Site response projections for deep sediment columns and earthquake microzonation for the Las Vegas basin

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SITE RESPONSE PROJECTIONS FOR DEEP SEDIMENT COLUMNS AND
EARTHQUAKE MICROZONATION FOR THE LAS VEGAS BASIN

by

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ABSTRACT

Site Response Projections for Deep Sediment Columns and Earthquake Microzonation for the Las Vegas Basin

by

Ying Liu

Dr. Barbara Luke, Examination Committee Chair
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Deep sediment columns play a significant role in defining surface response to earthquakes. For the Las Vegas basin (Nevada, U.S.), the basin sediments are capable of amplifying ground motions by a factor of up to 5 to 10 with respect to motions recorded at near-rock sites at the edges of the basin, over the period range 0.3 to 5 sec.

A one-dimensional (1-D) equivalent-linear model is used to study the impact of sediment columns on surface response. The 1-D model is optimally parameterized through iterative assessment to select the depth to halfspace so that the projected response spectrum best matches the measured or anticipated response. This approach compensates for intrinsic uncertainties associated with shear wave velocity and dynamic soil properties at great depths.

The established procedure adequately captured ground motion amplification in the period range 0.2 to 1 sec, which has engineering significance for 2- to 10-story structures. The appropriate depth to halfspace was 400 m.
Monte-Carlo simulation can be used to generate bounding response spectra. This approach is especially useful for site response studies for deep deposits (>200 m).

Shear wave velocity profiles can be effectively and efficiently characterized to significant depths (to 300 m or more in Las Vegas) using combined active- and passive-source surface wave measurements.

Parametric studies using Monte Carlo simulation revealed that (1) a high $V_s$ inclusion can amplify as well as deamplify the surface response; and (2) site response analyses using $V_s$ averaged over the upper 30 m can underestimate surface response.

Earthquake hazard in the Las Vegas basin is significant. Six faults are found to have high earthquake potential. The basin can be zoned in two major site response units according to predominant near-surface grain size. Bounding response envelopes are developed for each zone based on a deterministic site response projection, using the 1-D model, lithologic and shear wave velocity databases, and Monte-Carlo simulation. The upper bound values of peak ground acceleration and peak spectral acceleration for the Las Vegas basin are on the order of 0.3 g and 1 g, respectively.
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CHAPTER 1

INTRODUCTION

1.1 Purpose of the Study

The goals of this research are to develop an appropriate procedure for assessing earthquake ground motion hazard for deep sediments and to map seismic response zones and determine their surface response envelopes for the Las Vegas basin. This research also addresses the influences of thin, stiff layered inclusions on surface response and the appropriateness of assigning site classes based solely on an averaged shear wave velocity \( (V_S) \) over the upper 30 m.

1.2 Research Questions

This research used a one-dimensional (1-D) equivalent-linear site response model, placing the model halfspace within the soil column, to study seismic response of deep sediments. To establish surface response envelopes, this research incorporated the Monte-Carlo simulation technique. The terminology and the logic for using these methods are explained in the subsequent chapters. This research answers two main questions.

1) What are the effects of shallow sediments on surface shaking of deep soil deposits?

To answer this question, the following subordinate questions need to be answered:
a) For a 1-D site response analysis of a deep soil deposit, what is the appropriate procedure to select the optimum depth to model halfspace?

b) Over what frequency ranges do shallow sediments influence site response in deep deposits?

c) Can a credible response envelope be constructed by Monte-Carlo simulation using the 1-D site response analysis, taking into consideration uncertainties introduced by lateral variability and limited knowledge of $V_s$ and dynamic soil properties at depth?

d) How do high-velocity inclusions at shallow depths affect surface shaking?

e) For deep deposits with complex $V_s$ profiles, is it appropriate to define the shallow site response based solely on $V_s$ averaged over the upper 30 meters?

2) Based on current geological, geotechnical, and geophysical knowledge about Las Vegas basin, what is the geographic distribution of the intensity of surface shaking in the Las Vegas basin, under the influence of a credible seismic event?

To answer this question, the following subordinate questions need to be answered:

a) What is the shallow $V_s$ structure of the Las Vegas basin?

b) Can the $V_s$ profiles be correlated with sediment types to form a viable and more detailed basin-wide site response model?

c) What are the dominant sediment types in the near surface of the Las Vegas basin, and how are they distributed?

d) What are the potential seismic sources that are most likely to have severe impact on the Las Vegas basin?
1.3 Significance of the Study

For 1-D equivalent-linear seismic response analysis of deep sediments, researchers are facing two impediments: 1) the uncertainty associated with dynamic soil properties appropriate for elevated confining pressures existing at great depths, and 2) the uncertainty in defining the soil-bedrock interface. This study establishes a procedure that addresses the two impediments by properly parameterizing the soil column and incorporating Monte-Carlo simulation. The approach established in this study is applicable to any seismic response analyses for deep soil deposits.

During historical high-energy underground explosion events and earthquakes, the Las Vegas basin has demonstrated its capacity to amplify ground motions. Due to its large population, rapid growth rate, unique building inventory (unique casino hotel resorts and high-rise condominiums along the famous Las Vegas strip), and the continuing disclosure of significant young faults (e.g., Black Hills fault (Fossett et al., 2003) and California Wash fault (Bidgoli et al., 2003; Saldaña et al., 2004)), the seismic risk in the Las Vegas basin is high. However, the ground motion hazard in the Las Vegas basin has not been well understood. In this study, the basin is divided into two major seismic response units and response envelopes for each are generated. The results of this study are beneficial for developing seismic hazard mitigation measures and planning future development of the metropolitan area.
1.4 Definition of Key Terms

Microtremor: a faint earth tremor over a wide frequency range that is unrelated to any earthquake and caused by a variety of usually incoherent natural and artificial sources.

Seismic hazard: “Any physical phenomenon (e.g., ground shaking, ground failure) associated with an earthquake that may produce adverse effects on human activities” (EERI committee on seismic risk, 1984).

Seismic microzonation: “The process of determining absolute or relative seismic hazard at many sites, accounting for the effects of geologic and topographic amplification of motion and of soil stability and liquefaction, for the purpose of delineating seismic microzones. Alternatively, microzonation is a process for identifying detailed geological, seismological, hydrological, and geotechnical site characteristics in a specific region and incorporating them into land-use planning and the design of safe structures in order to reduce damage to human life and property resulting from earthquakes” (EERI committee on seismic risk, 1984).

Seismic microzone: “A generally small area within which seismic-design requirements for structures are uniform. Seismic microzones may show relative ground motion amplification due to local soil conditions without specifying the absolute levels of motion or seismic hazard” (EERI committee on seismic risk, 1984).

Seismogenic: capable of generating earthquakes. Seismogenic sources may or may not be directly associated with known faults. Seismogenic sources are normally determined by combing active fault data and historical earthquake information (http://earthquake.usgs.gov/learning/glossary.php, as of 03/14/2006).
1.5 Organization of this Dissertation

Following this introduction, Chapter Two gives background information on the geologic setting of the Las Vegas basin, the observed ground motions, and a literature review for seismic response analyses and earthquake microzonation. The methodology used in this study is developed based on the literature review, previous seismic response studies in the Las Vegas basin and available data.

Chapter Three documents the $V_S$ measurements in the Las Vegas basin. Existing $V_S$ datasets in the Las Vegas valley were archived and twelve new $V_S$ measurements at selected sites across the basin were conducted. For the twelve new $V_S$ measurements, a combined usage of active- and passive-source surface wave methods was adopted.

In Chapter Four, the spatial relationships among surface response, basin depth, sediment distribution, and $V_S$ are discussed. A direct correlation between sediment type and $V_S$, taking into consideration the depth factor, is conducted. The correlation between site stiffness, which is controlled by sediment distribution, and observed surface response is established.

Chapter Five contains a detailed site response study of a paired “near-rock” and soil sites in the basin based on observed ground motion data. This study yielded an appropriate procedure to properly parameterize the soil column by placing the model halfspace within the sediment column through an iterative process.

In Chapter Six, the site parameterization developed in Chapter Five and its applicability to the entire basin are validated through Monte-Carlo simulation. The Las Vegas basin is separated into two site response units.
Chapter Seven contains a ground motion projection study for the two site response units. Surface response envelopes were developed by considering credible active faults within 150 km of Las Vegas according to the USGS Quaternary Fault Database, employing appropriate attenuation relationships and taking into consideration local sediment effects.

Chapter Eight presents focused discussions about the impact of thin, stiff layers on surface response, and the adequacy of determining site classes based solely on $V_s$ averaged over the upper 30 m, regardless of the thickness of the shallow sediments.

Chapter Nine summarizes the answers to the research questions and presents recommendations for future research.
CHAPTER 2

BACKGROUND AND OVERVIEW

This chapter begins with a brief review of the geometry, geological setting, and lithology of shallow sediments of the Las Vegas basin. Then, ground motions recorded during historical high-energy explosive events and earthquakes are examined. The pertinent literature on effect of local sediments on ground motions, seismic response analyses of deep soil deposits, seismic hazard analysis, and seismic microzonation is reviewed. The state of knowledge about the seismic response in Las Vegas basin is discussed. Finally, the methodology used in this research is presented.

2.1 Geometry, Geological Setting, and Lithology of Shallow Sediments of the Las Vegas Basin

The Las Vegas metropolitan area currently houses approximately 1.7 million people (http://quickfacts.census.gov/qfd/states, as of March 2006). It maintains a rapid growth pattern that has been ongoing for decades. It is located in the Las Vegas Valley, a northwest-southeast trending alluvial basin formed by extensional tectonics (e.g., Wernicke et al., 1988). The basin is about 30 km across from east to west. The ground surface inclines downward from the Spring Mountains on the west to a basin-bounding high-angle normal fault on the east, at the foot of Frenchman Mountain. Langenheim et al.
(2001) studied basin geometry by gravity and seismic reflection methods. They reported that the maximum depth to bedrock approaches 5 km. The shallow part of the basin is filled with Quaternary alluvial-fan deposits derived primarily from the Spring Mountains, underlain by older Oligocene and Miocene deposits. The fan deposits are composed of clays, silts, sands, gravels and erratically-occurring carbonate-cemented lenses.

Sediments generally become finer toward the east and south (Wyman et al., 1993). Taylor et al. (2004) explained the sediment distribution within a basin formed and bounded by a predominant normal fault as follows: As the basin develops, the fine-grained sediments, such as river overbank (flood), lake and swamp deposits, migrate toward and become trapped by the steep, basin-bounding fault. The part of the basin near the fault on the down-dropped side evolves to house the thickest sediment deposits. By studying well logs across the Las Vegas basin, Taylor et al. (2004) have verified that the deep part of the basin (central and south) are dominated by clay-rich deposits, and the shallow part of the basin (west) are dominated by coarse- and mixed-grain size deposits. Depth to shallow groundwater varies across the basin from less than 10 m in the middle of the basin to more than 20 m toward the basin edges. Below this shallow system, the main confined aquifer for the basin appears. The bottom of the main aquifer varies from about 70 to 100 m below ground surface (Zikmund, 1996).

2.2 Observed Ground Motions

Historical ('legacy') ground motions recorded in and around the Las Vegas basin during underground nuclear testing conducted at the Nevada Test Site (NTS), located approximately 150 km from Las Vegas, show large geographical variation. Motions
recorded at the "near-rock" sites on the east and west edges of the basin are significantly smaller in amplitude than those recorded within the basin (Fig. 2.1). Rodgers and McCallen (2002) and Rodgers et al. (2004) demonstrated that the observed ground motion amplifications in the Las Vegas valley exhibit spatial correlations with basin depth; i.e., large amplifications are observed in the deep part of the basin and small amplifications are observed in the shallow parts of the basin. In this study, fifty-six legacy ground motions across the Las Vegas valley from four nuclear events, namely Barnwell (BA), Bodie (BO), Cottage (CO) and Gascon (GA), are examined (Table 1). These four datasets are selected because they had the best signal to noise ratios, as recommended by the research team's lead seismologist (Arthur Rodgers, Lawrence Livermore National Laboratory, personal communication). To facilitate examinations of general trends of observed site amplifications with respect to basin depths, basin sites have been grouped into shallow (less than 0.6 km), intermediate (0.6 - 2 km) and deep (2 - 5 km) according to the basin depth map of Langenheim et al. (2001) (Fig. 2.1). The depth groups have been contoured by the Jenks optimization method, coded in the program ArcGIS, which minimizes the squared deviations of the class means. The legacy recording network includes two near-rock sites: CALB and SGS, on the western and eastern edges of the basin respectively. Source-to-site distances for all four events are summarized in Fig. 2.2. The near-rock site CALB is about 20 km (~ 15 %) closer to the source than is SGS. With respect to source-to-site distances, the two near-rock sites bracket most of the other legacy ground motion sites in the basin.

The acceleration response spectra ($S_a$) of the four nuclear events, calculated using 5% damping, are grouped according to basin depth in Fig. 2.3. Considering the available
data (not all data are available for every event), several patterns appear. The near-rock motion at SGS is consistently smaller than at CALB. Considering the differences in source-to-site distance, this likely reflects attenuation due to geometric spreading. Spectral ordinates for most of the other legacy sites, particularly those in the intermediate-depth category, are much larger than the CALB motion, despite the greater source-to-site distance. Comparing the shallow sites to the intermediate-depth sites, with few exceptions, there is a clear tendency for increasing site response with increasing basin depth.

Only two legacy ground motion sites, S51 and S16, were located in the deepest part of the basin, and recorded data for those sites are sparse for the four events studied. The few available deep-basin datasets do not show an increase in amplification with respect to most of the intermediate-depth basin datasets.

Figure 2.4 contains response spectra normalized by peak ground acceleration (PGA) and then averaged within site groups. In general, the predominant period increases with increasing basin depth. At periods longer than 0.4 sec, spectral ordinates are much higher for the sites located within the basin than for the near-rock sites; however, the opposite is true for shorter periods. In the period range 0.4 to 1.2 sec, amplifications are consistently higher for the deep and intermediate-depth sites than for the shallow basin sites. For longer periods, the shallow, intermediate, and deep basin sites have about the same spectral acceleration, and both site categories have higher surface responses than that of the near-rock sites. Compared to the average normalized response spectra for four generic site categories constructed by Seed and Idriss (1982), the near-rock and shallow
sites behave like "rock" sites and the deep and intermediate-depth sites behave like "stiff and deep cohesionless soils."

In addition to the legacy motion records, on June 29, 1992, the Little Skull Mountain (LSM) earthquake ($M_l = 5.6-5.8$), which was felt throughout the Las Vegas basin, triggered 10 strong motion stations. Nine of the records are usable (Table 2.1). The earthquake occurred at lat 36.72° N, long 116.30° W, approximately 120 km from Las Vegas and within the boundaries of the NTS. The focus depth was 11.8 km (Su et al., 1998).

The acceleration response spectra ($S_a$) of the nine available strong motion records, calculated using 5% damping, are grouped according to basin depth in Fig. 2.5. Similar to the ground motions recorded during underground nuclear testing, considering straight line source-to-site distance, the CALB site is about 30 km (~ 25 %) closer to the source than is SGS. Not surprisingly, then, the near-rock motion recorded at CALB is much larger than that recorded at SGS. The near-rock site CALB and shallow site ANN have the same source-to-site distance and similar azimuth angle; ground motion recorded at ANN has the same peak $S_a$ as that of the CALB motion. The SGS motion is consistently smaller than motions recorded in all the other sites. The largest ground motions were recorded at the deep basin sites.

For the LSM earthquake event, the peak $S_a$ recorded at the deep basin sites is as much as three times greater than that recorded on the near-rock site SGS. For the underground nuclear test, the maximum amplification factor with respect to near-rock site, for peak $S_a$, is about four. Considering $PGA$, for both cases, the amplification factors with respect to near-rock site varied and were generally smaller than those for the peak $S_a$. 

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2.3 Effect of Local Sediments on Ground Motions

The influence of local sediment conditions on the intensity of surface motions has been recognized for many years (Kramer, 1996). One of the first attempts to quantify such effects was reported on the 1906 San Francisco earthquake (Kramer, 1996), where the intensity of ground shaking was related to local soil and geologic conditions. A most dramatic example of the effect of local sediment on surface shaking was observed in Mexico City during the 1985 Michoacan earthquake (M_s=8.1). Mexico City is located about 400 km away from the epicentral area. With respect to the reference rock sites, the PGA measured at the soil sites was amplified by as much as a factor of four. The fundamental periods, namely, the periods corresponding to the first modes of vibration, of the deep, soft lakebed deposits underlying parts of Mexico City, were about 1.9 to 2.8 seconds, which closely matched the predominant period (T_p), namely, the period corresponding to the peak S_a with 5% damping, of the bedrock ground motion. Unfortunately, the fundamental periods of many structures in that area fell within the same range. As a result of this double resonance, the city experienced devastating damage (Seed et al., 1988; Kramer, 1996).

An overview of site-specific amplification can be found in Kramer (1996). Ground motion can be significantly amplified or deamplified by the configuration and dynamic properties of the near-surface soil deposits through which the seismic energy propagates. Three major factors control the site-specific amplification effect in a soil column: impedance, damping, and resonance. For horizontally polarized shear waves, the impedance is the product of material density, shear wave velocity (V_S), and the cosine of the angle of incidence, which is the direction of seismic wave propagation with respect to...
vertical. As seismic waves approach the earth’s surface, the angle of incidence becomes very small and its cosine can be assumed to be equal to one. Because variation in density of geologic media is relatively small, the impedance of a soil layer mainly depends on its $V_S$. As a seismic wave travels from depth to the ground surface, it passes through soil strata with ever-decreasing $V_S$ and, hence, impedance. Ideally, the particle velocity will increase to satisfy the law of conservation of energy. However, because geologic media are not purely elastic, material damping will mitigate the amplification effect to some degree. Material damping is greater on soft soil than on rock, and greater at high frequencies than low frequencies (e.g., Reiter, 1990). These two contradictory elements combine uniquely in each soil column to produce motions at the surface that can be very different from the base motion (e.g., Liu et al., 2003).

Site-specific seismic response is often analyzed using the one-dimensional (1-D) equivalent-linear model SHAKE, which calculates the seismic response of a horizontally-layered soil system over a halfspace subjected to vertically-propagating, horizontally-polarized shear waves (Schnabel et al. 1972; Idriss and Sun 1992; Ordonez 2000). This process begins with estimation of the initial shear strain. The initial shear strain is calculated by $(M - 1)/10$, where $M$ is the magnitude of the seismic event. (The type of earthquake magnitude is not specified in the SHAKE manuals.) The program assigns user-specified strain-dependent shear moduli and damping ratios corresponding to the soil type and confining pressure normalized by the estimated initial shear strain. The shear strain is recalculated and compared against the initial guess. An iterative procedure is used to modify strains to arrive at shear moduli and damping ratios compatible with the calculated strains. The acceleration resulting from this procedure is the modeled output.
(surface) acceleration. This method is computationally efficient and has been validated by many back analyses (Kramer, 1996).

An alternative approach is to use a fully nonlinear model, which involves direct numerical integration in the time domain. A well-known program for this type of analysis is DESRA-2 (Lee and Finn, 1978). Siddharthan (2004) has shown that with respect to DESRA, SHAKE yields consistently conservative results. Because uncertainties in site parameterization overshadow benefits of incremental increases in accuracy in dynamic modeling, particularly for the low-strain events studied in this research, this study chose to use the equivalent linear model but put more effort on proper site parameterization. Adequacy of the equivalent linear model was tested on a paired rock and soil sites and discussed in detail in Chapter 5.

2.4 Seismic Response Analyses of Deep Soil Deposits

Deep soil deposits (defined here to be greater than 100 m) are usually formed in basin settings. Because many large cities are situated over deep alluvial basins, many researchers have studied the effects of basin geometry and soil layering on ground motions (Kramer, 1996). A basin can trap body waves and generate surface waves, thus producing stronger ground motion and longer duration than would be predicted by 1-D analyses. Typically, the 1-D analyses can adequately predict the averaged response near the center of the basin but not at the basin edges (Kramer, 1996).

To investigate basin effects, researchers have performed two-dimensional (2-D) and three-dimensional (3-D) seismic response analyses. Bakir et al. (2002) reported basin edge effects in the 1995 Turkey earthquake where damage to buildings located near the
east edge of an alluvial basin in Southeast Anatolia, Turkey, was severe. The maximum
depth of the basin is about 100 m. They applied both 1-D (SHAKE) and 2-D finite
element analyses. The authors concluded that the 1-D analysis considerably
underestimated the surface response for sites close to the basin edge. As the distance
from basin edge increases, the discrepancy between 1-D and 2-D projections diminishes.
This research supports Kramer’s (1996) claim discussed previously.

Sánchez-Sesma et al. (1988) studied the seismic response of the valley of Mexico
City during the 1985 Michoacan earthquake using a 1-D equivalent linear model and a
simple 2-D triangular basin model. The authors concluded that 1-D phenomena
dominated in the uppermost clay layer. The 2-D basin effect contributed to larger
amplification, longer duration and lateral variability of the observed ground motions. The
author stated that the observed ground motion could be better matched if both 1-D and 2-
D effects were combined. However, no approach was given to combine the two analyses.

Semblat et al. (2004) investigated the effect of basin geometry and soil layering at
sites in Volvi, Greece having a maximum depth to bedrock of 200 m, using the boundary
element method. Their findings supported conclusions of Sánchez-Sesma et al. (1988) in
that with respect to the surface motions predicted by the 1-D analysis, the 2-D effects
contribute to larger amplification as well as longer duration.

Wald and Graves (1998) used a 3-D finite-difference numerical modeling
approach to study the seismic response of the Los Angeles basin during the 1992 Landers
earthquake. The authors tested three well-known geologically-based velocity-structure
models for the Los Angeles basin. They observed that the simulation outcome was highly
sensitive to the velocity-structure model of the basin, which presents real problems for
such analyses because deep-basin structure models have high uncertainty. Interestingly, the basin model having similar background velocity to that used in 1-D modeling by other researchers yielded best projections. The authors also examined the amplification caused by considering 1-D site response, and, similar to Sánchez-Sesma et al. (1998) and Semblat et al. (2004), concluded that 3-D basin effects augmented amplifications and duration of ground motion.

In conclusion, all four researchers concluded that basin effects account for larger amplification and longer duration. One researcher found that the discrepancy between 1-D and 2-D results was the largest at basin edges and diminished toward the center of the basin, whereas the other three researchers did not report such a geographically-consistent trend.

Despite the above-mentioned multi-dimensional basin effects, in recent years, a number of studies have been conducted to investigate seismic response of deep sites, using 1-D modeling only, and obtained satisfactory results. The challenge of performing meaningful 1-D analyses for deep soil deposits is appropriate parameterization of strain-dependent modulus reduction and damping functions, $V_s$, and depth to model halfspace. Different researchers have handled this challenge differently. Three approaches have been practiced with regard to placing depth to model halfspace: 1) placing the halfspace at the actual soil-bedrock interface, based on the investigator's best understanding of the study site, 2) placing the halfspace using $V_s$-based criteria, and 3) placing the model halfspace within the soil column using an iterative procedure to match certain properties of the projected motion with the measured or expected motion. Chang (1996) used SHAKE91 to study seismic response for three sites in the Los Angeles basin for the 1994
Northridge Earthquake. The maximum depth to bedrock was about 200 m. The halfspace was placed at the soil-bedrock interface. The author acknowledged the uncertainties involved for locating the soil-bedrock interface because of the lack of boring logs and the disagreement over how to discriminate between hard soil and soft rock. The author used all available information such as published literature and discussion with other researchers to define the soil-bedrock interface, yet pointed out that still more precise and reliable estimation of the depth to bedrock was desired. Material-specific damping ratios that had not been adjusted for confining pressure were used. Still, the author concluded that the 1-D equivalent-linear model was able to capture much of the observed amplification.

Wong and Silva (1993) used stochastic numerical ground motion modeling and 1-D equivalent-linear site response analyses to study earthquake ground motions for sites in the Salt Lake Valley, Utah having maximum depth to bedrock approaching 600 m. They pointed out that proper representation of strain-dependent soil damping is particularly important for such deep sites. Unfortunately, no measured ground motion records were available to verify the 1-D model projections. Their 1-D equivalent-linear model has also been incorporated in an earthquake scenario and probabilistic ground shaking study for the same area (Wong et al., 2002).

Hashash and Park (2001) used a non-linear 1-D model to study seismic ground motion propagation for sites in the Mississippi embayment having maximum depth to bedrock approaching 1 km. The authors used confining-pressure dependent dynamic soil properties recommended by Laird and Stokoe (1993) (Hashash and Park, 2001). They claimed that to perform site response analysis in deep soil deposits, the entire depth of the
soil column should be considered; the use of an “arbitrary” cutoff depth results in erroneous predominant period. For the same embayment, Park and Hashash (2004) used the 1-D equivalent-linear method to back-calculate depth-dependent small-strain damping properties. However, their study involved several significant uncertainties. First, they had measured $V_S$ data only to 70 m. From 70 to 1000 m, they used a generic regional profile, and the bedrock $V_S$ was assumed to be 3000 m/sec. For the input ground motion, lacking measured outcrop motion in the study area, they generated a synthetic motion using a point-source stochastic model. Using this input motion, the authors adjusted the damping function to match the projected surface motion with measurements. It is this writer’s conclusion that because any changes of either the $V_S$ profile or the input motion can yield a significantly different projected motion, the back-calculated damping function is not likely to be reliable.

Ni et al. (1997) performed 1-D nonlinear seismic response analyses of hypothetical deep, saturated soil columns. Comparing 100- and 200-m deep soil columns, they confirmed that soil-column depth strongly affects frequency response. They also observed that the use of dynamic soil properties unadjusted to compensate for confining pressure is unconservative for deep deposits.

The same 1-D equivalent-linear model RASCALS used by Wong et al. (1993), which was developed by Silva using random vibration theory, was adopted by Romero and Rix (2001) for a seismic response study for the Upper Mississippi Embayment. For $V_S$ profiles, the authors compiled existing $V_S$ measurements (up to 70 m) and a combined a regional crustal model (to 1000 m) to form characteristic $V_S$ profiles for different parts of the Embayment. The $V_S$ at the halfspace was set to be 2100 m/s based on published
literature. The authors used dynamic soil properties recommended by EPRI (1993). They compared their response spectra with those projected using criteria advocated by the National Earthquake Hazards Reduction Program (NEHRP; Building Safety Council 2000). They observed that the agreement between the two deteriorates as site stiffness decreases. For soft sites, with respect to the investigators’ 1-D analysis, the NEHRP projection significantly underestimated the spectral shape at longer periods.

All the researchers mentioned above placed the halfspace at the soil-bedrock interface, based on their best understanding about the geologic setting of the study area and the dynamic properties of the sediments. The drawback of this approach is that for most deep soil deposits, the true soil-bedrock interfaces are not well-defined and the knowledge of confining-pressure-adjusted dynamic soil properties and $V_S$ at great depths is limited. The highest known strain level to which dynamic soil properties have been measured in the laboratory is 1 percent (Stokoe and Darendeli, 2001). The maximum pressure reported in this dataset corresponds to a maximum depth of 200 m, assuming that groundwater table is shallow and the total density of soil is 1830 kg/m$^3$.

One alternative to cope with the uncertainties is to apply the so-called “engineering bedrock” criterion, which is based on the claim that sediments overlying true bedrock can be so stiff as to behave like bedrock for purposes of site response analysis (Jonathan Bray, University of California, Berkeley; personal communication). As embodied in the current code-based practice, such as the International Building Code (2003, Chapter 16, Table 1615.1.1), engineering bedrock has a threshold $V_S$ of 760 m/s.

To address the uncertainties associated with locating the true soil-bedrock interface as well as assigning dynamic soil properties and $V_S$ for deep soil deposits, Luke
et al. (2001) recommended a practical alternative: In their study of a sandy soil deposit on the NTS, with depth to bedrock estimated to be more than 300 m, they obtained satisfactory surface projections by adjusting model halfspace depth to match the $PGA$ of the projected motions with the expected values. The authors found that optimum halfspace depths fell within the soil column, well above the estimated true bedrock surface, and well below the depth corresponding to the engineering bedrock criterion.

2.5 Study of Soil Nonlinearity and Frequency-Dependent Damping

Nonlinearity refers to the complex material deformation resulting from cyclic loading. Soils respond nonlinearly to strong earthquake motions; this response has a profound effect on ground shaking. In a linear system, the material response scales proportionally to the dynamic load and therefore can be reliably predicted. However, nonlinear soil response remains difficult to predict. Much of the reason is that in situ direct observations of nonlinear soil response are not available to test the existing physical models. Compared to the equivalent-linear model, nonlinear models also involve more difficulties and uncertainties in model parameterization, which, in the writer's opinion, could overshadow the benefits it can provide. Another complex issue with site response analysis that is not yet fully understood is the effect of frequency-dependent damping on ground shaking. These two issues, namely, soil nonlinearity and frequency-dependent damping, have been investigated by several researchers.

Hartzell et al. (2004) studied the nonlinear soil effect by comparing equivalent-linear and nonlinear soil models with and without frequency-dependent shear moduli and damping, for a wide range of input ground motions and site conditions. Their study
showed that for stiff sites and low levels of input motions, differences in ground motion projections between the equivalent-linear model and the nonlinear model are small. For soft site conditions, compared with the nonlinear solution, the equivalent-linear solution without frequency-dependent damping tends to over-damp higher frequency response (> 4 Hz), and the equivalent-linear solution with frequency-dependent damping tends to under-damp the ground motion. The frequency range over which the ground motion was under-damped was not specified. The authors recommended a nonlinear approach for NEHRP (Building Safety Council 2000) site classes D and E, and for site class C having input motions greater than a few tenths of acceleration of free-fall (g).

Kausel and Assimaki (2002) compared a series of “true” nonlinear numerical solutions with equivalent-linear solutions that incorporated frequency-dependent moduli and damping. Their result indicated that it is possible to simulate closely the non-linear soil response by means of equivalent-linear analyses with frequency-dependent damping. They also studied the differences between equivalent-linear solutions with and without frequency-dependent damping, and concluded that for deep, soft soil deposits, the frequency independent solution tend to over-damp high frequency response (> 3 Hz).

Hashash and Park (2002) studied the effect of incorporation of frequency-dependent damping on nonlinear site response. The authors implemented the full form of frequency dependent damping to represent viscous damping in a time-domain nonlinear site response analysis. They conducted a comparative study and concluded that the implementation of frequency dependent damping addressed the long-standing problem that non-linear site response analysis tends to underestimate high frequency response. The new frequency-dependent damping formulation they proposed suggested that in
addition to the first mode of the soil column, the higher modes also make important
ccontributions to the viscous damping component.

In conclusion, the aforementioned researchers found that for stiff soil deposits
and/or low levels of ground motions, the soils tend to behave linearly, and the ground
motion projections of equivalent linear and nonlinear models generally agree with each
other. For soft soil deposits and/or high levels of ground motion, the site tends to behave
nonlinearly. Compared to a nonlinear model, the equivalent-linear model tends to over­
damp high frequency response. For high frequency response, the models with frequency­
dependent damping tend to overdamp site response compared with those with
frequency dependent damping, for both linear and nonlinear approaches. Incorporation of
frequency-dependent damping tends to yield greater high frequency response for both
linear and nonlinear analyses.

For small-strain shaking and reasonably stiff sites, the site response analysis result
of this work (chapter 5) will show that the ground motion projections of a 1-D
equivalent-linear model, using frequency-independent damping, compare favorably with
measured ground motions, provided that the soil profile is properly parameterized
(including $V_s$, depth-dependent damping and shear modulus, and depth to model
halfspace). A benchmark comparison study conducted by Siddharthan (2004) indicated
that the ground motion projections from the well-known 1-D equivalent-linear model
SHAKE and 1-D nonlinear model DESRA both compared favorably to measured ground
motions. At high frequencies, for relatively stiff sites and small-strain shaking, the
SHAKE model yielded slightly conservative projections.
2.6 Seismic Hazard Analysis

Estimation of ground motion parameters such as peak ground acceleration (PGA), peak ground velocity (PGV) etc., is often accomplished by conducting a seismic hazard analysis. There exist two types of seismic hazard analysis (SHA): deterministic (DSHA) and probabilistic (PSHA). Detailed description of the two methods can be found, e.g., in Kramer (1996). The DSHA involves the development of a particular seismic scenario, independent of return period. The advantage of the DSHA is that it provides worst-case scenarios efficiently (Kemnitz, 1999). For the PSHA, the ground motion parameters are described by their probability of exceedance for a given return period. The PSHA often uses the well-known Gutenberg-Richter recurrence law to define the mean annual rate of exceedance of a particular magnitude event for a certain seismic source (Kramer, 1996). The Gutenberg-Richter coefficients are obtained by regression on datasets of paleoseismic evidence and historic seismic events from the source zone of interest. According to Kramer (1996), “unless the source zone is extremely active, the database is likely to be relatively sparse.” The advantage of the PSHA is that it allows uncertainties in earthquake magnitude, location, and rate of recurrence to be factored into the analysis.

2.7 Seismic Microzonation

As shown in Fig. 2.1, the Las Vegas basin is capable of amplifying ground motions to a great extent. As mentioned in Chapter One, given the large population, rapid growth rate, unique building inventory, and the continuing disclosure of significant young faults, the seismic risk in the Las Vegas basin is high. Hess and dePolo (2005) conducted an earthquake loss estimation study for the Clark County, Las Vegas area,
using FEMA’s loss estimation model HAZUS-MH. For a magnitude 6.6 earthquake on the Frenchman mountain fault, the HAZUS-MH estimated the following: 1) 200 to 800 fatalities, 2) 700 to 3,000 people needing hospital care, 3) 11,000 people needing public shelter, 4) 14,000 to 60,000 buildings suffering major damage, and 5) $4.4 to 17.7 billion economic loss.

Seismic microzonation maps, also referred to as seismic hazard maps, are effective tools for land use planning and earthquake hazard mitigation. Seismic microzonation maps may address one or more seismic hazards. They are compiled from geological, geotechnical and geophysical data and they reflect local ground conditions. The primary seismic hazards can be grouped into six categories for mapping purposes: amplification of ground motion, landslides, liquefaction, tsunamis and seiches, tectonic subsidence or uplift, and ground rupture (Klohn-Crippen Consultants Ltd., 1994).

Marcellini et al. (2001) discussed regional and local seismic hazard assessment. The authors pointed out that regional seismic hazard assessment usually addresses local sediment effects only in a limited way, using different attenuation laws for ‘soft soil’ and ‘rock’. But for microzonation, the local sediment conditions must be taken into consideration explicitly.

Seismic microzonation maps can be based on observed ground motions, predominant period from microtremor measurements, damage of buildings, and amplification factors. The most comprehensive microzonation studies include source characterization, site-response unit characterization, calculation of amplification factors, application of attenuation relationships and projection of surface ground motion. Following are examples of microzonation studies based on these different criteria.
2.7.1 Observed ground motions

Murphy and Hewlett (1975) performed a preliminary microzonation for the Las Vegas Valley based on observed ground motions. They studied ground motions recorded at 26 different locations in Las Vegas from six underground nuclear events. They selected a reference station which has relatively low ground motion amplitude and computed the Fourier spectral ratios for all the other stations. The microzonation results were presented in the form of contour maps in 12 different period bands ranging from 0.16 to 6.0 sec. Their work showed that most parts of the basin amplify ground motions by a factor of two over the frequency range 0.2 to 1 Hz. Because their reference site was located well within the alluvial basin, with respect to true rock motion, their amplification factors would have been underestimated.

2.7.2 Predominant period from microtremor (H/V) ratio

Tuladhar et al. (2004) constructed a seismic microzonation map for the greater Bangkok area, Thailand, using microtremor observations. They monitored microtremors at more than 150 sites and calculated the predominant periods of each site by computing the Fourier spectral ratio of horizontal- to vertical-component ground motion. At selected sites where detailed $V_S$ profiles were available, the predominant period obtained from microtremor measurements was validated by computing the transfer function between halfspace motion and surface motion using SHAKE91 (Idriss and Sun, 1992). The final results were presented in a contour map of predominant period.

2.7.3 Both observed ground motion and microtremors

Chávez-García and Cuenca (1998) conducted earthquake microzonation for Acapulco, Mexico, using measured strong motion records at 9 locations, weak motion
records at 6 locations and microtremor records at 35 locations. The relative amplification factor was determined from strong and weak motion records as the Fourier spectral ratio with respect to a reference rock site. The site predominant periods were determined by the horizontal-to-vertical spectral ratio of the microtremor observations. As a result, two contour maps, predominant period and relative amplification, were produced.

2.7.4 Comprehensive considerations

In their seismic response study for the Salt Lake City, Utah, metropolitan area, Wong et al. (2002) produced earthquake scenario and probabilistic ground shaking maps. The procedures they followed are listed below:

1) Seismic source characterization: The authors pointed out that seismic source characterization is concerned with the following elements: a) the location and geometry of significant potential seismic sources; b) the earthquake magnitude distribution for each source; and c) the recurrence rates of different magnitudes for each source. For the earthquake scenario study, no recurrence rate information was used. For the probabilistic earthquake ground motion study, all seismic sources, both discrete and areal source zones, capable of generating significant ground shaking at the study site (usually within 100 to 200 km in the western U.S.) should be characterized. For the probabilistic seismic hazard analysis, the authors adopted a logic tree approach. To characterize the seismic sources, they reviewed fault information from numerous recent studies and contacted numerous geoscientists regarding their unpublished and ongoing work in the region. For each fault, the rupture model, maximum rupture length, maximum magnitude, dip, approximate age of youngest offset, probability of activity and rate of activity were characterized.

2) Geologic site-response unit characterization: The authors defined five distinct
site-response units based on sediment type: silt and clay, sand, and three types of gravel. A representative near-surface $V_s$ was assigned to each response unit based on measured data and values inferred from published literature.

3) Amplification factor calculation: Amplification factors were calculated for each response unit using the equivalent-linear approach embodied in Silva's code RASCAL. A stochastic numerical model was used to generate input motions for a M 6.5 earthquake. Each site response unit was assigned a unique amplification factor.

4) Attenuation characterization: Due to the lack of historical strong-motion records, no attenuation relationships were available for the Salt Lake Valley or the Basin and Range province. So empirical attenuation relationships appropriate for shallow earthquakes in the western U.S. were used.

5) Ground motion calculations: Both scenario ground motions (due to a $M_w$ 7.0 scenario earthquake on the Salt Lake City segment of the Wasatch fault) and probabilistic ground motions (at return periods of 500 and 2,500 years) were calculated using RASCAL to assess horizontal acceleration and spectral acceleration at periods of 0.2 and 1.0 sec.

6) Map development: Ground shaking maps were developed using a vector- and raster-based GIS. The map was gridded at 200 by 200 m. A site-response unit was assigned to each grid point according to lithology. The surface ground motions were calculated by multiplying the scenario or probabilistic ground motions for rock by the appropriate amplification factors.

In a preliminary seismic microzonation assessment for British Columbia (Klohn-Crippen Consultants Ltd., 1994), investigators presented ground motion amplification in
a series of three maps. The so-called “Level 1” map shows the susceptibility to amplification by compiling geologic and geotechnical data and grouping the sites into response units. The sites were classified into 8 units based on general description of soils and averaged \( V_s \). The susceptibility to amplification was divided into low, moderate and high. Building on the Level 1 map, the Level 2 map shows the surface motion for each soil unit. The surface motion was computed by scaling the input bedrock-level motion by empirical amplification factors. The bedrock-level motion was determined using a probabilistic approach. The Level 3 map incorporated SHAKE analyses at sites where \( V_s \) profiles were available. The principal difference between Level II and Level III maps is that Level II maps use global empirical predictions of amplification whereas Level III uses site-specific analyses. With respect to the Level II map, the Level III map gives more accurate ground motion projections, but only for isolated areas.

Torregosa et al. (2002) performed a comprehensive study about seismic hazard assessment and microzoning in the Philippines. The authors examined about 6000 historical earthquake datasets recorded since 1907 and incorporated 59 active fault segments to model the seismogenic source zones. The seismogenic source zones were assessed by a probabilistic approach. An attenuation formula was used in this study to generate input “rock motion.” The amplification factor was computed from S-coda waves. The S-coda wave is the concluding portion of the time history after the identifiable shear waves have passed. For the microzonation, the surface geology was divided into 6 response units based on geologic ages. For each response unit, a soil softness index based on the standard penetration test was calculated; this in turn was correlated with the amplification factor. Three earthquake scenarios, having annual exceedance probabilities
of 0.01, 0.002 and 0.001, were considered. The ground acceleration was obtained by the same approach adopted by Wong et al. (2002).

2.7.5 Building vulnerability

In their work to study the earthquake scenarios for the city of Basel, Switzerland, Fäh et al. (2001) assessed the vulnerability of buildings and produced a contour map showing overall building damage. Two earthquake scenarios were considered: an event with a modified Mercalli intensity between VII and VIII and a return period of 475 years and an event that simulates the 1356 Basel earthquake, intensity IX.

2.7.6 Amplification Factors

After the 1997 Umbria Marche earthquake, an extensive microzonation study was conducted for 60 villages in the Umbria-Marche Apennines, central Italy (Marzorati et al., 2003). The study began by selecting the villages that showed the most damage from the 1997 Umbria Marche earthquake. Then, a field campaign was carried out to characterize in detail the geological and geomorphologic features of those areas. The site amplification was calculated through 1-D and 2-D soil response modeling. The 2-D modeling was performed using finite and boundary element methods. The input motion was defined as the uniform probability spectrum having a return period of 475 years. The amplification factor for each site was calculated by computing the ratio of the spectral intensities of surface motion to input motion.

2.8 State of Knowledge about the Seismic Response in Las Vegas Basin

It has long been noted that the seismic response in the Las Vegas basin is quite variable. As mentioned previously, the first effort to construct microzonation maps for
Las Vegas was made by Murphy and Hewlett in 1975. In 1998, Su et al. studied the S-wave site amplification in the Las Vegas basin using a regional layered crustal model. They used data recorded in the Las Vegas basin during the 1992 LSM earthquake ($M_L=5.6$-$5.8$). They reported a maximum amplification factor of 5 over the frequency range 0.5 to 2 Hz, with respect to a representative near-rock motion. They calculated the near-rock motion as the average of the responses recorded at CALB and SGS (Fig. 2.1), which are the near-rock sites on the east and west edges, respectively, of the basin, described previously. Recently, McCallen et al. (2003) studied historical ground motion datasets from both nuclear explosions and the Little Skull Mountain earthquake. Their study showed that the band-averaged amplification could approach a factor of ten for frequencies between 0.2 and 2 Hz in some locations. They found that the spatial pattern of site-specific amplification correlates strongly with basin depth, with amplification increasing with increasing basin depth. Rodgers et al. (2006) incorporated a 2-D model to study the site response in the Las Vegas Valley. By changing the shallow $V_S$ (within 200 m), they were able to reproduce some of the observed amplifications. However, no detailed shallow geotechnical and geophysical structure were included in their study. There has been no systematic study about ground motion projections taking into consideration the near-surface sediment effect on surface response, in a detailed manner, for the Las Vegas basin, as has been performed for other deep basins described previously (Wong and Silva, 1993; Wong et al., 2002; Romero and Rix (2001)).
2.9 Methodology

This research was initiated out of concerns over the potential for structural damages in the Las Vegas valley induced by high-energy nuclear test explosions (McCallen et al., 2003). The original research involved multi-institutional collaboration among Lawrence Livermore National Laboratory (LLNL), University of California, Berkeley, University of Nevada, Reno (UNR), and University of Nevada, Las Vegas (UNLV). The 2-D seismic response of the Las Vegas basin was investigated by seismologists at LLNL (Rodgers et al., 2006). The contribution of 1-D seismic response to observed ground motion amplification was investigated by the writer, Prof. Luke, many student research assistants in the department of civil engineering at UNLV, and Prof. Siddharthan at UNR. The analysis results showed that much of the ground motion amplification observed during underground nuclear tests could be modeled by the 1-D equivalent linear approach. This result supported the conclusions made by other researchers who had used 1-D equivalent linear approach to study seismic response of deep soil deposits, as discussed previously. Detailed analyses and discussions are presented in Chapter 5, where a benchmark site response study was performed for a pair of near-rock and soil sites.

Considering the foregoing discussion, this study will use the 1-D model to study seismic response in the Las Vegas basin. The challenge for properly employing the 1-D equivalent-linear model is appropriate parameterizing of the soil column. This study will refine the approach recommended by Luke et al. (2001), which places the model halfspace within the soil column and obtains optimum halfspace by an iterative procedure.
The objective of this research is to project bounding response spectra for the Las Vegas basin, taking into account the local site conditions. To obtain realistic time histories for input rock motions, all known active faults that have significant earthquake potential need to be considered. Compared to earthquake magnitudes resulting from PSHA analysis, the DSHA projects upper-bound values. In this research, a multiple DSHA approach was selected to project upper bound-response spectra for different soil units in the Las Vegas basin.

For the purpose of seismic microzonation, it is good practice to generate envelopes of credible seismic response (Luke et al., 2001). This is accomplished through Monte-Carlo simulation, which uses random theory to sample statistical parameterizations of system variables. Thus, the spatial variations within the same response unit and the uncertainties associated with $V_s$ profiles are taken into consideration. The techniques and procedures elaborated in this study are applicable to any seismic response analyses for deep soil deposits.
Table 2.1  Legacy ground motions for four nuclear events and the LSM earthquake ground motions recorded in the Las Vegas basin, sites grouped by basin depth. Blank cell indicates missing data.

<table>
<thead>
<tr>
<th>Event</th>
<th>Near-rock</th>
<th>Shallow</th>
<th>Intermediate-depth</th>
<th>Deep</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CALB</td>
<td>SGS</td>
<td>ANNN</td>
<td></td>
</tr>
<tr>
<td>Barnwell</td>
<td>X X X</td>
<td>X X</td>
<td>X X</td>
<td>X X</td>
</tr>
<tr>
<td>Bodie</td>
<td>X X</td>
<td>X X X X</td>
<td>X X X X X X</td>
<td>X X</td>
</tr>
<tr>
<td>Cottage</td>
<td>X X</td>
<td>X</td>
<td>X X X X X X X X X X</td>
<td>X X</td>
</tr>
<tr>
<td>Gascon</td>
<td>X X X X</td>
<td>X X X X</td>
<td>X X X X X X X X X X</td>
<td>X X</td>
</tr>
<tr>
<td>LSM Earthquake</td>
<td>X X X</td>
<td></td>
<td>X</td>
<td>X X</td>
</tr>
</tbody>
</table>

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Figure 2.1  Shaded relief map of Las Vegas basin, with legacy recording sites, selected ground motions (BA event, 0.2-1 Hz, 200 sec duration) illustrating variable amplifications, and depth-to-bedrock zones.
Figure 2.2 Source-to-site distances for selected legacy sites during four nuclear events, highlighting near-rock sites CALB (blue filled circles) and SGS (red squares)
Figure 2.3 Summary of acceleration response spectra from four events; sites grouped by basin depth.
Figure 2.4 Averages of normalized response spectra (5% damping) for different site groups for underground nuclear tests.

Figure 2.5 Summary of acceleration response spectra from LSM earthquake; sites grouped by basin depth.
ACQUIRING SHALLOW $V_S$ PROFILES IN THE LAS VEGAS BASIN

As discussed in Chapter 2, the observed ground motions in the Las Vegas basin show great geographical variation. Because reliable $V_S$ profiles are key to understanding seismic response of basin sediments, one important component of this research is to investigate the shallow $V_S$ structure of the basin. This work was accomplished by 1) compiling existing $V_S$ data, 2) making new $V_S$ measurements, and 3) creating publicly-accessible archives. Most of the material presented in this chapter has been reported in a peer-reviewed conference paper (Liu et al., 2005).

The new $V_S$ dataset consists of twelve surface wave measurements conducted in the Las Vegas basin. The intermediate data processing steps and the final $V_S$ profiles are documented in Appendix 1. All of the $V_S$ profiles gathered to date have been posted to an on-line archive for the Las Vegas basin.

3.1 Archiving Existing $V_S$ Data

In recent years, researchers at the UNLV Engineering Geophysics Laboratory (EGL) and their collaborators at UNR have been characterizing shallow $V_S$ at various locations in the Las Vegas basin using various methods including non-intrusive surface-wave based methods, downhole and crosshole seismic methods. The EGL has also
compiled available shallow $V_p$ and $V_S$ datasets collected by local practitioners. Results to date were summarized in Fall 2002 (Luke et al., 2002). Among other things, they were used to construct a "background" $V_S$ profile for the valley, which is particularly useful for locations where credible data at depth are scarce or not available. The $V_S$ datasets were configured to a standard format and posted on the EGL's data archive webpage (http://www.ce.unlv.edu/egl/lv_archives/). The archive webpage includes a "readme.txt" file which explains the formatting for the data files. For sites where multiple measurements exist, a "preferred" profile is identified. As of March, 2005, nineteen sites representing 50 measurements across the valley, and a velocity transect including 20 stations along the Las Vegas Boulevard, were archived. Since the local adoption of the 2000 International Building Code (IBC 2000), the design base earthquake has been changed from 10% probability of being exceeded in 50 years, as required by UBC 1997, to 2% probability of being exceeded in 50 years. Therefore, it becomes more cost effective for developers to carefully classify the site class. The 30-meter averaged $V_S$ measurements ($V_{S30}$) have become beneficial for many new land development projects for the purpose of obtaining site class for earthquake-resistant design purposes in a fast and economical way. The EGL is continuing to work to incorporate those records in their archives, which would richly supplement the archives.

3.2 New $V_S$ Measurements

3.2.1 Resolution requirements for new testing sites

Although current code-based practices, as embodied in the National Earthquake Hazards Reduction Program (NEHRP; Building Safety Council 2000) and the
International Building Code 2003 (Chapter 16, Table 1615.1.1), use $V_{S(30)}$ as the criterion for seismic design site classification, recent studies of deep basin sites suggest that a deeper sediment profile should be considered to obtain a reasonable projection of surface motion (Wong and Silva 1993; Bodin and Horton 1999; Romero and Rix 2001; Park and Hashash 2004; Luke et al. 2001).

The shallow sediments in the Las Vegas basin contain carbonate-cemented lenses, a feature that is unique in the desert environment. These media are ubiquitous and could have $V_s$ higher than 2,000 m/s (Stone and Luke, 2001; Tecle et al., 2003). These heavily-cemented media can exist at various depths (Prof. Wanda Taylor, UNLV Geoscience, personal communication). Individual lenses are usually less than 3 m thick but can extend laterally over tens of meters (Stone and Luke, 2001). When occurring at shallow depths and bounded by less stiff materials such as uncemented clays, silts and sands, the cemented inclusion presents a tremendous impedance contrast, thus potentially changing the intensity of ground shaking under seismic loading. One component of this research is to investigate the impact of cemented inclusions on surface response. In order to study these effects, it is important to capture detailed velocity variations at shallower depths, as well as characterize velocities to greater depths than commonly explored in conventional geotechnical site investigation.

3.2.2 Methodology

To meet the requirements of preserving high resolution at shallow depths while also extending the $V_s$ measurement to greater depths, combined usage of active- and passive-source surface wave methods was adopted. Recently, two other research teams have explored the integrated use of active- and passive-source surface wave methods.
Suzuki and Hayashi (2003) used a 48-channel linear array for active-source measurements and two-dimensional triangular- and “L”-shaped arrays for passive-source measurements at 22 sites. They were able to sample wave trains over frequencies from 5 to 30 Hz in the active-source measurements and 2 to 10 Hz in the passive-source measurements. Yoon and Rix (2004) tested an irregularly-spaced linear array for active-source measurements and a circular array for passive-source measurements at two sites. They recorded wave trains in the frequency ranges of 4 to 100 Hz and 1 to 10 Hz in the active- and passive-source measurements, respectively. Both studies reported that the active-source data tended to give a slightly (about 5%) lower velocity in the zone of overlap. In combining the active- and passive-source datasets, Suzuki and Hayashi used all data in the overlap zone. Yoon and Rix, citing an increased influence of near-field effects on active-source data as the reason for the observed discrepancy, discarded the active-source data in the overlap zone in favor of the passive-source data. Both research teams concluded that the combined usage of active- and passive-source surface wave measurement was constructive.

In this study, for the active-source measurements, the Spectral Analysis of Surface Waves (SASW) method (Stokoe et al. 1994) was adopted. When a Raleigh-type surface wave travels in a layered medium, for the same wave train, the velocities are different at different frequencies. The SASW method utilizes this dispersive characteristic to resolve subsurface $V_s$ profiles. Coupled with sophisticated inversion techniques, this method proves to be powerful to resolve detailed velocity variations at shallow depths (e.g., Luke et al., 2003a). By the conventional procedure (Stokoe et al., 1994), an “effective” dispersion curve, meaning that the energy contributions of all
modes of surface waves and body waves are superimposed (e.g., Lai and Rix, 1999), can be constructed. For the passive-source measurements, the ReMi method was adopted (Louie 2001). This method uses multiple (12–24) vertical geophones spaced regularly along a linear array. A dispersion relationship, in terms of slowness and frequency, can be developed by the following transformations: 1) $p - \tau$ (slowness – intercept time) transformation, 2) two-dimensional Fourier transformation, and 3) power spectral analysis. The user interprets the fundamental-mode dispersion relationship from the transform by selecting the low boundaries of the power spectral ratio. The ReMi method has been demonstrated to be capable of resolving $V_s$ profiles to depths greater than 100 meters (Louie 2001), but it is particularly efficient in urban environments for conveniently determining $V_{S(30)}$.

Both the active- and passive-source methods involve some subjectivity in the process of developing dispersion datasets (measured wave velocities with respect to frequency or wavelength). In this research, the two datasets were interpreted independently so that one dataset would not influence the interpreter’s opinion toward the other dataset. This is particularly important for sites that exhibit complex dispersion characteristics and are therefore more challenging to resolve. The interpreted active- and passive-source dispersion datasets were then superimposed to form the overall combined dispersion curves for the test sites. For the overlap zone, dispersion data from both measurement types were kept. The combined dispersion dataset is then used in the inversion process to invert the $V_s$ profile.

Luke et al. (2003b) demonstrated that the inversion of seismic surface wave data can yield a non-unique solution; i.e., virtually identical theoretical dispersion curves.
could be obtained from multiple $V_s$ profile parameterizations. This problem can be mitigated to some extent by using a stochastic optimization process, which can incorporate prior knowledge of the site conditions, when available, and guide the solutions within expected ranges. To accomplish this, a two-step optimization process of simulated annealing fine-tuned by linearized inversion starting from a data-driven initial model (Liu et al., 2002), is used in this study. In the simulated annealing step, the global-minimum-error solution is obtained through stochastic searching within guided velocity search ranges. The velocity search ranges are fixed using independent knowledge of the site, primarily in the form of borehole logs and regional geologic frameworks. Thus, a solution compatible with geologic constraints is favored. In the final step of linearized inversion, the misfit between the theoretical dispersion curve and the target dispersion curve is narrowed. This final optimization step is not constrained.

To assess credibility of the final solutions, confidence measures are consulted. As mentioned above, different $V_s$ configurations could yield similar theoretical dispersion datasets. It is good practice to investigate the range of velocity variations that yield equally acceptable theoretical dispersion datasets, thus providing a visual sense of confidence levels and their variability among the layers. Past experience (Luke and Calderón, in review) has shown that the variability that would result from a statistically significant number of runs can be estimated from three inversion runs. The resolution matrix is another indicator of the quality of the solution. For a perfect match of the theoretical dispersion curve to experimental data, an identity matrix is expected. The farther the resolution matrix deviates from the identity matrix, the more the inverted velocity profile becomes a depth-filtered-velocity version of the true profile. However,
the resolution matrix is also not adequate as a stand-alone indicator of solution quality: because of the potentially non-unique nature of the inversion process, a high-quality resolution matrix does not necessarily indicate the correct solution (Luke et al. 2003b).

3.2.3 Data Collection and Processing

3.2.3.1 Testing locations

A field campaign was carried out in summer 2003, by the EGL and its collaborators at Utah State University and UNR, to map $V_s$ at 12 sites across the valley (Fig. 3.1). The sites were selected to fill in data gaps according to lithology and basin depth, to establish bases for comparison with independent $V_s$ datasets, to characterize $V_s$ for key legacy sites, and to support efforts to build correlations of velocities with lithologies. Basin depth at each site was estimated from the basin depth model of Langenheim et al. (2001). The numbers in parentheses following the site names listed below are the approximate basin depths in km. Out of the 12 sites, three sites (ANN(0.36), GRP(0.12), TRD(0.20)) are located in the shallow part of the basin; three sites (EGT(1.00), WHT(1.30), EFL(1.40)) are located in the intermediate-depth part of the basin, and four sites (MNL(2.50), DOE(2.00), CSN(3.50), NGC(3.50)) are located in the deep part of the basin. The sites CLB and SGS are the two “near-rock” legacy sites located on the edges of the basin.

3.2.3.2 Borehole logs

Water well and/or geotechnical borehole logs close to the testing sites, as available, were catalogued by Prof. Taylor. (Taylor et al. 2004). Figure 3.2 shows selected simplified logs, projected to an east-west line. The distances of the boreholes from the seismic test sites vary from a few meters to as much as 2 kilometers. The sites
located near the edge of the basin (shallow basin sites ANN and GRP) are dominated by coarse-grained sediments, whereas the sites in the middle of the basin (intermediate-depth and deep basin sites MNL, EGT, NGC and EFL) are dominated by fine-grained sediments. Shallow carbonate-cemented inclusions are found at two sites, EGT and EFL. The CLB site on the western edge of the basin is sited on a thin layer of unconsolidated sediment over sedimentary rock.

3.2.3.3 Field test configuration

For the active-source testing, the primary seismic source was a 2040-kg trailer-mounted dropped weight (Fig. 3.3) from Utah State University, designed and built by Prof. James Bay. This source was used for receiver spacings up to 80 m, which allows maximum depth of resolution to about 50 m, depending on the overall stiffness of the site. An instrumented sledgehammer was used for shorter spacings (16 m and shorter). The shortest spacing, sometimes 1 m but usually 2 m, was determined on-site to measure wavelengths of at least 1 m.

Three 1-Hz vertical geophones were used to record ground motions. Data were collected using a four-channel Agilent Technologies’ HP35670A spectrum analyzer. The testing array used in this study deviates slightly from the conventional common-center-point array. Figure 3.4 shows a typical setup of the testing array. Each setup accommodates two spacings, one double the other. Detailed source and receivers layout is documented in Appendix E.

For each spacing setup, multiple excitations were conducted and frequency-domain averaging was performed to obtain clean phase differences; thirteen files, including 3 coherence functions, 5 phase shift diagrams, 4 time series and the Fourier
spectrum of the source function, were saved. An example case, from the 80-40 m spacing, reverse direction measurement at TRD site, is plotted in Figs. 3.5 to 3.8.

The analyzer was configured in such a way that the phase differences between receivers were calculated from transfer functions between source and receiver, averaged in the frequency domain. This is done to improve the signal-to-noise ratio.

For the passive source (ReMi) measurement, a linear array with twenty-four 4.5-Hz vertical geophones, spaced 10 m apart, was used. Ten 24-second records were collected at each site, at 500 samples per second, using a Geometrics 24-channel StrataVisor NX seismograph. The mid-point of the ReMi arrays was placed as close as possible to the center point of the SASW arrays.

3.2.3.4 Development of combined dispersion curves

The effective dispersion curves from the SASW measurements were developed by the author. Dr. Satish Pullammanappallil and Prof. John Louie (UNR, Nevada Seismological Laboratory) developed the dispersion curves for the ReMi measurements. The dispersion datasets developed from SASW measurements normally contain more than a thousand data points, while the hand-picked ReMi dispersion curves contain less than a hundred. To balance the number of data points from both methods, the SASW dispersion datasets were condensed to less than 100 points by logarithmically-averaged binning with respect to wavelength. The two dispersion curves were then superimposed (Fig. 3.9). Except for the CSN site, the SASW measurements captured more high frequency energy and filled in the curve at short wavelengths, whereas the ReMi measurements captured more low frequency energy and provided data at longer wavelengths.
The frequency ranges measured during active-and passive-source testing were from about 3.5 to 500 Hz and 1 to 40 Hz, respectively. Maximum wavelengths for the passive-source data are often as long as 1000 m. At the CSN site, however, the maximum wavelength from ReMi was less than the 160-m maximum afforded by the SASW measurements. The author suspects that this is because (1) it is a low-velocity site, so high signal attenuation is expected, and (2) as a vague possibility, discernible low frequency ambient noise is lacking for this site. For most of the measurements, the two dispersion curves exhibit close agreement in the overlap zone. Exceptions are GRP and CLB, where the wave velocities obtained from ReMi measurement are 10 to 50% lower than those from the SASW measurement. Due to this discrepancy, for the GRP site, ReMi data were omitted from further analyses. For the CLB site, since the wave velocities of the two measurements at short and long wavelength limits agree (Fig. 3.9), active- and passive-source data were superimposed as usual.

For the twelve sites studied, the SASW and ReMi dispersion datasets overlap roughly over the wavelength range of 10 to 100 m. In comparison with the studies cited earlier (Suzuki and Hayashi 2003, Yoon and Rix 2004), the current datasets have a broader overlap zone between active- and passive-source measurements. In contrast to the studies cited earlier, no consistent trend for one measurement (passive- or active-source) having consistently higher or lower velocities than the other was observed.

3.2.3.5 Inversion and data reporting

The forward model applied in this process makes the simplifying assumption that the measured motion is dominated by fundamental-mode Rayleigh waves. This is a simplifying assumption because the effective dispersion curves usually contain some
higher-mode energy, and this is more likely to be a significant factor for a complex profile (Jin et al., 2006). As discussed by Wu et al. (2003), before inverting the dataset, the experimental dispersion curve should be smoothed. The purpose of smoothing is to make it possible to develop a reasonable theoretical match using the simplified forward model. Thus, the combined dispersion curve was smoothed by convolving it with a 5- to 9-point kernel. The greater the number of points used in the kernel, the greater the degree to which the dispersion data are smoothed. The smoothed dataset then became the target for inversion, using simulated annealing fine-tuned by linearized inversion (Luke et al., 2003). The depth of resolution was fixed at one quarter of the maximum wavelength. The velocity search range for optimization by simulated annealing was bounded using data-driven expectations (e.g., Jin et al., 2003) and independent knowledge of the sites from borehole logs, such as depth and velocity ranges of carbonate cemented lenses and water levels.

For the starting model, appropriate values of density and compression wave velocity or Poisson’s ratio need to be assigned to each layer. To fix these values, first, a detailed investigation was conducted for the Engineering Geophysics Test Site (EGTS) on UNLV campus.

Six boreholes have been drilled at the EGTS from 1996 to 2003. The depths range from 6.10 m (20 ft) to 30.48 m (100 ft). The soil types encountered in these boreholes were dense sand, stiff clay and/or silt interbedded with layers of partially to fully carbonate-cemented sediments. In all of these boreholes, carbonate-cemented sediments were encountered between 2.5 and 5 m depth; this reflects a relatively uniform distribution pattern. Below 5 m, silty and sandy clay predominates. The groundwater
The table encountered in these boreholes ranged from 2.4 m (8 ft) to 4.0 m (13 ft). Differences may be due to periodic groundwater fluctuation.

Site-specific measurements conducted by Tecle et al. (2003), and published values from different researchers, as discussed in detailed below, were examined to select appropriate density and Poisson’s ratio for the study site. Table 3.1 is a summary of recommended values of Poisson’s ratio and density from the literature, categorized by soil type and density. The authors did not specify the moisture conditions.

Inferred from Table 3.1, the density of both stiff clay and dense sand was assigned to be 17 kN/m³.

Degree of saturation is known to have a great influence on Poisson’s ratio of unconsolidated sediments. In their study of model parameters for surface wave data inversion, Foti and Strobbia (2002) recommended Poisson’s ratio of 0.2 for soils above the ground water table and 0.49 for soils below the ground water table. They noted that for dry soils, the Poisson’s ratio is typically in the range 0.1 to 0.3, but for saturated soils, the Poisson’s ratio is close to 0.5.

Inci et al. (2003) examined the influence of degree of saturation and plasticity on Poisson’s ratio of soils in the laboratory. They tested three types of soils with low, medium and high plasticity, using the ultrasonic pulse transmission method. The samples were initially at optimum water content and then gradually dried to test the influence of water content on Poisson’s ratio. They found that the Poisson’s ratio depends highly on the degree of saturation and slightly on the plasticity of the soils. The Poisson’s ratio for moist samples ranged between 0.4 to 0.5 and decreased to 0.1 to 0.2 at the end of drying.
The testing results were very scattered. For the same soil with the same degree of saturation, the difference in Poisson's ratio was as large as 0.2.

Gazetas (1991) proposed the following values for Poisson's ratio of soils: 0.4 for nearly saturated clays above the water table, and 0.5 for saturated clays and sands beneath the water table. Bowles (1988) proposed Poisson's ratio of 0.1-0.3 for unsaturated clay and 0.4-0.5 for saturated clay.

Tecle et al. (2003) calculated Poisson's ratio at the EGTS from compression and shear wave velocities measured using the seismic downhole test, to 7 m depth. The water table at the time was at 2.7 m depth. The Poisson's ratio of materials above the water table was found to be between 0.26 and 0.3, which falls close to the values recommended by Gazetas and Bowles. The Poisson's ratio of materials below the water table was between 0.4 and 0.5, which is the same as that which has been reported by Bowles and close to the value recommended by Gazetas.

Based on literature search and the testing result from Tecle et al., the Poisson's ratio for unsaturated soils in this study is set to be 0.3. For saturated soils, the Poisson's ratio is set to be 0.4, which equals to the lower bound measured for the EGTS by Tecle et al., and corresponds to the lower bound value recommended for saturated soils and the upper bound value recommended for soils with unspecified moisture condition.

A few studies about Poisson's ratio of carbonate cemented soils have been conducted. Stone and Luke (2001) tested the density and Poisson's ratio of cemented material cored from the EGT site. The density was found to be 2500 kN/m$^3$ (160 pcf). Velocity measurements on cores in free-free vibration yielded a Poisson's ratio of 0.23 while field cross-hole tests yielded Poisson's ratio of 0.33. This difference might be due
to the fact that the sample tested in the lab is intact and very well cemented, while field measurements are affected by discontinuities, variable degree of cementation, and other irregularities occurring at the macro scale. Tecle et al. (2003) calculated Poisson’s ratio at the EGT site for a carbonate-cemented layer at 3.25 m depth, in the same study discussed previously. The Poisson’s ratio for that layer was 0.33. It is important to note, however, that due to differing degree of cementation, extent of discontinuities, particle size of the cemented media, and varying stiffness of the cemented lenses, the stiffness and Poisson’s ratio for cemented soils will vary.

Since carbonate-cemented soil is stiff (as reflected by a very high $V_s$), it can be considered as rock for engineering purposes. Thus, to understand the appropriate range of Poisson’s ratio of cemented soils, it is helpful to consider the recommended values of Poisson’s ratio of rock (Table 3.2).

Thus, for cemented media, the Poisson’s ratio is set to be 0.25, which falls between values measured in the laboratory (0.23; Stone and Luke, 2001) and the field (0.33; Tecle et al., 2003) and the recommended range of Poisson’s ratio for different rocks (0.15-0.33). Because the heavily cemented media behave like rock, the same value of Poisson’s ratio is applied regardless of moisture conditions. For density, considering spatial variability of field conditions, the in situ density is likely to be less than that of the intact specimen tested by Stone and Luke; as a result, a density of 2200 kN/m$^3$ is selected.

In summary, the values listed in Table 2.3 are recommended for the EGTS. The same values were applied to similar soils at the other 11 test sites, because no site-specific data were available.
As discussed earlier, a preferred $V_S$ profile averaged from three different inversion runs using the same input parameters was reported for each site. An example for the EGT site is shown in Fig. 3.10. The variability among the three iterations for this site is generally small (less than 10%), except at the depth of the cemented inclusion. Two of the three solutions correctly resolved the high-velocity cemented inclusion. The resolution matrices (Fig. 3.11) are similar, and all of good quality. The portions of the matrices having the most smearing are the layers surrounding the stiff inclusion. This outcome is to be expected in a profile having a stiff inclusion (Luke and Calderón, submitted). Considering the available confidence measures for all twelve sites studied, quality of solutions was generally good but variable, with no geographically- or geologically-consistent patterns.

The $V_S$ profiles of the 12 sites are plotted with all the pre-existing $V_S$ profiles including crosshole, downhole and ReMi measurements in Fig. 3.12. For the purpose of legibility, the ReMi profiles of the UNR transect are grouped and a single $V_S$ profile is chosen for each 2 kilometer interval started from south of Cheyenne Avenue (Scott et al., 2006). It is observed that the new $V_S$ profiles from combined SASW and ReMi measurements for the deep and intermediate-depth parts of the basin, and the pre-existing $V_S$ profiles have the same trend: velocities are lower compared to those of the shallow part of the basin and they increase gradually with depth. The new $V_S$ profiles have detailed layer geometry whereas the ReMi measurements, which dominate the pre-existing profiles, have simple layer geometries.
Table 3.1 Summary of Poisson’s ratio and density of stiff clay and dense sand from literature

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Poisson’s ratio</th>
<th>Density (kN/m²)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiff clay</td>
<td></td>
<td>17</td>
<td>Das (1994)</td>
</tr>
<tr>
<td></td>
<td>0.1-0.3</td>
<td></td>
<td>Bardet (1997)</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td></td>
<td>Gazetas (1991)</td>
</tr>
<tr>
<td>Sandy clay</td>
<td></td>
<td>0.2-0.3</td>
<td>Bowles (1988)</td>
</tr>
<tr>
<td>Silt (dense)</td>
<td></td>
<td>16-17</td>
<td>Bardet (1997)</td>
</tr>
<tr>
<td>Silt (hard)</td>
<td>18-19</td>
<td></td>
<td>Bardet (1997)</td>
</tr>
<tr>
<td>Dense sand</td>
<td></td>
<td>17.1</td>
<td>Dunn (1980)</td>
</tr>
<tr>
<td></td>
<td>0.3-0.45</td>
<td></td>
<td>Das (1994)</td>
</tr>
<tr>
<td></td>
<td>0.3-0.4</td>
<td></td>
<td>McCarthy (1988)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Hunt (1984)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Coduto (1999)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bardet (1997)</td>
</tr>
<tr>
<td>Dense silty sand</td>
<td></td>
<td>19</td>
<td>Das (1994)</td>
</tr>
<tr>
<td>Dense coarse sand</td>
<td></td>
<td>17-18</td>
<td>Bardet (1997)</td>
</tr>
<tr>
<td>Fine uniform dense sand</td>
<td></td>
<td>17-18</td>
<td>Bardet (1997)</td>
</tr>
</tbody>
</table>

Table 3.2 Summary of Poisson’s ratios of rock (Bardet 1997)

<table>
<thead>
<tr>
<th>Material</th>
<th>Poisson’s ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite, sound</td>
<td>0.15-0.24</td>
</tr>
<tr>
<td>Granite, partially decomposed</td>
<td>0.15-0.24</td>
</tr>
<tr>
<td>Limestone</td>
<td>0.16-0.23</td>
</tr>
<tr>
<td>Sound, intact igneous and metamorphics</td>
<td>0.25-0.33</td>
</tr>
<tr>
<td>Sound, intact sandstone and limestone</td>
<td>0.25-0.33</td>
</tr>
<tr>
<td>Sound, intact shale</td>
<td>0.25-0.33</td>
</tr>
</tbody>
</table>

Table 3.3 Values of Poisson’s ratio and density adopted for this study

<table>
<thead>
<tr>
<th>Material</th>
<th>Poisson’s Ratio</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soils, above water table</td>
<td>0.3</td>
<td>1700</td>
</tr>
<tr>
<td>Soils, below water table</td>
<td>0.4</td>
<td>1700</td>
</tr>
<tr>
<td>Cemented material</td>
<td>0.25</td>
<td>2200</td>
</tr>
</tbody>
</table>

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Figure 3.1 Las Vegas basin map showing $V_S$ testing locations, overlaid on depth contour map (from Langenheim et al. 2001). Red filled circles are new $V_S$ locations; green filled circles are existing $V_S$ locations; black triangles are the UNR transect.
Figure 3.2 Borehole logs available for the test sites, projected to an E-W line; 32x vertical exaggeration. The vertical exaggeration emphasizes the net west-to-east slope of the basin. "P CEMENTED": partially cemented.
Figure 3.3  Trailer-mounted dropped weight source from Utah State University.

Figure 3.4  Setup of SASW testing array: S – source; R1, R2, R3 – receivers; L – spacing
Figure 3.5  SASW data collection: example time records (TRD site, 80-40 m spacing, reverse direction): a) Source; b) Receiver R1 (40 m from source); c) Receiver R2 (80 m from source); d) Receiver R3 (160 m from source).

Figure 3.6  Fourier spectrum of the SASW source

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Figure 3.7  
Sample SASW dataset: phase shift functions between different sensors. 

H21, H31, and H41 are phase shifts between receivers 2, 3, 4 and source. 

H32 and H43 are the phase shifts calculated from transfer functions 

between receiver pairs.
Figure 3.8 SASW dispersion curve: coherence functions. “Cxy” is coherence between sensors x and y. Coherence function is defined in Section 5.1.
Figure 3.9  Combined dispersion curves arranged in order of basin depth, from shallow to deep.
Figure 3.10  Example $V_s$ profile developed through inversion (EGT site)

Figure 3.11  Example resolution matrices (EGT site). Each is normalized independently.
Figure 3.12  Available $V_s$ profiles in the Las Vegas valley. Black: representative pre-existing $V_s$ profiles; Colored: 12 new measurements; Deep and intermediate-depth: green; Shallow: blue; 'x': pre-existing point measurements.
CHAPTER 4

SPATIAL VARIATION OF $V_s$ AND ITS RELATIONSHIP WITH SEDIMENT DISTRIBUTION

As reported by Rodgers and McCallen (2002) and Rodgers et al. (2004 and 2006), observed ground motions in the Las Vegas valley exhibit strong spatial patterns with regard to basin depth; namely, high ground shaking was recorded in the deep and intermediate-depth part of the basin, and relatively low ground shaking was recorded in the shallow part of the basin. As discussed in chapter 2, the local ground motion amplification is caused by impedance contrast. Seed and Idriss (1982), as mentioned earlier, have produced average normalized response spectra for four generic site categories based solely on qualitative descriptions of site stiffness ($G = \rho \times V_s^2$, where $G$ is shear modulus (stiffness) and $\rho$ is density). It is suspected that the observed spatial pattern of site amplification in the Las Vegas basin is directly related to the geographic distribution of shallow sediments, which has control of overall site stiffness. To prove this hypothesis, the relationships of $V_s$ to basin depth, sediment distribution to basin depth, and $V_s$ to sediment type, are examined in the following, using the 12 new $V_s$ datasets and other existing $V_s$ measurements introduced in Chapter 3.

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4.1 $V_S$ versus Basin Depth

In chapter 3, the twelve sites where new $V_S$ measurements have been made were grouped into three categories according to basin depth, namely, shallow, intermediate-depth, and deep. The dispersion curves and average $V_S$ profiles of the 12 sites are superimposed in Figs. 4.1 and 4.2 respectively. From the two plots, the following trends are observed:

- In general, the basin depth is inversely correlated to $V_S$.
- The contrast in $V_S$ between intermediate-depth and deep basin sites is not as strong as that between the intermediate-depth and shallow basin sites.
- At short wavelengths ($\leq 10$ m), the $V_S$ profiles are not distinguishable with respect to basin depth. This implies predominant influence of features that are less strongly related to geographic location in the basin, such as cementation, weathering, and low confinement stresses.
- An exception occurs for one site, TRD, which is in the shallow part of the basin yet has low $V_S$. No geotechnical borehole data are available at this site to help explain the anomaly.

In general, this spatial variation of $V_S$ explains the observed trend of site amplification. With respect to the near-rock and shallow parts of the basin, the intermediate-depth and deep parts of the basin have lower $V_S$, and produce higher site amplifications during seismic events. The details of velocity differences among basin depth categories and their correlations with observed ground motions are discussed in the following.
For the test-generated ground motions considered in this research (56 records), an increase of site amplification was distinctly observed for the intermediate-depth site category and the shallow site category with respect to near-rock sites, especially the SGS site. However, no amplification was observed for the deep site category with respect to the intermediate-depth site category. For the LSM earthquake (9 records), the near-rock site CALB and the shallow site ANN are of equal source-to-site distance, and are 25% closer to the source than is the other near-rock site (SGS). Despite the fact that CALB and ANN are closer to the source than are the other basin sites, most probably because of local site effects, the amplitudes of motions recorded at these two sites are about the same as those recorded in the intermediate-depth part of the basin, and smaller than those recorded in the deep part of the basin. Similar to the test-generated ground motions, with respect to SGS, the intermediate-depth sites clearly show large amplification, but the amplification for deep sites with respect to intermediate-depth sites, though much more significant than that for the nuclear test data, is not as dramatic.

From this section, it is observed that the geographic distribution of site amplification is directly related to different site stiffnesses in the basin, reported in terms of $V_s$. Based on observed ground motions and $V_s$ profiles, for seismic response evaluation, it is not necessary to differentiate the intermediate-depth and deep zones of the basin. Thus, these two categories are grouped together and simply referred to as “deep” (> 600 m) in the remaining discussions.
4.2 Sediment Distribution, $V_s$ and Basin Depth

As mentioned previously, Wyman et al. (1993) observed that the sediments in the basin are primarily derived from its western boundary, the Spring Mountains, and generally grade into finer material toward the east and south.

Of the 12 new study sites introduced in Chapter 3, lithologic logs are available for 7 of them. These seven wells, together with the mapped $V_s$ in their vicinity, are projected to a W-E line in Fig. 4.3.

From Figure 4.3, it is observed that the coarse- and mixed-grain size deposits are associated with relatively high $V_s$ and occupy the shallow part of the basin; whereas the fine-grained clay-rich sediments are associated with relatively low $V_s$ and occupy the intermediate-depth and deep part of the basin.

As mentioned in Chapter 2, this observed geographic distribution of basin sediments is confirmed by a recent, comprehensive, basin-fill study conducted by Taylor et al. (2004), where the investigators verified that clay-rich deposits occupy the deep part of the basin (central and south), and coarse- and mixed-grain size deposits dominate the shallow part of the basin (west).

To recap, the site amplification in the valley is directly related to overall site stiffness, which, in turn is governed by the sediment distribution. To define the geographic distributions of seismic response in the Las Vegas valley, a sediment distribution map, based on material predominance over the upper 30 m, is developed (Fig. 4.4). The reasons for choosing the upper 30 m are the following: 1) most geotechnical site investigations stop at 30 meters depth or less, 2) the $V_s$ of the upper 30 m generally has “controlling” influence on surface response (Anderson et al., 1996), and 3) code-
based practice such as the IBC 2000 considers the upper 30 m only for site classification. This map is based on Taylor’s well log database. Jeff Wagoner (LLNL) provided expertise for map development, using the software program EarthVision. Three predominant sediment types, gravel, clay and cemented, were identified from Taylor’s database. The sediment is deemed as predominant if it constitutes 50% or more of the sediments over the upper 30 m according to the 3-D EarthVision model. Sand deposits are sparse and do not show predominance in the basin. If there is no dominant sediment at a certain area or the sediment type was not determined, the map shows its background color.

The basin-depth-category map and predominant-sediment-type map are shown in Fig. 4.5. In general, as has been reported by Taylor et al. (2004), the deep part of the basin, particularly the part encompassing sites CSN, NGC, SDS, EFL, WHT, EGT, LRE, LVS, 115, and DOE, is dominated by clay; the shallow part of the basin, particularly the west part where ANN, RAI, GRP, and TRD are located, is filled by coarse- and mixed-gain size deposits and cemented media. Since the wells were not logged to a common standard, the degree of cementation can not be differentiated. It is possible that a material logged as “cemented clay” turns out to be only lightly cemented in engineering terms, and thus behaves more like uncemented clay. On the other hand, if the material logged as cemented clay turns out to be heavily cemented, it behaves more like rock. The deep part of the basin in the north where MNL is located is dominated by clay. However, the cemented material shown as occupying a large area in the north where the basin is deep was interpreted from only two wells (Fig. 4.4). Thus the “cemented” designation is not considered to be meaningful. In general, the sediment types at the edges of the mapped
space, particularly at the north and west edges, were extrapolated from sparse data; thus, more well logs are needed to verify the extrapolated sediment types in those areas.

4.3 Correlation of $V_S$ with Lithology

One objective of this research is to develop a seismic microzonation map for the Las Vegas basin. As observed by many researchers, the lack of sufficient, reliable $V_S$ data is a common impediment for earthquake microzonation mapping in deep alluvial basins. To fill in the data gaps, characteristic $V_S$ profiles can be developed by correlating $V_S$ with sediment units or geological age (Romero and Rix 2001; Zhang et al. 2004). In the following, a direct correlation and a basin-wide correlation between $V_S$ and sediment units was investigated.

4.3.1 Direct correlation between $V_S$ and lithology

Direct correlations between $V_S$ and lithology were conducted for all thirteen sites where both $V_S$ measurements and lithology logs are available. At six sites the wells are within 50 m of the $V_S$ measurements and so are considered to be collocated. At the other seven sites, the distances between center points for $V_S$ measurements and well locations range from 180 to 1500 meters (Table 4.1). The sediments were grouped into clay, silt, sand, gravel, cemented material, and partially cemented material. Shear wave velocity increases with the increase of depth and, consequently, confining pressure. To account for this, the profiles were divided into five depth ranges: 0-15 m, 16-30 m, 31-90 m, 91-200 m and > 200 m. The sub-range thicknesses increase with depth to reflect that resolution of $V_S$ usually decreases with depth. The result is summarized in Fig. 4.6. The data densities for clay and gravel are larger than those of the rest, especially at shallow
depths. Considering lithologies and velocities at the same depth, the finer sediments are associated with lower $V_s$, and the coarser sediments are associated with higher $V_s$. As depth increases, especially for clay and gravel, the $V_s$ generally increases, reflecting the influence of confining pressure. For the cemented and partially cemented media, a higher velocity is expected compared to their uncemented counterparts. However, this is not always the case. As mentioned previously, these apparent discrepancies are not surprising considering the variable nature of the borehole logging. The SGS log is particularly questionable, because it shows a very high velocity, non-cemented clay layer where rock would be expected. For these reasons and since the sample size is not large enough for meaningful statistical calculation, these correlations can be taken as general guidance only.

4.3.2 Basin-wide correlation of $V_s$ with geologic database

To build a digitized model of shallow $V_s$ for the Las Vegas basin, the discrete $V_s$ measurements are being tied to basin-wide correlations of a velocity map with an extensive geologic database, in collaboration with Prof. Taylor and Jeff Wagoner. Professional expertise and software capable of performing spatial geo-statistical analysis for large databases is needed. At the current stage, a lithology-adjusted map of $V_{s(30)}$, the slowness-averaged $V_s$ over the upper 30 m, has been developed (Figure 4.7). The $V_{s(30)}$ is defined as follows (IBC 2000):

$$ V_{s(30)} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_s(i)}} $$
where \( d_i \) is the thickness, \( V_{S(i)} \) is the shear wave velocity of layer \( i \). The quantity 
\( 1/V_{S(i)} \) is called the “slowness” of layer \( i \).

Figure 4.7 indicates that the alluvial sediments in the central and east parts of the Las Vegas basin, where the alluvial deposits are generally deep, have relatively low \( V_{S(30)} \) of 500 m/s or less. Research to develop depth-adjusted correlations is ongoing.

4.4 Summary

The study of observed ground motions, shallow \( V_S \) structure, basin sediment distribution, basin depth contours, and the correlations among them confirmed the following:

• The deep part of the basin (central and south) is filled predominantly with clay-rich deposits to at least 30 m, which is associated with lower \( V_S \) and higher site amplification.

• The shallow part of the basin is filled predominantly with coarse- and mixed-grain size deposits, which are associated with higher \( V_S \) and lower site amplification.

• Different degrees of cementation occur at varying locations and depths across the basin. Further geotechnical and geophysical studies are required to delineate zones of cementation.

• The spatial pattern of observed ground motion amplification appears to be related with sediment type. Different sediment types are associated with different ranges of \( V_S \). Sediment-type distribution is correlated with basin depth.
Table 4.1  Distance between $V_s$ measurement locations and the corresponding borehole locations

<table>
<thead>
<tr>
<th>Site</th>
<th>SGS</th>
<th>ANN</th>
<th>GRP</th>
<th>RAI</th>
<th>TRD</th>
<th>EFL</th>
<th>EGT</th>
<th>I15</th>
<th>LVS</th>
<th>SDS</th>
<th>WLV</th>
<th>CSN</th>
<th>NGC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dist.(m)</td>
<td>180</td>
<td>560</td>
<td>560</td>
<td>0</td>
<td>940</td>
<td>1450</td>
<td>30</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>50</td>
<td>990</td>
<td>1570</td>
</tr>
</tbody>
</table>

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Figure 4.1 Experimental dispersion curves for all 12 sites. Blue: shallow basin sites, Magenta: intermediate-depth basin sites, Green: deep basin sites.
Figure 4.2 $V_s$ profiles for all 12 sites. Blue: shallow basin sites, magenta: intermediate-depth basin sites, green: deep basin sites.
Figure 4.3 $V_s$ (minor unit: 2000 m/s) and lithology from nearby well logs, projected to an E-W line; 40× vertical exaggeration. "P CEMENTED": partially cemented. Note: refer to Fig. 3.2 for more detailed lithology logs.
Figure 4.4 Sediment distribution map (dark grey: clay; yellow: coarse- and mixed-grained deposit; light grey: cemented) with well locations (black dots); background color indicates no data or no predominant sediment.
Figure 4.5  a) Predominant sediment type map (dark grey: clay; yellow: coarse- and mixed- grained deposit; light grey: cemented; background color indicates no data or no predominant sediment) and b) basin depth contour map with $V_S$ measurement locations superimposed. Red dots: ReMi transect; green dots: other $V_S$ measurements.
<table>
<thead>
<tr>
<th>Depth in meters</th>
<th>Clay</th>
<th>Silt</th>
<th>Sand</th>
<th>Gravel</th>
<th>Cemented</th>
<th>Partially Cemented</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
</tr>
<tr>
<td>16-30</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
</tr>
<tr>
<td>31-90</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
</tr>
<tr>
<td>91-200</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
</tr>
<tr>
<td>200 &amp; deeper</td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
</tr>
</tbody>
</table>

Sites arranged in the following order: SGS,ANN,GRP,RAL,TRD,EFL,EGT,H5,LVS,SDS,WLV,CSN,NGC

Figure 4.6 Correlation of sediment types with $V_s$. Sites in general order from shallow to deep. Black: $V_s$; white: absent material; gray: no data.
Figure 4.7 Lithology-adjusted $V_{S30}$ contour map. $V_{S30}$ varies from 350 m/s to 2000 m/s from cool color to hot color (earthVision graphic, courtesy Jeff Wagoner, LLNL).
CHAPTER 5

SELECTION OF OPTIMUM DEPTH TO MODEL HALFSPACE

Because of the ambiguity in defining the soil-bedrock interface and the lack of confining-pressure adjusted dynamic soil properties at great depth, one main challenge of performing 1-D site response analyses for deep soil deposits is properly parameterizing the soil column. As discussed previously, in this study, to address the above-mentioned ambiguities, the halfspace will be placed within the soil column. In this chapter, the existing criteria of placing model halfspace are examined and a refined criterion is proposed and tested. Most of the material presented in this chapter has been published in a peer reviewed conference paper (Liu and Luke, 2004).

To investigate the site amplification effect due to shallow basin sediments, minimizing path effect, a pair of rock and soil sites that have equal source-to-site distances and similar azimuth angles were desired. By examining both test-generated and earthquake ground motion records, it is found that the near-rock site on the east side of the basin, SGS, and the soil site within the basin, SE6, at which ground motions were monitored during nuclear test events, fulfill this requirement (Table 2.1). Ideally, then, the deconvolved rock motion recorded at SGS would be representative of the input motion at SE6. Therefore, the SGS site is selected as the reference (“rock”) site and the SE6 site is selected as the subject (“soil”) site for this study.
5.1 Coherence study

As mentioned previously, ground motions were recorded at both near-rock sites and soil sites in the Las Vegas basin during historic seismic events. These measured motions serve as valuable datasets to test and calibrate a seismic response model. The measured near-rock motions can be used as the basis for input motions at the soil sites and comparisons can be made between the projected and the measured motions at the soil sites. To be able to do so, the measured near-rock motion and soil motion should share the same characteristics in the frequency domain, even though the amplitudes are different because of site amplification effects (Finn et al., 1993). This comparison can be achieved by computing the coherence function between the measured near-rock and soil motions.

The coherence function is defined as the following:

\[ \gamma^2 = \frac{G_{xy} G_{xy}^{*}}{G_{xx} G_{yy}} \]

where \( G_{xy} \) is the cross power spectrum between the input and output signals, \( G_{xx} \) and \( G_{yy} \) are the auto power spectra of the input and output signals, and \( G_{xy}^{*} \) is the complex conjugate of \( G_{xy} \).

The coherences between recorded “near-rock” (SGS) and soil (SE6) motions for the four nuclear events targeted in this study, namely Barnwell (BA), Cottage (CO), Bodie (BO) and Gascon (GA), were evaluated (Fig. 5.1). It is observed that for all four events, between 1 and 5 Hz (0.2 to 1 sec), the coherences are generally greater than 0.5, thus can be considered to be fairly closely related. For the BA event, the near-rock and
soil motions are well related to about 10 Hz. The coherences vary, and generally are lower than 0.5, above 10 Hz.

5.2 Pre-processing of time-acceleration records

As mentioned in Chapter 2, two different sets of seismic records are available and are used in this study, namely, the nuclear-test-generated ground motions and the 1992 LSM earthquake ground motions. Because of different source-time functions and mechanisms, high-energy explosions produce a greater proportion of high-frequency compression- (P-) wave energy than do earthquakes. As a matter of fact, the P/S amplitude ratios have been successfully used to discriminate high-energy explosion events from natural earthquakes (Walter et al., 1995). This is illustrated by comparing time histories of explosion and earthquake ground motion records at soil (SE6) and near-rock (SGS) sites (Fig. 5.2A). A line marks the onset of shear wave energy. All the acceleration time histories used in this study are east components. Comparison of response spectra of east and north component for the four nuclear events, BA, BO, CO, and GA, at SGS indicated that both time histories have similar $PGA$ and peak $S_a$ (Figure 5.2B).

It is observed that the high-frequency P-wave energy has about the same amplitude for both near-rock and soil sites, whereas after the S-wave energy arrives, the amplitude of the soil motion is much larger than that of the rock motion, as demonstrated by ground motions recorded at these two sites during the four nuclear test events studied (Fig. 5.3). The yields of these events ranged from 20 to 150 kilotons, corresponding to body wave magnitudes of 5.0 to 5.9.
Because shallow sediments typically amplify vertically-propagating horizontally-polarized shear waves, if the high-frequency P-wave energy is not filtered out from the input motion, the high-frequency energy preserved on the rock site will be erroneously amplified through the sediment column, which will cause over-prediction of $PGA$ and $S_{a(max)}$, and inappropriate energy shift to shorter periods at the soil sites. This situation is illustrated in a section, "Discussion: Minimization of P-wave energy in input motion," which appears at the end of this chapter, after the SHAKE input parameters have been discussed. Therefore, to obtain reasonable projections at the soil site, it is necessary to exclude most of the P-wave energy that was preserved on the rock site. This was accomplished by processing only the portion of the time history that starts a few seconds before the onset of the S-wave energy. The record was ended where accelerations are less than approximately 25% of the peak. The remaining time history is 56 seconds long.

With a sampling interval of 0.005 sec, the 56-second time history has more than 10,000 points. For SHAKE analyses, it is recommended that the number of points of the input motion be less than 4096 (Ordonez, 2000). Thus, the truncated time history is down-sampled by a factor of 4, which leaves 2801 points at a sampling rate of 50 Hz. This equates to the preservation of frequency content up to 25 Hz. Observation of Fourier spectra confirms that down-sampling did not cause the loss of important information in the frequency range of interest (Fig. 5.4).

After the truncation, there is no “quiet time” at the beginning and at the end of the record (Fig. 5.5). For the purpose of Fourier analyses, to prevent the transformation from aliasing, it is preferred to have a period of quiet time at both ends to force periodicity. This can be achieved by applying windowing techniques. Different windows (e.g.,
Hanning window, Hamming window, exponential window, rectangular window, triangular window etc.) serve different purposes. At the beginning of the time record, it is preferred to have a cosine-style taper which forces the signal to start from zero amplitude and allows the amplitudes to increase rapidly to the recorded value within a short period of time as specified. At the end of the record, it is preferred to have a window which makes the signal die out quickly. Both Hanning and Hamming windows serve for the first purpose. To the author’s knowledge, exponential window is the only one which serves for the second purpose.

The Hanning window \( W_{\text{Hann}} \) for \( N \) points is defined as:

\[
W_{\text{Hann}}(i) = 0.5 + 0.5 \cos\left(\frac{2\pi i}{N}\right)
\]

The Hamming window \( W_{\text{Ham}} \) for \( N \) points is defined as:

\[
W_{\text{Ham}}(i) = 0.54 + 0.46 \cos\left(\frac{2\pi i}{N}\right)
\]

where \(-N/2 \leq i \leq N/2\)

The two windows differ only in the constants, thus, a similar windowing effect can be achieved by applying either of them. In this study, the Hanning window was adopted.

Based on the above-mentioned comparison, a Hanning window was applied to the first second of the truncated time history and an exponential window was applied to the ending 5 sec (Fig. 5.6).
5.3 Site response study of the paired near-rock and soil sites

5.3.1 Period range of amplification

As discussed above, SGS and SE6 were selected as the paired near-rock and soil sites. To define the period and amplitude range of amplification, the $S_a$ for these two study sites during the four nuclear events were computed (Fig. 5.7). The entire acceleration time histories, having a record length of more than 300 sec and a sampling interval of 0.005 sec, were used to calculate the response spectra. With respect to the rock site, significant amplification is observed at the soil site in the period range from 0.3 to 5 sec. For the period range from 0.1 to 0.3 sec, the two sites have about the same spectral accelerations, except for the GA event, where a narrow spike appears at 0.2 sec for the soil site. Predominant period shifts from shorter to longer for the soil site with respect to the rock site.

The acceleration response spectral ratios (soil over rock), averaged for the four events, indicate that with respect to the rock site, the motion recorded at the soil site is amplified by factors from 2 to more than 5, from 0.3 to 5 sec (Fig. 5.8).

5.3.2 Input motion

The preprocessed (truncated, down-sampled, and windowed) time acceleration records were used for the following analysis. In the 1-D analysis, an appropriate cut-off frequency must be specified. A set of filters, namely, low-pass, band-pass and high-pass filters, were applied to a sample time history for the rock site to study the energy distribution (Fig. 5.9). Frequencies higher than 10 Hz carry negligible amounts of energy. Since the SHAKE manual recommends a cut-off frequency of 10 to 15 Hz, to be conservative, the cut-off frequency is chosen to be 15 Hz, which is still well below the

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peak frequency preserved in the down-sampled dataset. However, the coherence study indicated that useful data is generally below 5 to 10 Hz.

Because a thin layer of superficial deposit covers the near-rock site, surface motion recorded at this site was deconvolved through a short soil column to develop the appropriate input motion. Rock outcrop was observed about 20 m east of the $V_S$ measurement location. Two $V_S$ profiles, developed independently using surface wave methods, were available for this site (Fig. 5.10). One profile was developed in the EGL using the SASW method. The source energy was an instrumented sledge hammer. The $V_S$ profile was developed using the same process described previously. Prof. John Louie developed the other profile using the ReMi method. This $V_S$ profile was generated through iterative forward modeling, manually, by trial and error. Considering differences in layer geometry of the models, the two profiles agree closely. A sharp velocity increase from approximately 600 to more than 1000 m/s occurs at 12 to 15 m depth; this depth is interpreted as the depth to bedrock. No borehole data are available for this site. Since the SASW method provides higher resolution at shallower depths, its profile is used in the deconvolution process. The model halfspace is placed at 12 m depth. The response spectra of the measured surface motion and deconvolved rock motion are shown in Fig. 5.11.

5.3.3 Soil column and dynamic soil properties

For the soil site (SE6, which is close to EGTS), numerous $V_S$ profiles from tests including SASW, ReMi, crosshole and downhole, are available. These datasets have been collected over several years by the EGL and others. The deepest profile is about 200 m. Geotechnical borehole logs to 30 m deep are also available for this site. The $V_S$ profile for
the first 30 meters was interpreted by considering both $V_s$ measurements and borehole logs. Below 30 m depth, a background $V_s$ profile developed by Luke et al. (2002) for the Las Vegas basin, based on all available $V_s$ measurements and published literature, was used. The background $V_s$ profile can be found in Appendix 2. The final profile used in this study is 400 m deep, consisting of 39 layers, generally increasing in thickness with depth, from 0.30 m to 50 m (Fig. 5.12).

Since the sediment column is deep, it is essential to take into consideration the effect of confining pressure on small-strain dynamic soil properties in the 1-D analyses. For a given shear strain, an increase in effective confining pressure causes modulus ($G$) to increase and damping ratio ($D$) to decrease; and the elastic thresholds of both shift to higher strains (Stokoe and Darendeli, 2001). Since the shear strains in this study are small, the small-strain limits of $G$ and $D$ are of greatest importance. Lacking site-specific data, Stokoe and Darendeli's (2001) depth-adjusted dynamic material properties for soils with PI=0 for sandy deposits and PI=15 for clayey deposits were used. Their $G$ and $D$ curves were derived from comprehensive laboratory studies at strains as low as 0.00001 percent (Fig. 5.13). As mentioned previously, the maximum pressure reported in Stokoe and Darendeli's (2001) dataset corresponds to a maximum depth of 200 m. For the halfspace, rock and cemented material, Schnabel's values of $G$ and $D$ for rock (Schnabel, 1973) are adopted.

5.3.4 Fixing depth to model halfspace

As discussed in Chapter 2, for seismic response analysis of deep soil deposits, researchers are facing two impediments: 1) the lack of dynamic soil properties adjusted for confining pressure at great depths, and 2) the ambiguity in defining the soil-bedrock
interface. Also mentioned previously, three approaches have been practiced by other researchers with regard to placing depth to model halfspace: 1) placing the halfspace at the actual soil-bedrock interface, based on the researcher’s best understanding of the study site, 2) placing the halfspace at the engineering bedrock interface, based on a threshold $V_s$ of 760 m/s, and 3) placing the model halfspace within the soil column by matching projected $PGA$ with the expected values. In the following, all three criteria were tested and a refined procedure was developed.

5.3.4.1 Placing halfspace at the estimated soil-bedrock interface

When characterizing a deep sediment column, inappropriate use of dynamic material properties that were developed for lower confining pressures yields erroneous results. This is illustrated by the following example case. According to Langenheim (2001), the basin depth at the soil site (SE6) is about 1 km. Applying the SHAKE analysis to the parameterization described above, for depth to model halfspace ($H$) equal to 1 km, the projected motion from the BA event is excessively attenuated and shifted to shorter periods (Fig. 5.14). In general, appropriately characterized, depth-adjusted $G$ and $D$ data are key to projecting credible surface motions. Unfortunately, as discussed previously, our understanding of these parameters is restricted to approximately 200 m. Stokoe and Darendeli (2001) discourage extrapolation of $G$ and $D$ to greater confining pressures. For the Las Vegas basin, at up to 5 km deep, it is not feasible to obtain depth-dependent knowledge of the dynamic material properties that is adequate to properly characterize the entire sediment column. Therefore, it is not appropriate to place model halfspace at the estimated soil-bedrock interface for the study site.
5.3.4.2 Placing halfspace following engineering bedrock criterion

At the soil site, the ReMi measurement indicates a sharp jump in $V_s$ from 450 to 1340 m/s, at 51 m depth (Fig. 5.12). Therefore, the engineering bedrock criterion is satisfied at this depth. The $H$ is set at 51 m, and $V_s$ of the halfspace is set at 1340 m/s. Using the input data and parameterization described above, application of the SHAKE analysis for the BA event yields the results shown in Fig. 5.14. The $S_{a(max)}$ and the $PGA$ of the projected motion matches that of the measured. However, with respect to the measured, the $T_p$ of the projected motion shifts from 0.50 to 0.33 sec, and therefore the damage patterns from the projected motion would be very different from that of the measured motion. Therefore, the engineering bedrock approach is also inappropriate for the study site, and an alternative approach is needed.

5.3.4.3 Placing model halfspace within sediment column by matching PGA

As a part of the current study, Skidmore et al. (2003) applied the criterion of placing model halfspace within the sediment column by matching $PGA$ of the projected motion to the measured. Four potential halfspace depths between 60 and 480 m were investigated for the LVW1 site (Fig. 2.1). It was found that $H$ equal to 250 m yielded an acceptable solution, which was preferable to the results corresponding to the engineering bedrock criterion.

The $PGA$ is an important parameter to quantify ground motions. Ground motions having higher $PGA$ tend to be more destructive than those having lower $PGA$ (Kramer 1996). However, the $PGA$ does not provide information on frequency content. The same $PGA$ occurring at different frequencies will cause different damage patterns. Considering the BA event recorded at the study site and the parameterizations described above,
SHAKE analyses revealed that several depths to halfspace yield PGA projections that match the measured PGA (Fig. 5.15). The relationship between halfspace depth and its corresponding PGA has a sinusoidal characteristic. As $H$ increases, because of increasing influence of material damping, the sinusoidal effect tapers off. Although the PGA remains constant for larger values of $H$, the $T_p$ of the projected motions for different $H$ will be different because the changes in profile depth cause overall site stiffness to change.

To avoid ambiguity, the criterion for selection of $H$ should address both amplitude and frequency content. Here, a refined criterion is proposed and tested: the optimum depth to model halfspace corresponds to the one that yields the best matches of the acceleration spectrum intensity ($ASI$) over a target period range (Thun et al. 1988), and $T_p$, with PGA and $S_{a(max)}$ given secondary consideration.

Recall that the soil site amplifies motions over the range 0.3 to 5 seconds. It is suspected that the 1-D effect is not likely to explain the long-period amplification. Therefore, only that portion of the response spectrum whose $S_a$ is greater than the PGA was considered. For the four events studied, this cutoff occurs at 1.3 sec (Fig. 5.7). Thus, the target period range is set to be 0.3 to 1.3 sec. The coherence study indicated that the recorded near-rock and soil motions are well related between period 0.2 and 1 sec, which is close to the target period range.

The target range was further subdivided into smaller sub-ranges and the $S_a$ for each sub-range was integrated to determine:

$$ASI = \int_m^n S_a(T) dT$$  \hspace{1cm} (1)

where $m$ and $n$ are the target period sub-range bounds, and $T$ is the period. To
facilitate comparisons, the normalized deviation ($\delta$) between measured and projected spectral intensity for each sub-range was computed:

$$\delta = \frac{ASI_{\text{projected}} - ASI_{\text{measured}}}{ASI_{\text{measured}}}$$  \hspace{1cm} (2)

This procedure is illustrated using the BA event. The halfspace $V_s (V_{S(H)})$ was assigned at 2600 m/s, based on refraction data (Snelson et al. 2003) and a regional layered crustal model (Su et al. 1998). Halfspace depths from 50 to 500 m were tested, in increments of 25 m (Fig. 5.16). Changes in model halfspace depths cause changes in the site fundamental period, which are reflected in the projected motion by changes in $T_p$. To select the appropriate $H$, first, the projections that best match the $T_p$ of the measured motions were selected.

Out of the 18 different values of $H$ tested, only the $T_p$ of the projected motions for $H$ equal to 100 and 375 m match that of the measured motion, so the $ASI$ and $\delta$ are computed for these two cases over five period sub-ranges (Fig. 5.17). For the sub-ranges 0.3 to 0.5 sec and 1.1 to 1.3 sec, the model with $H = 375$ m yields better projections. For one of the intermediate period ranges, the model with $H = 100$ m yields a better projection. For the other two intermediate period ranges, $\delta$ approaches or exceeds 50%, implying that the amplification can not be explained by the 1-D model. Next, the $S_{a(max)}$ was examined. The $S_{a(max)}$ for the model with $H = 375$ m matches the measured value quite well, whereas for the model with $H = 100$ m, it is over-predicted by a factor of 1.3 (Fig. 5.18). Lastly, the $PGA$ was considered. The $PGA$ from the engineering bedrock criterion matches the measured closely, whereas, the $PGA$ for the other two cases are about the same and deviate from the measured value by 20%. Considering all four
factors, the appropriate $H$ for this case is determined to be 375 m.

As demonstrated above, three model halfspace depths, namely, $H = 375$ m, 100 m and 51 m, merit further investigation. Therefore, the same procedure was followed to compute response spectra using data from the other three nuclear test events, CO, BO and GA, for the three halfspace depths. Results are provided in Fig. 5.17 b, c, d, and Fig. 5.19.

First, the matches of $\Delta S_I$ between the projected and measured were examined. Considering all four events, for every sub-range tested except for one each in the GA and BA events, the solution for $H = 375$ m yielded the smallest deviation. As period increases, the quality of $\Delta S_I$-match decreases.

Next, the $T_p$ was examined. For the BO and GA events, the response spectra have multiple peaks. In this situation, the spectral concentration of energy can not be adequately measured by $T_p$ alone. A more representative parameter such as “mean period” (Rathje et al. 1998) might be considered. Here, the periods corresponding to each peak were examined. For both events, the solution for $H = 375$ m resulted in best period match. For the CO event, the solution for $H = 51$ m yielded another peak at 0.3 to 0.4 sec, implying an erroneous energy shift to shorter period with respect to the measured. To some extent, for the engineering bedrock criterion, this energy shift to shorter period is noticeable for all four events. This phenomenon is not surprising: SHAKE amplifies the first and higher modes of the soil column, and for $H = 51$ m, the fundamental period of the soil column is 0.3 sec. Therefore, motions at periods greater than that value can not be significantly amplified by SHAKE.

Finally, the $S_{a(\text{max})}$ and $PGA$ are examined. Regarding $S_{a(\text{max})}$, the solution for the
engineering bedrock criterion generally under-predicted accelerations over the period range of interest. The solution for $H = 375$ m yielded the best match for three events, whereas the solution for $H = 100$ m noticeably over-predicted it for three events. Regarding $PGA$, the match for engineering bedrock criterion varied in quality among events, the solution for $H = 100$ m yielded best for all four events, and the match for $H = 375$ m was of intermediate quality. Considering both primary criteria and both secondary criteria, the check on response projections for the other three events supports the finding that 375 m is an appropriate depth to model halfspace.

For all four events, considering the solution for $H = 375$ m, ground response was modeled best over the period range 0.3 to 0.5 sec and is considered to be modeled adequately in the period range 0.3 to 1.3 sec. The 1-D model was not able to capture amplifications occurring at periods longer than 1.3 seconds. This result is not surprising considering the fact that the coherences between the input near-rock and measured soil motions are poor beyond the range of 0.2 to 1 sec. It is believed that the long-period amplifications result from two- and three-dimensional basin effects (e.g., Chang, 1996) which are not modeled here.

5.4 Key simplifying assumptions

This analysis involved two key simplifications that bear further discussion: 1) All models have the same $V_{S(H)}$ of 2,600 m/s, and 2) for a given event, regardless of $H$, the input motion remains the same. It would be preferable to use depth-dependent $V_{S(H)}$ and to use input motions developed by matching the stiffness of the halfspace at both soil and rock sites. However, as would be true for most if not all deep soil deposits, it is difficult
to obtain sufficiently deep, detailed $V_s$ profiles at both rock and soil sites. The consequence of these simplifying assumptions was explored by considering the effect of a lower value of $V_{S(H)}$, 1,416 m/s, on the $H = 375$ m model. This is the velocity of the layer directly above the soil model halfspace, and it is close to both the $V_{S(H)}$ of the engineering bedrock model (1340 m/s) and the model used to deconvolve input motion (1040 m/s). Results for the four events are shown in Fig. 5.20. For this sample case, the decrease in $V_{S(H)}$ resulted in a slightly poorer match to $S_a$ in all three cases, but no significant period shift. Thus, the optimum $H$ for the study site is not likely to be changed much by a slightly different $V_{S(H)}$. However, it is recommended that if credible deep $V_s$ profiles are available for both soil site and rock site, the case with depth-dependent $V_{S(H)}$ should be tested and the halfspace stiffness at the rock and soil sites should be matched.

5.5 Discussion: Minimization of P-wave energy in input motion

As mentioned previously, compared to earthquake ground motions, ground motions generated by high-energy explosion contain a larger proportion of high-frequency P-wave energy. At a rock site, this P-wave energy would be well preserved, whereas at a soil site, it will be attenuated to some degree (Fig. 5.2). Since shallow sediments typically amplify vertically-propagating horizontally-polarized waves, if the high-frequency P-wave energy is included in the input motion, instead of being attenuated, it will be falsely amplified through the sediment column. This phenomenon is illustrated for the BA event with depth to halfspace of 375 m, using parameters set out earlier in this chapter. The input motion was developed by deconvolving the entire time history of the rock motion, including the P-component. Figure 5.21 shows the time...
histories and response spectra of input motion, projected surface motion and measured surface motion. The sediment column erroneously amplified the high-frequency P-wave energy preserved from the rock site. As a result, the PGA and $S_{a(max)}$ were over-predicted, and the energy was inappropriately shifted to shorter periods. Therefore, to obtain reasonable ground motion projection, effort should be made to minimize the compression energy from the input motion. As discussed previously, this can be accomplished by using only the portion of the time history that begins shortly before the onset of the S-energy.

5.6 Summary

- Small-strain ground motions recorded at the paired near-rock and soil site in the Las Vegas basin during underground nuclear testing indicated that with respect to the rock site, the soil site amplifies ground motions over the period range 0.3 to 5 sec.
- For the 1-D analysis, to obtain credible projections, the high-frequency P-wave energy preserved at the rock site should be excluded.
- For the study site, a deep soil deposit with complex $V_S$ profile at shallow depths, lacking confining-pressure-adjusted $G(\gamma)$ and $D(\gamma)$ at great depths, the two commonly-used criteria for placing model halfspace, true bedrock interface and engineering bedrock interface, yielded unfavorable ground motion projections.
- To compensate for unknowns, namely, $V_S$ and confining-pressure-adjusted $G(\gamma)$ and $D(\gamma)$ at great depths and location of the true soil-bedrock interface, a refined criterion is proposed, which identifies the optimum halfspace depth through
iterative assessment of model halfspace depths to select the one whose projected response spectrum best matches measured or expected spectra.

- Using the refined criterion, the optimum depth to model halfspace for the study site is determined to be 375 m.

- The measured near-rock and soil motion are best related in the period range 0.2 to 1 second. For the study site, ground response was modeled best over the period range 0.3 to 0.5 sec and modeled adequately in the period range 0.2 to 1.0 sec. This period range covers predominant periods of most of the residential and commercial structures (2 to 10 stories).

- This proposed procedure cannot model long period motion (>1.0 sec), which particularly affects the design and performance assessment of high rise structures (typically > 10 stories).
Figure 5.1 Coherence between measured “near-rock” and soil motions
Figure 5.2A Comparison of explosion and earthquake ground motions recorded at near-rock site and soil site during the BA event and the 1992 LSM earthquake. Vertical line indicates onset of shear waves.

Figure 5.2B Comparison of response spectra of N-S and E-W components, four nuclear events, SGS motions. Blue solid: E-W components; Red dotted: N-S motions.
Figure 5.3  Acceleration time histories of measured soil and rock motions, for four underground nuclear test events. First arrival picks of S-wave energy are indicated (from Liu and Luke (2004)).
Figure 5.4 Comparison of Fourier spectra of a distilled time history before and after down-sampling (rock site, BA event).
Figure 5.6 Example of application of windowing to start and end of a fifty-six second time history (soil site, BA motion). Dotted line: before windowing; solid line: after windowing.
Figure 5.7 Response spectra of measured rock (dashed) and soil (solid) motions from four tests, 5% damping (from Liu and Luke, 2004)

Figure 5.8 Averaged response spectral ratio (soil motion over rock motion) for four nuclear events monitored at soil site (from Liu and Luke, 2004)
Figure 5.9   Applying low-pass, high-pass and band-pass filters to the input motion

(rock site, BA event)
Figure 5.10  Shear wave velocity profiles at rock site (from Liu and Luke, 2004)
Figure 5.11  Deconvolution of surface motion for the rock site (BA event) (from Liu and Luke, 2004)
Figure 5.12 $V_S$ measurements and simplified borehole log of the soil site (SE6), with shear wave velocity profile interpreted for SHAKE analyses for the 375-m halfspace depth. Second plot shows expanded detail of shallow layers (from Liu and Luke, 2004).
Figure 5.13 Shear modulus reduction and damping ratios for soils with PI=0 and PI=15, for effective mean stress equal to 0.25, 1, 4 and 16 atm; and for rock; Soil data is from Stokoe and Darendeli (2001) and is confining-pressure adjusted. Rock data is from Schnabel (1972) and is not confining-pressure adjusted.
Figure 5.14 Response spectra resulting from placement of halfspace according to true bedrock ($H=1$ km) and engineering bedrock ($H=51$ m) criteria (BA event)

Figure 5.15 Peak ground acceleration with respect to halfspace depth (BA event)

Figure 5.16 Response spectra for halfspace study, BA event: summary of projections for all H studied (from Liu and Luke, 2004)
Figure 5.17: Deviation in ASI of different halfspace depths and criteria; dotted line, dashed line and solid line are for engineering bedrock criterion, \( H = 100 \) m and \( H = 375 \) m respectively (from Liu and Luke (2004)).
Figure 5.18 Isolating spectra with matching $T_p$ and comparing two criteria for selection of $H$ (BA event) (from Liu and Luke (2004))
Figure 5.19 Response spectra for CO, BA and GA events (from Liu and Luke, 2004)
Figure 5.20 Impact of halfspace shear wave velocity, $V_s(H)$, on projected surface motion, for halfspace depth 375 m (from Liu and Luke, 2004)
Figure 5.21 Effects of compression energy on seismic response analyses. Time histories and response spectra of input, projected and measured motions (BA event, H = 345 m). First arrival picks of shear wave energy are indicated.
CHAPTER 6

SITE PARAMETERIZATION VALIDATION: APPLICABILITY TO THE ENTIRE BASIN

One main objective of this research is to investigate the geographic distribution of the intensity of surface shaking in the Las Vegas basin, and construct zone-specific bounding response spectra for the basin considering multiple seismic sources. To do this, an appropriate depth to model halfspace for the entire basin should be established. Compared to determining halfspace depth for a single site, in the process of validating model halfspace depth for the entire basin, factors such as differences in source-to-site distances and spatial variations of site stiffnesses should be taken into consideration. As discussed previously, these issues can be addressed by conducting a Monte-Carlo simulation. In this chapter, the model halfspace depth for the entire basin is established.

6.1 Defining site response units

In Chapter 4, from the paired near-rock and soil site study, it is concluded that to obtain a reasonable surface response, the halfspace should be placed at about 375 m depth. For general purposes of seismic response studies in the valley, considering the uncertainties involved, the use of 400 m as the optimum depth to model halfspace is proposed. In the following, the applicability of this proposed halfspace depth for the
whole basin under both high-energy explosion and earthquake events is tested using Monte-Carlo simulation.

As discussed in chapter 3, the basin sediments exhibit geographical distribution patterns, namely, the fine-grained clay-rich deposits occupy the central and east part of the basin, whereas the coarse grained- and mixed- grain size deposits occupy the west part of the basin (Fig. 4.4). The observed site amplification in the basin appears to be directly related to these predominant sediment types averaged over the upper 30 meters. Figure 4.4 also shows that the predominantly cemented media occupy the southwest and northeast parts of the basin. Since the cemented media were interpreted using a sparsely populated dataset, however, the credibility of the dominance of this lithologic category is low. Further, as discussed in chapter 3, because of the variability of the logging, the degree of cementation could not be distinguished, and the cemented material does not show a distinguishable higher velocity comparing to the coarse- and mixed- grained deposits. For these reasons, for the purpose of seismic microzonation, the cemented material will not be treated differently. It is grouped together with the coarse- and mixed-grain size deposits and simply referred to as “coarse sediment.” And the fine grained clay rich deposit is simply referred as “fine sediment.”

The SGS site recorded the smallest ground motion during all the historical seismic events. For the CALB site, because it was about 20% closer to the seismic sources during historical events, it did not show considerable deamplification with respect to the shallow basin sites. The two near-rock sites SGS and CALB are the reference sites for this study.
The fine sediment is associated with low $V_s$, and thus has greater capability to amplify ground motions. Conversely, the coarse sediment is associated with high $V_s$, and thus has lower capability to amplify ground motions. So, for the purpose of seismic microzonation, the fine sediment is more problematic and is of primary concern. These two categories form the basic site response units for the basin.

6.2 Developing characteristic $V_s$ profiles for each response unit

As discussed previously, different $V_s$ measurements exist in the valley, including crosshole, downhole, and surface-wave-based measurements. Comparing to the point measurements, namely, downhole and crosshole measurements, the surface-wave-based measurement yields a laterally-averaged $V_s$ profile. Because local velocity anomalies caused by small irregularities are less likely to influence surface response, the laterally-averaged $V_s$ profile is preferred for seismic response analyses. As mentioned previously, for the purpose of seismic microzonation, the $V_s$ profiles resolved from combined usage of active- and passive-source surface wave measurements are preferred because they provide detailed $V_s$ variation at shallow depths and also extend the profiles to greater depths. The Las Vegas transect datasets, generated using the ReMi method, were not used in this study because the $V_s$ profiles have only simple layer geometry and don’t show detailed $V_s$ variations. A $V_s$ profile of 100 – 300 m deep from ReMi measurement typically consists of three or four layers. If they were to be included in the analyses, the detailed information from the active- source measurements would tend to be averaged out. In this study, the $V_s$ profiles from SASW measurements and the combined SASW+ReMi measurements are used.
Sites are grouped by their response unit. The fine sediment response unit, which occupies the deep and intermediate-depth parts of the basin, includes the following sites: CSN, DOE, EFL, EGT, LRE, LVS, MNL, NGC and WHT. The coarse sediment response unit, which occupies the shallow parts of the basin, has the following sites: ANN, BRC, GRP and TRD. The sites CLB and SGS are near-rock sites; thus they are not included in either of the sediment units. To be noticed is that $V_S$ sites CSN, LVS, LRE and EGT are located in the interfingered zones that define the clay-rich deposit boundaries (Fig. 4.5). Referring to Fig. 3.12, the $V_S$ profiles from SASW+ReMi measurements, for the fine sediment response unit, and the pre-existing $V_S$ profiles, which are dominated by the ReMi transect data, have the same trend: 1) lower velocities compared to those of the shallow part of the basin, and 2) velocities increasing gradually with depth.

Figure 6.1 shows all the 13 $V_S$ profiles used in the site response study. As a recap, considering only the portions of the $V_S$ profiles that are below 50 meters, the overall velocities of fine sediments are consistently lower than those of the coarse sediments. For shallower depths, the differences are not as distinct, indicating the overriding influence of features less dependent on grain size such as cementation, weathering, and lack of confinement.

Marosi and Hiltunen (2004) performed an uncertainty assessment for the SASW measurement using a large sample of test data from two sites. They found that the phase velocity data are normally distributed. Because the primary testing methods used in this research are SASW and the combined SASW+ReMi methods, the $V_S$ for each response unit in the valley is assumed to be normally distributed. The characteristic $V_S$ profile for
each response unit is expressed as a mean value (μ) and standard deviation (σ). To form the characteristic $V_S$ profiles for each response unit, two factors were considered: 1) $V_S$ resolution generally decreases as depth increases; and 2) sharp $V_S$ contrasts define layer boundaries. As a result, the characteristic $V_S$ profiles consisted of 30 layers, with thicknesses increasing with depth and layer boundary placement mildly influenced by the observed data (Fig. 6.2). The mean velocity was calculated using layer-thickness-weighted data for each depth range. For the fine sediment response unit, the standard deviation was calculated for layers where at least five $V_S$ measurements exist. For the deep layers, where less than five measurements are available, the standard deviation for the lowest layer having at least five $V_S$ measurements was assigned. For the coarse sediment response unit, having only 4 datasets to average, the same procedure to set standard deviations was used, but with the minimum of three $V_S$ profiles. The mean and statistical range of $V_S$ for the two response units are illustrated in Fig. 6.2. In general, the standard deviation of the coarse sediment unit is about 1.5 times larger than that of the fine sediment response unit.

6.3 Suitability of the selected optimum depth to halfspace

As mentioned previously, geographically, the two near-rock sites at the east and west edges of the Las Vegas basin, SGS and CALB, respectively, bracket most of the soil sites in the basin. Ideally, if only source-to-site geometric attenuation is considered, and assuming the underlying geology is the same, the bedrock motions at these two sites will bracket the bedrock motions at the soil sites in the basin. Therefore, if the selected optimum depth to halfspace is suitable for the entire basin, ground motions projected
using the motions recorded at these two near-rock sites as input motions should envelop the surface response in the basin. In the following, Monte-Carlo simulations were conducted to test the suitability of the optimum depth to model halfspace of the rock and soil sites, for the rest of the basin. The random variable is $V_S$, defined for each of the two site response units by a characteristic mean value and its standard deviation, calculated from the $V_S$ data. The layer geometry and $V_S$ of the halfspace are not varied. For each layer, a random $V_S$ is generated according to normal distribution. Any parameterizations yielding negative $V_S$ were discarded and reselected. For dynamic material properties, the same parameterization as that for the paired rock and soil site study discussed in chapter 5 was used. For each simulation, 10,000 iterations were performed. To ensure that the number of iterations was adequate, a duplicate analysis for 20,000 iterations was conducted for two test cases. One example is given in Fig. 6.3. No differences are visible between the two sets of iterations. For each period step (a total of 128 steps from 0.06 to 5 seconds), the mean and standard deviation of the $S_a$ were calculated. The bounding spectral ordinates were defined as $\mu \pm 2\sigma$ or $\mu \pm \sigma$, as appropriate, of the projected $S_a$; in this way, ninety-five or eighty-four percent, respectively, of the possible surface response was represented. The detailed parameterization and model inputs are discussed in the following. The Matlab scripts used for Monte-Carlo simulation are documented in Appendix 3.

6.3.1 Model Inputs

The key model inputs for the Monte-Carlo simulation are the same as those of the site-specific analyses discussed in Chapter 5 except as stated here. For the coarse sediment response unit, dynamic soil properties developed for non-plastic (PI=0) soil was
used. For the fine sediment response unit, dynamic soil properties developed for soil having PI=15 was used. For the fine sediment, as discussed in chapter 4, $V_S$ of the basement rock, 2600 m/s, was assigned to the halfspace. For the coarse sediment, the mean $V_S$ exceeds 2600 m/s at 310 meters depth, therefore, dynamic properties of rock (Schnabel et al., 1972) were assigned for that depth and below, and the mean $V_S$ at 400 m depth, 3200 m/s, was used as the $V_{S(H)}$. The seven available near-rock motions recorded at CALB and SGS during the four nuclear events studied (the CALB motion during the BO event is not available) as well as the near-rock motions recorded at both sites during the LSM earthquake were used as input motions.

6.3.2 Bounding Spectra

6.3.2.1 Nuclear explosion events

Figure 6.4 shows histograms of $S_a$ from the Monte-Carlo simulations for $T=0.5$ and 2 sec, for a randomly selected example case. As expected, since the site response is more sensitive to low velocities, which produce higher $S_a$, and because the velocity sampling was biased by discarding non-negative values, the projected $S_a$ appears to be lognormally distributed. At longer periods, generally greater than 2 sec, the variation in velocities has less influence on the surface response. Ninety-five percentile response envelopes were considered for both response units. The results are summarized and compared to measured motions in Figures 6.5 and 6.6. As expected, when the SGS dataset is used as input motion, the projected motion tends to under-predict the surface responses, whereas when the CALB dataset is used as the input motion, the projections tend to over-predict the surface response. It is proposed that the median value between the two upper-bound response spectra formed by using SGS and CALB as input motion,
also shown in Figs. 6.5 and 6.6, be taken to represent the upper-bound acceleration spectrum for the corresponding response unit for similar events.

For the fine sediment, with respect to the real data, the bounding spectra closely matched the measured PGA and peak $S_a$. However, they over-predicted the short period response (less than 0.5 sec). This finding conforms to Siddharthan’s (2004) observation, mentioned earlier, that SHAKE yields consistently conservative results. The bounding spectra under-predicted the long-period response (greater than 1.0 sec). It is suspected that this occurs because the long-period response is dominated by surface wave energy and scattered body wave energy, which SHAKE does not model.

For the coarse sediment, with respect to the real data, the surface responses are over-predicted. Because only four $V_s$ profiles are available for this response unit, and two of them are shallow (only about 50 m deep), it is suspected that the average $V_s$ profile constructed from this small number of measurements does not represent the true mean. Therefore, the $S_a$ projections are not as credible as those for the fine sediment response unit. More $V_s$ measurements would be needed for this response unit to obtain high-quality projections. On the other hand, since the coarse sediment is stiff, its capability of amplifying ground motions is low. So, it is of lower priority to study the seismic response of this unit.

As discussed previously, the standard deviation of the average $V_s$ profile for the coarse sediment response unit is about 1.5 times larger than that for the fine sediment unit, and the 95th percentile response envelope for the coarse sediment unit tends to over-estimate the surface motion. Thus, a lower response envelope, namely, the 84th percentile envelope, is selected for the coarse sediment response unit.
Figure 6.7 contains a summary of the mean spectral accelerations from all seven pairs of Monte-Carlo simulations. In comparison to the coarse sediment unit, the fine sediment unit has a slightly higher $S_{a(max)}$ and longer $T_p$. The trends are the same but not as obvious for the simulations as they are for the observed motions. As mentioned previously, it is suspected that this happens because the $V_s$ dataset of the coarse sediment response unit is too small; as a result, the standard deviations for the coarse sediment unit are larger than those of the fine sediment unit (Fig. 6.2). And the characteristic $V_s$ profile for the coarse sediment unit is not as representative compared to that of the fine sediment unit. Given a more statistically representative database, it is suspected that the comparative trends would better track those observed for direct observation.

6.3.2.2 LSM earthquake

Following the same procedure described above, using deconvolved CALB and SGS motion recorded during LSM earthquake, the bounding response spectra were constructed for both fine sediment and coarse sediment response units.

Figure 6.8 shows the 95th and 84th percentile bounding spectra for the fine sediment response unit, using deconvolved CALB motion as input. As discussed previously, the CALB site is much closer to the source than the other sites; as a result, the projected surface motion is overestimated with respect to the real data.

Figure 6.9 shows the same information as that of Fig. 6.8 except that the deconvolved SGS motion was used as input. Comparing the 95th and 84th percentile envelopes, with respect to the measured, the 84th percentile envelope yields a better match of $PGA$ and $S_{a(max)}$, but it can not envelop long period motions (<0.5 sec). On the other hand, the 95th percentile envelope bracketed surface motions almost up to 1.3 sec.
The coverage of the ground motion across the valley during the LSM earthquake was sparse, only 7 stations. They were not likely to record the highest motion in the valley. So, the 95\textsuperscript{th} percentile presents a more realistic response envelope for the entire basin.

Figure 6.10 shows the histograms of projected $S_a$ for periods 0.5 and 2 seconds. The distribution pattern is similar to that observed for the nuclear explosions, namely, the influence of variation of $V_s$ on surface response decreases as the period increases.

Figure 6.11 is the 84th percentile bounding spectra for the coarse sediment response unit. Only one measured motion was available for this response unit. As with the nuclear testing events, with respect to the measured, the projected surface motion was over-estimated.

\subsection*{6.4 Summary}

The Monte-Carlo simulation validates that the site parameterization obtained from the paired near-rock and soil sites study, generalized to 400 m depth to half-space, tested for both high-energy explosion events and an earthquake event, is suitable for the basin.

By using the proposed site parameterization, for the fine sediment unit, the 95th percentile response envelope generated by Monte-Carlo simulation generally brackets measured site responses for periods up to 1.3 seconds.

For the coarse sediment unit, since there were only four measured $V_s$ profiles available, and two of them are shallow, the standard deviation is large, and the surface response envelope tends to be over-estimated. Since this response unit has high $V_s$, the associated ground motion amplification hazard is low. The measured amplification factor
was 1.5 in average for both $PGA$ and peak $S_a$. Therefore, it is of secondary importance to study site response for this part of the basin.

From the Monte-Carlo simulation, for earthquake hazard mitigation and city development planning purposes, it is recommended that for the fine sediment response unit, the 95th percentile response envelope should be used; for the coarse sediment response unit, the 84 percentile response envelope should be used.

Through Monte-Carlo simulation, it is confirmed that the $V_s$ variation particularly affects short period response, generally less than 2 seconds for the study basin.
Figure 6.1 Summary of $V_s$ data used in this study. Green: fine sediment response unit; Blue: coarse sediment response unit.
Figure 6.2  Characteristic $V_S$ profiles with 95% confidence interval (shaded area): green lines: measured $V_S$; black lines: mean $V_S$. Part a.) fine sediment response unit; part b.) coarse sediment response unit.
Figure 6.3  Monte-Carlo simulations: Response spectra from 10,000 iterations (thick lines) and 20,000 iterations (thin lines); BA event; SGS as input motion; mean and 95% confidence intervals.
Figure 6.4 Example histograms of $S_a$ from Monte-Carlo simulation; CO event; deconvolved SGS as input motion
Figure 6.5 Bounding spectra generated by Monte-Carlo simulation for the fine sediment response unit, using SGS and CALB as input motion. Measured legacy data from the same response unit (green) and mean spectra (blue) are shown.
Figure 6.6 Bounding spectra generated by Monte-Carlo simulation for the coarse sediment response unit, using SGS and CALB as input motion. Measured legacy data from the same response unit (green) and mean spectra (blue) are shown.
Figure 6.7 Comparison of mean $S_a$ from all seven Monte-Carlo simulations. Blue: coarse sediment response unit; Green: fine sediment response unit.
Figure 6.8  Bounding spectra for fine sediment response unit, using deconvolved CLB motion recorded during LSM earthquake as input motion: blue solid line: mean and 95th or 84th percentile lines; green dotted line: measured.
Figure 6.9 Bounding spectra for fine sediment response unit, using deconvolved SGS motion recorded during LSM earthquake as input motion: blue solid line: mean and 95th or 84th percentile lines; green dotted line: measured.
Figure 6.10 Histograms of $Sa$ from Monte-Carlo simulation; LSM earthquake; deconvolved SGS as input motion

Figure 6.11 Bounding spectra (84th percentile) for the coarse sediment response unit using deconvolved SGS motion from LSM earthquake as input motion. Solid lines: mean and 84th percentile; dotted line: measured.
CHAPTER 7

GROUND MOTION HAZARD PROJECTIONS

As discussed previously, during historical high-energy explosion events and earthquakes, ground motions recorded in the Las Vegas basin exhibited great geographic variations. Motions recorded on the fine-grained clay-rich deposits in the central and eastern part of the basin are much larger than those recorded on the coarse-grained and mixed-grain size deposits in the shallower western part of the basin. Due to its large population, rapid growth rate, unique building inventory, and the continuing disclosure of significant young faults, the seismic risk in the Las Vegas basin is high. For the purpose of future land use planning and earthquake hazard mitigation, it is beneficial to develop ground motion hazard microzonation maps for the Las Vegas basin. As mentioned previously, an earlier microzonation study for the Las Vegas basin (Murphy and Hewlett, 1975) was based on motions recorded during historical nuclear test events. No potential earthquake sources were considered and the local soil conditions were not taken into consideration. In this chapter, a comprehensive ground motion hazard microzonation study, which considers multiple potential earthquake sources, energy attenuation, and local soil effects, is conducted.
7.1 Procedures for Ground Motion Hazard Microzonation

As discussed in chapter 1, in this study, the following procedure for ground motion hazard microzonation is followed:

1) Compiling data on active faults in and around the Las Vegas basin. This includes literature searching, communicating with local scientists, internet searching and incorporating recent studies of local faults.

2) Applying attenuation laws to obtain the rock-level motions (input motions) in terms of $PGA$ for each zone. Scaling the measured motions from the LSM earthquake and using other strong motions that bear similar source type, depth, source-to-site distances and underlying geology as appropriate.

3) Defining response unit: The response units have been determined by intelligently interpreting the basin fill model and basin shallow $V_s$ model (chapter 5).

4) Constructing surface response envelope by Monte-Carlo simulation for each response unit.

This procedure in general is similar to that which has been used by Wong et al. (2002) in a seismic study for Salt Lake City, Utah, except for the following two aspects: 1) a different site response model and different procedure for dynamic soil property parameterization were adopted, as discussed in chapters 5 and 6, and 2) a surface response envelope for each response unit is generated by Monte-Carlo simulation.
7.2 Seismic Source Characterization

The first step of any earthquake ground motion hazard assessment is to characterize the seismic sources that have the potential of producing ground motions of engineering significance at the area of interest. As discussed in chapter 2, in this study, a multiple DSHA, which incorporates the worst-case scenarios for all the known active faults that have significant earthquake potential, is conducted. In the western U.S., active faults within a distance of 100 to 200 km of the study area are usually considered (Wong et al. 2002). To assess the potential seismic hazard at Yucca Mountain, Nevada, Stepp et al. (2001) considered all the faults within 100 km of the study site.

In this study, all 67 known Quaternary faults within 150 km of Las Vegas were compiled. The distances were measured from the midpoint of the fault rupture to a point in the center of Las Vegas (Table 7.1 a, b and c). Detailed references for these faults can be found at http://earthquake.usgs.gov/qfaults/nv/index.html. The version used for this study was last updated on Sep. 2004. It contains all the faults that are believed to be sources of M 6.0 or greater earthquakes the U.S. in the past 160,000 years.

The Basin and Range Province is characterized by extensional deformation. The Quaternary faults within 150 km of Las Vegas are principally normal. One exception is the Mead Slope fault which is characterized as a reverse and probably left-slip fault by the USGS and oblique-slip with significant reverse and possible left-lateral strike-slip movement by Anderson and O’Connell (1993). Out of the 67 faults, 30 of them (Table 7.1 b) have been compiled and reported by Piety (1996) and have been used by Stepp et al. (2001) and Kemnitz (1999). In a recent study of the design motions for Hoover Dam Baypass, Keaton (2004) identified the Mead Slope fault and the California Wash fault as
the faults controlling the design ground motion. The maximum magnitudes for these two faults were selected based on the work by Anderson and O’Connell (1993). For the 30 faults compiled by Piety (1996), the same magnitudes used by Kemnitz (1999) were used in this study. For the other 37 faults, the empirical relationship for normal faults developed by Wells and Coppersmith (1994) was used to determine the maximum moment magnitude (Table 7.1 a). This relationship is well known and has been widely used (e.g., Wong et al. (2002); Stepp et al. (2001); Kemnitz (1999)).

Evidence of the most recent prehistoric deformation is one of the most important factors for evaluating earthquake risks. According to USGS, faults are commonly considered to be active if they have moved one or more times in the last 10,000 years (http://earthquakes.usgs.gov/image_glossary). In this study, all the faults within 150 km of Las Vegas that moved within the past 15,000 years (Latest Quaternary) are considered to be having most significant earthquake potential. As a result, 11 faults were identified. One of these, the Boundary fault has short rupture length (5.6 km) and is far away (150 km) from Las Vegas, so it is excluded from this study. The 10 faults considered in this study are listed in Table 7.2.

7.3 Development of Rock-Level Acceleration Time Histories for the Las Vegas Valley

7.3.1 Attenuation relationships

As pointed out by Stepp et al. (2001) and Wong et al. (2002), due to the lack of strong motion records, no specific attenuation relationships are available for the Basin and Range region. Instead, the attenuation relationships developed for western North
America are often used. A complete literature review of the attenuation relationships for western North America was given by Kemnitz (1999).

Stepp et al. (2001) used the attenuation relationships developed by Boore et al. (1997) and Abrahamson and Silva (1997). Wong et al. (2002) used relationships developed by Abrahamson and Silva (1997), Spudich et al. (1999), Sadigh et al. (1997), and Campell and Bozorgnia (1994). In a study of seismic response of deep sandy soil deposits at the Nevada Test Site, Luke et al. (2001) used those of Sadigh et al. (1997) and Abrahamson and Silva (1997). The parameters that can be predicted with these relationships are listed in Table 7.3.

The relationship of Spudich et al. for normal faults may be the most appropriate attenuation relationship for the normal faults modeled in this study. However, as has been summarized by Kemnitz (1999), recent studies pointed out that the earthquake database used to form that attenuation relationship was sparse, and as a result, many of the attenuation model coefficients could not be well constrained. Therefore, this attenuation relationship is not used. In this study, the same attenuation relationships used by Luke et al. (2001) were selected. They are based on strong motion records primarily from California where strike-slip faults dominates. The Sadigh et al. (1997) relationship has the provision to distinguish between normal faults and strike-slip faults. The normal faulting data have not been found to be statistically different from those predicted for strike-slip faults (Westaway and Smith 1989).

One limitation of all the above-mentioned attenuation relationships is that they are limited to 100 km from the source. As some active faults considered in this study are more than 100 km from Las Vegas, an appropriate attenuation relationship with longer
source-to-site distances (extending to at least 150 km) should be found. After a good deal of searching, this problem was solved by finding an attenuation curve provided in SHAKE 2000 (Ordonez, 2000) that yields the best fit to the prediction of Sadigh et al. (1997) and Abrahamson and Silva (1997) and also extends to 200 km (Fig. 7.1).

7.3.2 Development of Target Spectral Envelope

To develop representative acceleration time histories for the area of interest, a target response spectrum envelope needs to be determined. Then, a suite of time histories (real or synthetic) is selected to fit in the target envelope. The time histories that provide the best fit to the target envelope are considered to be representative. The details of this methodology can be found in Idriss (1993).

As discussed by Doser and Smith (1985) and Smith and Bruhn (1984), the focal depth of main shocks in the Great Basin is commonly near 10 km. Thus, a focal depth of 10 km is assumed for all the earthquakes in this study. The target spectral envelopes were formed by projecting the mean and 84th percentile rock level motion (Fig. 7.2 a and b). For engineering purposes, the 84th percentile motion (mean plus one standard deviation) should account for most uncertainties in ground motion prediction (Reiter, 1990).

To select the representative time histories, the PEER Strong Motion Database (PSMD; http://peer.berkeley.edu/smcat/index.html) was searched. This database contains 1557 records from 143 earthquakes from tectonically active regions. The datasets were processed by Dr. Walt Silva of Pacific Engineering using publicly available data from Federal, State, and private providers of strong motion data. The key searching parameters are earthquake magnitudes, distance from the source, fault type, site conditions and PGA. A wide range of values was tried and the best fitted motions are shown here. This suite of
motions did not need to be scaled. Because earthquakes recorded by the Nevada Seismological Laboratory (NSL) carry some general path effects of the Basin and Range region, this database was also searched, and more earthquake motions were included as appropriate. Other strong motion databases searched were University of Utah Seismograph Stations and Caltech Seismological Laboratory, but no additional suitable time histories were found.

As discussed previously, the 1992 Little Skull Mountain (LSM) earthquake (M=5.6) was distinctly felt in the Las Vegas valley and triggered 10 strong motion stations including two near-rock locations. Because these records reflect the influences of path effects of the region, the amplitudes of the SGS motions were scaled up by factors from 4 to 40 and plotted against the target envelopes (Figure 7.3 a and b). The twenty selected time histories are documented in Table 7.4 and Figure 7.4.

7.4 Defining site response units and corresponding characteristic $V_s$ profiles

As has been discussed in detail in chapter 5, two site response units, namely, fine sediment and coarse sediment, were identified for the basin, based on predominant sediment type over the upper 30 meters and $V_s$ measurements. The characteristic $V_s$ profile for each response unit is expressed by its mean value and standard deviation, developed from existing $V_s$ measurements. The same profiles are used here for projecting surface response.
7.5 Projected surface responses

As discussed in chapter 5, for the fine sediment response unit, the 95\textsuperscript{th} percentile response envelope is constructed for each potential earthquake motion. For the coarse sediment response unit, the 84\textsuperscript{th} percentile response envelope is constructed.

During the Monte-Carlo simulations, it was observed that some random $V_S$ profiles generated using the approach described above caused the program SHAKE to crash. Fifty of the profiles that caused SHAKE to crash were examined. Through modifications of the $V_S$ parametric investigation by trial and error, it was found that the velocity contrast between adjacent layers was the root cause. Typically, a single layer in the profile having exceptionally low velocity with respect to its adjacent layers caused the program to crash. If this velocity contrast was manually reduced by increasing the velocity for one of the adjacent low-velocity layers, starting at a certain velocity contrast ratio, the profile could be made to be acceptable by SHAKE.

To address this problem, at first, the Monte-Carlo simulation code was modified so that the profile having a velocity contrast ratio greater than a maximum allowable value would be disregarded in the computation. The maximum allowable value was determined from examination of the profiles that caused SHAKE to crash. Through a series of tests, it was found that the maximum allowable ratio is as low as 1.5. If a pre-specified ratio of 1.5 is used, the computational time needed to generate one profile is very long (many minutes). Also, the ratio is considered to be too restrictive because true $V_S$ profiles could have a velocity contrast greater than 1.5. Therefore, this problem could not be solved satisfactorily by limiting the velocity contrast between adjacent layers. Also the problem was found to be further dependent on the input motion; namely, a profile that
caused the program to crash using one input motion will not necessarily cause the program to crash if different motion is used.

To solve this problem, a more flexible approach needed to be developed. It is observed that after the SHAKE program crashes, it returns to the Matlab environment and the computed spectral accelerations are shown as having negative values. Thus, a flag was set to examine the spectral acceleration so that the program will re-invoke SHAKE when a negative spectral acceleration is encountered. Thus, the simulation disregards the profiles that caused SHAKE to crash without further restriction of the velocity contrast. Only about 0.5% of $V_s$ profiles had to be disregarded in the simulation process.

7.6 Fine sediment response unit

For the fine sediment response unit, the surface response envelopes for each selected input time history are plotted in Fig. 7.5. The spectral distribution of the surface response is highly influenced by the input motions. For most of the cases, the energy is concentrated between $T=0.1$ to $1$ sec except for one each time history for the BH and CW faults (CHV, LAN1) where the concentration of energy extended to $T=3$ sec (Fig. 7.5). The 95th percentile values of $PGA$ and $S_{a(max)}$ for the fine sediment response unit using different seismic sources are plotted in Fig. 7.6. Out of the 10 selected faults, six, namely, MS, BH, CW, WSM, PRP and RV, are within 100 km from Las Vegas. The 95th percentile values of $PGA$ and $S_{a(max)}$ of these six faults are $308$ cm/s$^2$ (0.31 g) and 1131 cm/s$^2$ (1.15 g), respectively. According to a study conducted by Wald et al. (1999), a $PGA$ of $308$ cm/s$^2$ corresponds to a peak Modified Mercalli Intensity of VII, which implies strong shaking severity. Therefore, the potential seismic risk for the Las Vegas
valley imposed by these six faults is high. In contrast, the other four, namely WSR, YMW, DV and GRE, are located more than 100 km from Las Vegas. The maximum $PGA$ of these four faults is below $75 \text{ cm/s}^2$ (0.08 g), and the $Sa_{(max)}$ of these four faults is below $285 \text{ cm/s}^2$ (0.30 g). A $PGA$ of $75 \text{ cm/s}^2$ corresponds to Modified Mercalli Intensity of V, which implies light shaking severity. Therefore, the potential seismic risk for Las Vegas valley imposed by these four faults is low, and can safely be excluded from the seismic hazard study.

Figure 7.7 shows the overall surface response envelope for the fine sediment response unit. The spectral acceleration envelopes shown in Fig. 7.7 are superimposed and the upper and lower bounds are the maximum spectral acceleration at each period of the 95th percentile and the mean spectra, respectively. The IBC uses the probabilistic approach for the design earthquake motion, and, as discussed previously, this study used multiple DSHA to produce bounding response spectra. For the sake of comparison, the computed response envelope is compared with the deterministic design spectra from the UBC. Based on $V_{S(30)}$ calculated for individual sites located on the fine sediment response unit ($V_{S(30)} = 388, 364, 413, 420, 410, 375, 433, 610, 575, \text{ and } 315 \text{ m/s}$ for CSN, DOE, EFL, EGT, LVP, MNL, NGC, WHT, WLV, and 115, respectively), the site classes for the fine sediment response unit are between site class C (very dense soil and soft rock) and site class D (stiff soil profile); thus, the UBC design spectra for these site classes are shown. The MS fault, which is the nearest one studied and is capable of a M 6.75 earthquake, is used as the source to calculate the UBC spectra. The other parameters used are seismic zone 2B and fault type B (UBC 1997, Table 16-A-U), which are appropriate for the faults in Las Vegas according to the 1997 UBC. Compared to the UBC spectra,
the $S_{a(\text{max})}$ of the overall response envelope for this study is about 1.5 times higher. Also, the overall response spectrum is broader, with significant response up to 3 sec. However, the UBC spectra predict a higher $PGA$.

The projected upper bound $PGA$ was compared with the U.S. Geological Survey (USGS) probabilistic seismic hazard maps (PSH) (http://earthquake.usgs.gov/hazmaps/products-data/1996/canvmap.html, accessed March 16, 2006). Considering the case 1% to 10% probability of exceedance in 50 years for NEHRP B-C boundary sites, the USGS PSH map projected $PGA$ of 0.1 to 0.31 g for the Las Vegas area, which is very close to the $PGA$ ranges projected in this study, namely, 0.05g - 0.31g. This match to a very low probability of occurrence is a favorable outcome for a deterministic analysis.

7.7 Coarse sediment response unit

Similar to that of the fine sediment response unit, the spectral distribution of the surface response is highly influenced by the input motions. The potential seismic risk from the same four faults that are located more than 100 km away from Las Vegas, as identified above, is low, with maximum $PGA$ and $S_{a(\text{max})}$ of 69 cm/s$^2$ (0.07 g) and 304 cm/s$^2$ (0.31 g), respectively (Fig. 7.8 & 7.9). Figure 7.10 shows the overall surface response envelope for the coarse sediment response unit. The method used to construct the overall surface response envelope is the same as that for the fine sediment response unit except that 84th percentile envelope is used. This computed response envelope is also compared with the UBC design spectra. Based on $V_{S(30)}$ computed for different sites, the site classes for the coarse sediment response unit are very dense soil and soft rock and rock, thus, the UBC design spectra for site class C and B are computed. The other
parameters used to compute the UBC design spectra are the same as those of the fine sediment response unit. Compared to the UBC spectra, the $S_{u(\text{max})}$ of the overall response envelope constructed in this study is about 2.3 times higher than that predicted by the UBC spectra. The $PGA$ of the UBC spectra agree with the upper bound of the overall response spectra envelope. As discussed in the previous chapters, as more $V_S$ profiles are obtained, the characteristic $V_S$ profile of this response unit will be refined, and the response envelope should be updated accordingly.

7.8 Summary

Through Monte-Carlo simulation, overall surface response envelopes were constructed for the fine and coarse sediment response units. The six latest Quaternary faults that are located within 100 km from Las Vegas, namely, MS, BH, CW, WSM, PRP and RV, impose high seismic risk. The surface responses for both seismic response units are highly influenced by the spectral energy distribution of the input motion. For the fine sediment unit, compared to that predicted by the UBC design spectra using the closest fault studied, the $S_{u(\text{max})}$ of the overall response envelope (95$^{\text{th}}$ percentile) is about 1.5 times higher. The $PGA$ range projected in this study is very close to that projected by the USGS PSH map for 1 to 10% probability of being exceeded in 50 years. Also, the overall response spectrum has significantly larger response at longer period. However, the UBC spectra predicted a higher $PGA$. For the coarse response unit, the $S_{u(\text{max})}$ of the overall response envelope (84$^{\text{th}}$ percentile) constructed in this study is about 2.3 times higher than that predicted by the UBC spectra. Also, the overall response spectrum has slighter broader shape, with significant response to 1.5 sec. The $PGA$ of the UBC spectra agrees
with the $PGA$ of the overall response spectra. Because the $V_s$ data of the coarse sediment response unit are sparse, it is likely that the response envelope will be different, and probably smaller, if more $V_s$ profiles are available.
Table 7.1a Faults compiled in this study, maximum magnitudes estimated from empirical relationship for normal faults developed by Wells and Coppersmith (1994)

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<th>Fault Name</th>
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<th>Distance (km)</th>
<th>Average Strike</th>
<th>Sense of Movement</th>
<th>Fault Dip</th>
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<th>Slip Rate (mm/yr)</th>
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Table 7.1b Faults compiled in this study, maximum magnitudes studied by other researchers (Kemnitz, 1999).

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<th>Distance (km)</th>
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Notes:
- Most recent prehistoric deformation:
  - LQ: Late Quaternary (< 130 ka); LTQ: Latest Quaternary (<15 ka);
  - MLQ: Middle and late Quaternary (<750 ka); Q: Quaternary (<1.6 Ma);
- Sense of Movement:
  - N: Normal, R: Reverse, ND: Normal and Dextral, NS: Normal and Sinistral
- Fault dip:
  - SE: south east, W: west, N: north, V: vertical
- NM: not measured (applies to faults that are not considered active in this study)
- Blank cell: Information not available

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### Table 7.1c Fault names

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<td>Sheep Basin fault</td>
</tr>
<tr>
<td>SR</td>
<td>Sheep Range fault</td>
</tr>
<tr>
<td>BH</td>
<td>Black Hills fault</td>
</tr>
<tr>
<td>DEV</td>
<td>Detrital Valley faults</td>
</tr>
<tr>
<td>PC</td>
<td>Peace Camp faults</td>
</tr>
<tr>
<td>WSM</td>
<td>West Spring Mountains fault</td>
</tr>
<tr>
<td>YME</td>
<td>Yucca Mountain faults, eastern group</td>
</tr>
<tr>
<td>YMW</td>
<td>Yucca Mountain faults, western group</td>
</tr>
<tr>
<td>GRE</td>
<td>Garlock fault zone, eastern Garlock section</td>
</tr>
<tr>
<td>GW</td>
<td>Grand Wash fault zone</td>
</tr>
<tr>
<td>MES</td>
<td>Mesquite fault</td>
</tr>
<tr>
<td>LFM</td>
<td>Littlefield Mesa fault</td>
</tr>
<tr>
<td>MS</td>
<td>Main Street fault zone</td>
</tr>
<tr>
<td>WSD</td>
<td>Washington fault zone, Sullivan Draw section</td>
</tr>
<tr>
<td>CR</td>
<td>Carp Road fault</td>
</tr>
<tr>
<td>UMV</td>
<td>Unamed faults of Meadow Valley Wash</td>
</tr>
<tr>
<td>KSW</td>
<td>Kane Spring Wash fault</td>
</tr>
<tr>
<td>CS</td>
<td>Coyote Spring fault</td>
</tr>
<tr>
<td>ML</td>
<td>Maynard Lake fault</td>
</tr>
<tr>
<td>DM</td>
<td>Delamar Mountains fault</td>
</tr>
<tr>
<td>NBV</td>
<td>Unamed faults of Badger Valley</td>
</tr>
<tr>
<td>PH</td>
<td>Pahroc fault</td>
</tr>
<tr>
<td>UWPV</td>
<td>Unamed faults of western Pahranaget Valley</td>
</tr>
<tr>
<td>CSM</td>
<td>Central Springs Mountains fault</td>
</tr>
<tr>
<td>WSR</td>
<td>West Specter Range fault</td>
</tr>
<tr>
<td>BD</td>
<td>Boundary fault</td>
</tr>
<tr>
<td>AT</td>
<td>Area 3 fault</td>
</tr>
<tr>
<td>YF</td>
<td>Yucca fault</td>
</tr>
<tr>
<td>CRPL</td>
<td>Cockeyed Ridge-Papoose Lake fault</td>
</tr>
<tr>
<td>MM</td>
<td>Mine Mountain fault</td>
</tr>
<tr>
<td>CS</td>
<td>Cane Springs fault</td>
</tr>
<tr>
<td>BUH</td>
<td>Buried Hills fault</td>
</tr>
<tr>
<td>ER</td>
<td>Eleana Range fault</td>
</tr>
</tbody>
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Table 7.2  Fault parameters of 10 significant faults (most recent prehistoric deformation less than 15 ka) considered in this study

<table>
<thead>
<tr>
<th>Fault</th>
<th>Rupture Length (km)</th>
<th>Maximum Magnitude (Mw)</th>
<th>Distance (km)</th>
<th>Average Strike</th>
<th>Sense of Movement</th>
<th>Fault Dip</th>
<th>Slip Rate (mm/yr)</th>
<th>Recurrence Interval (ka)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BH</td>
<td>8.8</td>
<td>6.1</td>
<td>36</td>
<td>N31°E</td>
<td>N</td>
<td>SE</td>
<td>&lt;0.2</td>
<td></td>
</tr>
<tr>
<td>CW</td>
<td>32.3</td>
<td>6.9</td>
<td>49</td>
<td>N3°E</td>
<td>N</td>
<td>W</td>
<td>0.2-1</td>
<td></td>
</tr>
<tr>
<td>DV</td>
<td>100.0</td>
<td>7.4</td>
<td>150</td>
<td>ND</td>
<td></td>
<td></td>
<td>&lt; 0.2</td>
<td>0.7-1.3</td>
</tr>
<tr>
<td>GRE</td>
<td>57.5</td>
<td>7.2</td>
<td>150</td>
<td>N88°W</td>
<td>NS</td>
<td>V</td>
<td>1.0-5.0</td>
<td>0.2-3</td>
</tr>
<tr>
<td>MS</td>
<td>6.9</td>
<td>6.7</td>
<td>34</td>
<td>N44°E</td>
<td>R</td>
<td>SE, V</td>
<td>&lt;0.2</td>
<td></td>
</tr>
<tr>
<td>PRP</td>
<td>70.0</td>
<td>7.2</td>
<td>80</td>
<td>N39°W</td>
<td>D</td>
<td></td>
<td>&lt;0.2</td>
<td></td>
</tr>
<tr>
<td>RV</td>
<td>65.0</td>
<td>7.2</td>
<td>100</td>
<td>N58°E</td>
<td>SN</td>
<td>N</td>
<td>&lt;0.2</td>
<td>5.0 - 10.0; 50.0 - 100.0</td>
</tr>
<tr>
<td>WSM</td>
<td>60.0</td>
<td>7.1</td>
<td>73</td>
<td>N7°W</td>
<td>ND</td>
<td>WE</td>
<td>&lt;0.2</td>
<td>28-124</td>
</tr>
<tr>
<td>WSR</td>
<td>8.6</td>
<td>6.1</td>
<td>104</td>
<td>N2°W</td>
<td>N</td>
<td>W</td>
<td>&lt;0.2</td>
<td></td>
</tr>
<tr>
<td>YMW</td>
<td>25.1</td>
<td>6.7</td>
<td>145</td>
<td>N10°E</td>
<td>NS</td>
<td>W</td>
<td>&lt;0.2</td>
<td>17-40</td>
</tr>
</tbody>
</table>

Table 7.3  Attenuation relationships and their associated predictable parameters

<table>
<thead>
<tr>
<th>Relationship</th>
<th>Predictable Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abrahamson and Silva (1997)</td>
<td>PVA, PHA, HSA, VSA</td>
</tr>
<tr>
<td>Boore et al. (1997)</td>
<td>PHA, PSA</td>
</tr>
<tr>
<td>Campell and Bozorgnia (1994)</td>
<td>PHA</td>
</tr>
<tr>
<td>Spudich et al. (1999)</td>
<td>PHA, PSV</td>
</tr>
<tr>
<td>Sadigh et al. (1997)</td>
<td>PHA, PVA, PSA</td>
</tr>
</tbody>
</table>

Note: PVA: Pseudo-vertical acceleration; PHA: Pseudo-horizontal acceleration; HSA: Horizontal spectral acceleration; VSA: Vertical spectral acceleration.
Table 7.4  Selected earthquake ground motions for site response study

<table>
<thead>
<tr>
<th>Date</th>
<th>Earthquake Name</th>
<th>Location</th>
<th>Time</th>
<th>Magnitude</th>
<th>Station</th>
<th>Faults Represented</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/21/86</td>
<td>Chalfant Valley (CHV)</td>
<td>California</td>
<td>14:42</td>
<td>6.2</td>
<td>54214 Long Valley Dam L Abut</td>
<td>BH</td>
</tr>
<tr>
<td>6/28/92</td>
<td>Landers (LAN1)</td>
<td>Southern CA</td>
<td>11:58</td>
<td>7.3</td>
<td>Palm Springs Airport</td>
<td>CW</td>
</tr>
<tr>
<td>5/02/83</td>
<td>Coalinga (COA)</td>
<td>Southern CA</td>
<td>23:42</td>
<td>6.5</td>
<td>Parkfield-Stone Corral</td>
<td>MS</td>
</tr>
<tr>
<td>1/16/95</td>
<td>Kobe (KOB1)</td>
<td>Japan</td>
<td>20:46</td>
<td>6.9</td>
<td>Okayama</td>
<td>PRP</td>
</tr>
<tr>
<td>6/28/92</td>
<td>Landers (LAN2)</td>
<td>Southern CA</td>
<td>11:58</td>
<td>7.3</td>
<td>Las Palmas</td>
<td>RV</td>
</tr>
<tr>
<td>1/16/95</td>
<td>Kobe (KOB2)</td>
<td>Japan</td>
<td>20:46</td>
<td>6.9</td>
<td>Okayama</td>
<td>WSM</td>
</tr>
<tr>
<td>6/28/92</td>
<td>Landers (LAN3)</td>
<td>Southern CA</td>
<td>11:58</td>
<td></td>
<td>Glenoaks</td>
<td>DV</td>
</tr>
<tr>
<td>6/28/92</td>
<td>Landers (LAN4)</td>
<td>Southern CA</td>
<td>11:58</td>
<td>7.3</td>
<td>N Las Virg</td>
<td>GRE</td>
</tr>
<tr>
<td>8/10/01</td>
<td>Portola (POR1)</td>
<td>California</td>
<td>20:19</td>
<td>5.5</td>
<td>SF02 / 317</td>
<td>WSR</td>
</tr>
<tr>
<td>8/10/01</td>
<td>Portola (POR2)</td>
<td>California</td>
<td>20:19</td>
<td>5.5</td>
<td>SF02 / 227</td>
<td>YMW</td>
</tr>
<tr>
<td>8/10/01</td>
<td>Portola (POR3)</td>
<td>California</td>
<td>20:19</td>
<td>5.5</td>
<td>SF07 /43</td>
<td>MS, BH</td>
</tr>
<tr>
<td>6/29/92</td>
<td>Little Skull Mountain</td>
<td>Nevada</td>
<td>10:14</td>
<td>5.6</td>
<td>SGS</td>
<td>All Ten Faults</td>
</tr>
</tbody>
</table>
Figure 7.1  Comparison of attenuation curves for a M=7 earthquake
Figure 7.2  Target acceleration spectral envelopes. Blue: Abrahamson and Silva (1997); Green: Sadigh et al. (1997); Red: SHAKE 2000; Solid: mean; Dashed: 84th percentile
Figure 7.3  Acceleration response spectra of scaled SGS motion of LSM earthquake (magenta dashed line), from PSMD (red solid line), and from NSL (black solid line) against the target envelopes developed from Abrahamson and Silva (1997), Sadigh et al. (1997) and SHAKE 2000.
Figure 7.4a Earthquake motions selected from PSMD (continued on next page)
Figure 7.4a  Earthquake motions selected from PSMD (continued)
Figure 7.4b  Earthquake motions selected from NSL database
Figure 7.4c  Scaled LSM earthquake motion, recorded at SGS site, east component
Figure 7.5  Ninety-five percentile response envelopes for the fine sediment response unit using different input motions. Shaded area: 

\[ \mu \pm 2\sigma \]; blue line: mean (page 1 of 2).
Figure 7.5 Ninety-five percentile response envelopes for the fine sediment response unit using different input motions. Shaded area: $\mu \pm 2\sigma$; blue line: mean (page 2/2).
Figure 7.6 Ninety-five percentile values of $S_{a(\text{max})}$ (circle) and PGA (triangle) for the fine sediment response unit in the Las Vegas valley, for earthquake scenarios generated by the ten faults considered.
Figure 7.7  Bounding response spectra for fine sediment response unit and UBC design spectra
Figure 7.8 Eighty-fourth percentile response envelopes for the coarse sediment response unit using different input motions. Shaded area: $\mu \pm 2\sigma$; blue line: mean.

(page 1 / 2).
Figure 7.8  Eighty-four percentile response envelopes for the coarse sediment response 
unit using different input motions. Shaded area: $\mu \pm 2\sigma$; blue line: mean 
(page 2 / 2).
Figure 7.9 Eighty fourth percentile values of $S_{a(\text{max})}$ (circle) and PGA (triangle) for the fine sediment response unit in the Las Vegas valley, for earthquake scenarios generated by the ten faults considered.
Figure 7.10 Bounding response spectra for coarse sediment response unit and UBC design spectra
CHAPTER 8

SPECIAL INVESTIGATIONS

This chapter contains the following investigations: 1) the impact of cemented inclusions on surface response, and 2) the adequacy of using $V_{(50)}$ to capture geotechnical properties for seismic site response characterization.

8.1 Impact of cemented inclusions on surface response

As mentioned previously, the alluvial deposits in the Las Vegas basin include ubiquitous lenses of heavily cemented media. These lenses of carbonate-cemented sand, gravel, and/or clay particles are very stiff materials. At shallow depths, when bounded by less stiff materials such clay, silt and sand, the cemented materials present a tremendous impedance contrast. It is not clear yet to what extent they affect seismic response. To investigate the impact of these cemented inclusions on surface response, the following studies were conducted: 1) a simple sensitivity study based on a hypothetical soil column, and 2) a Monte-Carlo simulation based on a characteristic $V_s$ profile for fine sediment in the Las Vegas basin.

8.1.1 Simple sensitivity study

In the simple sensitivity study, the SHAKE analysis was applied to a 50-ft (15.24-m) deep hypothetical sand deposit, $V_s = 300$ m/s, containing a 5-ft (1.52-m) thick cemented layer ($V_s = 1230$ m/s) inserted at different depths. The results (Fig. 8.1) indicate
that the impact of this cemented inclusion on surface response is depth-dependent. The cemented inclusion usually helps to de-amplify the motion and it changes the site fundamental frequency by about 6%. However, with respect to the uniform sand deposit, the motion was slightly amplified (less than 3 percent) for a cemented layer placed at 13 m depth. Therefore, in 1-D site response analyses, if the sediment column contains cemented media, their impact on surface shaking should be evaluated. Omitting the cemented inclusions in such analyses will usually yield conservative results, but not in every case. Fortunately, as revealed by this simple study, the potential site amplification caused by the cemented inclusion is not large. This brief study also implies that for site-specific seismic response projection, an overall site classification based on averaged $V_s$ might not be adequate. Instead, to improve the accuracy of surface motion projections, a more detailed $V_s$ profile might be needed, particularly when strong stiffness contrasts exist. The outcome suggests that a more rigorous study, possibly using Monte-Carlo simulation, is needed to investigate the significance of such localized stiffness anomalies on surface shaking.

8.1.2 Monte-Carlo simulation

8.1.2.1 SHAKE input

As discussed previously, the degree of impact of the cemented inclusions on surface shaking is controlled by the stiffness contrasts they present in the soil column. If the cemented inclusions are present in a high-stiffness sediment column such as the coarse sediment unit, their influences on surface shaking are not expected to be high. Or if they are present at great depths, where the overburden pressure is high and the sediments are well consolidated and stiff, their influences on surface response are also
expected to be small. For this study, the characteristic $V_S$ profile of the fine sediment in the basin is considered. To take advantage of the maximum capacity of SHAKE, the upper 105 m of the profile is subdivided into 41 layers with thickness ranging from 1.5 to 3 m (Fig. 8.2). The input motion used is deconvolved SGS motion, east component, recorded during the 1992 LSM earthquake. Other than specified here, the same parameterization as described in chapter 5 is used.

A uniform distribution is used to generate randomly-placed cemented inclusions; in other words, the probability of occurrence of a cemented inclusion at any depth is equal, except for the restrictions noted above. Up to three cemented inclusions are substituted for entire layers in the layer geometry described below. The $V_S$ of the cemented layer is set to be 2000 m/s. The occurrence of cemented inclusions is limited to the depth range 0 to 105 m, because layers below 105 m are so thick (at least 13 meters) that it is not logical to superimpose cemented media; and also at that depth the velocity contrast will be much smaller than that at near surface, so the presence of cemented layer will have less effect on surface response. Figure 8.3 shows an example $V_S$ profile from a single run. To evaluate the adequacy of this 105 m depth limit, a single cemented layer is superimposed at 105 m depth, and the corresponding projected surface response is compared with that of the base case (Fig. 8.4). It is observed that differences between the two projected surface motions are negligible. Therefore, thin layers of cemented inclusions below 105 m deep are not likely to have significant impact on the surface response.
8.1.2.2 Projected surface response

Using the Monte-Carlo analysis approach with the SHAKE code in the parameterization described in Chapter 6, the surface responses were calculated with 2000 iterations. The results are shown in Fig. 8.5, 8.6 & 8.7. It is observed that the cemented inclusions particularly influence short period response (< 0.5 sec). The simulation result confirms the observations from the simple sensitivity study, namely, the presence of cemented inclusions could deamplify as well as amplify ground motions. The histogram for $S_a$ at $T=0.21$ sec, which corresponds to the predominant period of the projected surface motion from the no-cemented-inclusion case, is shown in Fig. 8.6. Compared to the no-cemented-inclusion case, the cemented inclusions can amplify or deamplify the projected ground motion by as much as a factor of 1.5. The likelihoods for amplification and deamplification are approximately equal. They particularly impact higher mode response (Fig. 8.7).

8.1.3 Conclusions for the impact of cemented inclusions on surface response

From the simple case sensitivity study and Monte-Carlo simulation to explore the influence of cemented inclusions on surface response, it is concluded that:

1) The impact of cemented inclusions on surface response is depth-dependent

2) The cemented inclusions particularly affect high-frequency response (>2 Hz)

3) The cemented inclusion can deamplify as well as amplify the surface response

4) For the 1-D analysis, if cemented media exist, their influence on surface response should be evaluated.
8.2 The adequacy of using $V_{s(30)}$ to capture geotechnical properties for seismic site response characterization

The $V_s$ averaged over the upper 30 meters ($V_{s(30)}$) has been widely used by the engineering community (as embedded in the IBC 2003) for site classification for seismic design. Two questions arise regarding this simplified approach:

1) For sites with the same $V_{s(30)}$, what is the credible range of projected surface motion?

2) How do velocities below 30 m affect the projection?

This study uses Las Vegas basin as an example to examine how well the $V_{s(30)}$ captures geotechnical properties for seismic site response characterization.

8.2.1 Credible range of projected motion

8.2.1.1 Shear Wave Velocity Profile

In this study, the upper 30 meters of the characteristic $V_s$ profile of the fine sediment unit in the Las Vegas basin is used (Fig. 8.8). Two thousand pseudo-random $V_s$ profiles with the same $V_{s(30)}$ were generated by assuming normal distribution of $V_s$ for the upper 9 layers and calculating the required $V_s$ of the 10$^{th}$ layer. To generate credible $V_s$ profiles, for the upper 9 layers, only the shear wave velocities that fall within the range of 100 to 2200 m/s are permitted in the analysis. For the 10$^{th}$ layer, the $V_s$ is restricted to between 300 and 760 m/s, which correspond to $(\mu - \sigma)$ and $V_s$ of the engineering bedrock criterion, respectively. Solutions that don't satisfy this criterion are discarded in the computations. The $V_s$ of the halfspace is assumed to be 760 m/s. This $V_s$ configuration simulates the following situation: 1) the engineering bedrock criterion is satisfied at 30 m depth; 2) the $V_s$ in the upper 30 meters is everywhere greater than 100 m/s.
m/s, which is appropriate considering observed data for the fine sediment deposits in the Las Vegas Valley; and 3) cemented inclusions could be present in the soil column. The pseudo-random $V_s$ profiles and other parameters used in the analysis are summarized in Fig. 8.9.

8.2.1.2 Projected Surface Response

The surface responses were computed using the 2000 different $V_s$ profiles generated as described above. A response envelope was formed by considering the $\mu \pm 2\sigma$ (Fig. 8.10). It is noticed that for the case studied (site class C), detailed variations of $V_s$ in the upper 30 meters affected surface response for T up to 1 sec. The $S_{a(max)}$ ranged from 14 to 26 cm/s$^2$ , while the $S_{a(max)}$ from the uniform velocity profile is about 23 cm/s$^2$. From T =0.2 to 0.3, the projected $S_a$ using the uniform velocity profile is close to the upper-bound value projected in the Monte-Carlo simulation. For the other period ranges, the $S_a$ projected using the uniform velocity profile tends to fall in the middle of the range projected by the Monte-Carlo simulation. Overall, the uniform velocity profile captured the geotechnical properties for seismic site response characterization well.

8.2.1.3 Site Fundamental Period ($T_s$)

Variation of $V_s$ profiles in the upper 30 meters will change the site fundamental period. The histogram of site fundamental periods of 2000 different soil profiles is plotted in Fig. 8.11. It is noticed that the $T_s$ can vary from about 0.2 to 0.33 second, whereas the middle value (0.258 sec) will be most likely captured by uniform velocity profile.
8.2.2 Effects of deeper strata on surface response

One major concern about $V_{S(30)}$ is that it doesn’t take consideration of the velocity structure below 30 meters depth, which might contribute tremendously to surface response for deep soil deposits. To examine this issue, the following cases were studied:

1) Case 1: As discussed in Chapter 5, by using the characteristic $V_S$ profile for a clay-rich deposit, considering all the $V_S$ values to 400 m depth, the projected surface responses enveloped the measured motions. Here, the same $V_S$ profile and its associated standard deviation are used to construct the surface response envelope for earthquake motions, using the SGS east component recorded during the 1992 LSM earthquake as input.

2) Case 2: In this case, the $V_{S(30)}$ and its standard deviation are used for the upper 30 meters. For depths below 30 meters, the characteristic $V_S$ profile and its associated standard deviation are used.

3) Case 3: The $V_S$ profile is the same as that of Case 2 but with a constant value for depths below 30 meters. The idea is to examine such situations where only $V_{S(30)}$ is available, but through literature search or regional geologic study, a background $V_S$ profile can be estimated.

4) Case 4: Only $V_{S(30)}$ and its standard deviation is used, no $V_S$ below 30 meters is considered.

The $V_S$ profiles used in all the four cases are illustrated in fig. 8.12.

8.2.2.1 Projected Surface Response

Figure 8.13 summarizes the four cases and plots the projected response envelopes against measured motions. Limited by graphic visibility, the results are presented in two
separate plots. It is observed that, for a deep soil deposit, if only $V_{S(30)}$ is used for site response calculation without considering the $V_s$ profile below 30 m depth (Case 4), the projected surface response will be significantly underestimated. If $V_{S(30)}$ are the only $V_s$ data available, by incorporating a background $V_s$ profile to an appropriate depth (Case 3), the surface response envelope is likely to capture the upper-bound response. Building on Case 3, if an appropriate standard deviation is incorporated for the profile below 30 m depth (Case 2), the projected surface response tends to be improved in that it will have a good representation for the low response values also. As discussed in chapter 5, the best ground motion projection is achieved by considering a detailed $V_s$ profile to an appropriate depth and incorporating its associated standard deviation (Case 1).

8.2.2.2 Summary and Discussions

1. For cases where depth to bedrock is 30 meters, $V_{S(30)}$ is a good indicator for predicting surface response and site fundamental period. As such case is not likely to happen, caution should be taken when applying the $V_{S(30)}$ criterion alone to classify site category for seismic design purposes.

2. If the depth to bedrock is greater than 30 meters, neglecting the $V_s$ structure below 30 meters could significantly underestimate surface response. For the case studied here, the $PGA$ was underestimated by a factor of 1.8.

3. The projected surface response can be improved by incorporating a background $V_s$ profile to an appropriate depth.

4. Overall, for site response analysis, $V_{S(30)}$ alone is not sufficient. Depth to halfspace is a factor that can not be neglected. For cases that the exact basement rock and soil interface is not known, or the soil deposit is so deep that no reliable damping and
modulus reduction functions are known, an iterative procedure to select depth to model halfspace as discussed in previous chapters should be considered.
Figure 8.1 Impact of cemented layers in the shallow sediments on surface shaking: depth of occurrence
Figure 8.2 Fine sediment response unit parameterization for the Monte-Carlo simulation to study the influence of cemented inclusions on surface response. Layer boundaries indicated by “+” symbols.
Figure 8.3  Example $V_s$ profile for the Monte-Carlo simulation from a single run.
Figure 8.4 Comparison of response spectra for cases with and without cemented inclusions
Figure 8.5  Projected surface motion. Green line: the base case; blue lines: with randomly generated cemented inclusions.
Figure 8.6 Histogram of spectral acceleration at $T = 0.21$ sec. Red line: spectral acceleration for soil profile without cemented layer.
Figure 8.7 Amplification ratio. Grey line: the base case; black lines: with randomly generated cemented inclusions.
Figure 8.8 The upper 30 m of the characteristic $V_s$ profile for the fine sediment unit in the Las Vegas basin. Dotted line: measured; solid line: $\mu$ and $\mu \pm 2\sigma$. (The entire 400-m deep profile is shown in Chapter 6.)
Figure 8.9 Two thousand pseudo-random $V_s$ profiles with the same $V_{soj}$ of 468 m/s.

The red line is $V_{soj}$. The regularly-spaced pattern of black lines is a computer-plotting peculiarity.
Figure 8.10: $S_a$ for using $V_{s(30)}$ only (green line) with the ninety-five percentile surface response envelope of 2000 different soil profiles (shaded area).

Figure 8.11 Histogram of site fundamental period for 2000 randomly sampled soil profiles. Green line is site fundamental period by using $V_{s(30)}$ only.
Figure 8.12  Mean $V_S$ and ranges used in the four cases
Figure 8.13 Surface response envelopes: a) Case 1, 2 and 4; b) Case 1 and 3; Dotted lines: measured surface motion.
CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

This chapter contains answers to research questions raised in Chapter 1 and provides recommendations for future research.

9.1 Significance of Shallow Sediments in Surface Shaking of Deep Soil Deposits

9.1.1 Observed Amplification

Deep soil deposits have a significant effect on surface response. For the Las Vegas Basin, deep soil sites have exhibited amplification by a factor of up to 5 to 10 with respect to motions recorded at near-rock sites at the edges of the basin. Most amplification occurred over the period range 0.3 to 5 sec (0.2 to 3.3 Hz).

9.1.2 Site Response Model and its Applicability

For deep soil deposits (>200 m), 1-D site response analyses are best conducted using an equivalent-linear model with the halfspace placed deep within the column, but not necessarily at the bedrock interface. When parameterized appropriately, the 1-D equivalent model can adequately capture ground motion amplification over a limited frequency range.

For the Las Vegas study, the 1-D equivalent linear model adequately captured ground motion over the period range 0.2 to 1 sec (1 to 5 Hz). This period range corresponds to the fundamental modes of vibration for structures having approximately 2
to 10 stories. Ground motion having frequencies greater than 5 Hz will be strongly influenced by very-near-field effects, so regional predictions are not meaningful. Also, ground motion at these frequencies affects very short structures, and so is of lesser engineering importance. For low frequency response (less than 1 Hz), the 1-D model is no longer appropriate; two- and three-dimensional effects and regional-scale structural/velocity models must be evaluated.

9.1.3 Model Parameterization

The 1-D site response model is optimally parameterized through iterative assessment to select the parameter set whose projected response spectrum best matches the measured or anticipated response. Acceleration spectral intensity and predominant period are key criteria for matching. Peak ground acceleration and peak spectral acceleration should also be considered.

This research demonstrated the beneficial use of combined active- and passive-source surface wave measurements to develop soil-column $V_s$ profiles that are both detailed and deep.

9.1.4 Projecting Surface Response

To account for uncertainties in the seismic source, vertical and lateral ground variability, and knowledge of $V_s$ at depth, site response projections are best presented in terms of surface-response envelopes. The envelopes can be created through Monte-Carlo simulation, using $V_s$ parameterizations selected at random from the normal distribution of a baseline model. The surface response envelopes are useful for earthquake hazard mitigation. Characteristic envelopes can be developed independently for each seismic microzone.
9.1.5 Influence of High Velocity Inclusions on Surface Response

High $V_\text{s}$ inclusions, like the carbonate-cemented lenses that appear in some desert settings, particularly affect surface response at frequencies greater than 2 Hz. They can deamplify as well as amplify surface response, as well as change site fundamental periods. The surface motions can be amplified or deamplified by a factor as large as 1.5. For the 1-D analysis, if such high velocity inclusions exist, their influence on surface response should be evaluated.

9.1.6 Adequacy of Using $V_{s(30)}$ for Site Response Projections

For deep soil deposits, surface response can be underestimated if site response analysis is based solely on $V_\text{s}$ averaged over the upper 30 m. Inclusion of strata below 30 m in the site response analyses can significantly improve surface motion projections. Lacking site-specific information, the projected surface response can be improved by extending the profile to greater depths by appending an appropriate background $V_\text{s}$ profile.

9.2 Earthquake Microzonation for the Las Vegas Basin

9.2.1 Earthquake Sources

Six active faults that lie within 100 km from Las Vegas have been identified that present earthquake hazards for the Las Vegas Basin.

9.2.2 Site Response Units

Site response units are best identified considering both lithologic and dynamic characteristics of soil deposits. Two major site response units are established for the Las Vegas basin, namely, a fine-sediment response unit and a coarse-sediment response unit. The fine sediment unit dominates the central and eastern parts of the Basin.
It is primarily clay and has relatively low $V_s$ and high amplification potential. The coarse sediment unit includes coarse-grained and mixed-grain size deposits. It dominates the western part of the basin and has relatively high $V_s$ and low amplification potential.

9.2.3 Surface Response Projections

For the fine-grained response unit, Monte-Carlo simulation based on deterministic seismic response analysis yielded upper bound $(\mu + 2\sigma) PGA$ and $S_{a(max)}$ of 0.31 g and 1.1 g, respectively. Compared to the 1997 UBC spectra, the $S_{a(max)}$ is about 1.5 times higher and the envelope is broader, with significant response up to 3 sec. However, the UBC spectrum predicts a $PGA$ that is 10 to 20% higher. The projected upper bound $PGA$ is very close to the case of 1% probability of exceedance in 50 years for NEHRP D-C boundary sites projected by the USGS. This finding is appropriate for a deterministic seismic hazard analysis.

For the coarse sediment response unit, Monte-Carlo simulation based on deterministic seismic response analysis yielded upper bound $(\mu + \sigma) PGA$ and $S_{a(max)}$ of 0.31 g and 1.4 g, respectively. Compared to the 1997 UBC spectra, the $S_{a(max)}$ is about 2.3 times higher and the envelope is broader, with significant response up to 1.5 sec. The $PGA$ of the UBC spectra agrees with the $PGA$ of the upper bound response spectra. The projected upper bound $PGA$ is about 13% lower than the case of 1% probability of exceedance in 50 years for NEHRP B-C boundary sites, projected by the USGS. The $V_s$ dataset for the coarse sediment response unit is sparse; it is likely that the response envelope will shrink when more $V_s$ profiles are incorporated.
9.2.4 Application of These Research Results

The bounding spectra were developed based on 1-D site response modeling and Monte-Carlo simulation. The site response model was validated using historically recorded ground motions. These response envelopes can be used as a general guide for earthquake hazard mitigation in the period range 0.2 to 1 sec (1 to 5 Hz).

9.3 Future Research Recommendations

Seismic hazard evaluation is a multi-disciplinary subject that integrates geology, seismology, geophysics, structural earthquake engineering, soil dynamics and geotechnical earthquake engineering. Uncertainties are present in every field mentioned above. The accuracy of surface shaking projections is largely constrained by the quality and quantity of information incorporated in the evaluation process. To better understand the seismic hazard for the Las Vegas basin and beyond, there are four aspects that most need further investigation:

1) Calibrating site response model: The procedure established in this research for soil-column parameterization is directly applicable when recorded responses on paired rock and soil sites are available to calibrate the model. For cases where recordings on paired rock and soil sites are not available, projections might be calibrated against credible ground motions developed using scaled historic data or other accepted practices, such as appropriate attenuation relationships. It is recommended that this approach be tested in future research.

2) Incorporating credible earthquake sources: In this research, an active fault is
defined as a fault that has moved within the last 15,000 years. All the known active faults within 150 km of Las Vegas were screened, and six of them were identified as being potentially significant earthquake sources for the Las Vegas basin. This study was based on the USGS published Quaternary fault map. There are local faults that lie within the Las Vegas basin that are not included in the USGS fault map, such as the Las Vegas Shear Zone. Experts in UNLV's Geoscience department are studying previously unmapped faults that exhibit Late Pleistocene to Holocene offsets. If any of these local faults is confirmed to be active, it would impose high seismic risk to the Valley, and therefore should be incorporated into the seismic hazard evaluation model. Also, because they are close to the Las Vegas metropolitan area and some of them run through the populated area, in addition to ground shaking hazard, the ground rupture hazard should be evaluated. For the local faults, the near-fault effects, such as influence of rupture directivity on ground motion, should be studied. And the suitability of the 1-D model should be assessed for such cases. The author is not aware of any published reference that gives guidance regarding the minimum distance from active fault that the 1-D site response analysis is valid.

3) Refining seismic response units: In this research, two response units were established based on the dominant sediment type over the upper 30 meters, and the corresponding response envelope was constructed for each. To make the ground motion map more useful for engineering practice, sub-response units within each main unit can be developed, preferably based on measured $V_S$. More data should be collected within the interfingered zone to more clearly define the boundary between the two main units.

4) Developing site fundamental period contours for the Las Vegas basin: In
addition to the ground motion map that has been developed in this study, it is recommended that a site fundamental period contour map be developed. Such a map will be useful in structural design: engineers can avoid designing structures having resonance periods matching those of their sites. The site fundamental period contour map can be developed through computation of the transfer function using SHAKE, or by taking microtremor measurements at selected sites in the basin and computing the horizontal-to-vertical spectral ratios.
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