SEISMIC EFFECTS ON SOIL-STRUCTURE INTERACTIONS

Prepared for

U.S. Nuclear Regulatory Commission
Contract NRC–02–07–006

Prepared by

Sui-Min (Simon) Hsiung
Asadul H. Chowdhury

Center for Nuclear Waste Regulatory Analyses
San Antonio, Texas

September 2011

CONTENTS

FIGURES

TABLES

EXECUTIVE SUMMARY

U.S. Nuclear Regulatory Commission (NRC) regulations, including 10 CFR Part 63, require safety review of nuclear facilities license applications for naturally occurring hazards. The assessment of seismic hazard and its impact on nuclear and other critical facilities is important to NRC, the license applicants for nuclear facilities, self-regulators such as the U.S. Department of Energy and the U.S. Department of Defense, and owners of non-nuclear facilities. Depending on the relative characteristics of a structure and geological medium such as natural soil, natural soft or hard rock, or engineered foundation system, the effects of interaction between the structure and geological medium under seismic load may be significant depending on the response of both the structure and the geological medium. The key characteristics of the structure and the site geological condition include, among others, weight of the structure and its dynamic properties, variations of geologic profile and geotechnical properties of the geological medium, and dynamic properties of the geological medium.

The objective of this report is to present a detailed overview of work performed by the Center for Nuclear Waste Regulatory Analyses (CNWRA®) supporting the NRC high-level waste repository safety program (Hsiung, et al., 2006, 2007). This report summarizes the soil-structure interaction analyses of a hypothetical waste handling facility at Yucca Mountain, Nevada, a seismically active site, for which six deaggregation earthquake response spectra on a hypothetical reference rock outcrop corresponding to the 5×10^{-4} annual exceedence frequency are available (Bechtel SAIC Company, LLC, 2004). The waste handling facility is located on an alluvium soil of varying thickness, which is above soft rock units of varying thicknesses and depths relative to the reference rock outcrop. The soil-structure interaction analyses were performed using two sets of free-field ground motions. One set of ground motions was from Bechtel SAIC Company, LLC (2004) and the other set was developed by CNWRA.

Bechtel SAIC Company, LLC (2004) developed free-field response spectra at the top of alluvium soil by conducting a ground response analysis with the six deaggregation response spectra for the reference rock outcrop as the input. The design-basis response spectrum at the top of the alluvium soil was then developed by enveloping these free-field response spectra. Bechtel SAIC Company, LLC (2004) further developed nine acceleration time histories corresponding to this design-basis response spectrum. The CNWRA developed 24 acceleration time histories on the reference rock outcrop by spectrally matching the six deaggregation response spectra Bechtel SAIC Company, LLC (2004) developed and then performed ground response analyses to calculate the free-field acceleration time histories on the top of the alluvium soil considering variations of geologic profiles and soil materials. Thus, the Bechtel SAIC Company, LLC generated nine acceleration time histories corresponding to a single design-basis response spectrum and the CNWRA generated free-field acceleration time histories corresponding to the six deaggregation response spectra developed by Bechtel SAIC Company, LLC (2004).

This report assesses the effects of geologic profile and soil material variations on the seismic structural responses at various locations of the waste handling facility. This report also compared the structural responses of the waste handling facility calculated by CNWRA using the Bechtel SAIC Company, LLC (2004) acceleration time histories and the CNWRA generated acceleration time histories.

The soil-structure interaction analysis results of the hypothetical waste handling facility reported herein showed significant influence of the geologic profiles of the foundation medium and their geotechnical and dynamic properties on the seismic structural response of the facility. The simplistic assumption of a homogeneous and isotropic medium representing a geological medium with complex and highly heterogeneous geologic profiles and material properties may not provide reasonable results for the seismic response of a facility supported by that type of geological medium. The soil-structure interaction analysis results show that a facility with a large footprint resting on a complex and heterogeneous geological medium needs to be designed for a broadened design response spectrum. The soil-structure interaction analysis results could also identify the segments of the heterogeneous foundation medium that could be improved by engineered measures to reduce the effect of soil-structure interaction on the seismic response of the facility that would result in a more efficient and economical design.

Comparison of the results of soil-structure interaction analyses conducted using the free-field acceleration time histories CNWRA developed and those generated by Bechtel SAIC Company, LLC shows reasonable agreement. The bounding acceleration responses of the operating floor at a height of 8.2 m [27 ft] and the top of the facility at a height of 25.6 m [84 ft] show general agreement except at some isolated frequencies. In developing the free-field acceleration time histories, CNWRA explicitly modeled the alluvium soil and each geologic stratum with the specific material properties provided in Bechtel SAIC Company, LLC (2004), whereas the free-field acceleration time histories Bechtel SAIC Company, LLC developed were based on site response analyses that used statistically processed homogenized material properties for the entire geological medium.

REFERENCES

Bechtel SAIC Company, LLC. "Development of Earthquake Ground Motion Input for Preclosure Seismic Design and Postclosure Performance Assessment of a Geologic Repository at Yucca Mountain, NV." MDL–MGR–GS–000003. Rev. 01. Las Vegas, Nevada: Bechtel SAIC Company, LLC. 2004.

Hsiung, S.M., A. Ghosh, and A.H. Chowdhury. "Site Response and Soil-Structure Interaction Analyses—A Progress Report." San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2007.

Hsiung, S.M., S.H. Gonzalez, and A.H. Chowdhury. "Site Response and Soil-Structure Interaction Analyses—A Progress Report." San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2006.

ACKNOWLEDGMENTS

This knowledge capture report was prepared to document a detailed overview of the work performed by the Center for Nuclear Waste Regulatory Analyses (CNWRA®) for the U.S. Nuclear Regulatory Commission (USNRC) under Contract No. NRC–02–07–006. The activities reported here were performed on behalf of the USNRC Office of Nuclear Material Safety and Safeguards, Division of High-level Waste Repository Safety. This report is an independent product of the CNWRA and does not necessarily reflect the view or regulatory position of USNRC.

The authors thank D. Pomerening for technical review, D. Pickett for programmatic review, and E. Pearcy for his editorial review. The authors also appreciate L. Selvey for her administrative support.

QUALITY OF DATA, ANALYSES, AND CODE DEVELOPMENT

DATA: CNWRA-generated hand calculation results are contained in this report. Sources of other data should be consulted for determining the level of quality for those data.

ANALYSES AND CODES: ProShake® Version 1.11 (EduPro Civil Systems, 2001) was used for one-dimensional equivalent-linear site response analyses. EZ-FRISK™ Version 7.20 (Risk Engineering, Inc., 2005) was used to spectrally match acceleration time histories with response spectra. SASSI 2000, Revision 1 (Lysmer, et al., 1999) was used for soil-structure interaction analyses. These codes are controlled under the CNWRA quality assurance program. Analyses described in this report are documented in CNWRA Scientific Notebooks No. 726 (Hsiung, 2005).

REFERENCES

EduPro Civil Systems. “ProShake.” Version 1.11. Sammamish, Washington: EduPro Civil Systems. 2001.

Hsiung, S.M. “Soil-Structure Interaction Analyses.” Scientific Notebook No. 726. San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2005.

Lysmer, J., F. Ostandan, and C.C. Chin. “SASSI 2000—A System for Analysis of Soil-Structure Interaction, User’s Manual.” Rev. 1. Berkeley, California: University of California. 1999.

Risk Engineering, Inc. “EZ-FRISK.” Version 7.20. Golden, Colorado: Risk Engineering, Inc. 2005.

1 INTRODUCTION

1.1 Background

Appropriate consideration of seismic hazard and its impact on nuclear and other critical facilities is important to the U.S. Nuclear Regulatory Commission (NRC), the license applicants for nuclear facilities, other federal agencies such as the U. S. Department of Energy and the U.S. Department of Defense, and owners of non-nuclear critical facilities. Design of nuclear facilities to account for the seismic hazard is an important step to ensure operational safety and health and safety of the public.

Most civil engineering structures involve direct contact with the ground. When subjected to earthquake ground motions, the structural response and the ground displacements are not independent of each other. In an earthquake, seismic waves are transmitted from the bedrock through the soil medium to a structure of interest. The seismic waves excite the structure, which in turn modifies the input free-field ground motion. This difference in ground motion is due to interaction between the soil geological medium including soil and the superstructure, also known as soil-structure interaction.

Damage observed in several earthquakes such as the 1985 Mexico City earthquake (Resendiz, 1986; Avilés and Pérez-Rocha, 1998), the 1995 Kobe earthquake (Mylonakis, et al., 2000), and the 1999 Ji-Ji Taiwan earthquake (Earthquake Engineering Field Investigation Team, 2011) has shown that the seismic behavior of a structure is influenced by the responses of the structure, its foundation, and the ground. In general, the response of the structure is dependent on the properties of the soil, characteristics of the dynamic excitation, type of foundation of the structure, and the physical characteristics of the structure. For example, soft soil sediments could lead to elongation of the period of seismic waves resulting in the structure resonating with the long period ground vibration (Mylonakis, et al., 2000). The failure of soil may further aggravate the seismic response of the structure (Earthquake Engineering Field Investigation Team, 2011). Avilés and Pérez-Rocha (1998) identified the significant soil-structure interaction effects of an extended alluvial valley (Valley of Mexico) on medium- and long-period structures.

The soil-structure interaction effects between the structure and its geological medium under seismic load become more prominent for heavy structures (e.g., nuclear power plants, high-rise buildings, elevated highways, and other infrastructures). Whereas the conventional design approach of not considering soil-structure interaction may be acceptable for light structures, it is prudent to include soil-structure interaction as an integral part of the design process for heavy structures. Once seismic hazard assessment is completed and a design-basis response spectrum is determined, soil-structure interaction analysis should be the next step for design and performance analysis of heavy structures. This step will develop appropriate seismic input for structural analysis, design, and qualification of critical structures, systems, and components. If the seismic input for analysis of the structures—especially the systems and components to be located within the structures—is not appropriately developed, the performance of their intended safety functions may not be assessed adequately. As discussed previously, to develop reasonable seismic input for structural analysis, appropriate consideration of the parameters such as the variability of geology and geotechnical and dynamic properties of geological medium, characteristics of the dynamic excitation, and the physical characteristics of the structure is essential.

Analysis of soil-structure interaction includes two major parts: (i) free-field ground motion or site response to seismic waves, in the absence of the structure, propagating through the earth to the ground surface and (ii) interaction between the structure, including systems and components located within the structure, and the geological medium using the free-field site-response data developed in part (i).

1.2 Objective and Scope

In this report, the Center for Nuclear Waste Regulatory Analyses (CNWRA®) documents its independent analyses conducted previously (Hsiung, et al., 2006, 2007) on the effects of the complex site subsurface geological medium on soil-structure interaction of a hypothetical waste handling facility at Yucca Mountain, Nevada. The complexity involves large variability in geologic units, including spatial variation of thicknesses and dynamic properties of geologic units. The soil-structure interaction analyses also assess the proposed seismic design-basis spectrum for the site of interest through comparing the responses of the critical structures resulting from the ground motions for the design-basis spectrum with the structural responses subjected to the ground motions developed separately at the same hazard level for the same site. In addition, this report discusses the potential effects of the soil-model assumptions used in the soil-structure interaction analyses.

The computer code SASSI 2000 (A **S**ystem for **A**nalysis of **S**oil-**S**tructure **I**nteraction) (Lysmer, et al., 1999a) was used to investigate the seismic responses of a hypothetical waste handling facility. The input free-field acceleration time histories include two sets of ground motions. All of them correspond to the 2,000-year return period (i.e., annual exceedance frequency of 5×10^{-4}). The first set of ground motion time histories were developed by CNWRA. In developing the free-field ground motion time histories, one-dimensional site response analyses were performed using the computer code ProShake® (EduPro Civil Systems, 2001). The second set of ground motion time histories were from Bechtel SAIC Company, LLC (2004).

2 THEORETICAL BACKGROUND OF SOIL-STRUCTURE INTERACTION ANALYSIS

2.1 Theory

Soil-structure interaction problems can be analyzed using the substructuring approach by subdividing the problem into simpler subproblems. The subproblems may be solved individually and the results combined to provide a complete solution using the principle of superposition. Because superposition is used to obtain the solution, the substructuring method is limited to the linear domain (Lysmer, et al., 1999a).

Conceptually, the substructuring methods for soil-structure interaction analysis can be categorized into four types depending on the interaction at the soil and structure interface (Lysmer, et al., 1999a). These methods include (i) rigid boundary, (ii) flexible boundary, (iii) flexible volume, and (iv) substructure subtraction. All methods require site-response analysis, soil-impedance calculation, and structural response analysis. In addition, the rigid and flexible boundary methods require performing scattering analysis for the soil-structure interaction evaluation.

The SASSI 2000 computer program (Lysmer, et al., 1999a) was selected for the independent soil-structure interaction analyses the CNWRA staff conducted. This program uses the substructuring method to approach a soil-structure interaction problem and solves the problem in the frequency domain. The program includes both the flexible volume and substructure subtraction techniques (Lysmer, et al., 1999a). Users can specify which technique to use. For the analyses presented in this report, the flexible volume method was used.

The current version of SASSI 2000 has two major limitations: (i) the site should consist of horizontal soil layers (Lysmer, et al., 1999a) and (ii) the analytical method the program uses solves linear problems only.

2.2 Background for SASSI 2000 Computer Code

2.2.1 Formulation

Both the flexible volume and substructure subtraction methods in SASSI 2000 use the concept of partitioning the soil-structure system [Figure 2-1(a)] into three substructures [Figure 2-1(b), (c), and (d)] (Lysmer, et al., 1999b). Substructure I consists of the free field, Substructure II is made of the excavated soil volume, and Substructure III consists of the structure itself including the basement. The term basement is used here to mean any portion of a structure below grade. A basement can be a complicated structure or a basemat, as the term is used later in this report in the context of soil-structure interaction. In the flexible volume method, the interaction between the free field (Substructure I) and the excavated soil volume (Substructure II) occurs at the boundary and within the excavated soil volume, and the soil-structure interactions between Substructures II and III and Substructures I and III occur at the interfaces with the basement of Substructure III (Lysmer, et al., 1999b).

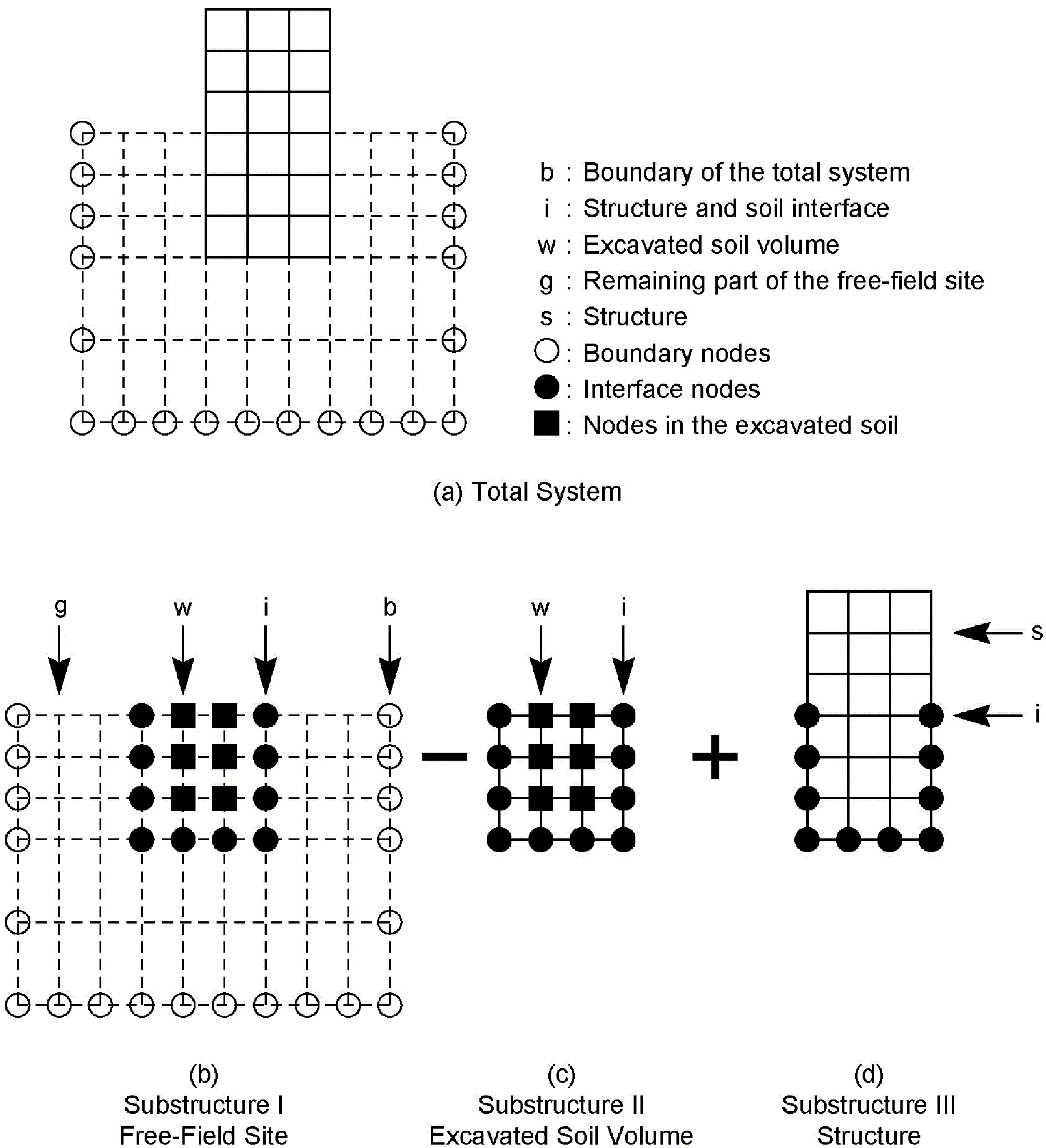


Figure 2-1. Substructures I [2-1(b)], II [2-1(c)], and III [2-1(d)] of a Total System [2-1(a)] for Soil-Structure Interaction Analysis (Modified From Lysmer et al., 1999b)

In the frequency domain, the equation of motion for each substructure in Figure 2-1 may be expressed as (Lysmer, et al., 1999b)

$$[C]\{U\} = \{Q\} \tag{2-1}$$

where $[C]$ is complex frequency-dependent dynamic stiffness matrix, $\{U\}$ is complex displacement matrix, and $\{Q\}$ is complex load matrix.

At a specific frequency ω,

$$[C] = [K] - \omega^2[M] \tag{2-2}$$

$$\{U\} = \{\underline{U}\}e^{-i\omega t} \tag{2-3}$$

$$\{Q\} = \{\underline{Q}\}e^{-i\omega t} \tag{2-4}$$

where $[K]$ is stiffness matrix, $[M]$ is mass matrix which is a combination of one-half lump mass matrix and one-half consistent mass matrix, $[\underline{U}]$ is displacement matrix in the time domain, and $[\underline{Q}]$ is load matrix in the time domain due to the seismic excitations.

A lump mass matrix is a mass matrix formed by assuming the total element mass is directly apportioned to nodal freedom, ignoring any cross coupling and the consistent mass matrix is a mass matrix derived using the same shape functions as those for the stiffness matrix. Combining the substructures, the equation of motion used for solving a soil-structure problem used in the SASSI 2000 can be expressed as follows.

$$\begin{bmatrix} C_{ii}^{III} - C_{ii}^{II} + X_{ii} & -C_{iw}^{II} + X_{iw} & C_{is}^{III} \\ -C_{wi}^{II} + X_{wi} & -C_{ww}^{II} + X_{ww} & 0 \\ C_{si}^{III} & 0 & C_{ss}^{III} \end{bmatrix} \begin{Bmatrix} U_i \\ U_w \\ U_s \end{Bmatrix} = \begin{Bmatrix} X_{ii}U_i' + X_{iw}U_w' \\ X_{wi}U_i' + X_{ww}U_w' \\ 0 \end{Bmatrix} \tag{2-5}$$

where superscripts II and III are types of substructures shown in Figure 2-1, subscript I is for nodes at the interface between the soil and the structure, subscript *w* is for nodes within the excavated soil volume, subscript s is for nodes for the structure excluding the basement, $[X_{fh}]$ is the impedance matrix representing the dynamic stiffness of the foundation at the interaction nodes where subscripts f and h represent either i or w, and $\{U_f'\}$ is the matrix of free-field motions where subscript f represents either I or w.

This formulation applies to both two- and three-dimensional soil-structure interaction problems. Notice that Substructure I is not included in Eq. (2-5), implying that it is sufficient to assess the interaction response of a structure based only on the free-field motions within the depth of the basement (Chen, et al., 1981).

The subproblem in the flexible volume method used in SASSI 2000 includes

(i) Determining $\{U_f'\}$ by conducting site response analysis

(ii) Solving for $[X_{fh}]$ by performing the impedance analysis

(iii) Solving the linear equation of motion to obtain the transfer functions that will be used to determine structural responses.

Thus, to analyze the site response, the free-field displacements at the interaction points at the soil-layer interface are calculated. Through the impedance analysis, the impedance matrix or the dynamic stiffness for the excavated soil volume at the interaction points will be determined. SASSI 2000 performs structural analyses using the finite element method as defined in Eq. (2-5) for both Substructures II and III.

2.2.2 Site Response Analysis

The site response analysis is performed using SASSI 2000 for a horizontally layered soil system with a semi-infinite halfspace. The equations of motion are formulated such that the displacements are solved at the interfaces of soil layers. The displacements in each soil layer are assumed to vary linearly with the thickness of the soil layer. SASSI 2000 can calculate the free-field displacements resulting from a combination of plane-wave fields including body waves, such as compressive (P-), vertically propagating shear (SV-), and horizontally propagating shear (SH-) waves, and surface waves, such as Rayleigh and Love waves.

2.2.2.1 Free-Field Displacements

To solve for the free-field displacements $\{U_f'\}$ at the interaction nodes (Figure 2-1) including those within the excavated soil volume in the flexible method, Substructure I [Figure 2-1(b)] is used (Lysmer, et al., 1999b). SASSI 2000 requires that all interaction nodes be placed at the soil layer interfaces.

The equation of motion of a layered soil system for the incident P- and SV-waves is (Lysmer, et al., 1999b; Chen, et al., 1981; Udaka, 1975)

$$\left([A]k^2 + [\overline{B}]k + [G] - \omega^2[M]\right)\{U\} = \begin{Bmatrix} 0 \\ P_b \end{Bmatrix} \tag{2-6}$$

where $[A]$ and $[G]$ are constant matrices for each soil layer and are a function of layer thickness, Lame's constant, and shear modulus of the soil layer; $[\overline{B}]$ is the constant matrix for each soil layer and is a function of Lame's constant and shear modulus of the soil layer; $\{P_b\}$ is the load matrix for the forces at the interface between the layered soil system and the half space; and k is the eigenvalue or wave number. The load matrix $\{P_b\}$ and wave number *k* can be determined if the information regarding the incidence angle of the wave and the nature of the wave field is provided.

The equation of motion of a layered soil system for the SH-waves takes the form (Lysmer, et al., 1999b; Chen, et al., 1981)

$$\left(\left[\overline{A}\right]k^2 + \left[\overline{G}\right] - \omega^2\left[\overline{M}\right]\right)\{U\} = \begin{Bmatrix} 0 \\ P_b \end{Bmatrix} \tag{2-7}$$

where $\left[\overline{A}\right]$ and $\left[\overline{G}\right]$ are constant matrices for each soil layer and are a function of layer thickness and shear modulus of the soil layer and $\left[\overline{M}\right]$ is the mass matrix of each soil layer.

The equation of motion of a layered soil system for the Rayleigh waves can be generalized as (Chen, et al., 1981; Waas, 1972)

$$([A]k^2 + i[B]k + [G] - \omega^2[M])\{V\} = 0 \tag{2-8}$$

where $[B]$ is constant matrix for each soil layer and is a function of Lame's constant and shear modulus of the soil layer and $\{V\}$ is associated eigenvectors (mode shapes).

The equation of motion of a layered soil system for the Love waves can be generalized as (Chen, et al., 1981; Wass, 1972)

$$\left(\left[\overline{A}\right]k^2 + \left[\overline{G}\right] - \omega^2\left[\overline{M}\right]\right)\{V\} = 0 \tag{2-9}$$

Note that for a soil system with *n*-number of horizontal layers, the matrices in the equation of motion for the SV- and P-waves [Eq. (2-6)] contain 2 (*n* + 1) × 2 (*n* + 1) elements. In the matrices in the equation of motion for the SH-waves [Eq. (2-7)] contain (*n* + 1) × (*n* + 1) elements. In the matrices in the equation of motion for the Rayleigh waves [Eq. (3-8)], each matrix contains 2 *n* × 2 *n* elements. The matrices in the equation of motion for the Love waves [Eq. (2-9)] contain *n* × *n* elements (Lysmer, et al., 1999b; Chen, et al., 1981).

After solving the equations of motion for displacements $\{U\}$ at the soil-layer interfaces, displacements along the horizontal direction can be determined using (Lysmer, et al., 1999b)

$$\{U(x)\} = \delta\{U\}e^{-ikt} \tag{2-10}$$

where x is the horizontal distance from the defined control-point location (an input by SASSI 2000 user) where the input control motion is applied and δ is the mode participation factor which can be obtained from the input control motion at the frequency of interest.

2.2.2.2 Boundary Conditions

2.2.2.2.1 Transmitting Boundary

To correctly transmit the seismic energy in the horizontal direction, SASSI 2000 implemented transmitting boundaries. Formulation of the transmitting boundary for two-dimensional problems adopted in SASSI 2000 was developed by Waas (1972) using the eigenvalues and eigenvectors obtained from solving the equation of motion for the Rayleigh waves [Eq. (2-8)]. The formulated force-displacement relationship for two-dimensional problems in the frequency domain for a layered soil system takes the form of

$$\{P\} = [R]\{U\} \tag{2-11}$$

where $\{P\}$ is force matrix and $[R]$ represents the dynamic stiffness matrix of the semi-infinite layered region beyond the transmitting boundary and (Lysmer, et al., 1999b)

$$[R] = i[A][V][K][V]^{-1} + [D] \tag{2-12}$$

where $[D]$ is a constant matrix and a function of Lame's constant and shear modulus of a soil layer.

Using the eigen-solutions for Rayleigh and Love waves [Eqs. (2-8) and 2-9)], a force-displacement relationship similar to Eq. (2-12) for a layered soil system in an axisymmetric condition may be obtained. However, the analytical solution for the dynamic stiffness matrix $[R]$ for the axisymmetric problems appears to be more complicated than that for the two-dimensional problems. In the axisymmetric case, the radius (distance from the central axis to the transmitting boundary) of the axisymmetric model is an important parameter in formulating the dynamic stiffness matrix (Lysmer, et al., 1999b). The transmitting boundary formulation for the axisymmetric problems is also used as the transmitting boundary formulation for the three-dimensional problems in SASSI 2000.

2.2.2.2.2 Viscous Boundary

The bottom boundary of the layered soil system in SASSI 2000 may be modeled as a rigid (fixed) boundary or a halfspace. The halfspace may be represented by several soil layers with variable thicknesses and a viscous boundary at the bottom (Lysmer, et al., 1999a). The rigid boundary tends to reflect some seismic energy back to the system and may, in turn, result in some erroneous natural frequencies that affect the overall response (Lysmer, et al., 1999a). Simulation of a halfspace will reduce the energy reflection problems and greatly improve the accuracy of the impedance calculation.

In SASSI 2000, the halfspace option is selected by specifying the number of sublayers to represent the halfspace. SASSI 2000 requires a minimum of four sublayers to simulate a halfspace. The total thickness of the sublayers specified for the halfspace is a function of frequency of interest and shear velocity of the halfspace. The thickness of each sublayer is a function of total thickness, number of sublayers, and thickness of the soil layer above the halfspace (Lysmer, et al., 1999b).

The viscous boundary consists of two dashpots per unit area of the boundary (Lysmer and Kuhlemeyer, 1969; Lysmer, et al., 1999b)—one for P-wave damping and the other for shear-wave damping. The damping coefficients for the dashpots can be expressed as

$$C_p = \rho V_p \tag{2-13}$$

and

$$C_s = \rho V_s \tag{2-14}$$

where C_p is damping coefficient for P-waves, C_s is damping coefficient for shear waves, ρ is mass density of the halfspace, V_p is P-wave velocity for the halfspace, and V_s is shear-wave velocity for the halfspace.

2.2.3 Impedance Matrix Determination

As with the flexible volume method, the impedance matrix $[X_{fh}]$ needs to be calculated by all interaction nodes in the excavated soil volume, as shown in Eq. (2-5) and Figure 2-1(c). The impedance matrix can be obtained by inverting the compliance matrix at each frequency of interest. The compliance matrix is a collection of the displacements responding to a unit harmonic force acting at the interaction nodes defined in the excavated soil volume. More specifically, the elements in the i^{th} column of the impedance matrix are the displacements of the interacting degree-of-freedom induced by a unit harmonic force acting at the i^{th} degree-of-freedom (Lysmer, et al., 1999b). To determine the compliance matrix, a soil column consisting of a single column of loaded interaction nodes (Figures 2-2 and 2-3) is used in the SASSI 2000. This soil column includes two transmitting boundaries for two-dimensional problems and a cylindrical transmitting boundary for axisymmetric and three-dimensional problems. The displacement responses at the interaction nodes (shown as the loaded interaction nodes in Figures 2-2 and 2-3) and the boundary nodes in the soil model can be determined by solving the following equations of motion (Lysmer, et al., 1999b) and successively applying the unit loads to the interaction nodes on the center line of the model.

$$\begin{bmatrix} C_{cc} & C_{cl} \\ C_{lc} & C_{ll} + R \end{bmatrix} \begin{Bmatrix} U_c \\ U_l \end{Bmatrix} = \begin{Bmatrix} Q_c \\ 0 \end{Bmatrix} \tag{2-15}$$

where subscript c is index referring to the degree-of-freedom for the interaction nodes along the center line, subscript l is index referring to the degree-of-freedom for the nodes along the transmitting boundary, $[C]$ is dynamic stiffness matrix which takes the form of Eq. (2-2), and $\{Q_c\}$ is load matrix (for each load case, this matrix has only one non-zero element corresponding to the applied unit load).

The solution for the compliance or displacement matrix for a single column of the interaction nodes can be used as the compliance matrix for the remaining columns of interaction nodes. As discussed previously, once the displacement matrix for all interaction nodes is obtained, this matrix can be inverted to determine the impedance matrix used to solve Eq. (2-5).

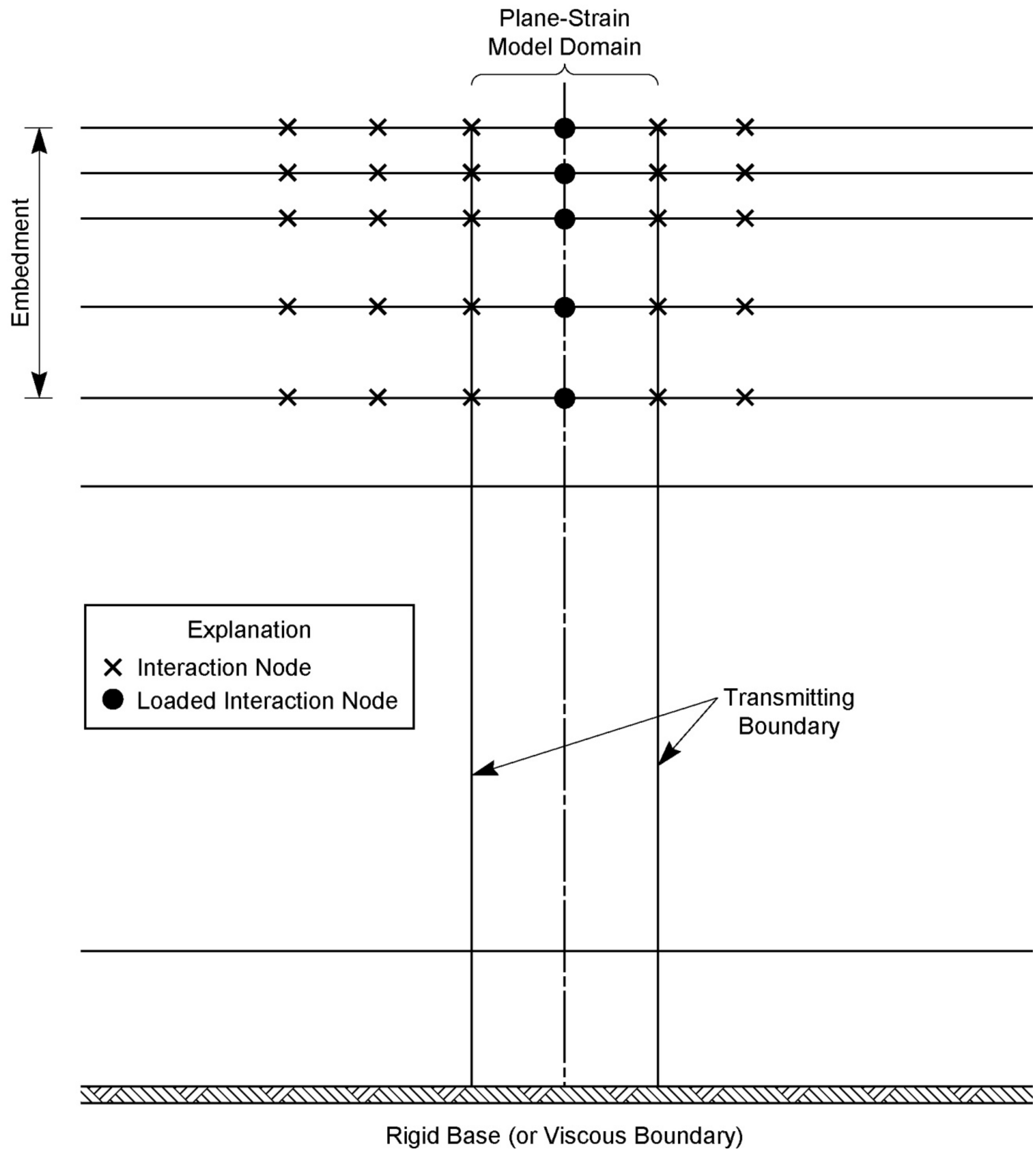


Figure 2-2. Two-Dimensional Plane-Strain Soil Model for Impedance Matrix Calculation (Modified From Lysmer, et al., 1999b)

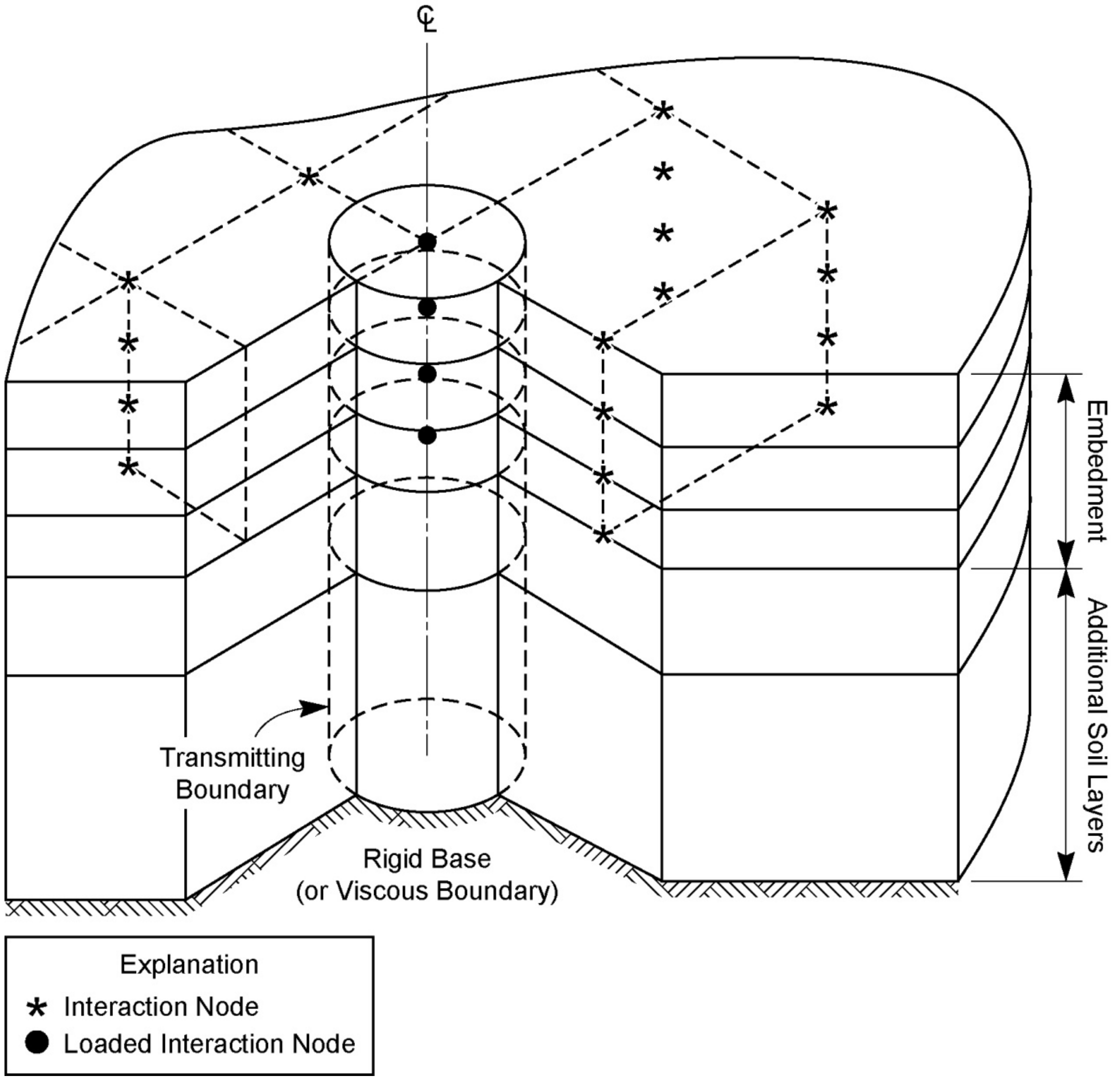


Figure 2-3. Axisymmetric and Three-Dimensional Soil Model for Impedance Matrix Calculation (Modified From Lysmer, et al., 1999b)

3 SEISMIC HAZARDS AND SITE RESPONSE

3.1 Seismic Hazards

Ground motion results for the probabilistic seismic hazard analyses were developed for a hypothetical reference rock outcrop at Yucca Mountain, referred to as Point A in Figure 3-1 (Stepp, et al., 2001). Point A in Figure 3-1 is located on the same rock unit and at the same elevation as Point B, which represents the proposed repository horizon. The only difference between Points A and B is that Point A is an outcrop site. Within the proposed surface facilities site, Point D refers to locations southwest of the Exile Hill fault splay with alluvium thickness of at least 15 m [49 ft] and Point E refers to the area on the west side for which the alluvium is approximately 4.6 m [15 ft] thick or less (Bechtel SAIC Company, LLC, 2004).

At Point A, six deaggregation earthquake response spectra were generated for each hazard level in the 10^{-3} to 10^{-7} annual exceedence frequency range for both vertical and horizontal components of ground motion (Bechtel SAIC Company, LLC, 2004). Bechtel SAIC Company, LLC developed these deaggregation earthquake response spectra based on the deaggregation of the probabilistic seismic hazard analyses and divided them equally into two groups. The three response spectra in the first group represent the 1–2 Hz structural response frequency range, while the response spectra in the second group represent the 5–10 Hz structural response frequency range. The response spectra of the three deaggregation earthquakes in each group were developed by first deaggregating the probabilistic seismic hazard with respect to magnitude only to obtain the 5^{th}, mean, and 95^{th} percentile magnitudes. The site-to-source distances corresponding to these magnitudes were obtained by deaggregating the probabilistic seismic hazard with respect to distance only and selecting the corresponding distances for the 5^{th}, mean, and 95^{th} percentile (Bechtel SAIC Company, LLC, 2004).

In this report, one set of free-field ground accelerations used for the soil-structure interaction analyses were developed by CNWRA by performing site response analyses using these six deaggregation earthquake response spectra representing the 5×10^{-4} annual exceedence frequency (Figure 3-2). Detailed discussion on developing this set of free-field ground accelerations are provided in the following three sections. The second set of ground motions used in the soil-structure interaction analyses is from Bechtel SAIC Company, LLC (2004) and is discussed in Section 4.3 of this report.

3.2 Spectral Matching of Input Time Histories

Before free-field acceleration time histories can be generated, site response analyses are required to propagate the accelerations from Point A in Figure 3-1. These accelerations should be consistent with the six deaggregation earthquake response spectra (Table 3-1) representing the 5×10^{-4} annual exceedence frequency at the reference rock outcrop location. To calculate the acceleration time histories corresponding to the deaggregation earthquake response spectra representing the 5×10^{-4} annual exceedence frequency, CNWRA used the recorded acceleration time histories of 24 earthquakes to spectrally match the six deaggregation earthquake response spectra. These time histories were from the European Strong-Motion Database (2006) and McGuire, et al. (2001). These earthquakes were chosen because they have magnitudes, distances, and faulting styles similar to the deaggregation earthquakes. Three to six spectrally matched acceleration time histories were developed for each deaggregation response spectrum shown in Figure 3-2.

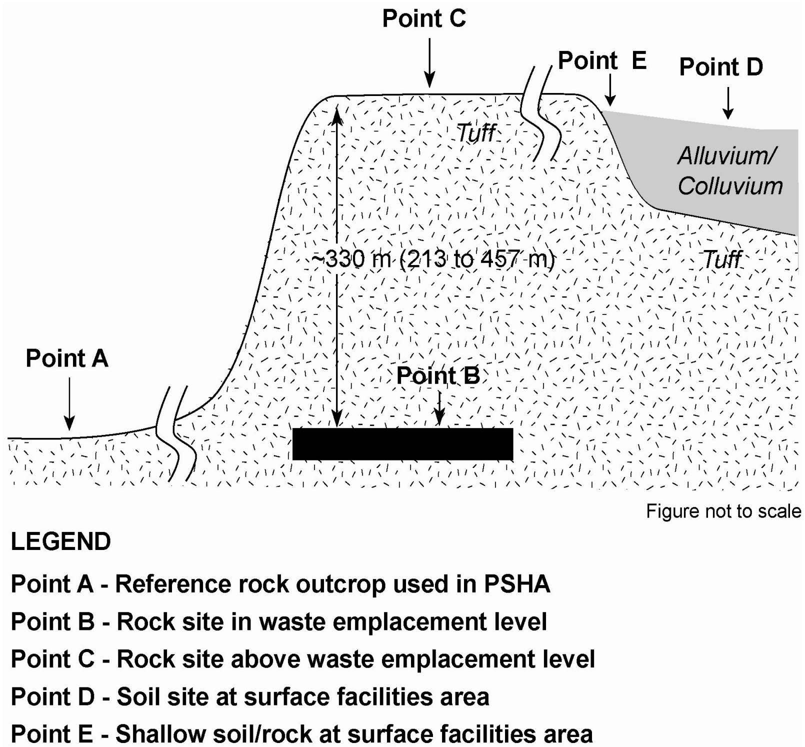


Figure 3-1. Schematic of the Site Showing the Relative Location Points A, B, C, D, and E [Note: PSHA = Probabilistic Seismic Hazard Analysis] (Modified From Stepp, et al., 2001)

The 24 spectrally matched acceleration time histories were developed using EZ-FRISK™ Version 7.20 (Risk Engineering, Inc., 2005). A convergence tolerance criterion was used for spectral matching to ensure the maximum deviation of the matched spectrum from the target response spectrum was 0.01.

3.3 Site-Specific Soil Model

To generate the free-field acceleration time histories at the ground surface necessary for soil-structure interaction analyses of the hypothetical waste handling facility, all 24 spectrally matched acceleration time histories were used as inputs to the site response analyses.

Considering the potential effects of spatial variations (both thicknesses and dynamic properties of geologic units), nine site-specific geologic profiles and three sets of shear wave (S wave) velocities were used for the soil-structure interaction analyses. Among the nine geologic columns (profiles), four (RF13, RF14, RF16, and RF17) were from Bechtel SAIC Company, LLC (2002) and the remaining five (SF01, SF02, SF03, SF04, and SF05) were extracted from a three-dimensional geological model representing the site built by CNWRA using the EarthVision™ software (Dynamic Graphics, Inc., 2002).

The lithologic information and thickness of each lithologic unit for these nine geologic profiles are shown in Figure 3-3. Differences in unit thicknesses for the nine geologic profiles are

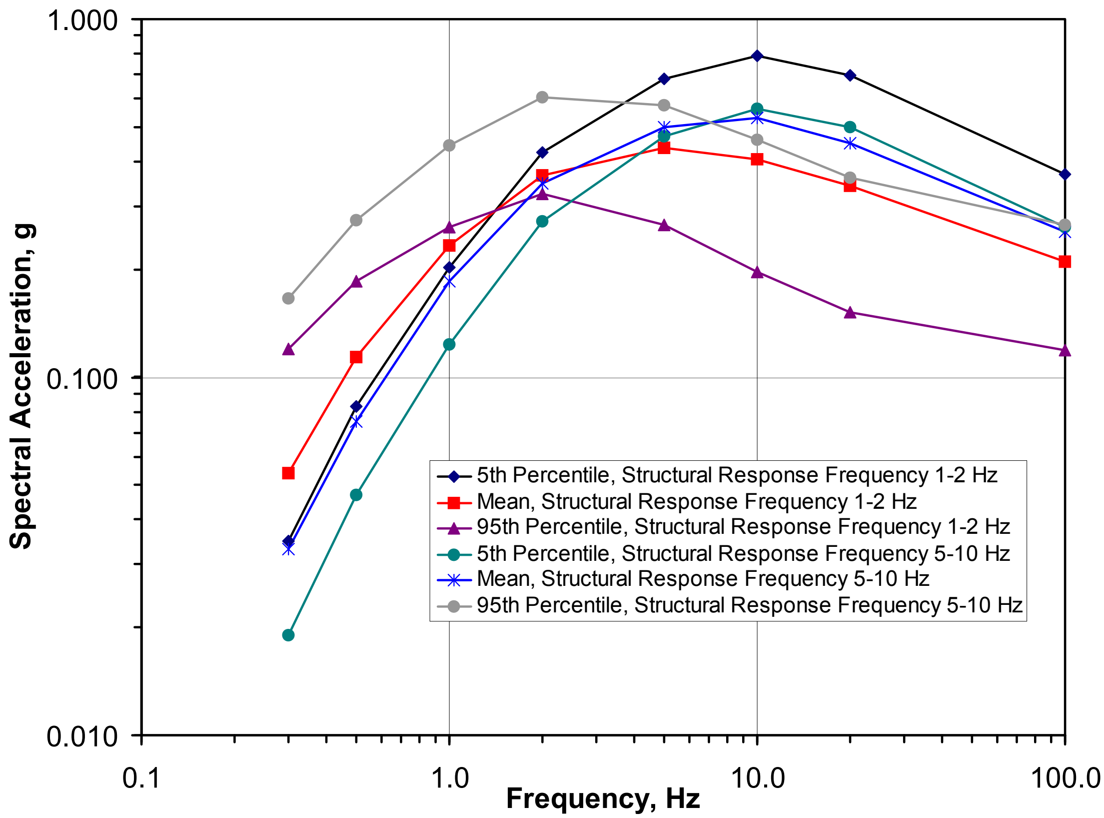


Figure 3-2. Six Deaggregation Earthquake Response Spectra Representing the 5×10^{-4} Hazard Level

substantial. Consequently, these nine geologic profiles should be sufficient to evaluate the significance of geologic spatial variations on the seismic response of the hypothetical waste handling facility assumed to be located at the site with this type of geological medium.

Three sets of S-wave velocities selected to investigate the potential effects of soil stiffness on dynamic structural responses were average, average minus one standard deviation (lower bound), and average plus one standard deviation (upper bound) S-wave velocities. These S-wave velocities along with the densities of the geologic profile are given in Table 3-2. For convenience, the geologic profiles with the average S-wave velocity (Bechtel SAIC Company, LLC, 2002) are referred to as average soils, the geologic profiles with the average minus one standard deviation S-wave velocity are referred to as soft soils, and the geologic profiles with the average plus one standard deviation S-wave velocities are referred to as stiff soils.

These nine geologic profiles and three sets of dynamic properties were used in the site response analyses to develop free-field ground acceleration time histories for the soil-structure interaction analyses.

Table 3-1. Characteristics of Earthquakes Used for Generating Spectrally Matched Acceleration Time Histories Representing 5 × 10^{-4} Annual Exceedence Frequency				
Earthquake	**Station**	**M_w**	**Epicentral Distance, km [mi]**	**DOE Deaggregation Response Spectrum Matched**
Umbria Marche 1, Italy (aftershock)	Nocera Umbra-Biscontini	5.5	10 [6.2]	5^{th} Percentile, Structural Response Frequency 1–2 Hz
Coalinga 1, California	Transmitter Hill	5.2	10.4 [6.5]	
Coalinga 2, California	Anticline Ridge Free-Field	5.2	11 [6.8]	
Kozani 1, Greece	Kozani-Prefecture	6.5	17 [10.6]	Mean, Structural Response Frequency 1–2 Hz
Umbria Marche 3, Italy (aftershock)	Borgo-Cerreto Torre	6	25 [15.5]	
South Iceland 1, Iceland (aftershock)	Hveragerdi-Church	6.4	24 [14.9]	
South Iceland 2, Iceland (aftershock)	Selfoss-City Hall	6.4	15 [9.3]	
Izmit 1, Turkey	Heybeliada-Senatoryum	7.6	78 [48.5]	95^{th} Percentile, Structural Response Frequency 1–2 Hz
Vrancea 1, Romania	Vrancioaia	7.2	49 [30.4]	
Izmit 2, Turkey	Gezbe-Tubitak Marmara Arastima Merkezi	7.6	47 [29.2]	
Vrancea 2, Romania	Petresti-Foscani	7.2	75 [46.6]	
Umbria Marche 2, Italy (aftershock)	Nocera Umbra-Biscontini	5.3	8 [5.0]	5^{th} Percentile, Structural Response Frequency 5–10 Hz
Coalinga 3, California	Oil City	5.8	8.2 [5.1]	
Coalinga 4, California	Anticline Ridge Pad	5.2	11 [6.8]	
Valnerina, Italy	Cascia	5.8	5 [3.1]	Mean, Structural Response Frequency 5–10 Hz
Friuli, Italy (aftershock)	Tarcento	6	12 [7.5]	
South Iceland 3, Iceland	Thjorsarbru	6.5	15 [9.3]	
Bingol, Turkey	Bingol-Bayindirlik Murlugu	6.3	14 [8.7]	
Parkfield, California	Temblor pre-1969	6	16 [9.9]	
Helena, Montana	Carroll College	6	5 [3.0]	
Montenegro, Yugoslavia	Hercegnovi Novi-O.S.D. Pavicic School	6.9	65 [40.4]	95^{th} Percentile Structural Response Frequency 5–10 Hz
Kocaeli, Turkey	Gezbe	7.4	17 [10.5]	
Kozani 2, Greece	Kozani-Prefecture	6.5	17 [10.5]	
Tabas, Iran	Dayhook	7.3	12 [7.5]	

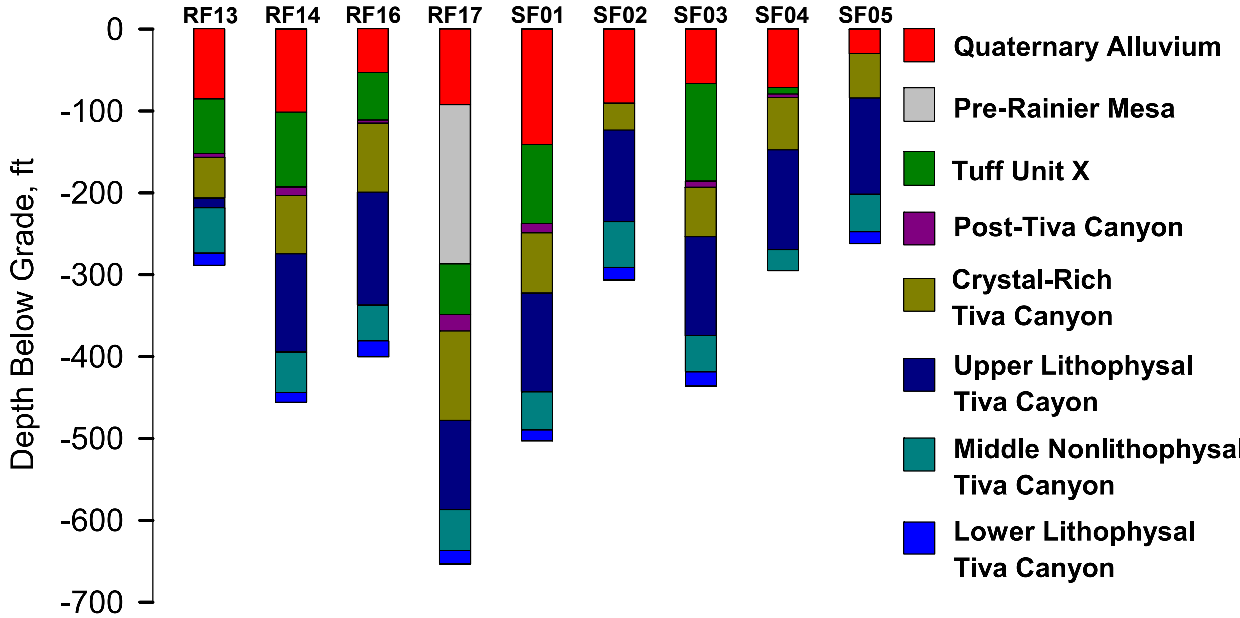


Figure 3-3. Geologic Profiles Used for Site-Response and Soil-Structure Interaction Analyses [1 ft = 0.3048 m]

Table 3-2. Density, Shear-Wave Velocities for Each Unit of the Nine Geologic Profiles

Unit	Unit Weight, kg/m3 [pcf]	S-Wave Velocity, m/s [ft/s]		
		Average − 1 SD	Average‡	Average + 1 SD‡
Quaternary Alluvium	1,852 [115.6]	559 [1,834]	712 [2,336]	865 [2,838]
Pre-Rainier Mesa Tuff	1,816 [113.4]	781 [2,563]	1,011 [3,318]	1,238 [4,063]
Tuff Unit X	1,572 [98.16]	946 [3,105]	1,063 [3,489]	1,180 [3,873]
Post-Tiva Canyon Tuff	1,789 [111.7]	809 [2,653]	1,017 [3,335]	1,224 [4,017]
Crystal-Rich Tiva Canyon Tuff	1,855 [115.8]	781 [2,563]	1,010 [3,313]	1,238 [4,063]
Upper Lithophysal Tiva Canyon Tuff	2,070 [129.2]	757 [2,483]	1,132 [3,713]	1,506 [4,942]
Middle Nonlithophysal Tiva Canyon	2,321 [144.9]	1,117 [3,666]	1,583 [5,193]	2,048 [6,720]
Lower Lithophysal Tiva Canyon	2,207 [137.8]	1,234 [4,050]	1,639 [5,378]	2,044 [6,706]
Lower Nonlithophysal Tiva Canyon*	2,127 [132.8]	1,528 [5,012]	1,894 [6,213]	2,260 [7,415]

†SD: standard deviation
‡Data from Gonzalez, S., A. Ghosh, J. Stamatakos, K. Murphy, and L. McKague. "Review of Scientific Analysis Document—Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project." Scientific Notebook No. 644. San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2004.
*Unit used as a half-space in the site-response and the soil-structure interaction analyses

3.4 Free-Field Ground Acceleration Time Histories

To generate the free-field acceleration time histories at the ground surface for soil-structure interaction analyses, ProShake® Version 1.111 (EduPro Civil Systems, 2001) was used to perform the one-dimensional equivalent-linear site-response modeling. The equivalent-linear model approximates nonlinear site response by iteratively adjusting the stiffness and damping parameters of the soil or rock layers until they are compatible with the strain levels induced by the earthquake loading (Kramer, 1996; Lo Presti, et al., 2006). ProShake calculates the response of a horizontally layered soil deposit overlying a uniform half-space subjected to vertically propagating shear waves (Schnabel, et al., 1972). The 24 spectrally matched acceleration time histories were used as inputs to the site response analyses for each geologic profile shown in Figure 3-3 with the three S-wave velocity sets listed in Table 3-2.

To conduct the site-response analyses, the upper mean alluvium and upper mean tuff shear modulus reduction and damping curves in Bechtel SAIC Company, LLC (2004) were used as inputs to the site-response models. The upper mean alluvium and upper mean tuff shear modulus reduction and damping curves were selected for the calculations because the site-response modeling results presented in Bechtel SAIC Company, LLC (2004) indicate that this combination of curves generally produced larger ground motions than other combinations of curves. These modeling results also show that, depending on frequency, alluvium thickness is an important factor.

4 SOIL-STRUCTURE INTERACTION ANALYSES

4.1 Model of a Hypothetical Waste Handling Facility

4.1.1 Facility Description

For the soil-structure interaction analyses, a SASSI (Lysmer, et al., 1999a) model simulating a hypothetical waste handling facility (Figure 2-1) was developed. This facility consists of

(i) A ground floor area 64 m [210 ft] long in the east-west direction and 47.1 m [154.5 ft] wide in the north-south direction

(ii) An operating floor located at the northern portion of the facility at a height of 8.2 m [27 ft] above the ground floor

(iii) A partial mezzanine floor located at the northeast portion of the facility and located 13.7 m [45 ft] above the ground floor

(iv) A rooftop at 19.5 m [64 ft] above the ground level

(v) An enclosed rectangular tower located at the southeast corner of the facility with its rooftop at the 25.6-m [84-ft] level

Figure 4-1 shows the top and cross-sectional views of this hypothetical facility. The basemat of this hypothetical facility is 3.05 m [10 ft] thick, and the top of the basemat is at ground level. The thickness assumed for the east-west walls is 1.37 m [4.50 ft], south-north walls is 1.52 m [5.00 ft], north outer wall is 1.22 m [4.00 ft], operating and mezzanine floors is 0.38 m [1.25 ft], roof at the 19.5 m [64 ft] level is 0.99 m [3.25 ft], and roof at the 25.6 m [84 ft] Level is 0.76 m [2.50 ft].

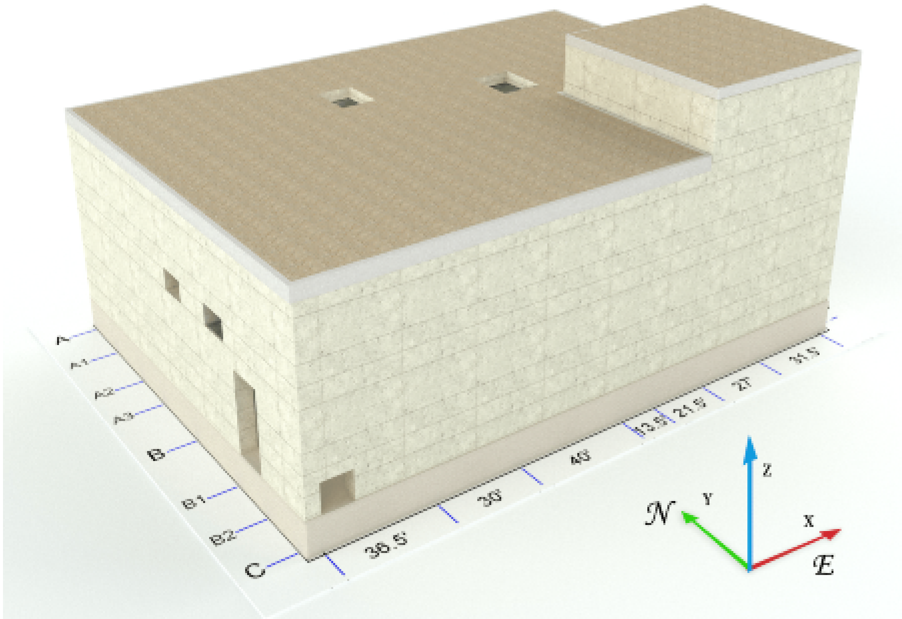
4.1.2 Structural Model

The hypothetical waste handling facility was modeled as a three-dimensional structure. The SASSI structural model consisted of 9,101 nodes and 5,132 elements. Out of the 5,132 elements, 594 elements represent the excavated soil and 4,538 elements represent the structure. For the element nodes located above grade, six degrees of freedom of motions were permitted. The structure was assumed to be made of concrete with a unit weight of 2,403 kg/m^3 [150 pcf]. The Young’s modulus and Poisson ratio assumed for the concrete were 24,821 MPa [5.184×10^5 ksf] and 0.2, respectively.

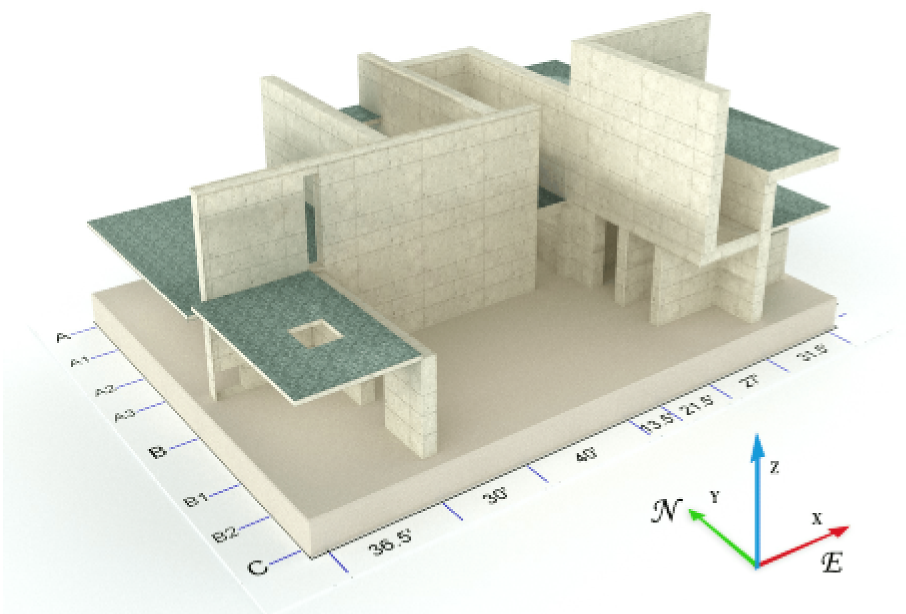
The excavated soil was modeled using eight-node rectangular solid elements. Most solid element dimensions were controlled by the physical dimensions of the components of the structure above. Otherwise, the maximum dimension was set to ensure accurate transmission of the seismic waves.

4.2 Soil Model

The soil-structure interaction analyses were conducted using the nine different geologic profiles shown in Figure 3-3. For each geologic profile, the soil-structure interaction analysis considered



(a) Full View



(b) Cutaway View With Outer Walls and Roofs Removed

Figure 4-1. Full and Cutaway Views of the Hypothetical Waste Handling Facility

three soil stiffness variations. Each layer in a geologic profile was subdivided into smaller layers as necessary to satisfy the maximum permissible layer thickness recommended in the SASSI 2000 computer program to adequately transmit the waves. The maximum thickness permissible for a soil layer should be small enough to capture the particle motion associated with high frequency vibrations (e.g., Ofoegbu and Gute, 2002). Lysmer and Kuhlemeyer (1969) show that a better than 10-percent solution accuracy can be obtained if the layer thickness is less than or equal to (i) one-eighth of the shortest wavelength of interest if either lumped or consistent mass matrix is used in the analysis or (ii) one-fifth of the shortest wavelength of interest if a mixture of the lumped and consistent mass matrix is used in the analysis.

The SASSI 2000 computer program calculates the mass matrices using a combination of half consistent mass matrix and half lump mass matrix (Lysmer, et al., 1999b). Consequently, in determining the maximum soil-layer thickness, it is sufficient to use the recommended one-fifth of the shortest wavelength of interest. Lysmer, et al. (1999b) indicate that larger soil layer thickness can be used in soil zones with higher S-wave velocities.

To be effective, if a soil-layer thickness exceeds the permissible thickness, the soil layer should be subdivided into several sublayers. The same maximum permissible thickness should also be applicable to the finite element model of the structure sharing the interaction nodes with the surrounding soil. Given that the largest element dimension used in the soil-structure interaction analyses was 3.89 m [12.75 ft], the cutoff frequency was approximately 29 Hz for the soft soil cases, 37 Hz for the average soil cases, and 45 Hz for the stiff soil cases.

For each geologic column, five extra layers and a set of viscous dashpots were added at the bottom of the column to simulate the half space condition. In the SASSI 2000, the total thickness of the added layers is 1.5 times the shear-wave length of the half space (Lysmer, et al., 1999a,b). The choice of this thickness is intended to minimize the potential effects of the fundamental mode Rayleigh waves because this fundamental mode decays rapidly with depth. In addition, at the surface, the Rayleigh wave essentially vanishes after a distance of 1.5 times wavelengths (Lysmer, et al., 1999b). The material properties used in this study for the extra layers were the same as those for the layer located at the bottom of the column (Table 3-2).

4.3 Input Ground Motions

As discussed previously, the free-field acceleration time histories CNWRA developed using the 24 spectrally matched acceleration time histories (Section 3.2) for the soil profiles, including consideration of soil property variations through site-response modeling, were used as the input ground motions for soil-structure interaction analyses.

Besides the free-field acceleration time histories generated from the 24 spectrally matched acceleration time histories, the five sets of free-field acceleration time histories developed in Bechtel SAIC Company, LLC (2004) for the design-basis response spectrum at Point D/E of Figure 3-1 were also used in the soil-structure interaction analyses. The design-basis response spectrum was consistent with the hazard level of 5×10^{-4} annual exceedence frequency. In developing the design-basis response spectrum for seismic design, Bechtel SAIC Company, LLC (2004) used a one-dimensional equivalent-linear approach similar to that of CNWRA to calculate site responses of the soil and rock beneath the site. The design-basis response spectrum represents the envelope of the individual site response analyses results. An important distinction between the site response analyses approach CNWRA used and that Bechtel SAIC Company, LLC used was that CNWRA considered lithology when developing velocity, density, and layer thickness profiles (see Figure 3-3 and Table 3-2); whereas, Bechtel SAIC Company, LLC incorporated randomized velocity/thickness (Hsiung, et al., 2005). Each set of the acceleration time histories consists of two horizontal and one vertical acceleration time history except Set 4, which consists of only one horizontal and one vertical acceleration time history. All nine horizontal acceleration time histories were used as the free-field ground motions for the soil-structure interaction analyses presented in this report.

For convenience, the input ground motions using the free-field acceleration time histories generated from the 24 spectrally matched acceleration time histories are referred to as Set A ground motions and the input ground motions using the nine free-field acceleration time histories developed in Bechtel SAIC Company, LLC (2004) are referred to as Set B ground motions.

The input ground motions were specified at the ground surface and applied to the soil-structural model along the east-west direction. A uniform damping of 4 percent was assumed for the structure and basemat.

5 RESULTS AND DISCUSSIONS

Three primary factors control soil-structure interactions: (i) soil geometry (geologic profile geometry), (ii) soil material property, and (iii) ground motion. Generally, the influence of these three factors is complex. It is often difficult to evaluate the effects of one factor on the dynamic response of a given structure independent of the others. The structural response results from the soil-structure interaction analyses discussed in the following sections are measured along the horizontal east-west direction. For convenience, the terms "soil profiles" and "geologic profiles" are used interchangeably in this chapter.

5.1 Effects of Soil Profiles

As discussed previously, nine geologic profiles were considered in these soil-structure interaction analyses. The wide variation in thickness of each unit or layer in the nine geologic profiles (Figure 3-3) should be sufficient to allow qualitative assessment of the potential influence of spatial variation of geological medium on dynamic structural responses. Figure 5-1 shows the envelope structural responses at the tower-top level of the hypothetical waste handling subjected to the Set A earthquake ground motions developed from the six deaggregation response spectra by CNWRA. The tower top discussed here is the rooftop of the tower located at the southeast corner of the facility at the 25.6-m [84-ft] level. The envelope response spectra discussed in this section represent the largest spectral acceleration at each frequency from the results of the soil-structure interaction analysis cases for a geologic profile of interest.

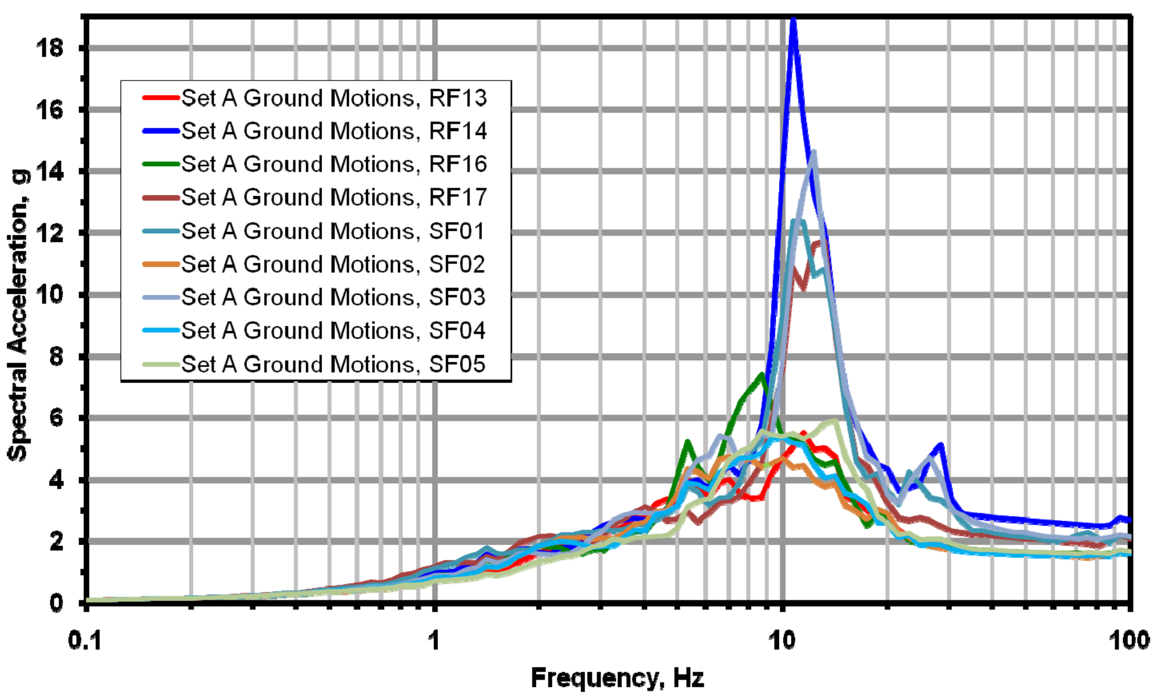


Figure 5-1. Envelope Spectral Responses at Tower-Top for Set A Ground Motions for the Nine Geologic Profiles

Figure 5-1 indicates that geologic profile (spatial) variations at the hypothetical surface facility site could cause significant variation in spectral responses of the hypothetical waste handling facility. The soil-structure system under investigation appears to exhibit two types of responses to seismic ground motions. For the first type (type 1), the response spectral acceleration peaks are in the frequency range of 5–14 Hz. For the second type (type 2), the response spectral acceleration peaks are in the frequency ranges of 9–17 Hz with much larger spectral amplitudes than those observed for the first type. The type 2 responses are the results of the soil-structure system resonance. The type 2 responses may be observed for the hypothetical waste handling facility located on the RF14, RF17, SF01, and SF03 (Group 2) geologic profiles, but not observed at any in-structure locations of the hypothetical waste handling facility for the RF13, RF16, SF02, SF04, and SF05 (Group 1) geologic profile cases. It appears that the soil-structure system resonates with the Group 2 geologic profiles for Set A ground motions in the 9–17 Hz frequency range. Similar observations can also be made for the spectral responses at the operating floor level, which is at a height of 8.2 m [27 ft] above the ground floor with smaller spectral acceleration amplitudes (Figure 5-2).

The effects of geologic variations on spectral responses at the tower top and on the operating floor for the hypothetical waste handling facility subjected to Set B ground motions developed in Bechtel SAIC Company, LLC (2004) show the same behavior as observed for Set A ground motions (Figure 5-3). The tower top and operating floor resonate with Group 2 geologic profiles for the Set B ground motions in the 9–17 Hz frequency range. As expected, the spectral

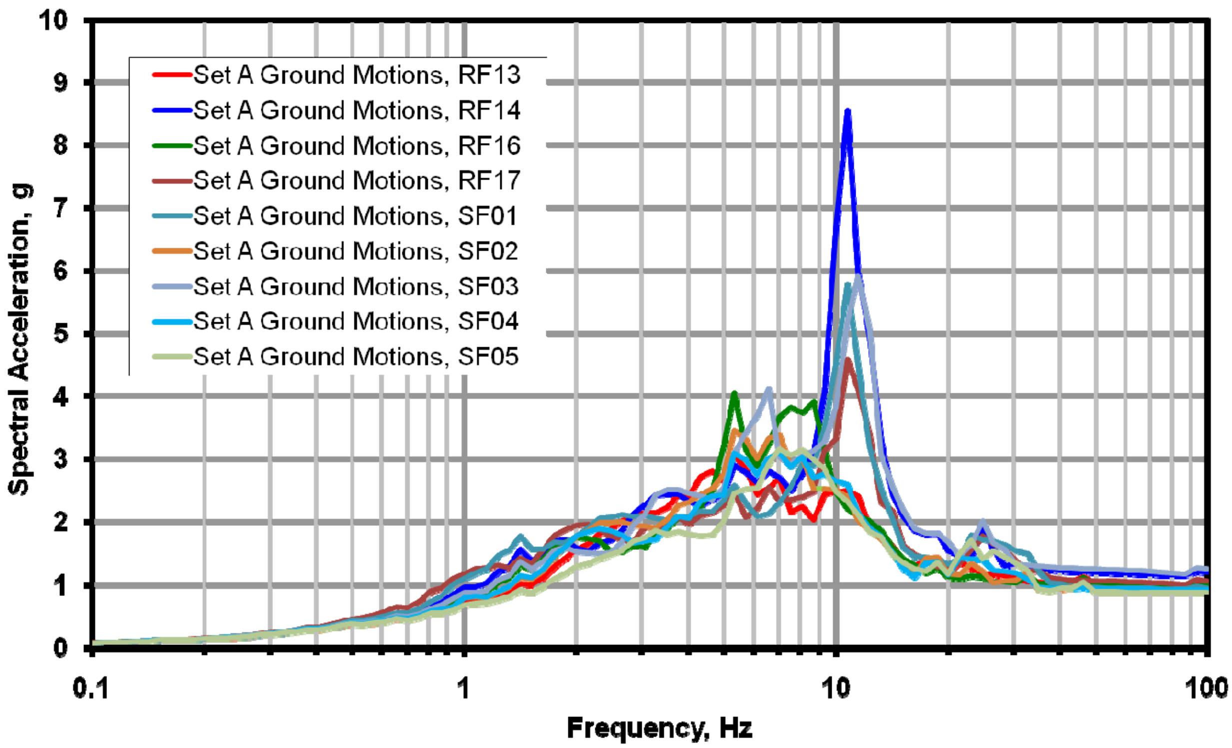
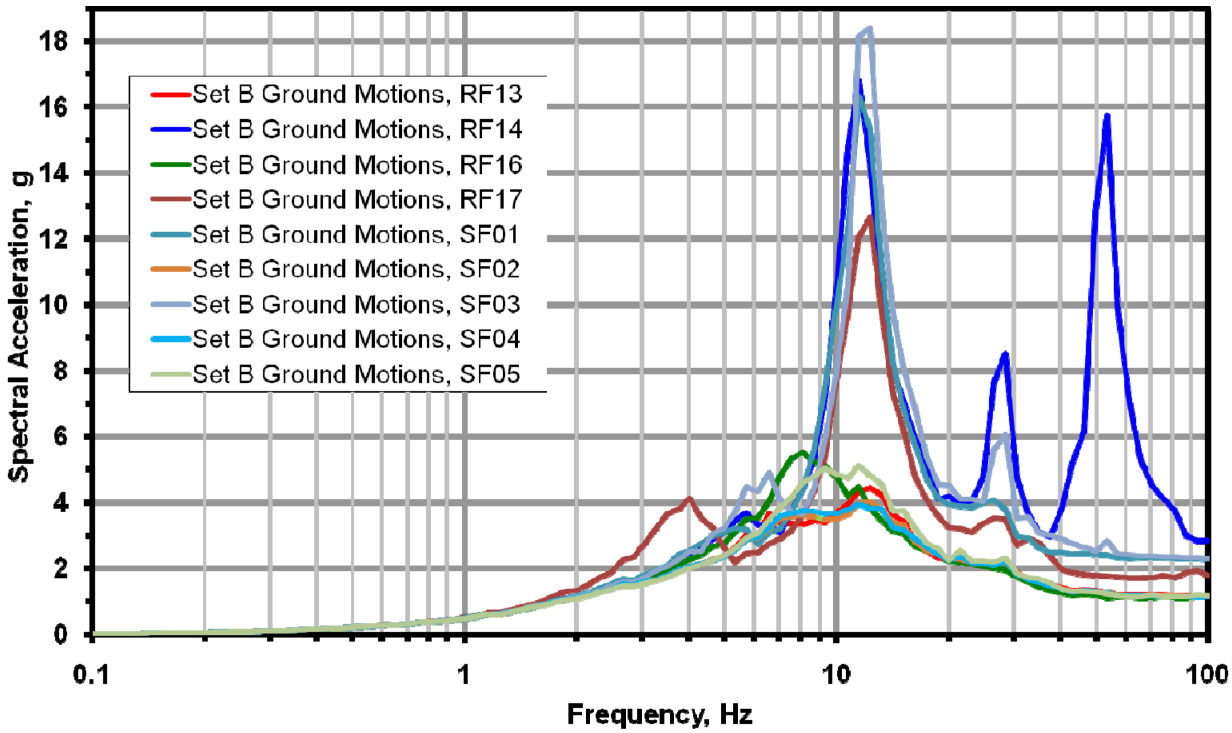
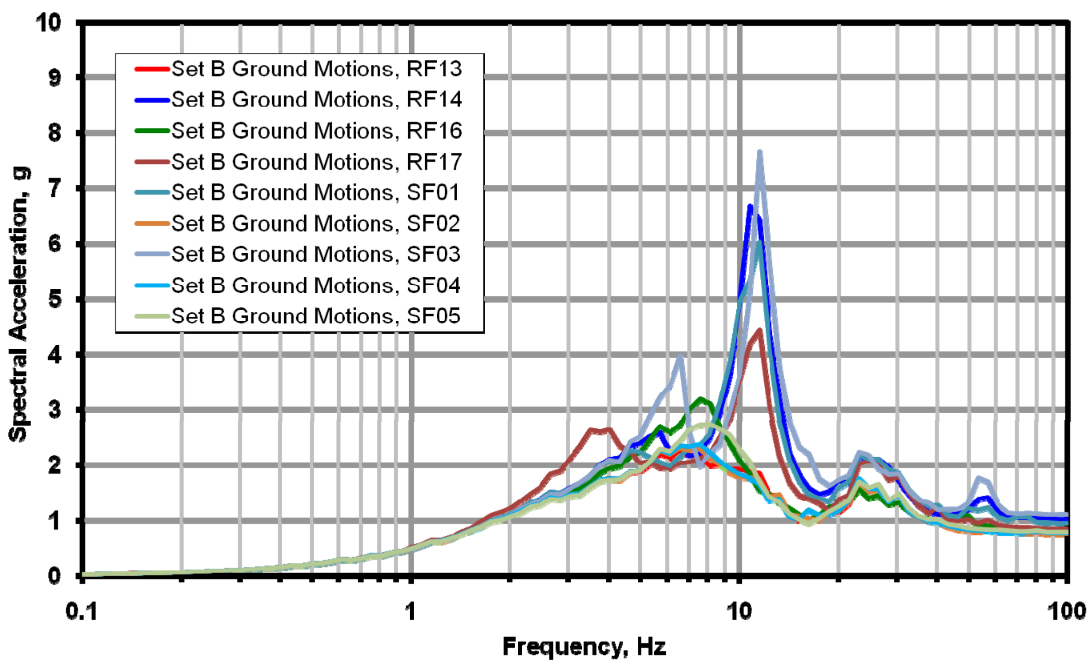


Figure 5-2. Envelope Operating Floor Spectral Responses for Set A Ground Motions for the Nine Geologic Profiles



(a) Tower Top



(b) Operating Floor

Figure 5-3. Envelope Spectral Responses for Set B Ground Motions for the Nine Geologic Profiles

acceleration peaks for the operating floor are smaller than those at the tower top. For the Set B ground motion case, the SF03 geologic profile produces the largest spectral response in the 9–17 Hz frequency range; it is the RF14 geologic profile for the Set A ground motions.

For the Set B ground motion cases, two other soil-structure resonances with large local spectral acceleration peaks can also be observed on the tower top [Figure 5-3(a)] at the 25–30 Hz and 45–65 Hz frequency ranges for the RF14 geologic profile case. The resonance behavior at the 25–30 Hz frequency range can also be observed for the SF03 geologic profile case with relatively smaller spectral acceleration amplitude. The local spectral peak in the 25–30 Hz frequency range exists also for the cases related to the Set A ground motion cases, however, with negligible acceleration values (Figure 5-1). The resonance at the 45–65 Hz frequency range for the Set B ground motion case is not shown in the results of the Set A ground motion cases and has a high spectral amplitude of 15.75 g. This particular high spectral amplitude is associated with one time history in the Set B ground motion group. If the results for the time history that caused this high local peak are removed from Figure 5-3(a), a local spectral peak in the 45–65 Hz frequency range still exists, however, with a much smaller spectral amplitude (Figure 5-4). Examining the response spectral results for the RF14 geologic profile cases, it was found that more than half of the acceleration time histories in the Set B ground motion group produced resonance in the 45–65 Hz frequency range, with local spectral peaks ranging from 3.5–6.9 g besides the 15.8 g case mentioned previously. To understand why the tower of the hypothetical waste handling facility resonates at the 45–65 Hz frequency range with this high acceleration peak for only the RF14 geologic profile and the Set B ground motions, understanding the natural frequencies of the structure and the associated modal participation factors may offer valuable insights.

The thickness of the Quaternary Alluvium top soil unit alone does not appear to explain the phenomena of the seismic responses of the hypothetical waste handling facility on Group 2 geologic profiles that are different from those on Group 1 geologic profiles. As shown in Table 3-2, the Quaternary Alluvium unit has the lowest S-wave velocity. The S-wave velocity for the Pre-Rainier Mesa, Tuff Unit X, Post-Tiva Canyon, crystal-rich Tiva Canyon, and Upper Lithophysal Tiva Canyon tuff units are approximately 35 to 74 percent higher than those of the Quaternary alluvium. The S-wave velocities for the geologic units beneath the Upper Lithophysal Tiva Canyon tuff unit are more than twice those of the alluvium unit.

The thickness of the alluvium unit for Group 1 geologic profiles varies from 9.1 to 27.7 m [29.8 to 90.9 ft], with the thinner alluvium unit from the SF05 geologic profile and the thicker one from the SF02 geologic profile. The alluvium thickness for Group 2 geologic profiles varies from 20.5 m [67.1 ft] for the SF03 geologic profile to 42.9 m [140.7 ft] for the SF01 geologic profile (Figure 3-3). Notice there is overlap between the two thickness ranges. For example, the alluvium thickness is 26.1 m [85.5 ft] for the RF13 geologic profile, 27.7 m [90.9 ft] for the SF02 geologic profile, and 21.8 m [71.5 ft] for the SF04 geologic profile. These thicknesses are larger than the alluvium thickness {20.5 m [67.1 ft]} associated with the SF03 geologic profile. However, the resonance structural responses found at the tower top and on the operating floor in the 9–17 Hz frequency range for the SF03 geologic profile are not observed for the RF13, SF02, or SF04 geologic profiles (Figures 5-1 and 5-2). Therefore, even though the thickness range for Group 2 geologic profiles is relatively larger than that for Group 1 geologic profiles, alluvium thickness does not seem to be the only factor that causes the soil-structure system with the geologic profiles in these two groups to respond differently.

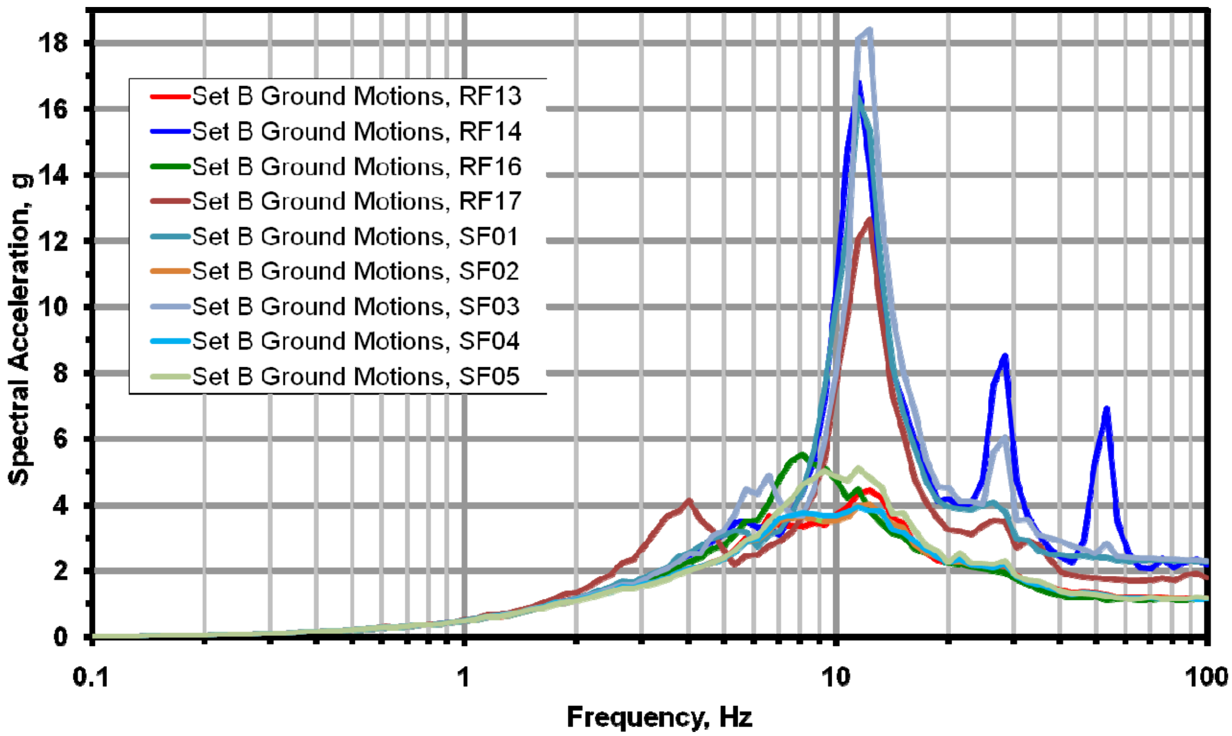


Figure 5-4. Envelope Spectral Responses for Set B Ground Motions for the Nine Geologic Profiles Except the Ground Motion that Results High Spectral Acceleration in the 45–60 Frequency Range

Among the nine geologic profiles, the Pre-Rainier Mesa Tuff unit is present only in the RF17 geologic profile. According to Figures 5-1 and 5-2, this unit does not appear to have a strong effect on soil-structure interaction. Table 3-2 shows that the Pre-Rainier Mesa Tuff, Tuff Unit X, Post-Tiva Canyon Tuff, and the Crystal-Rich Tiva Canyon Tuff units beneath the alluvium soil appear to have similar S-wave velocities. The total thickness of these four units varies from 16.7 to 44.6 m [54.7 to 146.3 ft] for the Group 1 geologic profiles and from 52.8 to 117.5 m [173.2 to 385.6 ft] for Group 2 geologic profiles (Figure 3-3). The total thickness of the four units in Group 2 geologic profiles is greater than that in Group 1 geologic profiles. Furthermore, the total thickness of these four units and the alluvium unit in Group 2 geologic profiles is greater than that in Group 1 geologic profiles. The largest total thickness is 63.0 m [206.6 ft] for the Group 1 geologic profiles and the smallest thickness for Group 2 geologic profiles is 77.4 m [253.8 ft]. The combined effect of the alluvium unit and the four units immediately beneath it appears to be responsible for the soil-structure resonance behavior observed for Group 2 geologic profiles.

A thicker alluvium unit or a thicker combined first five units does not necessarily cause a higher resonance response in the 9–17 Hz frequency range for Group 2 geologic profiles. For Group 2 geologic profiles, the thickness of the alluvium unit is 42.9 m [140.7 ft] for the SF01 geologic profile, 31.0 m [101.8 ft] for the RF14 geologic profile, 28.2 m [92.4 ft] for the RF17 geologic profile, and 20.5 m [67.1 ft] for the SF03 geologic profile (Table 3-2). Even though the alluvium unit for the SF01 geologic profile is the thickest among the four geologic profiles, the largest peak spectral accelerations observed for the tower top and operating floor are with the RF14 geologic profile for the Set A ground motion case. For the Set B ground motion case, the largest tower and operating floor peak spectral accelerations are from the SF03 geologic profile, for which the thickness of the alluvium unit is the thinnest among the four Group 2 geologic profiles.

Figure 5-5 shows the mean response spectra at the tower top for both Set A and Set B ground motions. Similar observations made for the envelope in-structure response spectra are also applicable to the mean response spectra. Notice that the mean response spectra are much smaller than the corresponding envelope response spectra with the Group 2 geologic profiles for the Set A ground motion case. This observation suggests that a large number of the acceleration time histories in Set A ground motions generated small spectra accelerations in the 9–17 Hz frequency range for Group 2 geologic profiles. The median tower top spectral response plot in Figure 5-6 supports this observation. Figure 5-6 indicates that more than half of Set A ground motions generate smaller spectral response in the 9–17 Hz frequency range for Group 2 geologic profiles, such that the median is smaller than the mean as shown in Figure 5-5(a).

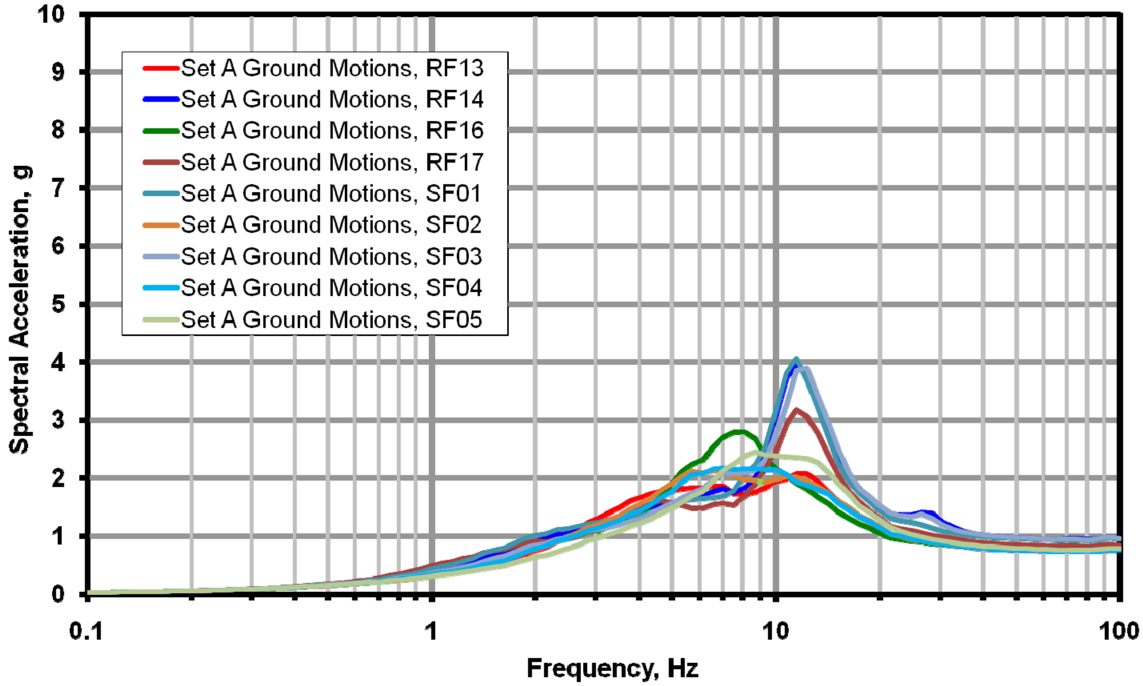
The mean response spectra on the operating floor also exhibit structural resonances in the 9–17 Hz frequency range with much smaller acceleration amplitudes than those at the tower top. Additionally, for the Set A ground motion case, the amplitudes of the mean resonant response spectra on the operating floor for the Group 2 geologic profiles are not any larger than other mean response spectra for the Group 1 geologic profiles.

The envelope and mean in-structure response spectra for the nine geologic profiles represent a broadened structural response (Figures 5-1, 5-2, 5-3, and 5-5) (i.e., the frequency content containing the spectral acceleration peaks for the resulting envelope has been widened). In other words, soil profile variation has a broadening effect on the structural response spectra. The broadening effect of soil profile variation on the in-structure response spectra is observed for all earthquake input conditions. For the individual response spectra in the envelope, the response spectra corresponding to a geologic profile may be considered to have a spectrum shift with respect to the response spectra of another geologic profile if the frequency contents for the spectral acceleration peaks are not the same. For example, the response spectra for the SF01 geologic profile shift to a higher frequency range relative to the response spectra for the RF13 geologic profile because the frequency content for the latter is lower.

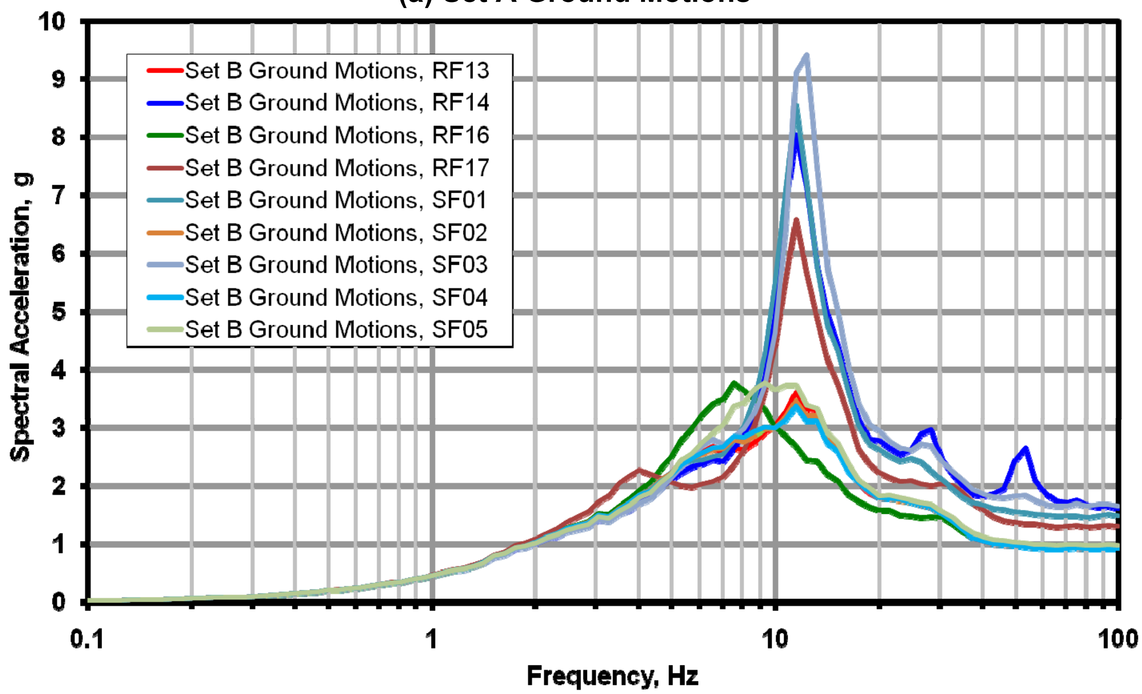
As discussed previously, the soil-structure interaction analyses results suggest that, if the hypothetical waste handling facility were to be located on the Group 2 geologic profiles, it would experience high spectral accelerations at a number of resonant frequencies. These high seismic responses may be reduced if necessary through several engineering measures. These measures include (i) avoiding locating the structure in areas with a subsurface geological medium similar to those of Group 2 geologic profiles, (ii) conducting ground improvements in areas with a subsurface geological medium similar to those of Group 2 geologic profiles, and (iii) modifying the design of the structure to change natural frequencies to prevent or mitigate resonance.

5.2 Effects of Soil Material Variation

Figure 5-7 compares the envelope structural seismic responses caused by Sets A and B ground motions at the tower top resulting from soil material variations. The envelope response spectra discussed in this section represent the largest spectral acceleration for all nine geologic profiles at each frequency from the results of the soil-structure interaction, analysis cases under a



(a) Set A Ground Motions



(b) Set B Ground Motions

Figure 5-5. Mean Tower-Top Spectral Responses for Set A and B Ground Motions for the Nine Geologic Profiles

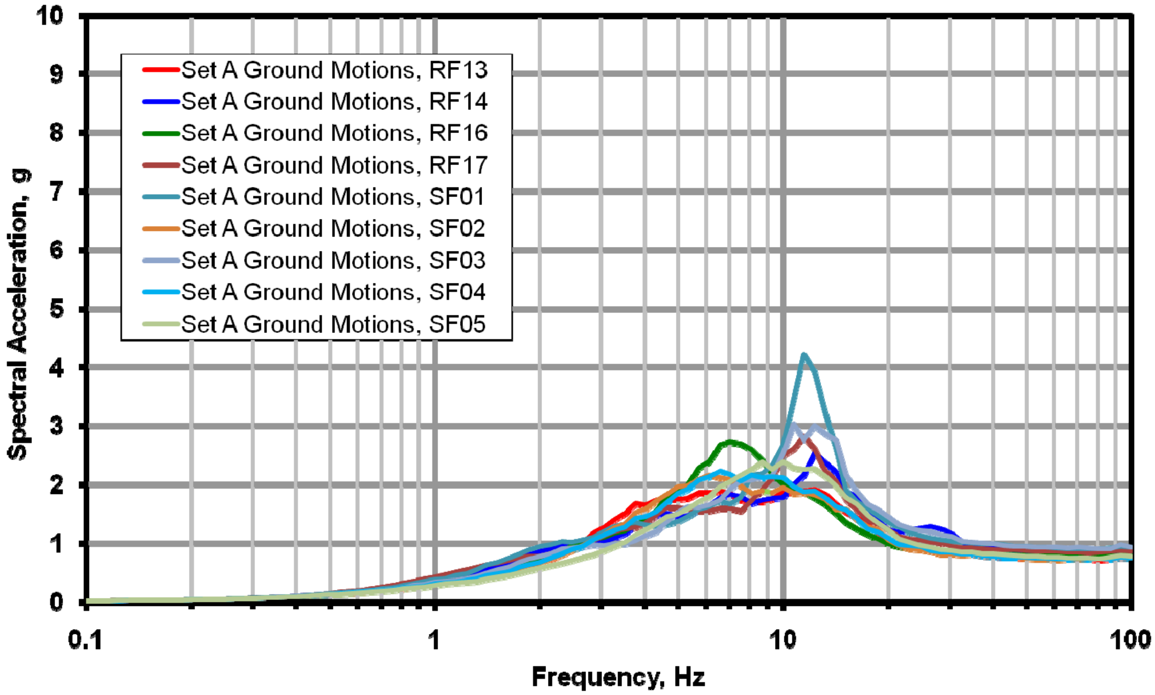
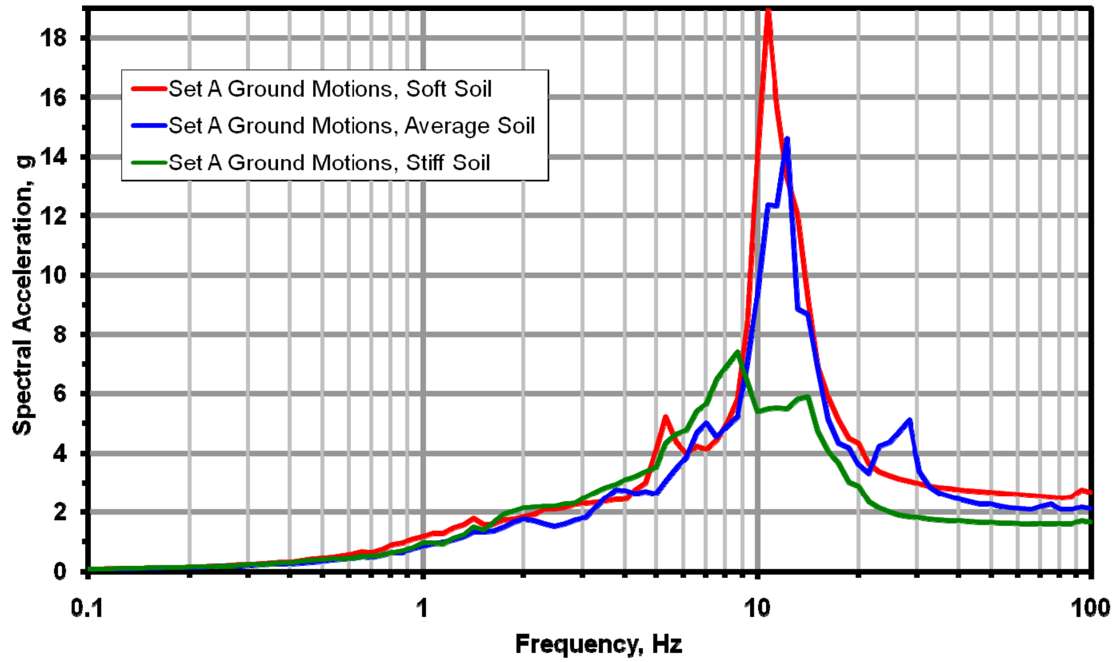


Figure 5-6. Median Tower-Top Spectral Responses for Set A Ground Motions for the Nine Geologic Profiles

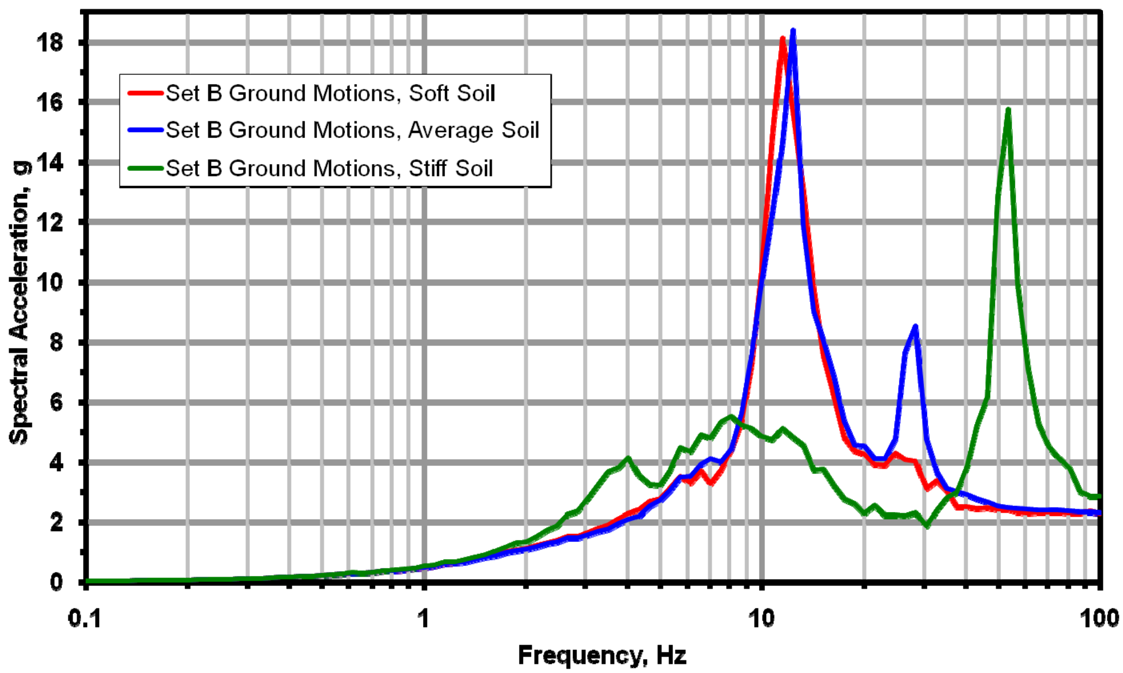
particular soil material condition of interest. Note that the soft soil referred to in the caption of the figure used the average minus one standard deviation S-wave velocities as input as discussed in Section 3.3 of this report, the average soil used the average S-wave velocities, and the stiff soil used the average plus one standard deviation S-wave velocities. In the previous section, the structural resonance behavior was discussed for Group 2 geologic profiles in the 9–17 Hz frequency range. This observation needs to be conditioned based on the results shown in Figure 5-7. According to Figure 5-7, resonance behavior is only for the soft and average soil conditions and not for the stiff soil condition, irrespective of the soil profile variations.

The two soil-structure resonances with large spectral acceleration peaks at frequency ranges of 25–30 Hz and 45–65 Hz for Set B ground motions, as discussed in the previous section, are also shown in Figure 5-7(b). Figure 5-7(b) indicates that, for Set B ground motions, the resonance at the 25–30 Hz frequency range is related to the average soil condition and the resonance in the 45–65 Hz frequency range is associated with the stiff soil condition. As discussed previously, the resonance in the 25–30 Hz frequency range is specific to the RF14 and SF03 geologic profiles, while the resonance in the 45–65 Hz frequency range is specific to the RF14 geologic profile only. The spectral acceleration responses for Set A ground motions also exhibit the resonance behavior [Figure 5-7(b)] with much smaller spectral acceleration amplitude compared to those for the Set B ground motion case. The resonance behavior in the 45–65 Hz frequency range is not observed for any geologic profiles subjected to the Set A ground motions.

The two resonances at the high frequency ranges (higher than 20 Hz) mentioned in the previous paragraph are not obvious from the results of mean tower-top response spectra (Figure 5-8).

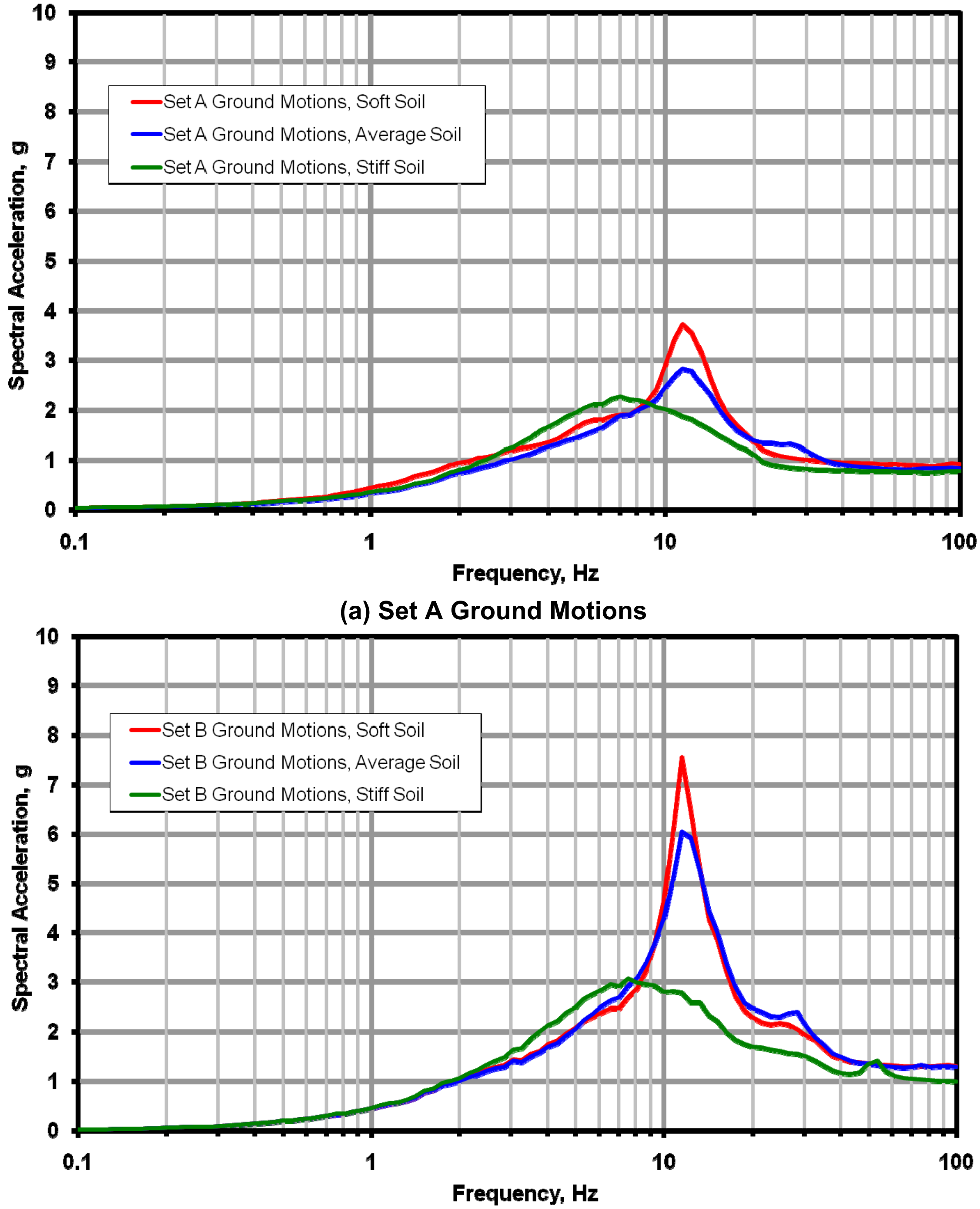


(a) Set A Ground Motions



(b) Set B Ground Motions

Figure 5-7. Envelope Tower-Top Spectral Responses for Set A and B Ground Motions for the Three Soil Conditions



(a) Set A Ground Motions

(b) Set B Ground Motions

Figure 5-8. Mean Tower-Top Spectral Responses for Set A and B Ground Motions for the Three Soil Conditions

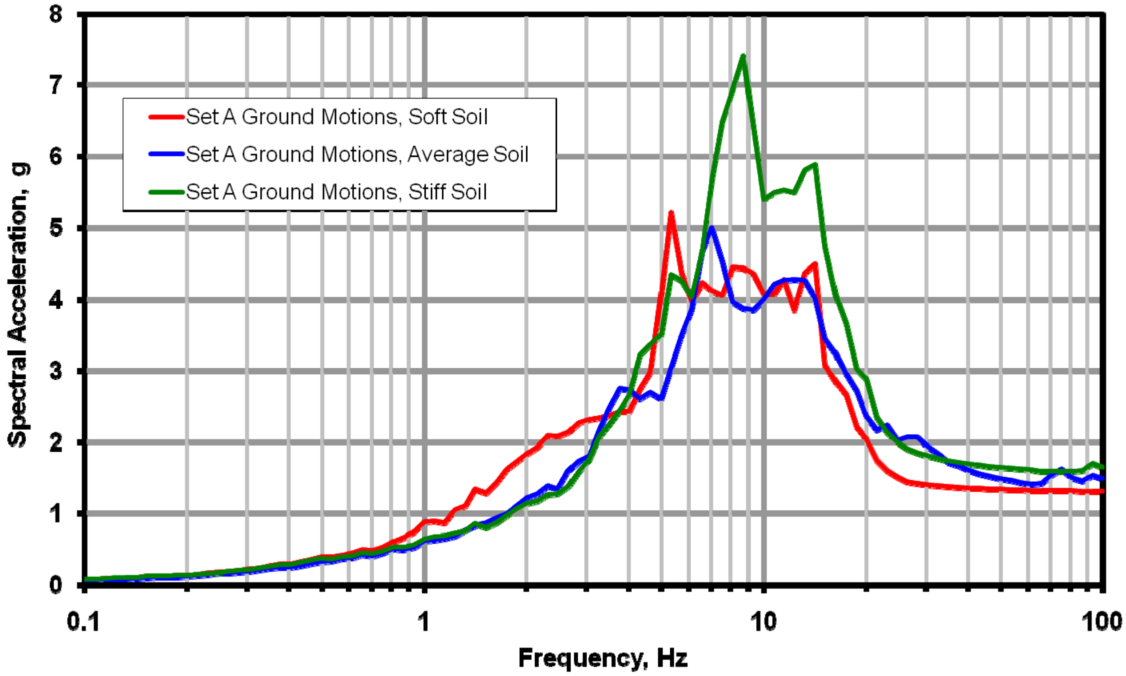
The mean response spectra for both Set A and B ground motions show type 2 resonant behavior in the 9–17 Hz frequency range for the soft and average soil conditions. This resonant behavior is not apparent in the mean response spectra for the stiff soil condition for either ground motion set. In addition, the spectral acceleration peaks at the resonant frequency ranges are the highest for the soft soil condition and smallest for the stiff soil condition. This trend is not as clear for the envelope tower-top spectral acceleration peaks subject to the Set B ground motions, for which the spectral acceleration peaks for the soft and average soil conditions are comparable [(Figure 5-7(b)].

Consistent with the observation on the effects of spatial variation of areal geology, the structural systems located on the Group 1 geologic profiles show no resonance responses at the tower top for all soil material conditions. For these cases, the frequency contents of the spectral acceleration peaks tend to shift to the higher frequency range as the soils become stiffer (Figure 5-9). In addition, the spectral acceleration peaks are higher for the relatively stiffer soils. This trend is consistent irrespective of the geologic profiles used and free-field ground motions applied. This observation is consistent with the common understanding that the structural response of a stiffer soil-structure system generally has a higher frequency content than that of a softer system. It should also be noted that, for the Group 1 geologic profiles, the response spectral acceleration amplitudes are higher when the soil is stiffer.

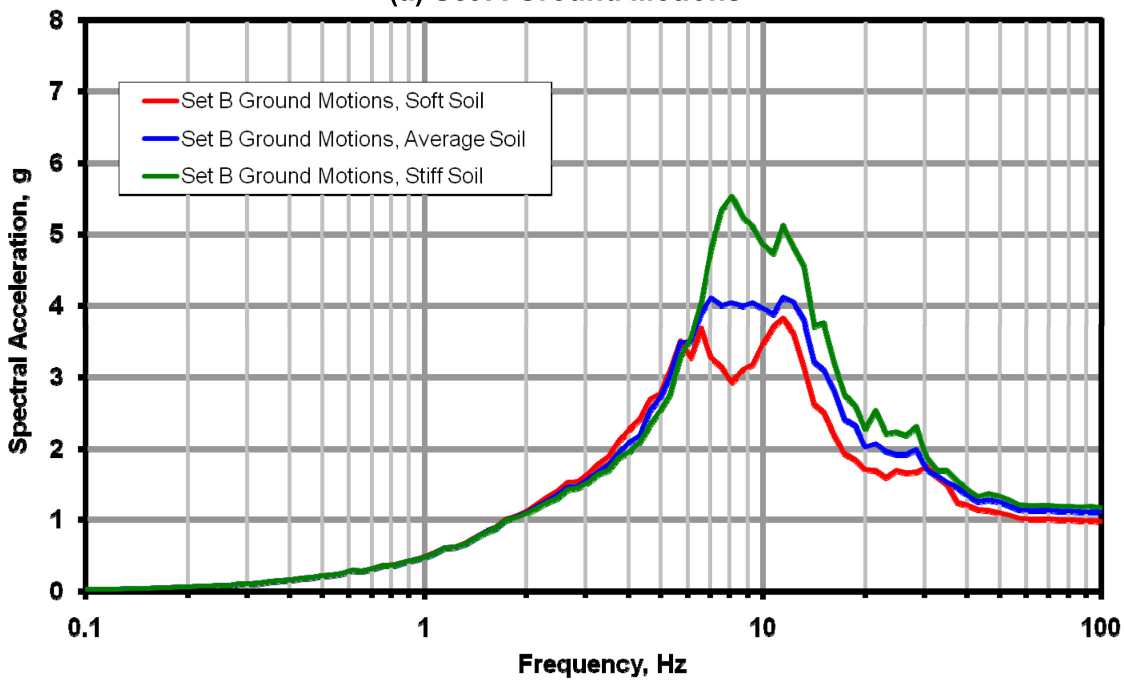
In the case of seismic responses generated by the Set A ground motions, the response spectral peaks are comparable for the average and soft soil conditions [Figure 5-9(a)]. This result may reflect the differences in how the Set A and B ground motions were developed. As discussed in Section 4.3, the free-field acceleration time histories used in the soil-structure interaction analyses for the Set B ground motions were generated by Bechtel SAIC Company, LLC (2004) using the design-basis response spectrum at Point D/E of Figure 3-1 corresponding to the hazard level of 5×10^{-4} annual exceedence frequency. The Set A ground motions, on the other hand, were developed using the six deaggragation response spectra at Point A/B of Figure 3-1 corresponding to the same hazard level of 5×10^{-4} annual exceedence frequency by spectral matching. The spectrally matched time histories were then propagated from Point B through the nine geologic profiles under the three soil condition sets to the ground surface to generate free-field acceleration time histories at the surface. Because of the effects of specific soil conditions (including both variability of areal geology and soil materials), the resulting free-field ground motions were conditioned by the variations of soil profiles and stiffness. As a result, the free-field acceleration ground motion input for each soil-structure interaction analysis case was different. For the Set B ground motions, on the other hand, the same acceleration time history was used for all geologic profiles and soil material conditions in the soil-structure interaction analyses.

The observed spectrum shift caused by the variation of soil materials (stiffness) broadens the response spectrum envelope at the tower top, similar to that observed from soil profile variation. This broadening effect is an important characteristic of a complex and heterogeneous geological medium. In this study, the complex and heterogeneous geological medium is represented by the nine geologic profiles and three sets of soil material conditions. For a facility with a large footprint on such a complex and heterogeneous geological medium, its design needs to be based on a broadened design response spectrum to properly include spatial variability of geological medium.

The soil-structure interaction analysis results show that the dynamic structural response observed for the tower top also occurs in other in-structure locations. The two types of structural responses and the broadening effect caused by the variation of soil stiffness typically



(a) Set A Ground Motions



(b) Set B Ground Motions

Figure 5-9. Envelope Tower-Top Spectral Responses of Group 1 Geologic Profiles for Set A and B Ground Motions for the Three Soil Conditions

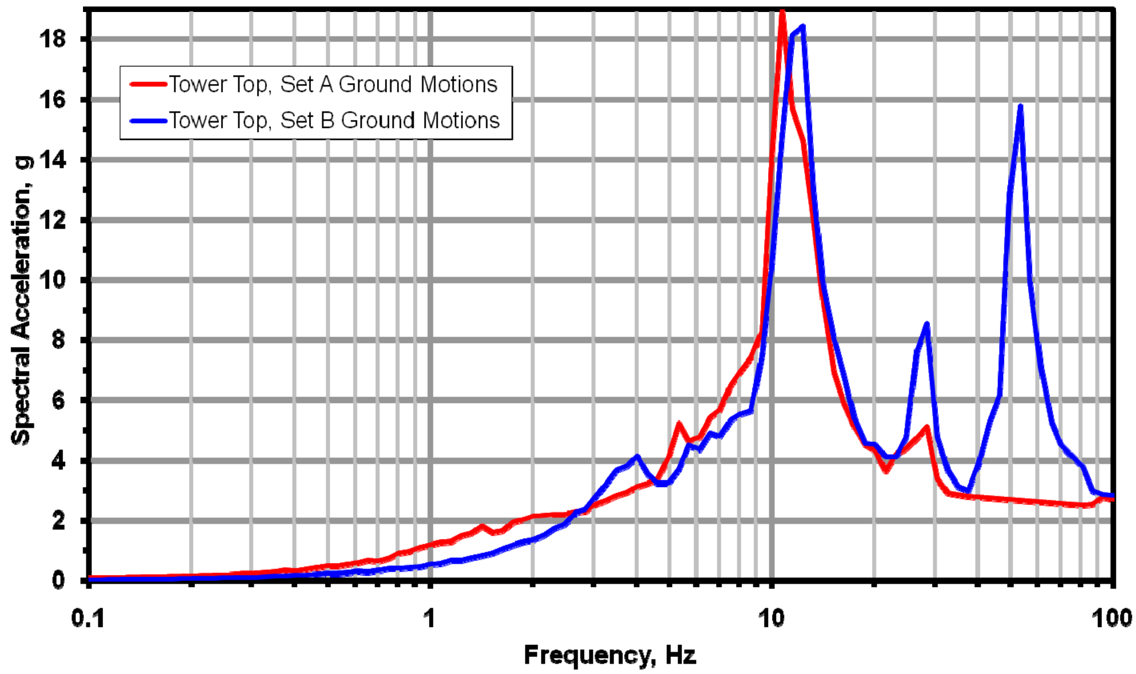
found at the tower top are also observed on the operating floor with only relatively smaller spectral acceleration amplitudes.

The results for the soil material effects provide additional information to identify regions at the site where the grounds could be improved or avoided to reduce effects of soil-structure resonance. For example, the tower-top spectral response envelopes for Set A and B ground motions in Figure 5-7 suggest that only when a structure is to be built on the soft and average soils, ground improvements may need to be considered. There is no need for ground improvement when the structure is to be built on a location with stiff soil.

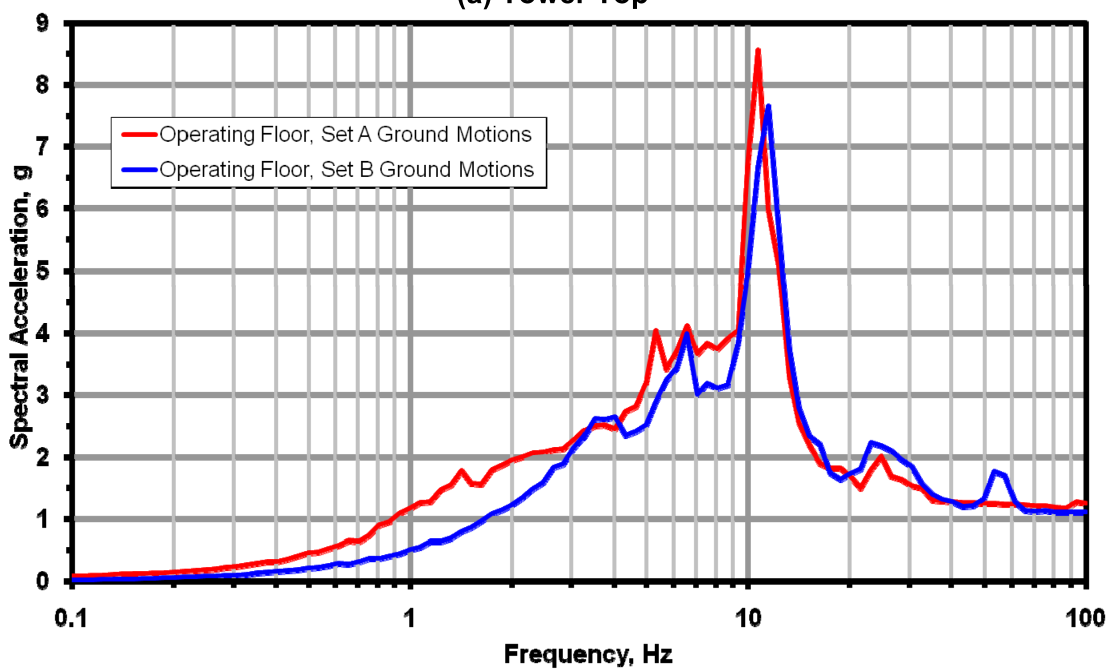
5.3 Comparison of Set A and B Ground Motion Effects

Figure 5-10 displays the response spectral envelopes at the tower top and on the operating floor subjected to both Set A and B ground motions. The envelope response spectra discussed in this section represent the largest spectral acceleration at each frequency from the results of all soil-structure interaction analyses cases, including all geologic profiles and soil material conditions of interest. Both tower-top and operating floor response spectral envelopes for the Set B ground motions appear to slightly shift to a higher frequency range relative to the response spectral envelope for the Set A ground motions. As a result, the response spectral acceleration amplitudes for the Set A ground motion case are slightly larger than those for the Set B ground motion case at the frequencies lower than approximately 11 Hz. In other words, the envelope response spectra for the Set B ground motions developed from the design-basis response spectrum corresponding to the hazard level of 5×10^{-4} annual exceedence frequency do not appear to bound the envelope spectra for the Set A ground motions developed using the six deaggragation response spectra. Recognizing the uncertainties associated with the modeling approaches and the complex geological medium present, the difference between the two envelopes at frequencies below 11 Hz can be considered small; therefore, the two envelopes are in a reasonable agreement in this frequency range.

At frequencies higher than 11 Hz, the Set B ground motions produced much larger spectral accelerations than the Set A ground motions in several frequency ranges. As discussed in Section 5.2, there are two resonances with large spectral acceleration peaks at frequencies higher than 20 Hz. The first resonance in the 25–30 Hz frequency range for the Set B ground motion case is related to the RF14 and SF03 geologic profiles under the average soil condition. The tower top experienced a resonance in the 45–65 Hz frequency range only when the structure was located on the RF14 geologic profile with a stiff soil for Set B ground motions. The Set A ground motion case does display the first resonance in the analysis results. However, the magnitude is not as pronounced as for the Set B ground motion case. For the operating floor, the spectral accelerations are slightly larger for the Set B ground motion case at frequencies higher than 11 Hz and the resonances at the two frequency ranges of 25–30 Hz and 45–65 Hz are negligible. Therefore, it can be concluded that the in-structure response spectral envelopes for the Set B ground motions are in a broad agreement with those for the Set A ground motions at frequencies higher than 11 Hz, with the tower-top envelope of Set B ground motions clearly bounding the tower-top envelope of Set A ground motions. In other words, the design-basis response spectra are appropriate for design of structure and generating in-structure response spectra for structural analysis, design, or qualification of systems and components in the structure if the envelope structural response spectra are intended to be used for these purposes.



(a) Tower Top



(b) Operating Floor

Figure 5-10. Envelope Tower-Top and Operating Floor Spectral Responses for Set A and B Ground Motions

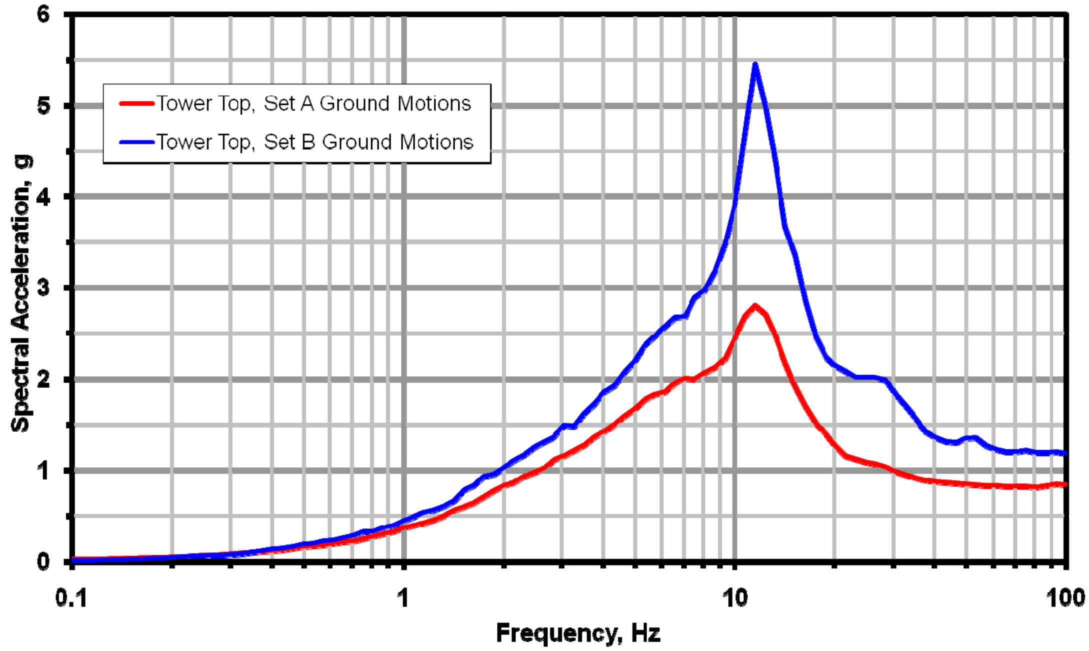
Figure 5-11 shows the mean tower-top and operating floor response spectra for both Set A and B ground motions. The mean response spectra at the tower top for both Set A and B ground motions clearly reflect the dominance of Group 2 geologic profiles for both soft and average soil conditions; the mean response spectra peak in the 9–17 Hz frequency range. Such dominance is not as clear for the mean operating floor response spectra. The mean response spectra associated with the Set B ground motions appear to envelope the response spectra for the Set A ground motions at frequencies larger than 0.4 Hz, and the differences between the two at frequencies smaller than 0.4 Hz are negligible. If the mean in-structure response spectra were to be used as seismic input for structural analysis, design, and qualification of the systems and components located within the structure analyzed, the design-basis response spectrum is more than sufficient for all frequencies of interest.

5.4 Potential Effects of Soil-Model Parameters

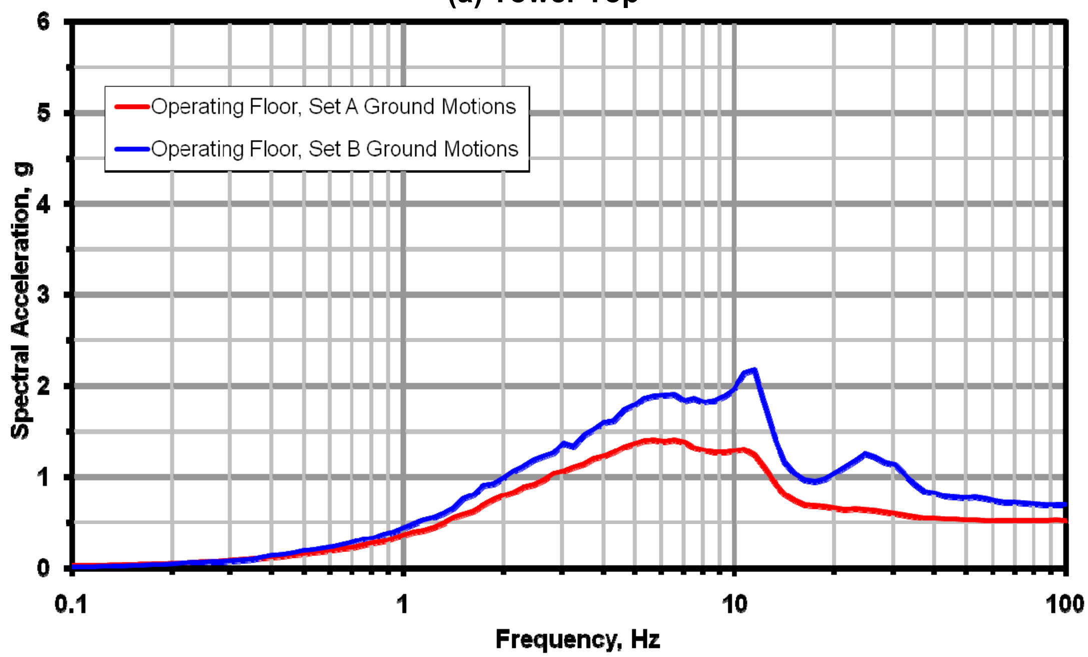
As discussed in Section 4.2, two soil-model parameters potentially affect the soil-structure interaction results: allowable soil-model-layer thickness and number of half-space layers. The allowable soil layer thickness used in the soil-structure interaction analysis should not be larger than one-fifth of the shortest wavelength of interest (Lysmer, et al., 1999a,b). Lysmer, et al. (1999b) further suggest that the maximum model-layer thicknesses can be different for geologic units with different S-wave velocities. Inclusion of the half-space is intended to minimize the potential effects of Rayleigh waves. SASSI 2000 simulates the half-space by adding extra layers to the bottom of a geologic profile with a total thickness 1½ times the shear wavelength of the half space. SASSI 2000 also allows users to select the number of half-space layers for the soil-structure interaction analysis. Lysmer, et al. (1999a) suggest that, for many practical cases, 10 extra half-space layers are sufficient. This section is intended to develop a better understanding of the influence of model-layer thickness and number of half-space layers on dynamic structural responses for the soil-structure system studied in this report. The results are valuable to inform conducting similar soil-structure interaction analyses.

In this investigation, the RF17 geologic profile is used to evaluate the potential effects of allowable layer thickness. This geologic profile, along with the RF14 geologic profile, is used to assess the potential effects of the number of half-space layers. The allowable model-layer thickness for the alluvium unit is the smallest because its S-wave velocity is the smallest among the geologic units (Table 3-2). The RF17 geologic profile is subdivided into 39 thinner layers for the case satisfying the allowable layer thickness for the respective unit and 52 thinner layers for the case meeting the allowable layer thickness for the alluvium unit. For the RF14 geologic profile, only one subdivision is used. This subdivision ensures that the subdivided model layer has a thickness consistent with the allowable layer thickness for the alluvium unit for all geologic units.

Figure 5-12 presents the spectral response results for the two soil-model layer subdivision alternatives. In general, the effects of layer subdivision alternatives (39 sublayers versus 52 sublayers in total) on the spectral responses at the basemat are relatively small, while the effects are more pronounced on the in-structure responses. In addition, the effects are larger at higher elevations than of lower elevations of the structure. The subdivision option with the sublayer thickness meeting the allowable layer thickness of each unit (39 sublayers in total) produces relatively larger in-structure spectral response at higher frequencies than the option with the sublayer thickness for all geologic units meeting the allowable layer thickness for the alluvium unit (52 sublayers in total), irrespective of the number of half-space layers used.

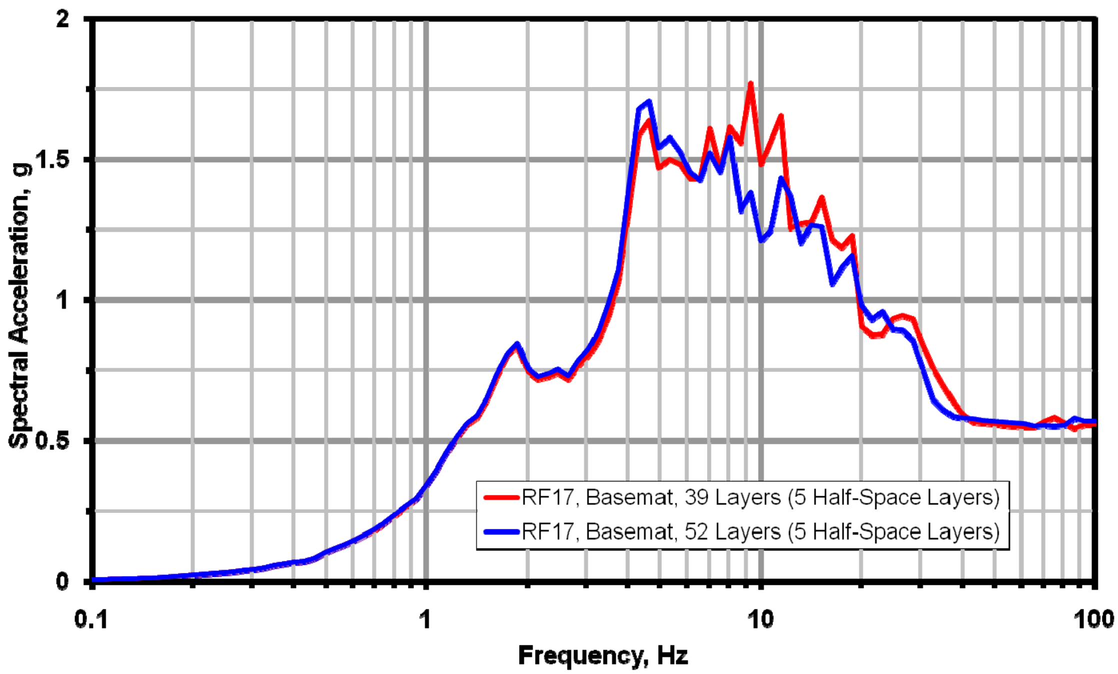


(a) Tower Top

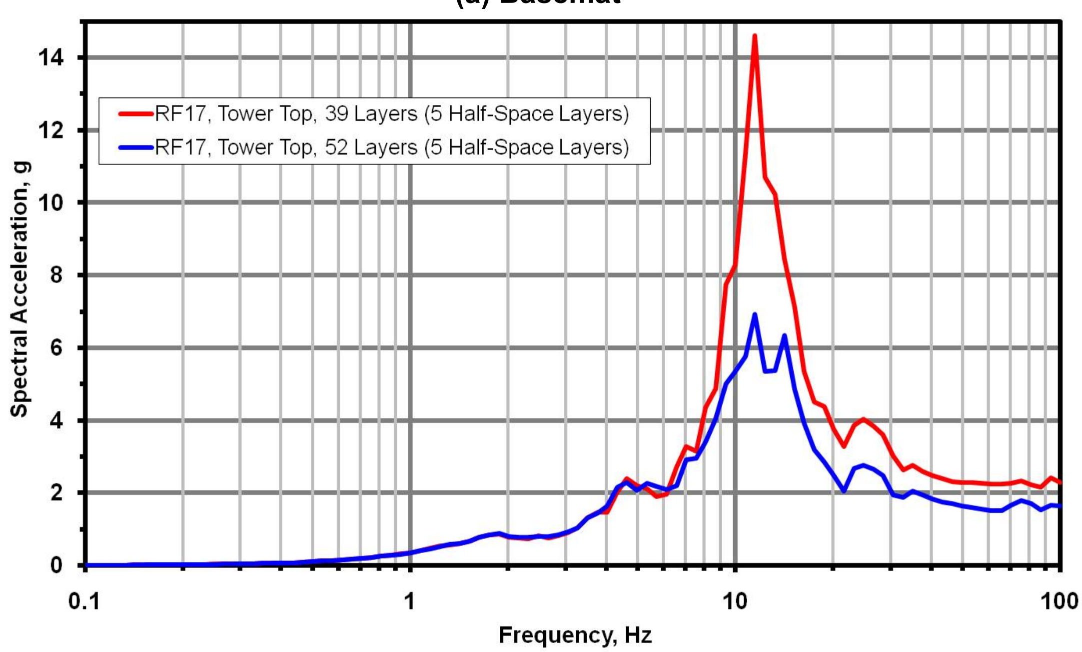


(b) Operating Floor

Figure 5-11. Mean Tower-Top and Operating Floor Spectral Responses for Set A and B Ground Motions



(a) Basemat



(b) Tower Top

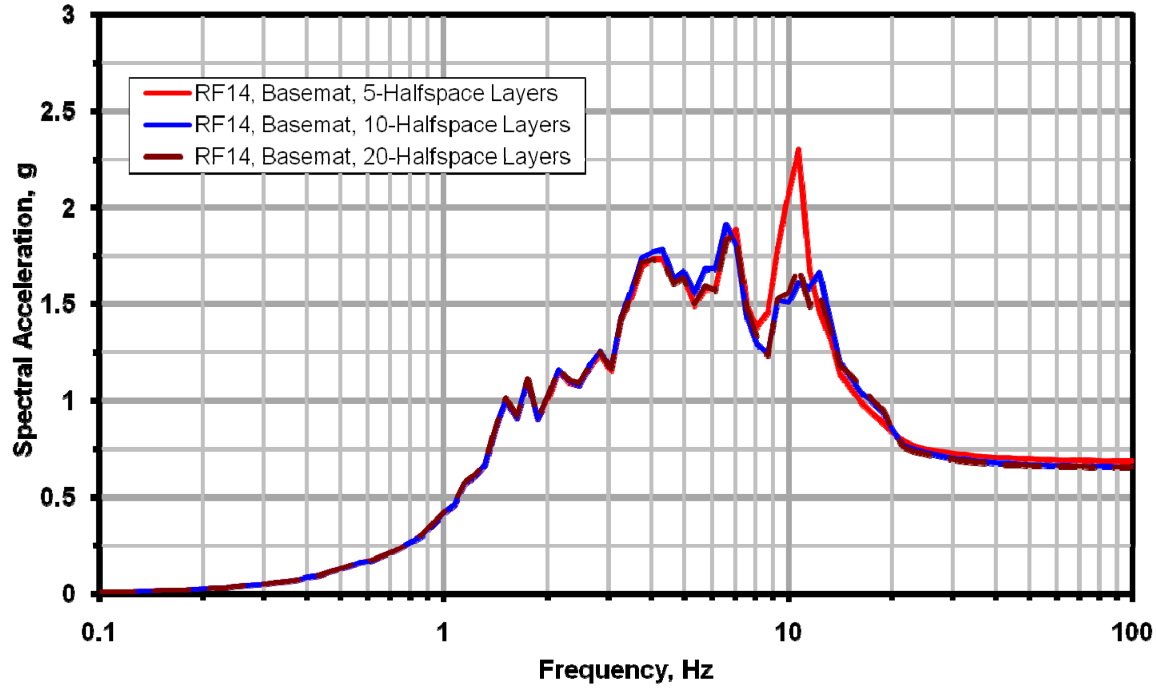
Figure 5-12. Effects of Soil-Model Layer Thickness on Structural Responses

Figure 5-13 compares the effects of the number of half-space layers on the spectral responses at the basemat for the RF14 and RF17 geologic profile cases. The effects of the number of half-space layers are small for response spectra at the basemat, irrespective of the geologic profiles used, except in the frequency range where the resonant behavior was observed. In that frequency range, the response spectra with 5 half-space layers are the largest among the three options for both RF14 and RF17 geologic profiles. The difference between 10 half-space layers and 20 half-space layers is almost negligible for all cases.

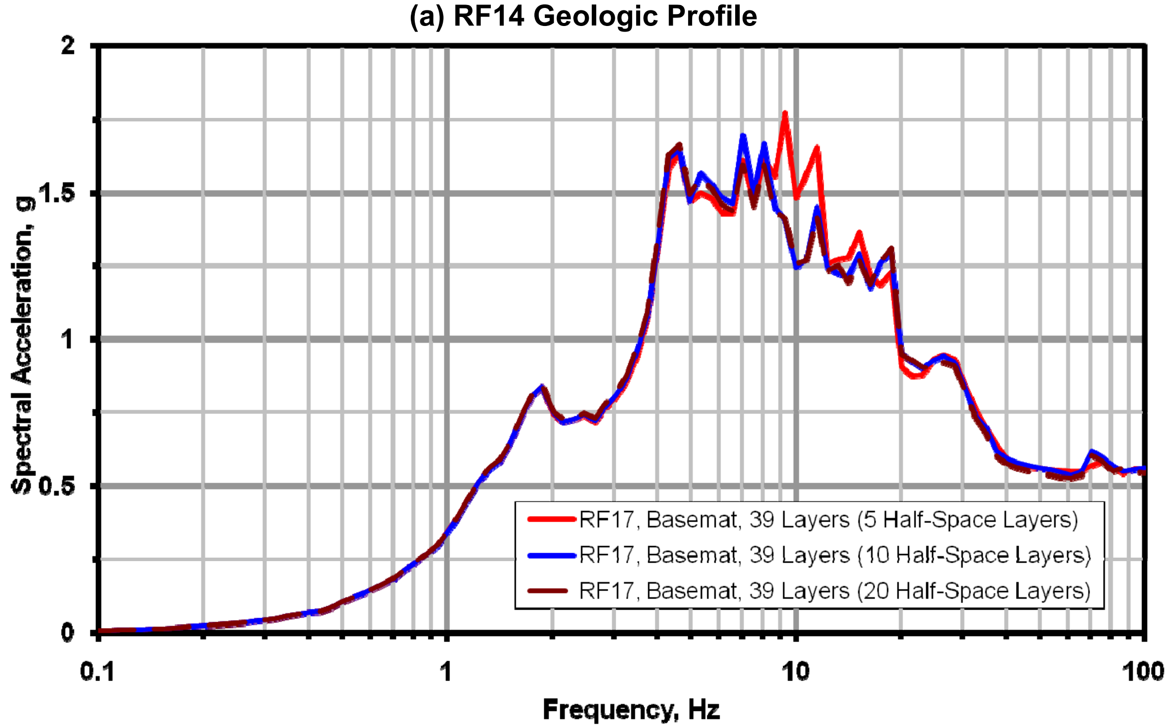
The effects of the number of half-space layers on response spectra are much greater on in-structure response spectra than the basemat (Figure 5-14). Similar to the basemat, the response spectra with 5 half-space layers are the largest among the three options in the resonant frequency ranges for the RF14 geologic profile and at frequencies higher than 9 Hz for the RF17 geologic profile. The response spectra for the 20 half-space layer case are the smallest among the three options at all frequencies greater than 6 Hz. As with the observation for the basemat, the difference in the tower-top response spectra between the cases with 10 and 20 half-space layers is negligible. This observation agrees with the suggestion Lysmer, et al. (1999a) made that 10 extra half-space layers are sufficient for most engineering cases.

Given that both soil-model layer subdivisions and number of half-space layers have effects on the structural spectral responses, it may be worthwhile to determine which factor dominates. Figure-5-15 shows that the subdivision option with the model-layer thickness meeting the allowable layer thickness of the respective unit (39 sublayers in total) and more half-space layers (20 layers) produces larger in-structure spectral responses than the option with the model-layer thickness meeting the allowable layer thickness for the alluvium unit (52 sublayers in total) and fewer half-space layers (5 layers). This result suggests that the model-layer subdivision may have more pronounced effects than the half-space layer option. The difference in effects between the subdivision options and number of half-space layers on the spectral responses at the basemat cannot be clearly determined; however, the difference is small.

In summary, for the hypothetical waste handling facility studied, the in-structure spectral response results are larger using the subdivision option with the model-layer thickness meeting the allowable layer thickness for each corresponding unit than using the subdivision option with the model-layer thickness meeting the allowable layer thickness of the alluvium unit. The response spectra for the former, in general, bound those for the latter. Similar observations may also apply to the use of half-space layers; the cases with fewer half-space layers may potentially result in conservative in-structure spectral responses relative to the cases with more half-space layers. Effects of both subdivision options and the number of half-space layers on basemat response spectra are substantially smaller than those on the in-structure response spectra and, therefore, may be ignored.

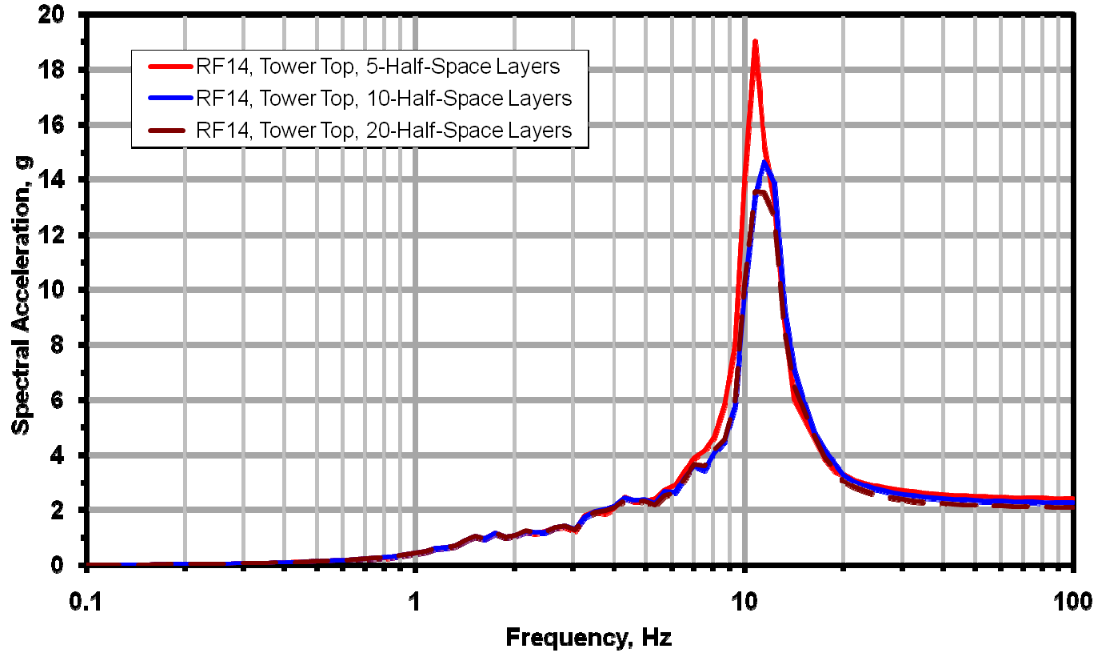


(a) RF14 Geologic Profile

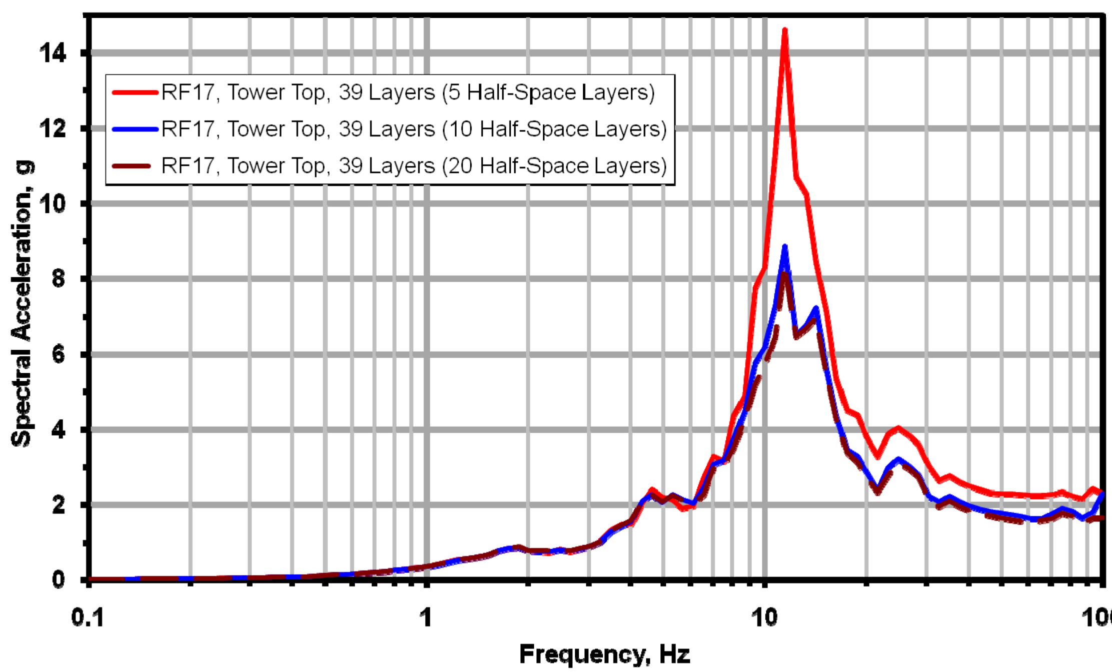


(b) RF17 Geologic Profile

Figure 5-13. Effects of Number of Half-Space Layers on Basemat Spectral Responses

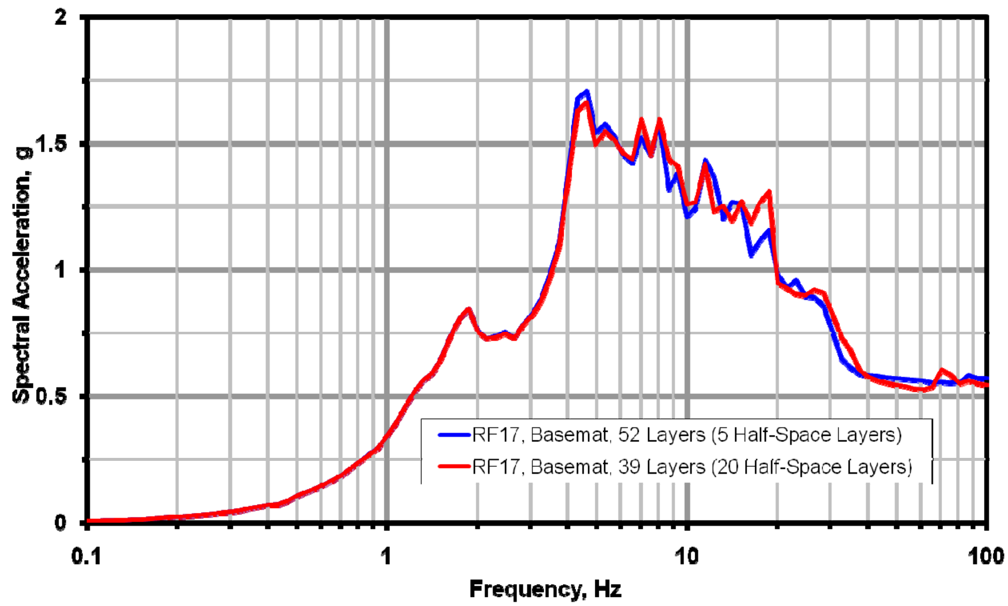


(a) RF14 Geologic Profile

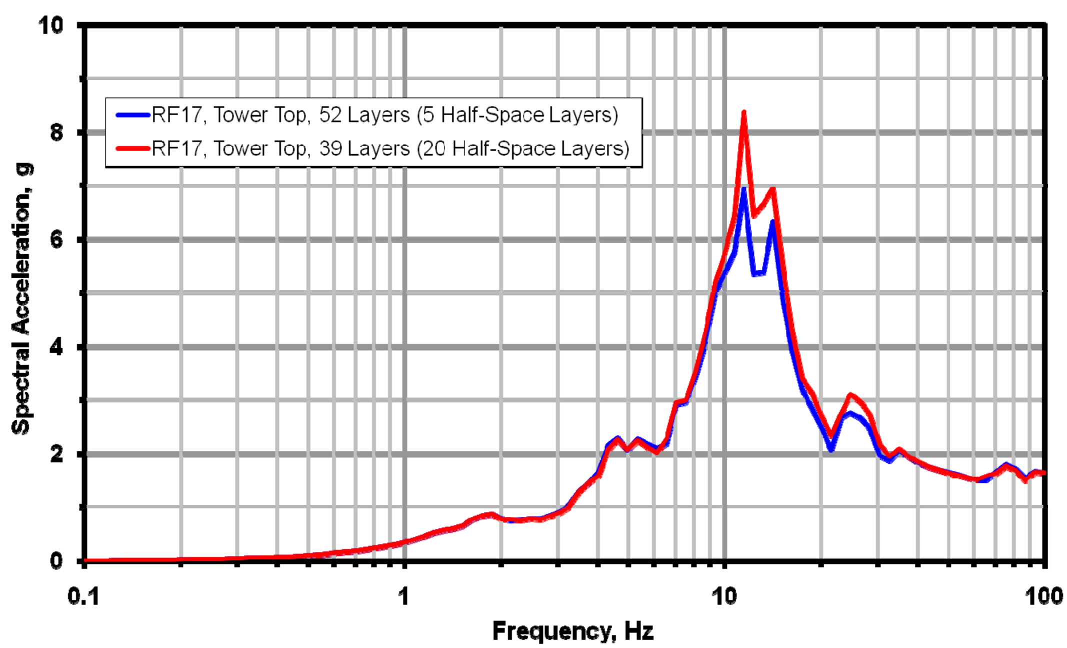


(b) RF17 Geologic Profile

Figure 5-14. Effects of Number of Half-Space Layers on Tower-top Spectral Responses



(a) Basemat



(b) Tower Top

Figure 5-15. Combined Effects of Subdivision Options and Number of Half-Space Layers on Structural Responses

6 CONCLUSIONS

The effects of complex and heterogeneous geological media on soil-structure interaction of a hypothetical waste handling facility at Yucca Mountain were investigated using the SASSI computer code. The investigation includes spatial variations of geologic profiles and soil material conditions. This report compared the dynamic structural responses using the Set B acceleration time histories developed consistent with the design-basis response spectrum at the soil top for the site of interest with the Set A acceleration time histories generated by spectrally matching the deaggregation response spectra at the reference rock outcrop of the same site. The Set A ground motions were developed by CNWRA whereas the Set B ground motions were from Bechtel SAIC Company, LLC (2004). The investigation conducted in this report included nine geologic profiles to capture a wide variation in thicknesses of geologic units at the site to permit qualitative assessment of the potential effects of spatial variability of complex site geology. Three sets of soil material conditions (soft, average, and stiff) defined based on S-wave velocities of the geologic units were also analyzed to examine the potential influence of uncertainty associated with the S-wave velocity measurements.

The soil-structure interaction analysis results showed soil-structure resonance at the top of a rectangular tower located at one corner of the hypothetical waste handling facility roof, 25.6 m [84 ft] above the ground floor, and on the operating floor at the 8.2-m [27-ft] level. This resonance, with sharp spectral acceleration amplitude increase in the frequency range of 9–17 Hz, was observed for four of the nine geologic profiles irrespective of the set of ground motions. For other geologic profiles where the resonant behavior was not observed, the response spectral acceleration peaks were reported in the 5–14 Hz frequency range.

A resonance in the 45–65 Hz frequency range with the response spectral acceleration peaks ranging from 3.5–15.75 g was observed for various acceleration time histories in the Set B ground motions. This resonance behavior with high response spectral peaks is specific to the RF14 geologic profile under the stiff soil condition. The presence (or absence) and thickness variations of the geologic units in the RF14 geologic profile are not any different from other geologic profiles used to explain this behavior.

The soil-structure interaction analyses results yield valuable information on how a structure sitting on the ground with a complex geological medium reacts to ground motions. The results could be used to identify regions of a site where the soil-structure interaction may be too large to be tolerable for performance of the structure, including the systems and components in the structure. These insights are valuable for selecting alternatives to mitigate the complex geological effects on dynamic structural performance and result in a more efficient and cost-effective design. Many engineering measures are available to improve performance. For example, if the structure footprint is sufficiently small or the available site area is large enough, the structure could possibly be located in areas where the structural resonance behavior is less pronounced. Ground improvements to increase soil stiffness may prove to be effective as well because the soil-structure analysis results show that the resonant behavior is less pronounced when the structure is located on stiffer ground. The soil-structure interaction analyses could be used to assess the effectiveness of various ground improvement techniques provide input for cost estimation. Such information is helpful for decisionmaking on selecting reasonable techniques for ground improvement.

Besides the resonant behavior, the soil-structure interaction analyses also highlight the need for the structure to be designed for a broadened design spectrum. The soil-structure interaction analysis results showed that, with a complex geology, the variation of geologic profiles results in shifts in frequency contents of the spectral acceleration peaks and different soil material conditions tend to produce spectral acceleration peaks with different frequency contents. This shift in frequency contents of the spectral acceleration peaks suggests a broadened response spectra curve could be developed to account for the spatial geological variability to ensure reasonable consideration of possible seismic effects.

Based on comparison of the soil-structure interaction analysis results from the Set A and Set B ground motions, the response spectral acceleration envelopes at various locations of the hypothetical waste handling facility for both Set and A and B ground motions are in a broad agreement even though at some frequencies the spectral accelerations for Set A ground motions are larger. The differences are not significant considering the uncertainties associated with modeling assumptions, the approaches used, and complex site geology. The mean response spectra from the Set B ground motions, on the other hand, bound those from the Set A ground motions at various locations of the hypothetical waste handling facility with a large margin. Because the Set B ground motions were developed consistent with the design-basis response spectrum, the design-basis response spectrum appears to be reasonable for structural analyses, design, and qualification of systems and components in the hypothetical waste handling facility.

In performing the soil-structure analyses using the SASSI computer code, it is important that the layer thickness used in the soil model should, at least, be smaller than or equal to the allowable soil layer thickness for each corresponding soil-layer unit. The allowable thickness is one-fifth of the wavelength of interest. Using the allowable soil-layer thickness determined based on the smallest S-wave velocity among the geologic units in the geologic profile is preferable to using the allowable thickness based on each corresponding geologic unit. However, the latter appears to yield conservative results relative to the former.

7 REFERENCES

Avilés, J. and Pérez-Rocha, L.E. "Site Effects and Soil-Structure Interaction in the Valley of Mexico." *Soil Dynamics and Earthquake Engineering*. Vol. 17, No. 1. pp. 29–39. 1998.

Bechtel SAIC Company, LLC. "Development of Earthquake Ground Motion Input for Preclosure Seismic Design and Postclosure Performance Assessment of a Geologic Repository at Yucca Mountain, NV." MDL–MGR–GS–000003. Rev. 01. Las Vegas, Nevada: Bechtel SAIC Company, LLC. 2004.

———. "Geotechnical Data for a Potential Waste Handling Building and for Ground Motion Analyses for the Yucca Mountain Site Characterization Project." ANL–MGR–GE–000003. Rev. 00. Las Vegas, Nevada: Bechtel SAIC Company, LLC. 2002.

Chen, J.C. J. Lymer, and H.B. Seed. "Analysis of Local Variations in Free Field Seismic Ground Motion." Report No. UCB/EERC–81/03. Berkeley, California: University of California. 1981.

Dynamic Graphics, Inc. "EarthVision User's Guide 7.5." Alameda, California: Dynamic Graphics, Inc. 2002.

Earthquake Engineering Field Investigation Team. "The Ji-Ji, Taiwan Earthquake of 21 September 1999—Short Report." <http://www.istructe.org/knowledge/EEFIT/Documents/Ji_Ji_Taiwan.pdf> July 2011.

EduPro Civil Systems. "ProShake." Version 1.11. Sammamish, Washington: EduPro Civil Systems. 2001.

European Strong-Motion Database. <http://www.isesd.hi.is/ESD_Local/Database/Database.htm> October 2006.

Hsiung, S.M., A. Ghosh, and A.H. Chowdhury. "Site Response and Soil-Structure Interaction Analyses—A Progress Report." San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2007.

Hsiung, S.M., S.H. Gonzalez, and A.H. Chowdhury. "Site Response and Soil-Structure Interaction Analyses—A Progress Report." San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2006.

Hsiung, S.M., S.H. Gonzalez, K.J. Smart, and A.H. Chowdhury. "Soil-Structure Interaction—Progress Report: Site Response Analyses." San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2005.

Kramer, S.L. *Geotechnical Earthquake Engineer*. Upper Saddle River, New Jersey: Prentice-Hall. 1996.

Lo Presti, D.C.F.I, C.G. Lai, and I. Puci. "ONDA: Computer Code for Nonlinear Seismic Response Analyses of Soil Deposits." *Journal of Geotechnical and Geoenvironmental Engineering*. Vol. 132, No. 2. 2006.

Lysmer, J. and R.L. Kuhlemeyer. "Finite Dynamic Model for Infinite Media." *Journal of Engineering Mechanics Division*. American Society of Civil Engineering. Vol. 95, No. EM4. pp. 859–877. 1969.

Lysmer, J., F. Ostandan, and C.C. Chin. "SASSI 2000—A System for Analysis of Soil-Structure Interaction, User's Manual." Rev. 1. Berkeley, California: University of California, Berkeley. 1999a.

———. "SASSI 2000—A System for Analysis of Soil-Structure Interaction, Theoretical Manual." Rev. 1. Berkeley, California: University of California, Berkeley. 1999b.

McGuire, R.K., W.J. Silva, and C.J. Costantino. NUREG/CR–6728, "Technical Basis of Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines." TIC: 251294. Washington, DC: NRC. 2001.

Mylonakis, G., G. Gazetas, S. Nikolaou, and O. Michaelides. "The Role of Soil on the Collapse of 18 Piers of the Hanshin Expressway in the Kobe Earthquake." Proceedings of 12th World Conference on Earthquake Engineering, Auckland, New Zealand. Tokyo, Japan: Earthquake Research Institute. 2000.

Ofoegbu, G.I. and G.D. Gute. "Dynamic Soil-Structure Interaction Analysis of a Storage-Cask Foundation Design." CNWRA 2003–03. San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2002.

Resendiz, D. "Soil-Structure Interaction in Mexico City During the 1985 Earthquakes." Proceedings of the International Conference on Mexico Earthquakes—1985, Mexico City, Mexico: Factors Involved and Lessons Learned. M.A. Cassaro and E. Martínez-Romero, eds. Reston, Virginia: American Society of Civil Engineers. 1986.

Risk Engineering, Inc. "EZ-FRISK." Version 7.20. Golder, Colorado: Risk Engineering, Inc. 2005.

Schnabel, P.B., J. Lysmer, and H.B. Seed. "SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites." Report No. UCB/EERC–72/12. Berkeley, California: University of California, Earthquake Engineering Research Center. 1972.

Stepp, J.C., I. Wong, J. Whitney, R. Quittmeyer, N. Abrahamson, G. Toro, R. Youngs, K. Coppersmith, J. Savy, T. Sullivan, and the Yucca Mountain Probabilistic Seismic Hazard Analyses Project Members. "Probabilistic Seismic Hazard Analyses for Ground Motions and Fault Displacement at Yucca Mountain, Nevada." *Earthquake Spectra*. Vol. 17, No. 1. pp. 113–151. 2001.

Udaka, T. "Analysis of Response of Large Embankments to Traveling Base Motions." Ph.D. dissertation. University of California. Berkeley, California. 1975.

Waas, G. "Linear Two-Dimensional Analysis of Soil Dynamic Problems in Semi-Infinite Layered Media." Ph.D. dissertation. University of California. Berkeley, California. 1972.