SOIL STABILITY AND SEISMIC SITE RESPONSE AT YUCCA MOUNTAIN, NEVADA

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ABSTRACT

This knowledge management report summarizes the methods and related information used to evaluate the soil stability of the proposed surface facilities site at Yucca Mountain under both static and dynamic loads. This site is located in Midway Valley, just to the east of Yucca Mountain. The stratigraphy comprises a variably thick layer of dry alluvium and colluvium resting unconformably atop variable bedrock of Miocene age volcanic tuff. The tuff comprises moderately to densely welded pyroclastic flows of the Tiva Canyon Tuff and nonwelded bedded tuff of the post-Tiva Canyon Tuff and the pre-Rainier Mesa Tuff.

For the static loads, the U.S. Department of Energy (DOE) relied on traditional geotechnical engineering approaches to determine the soil strength, settlement potential, and bearing capacity. DOE design of mat foundations at the surface facility under static and seismic loading is based on a finite element (FE) model in which the subsurface material is represented by soil springs and the resulting deformation is used to calculate foundation pressures and settlements. DOE characterized the granular soil media by a high friction angle of 39° and zero cohesion and low compressibility. Beneath the alluvium, the bedrock consists of welded and non-welded volcanic tuff units and is considered to be higher strength than the alluvial soil. Because of the large footprints of the mat foundations and dense nature of granular alluvium, shear failure of sand is less likely and soil settlement is typically of greater concern. Because the water table is approximately 390 m [1,300 ft] below the ground surface and the alluvium is not saturated, earthquake-induced bearing capacity failure, liquefaction, and post-earthquake instability are not of concern at the Yucca Mountain site.

For the dynamic analyses, DOE conducted seismic site response modeling. A random-vibration theory (RVT), one-dimensional (1D) equivalent-linear approach was implemented to calculate site response effects on ground motions. The model provided results in terms of spectral acceleration, including peak ground acceleration, peak ground velocity, and dynamically-induced strains as a function of depth. The site response model convolves the DOE probabilistic seismic hazard analysis (PSHA), which was derived for a reference base rock condition, with site specific geotechnical data to develop ground motions for the surface facilities area in Midway Valley and the repository host horizon in Yucca Mountain. The PSHA was developed for a reference bedrock outcrop, specified as a free field site condition with a mean shear wave velocity (V_S) of 1,900 m/s [6,200 ft/s] and located adjacent to Yucca Mountain. In addition to the PSHA ground motions, important inputs to the site response included the V_S, compression wave (V_P), and density of the underlying strata and their corresponding dynamic material properties. A summary description of these data and their application to the site response are discussed in this report. The confirmatory analysis developed by the U.S. Nuclear Regulatory Commission (NRC) staff to evaluate the DOE site response results is also described. This analysis was based on a 1D equivalent-linear approach using the SHAKE2000 code. Results of the NRC staff analyses, which were consistent with those from DOE, show that the alternating layers of densely welded and unwelded tuff layers beneath Midway Valley lead to significant deamplification of the high frequency ground motions. These results are also similar to deamplification observed at the Columbia Nuclear Generating Station in eastern Washington State. The Columbia site rests atop alternating layers of crystalline basalt and fine-grained unconsolidated fluvial and lacustrine sediments. The high frequency deamplification at the Columbia site was attributed to the strong impedance contrast between the interbed and basalt layers and material damping in the interbeds.

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ACKNOWLEDGMENTS

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QUALITY OF DATA, ANALYSES, AND CODE DEVELOPMENT

DATA: All CNWRA-generated data contained in this report meet quality assurance requirements described in the CNWRA Quality Assurance Manual. Sources of other data should be consulted for determining the level of quality of those data. No original data were generated in this report. Data from CNWRA Scientific Notebook 1227 (Stamatakos, 2014) was used in this report. That notebook is preserved in the NRC ADAMS system with accession number ML14365A146.

REFERENCE

Stamatakos, J. "One Dimensional Site Response Analysis to Support NRC Review of the DOE Seismic Hazard Information for Yucca Mountain." Scientific Notebook 1227. ML14365A146. San Antonio, Texas: Center for Nuclear Waste Regulatory Analyses. 2014.

1 INTRODUCTION

1.1 Background

Yucca Mountain is located in the southern part of the Great Basin of the central Basin and Range Physiographic Province (Figure 1-1). It is one of many fault-bounded ridges composed of volcanic tuff that separate arid desert basins that are largely filled by alluvial material and related soils. As envisioned in the U.S. Department of Energy (DOE) license application, the proposed geologic repository would be configured to dispose up to 70,000 MTHM (metric tons of heavy metal) of high-level radioactive waste and spent nuclear fuel in underground horizontal drifts that would be situated approximately 300 m [1000 ft] below the ground surface within Yucca Mountain. To emplace this material underground, DOE proposed a design that included waste handling facilities, a network of roads and rail for surface transportation of waste among the surface facilities, flood control structures, and related support systems. The waste handling facilities would accommodate initial waste receipt, canister transfer, thermal aging, wet and dry handling, and underground emplacement. DOE proposed to construct these surface facility installations above a thick sequence of alluvium and tuff in Midway Valley, which is located just east of the proposed repository footprint within Yucca Mountain (Figure 1-2).

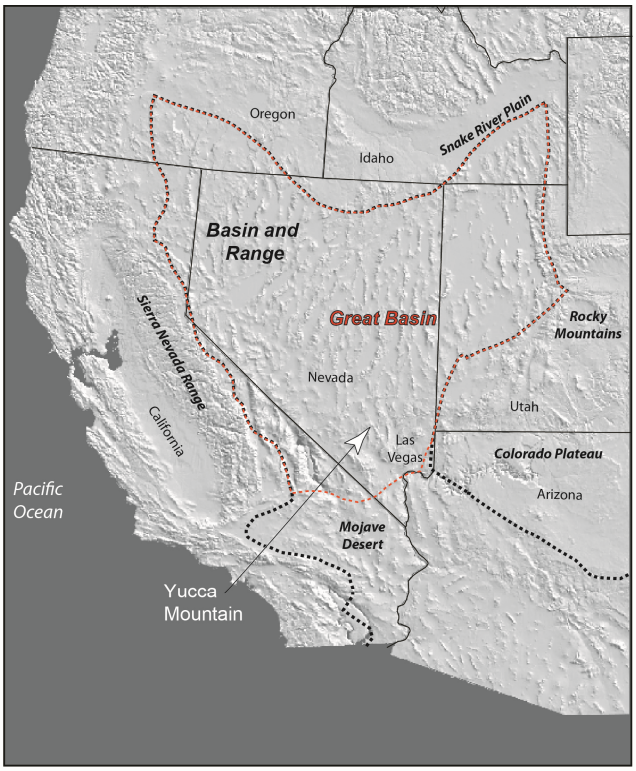

Figure 1-1. Digital elevation model of the western United States showing the location of Yucca Mountain within the Great Basin and Basin and Range.

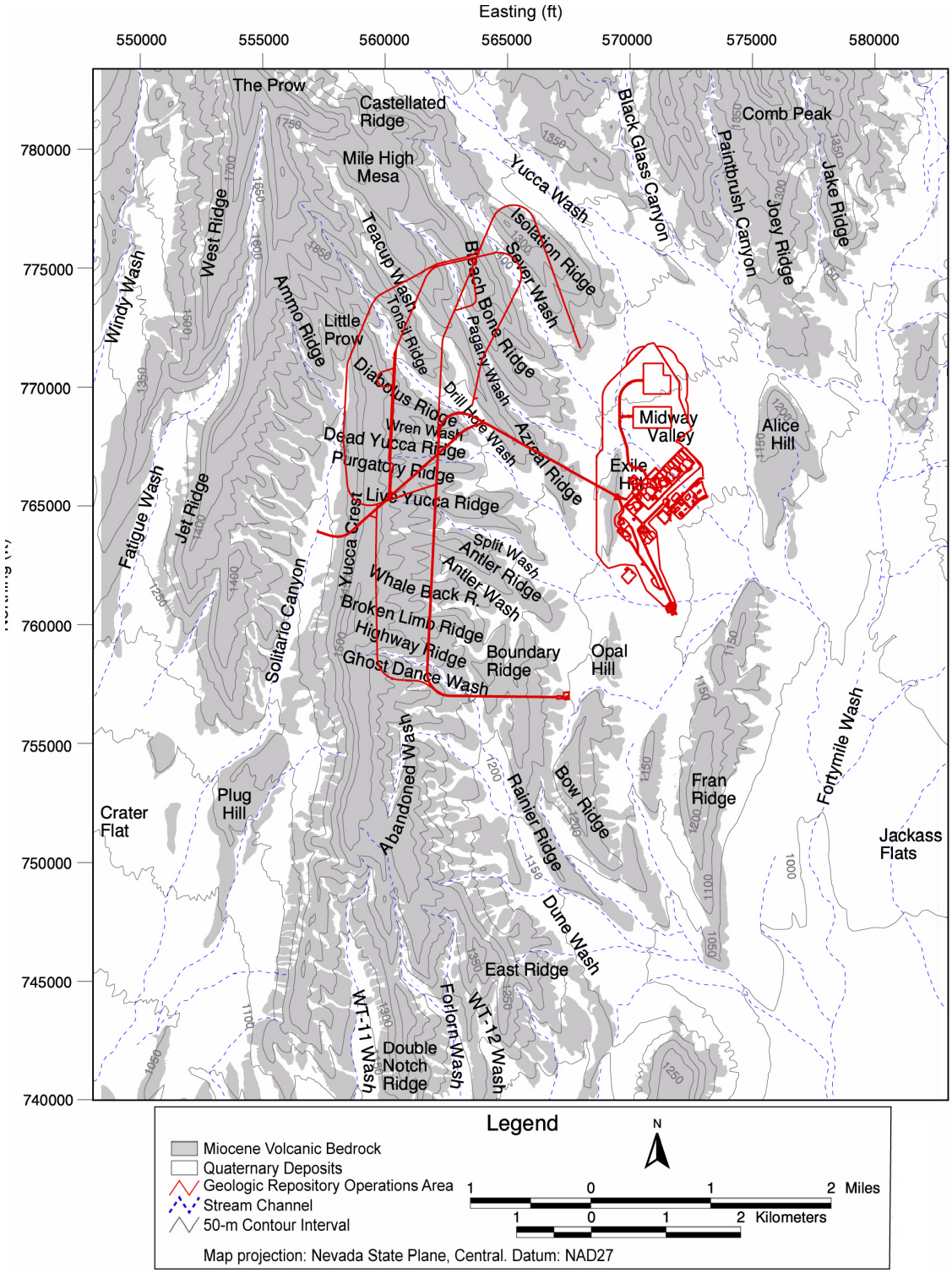

Figure 1-2. Map of the Yucca Mountain region showing the exhumed ridges of volcanic bedrock and the intervening basins filled with Quaternary alluvium and related soils. The red outline shows the proposed surface and subsurface facilities. This figure was adapted from Figure 1.1-5 of DOE (2008).

To develop design inputs for surface facilities and subsurface drifts and installations, DOE conducted extensive geotechnical investigations of the site bedrock, alluvium, and soils. These investigations included field and laboratory testing as well as finite element modeling. DOE conducted these investigations to obtain the necessary information to assess the static and dynamic material properties of the bedrock, alluvium, and soils to assess the bearing capacity and settlement under both static and dynamic loads.

As part of the site investigations conducted to evaluate Yucca Mountain as the site for a high-level nuclear waste repository, DOE identified seismic hazards as one of several credible natural hazards that could adversely impact safety and repository performance. DOE concluded that an accurate assessment of the seismic hazard at the site was needed for the design and safety analysis of the surface facilities for the preclosure period[1] and the performance assessment of the repository for the postclosure period.[2] During the 1980s and 1990s, DOE supported extensive geological, geophysical, and seismological studies by the United States Geological Survey (USGS), Bureau of Reclamation, DOE national laboratories, and other contractors that were used for their seismic hazard analysis.

DOE's overall approach to developing a seismic hazard assessment for Yucca Mountain involved the following two steps. In the first step, DOE conducted an expert elicitation in the late 1990s to develop a probabilistic seismic hazard analysis (PSHA) for Yucca Mountain (CRWMS M&O, 1998). The PSHA was developed for a reference bedrock outcrop, specified as a free-field site condition with a mean shear wave velocity (V_S) of 1,900 m/s [6,200 ft/s] and located adjacent to Yucca Mountain. As provided in Schneider et al. (1996), this value was derived from a V_S profile of Yucca Mountain with the top 300 m [1000 ft] of tuff and alluvium removed.

In the second step, DOE conducted site-response modeling to condition the PSHA results so they would be applicable to the surface facilities in Midway Valley and the repository drifts in the subsurface (Bechtel SAIC Company, LLC, 2004). Site response modeling accounts for changes in seismic energy (amplification or deamplification, attenuation, and damping) as the seismic waves propagate through the tuff layers in the repository strata directly beneath the proposed emplacement drifts and in the layers of soil and alluvium directly beneath the proposed surface facilities in Midway Valley (Figure 1-2). DOE relied on the resulting repository-level and surface-level seismic hazard curves as inputs to its postclosure performance assessment, surface and subsurface design, and preclosure seismic risk assessment used to evaluate the likelihood of accident event sequences initiated by earthquakes.

1.2 Objective and Scope

This knowledge management report focuses on the second of the two steps DOE used to develop a seismic hazard assessment for Yucca Mountain: the development of the DOE site

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[1] The preclosure period (period of operations) includes (i) the time during which emplacement would occur, (ii) any subsequent period before permanent closure during which the emplaced wastes are retrievable, and (iii) permanent closure.

[2] The postclosure period is the period following permanent closure of the repository through the period of geologic stability.

response modeling and some of the supporting studies the Center for Nuclear Waste Regulatory Analyses (CNWRA®) and U.S. Nuclear Regulatory Commission (NRC) staffs conducted in preparing for the review of the DOE license application. The first step of the assessment, involving the DOE PSHA, was the topic of a separate and recently completed knowledge management report (Stamatakos, 2017). In addition, this report describes the geodynamic and geomechanical analyses DOE conducted to develop the design inputs for the soil stability analysis. This report also describes some of the relevant new information and technical developments that have accrued since DOE submitted their license application in 2008. Chapter 2 of this report summarizes the DOE site geological and geotechnical information relevant to the seismic site response and soil stability analysis. Chapter 3 explains soil stability analyses. Chapter 4 explains the seismic site response analyses, including improvements to site response methods that have occurred since 2008. Chapter 5 gives a brief summary of this report and Chapter 6 documents the references cited. Although the purpose of the report is to convey the thought process and independent analyses conducted as part of the Yucca Mountain license application review, information presented in this report also may be of interest to other safety analyses and reviews involving soils stability and seismic design.

2 SITE GEOLOGIC AND GEOTECHNICAL INFORMATION

2.1 Site Geology

Yucca Mountain consists of a ridge of welded and nonwelded pyroclastic flows and air-fall tuffs that erupted from the southwest Nevada volcanic field calderas in the Late Miocene, between approximately 11 and 15 Ma (Sawyer et al., 1994). Tuffs exposed at Yucca Mountain include units of the Crater Flat and Paintbrush Groups, as well as the Calico Hills Formation. The repository is proposed to be housed within moderately to densely welded members of the Topopah Spring Tuff, which is one of the main tuff units of the Paintbrush Group. These volcanic strata rest uncomfortably on basement rocks consisting of Precambrian and Paleozoic clastic and carbonate sedimentary and metasedimentary strata with minor occurrences of Mesozoic and Tertiary igneous and meta-igneous sills, dikes, and flows.

Structurally, Yucca Mountain constitutes a central graben high that is part of the Crater Flat basin, which is bounded on the west by the east-dipping Bare Mountain fault and on the east by the north-trending anomaly beneath Jackass Flat, informally named the Gravity fault. Faults at Yucca Mountain trend north to north-northeast and form fault-bounded north-south ridges that are occasionally crossed by northwest-trending, dextral strike-slip faults. Faults dip almost uniformly to the west and separate blocks of gently to moderately east-dipping tuff and pyroclastic flows. Faulting activity was most active between approximately 11.0 Ma and 11.5 Ma, soon after deposition of the volcanic units (Stamatakos et al., 2000). Many of these faults remain active to the present, albeit with moderately low slip rates (e.g., Keefer et al., 2004).

At the proposed site for the surface facilities in Midway Valley, the subsurface strata consist of alluvium overlying moderately and densely welded pyroclastic flows of the Tiva Canyon Tuff and nonwelded bedded tuff of the post-Tiva Canyon Tuff and the pre-Rainier Mesa Tuff. Table 2-1 provides a summary description of the lithologic units encountered in the boreholes within Midway Valley and the surface facility site. The proposed site for the surface facilities is cut by several north-northeast to north-northwest trending normal faults. The largest of these is the Exile Hill fault splay, which has significant down-to-the-northeast displacement. In general, the bedded nonwelded tuffs are confined to the hanging walls of the normal faults. Because of the faulting, the underlying stratigraphy generally dips about 25° to the east-southeast, although locally some beds dip back to the west-northwest within several of the small grabens that form between normal faults or on relay ramps that form between *en-echelon* segments of normal faults.

Based on the stratigraphic and fault data in Bechtel SAIC Company, LLC (2002), the CNWRA staff developed a three dimensional digital geologic model of the site (Gonzalez at al., 2004). The purpose of the model was to depict the stratigraphic and the faulting relationships and ensure that these data provided a model of the subsurface geology beneath Midway Valley that was consistent with geological principals (Figure 2-1). The model also provided the basis for the one dimensional (1D) profiles derived from the model that were used by the CNWRA and NRC staffs to independently evaluate the DOE site response models (as described in Section 4.2 of this report).

Table 2-1. Summary description of the lithologic units comprising the subsurface strata in the site response modeling		
Unit Symbol	**Unit Name**	**Lithology**
Qal	Quaternary Alluvium	Poorly to well-cemented tuffaceous alluvium with mixture of layered gravel and cobble of clasts of densely welded ignimbrite in a matrix of smaller fragments of nonwelded tuff and silty sand
Tmbt1	pre-Rainier Mesa Bedded Tuff	Bedded and reworked tuff with up to 10 percent pumice
Tpki	Tuff unit “x”	Nonwelded pyroclastic flow with 10–30 percent pumice clasts
Tpbt5	post-Tiva Canyon bedded tuff	Devitrified and reworked fallout tephra and tuffaceous rocks and interbedded paleosols
Tpbt5	Crystal rich member of the Tiva Canyon Tuff	Moderately to densely welded crystal rich pyroclastic flow, some pumice fragments but no lithophysae
Tpbt5	Upper lithophysal zone of the Tiva Canyon Tuff	Moderately to densely welded crystal poor upper lithophysal zone. Up to 20 percent lithophysae. Moderately to intensely fractured
Tpcpmn	Middle nonlithophysal zone of the Tiva Canyon Tuff	Densely welded crystal poor pyroclastic flow. Moderately to intensely fractured
Tpcpll	Lower lithophysal zone of the Tiva Canyon Tuff	Densely welded crystal poor pyroclastic flow with up to 20 percent lithophysae, slightly fractured.
Tpcpln	Lower nonlithophysal zone of the Tiva Canyon Tuff	Densely welded crystal poor pyroclastic flow. Moderately to intensely fractured

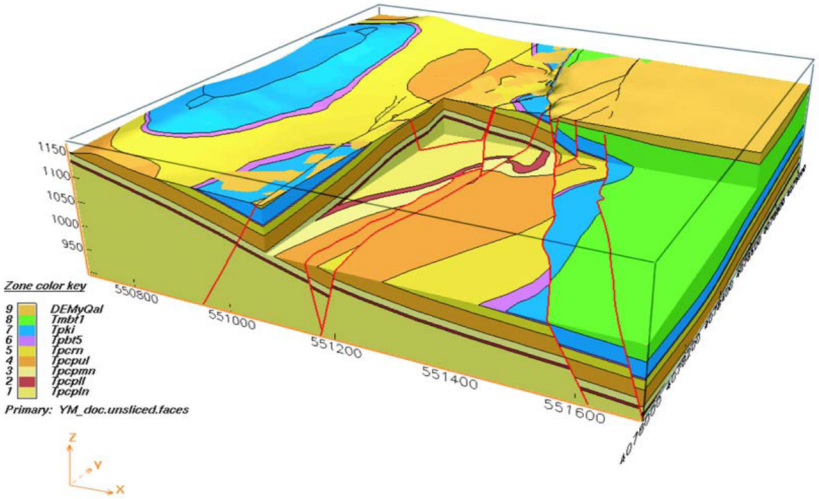

Figure 2-1. An image of the 3D EarthVision® model of the surface facility area in Midway Valley developed for the review and evaluation in Gonzalez et al. (2004).

2.2 Surface and Downhole Geotechnical Data

DOE relied on three types of geophysical measurements to obtain shear-wave (V_S) and compression-wave (V_P) velocity data for the surface facility site in Midway Valley, on the crest of Yucca Mountain, and within the drifts. These included (i) conventional downhole and downhole suspension seismic surveys from 43 boreholes and (ii) spectral analysis of surface waves surveys from 40 traverses across Midway Valley and within the Exploratory Studies Facility and in the Cross Drift. These data were collected by the DOE within three periods of data collection activities: (i) prior to 2005, (ii) the 2005–2006 campaign, and (iii) the 2006–2007 campaign (Bechtel SAIC Company, LLC, 2002; 2007a, 2007b, 2008). These three campaign periods reflect additional data needs associated with revisions of the Geologic Repository Operations Area (GROA) design during the pre-licensing period. The analyses in Gonzalez et al. (2004) included a review of the DOE V_S and V_P data collected prior to 2005. Review of the geotechnical data by the NRC and CNWRA staffs during the technical evaluation of the DOE license application showed that the data collected in the second and third DOE campaigns were consistent with the data from the first campaign (NRC, 2011).

2.2.1 Downhole Data

Comparison of the results from the suspension surveys and conventional downhole surveys are similar and show that V_S increases from approximately 200 m/s [660 ft/s] near the surface to greater than 1,800 m/s [5,900 ft/s] in the densely welded Tiva Canyon tuff (Figure 2-2). Both downhole and suspension survey results included uncertainties in averaged V_S and V_P values that reflect the variable nature of the host rocks and soils (e.g., composition, fracture density, and vein material), complexities in the source ray paths, and cultural noise. In the conventional downhole measurements, a relatively large volume of material is sampled because the source remains at the surface away from the borehole. As the source at the surface is activated, the receiver array is moved vertically within the borehole. This method reduces the variability in V_S or V_P from local effects because they are effectively integrated out. However, the signal-to-noise ratio in the downhole surveys increase as the receiver is moved down the boreholes and away from the source at the surface because the input waveform is dulled by attenuation and damping.

In contrast, the strength of the input waveform remains relatively uniform in suspension surveys because the receiver and source remain at fixed distance from each other for all measurements in the borehole. However in suspension surveys, local factors such as patches of fractured rock or vein fillings strongly influence a single measurement because only a small volume of material is sampled by each measurement. These local factors contribute substantially to the overall variability in the measurements. As indicated in Bechtel SAIC Company, LLC (2002), some smoothing of the suspension data was therefore necessary to compensate for these local effects.

2.2.2 Spectral Analysis of Surface Wave (SASW)

Spectral analysis of surface waves (SASW) is a method that relies on the dispersive characteristic of Rayleigh waves as they propagate through a layered medium due to changes in the material properties of the underlying rocks or soil, including shear wave velocity and stiffness. A Rayleigh wave is a seismic surface wave that produces an elliptical ground motion, with no transverse, or perpendicular, motion. Wave amplitudes decrease exponentially with depth. This method uses the dispersive characteristics of surface waves to determine the variation of the shear wave velocity (stiffness) of layered systems with depth. Spectral analysis

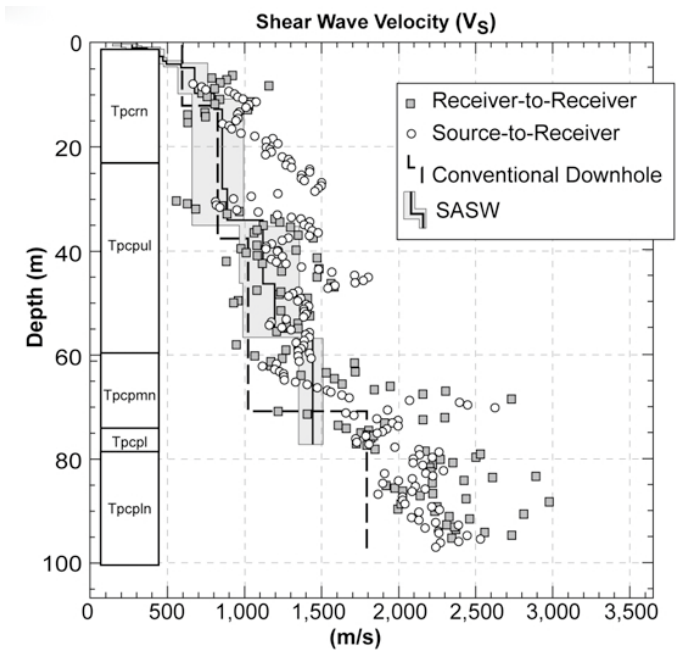

Figure 2-2. Comparison of V_S measurements from conventional downhole, source-to-receiver suspension surveys from DOE Borehole RF #15 (data re-graphed from Figures VII-3 and VII-19 in Bechtel SAIC Company, LLC, 2002). The SASW profile is the Area 2 composite profile, which was scanned and re-graphed from Figure 93 in Bechtel SAIC Company, LLC (2002).

is used to separate the waves by frequency and wavelength to arrive at individual or composite dispersion curves. Forward modeling is then used to develop 1D V_S profiles and associated dispersion curves that reasonably match the observed Rayleigh wave dispersion. V_P is not modeled by this method.

SASW surveys were conducted using either common-receivers midpoint geometry or fixed source configuration in which the receivers were progressively spaced up to a maximum of approximately 60 m [200 ft]. In SASW applications, the maximum depth that V_S can be estimated is half the longest wavelength of the generated Rayleigh wave. To generate energy with the requisite spectral frequencies, DOE used four sources (i) a handheld hammer striking the ground; (ii) a sledgehammer striking the ground; (ii) a bulldozer operated back and forth a distance of several meters, and (iv) a Vibroseis truck. Estimates of V_S from SASW profiles are comparable to the downhole and suspension results except at very shallow depths, less than approximately 6 m [20 ft], where the spectral analysis of surface waves readings indicate much slower V_S (Figure 2-2).

2.2.3 Density

DOE acquired rock and soil density data using standard wireline caliper and gamma-gamma density tools, which were run in a subset of the boreholes in Midway Valley (Bechtel SAIC Company, LLC, 2002; 2007a, 2007b, 2008). The alluvium and volcanic strata beneath Midway Valley have relatively low densities. Densities as low as 1.6 Mg/m³ were measured in Tpki unit (tuff unit “x”), which is nonwelded tuff containing up to 30 percent pumice (Table 2-1). The alluvium and the other nonwelded to moderately welded tuffs have densities near 2.0 Mg/m³.

Density values up to 2.4 Mg/m³ were measured in the nonlithophysal units of the densely welded Tiva Canyon Tuff.

2.2.4 Dynamic Material Properties

Equivalent-linear seismic site response modeling requires two inputs to characterize the dynamic material properties of the strata in the column of material above the reference bedrock. These are shear modulus reduction (changes in the ratio of stress to strain in the material under vibratory loading) and material damping ratios (changes in the percent damping of the material under increasing strain). These properties are obtained either from direct laboratory testing or from published data and are plotted in a functional form as best fit curves (Figure 2-3). In typical equivalent-linear analyses, the modulus reduction factors and damping ratios are obtained from the curves and as a function of the peak shear strain over the duration of excitation for the layer in question. Because the peak strains are not known when the 1D site response is initiated, initial values are assumed and the analysis iterates until "strain-compatible" values are found (e.g., Risk Engineering, INC, 2001). For site response modeling at Yucca Mountain, DOE obtained these data from combined resonant column and torsional shear tests performed at the University of Texas (Bechtel SAIC Company, LLC, 2002). Test were carried out on both alluvium and tuff samples collected from boreholes at the site over a shearing-strain range from about 10^{-4} percent to slightly more than 10^{-1} percent.

Because of variability in the test data and the limited range of testing, curves are fit to the measured data assuming a simple model of soil behavior under simple shear loading conditions. Electric Power Research Institute (EPRI) developed simplified hyperbola to represent this behavior (EPRI, 1993). The EPRI (1993) hyperbolic curve has an initial slope that is equal to Gmax (the shear modulus at small strains) and is asymptotic to τmax (the shear strength). The ratio of τmax to Gmax, defined as the reference strain, γ_r, is useful both in expressing the stress-strain relationship in mathematical form and as a measure of the relative values of τmax and Gmax. Soils with larger values of γ_r have greater shear strengths relative to their small strain modulus and show more elastic stress-strain behavior than soils with smaller values of γ_r. Thus, gravelly soils have low reference strain values and more plastic clays have high values.

For site response modeling at Yucca Mountain, DOE developed two sets of strain reduction and damping curves in order to account for the uncertainty in these properties between the values measured under laboratory conditions and the values that may arise because of actual *in-situ* conditions. Thus, the "upper mean tuff (UMT) curves" shown in Figure 2-3 represent best fit EPRI curves to the laboratory measurements and the "lower mean tuff (LMT)" curves in Figure 2-3 are adjusted downward to account for *in situ* fracturing and heterogeneity. Details of the development of these curves is provided in Bechtel SAIC Company, LLC (2004).

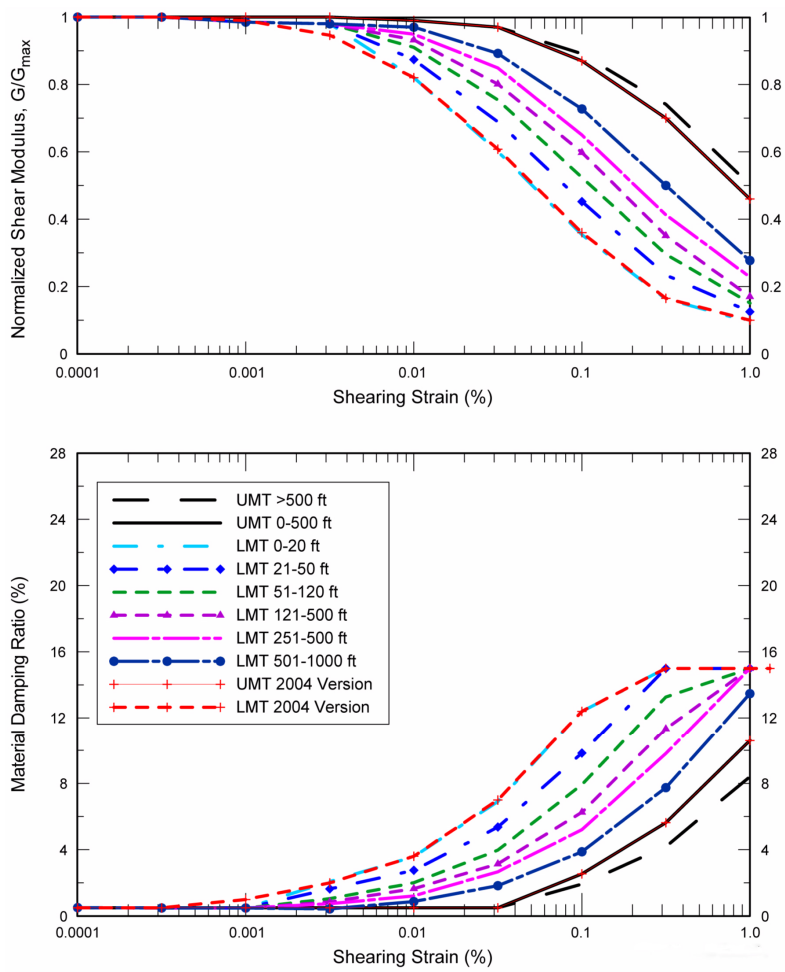

Figure 2-3. Shear modules and damping curves for tuff strata at Yucca Mountain based on analyses in Bechtel SAIC Company, LLC (2002) and updated analyses in Bechtel SAIC Company, LLC, 2008.

3 SOIL STABILITY ANALYSES

The purpose of evaluation of stability of subsurface materials and foundations is to determine if the soil beneath a facility exhibits properties that ensure the facility will remain stable under static and seismic conditions. The soil must be capable of carrying load from the structure without shear failure and the resulting settlements from the structure should be within tolerable limits. Instability in soil foundation may compromise structural integrity and intended safety functions of the in-structure equipment and components. To determine the bearing capacity and settlement potential of the *in situ* soils, a comprehensive soil stratigraphy should be developed that will adequately characterize the lateral and vertical extent of all soil units beneath the proposed facility. The static and dynamic engineering properties of soil and rock strata underlying the surface facilities are based on laboratory and *in situ* test results, and these data are collected and processed using accepted industry techniques. The adequacy of site and laboratory investigations and the stability evaluation requirements are reviewed using guidance in Section 2.5.4, NUREG-0800 (NRC, 2007) and the NRC regulatory guides referenced therein.

The DOE surface facility design at Midway Valley envisioned several building structures where radioactive waste would be handled and repackaged, and an aging facility where radioactive waste and spent nuclear fuel would cool down prior to emplacement underground (DOE, 2008). Similar to many of the large structures at the Nevada National Security Site, the DOE design proposes construction of surface building structures containing reinforced concrete interior and exterior shear walls, floor and roof diaphragm slabs, and mat foundations. In DOE's design, the foundation mat size varies with each building structure, with a maximum mat size of 120 × 130 m [390 × 430 ft]. The thickness of the mat varies between 1.8 and 2.1 m [6 and 7 ft]. The ratio of the length to width varies from 1.0 to 1.3.

The surface facility site is underlain by Quaternary alluvium and colluvium, which overlies a sequence of volcanic tuff. The alluvium thickness varies in the east-west direction from approximately 9.1 m [30 ft], increasing to approximately 61 m [200 ft] thick below the foundations of the critical structures. The alluvium was characterized using data from borehole logs, test pits, and trenches. The alluvium at the surface facility site consists of dense, coarse-grained granular deposits of gravel with sand or sand with gravel, with minor amounts of cobbles and fines (silt and clay). The soil at Yucca Mountain is not saturated, and the water table is approximately 390 m [1,300 ft] below the ground surface. As a result, liquefaction and lateral spreading hazards are not deemed credible at Yucca Mountain because these hazards require saturated soil conditions.

3.1 Soil Strength

The strength of soil is most commonly expressed as Mohr-Coulomb failure criteria. The shear stress at failure is a function of cohesion and friction angle. Soil data required for shear strength evaluation is obtained by standard experimental procedures (e.g., shear box, triaxial, and consolidation tests). Collection of soil data required for shear strength evaluation using these standard test procedures is difficult at sites similar to the proposed surface facility area because of the highly coarse-grained nature of the alluvium. In general, for granular soil deposits, cohesion is zero and the soil strength is defined by angle of internal friction.

To estimate the shear strength of the alluvium, the DOE conducted laboratory and field investigations to measure the relative density of the alluvium. Relative density is a measure of how dense sand is compared with its maximum density. DOE used relative density to estimate the angle of internal friction from several correlations and charts available in the literature (e.g., Bowles, 1996). To obtain relative density, DOE first determined the bulk density of the alluvium from geophysical measurements in seven boreholes (Bechtel SAIC Company, LLC, 2002a, b) and water-replacement and sand-cone density tests in test pits. Maximum and minimum density indices were measured in the laboratory from samples obtained from the test pit locations. Relative density of the alluvium was then calculated from the minimum and maximum density indices and *in-situ* measured bulk densities. DOE used the relative density data and empirical relationships between relative density and shear strength to define the shear strength of the alluvium. Using five correlations available in the literature and for the range of relative densities considered, the angle of internal friction ranged from 33° to 52°. DOE proposed an internal friction angle of 39° with zero cohesion to represent the shear strength parameters of the alluvium at the site (DOE, 2008). DOE modeled alluvium below the foundation as a single material.

Beneath the alluvium in Midway Valley, bedrock consists of welded and non-welded volcanic tuff units. Using the data collected from geophysical and geologic logs (e.g., shear wave velocity and density data and measurements of unconfined compressive strength of tuff at the North Ramp and repository block), the bedrock is considered to have greater strength and be less compressible than the alluvium. Thus, deformation of the tuff is not likely to have any significant effect on the stability of subsurface materials underlying the proposed surface facility structures. Deformation would be most likely to occur in the alluvium.

3.2 Settlement

Settlement is the permanent displacement of a foundation caused by compression and deformation of the underlying soil. Uniform settlement of a structure may not be of concern to structural performance; however, differential settlement, or variations in the amount of settlement between two different locations of large mat foundations, may induce stresses in a foundation, which could potentially compromise a foundation's structural integrity. Excessive uniform settlement of a structure relative to the surrounding soil also could damage equipment and components (e.g., piping) that interface between the soil and structure. Similarly, differential settlement between structures may affect the equipment and components that span the gap between the structures.

For maximum allowable settlement for mat foundations at the surface facilities, DOE relied on guidance and recommendations from Bowles (1996) and Terzaghi et al. (1996). In general, for mat foundations designed on sand, Bowles suggests a maximum differential settlement range of 35 to 65 mm [1.4 to 2.6 in] (Table 5-7, Bowles, 1996). Differential settlement is related to the total settlement by empirical relations. Based on observed data, Bowles (1996) recommends that differential settlement is three-fourths of the total settlement; thus, the range of the total settlement for a mat foundation is estimated to be 47 to 87 mm [1.9 to 3.4 in]. Terzaghi (Section 51.2.1, 1996) recommends a maximum settlement of 50 mm [2.0 in] for raft foundations. DOE used 50 mm [2 inch] as the maximum allowable settlement in their soil stability analysis. The limiting differential settlement between adjacent footings is assumed to be three-fourths of the maximum estimated settlement value, based on Peck et al. (1974). The allowable angular distortion (allowable differential settlement over a given distance) for a building is assumed to be 1/500, based on Fang (1991).

For facilities at Midway Valley, elastic settlement was estimated by DOE based on elastic theory, where the stress profile under the mat was computed using a Boussinesq equation for a uniform vertical load (Bechtel SAIC Company, 2007c). In practice, the incremental strain profile under the mat can be computed using an iterative procedure that accounts for the degradation of Young's modulus with strain. The short-term and long-term settlements of foundations at the site were computed by DOE using the Burland and Burbidge procedure and Schmertmann et al. method, which are discussed in Terzaghi et al. (1996, Section 50.2.5 and 50.2.6). Both approaches use field measurements and empirical relations to compute the settlement. The Burland and Burbidge approach uses the soil average standard penetration test blow count (N_{60}) values to estimate the soil's vertical compression. The foundation width, vertical compression coefficient, the bearing pressure, and the settlement are related through an empirical relation. The Schmertmann et al. method is based on field measurements of vertical strain beneath shallow footings. It uses the elastic soil modulus obtained from cone-penetration tip resistance. The soil modulus, vertical strain influence factor, foundation dimensions, and bearing pressure are related empirically to estimate settlements.

3.3 Bearing Capacity

Bearing capacity is the ability of soil to safely carry the pressure placed on the soil from any engineered structure without undergoing a shear failure with accompanying large settlements. Ultimate bearing capacity is the theoretical maximum pressure (developed under a foundation) that can be supported without exceeding the limiting shear resistance of the soil. The allowable bearing capacity is the ultimate bearing capacity (based on soil strength) divided by an appropriate factor of safety adequate to avoid base shear failure. Factor of safety against bearing capacity failure for mat foundation design is generally used as greater than 3.0 (Table 1-2, U.S. Army Corps of Engineers, 1994). Applying a bearing pressure that is safe with respect to soil failure may not, however, ensure that settlement of the foundation will be within acceptable limits. At Midway Valley, because of large footprints of mat foundations and the dense nature of granular alluvium, shear failure of sand is less likely and soil settlement is typically of greater concern, consistent with Peck et al. (1994).

In general, shallow foundations may experience a reduction in bearing capacity and increase in settlement and tilt due to seismic loading. During an earthquake, bearing capacity failure may be caused by cyclic degradation of soil strength and excess pore pressure may cause soil liquefaction beneath and around the foundation, leading to large settlement and tilting. In addition, redistribution of pore water pressure after an earthquake may adversely affect the stability of the foundation. Because the water table at Yucca Mountain is approximately 390 m [1,300 ft] below the ground surface and the alluvium is not saturated, earthquake-induced bearing capacity failure, liquefaction, and post-earthquake instability are not of concern at the site.

Building codes generally permit an increase in allowable bearing capacity when earthquake loads, in addition to static loads, are used in design of the foundation. This increase in allowable bearing capacity may be considered reasonable for dense granular soils, stiff to very stiff clays, or hard bedrocks, such as at Midway Valley. However, a large horizontal inertial force due to an earthquake (dynamic loading) may cause the foundation to fail by sliding or overturning, and therefore these possibilities should be evaluated.

The methodology to determine the allowable bearing pressure applicable to rectangular foundations on sand and non-plastic silts is described in Bowles (1996) and Terzaghi et al. (1996). For wide foundations (e.g., mat foundations of structures such as those proposed by

DOE), the bearing pressure is controlled by settlement criteria. Using the shear strength of alluvium on the basis of an internal friction angle of 39°, DOE evaluated the allowable bearing pressure for: (i) square and strip footings with no limit on settlement; and (ii) square and strip footings for maximum allowable settlement of 50 mm [2.0 in]. DOE found that the allowable bearing pressure from condition (i) increased as the footing width increased. The results for this condition determined potential limits on foundation loading without causing a generalized shear failure of the subsurface materials (i.e., rotational failure of the foundation and underlying materials). For condition (ii), DOE conducted analyses to determine potential limits on foundation loading without causing excessive settlement due to localized shear failure of the subsurface materials. The analysis yielded results of allowable bearing pressure that decreased as the footing width increased, but asymptotically approached a minimum value for large footing widths. This minimum value establishes foundation pressure criterion for the allowable settlement. For a maximum allowable settlement of 50 mm [2.0 in], DOE estimated a bearing pressure of 480 kPa [10 ksf] for foundation width greater than 46 m [150 ft] (DOE, 2009). For seismic loading conditions, DOE proposed an allowable bearing pressure of 2,400 kPa [50 ksf], based on laboratory and field test data. This analysis considers rotational shear failure of the foundation material, which does not include a limit on foundation settlement.

DOE design of mat foundations at the surface facility is based on a finite element (FE) model in which the subsurface material is represented by soil springs and the resulting deformation is used to calculate foundation pressures and settlements. The FE model is used to check that the calculated pressures and settlements are within the allowable limits. Similarly, DOE performed an FE analysis to design mat foundations for the design-basis seismic load.

3.4 Coefficient of Subgrade Reaction

In practice, the seismic design of foundations requires an evaluation of the soil springs for the soil-structure interaction. DOE's soil structure analysis is based on a three dimensional SAP2000 model (Bechtel SAIC Company, LLC, 2007d). In this model, foundation mats, modeled as "shell" elements, are discretized into several finite elements. Each foundation mat is coupled with a "beam-stick" model that represents the structure. The design analysis involves evaluation of bending moments and shear forces in each finite element mesh. In addition, the analysis evaluates the sliding and overturning potential.

To consider the stiffness properties of the soil underlying a foundation mat, a series of nonlinear (compression only) springs can be computed. The soil spring uses thickness of the alluvium to compute global spring (three translational and three rotational) and the global spring must be converted into individual springs applied to each node of the foundation finite element mesh.

Consistent with standard practice, DOE developed their dynamic foundation analysis using the following methodology. The DOE evaluation of frequency, independent soil springs, and corresponding percent of critical damping values for a rectangular foundation ewas based on ASCE 4-98 (ASCE, 1998, Table 3.3-3). Soil springs are a function of the foundation plan dimensions width and length, the soil dynamic shear modulus, and Poisson's Ratio. Damping values are computed using ASCE 4-98 (ASCE, 1998, Table 3.3-1). The shear wave velocity under surface facilities typically varies with depth. The dynamic shear modulus, which is a function of the shear wave velocity, also varies with depth. DOE computed an equivalent shear modulus to determine the frequency for independent soil springs using ASCE 4-98. A methodology developed by Hadjina and Ellison (1985) was used for evaluating the equivalent property of layered soil media assuming the soil layers to be linear-elastic. In this approach, the soil media is divided into several horizontal soil layers. The dynamic shear modulus is

computed using shear wave velocity, and soil modulus of each layer is calculated from shear modulus and Poisson's ratio. The vertical displacement of each layer is determined using the layer thickness, modulus of elasticity, and Boussinesq coefficient from Newmark's coefficient influence diagram (Bowles, 1996). Assuming equality of the total displacement of the layered site to the equivalent uniform soil media, the equivalent elastic modulus and shear modulus can be evaluated. Equivalent spring constants can be determined for horizontal, vertical, and rotational motions based on ASCE 4-98 (ASCE, 1998). Using this approach, DOE calculated the global spring constants for mat foundations of each structure based on the thickness of underlying alluvium. Local spring constants were then determined by dividing the global vertical and horizontal springs by the area of the mat foundation, and rotational spring was obtained by dividing the moment of inertia of the mat foundation with respect to the centroidal axis. The local spring and rotational spring constants were applied to the finite elements of the base mat model.

4 SITE RESPONSE

Amplification of earthquake energy is a well-known phenomenon at sites built on unconsolidated soil. Near-surface rock and soil materials generally cause an increase of seismic wave amplitude due to a decrease in material velocity near the surface. In an elastic system, the wave amplitude increases with decreasing wave velocity and this increase can be compensated by material damping in the layers. Resonance also occurs in soil and rock layers and between the surface and the soil/bedrock interface, and resonance frequency and amplitude are affected by material damping. Ground response modeling is used to account for soil amplification effects. These ground response models take earthquake time histories or response spectra, selected to represent the seismic hazard for the underlying bedrock, and transform them into the equivalent time histories or response spectra at the top of the soil column. The amplification of earthquake energy as it propagates from bedrock through the soil column is largely a function of physical properties of the soil; specifically, velocity, density shear, modulus reduction, and damping. Detailed information about the physical properties of the material above bedrock is needed to develop reliable ground response models.

At Yucca Mountain, the seismic hazard was defined at four control elevations (Figure 4-1). The PSHA was developed for a reference hard rock outcrop (Point A in Figure 4-1). The purpose of the site-response ground motion model is to incorporate the effects on earthquake ground motions of the approximately 300 m [980 ft] of rock above the emplacement levels beneath Yucca Mountain (Site B), the surface of Yucca Crest (Point C), and the soil (alluvium and colluvium) beneath the site of the Surface Facilities Area in Midway Valley (Site D). The resulting probabilistic ground motions at Points B and D were then used as inputs to the preclosure safety analysis, preclosure design, and postclosure performance assessment.

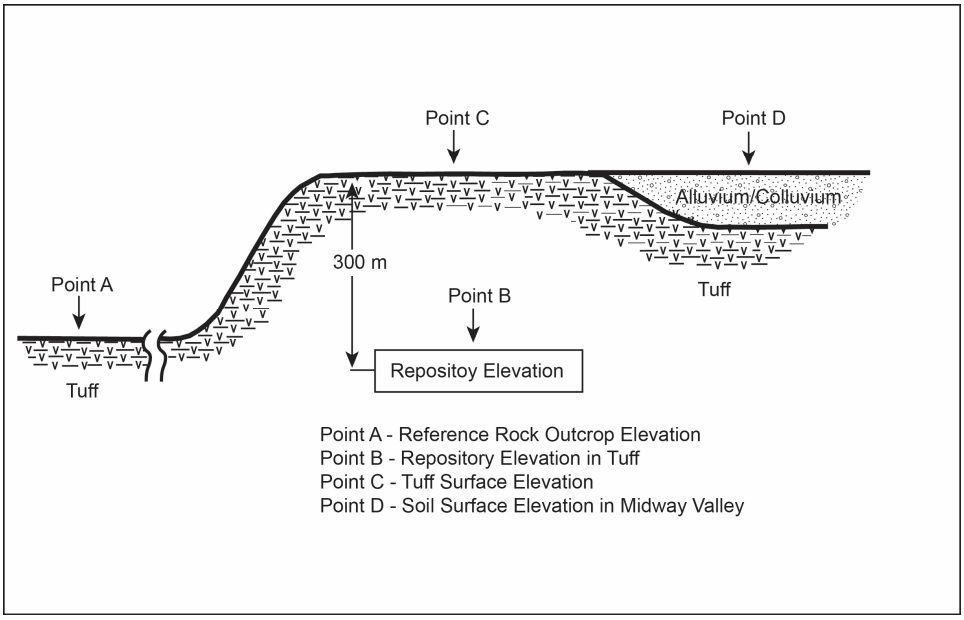

Figure 4-1. Schematic representation of the locations at Yucca Mountain where seismic ground motions were developed. This figure was adapted from Figure 2 in Stepp et al. (2001).

4.1 Methodology

Site response modeling involves a numerical model to represent the propagation of earthquake motions from the base rock through the overlying strata (soil, alluvium, colluvium, or other sedimentary layers) to the ground surface. Site response analysis provides surface acceleration-time series, surface acceleration response spectra, and/or spectral amplification factors based on the dynamic response of the local strata conditions. Seismic site response is typically achieved using a random-vibration theory, 1D equivalent-linear models. More sophisticated two-dimensional (2D) and three-dimensional (3D) models exist, including those that incorporate nonlinear effects, but these models require significant computational horsepower and more detailed characterization of the site than the 1D models. Kottke (2010) investigates and compares the results from equivalent-linear time series analysis, equivalent-linear random vibration theory analysis, and nonlinear time series analysis. Stewart et al. (2008) provides a complete review of various nonlinear models and discusses the calibration of nonlinear methods to the equivalent-linear method and recordings from borehole arrays.

Nearly all 1D site response analyses assume that plane shear waves (S-waves) propagate vertically upwards in the strata column. Departures of soil response from a linear constitutive relation are treated in an approximate manner through the use of the equivalent linear formulation (Idriss and Seed, 1968). Geological/geotechnical inputs to the site-response ground motion model include the site-specific base rock ground motion hazard curves, like the ones developed in the DOE 1998 PSHA (CRWMS M&O, 1998), and geotechnical information like that described in Chapter 2 of this report (namely V_S and V_P, density; and the dynamic material properties).

Traditionally, these 1D site response methods use a series of acceleration-time series as the input rock motion. These input time histories are modified to be compatible with the specified outcrop response spectra, and thus serve as control (or input) motions. The control motions are then used to drive an equivalent linear computational formulation to transmit the motions through the soil profile. Simplified analyses generally assume vertically propagating shear (S)-waves for horizontal components and vertically propagating compression (P)-waves for vertical motions. The time histories are selected from actual strong motion records from earthquakes with a magnitude, style of faulting, and site-to-source distance that match the controlling earthquakes derived from deaggregation of the input base rock hazard at the appropriate annual exceedance probabilities. To make them compatible, these time histories are scaled such that their corresponding response spectra match the response spectra of the input motions from the base rock hazard. The most common 1D site response code is SHAKE, which was originally developed by Schnabel et al. (1972). There have been many commercial and academic versions of the code since then (e.g., SHAKE91by Idriss and Sun 1993; SHAKE2000 by Ordónez, 2002; STRATA by Kottke and Rathje 2008) but the underlying algorithm remains similar to the original. Alternatively, random vibration theory can be applied to an equivalent linear analysis, such that only a Fourier amplitude spectrum is used as input, and the selection of input time histories and their scaling to match the hazard is avoided. The computer code RASCALS (Silva and Lee, 1987) was developed for this alternative analysis.

For the site response at Yucca Mountain, DOE relied on the RASCALS code. Bechtel SAIC Company, LLC (2004) provides a detailed description of the DOE analysis, including the selection of control motions, scaling of control motions, and deaggregation. The DOE relied on guidance in NUREG/CR–6728 (Risk Engineering, 2001). There are five approaches (1, 2A, 2B, 3, and 4) described in NUREG/CR–6728 to conduct this analysis. The approaches are each applicable under certain circumstances, according to available data and information. The

site-specific data and information needed increase with each successive approach, and the resulting analyses yield increasing levels of accuracy. Approach 4 requires site-specific soil attenuation models based on detailed observations of earthquake data at the site which were not available for Yucca Mountain. Initially, DOE relied on Approach 2B for the preclosure analysis and Approach 3 for the postclosure analysis. Subsequently, DOE modified the preclosure analyses to be consistent with Approach 3.

4.2 Staff Confirmatory Analyses

During review of the DOE license application, the NRC and CNWRA staffs developed a set of confirmatory analyses to evaluate the DOE seismic hazard results for the surface facility site in Midway Valley. The staff analysis used SHAKE2000, with stratigraphic profiles derived from the analysis of the DOE geotechnical data in Gonzalez at al. (2004). In addition to using SHAKE2000 rather than RASCALS, the main difference in the staff analyses was the way in which the V_S, V_P, and density data were varied statistically to account for uncertainty and variability.

In the DOE analysis, four base case profiles (for both shear-wave and compressional-wave velocity) were used, along with statistical correlation of layer thicknesses and layer velocities, to develop a suite of random velocity profiles for RASCALS model input. For uncertainty in the dynamic material properties, two sets of normalized shear modulus reduction and damping curves were developed (as described in Section 2.2.4 of this document and shown in Figure 2-3). One important aspect of the DOE model was that the V_S, V_P, and density values were allowed to vary both laterally and vertically simply based on the range of observed measurements and not correlated to specific stratigraphic layers. Thus, the layer thicknesses in the 1D profiles in these models can vary significantly from one run to the next (Figure 4-2). Details of the DOE approach are given in Bechtel SAIC Company, LLC (2004 and 2008).

In the NRC and CNWRA staff analysis, only a single base case profile was used based on the DOE mean profile. However, the V_S, V_P, and density data were averaged within each of the stratigraphic layers based on the conclusion in Gonzalez et al. (2004) that these values correlate well within the different tuff an alluvial layers (Figure 4-3). Thus, unlike the DOE model, layer thickness in the site response model is controlled by actual lithologic relationships. To account for variation in the thickness of these layers across the site, SHAKE2000 runs were completed for 26 of the DOE boreholes. Figure 4-4 illustrates the variability of the lithologic profiles across the site for several representative boreholes. Two input time histories were used, one record from the 1997 M_W 6.0 in Umbria, Italy and one from the 1994 M_W 6.7 Northridge earthquake that was recorded at Sylmar station. These earthquake time histories were selected because they were from moderate magnitude events that were recorded relatively close to the source, and therefore compatible with the dominant sources near Yucca Mountain (e.g., Stepp et al., 2001). Comparison of the response spectra to the target response spectra from the DOE 2008 PSHA showed a reasonable match. Because the comparison was favorable, these time histories were not scaled to get an exact match.

In the analysis, the borehole log was used to identify the stratigraphic layers, and these were input into SHAKE2000 code. For each of these stratigraphic layers, the weighted mean values for V_S and density for each layer, as determined in Gonzalez et al. (2004), were then input into the code. Results from the input ground motions were then compared with the output results to generate the amplification factors. Results were obtained for spectral frequencies between 0.125 and 25 Hz.

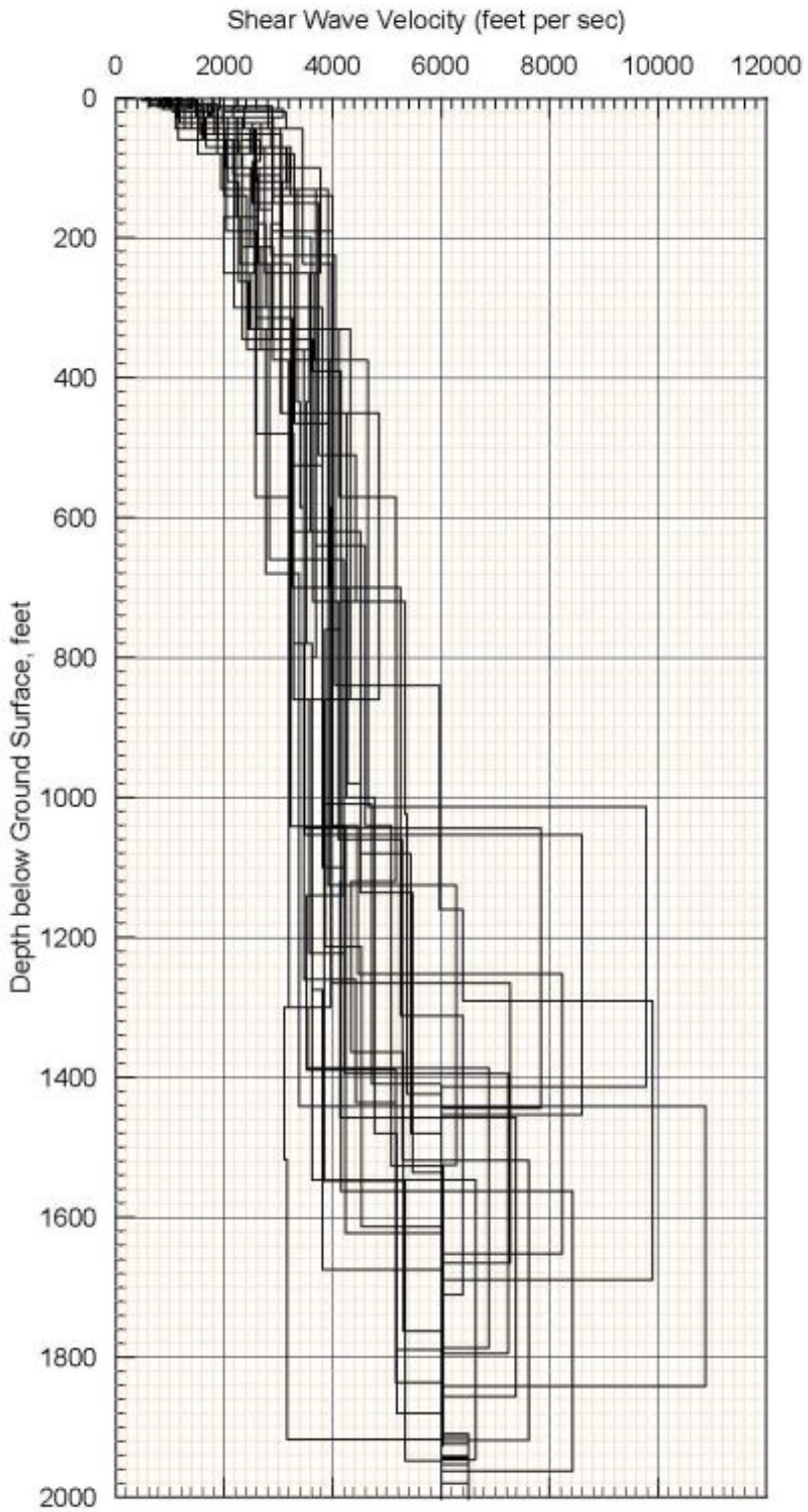

Figure 4-2. Sample of randomized V_S velocity profiles for surface facility site in Midway Valley, from Figure 6.2.4-95 of Bechtel SAIC Company, LLC (2008).

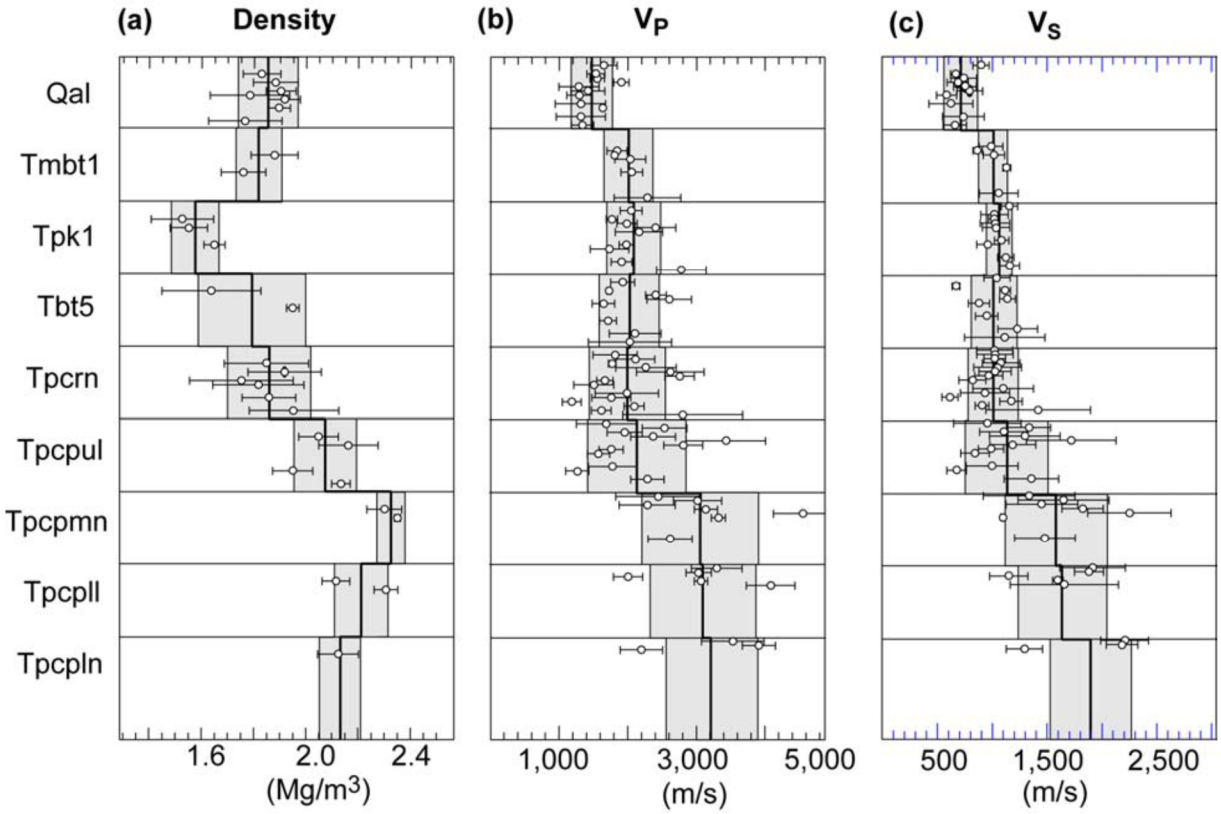

Figure 4-3. Density shown in (a), V_P shown in (b) and V_S shown in (c) plotted as a function of stratigraphy. The circles with 1-sigma error bars are the mean values from each borehole where velocity or density data for that unit were obtained. The thick black line with 1-sigma shaded error band is the weighted mean of the borehole data. Redrafted from Figure 3-5 of Gonzalez et al. (2004).

Results from this analysis showed that there is amplification at low frequencies - below about 5 Hz - but that at higher frequencies, the energy is deamplified (Figure 4-5). As documented in the NRC Safety Evaluation Report for Yucca Mountain (NRC, 2015), these results were consistent with the results from DOE (Bechtel SAIC Company, LLC, 2015). The deamplification of the high frequency ground motions is an interesting observation that appears to be consistent at sites underlain by alternating layers of rigid volcanic rocks sandwiching layers of unconsolidated sediments or poorly welded volcanic ash. For example, a similar deamplification of the high frequency ground motions was observed at the Columbia Nuclear Generating Station (Energy Northwest, 2013). At this site, the underlying stratigraphy consists of alternating layers of crystalline basalt and fine-grained unconsolidated fluvial and lacustrine sediments. The high frequency deamplification at this site was attributed to the strong impedance contrast between the interbed and basalt layers and material damping in the interbeds.

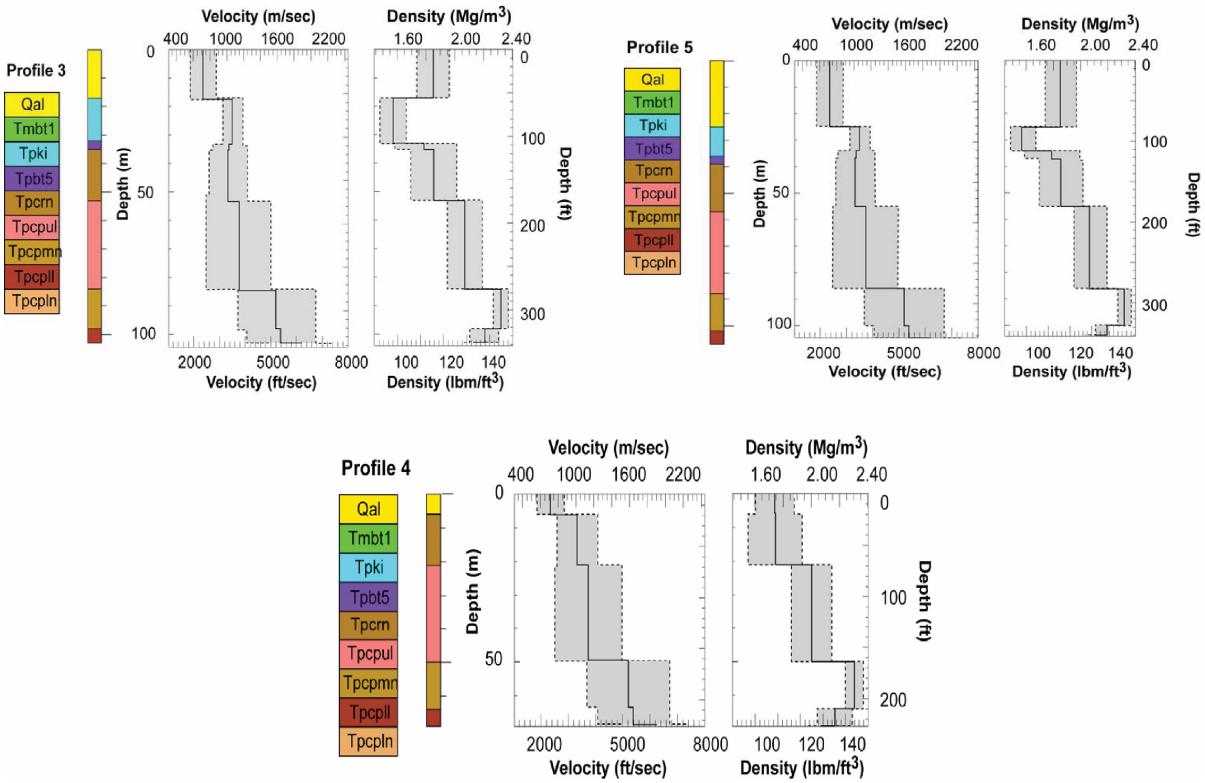

Figure 4-4. Example stratigraphic profiles used in the SHAKE2000-analysis. These profiles were redrafted from Figures 4-6, 4-7, and 4-8 of Gonzalez et al. (2004).

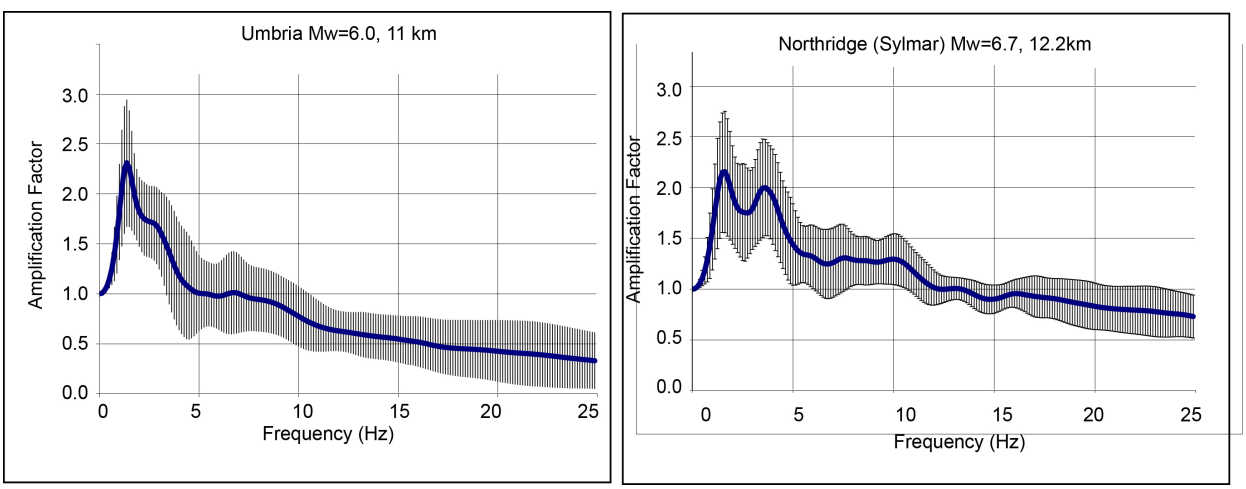

Figure 4-5. Composite amplification factor results from the SHAKE2000 site response analysis.

5 SUMMARY

To develop design inputs for surface facilities and subsurface drifts and installations, DOE conducted extensive geotechnical investigations of the site bedrock, alluvium, and soils. These investigations included field and laboratory testing as well as finite element modeling. DOE conducted these investigations to obtain the necessary information to assess the static and dynamic material properties of the bedrock, alluvium, and soils to assess the bearing capacity and settlement under both static and dynamic loads.

The surface structures at Midway Valley are proposed to be constructed as reinforced concrete shear walls, floor and roof diaphragm slabs, and mat foundations. In general, instability in soil foundations may compromise structural integrity and the intended safety functions of the in-structure equipment and components. Stability evaluation of subsurface materials and foundations determines whether the soil beneath a facility exhibits properties that ensure the facility will remain stable under static and seismic conditions. The surface facility site at Yucca Mountain is underlain by Quaternary alluvium and colluvium with thickness varying from approximately 9.1 to 61 m [30 to 200 ft] below the foundations of the proposed critical structures. The alluvium consists of dense, coarse-grained granular deposits of gravel with sand or sand with gravel and minor amounts of cobbles and fines. DOE characterized the granular soil media by a high friction angle of 39° and zero cohesion and low compressibility. Beneath the alluvium at Yucca Mountain, bedrock consists of welded and non-welded volcanic tuff units, which are considered to be higher strength than the alluvial soil. Because of the proposed large footprints of mat foundations and the dense nature of granular alluvium, shear failure of sand is less likely and soil settlement is typically of greater concern. Because the water table is approximately 390 m [1,300 ft] below the ground surface and the alluvium is not saturated, earthquake-induced bearing capacity failure, liquefaction, and post-earthquake instability are not of concern at the Yucca Mountain site. DOE's design of mat foundations at the surface facility under static and seismic loading is based on a finite element model in which the subsurface material is represented by soil springs, and the resulting deformation is used to calculate foundation pressures and settlements.

The purpose of the site-response ground motion model is to incorporate the effects from earthquake ground motions through the approximately 300 m [1,000 ft] of rock above the emplacement levels beneath Yucca Mountain and the soil and rock beneath the site of the Surface Facilities Area. The site response model convolves the DOE PSHA (CRWMS M&O, 1998), which was derived for a reference base rock condition, with site specific geotechnical data to develop ground motions for the surface facilities area in Midway Valley and the repository host horizon in Yucca Mountain. The DOE site response was based on a random-vibration theory, 1D equivalent-linear model. Geological/geotechnical inputs to the site-response ground motion model include the site-specific base rock ground motion hazard curves, V_S, V_P, density, shear modulus reduction, and damping. In addition, DOE followed the specific guidance for Approach 2B and Approach 3 given in NUREG/CR-6728 (Risk Engineering, 2001).

The NRC and CNWRA staffs developed a simplified confirmatory analysis of the DOE site response. Unlike the DOE approach, which randomized the V_S and density based on the geophysical measurements, the staff developed lithologic-specific V_S and density values. Randomization of these values was achieved by evaluating the site at 26 bore sites across Midway Valley. Results from this analysis show that there is amplification at low frequencies—below about 5 Hz—but that at higher frequencies, the energy is deamplified. As documented in

the NRC Safety Evaluation Report for Yucca Mountain (NRC, 2015), these results were consistent with the results from DOE (Bechtel SAIC Company, LLC, 2015). The deamplification of the high frequency ground motions is an interesting observation that appears to be consistent with site response results from other sites underlain by alternating layers of rigid volcanic rocks sandwiching layers of unconsolidated sediments or poorly welded volcanic ash.

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