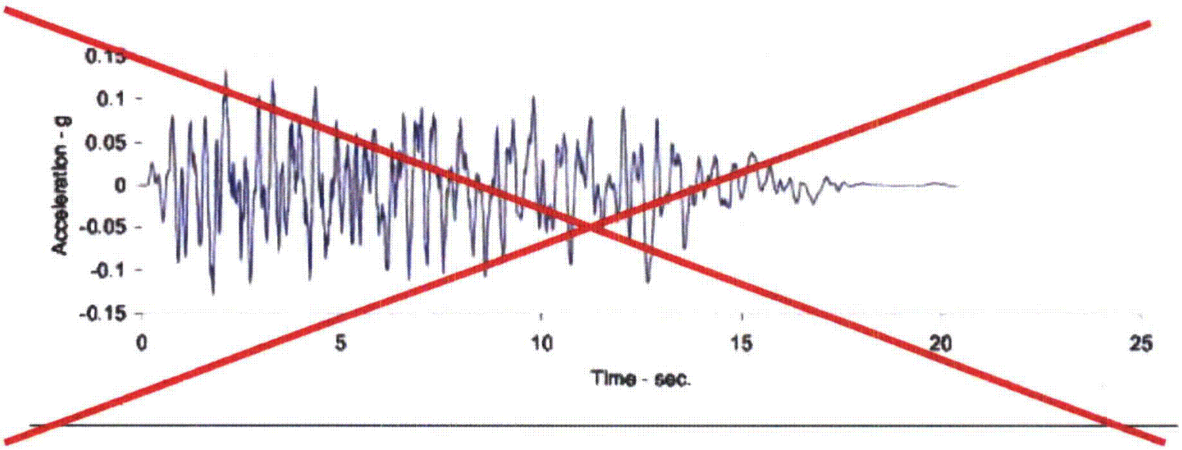


**Figure 3.7-18— {SSI "Within" Acceleration Time Histories for Input at CBIS Foundation (UB Soil Case)- Intake Area (37.5 ft Depth)}**

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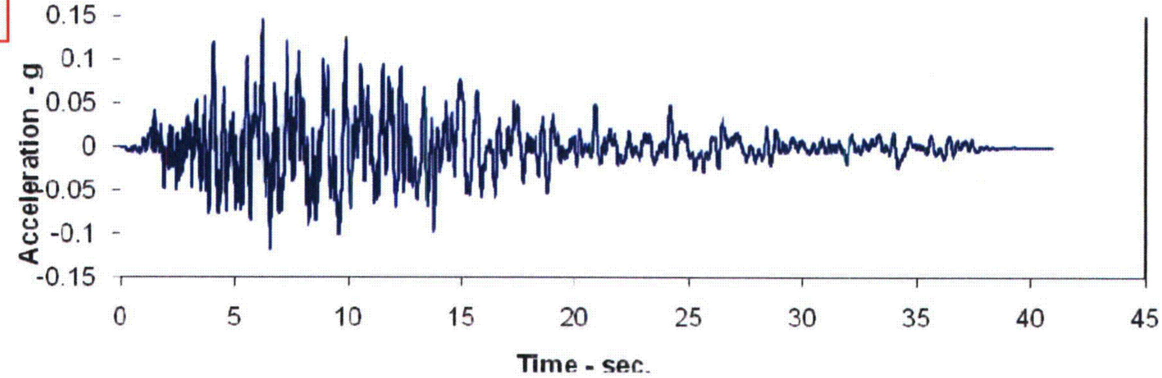
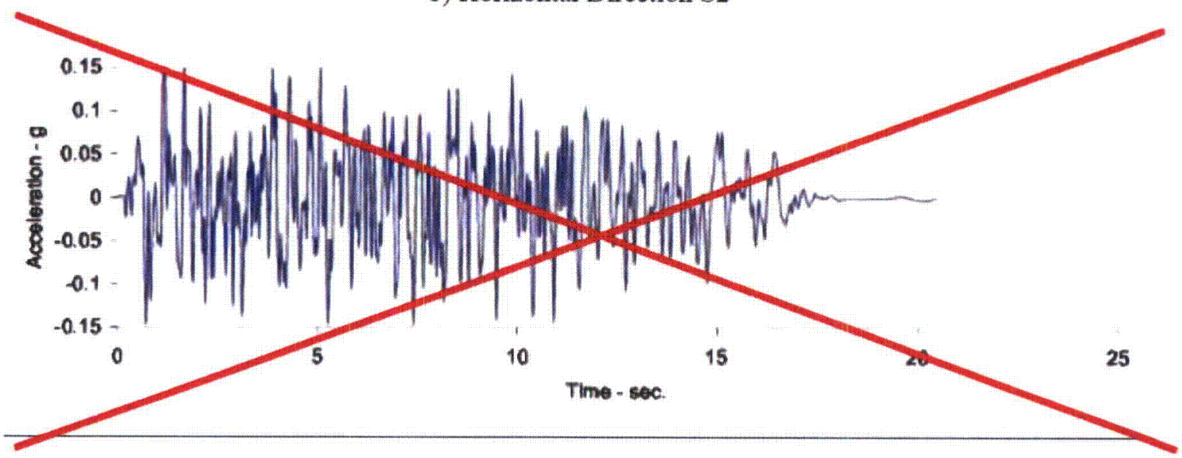
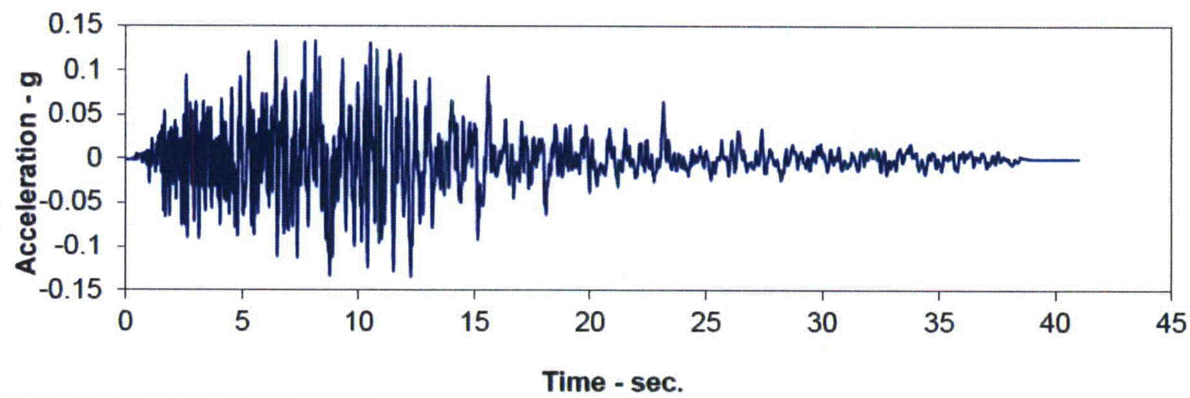



b) Horizontal Direction S2




c) Vertical Direction S3

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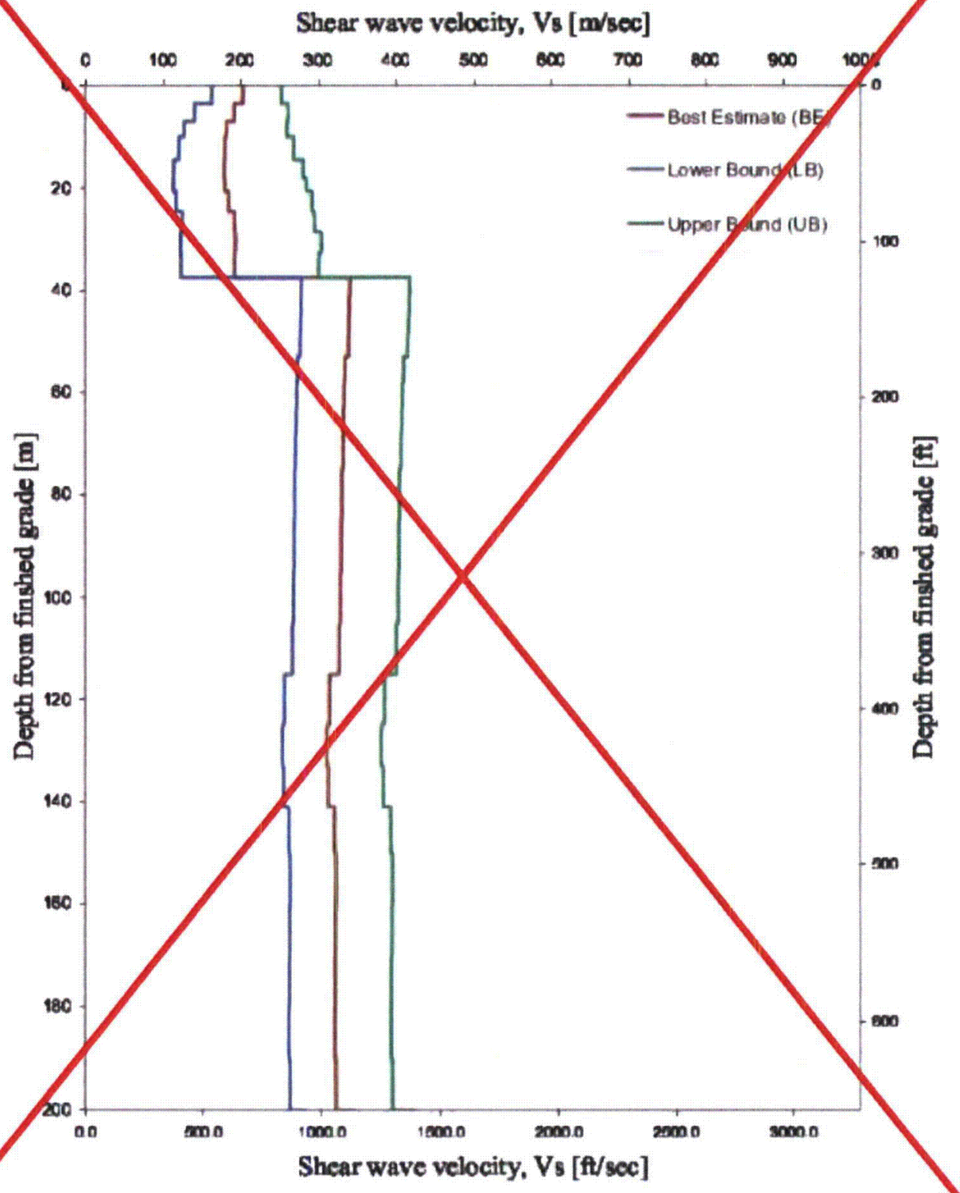
**Figure 3.7-22— {CCNPP Unit 3 Strain-Compatible profiles at the Intake Area for CBIS}**



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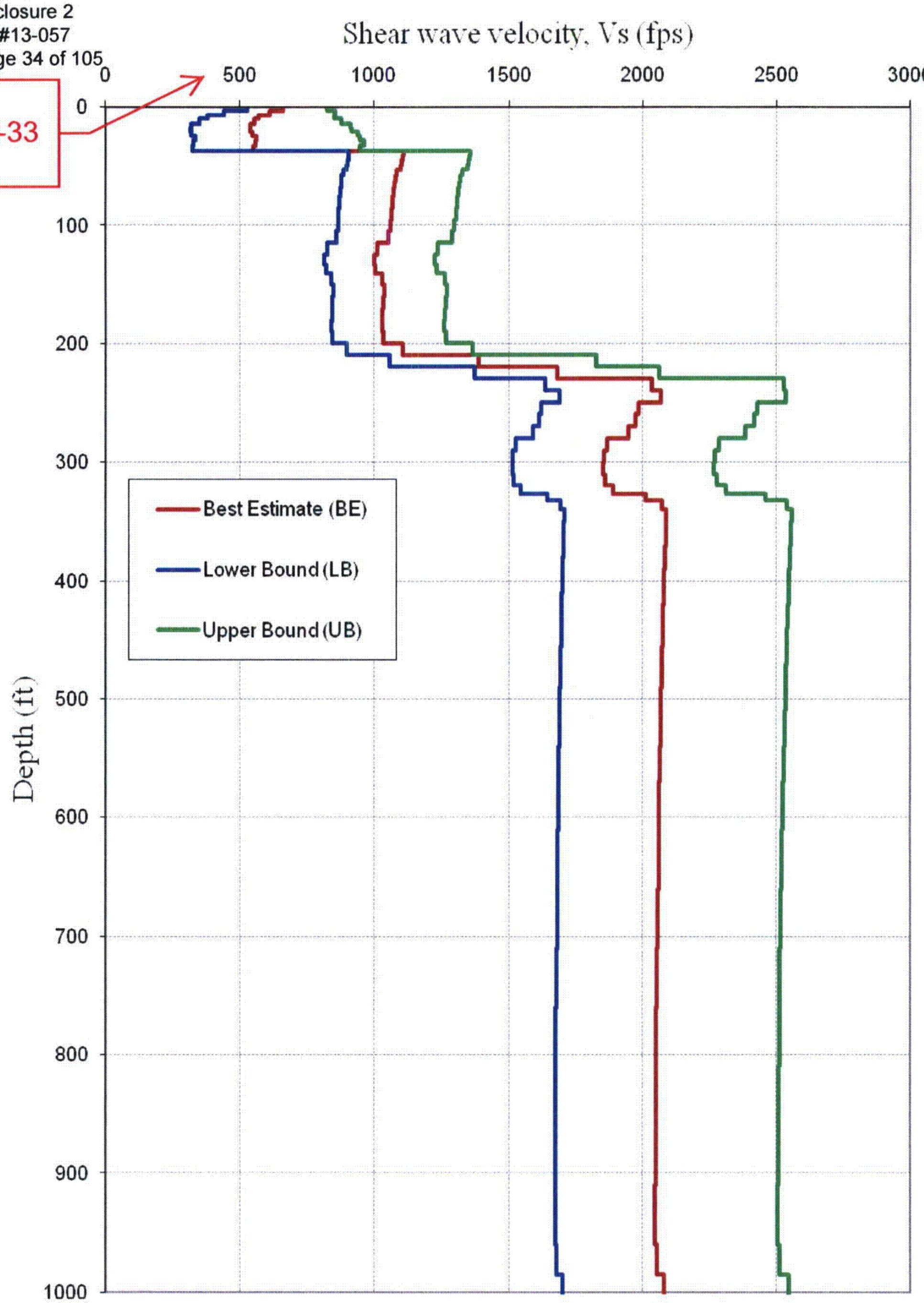


Insert for Figure 3.7-22

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

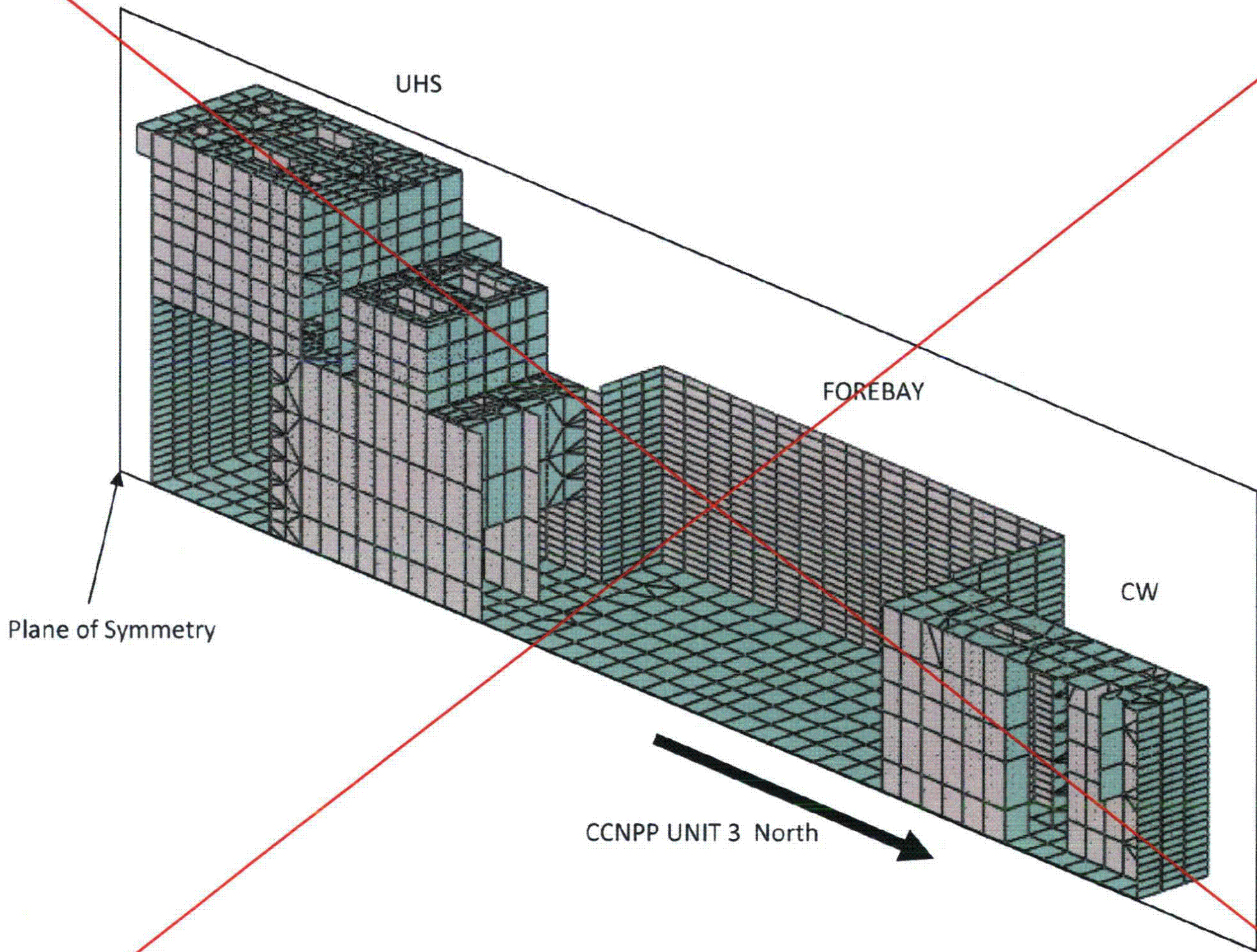

**Figure 3.7-23— {Isometric View of the Common Basemat Intake Structures}**

New Insert for
Figure 3.7-23

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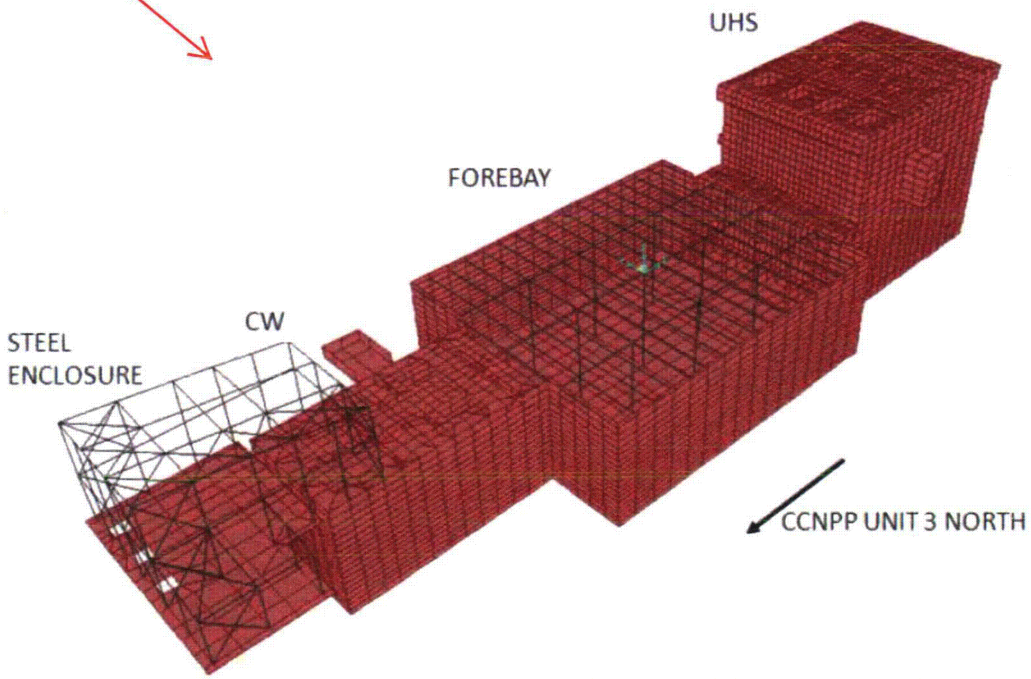
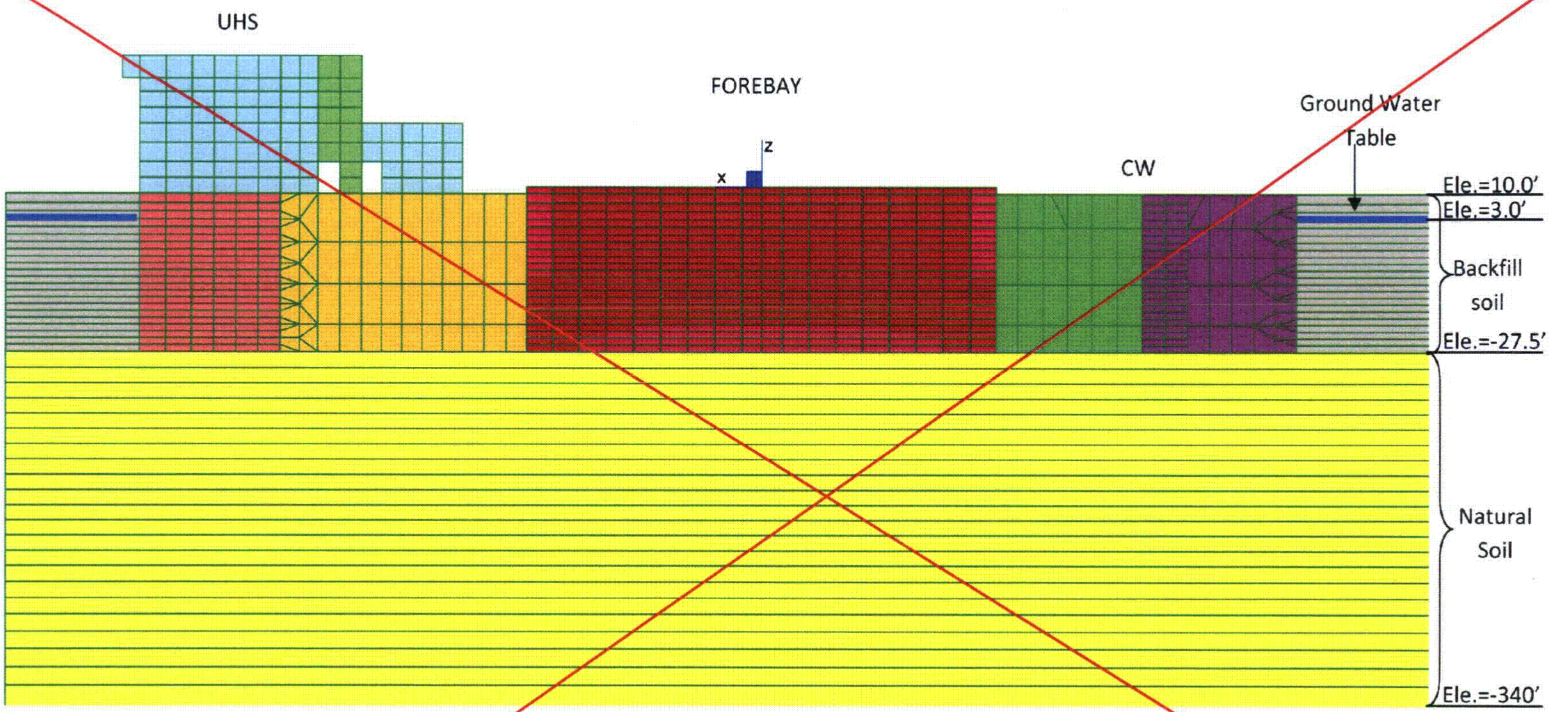


**Figure 3.7-24— {Soil-Structure Interaction (SSI) model for the Common Basemat Intake Structures (Elevations and plant coordinate system refer to CCNPP Unit 3)}**




The soil layering system shown is schematic.

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

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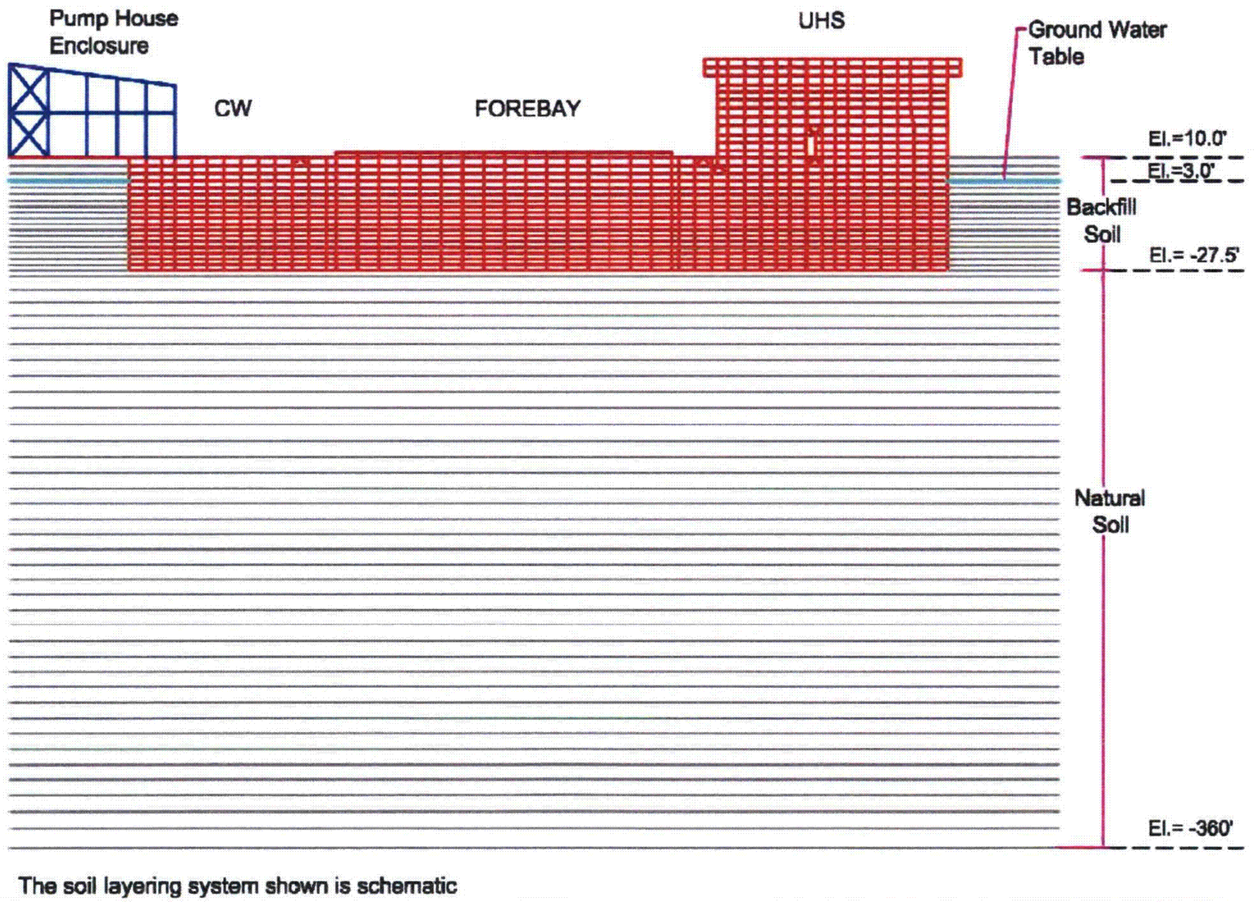


Insert for Figure
3.7-24

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}

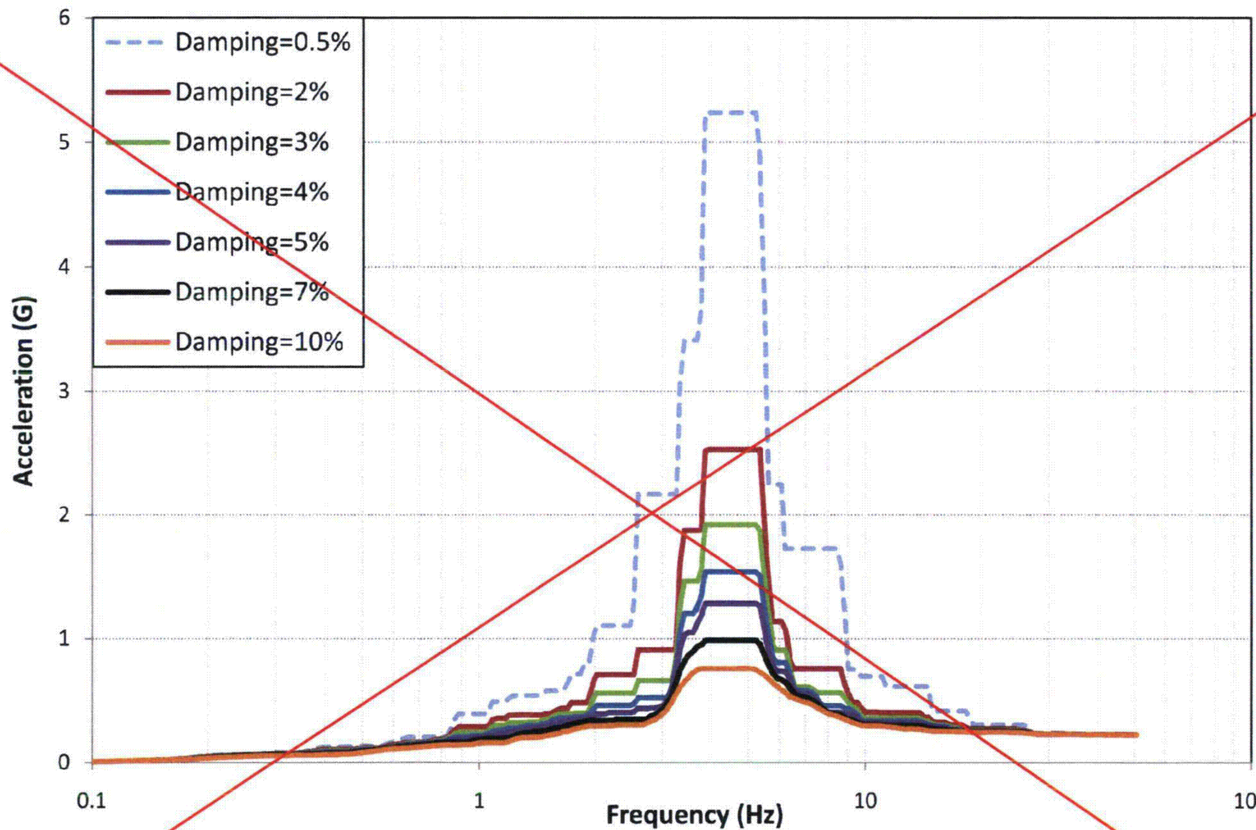

**Figure 3.7-73— {ISRS for UHS Makeup Water Intake Structure at location at Elev. -22.5 ft (-6.86 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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03.07.02-72

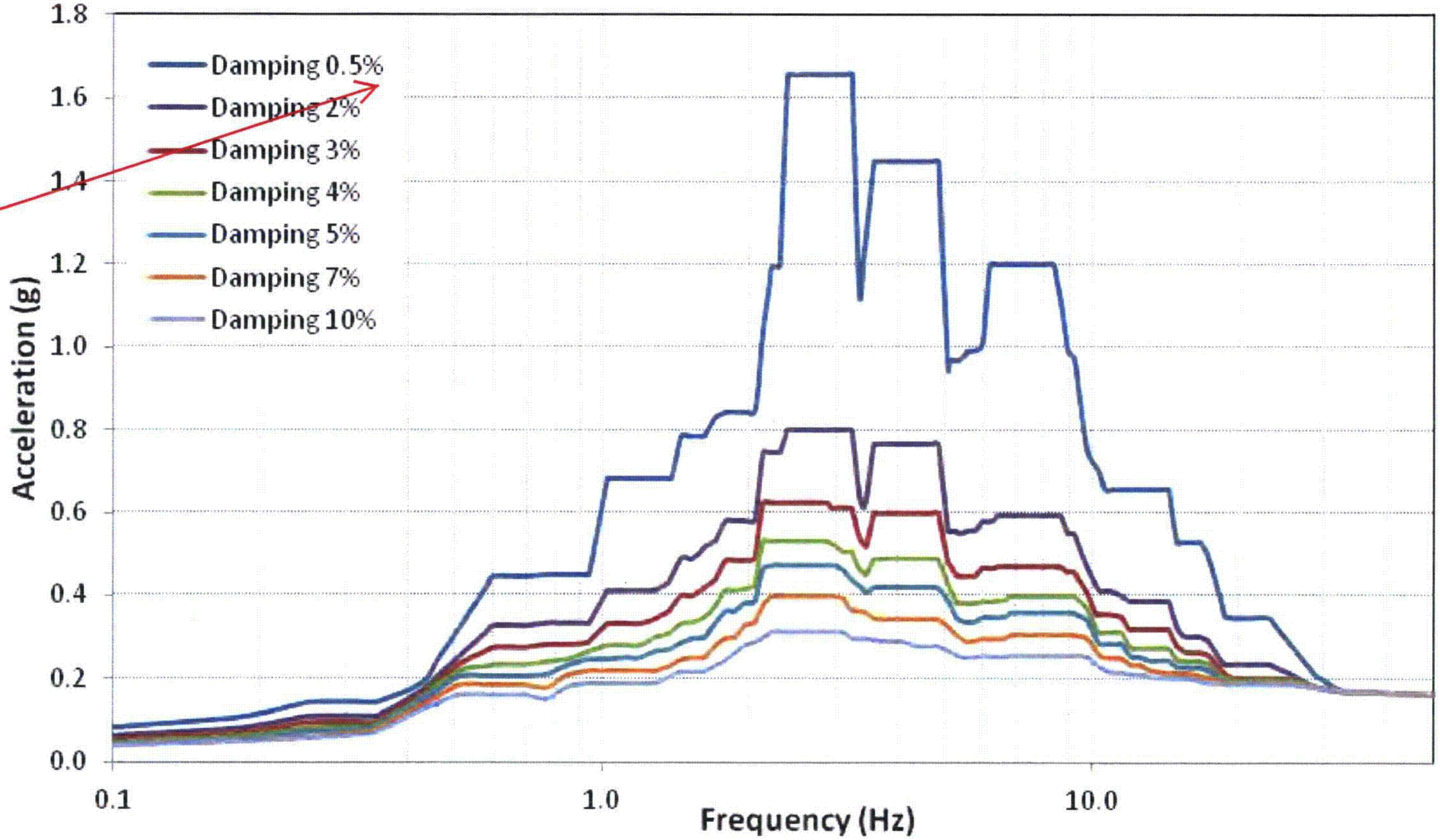

**Figure 3.7-74— {ISRS for UHS Makeup Water Intake Structure at Elev. -22.5 ft (-6.86 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3.}**

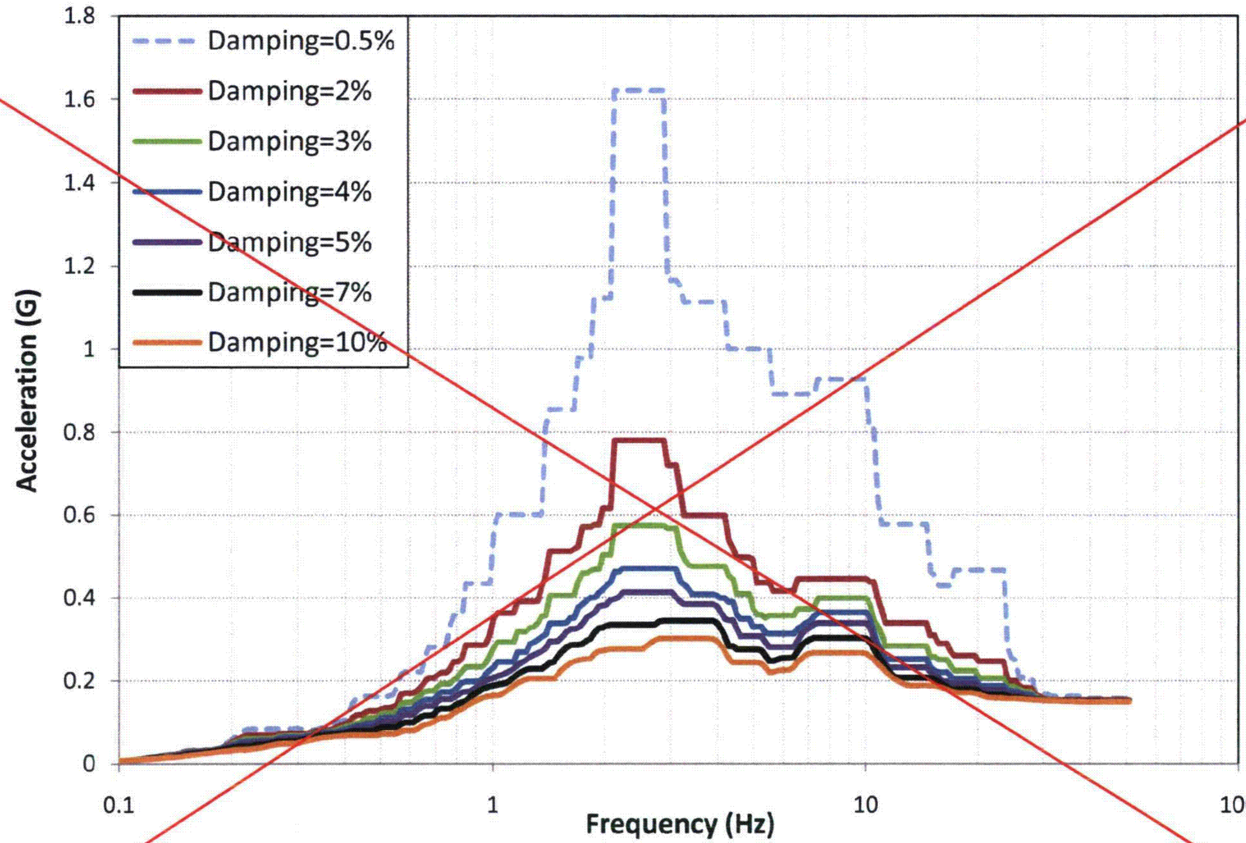

**Figure 3.7-74— {ISRS for UHS Makeup Water Intake Structure at Elev. -22.5 ft (-6.86 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3.}**

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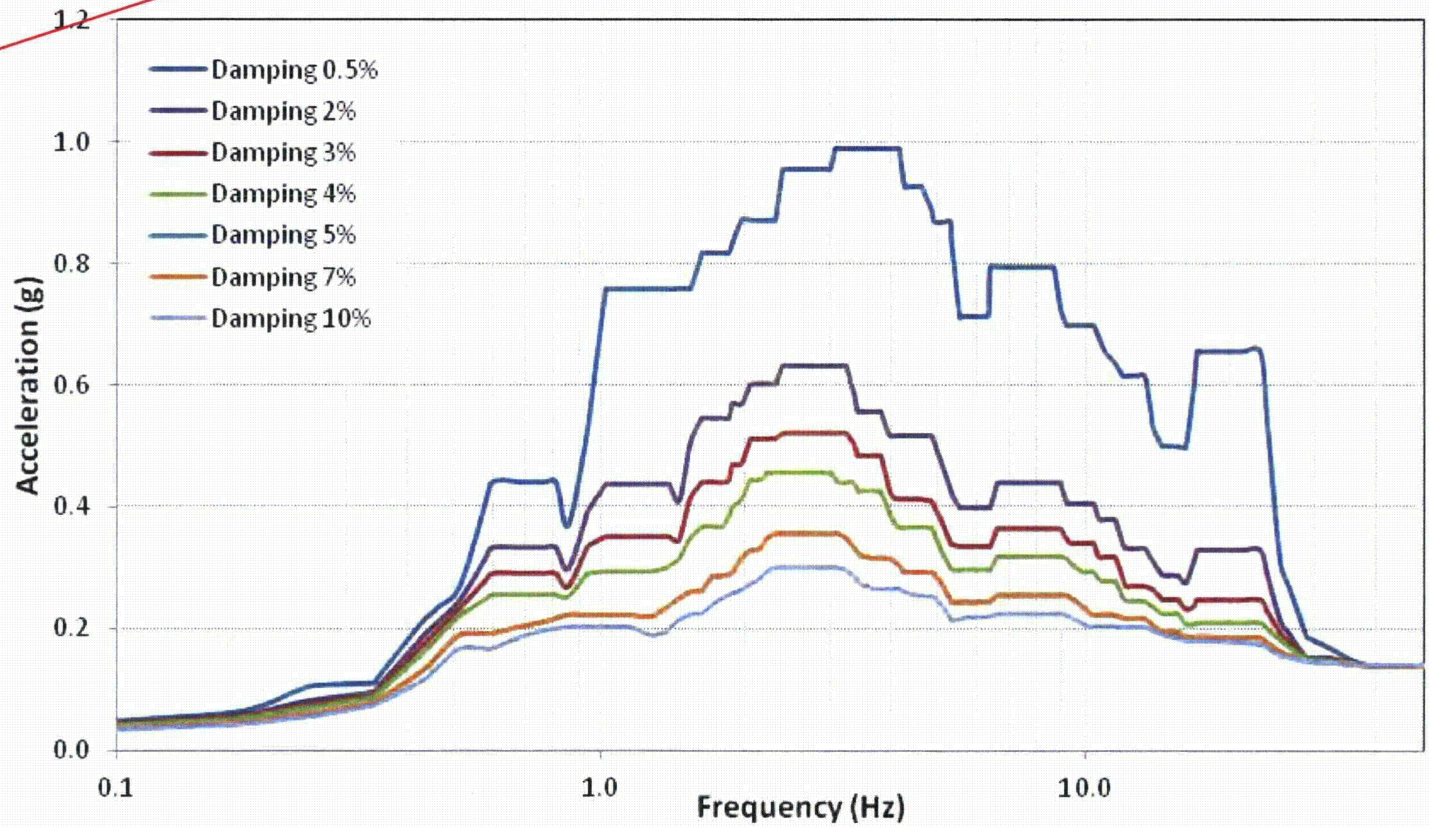

**Figure 3.7-75— {ISRS for UHS Makeup Water Intake Structure at Elev. -22.5 ft (-6.86 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

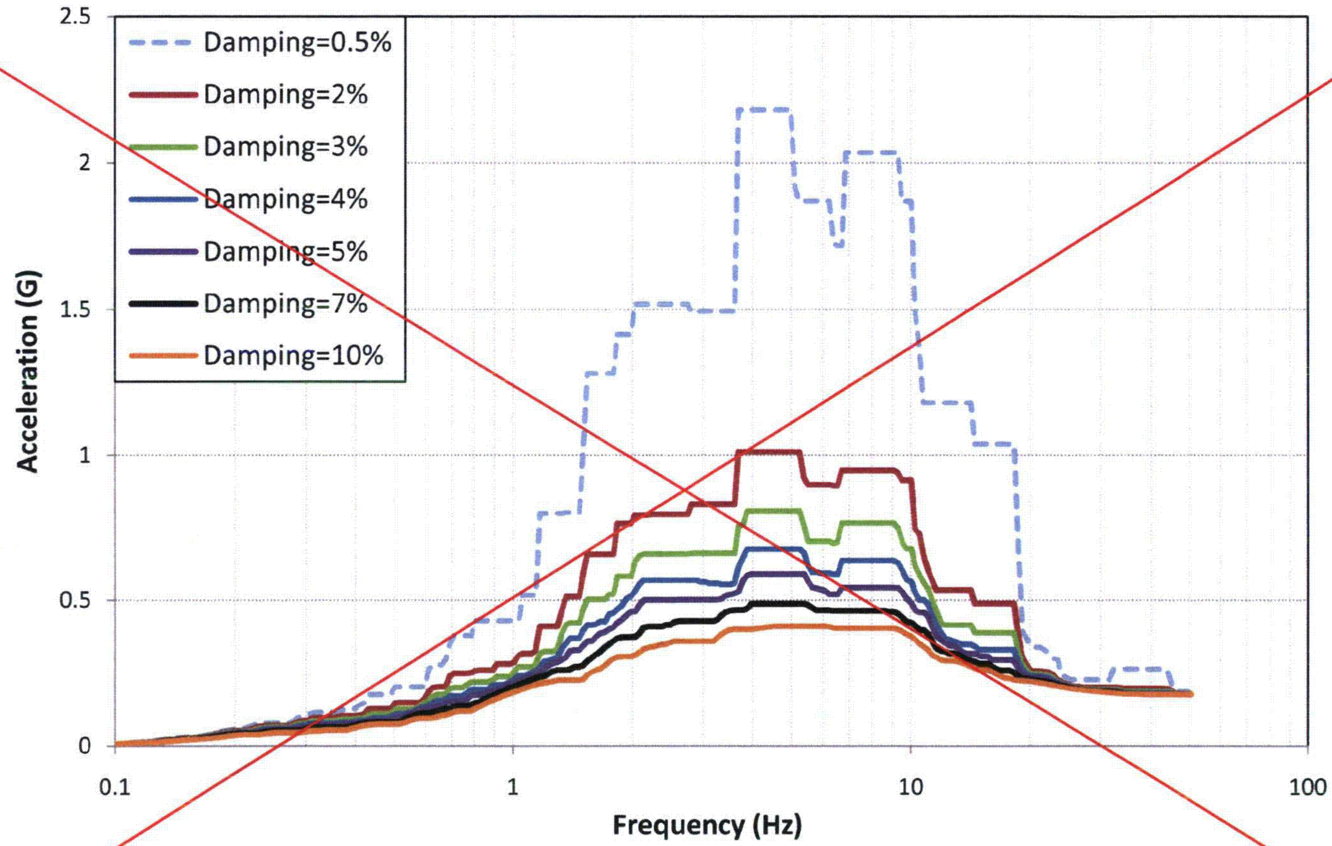

**Figure 3.7-75— {ISRS for UHS Makeup Water Intake Structure at Elev. -22.5 ft (-6.86 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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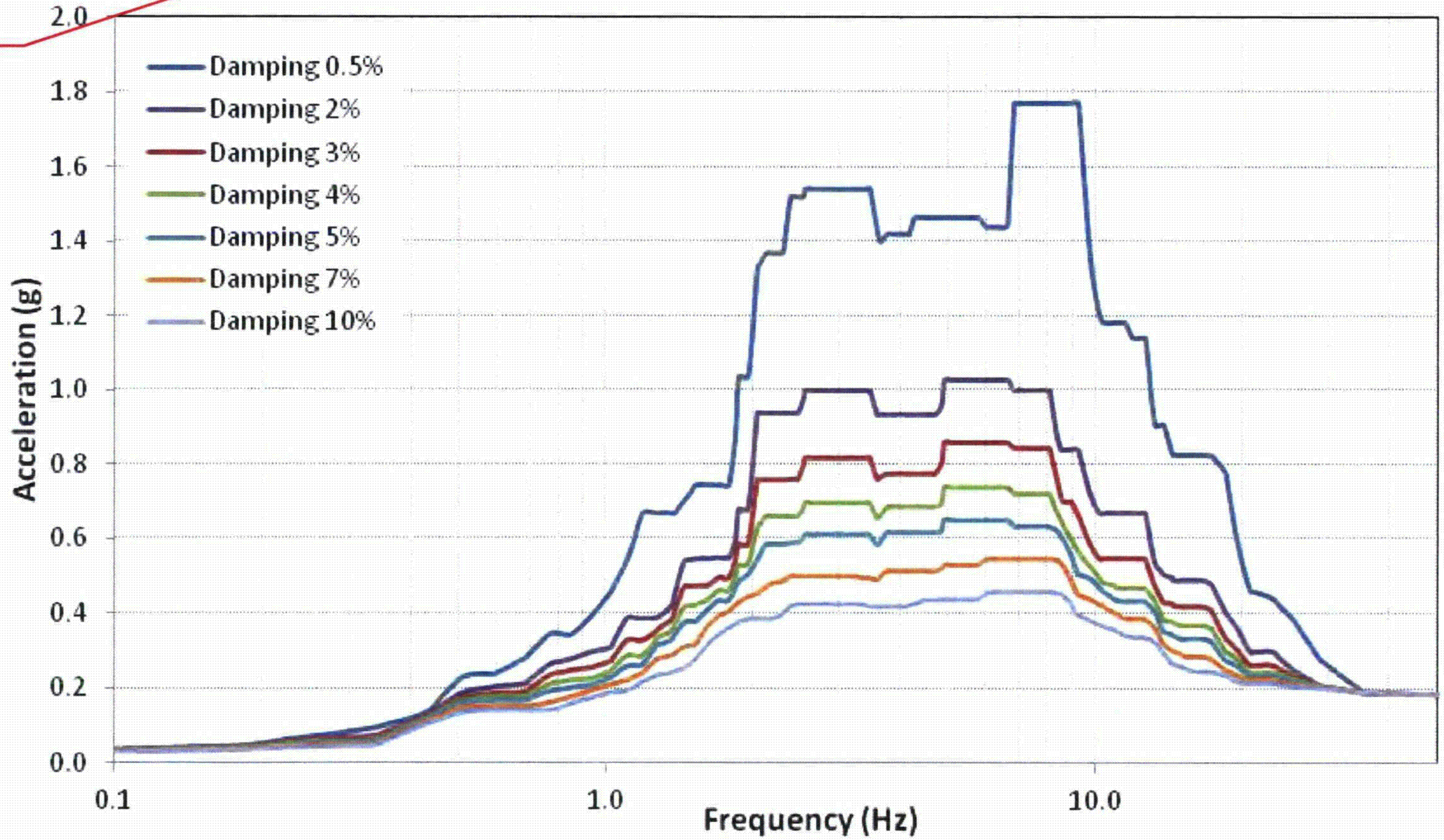

**Figure 3.7-76— {ISRS for UHS Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3.}**

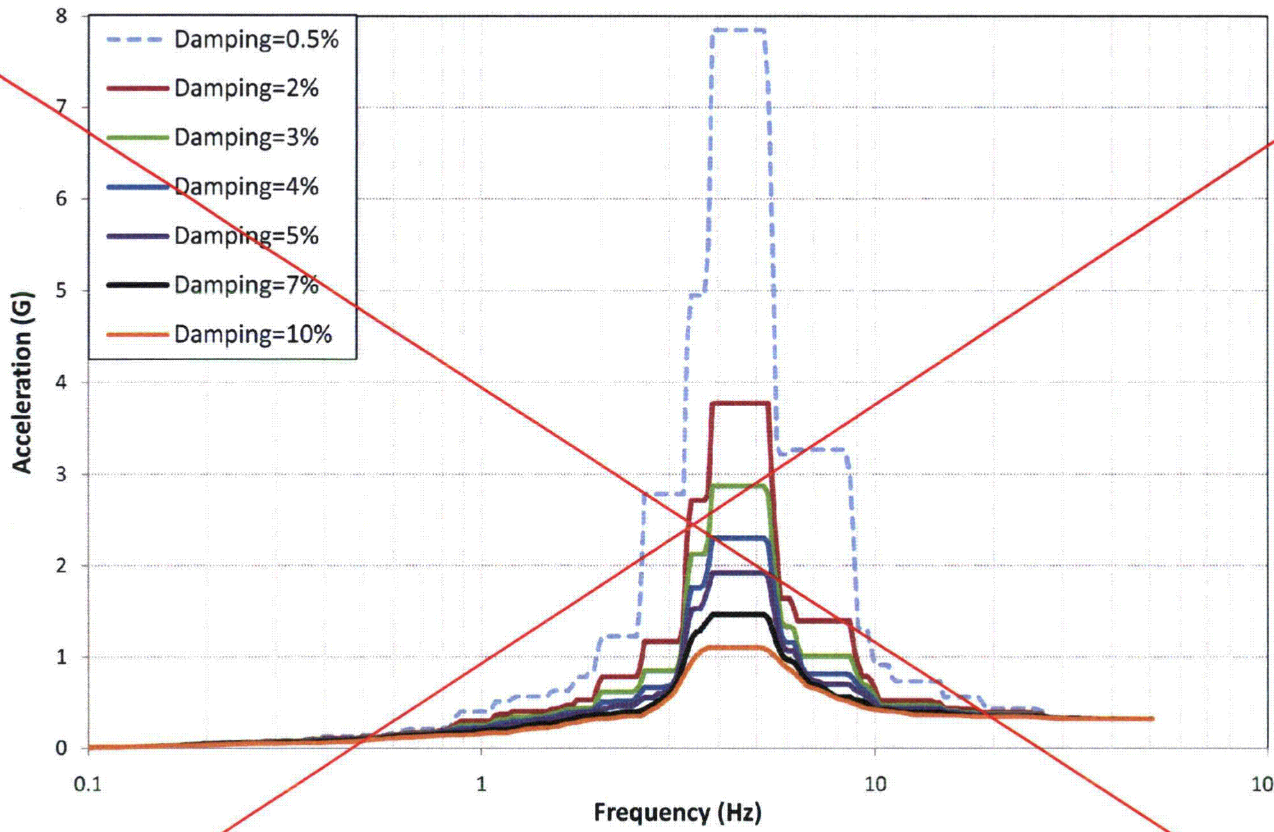

**Figure 3.7-76— {ISRS for UHS Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3.}**

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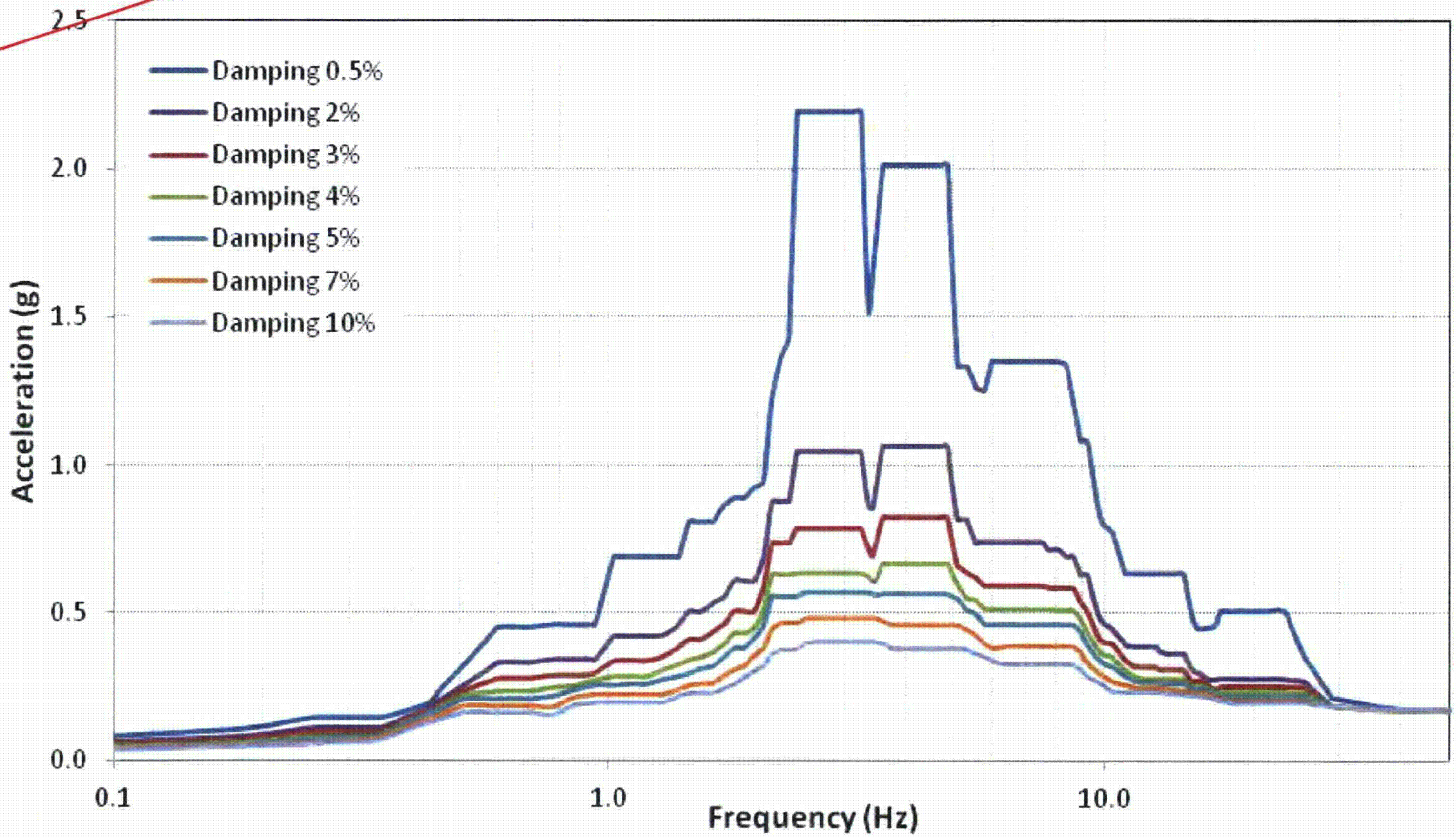

**Figure 3.7-77— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

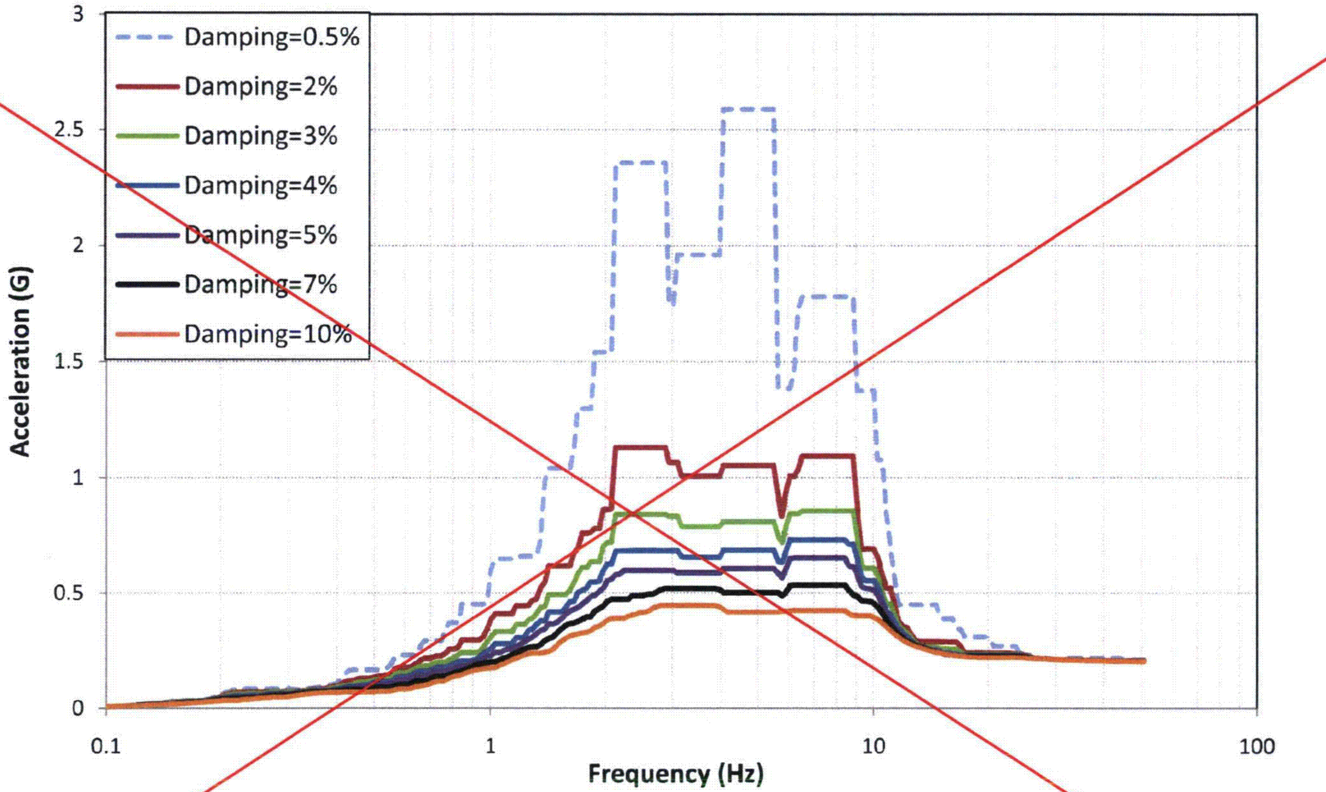

**Figure 3.7-77— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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03.07.02-72

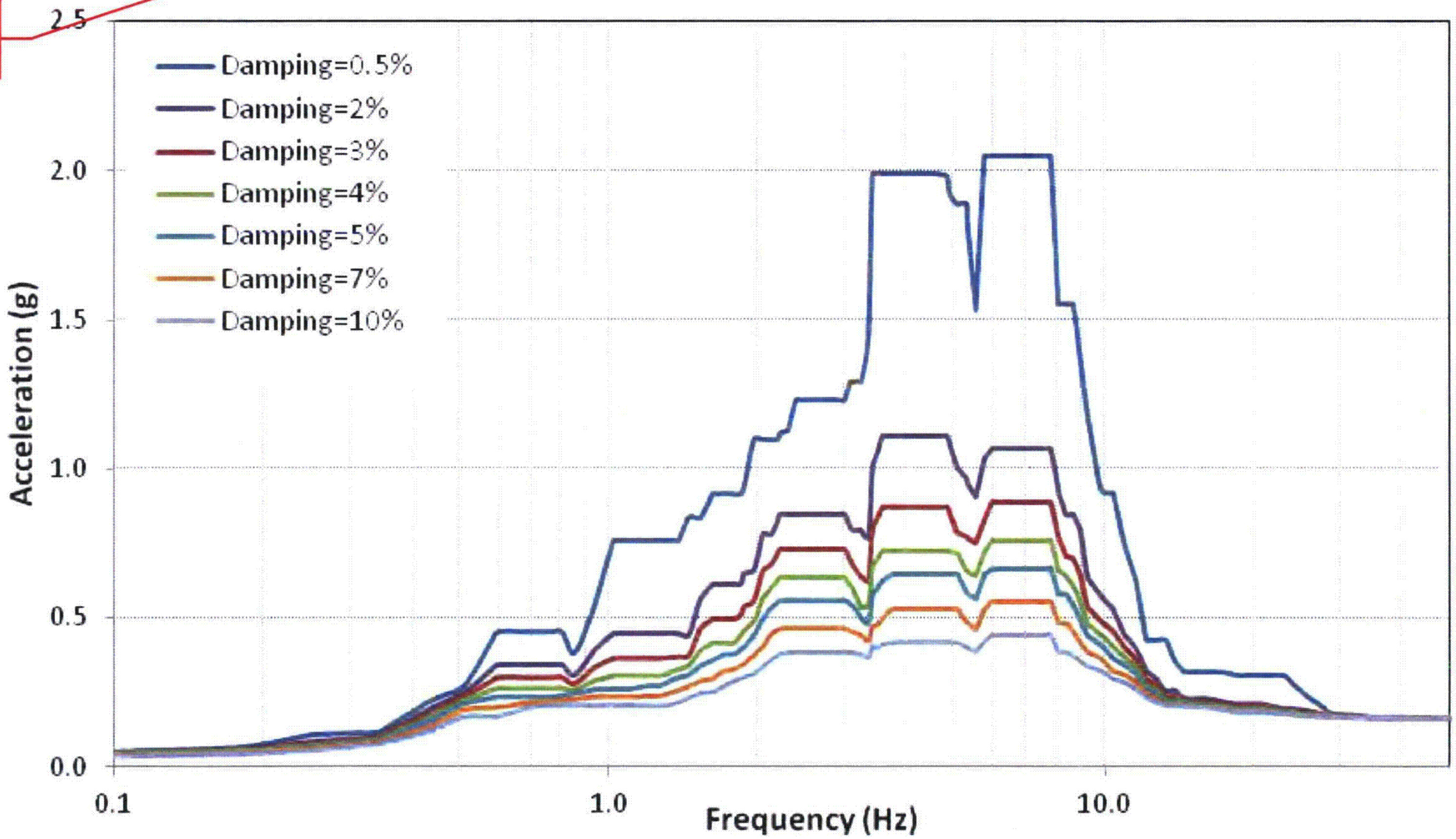

**Figure 3.7-78— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

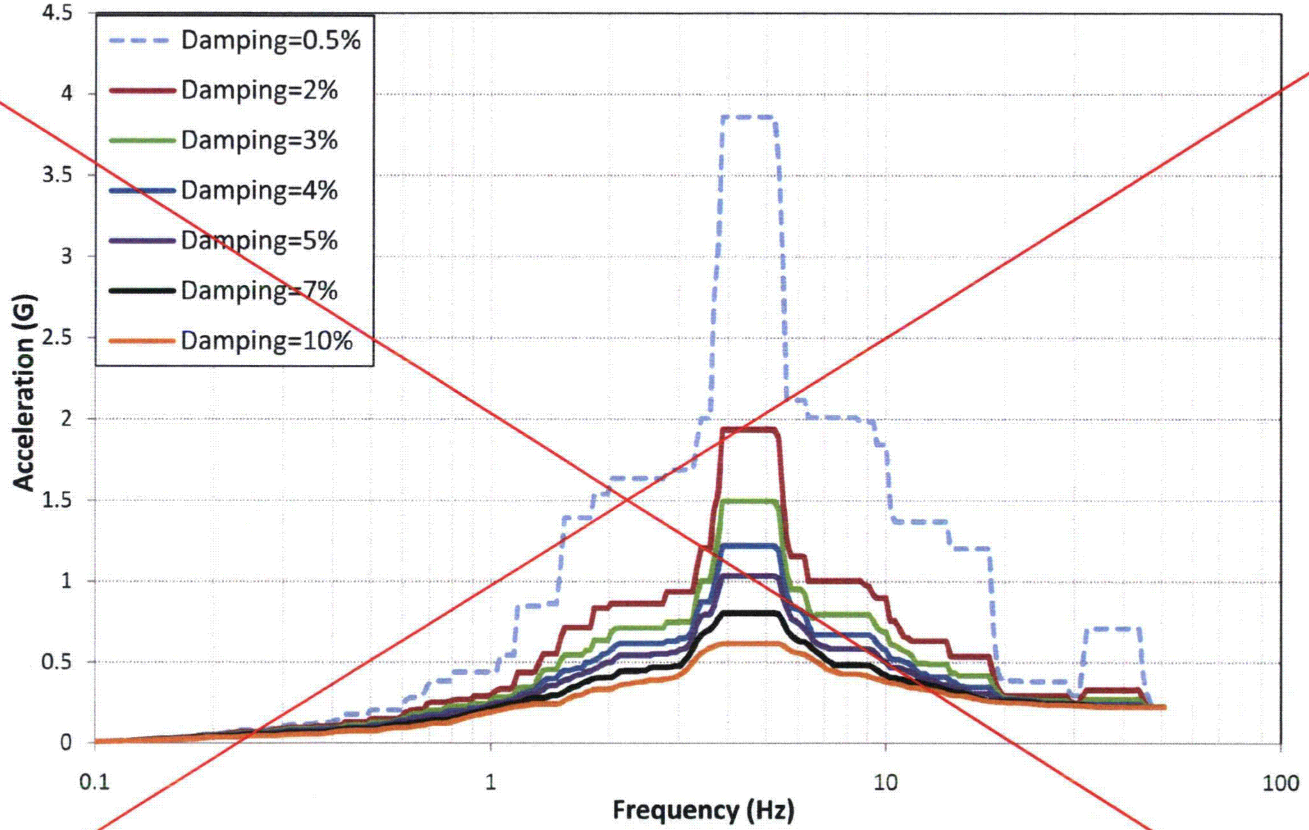

**Figure 3.7-78— {ISRS for Makeup Water Intake Structure at Elev. 11.5 ft (3.5 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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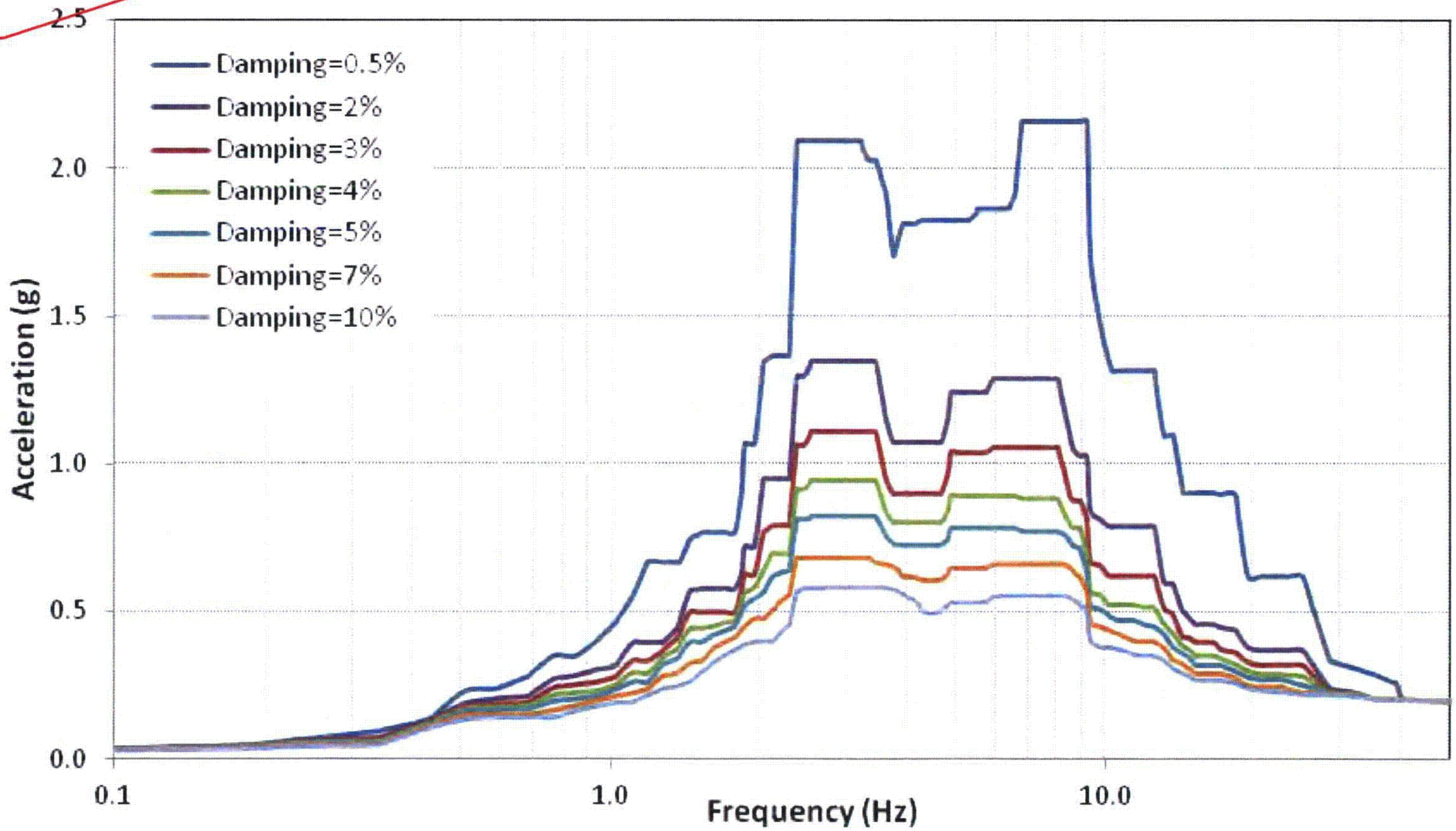

**Figure 3.7-79— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

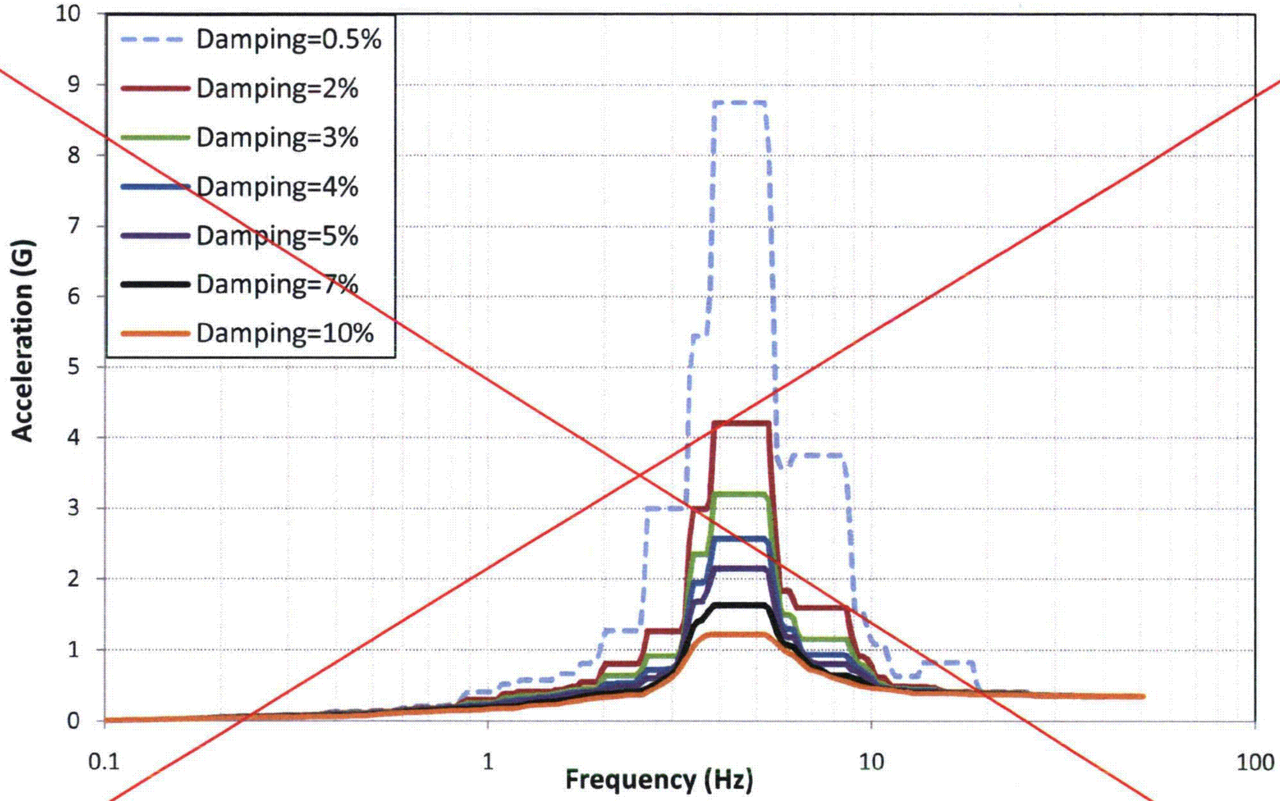

**Figure 3.7-79— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), North-South Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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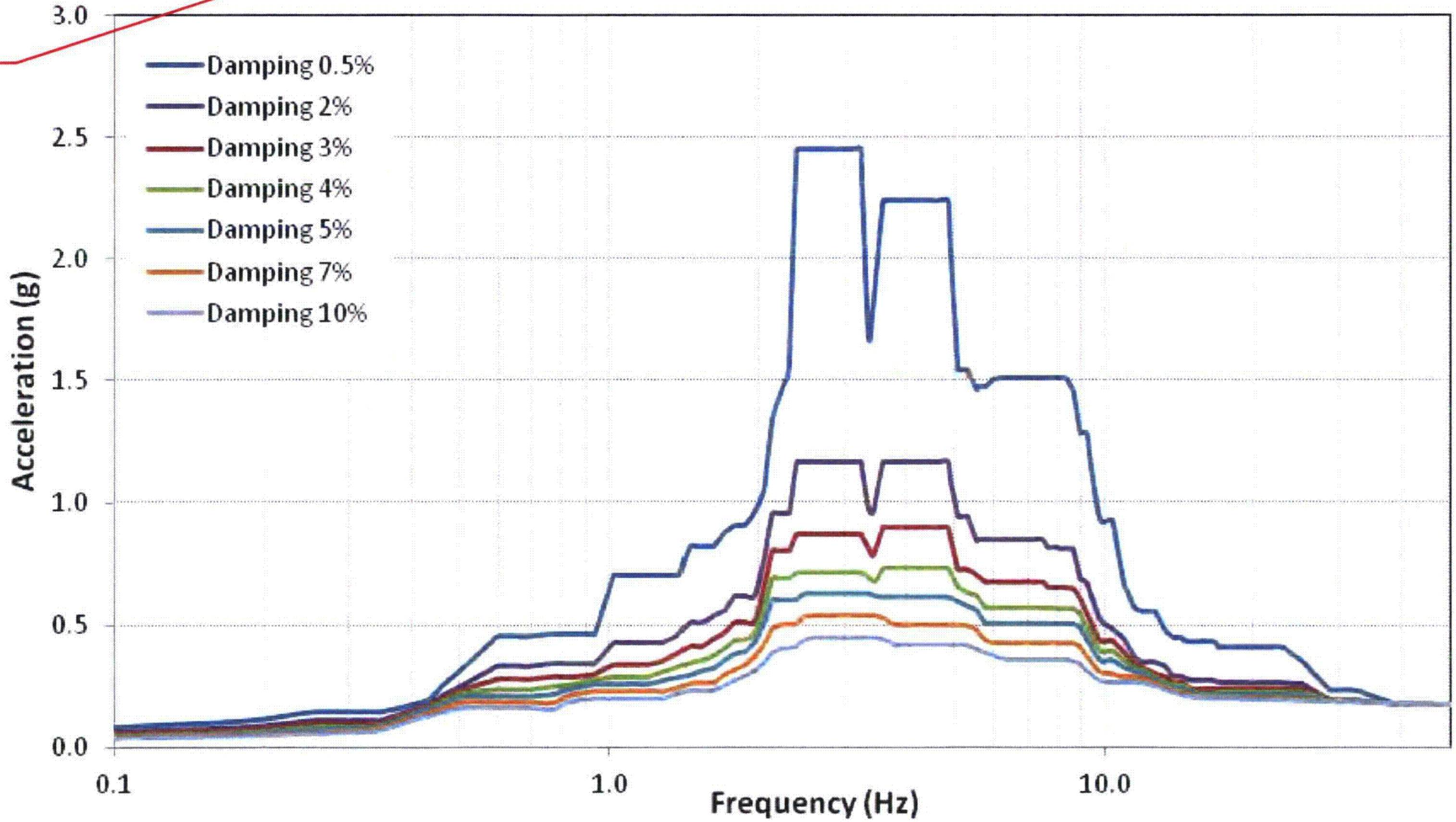

**Figure 3.7-80— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

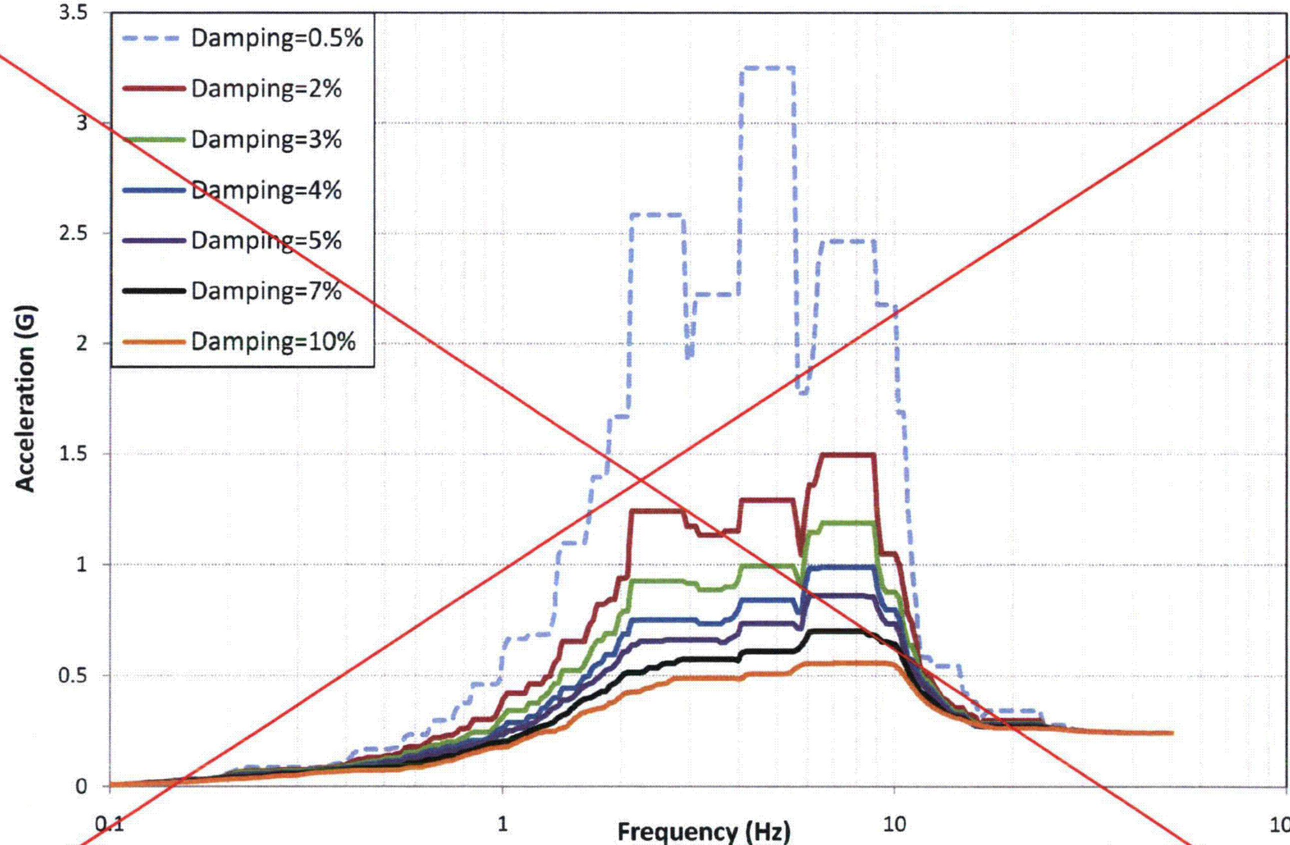

**Figure 3.7-80— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), East-West Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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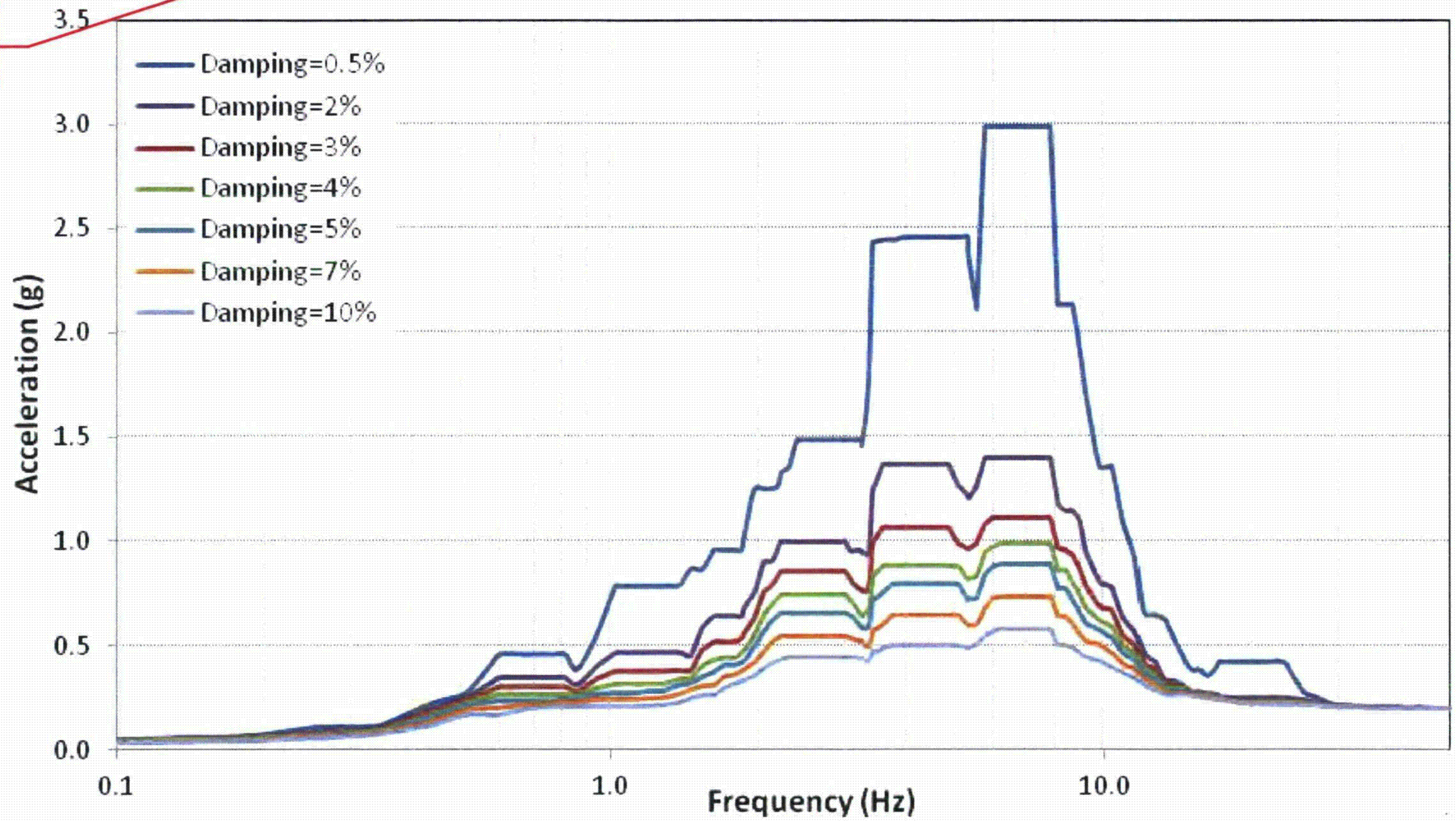

**Figure 3.7-81— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

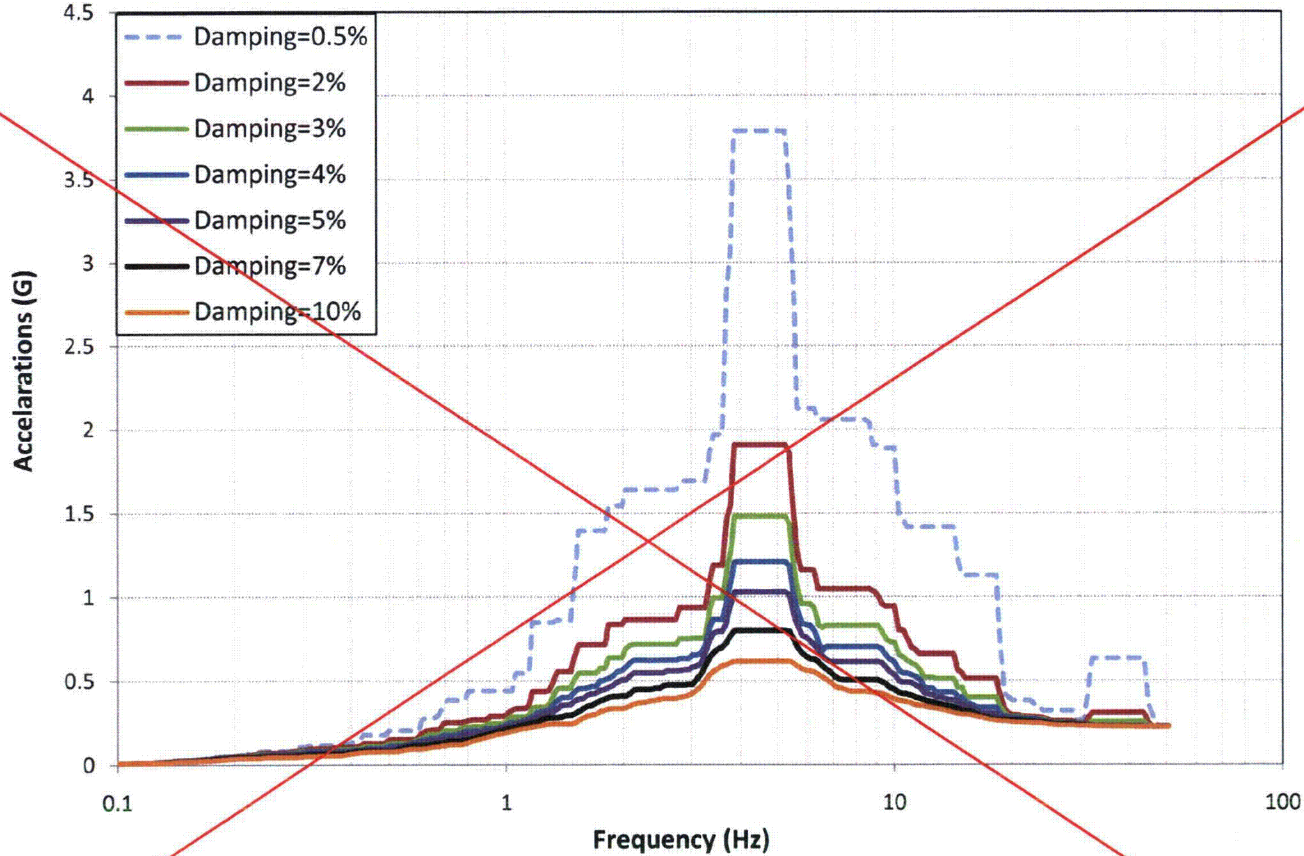

**Figure 3.7-81— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}**

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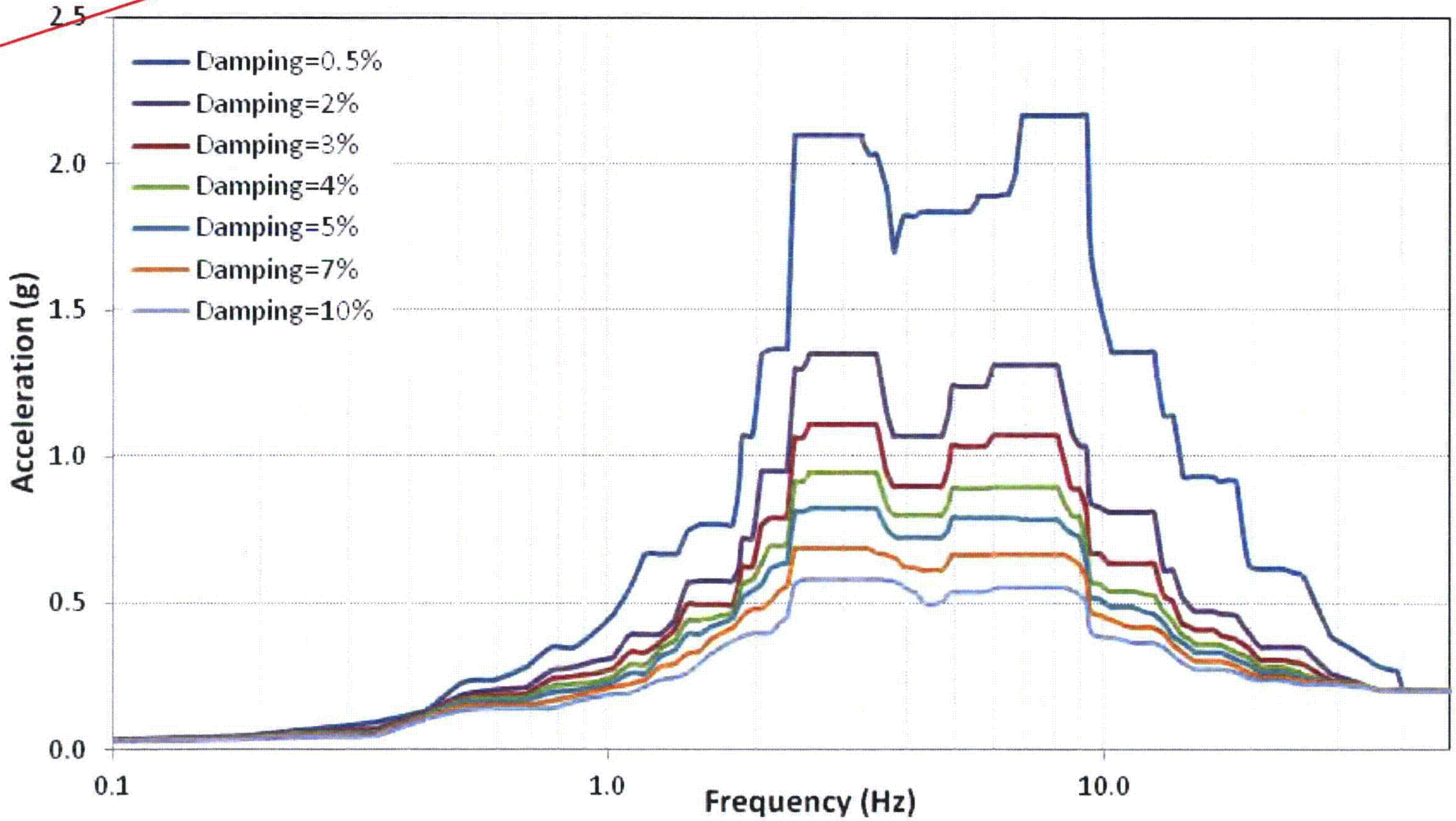
03.07.02-72



slope from manhole to manhole. The low point manholes have a sump with a pump for collecting and disposing water.

Included for Information only

Waterproofing membrane, as described in Section 3.8.4.6.1, is used, as necessary, to protect buried electrical duct banks from the corrosive effects of low-pH groundwater from the Surficial aquifer in the powerblock area.}

**3.8.4.1.9 Buried Pipe and Pipe Ducts**

The U.S. EPR FSAR includes the following COL Item in Section 3.8.4.1.9:

> A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried pipe and pipe ducts.

This COL Item is addressed as follows:

{Figure 3.8-3 provides an overall site plan of Seismic Category I buried pipe. Pipes run beneath the final site grade. Buried pipe ducts are not used for CCNPP Unit 3. Two buried Unit 3 Intake Pipes run from the CCNPP Unit 3 Inlet Area to the CCNPP Unit 3 Forebay (See Figure 2.4-56). Four UHS Makeup Water pipes emanate from the UHS Makeup Water Intake Structure and terminate at the ESWBs. These pipes run within the utility corridor, shown in Figure 3.8-3, and pass under the main Haul Road which runs in the East-West direction adjacent to the North side of the CCNPP Unit 3 powerblock.

Figure 3.8-4 provides a detail plan of Seismic Category I buried ESW pipe in the vicinity of the NI. As illustrated in the figure, the Seismic Category I buried ESW piping consists of:

- Large diameter supply and return pipes between the Safeguards Buildings and the ESWBs.
- Large diameter supply and return pipes from the EPGBs which tie in directly to the aforementioned pipes.

Fire Protection pipe traverses from the UHS Makeup Water Intake Structure to the vicinity of the NI, where a loop is provided to all buildings. In accordance with Section 3.2.1, Fire Protection piping to Seismic Category I structures that is classified as: 1) Seismic Category II is designed to maintain its pressure boundary after an SSE event; and 2) is designed to remain functional during and following an SSE event.

The buried piping is directly buried in the soil (i.e., without concrete encasement) unless detailed analysis indicates that additional protection is required. The depth of the soil cover is generally sufficient to provide protection against frost (top surface of the pipe is below the site-specific frost depth), surcharge effects, and tornado missiles. Structural fill is used as bedding material underneath the pipe. As an alternate, lean concrete may be used. Additionally, soil surrounding the pipe is compacted structural fill.}

**3.8.4.1.10 Masonry Walls**

{No departures or supplements.}

**3.8.4.1.11 {Forebay and UHS Makeup Water Intake Structure}**

{This section is added as a supplement to U.S. EPR FSAR Section 3.8.4.1.