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Independent Seismic Evaluation of the 24-580-980 South Connector Ramps

Site-Response Studies for Magnitude 7.25 Hayward Fault Earthquakes

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Lawrence Livermore National Laboratory

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Abstract

Site response analyses were performed using the computer program SHAKE at the I-24/580/980 site to provide seismic ground motions for independent evaluations of the freeway interchange structure. Analytical models and soil parameters for SHAKE analysis were developed from geotechnical data obtained from several site investigation programs conducted at the site in 1960, 1991 and 1995. Two sets of rock outcropping input motions were used: (1) modified Santa Cruz earthquake records provided by Caltrans, and (2) LLNL synthetic strong ground motions. The LLNL synthetic ground motions were developed using LLNL Empirical Green functions method simulating strong earthquakes of moment magnitude 7.25 from the nearby Hayward Fault about 4 km from the site. Calculated ground surface motions using LLNL median rock input-motions are compatible with Caltrans design/evaluation motions.

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1.0 Introduction

In support of the LLNL independent seismic evaluation of the freeway I-24/580/980 interchange structure, a series of site response analyses were performed using the LLNL version of computer program SHAKE to determine seismic ground motions at the site. The LLNL version of SHAKE has been validated and extensively used for various NRC and DOE projects. The SHAKE program (Schnabel et al. 1972) is the well known tool for evaluating the effect of local soil conditions on ground response during earthquakes. The SHAKE procedure generally involves several steps:

(1) Determine the characteristics of ground motions likely to develop in rock formation underlying the site, and select a set of acceleration time histories with these characteristics for use in the analysis.

(2) Determine the site model and the dynamic properties of the soil deposit at the site.

(3) Compute the response of the soil deposit to the base-rock motions.

The base-rock motions used in this study consists of two sets of motions. The first set of motions provided by Caltrans (Abghari and Jackura, 1992) are three components of Santa Cruz earthquake records modified for this site. The second set of motions developed by Hutchings et al. 1996 at LLNL are synthetic strong motions.

The synthetic strong motions were developed using Empirical Green's Functions method to simulate seismic ground motions of a strong earthquake of moment magnitude of 7.25 from the nearby Hayward Fault about 4 km form the site. A total of 100 rupture scenarios for earthquakes from Hayward Fault were calculated. Prior to LLNL's independent evaluation of site ground motion, LLNL did not receive a definitive policy statement from Caltrans staff on whether the analysis and retrofit design was based on median or 84th percentile motion. In light of this, ground motion estimates were initially developed for both median and 84th percentile motions. However, later information provided by Caltran on their policy for this site, only the median motions were used for analysis/evaluation of the interchange structure. The 84th percentile motions were also presented in this report for information and reference only.

The calculation site models were developed based on available information of site geology and the available data from geotechnical investigations at the site. Site investigation programs were conducted at the site in 1960, 1991, and 1995. Site investigations include site boring logs, soil samples taken, standard penetration tests and shear wave velocity measurements. Laboratory tests on soil classification, basic soil properties, undrained strength, consolidation characteristics were also conducted on soil samples taken from the site. Dynamic soil properties for alluvium soils at the site were estimated from the published data for similar soils. A best estimate site model was developed using the average value of soil parameters and our best judgment. Uncertainty in site response analysis was assessed in accordance with the latest guidance provided in ASCE Standard (ASCE-4, 1997). Sensitivity on modeling the variation of shear wave velocities for the depth below 290 ft from the ground surface was also investigated. Calculated LLNL median ground surface motions were compared to Caltrans design/evaluation motions.

2.0 Geologic And Seismic Characteristics

San Francisco Bay is a northwest trending depression, bounded on the west side by the San Andreas fault and on the east side by the Hayward and Calaveras Fault systems. The bay basin is largely infilled with alluvial deposits. The I-24/580/980 interchange site, located in the east bay, is underlain by older alluvial deposits. Most of the old alluvial deposits are primarily silty and sandy clays, silt, silty sands and gravels. This alluvial deposit is considerably deep, with the depth to bedrock of about 465 ft from the ground surface of the site. The bedrock unit is the Franciscan Complex, a diverse assemblage of sedimentary, volcanic, and metamorphic rock formation. In the vicinity of the interchange, the rock formation consists primarily of shale and graywacke sandstone.

2.1 Generalized Soil Profiles

The layout of I-24/580/980 interchange together with locations of seismic velocity measurements and testing boring holes are shown in Fig. 2-1. The boring holes locations for geotechnical investigation at the site in 1960 are not shown in this figure.

Three generalized soil profiles along the interchange were established from available boring logs at the site, Abghari and Jackura, 1992. Two shallow profiles, one along the WS line (Fig. 2-2) and one along ES line (Fig. 2-3), were constructed on the basis of information and data available through the logs of test borings in 1960.

These two profiles only show general subsurface information from the ground surface to about 75 ft deep which is the deepest bottom among the boreholes drilled in 1960. In 1991, four test borings were drilled by the Office of Engineering Geology, Division of New Engineering Technology, Material and Research of Caltrans. The deepest borehole, B4-91, as shown in Fig. 2-1 is located at the parking lot of the Telegraph Avenue Maintenance Station. The elevation of the ground surface is +67 ft from MSL and the elevation of the top of bedrock is -398 ft from MLS. The total depth of boring log is about 465 ft. This is the only borehole drilled to the bedrock in the interchange site. Immediately adjacent to this borehole (about 12 ft away), another borehole (B1-91), was drilled to a depth of 101 ft. The third borehole, B2-91, drilled to a depth of 254 ft is located in the Southern end of the interchange at 34th Street near the Martin Luther King way. The 4th borehole, B3-91, is located at Telegraph Avenue near to the merging point of WN line and WS line. This borehole was drilled to a depth of 244 ft from the ground surface. Soil samples were taken at every five feet intervals from boreholes B1-91 and B2-91 down to about 100 ft from the ground surface. Standard Penetration Tests (SPT) were also conducted at five foot intervals while soil samples were taken from these two boreholes. The soil boring logs were recorded to the bottom of each hole. Based on geotechnical data obtained from the 1991 logs of test borings, a deeper generalized soil profile was developed by Caltrans (Abhari and Jackura, 1992) along the WS line as shown in Fig. 2-4.

In 1995, Kleinfelder, Inc. was contracted to drill and sample seven additional borings, Kleinfelder (1996). The locations of seven boring holes (B1 to B7) are shown in Fig. 2-1. Seven boreholes were drilled to a depth of about 200 ft from the ground surface. Soil samples were taken at five foot intervals for the upper 100 ft and at ten foot intervals below 100 ft. SPTs were also conducted when soil samples were taken. The depth of water tables varies from 20 ft to 25 ft below the ground surface. The soil type and soil sublayers were found similar to those shown in Fig. 2-4. In general, the site is underlaid by alluvium which consists of primarily sandy/silty clays, silts, sands and gravels. The average blow count numbers from SPTs ranges from 19 to 100 indicating moderately compact to very dense cohesionless soils and stiff to hard cohesive soils.

2.2 Average Shear-wave Velocity Profile

A compilation of down hole seismic velocity measurements acquired at the 24/580/980 freeway interchange was provided by Vickery and Cole, 1995. Three boring holes were logged for shear wave velocity using a down-hole system in 1991. The locations of these three holes are shown in Fig. 2-1. The shear wave velocities were measured down to about 250 ft in boreholes B2-91 and B3-91. In borehole B4-91, the shear wave velocities were measured down to the bedrock at a depth of 465 ft from the ground surface. The logging of soil types and soil sublayers in this borehole were also complete to the bedrock. The geotechnical data and measure shear wave velocities from this borehole are very important for developing analytical models for this site. Shear wave velocities measured from each of these three boreholes are overplotted and shown in Fig. 2-5. As mentioned earlier, in September of 1995, Kleinfelder, Inc. was contracted to drill and sample seven additional borings. Seismic velocity measurements were acquired at testing B5 and B7 as shown in Fig. 2-1. Shear wave velocities were measured to about 185 ft from the ground surface. The measured shear wave velocities from these two boreholes are also overplotted in Fig. 2-5. Based on available measured shear wave velocities around the site, the weighting average of shear wave velocities in each sublayer of the site model were computed and defined as our best estimate model. A total of 41 sublayers whose thickness varying between 5 ft and 20 ft were used in our computational site model.

2.3 Soil Parameters and Properties

Caltrans conducted a series of laboratory testing of samples taken from boreholes B1-91 and B2-91. The laboratory tests included unit weight, moisture content, specific gravity, Atterberg limits, consolidation, triaxial and permeability tests. Densities of soil samples were determined before triaxial and consolidation tests. Gradation analysis on some samples were also performed in order to have proper classification of soil type. The results of the laboratory testing are summarized in Table 1. The unit weights and plasticity index obtained from laboratory test provide the basis for estimating these parameters for similar soils in the deeper part of the site.

Table 1 Results of laboratory tests on soil samples from boreholes B1-91 and B2-91, I-24/580/980.

Sample No. 1	Depth (Ft.)	Soil Description	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS* Soil Type	Moisture Content (%)	Unit Weight (pcf)
B1-2	17	Silty Clay	43	18	25	CL		
B1-3	22	Sandy Clay	—	—	—	CL	22.8	125
B1-4	23	Clay Silt - Silty Clay	36	19	17	MH-CL		
B1-5	28	Silty Clay	38	19	19	CL	22.0	130
B1-6	37	Clay Silt - Silty Clay	53	23	30	MH-CL	28.5	122
B1-11	68	Clayey Silt - Silty Clay	32	16	16	MH-CL	28.5	122
B1-14	83	Sandy Clay	—	—	—	CL	18.5	130
B1-15	88	Clayey Silt	—	—	—	MH	22.0	128
B2-3	18	Silty Clay	—	—	—	CL		127
B2-5	28	Gravelly Sand	—	—	—	SP	22.0	129
B2-6	32	Gravel/Clay	—	—	—	GC	16.1	118
B2-8	43	Silty Clay	49	20	29	CL	24.2	127
B2-9	48	Silty Sand	44	23	21	SM	30.0	124
B2-14	73	Silty Clay	53	27	26	CL	32.2	120
B2-15	78	Silty Clay	54	23	31	CL	24.0	126

USCS - (Unified Soil Classification System)
CH - inorganic clays of high plasticity
CL - inorganic clays of low to medium plasticity
MH - inorganic silts of high plasticity
SM - silty sands, sand-silt mixtures
SP - poorly graded sands, gravelly sands
GC - gravel-sand-clay mixtures

Laboratory tests were not specifically conducted on dynamic soil properties of soil samples obtained from the interchange site. However, for the need of performing site response analysis the dynamic soil properties may be estimated from the available data of similar soils. Dynamic properties required for SHAKE procedure consists of shear modulus and damping ratios at various shear strain levels. The variation of shear modulus and damping ratio with cyclic shear strains for clayey soils has been well established from a large number of studies. A good summary of published data is presented by Sun et al. (1988) as well as Seed and Sun (1989). A study on the influence of plasticity Index, PI, on the cyclic stress-strain parameters of saturated soils needed for site response evaluation was presented by Vucetic and Dobry (1991). The PI is the difference between the liquid and plastic limits of fine grained soils. It provides a measure of the range of water content that the soil remains in a plastic state. Vucetic and Dobry presented ready-to-use charts (Fig. 2-6) showing the effect of PI on variation of shear moduli and damping ratios with shear strains. The charts are based on experimental data from sixteen publications encompassing normally and overconsolidated clays as well as sands. The test results for Bay mud from Isenhower (1979), Sitar and Salgado (1989), as well as the data presented by Seed et al. (1970) and Sun et al. (1988) were all included in Vucetic and Dobry's study. The curves labeled PI values of 0, 15, and 30 were used in dynamic site responses. The PI values associated with soil types of each sublayer are shown in Fig. 2-5.

The variation of normalized shear modulus and damping ratios with shear strains for sandy and gravelly soils were presented by Seed et al. (1970), Stokoe and Lodde (1978), Seed et al. (1984), Sun et al. (1988), EPRI (1993). Figure 2-7 shows typical normalized shear modulus and damping curve from Seed and Idriss (1970), Sun et al. (1988) and testing data from Treasure Island fine sands (EPRI 1993). The influence of confining pressure on the normalized modulus reduction relationships for sand has been recognized. For clay this influence is not evident. To include the effect of confining pressure on the stiffness of sandy soils at the site, the data presented in Sun, Golesorkhi and Seed (1988) were used. Three effective confining pressures of 1.0, 2.0 and 3.0 ksc applied for the test data were converted to the equivalent depths of the site. The corresponding depths of these three confining pressures are about 30 ft., 60 ft. and 120 ft. For engineering analysis, it is reasonable to assign the modulus reduction curve with 1 ksc confining pressure to represent dynamic characteristics of those sandy soils within the upper 30 ft of the site. The modulus reduction curve of 3 ksc confining pressure was used for sandy soils deeper than 120 ft of the site. Similarly, the curve of 2 ksc confining pressure was used for the sublayers of sandy soils between 30 ft. and 120 ft.

The damping curves corresponding to modulus reduction curves of the three confining pressures were not available. However, measured values of the damping ratio for cohesionless soils proposed by Seed and Idriss (1970, 1984) as shown in Fig. 2-7 were widely used for site response calculations. The mean damping curve shown is adequate for most cohesionless soils up to a confining pressure of 2500 psf, (Schnabel 1973). The damping ratio will be affected by overburden pressure, relative density, degree of saturation and the number of loading cycles. The effect of relative density and number of loading cycles are minor. It has been found that the damping decreases with increasing effective overburden pressure and degree of saturation. In this study, the mean damping curve was used for the upper 30 ft of sandy soils and the lower bound of damping curve was used for the sandy soils deeper than 120 ft. While the average values of the mean and the low bound curves at various strain levels was assigned for those sandy soils in between 30 and 120 ft.

3.0 Ground-Response Analyses

Ground-response analyses were performed using SHAKE program for the I-24/580/980 interchange site. SHAKE program computes the response of horizontally layered soil profiles subjected to bedrock input motions from strong earthquakes. Each layer in the analytical model is completely defined by its value of shear modulus, critical damping ratio, density and thickness. These values are independent of frequency. The responses in the analytical model are caused by the upward propagation of shear waves or pressure waves from the underlying bedrocks. The strain dependence of modulus and damping is accounted for by an equivalent linear procedure based on an average effective shear strain level computed for each layer. The effect of shear modulus and damping of the bedrock (halfspace) on the calculated motions are included in the procedure. The input motion (or the object motion) can be given in any one layer in the model and new motions can be computed in any other layer.

The stress-strain characteristic of soils are strongly non-linear and may significantly influence the dynamic response of a site subjected to strong earthquake motions. A good site response analysis must therefore consider these non-linear effects. It is known that the non-linear behavior of soil material cannot be fully described by constant elastic moduli and damping coefficients. However, a good approximation of the effects of soil non-linearities on the response can be obtained by use of constant strain compatible moduli and damping ratios in a sequence of linear analyses. This method, which is known as the equivalent linear method (Seed and Idriss, 1969) can be briefly described in the following manner.

In a site response analysis, the equivalent linear method starts with a linear analysis using estimated soil properties in each layer of the soil system. This analysis yields complete time histories of shear strain, from which the effective shear strain amplitudes are calculated in each layer. (The effective shear strain amplitude is usually taken as 65% of the maximum shear strain or as the RMS value of the shear strain time history). Using the computed strain amplitudes, an improved set of soil moduli and damping ratios are obtained from the appropriate soil data curves of the type shown in Figs. 2-6 and 2-7 and a new linear analysis is performed with these properties. The process is repeated until the properties from two consecutive analyses differ by less than a specified tolerance, say 5 percent. This will usually require fewer than 5 to 7 iterations. The results of the last iteration are taken as the final solution to approximate a true nonlinear solution. This technique has been widely used in practice because it is an efficient method and is easy to implement in a computer program.

The program SHAKE has been widely used to calculate the response of soil sites. Studies of the ground response in Mexico City during the September 1985 earthquake (Seed et al. 1987) and on Treasure Island during the October 1989 Loma Prieta earthquake (Idriss, 1990, Hryciw et al., 1991 and Rollins et al. 1993) indicated that SHAKE could provide reasonable estimates of ground response during earthquake. The LLNL version of SHAKE program has been used to calculate ground response with various NRC and DOE projects. Validations of the SHAKE program has been reported by Bohn et al. (1983) and Chen (1986.) In those studies, validation involved comparisons of the observed mean site amplification factor and the calculated mean site amplification factor using several sets of motions recorded at soil/rocks station pairs from 1975 Oroville and 1976 earthquakes. Validation also involved comparisons of observed and calculated motions at the Richmond Field Station from the 1977 Briones earthquake as well as at Forgaria (soil)/S. Rocco (rock) station pair from 1976 Friuli earthquake. Our earlier studies indicated that overall site response spectra as predicted by one-dimensional equivalent linear techniques such as SHAKE procedures agree fairly well with those of recorded motions.

3.1 Input rock motions

The first step of site response analysis involved determination of the characteristics of motions likely to develop in rock-outcrop adjacent to the site or in the bedrock underlying the site. Acceleration time histories with these characteristics have been developed for SHAKE analysis at the site. Based on studies of the seismicity around the site, two major fault systems should be considered for the site: Hayward Fault about 2.5 miles (4 km) east of the site and San Andreas Fault about 16.5 miles (26.5 km) west of the site. Preliminary studies by Caltrans (Abghari and Jackura, 1992) using average ground motion attenuation curves published for the Bay Area showed that the peak bedrock acceleration at this site is about 0.6 g and 0.32 g from Hayward Fault and San Andreas Faults, respectively. Hence, the Hayward Fault is the controlling fault and the dynamic site response analysis should be conducted for an earthquake of magnitude about 7.5 at 4 km.

3.1.1 Modified Santa Cruz Earthquake Records

Three components of Santa Cruz earthquake records was modified by Caltrans for their site response analysis using computer program SUMDES (Li, 1993). The recorded motions were modified so that their response spectra match the magnitude 7.5 target spectrum based on the attenuation model developed by Sadigh et al. (1992). The time step of the acceleration time histories is 0.02 seconds. Only the first 2000 points (40 seconds) were used as a rock outcropping motion for SHAKE analysis. The time histories are shown in Fig. 3-1. The peak ground accelerations are 0.45 g, 0.51 g and 0.44g in the directions of H00, H90 and vertical. The response spectrum of each components are shown in Fig. 3-2.

3.1.2 LLNL Synthetic Strong Motions

A probabilistic assessment of seismic hazard to define the seismic ground motion for evaluation of the interchange at the site was not within the scope of the effort. LLNL seismologists simply relied on existing geophysical data to define the Hayward Fault earthquake likely to occur in the next thirty years. Strong ground motions (rock outcrop motions) at the 24/580/980 freeway interchange in Oakland, California from potential hazardous earthquakes on the Hayward fault were developed by Hutchings et al. 1997. They used the Green's function summation approach to model large earthquakes by solving the representation relation for a finite earthquake rupture. They also applied recordings of small earthquakes at the nearby rock sites to provide empirical Green's function for frequencies 0.5 to 33 Hz, and analytical calculation to provide synthetic Green's functions for frequencies between 0.05 to 0.5 Hz. Their studies indicate that an earthquake with a moment magnitude of 7.25 that ruptures 82 km of the Hayward fault is the major hazard to the interchange structure. This independently determined magnitude matches the magnitude determined in the Caltrans hazard study. Synthesized ground motions that have been developed are for three components and the full wavetrain, and include frequencies from 0.05 to 33.0 Hz.

A suite of 100 rupture scenarios for Hayward fault earthquake were developed. Each scenario considers variations of moment, fault geometry, hypocenter, rupture roughness, rupture velocity, healing velocity, slip vector and asperity location. Moment and fault geometry were held fixed, while the other parameters were allowed to vary within estimated limits. Acceleration time histories (three components) were computed for each scenario of rupture. The median and 84^{th} percentile spectra of 100 accelerations time histories were computed, two sets of time histories whose spectra most closely match the median and 84^{th} percentile spectra of 100 accelerations time histories of all 100 rupture scenarios were used for the site response analysis. The average spectrum of two horizontal components for each set of generated time hostories was used in matching process. Figures 3-3 and 3-4 show three components of acceleration time histories and the corresponding spectra which most closely matches the median value of 100 spectra. Figures 3-5 and 3-6 show the acceleration time histories and the corresponding response spectra most closely matches the 84^{th} percentile of 100 spectra. As stated in the Introduction that Caltrans policy decisons had let to their utilization of median motion. The results of 84^{th} percentile level are presented in this report for information and reference only.

3.2 Input Geotechnical Data

Several analytical site models containing horizontal layers with homogeneous properties in each layer were developed for dynamic site analyses. Required soil parameters and properties for each layer include soil layer thickness, density, low strain shear modulus, and the variation of shear modulus and damping ratios with shear strains. The depths of water table and bedrock is also needed. The underlying bedrock are treated as a halfspace with shear wave velocity of 5000 fps. This value is probably in the upper bound for underlying bedrock of shale and graywacke sandstone. However, the higher value of shear wave velocity of the bedrock would introduce higher value of impedance ratio between the soil deposit and bedrock and leads to the results of site response in conservative side. The low strain shear modulus, density, and damping ratio of the bedrock are also required to input for calculation. A total of 41 sublayers for the soil profile (Fig. 2-5) were used in the analytical site model. The variation of shear modulus and damping ratio with shear strains for each type of soils as described in Sec. 2.3 were assigned for each sublayers. The input files for the case of analyzing LLNL median synthetic motions are attached in Appendix A.

4.0 Calculated Ground Motions

The results of site response analyses for rock-input motions described in Secs. 3.1.1 and 3.1.2 are briefly summarized in this section. The motions may be computed at any layer in the soil deposit but only the surface ground motions are needed for this project. The computed motions are presented by acceleration time histories and corresponding response spectra with 5% damping. For detail comparisons, the response spectra of the rock-input motion and computed surface motions are overplotted in the same figure. The effect of varying shear wave velocities below 290 ft from the ground response was investigated by using modified Santa Cruz earthquake records. Investigations of the effects of modeling uncertainty of ground response were only performed on LLNL median synthetic motions.

4.1 For modified Santa Cruz Earthquakes Records

The computed ground surface motions for the best estimate model using rock-input motions (Fig. 3-1) of modified Santa Cruz earthquake records are shown in Fig. 4-1. The corresponding response spectra of 5% damping are shown in Fig. 4-2. The calculated peak ground accelerations (PGA) in both horizontal directions are 0.46 g (in the direction of H00) and 0.42 g (in the direction of H90) versus the input motions of 0.45 g (H00) and 0.51 (H90). The vertical peak ground acceleration is 0.43 g versus the input of 0.44 g. Comparisons of response spectrum of each component are shown in Figs. 4-3 to 4-5. It is observed that the high frequency contents above 2.7 Hz (or 0.37 seconds) are considerably filtered by the local soils of the site. On the other hand, the frequency content below 2.7 Hz are considerably amplified.

It has been mentioned that the best estimate shear wave velocity was taken from the average of measured values available from all boreholes around the site. The shear wave velocity below the depth of 290 ft was only measured from one deep boreholes located in the central area of the site. Therefore, the shear wave velocity measured from this borehole was adapted as the best estimate values in the computation model. In order to investigate the effect of variation of shear wave velocity on the calculated surface motion, a second model was developed for sensitivity study. The second model used the weighting average shear wave velocity of 2000 fps to represent the soil profile below 290 ft all the way to the bedrock. The 2000 fps of shear wave velocity was also used by Caltrans for the lower part of soil profile for site response analysis. The calculated responses from the H00 component for the best estimate and the second model are shown in Fig. 4-6. The effect of the shear wave velocity variation in the lower part of soil deposit on the surface response is not significant.

4.2 For LLNL Median Synthetic Motions

Three components of LLNL median synthetic motions (Fig. 3-3) were used as rock-input motions for calculations of site response at the interchange site. The calculated ground surface motions are shown in Fig. 4-7 for acceleration time histories and in Fig. 4-8 for 5% damping response spectra. The peak ground accelerations are 0.4 g (HA.145), 0.52 g (HA.235), and 0.42 g (vertical) with respect to the input motions of 0.32 g (HA.145), 0.41 g (HA.235) and 0.31 g (vertical). The comparisons of response spectra for rock-input and calculated soil surface for each component are shown in Figs. 4-9 to 4-11. Although the frequency content of the rock-input motions are considerably different from those of modified Santa Cruz earthquake records, a similar phenomenon is observed. The local soils considerably damped out the high frequency contents above 2.7 Hz (0.37 seconds) and amplified the spectral accelerations below 2.7 Hz.

In calculation of the site response to the vertical component of rock-input motions, the final iterated shear modulus in each layer has to be converted to constrained modulus by the assumed Poisson ratios of the layer. The site response analysis was performed again using the vertical input motion and converted constrained moduli of the system. The degradation of modulus and attenuation of damping with strain levels were assumed as the same shapes of shear wave analysis. However, since the damping curve used in the program is not a normalized curve (i.e. the actual value of damping ratio due to shear wave), the final iterated damping values in each sublayer needs to be converted in accordance with the following relationships for the p-wave

$$\alpha = \frac{4}{3}\left(\frac{V_s}{V_p}\right)\beta$$

where α is the damping ratio due to P-wave,
β is the damping ratio due to shear wave,
V_p is the p-wave velocity,
V_s is the shear wave velocity.

The computer input files for calculation of site responses of shear wave and p-wave using horizontal (H00) and vertical components of LLNL median synthetic motion are shown in Appendix A.

4.2.1 Uncertainties in Site Response Analysis

The modeling uncertainty in site response analysis was assessed by varying shear modulus in each sublayer in accordance with the latest ASCE Standard (ASCE-4, 1997). In lieu of a probabilistic evaluation of uncertainties, an acceptable method to account for uncertainties in site response analysis is to vary the soil shear modulus. The soil shear modulus should be varied by a factor of one standard deviation of the low strain shear modulus if sufficient, adequate soil investigation data are available to establish the value of the standard deviation. When available data are insufficient to address uncertainties of soil properties, the variation factor should be taken as no less than 1.0. At this site a factor of two in low strain shear modulus was used to assess the response uncertainties. The calculated responses for the cases of best estimate, low bound and upper bound shear modulus are shown in Fig. 4-12. The effect of low strain shear modulus on calculated ground surface motions is important.

4.2.2 Comparisons of Calculated motions with Caltrans Evaluation Motion

The horizontal components of LLNL median synthetic motions and its corresponding component of calculated soil surface motions are overplotted and shown in Fig. 4-13 as well as the Caltrans design/evaluation motion by Gates, (1992). From the comparisons of the calculated motions and Caltrans design/evaluation motions, it can be seen that the spectral accelerations are compatible for most frequency ranges important to the interchange structure. Calculated peak ground accelerations are also compatible with Caltrans motion. In high frequency ranges, Caltrans design motions envelop the LLNL median rock motions. In low frequency range (about 1.8 seconds), the calculated motions are higher than those of Caltrans motions due to amplification of local deep soil deposit. Since the predominant period of the interchange structure is less than 1.8 seconds, the effect of these motions on the seismic response of the structure is probably insignificant.

4.3 For LLNL 84th percentile Synthetic Motions

Dynamic site response analyses were first performed for the interchange site using LLNL 84th percentile rock-input motions. The calculated ground surface motions were used for the LLNL first independent seismic evaluations of the interchange structure. The independent evaluations were made in the absence of any information regarding Caltrans policy decisions on the classification of the structure. Caltrans policy establishes performance levels for all structures based on importance, serviceability, and damage levels. Later information provided by Caltrans regarding their policy for this particular structure at this site led to LLNL reassessment of the structure based on LLNL median synthetic motions.

Since the work has been done, the results are briefly documented in this report only for the purpose of information and reference. The calculated surface motions are shown in Fig. 4-14 and the corresponding spectral with 5% damping are shown in Fig. 4-15. The comparison of 84th percentile rock-input motion and the calculated motion for each component are shown in Figs. 4-16 to 4-18. Similar effect on local soil on the seismic ground motion are seen form these comparisons. The local soils considerably filter the frequency contents of the rock-input motion above 2 Hz but amplify the motions for frequencies lower than 2 Hz.

5.0 Summary and Conclusions

Site response analyses were conducted at the 24/580/980 freeway interchange site, Oakland to determine the seismic ground motions for LLNL independent seismic evaluations of the interchange structures. The equivalent linear procedures implemented in computer program SHAKE were used for the analysis.

Several sets of rock motions likely to develop in the base-rock underlying the site were used for the analysis. One set of the motions provided by Caltrans was modified Santa Cruz earthquake records. Another two sets of the motions were developed at LLNL using LLNL empirical Green's function method simulating seismic motions from a maximum credible earthquake with moment magnitude 7.25 from Hayward fault which is about 4 km away from the site. A total of 100 scenarios of fault's rupture were developed for 100 sets of acceleration time histories. Each set consists of two horizontal and one vertical components. Statistical analyses were performed to have the median and 84th percentile spectra. The motions whose spectral shapes most close match to the median and 84th percentile were selected for the input motions of site response analysis. The site response analysis were conducted on both median and 84th percentile input because prior to LLNL's independent evaluation of site ground motion, LLNL did not receive a definitive policy statement from Calstrans staff on whether the analysis/evaluation was based on median or 84th percentile motion. As the LLNL seismic study progress, and LLNL presented ground motion results to Caltrans staff, it was determined that Caltrans policy decisions had led to their utilization on median motion. Consequently, the emphasis of the LLNL structure response evaluation were placed on consideration of the median level of earthquake motion.

Generalized soil profiles were developed using geotechnical data obtained from boring tests conducted in 1960, 1991 and 1995. The average shear wave velocity obtained from shear wave velocity measurements acquired in five test boreholes was used as the best estimate of site model for site response analysis. The density, soil type, plasticity index for each layer were estimated from the results of laboratory tests on samples taken from boring holes. The dynamic characteristics of shear modulus and damping at various shear strain levels were assessed from those of similar soils published in literatures. The effect of confining pressures on damping characteristics of sandy and gravelly soils was included based on our experiences and judgments. Uncertainties in site response analysis were investigated in accordance with the guidelines provided in latest ASCE standard. Calculated response spectra on ground surface of the site for LLNL median synthetic motions were compared with Caltrans design/evaluation spectra.

Several conclusions may be made based on the results of site response analyses at the 24/580/980 site in Oakland.

1. The frequency contents (spectral accelerations) above 2.7 Hz of input baserock motions either from LLNL median synthetic motion or from modified Santa Cruz earthquake records are considerably filtered by the local soils, but the rock motions below 2.7 Hz are amplified considerably. Peak ground accelerations (at period of zero second) are slightly amplified for LLNL median motions and remain similar for modified Santa Cruz earthquake records except the horizontal component in H90 direction. In this particular component the peak ground acceleration of 0.51 g which is much higher than others would probably induce much higher shear strains in soil deposits during earthquake shaking and thus get higher damping to damp out the response. This type of reduction is consistent with the median relationship recommended by Idriss (1990).

2. The results of site response analyses using LLNL 84th percentile baserock motion indicate similar effects occurring on motions above and below frequency 2 Hz. Equivalent linear method of modeling nonlinear soil response tends to result in overdamping of higher frequencies. This can suppress some higher frequency spectral peaks to some extent. Time domain nonlinear analyses may be needed to perform for this site for 84th percentile baserock motions.

3. The calculated response spectra at the ground surface of this site using LLNL median baserock motions are generally compatible with Caltrans design/evaluation motions. The interchange has been classified as a non-collapse structure (non-service level requirement) according to Caltrans classification criteria for the site. Caltrans spectral acceleration of 1.2g between the periods of 0.25 to 1.3 seconds could be increased to 1.7 g to cover the uncertainties of lower strain shear modulus of the site. However, the upper bound shear moduli used in our calculations are based on the ASCE-4 guidelines which are basically recommendations for seismic analysis of safety related nuclear structures. It is probably too conservative for the interchange structure at this site.

4. The uncertainties in site response analysis were investigated in accordance with ASCE-4 Standard. The effect of lower bound and upper bound of low strain shear modulus on calculated response spectra is significant. The peak ground acceleration and peak spectral accelerations may increase about 15% and 40%, respectively, from those of the best estimate case.

5. Variations in shear wave velocities below 290 ft at the site were considered in two cases: One case used the shear wave velocity directly measured for each layer and other cases used overall weighting average of 2000 fps to represent all layers between 290 ft and 465 ft (bedrock). The effect of variations in shear wave velocities below 290 ft from the ground surface on site responses is not important.

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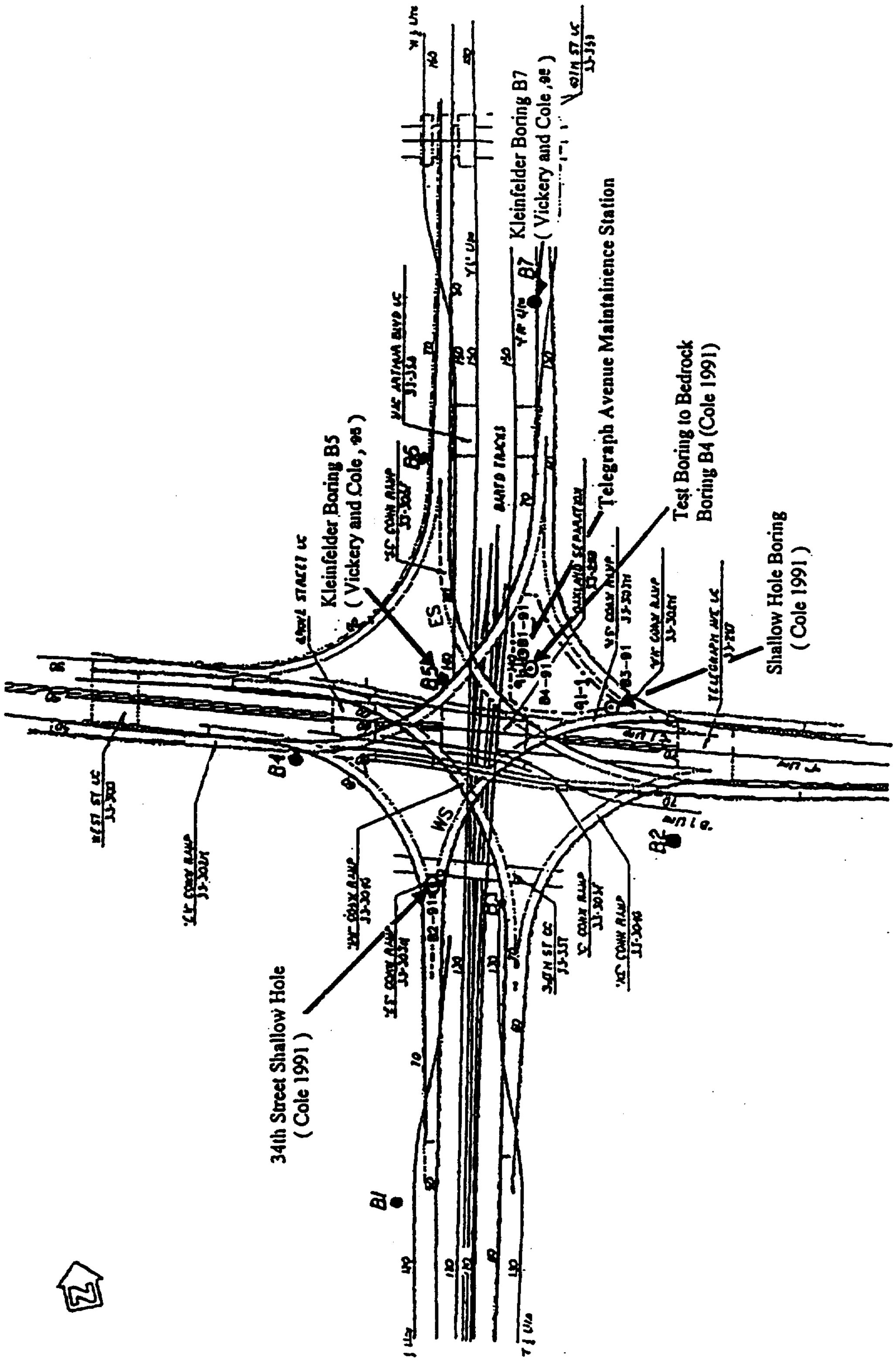


Fig. 2-1 Locations of seismic velocity measurements and test boring holes at the 24/580/980 freeway interchange site

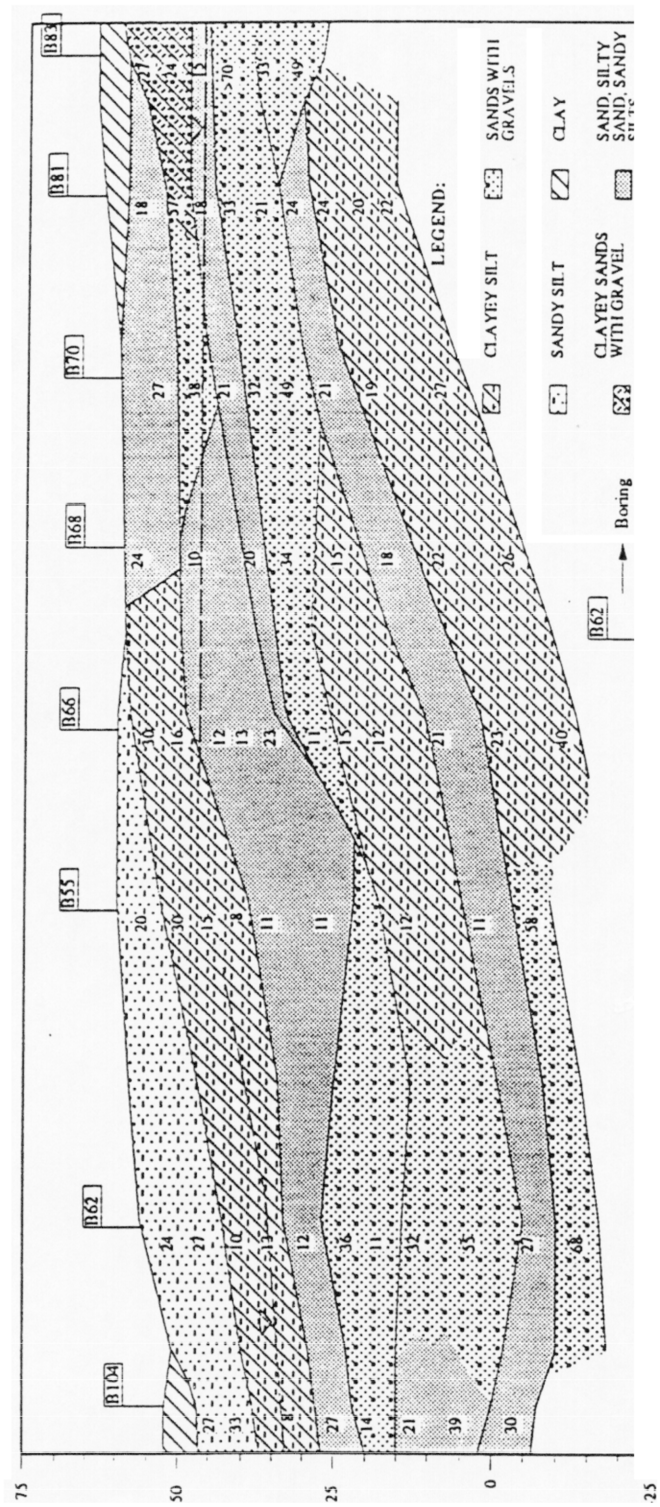


Fig. 2-2 Generalized soil profile for the 24/580/980 interchange along WS line based on 1960 boring logs, (after Abghari and Jackura, 1992)

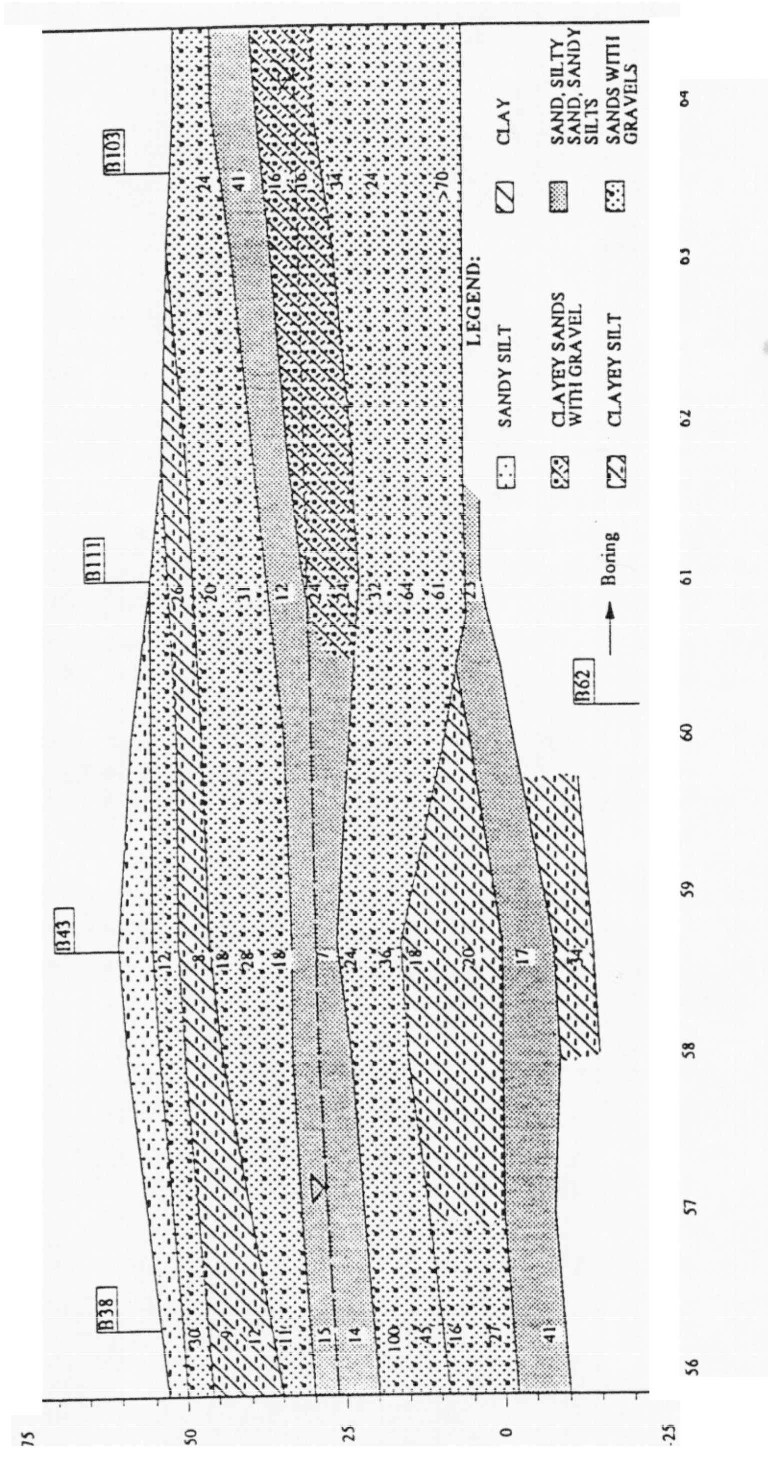


Fig. 2-3 **Generalized soil profile for the 24/580/980 interchange along ES line based on 1960 broing logs, (after Abghari and Jackura 1992)**

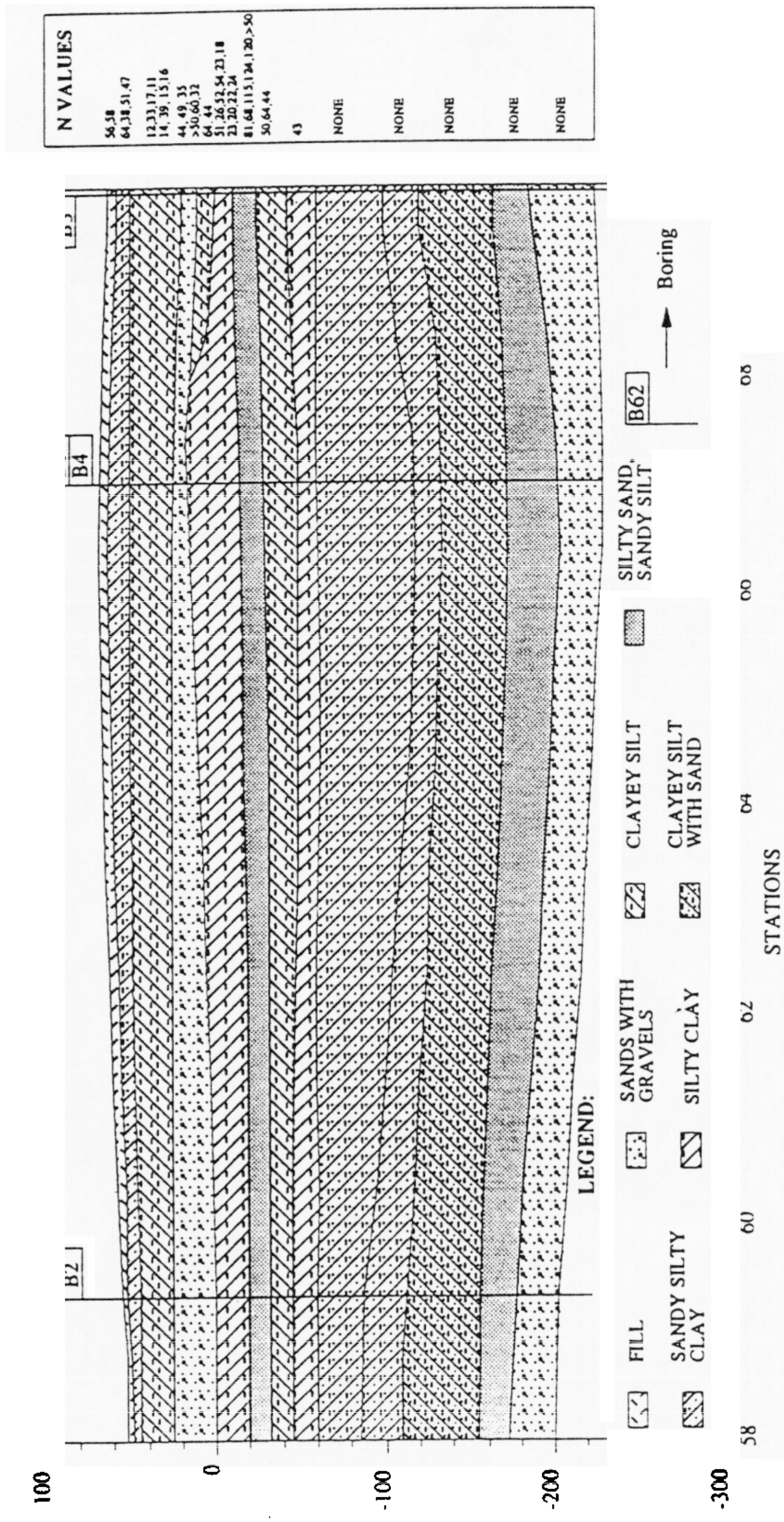


Fig. 2-4 Generalized soil profile for the 24/580/980 interchange alons WS line (after Abghari and Jackura 1992)

Measured Vs values and the Average Vs profile

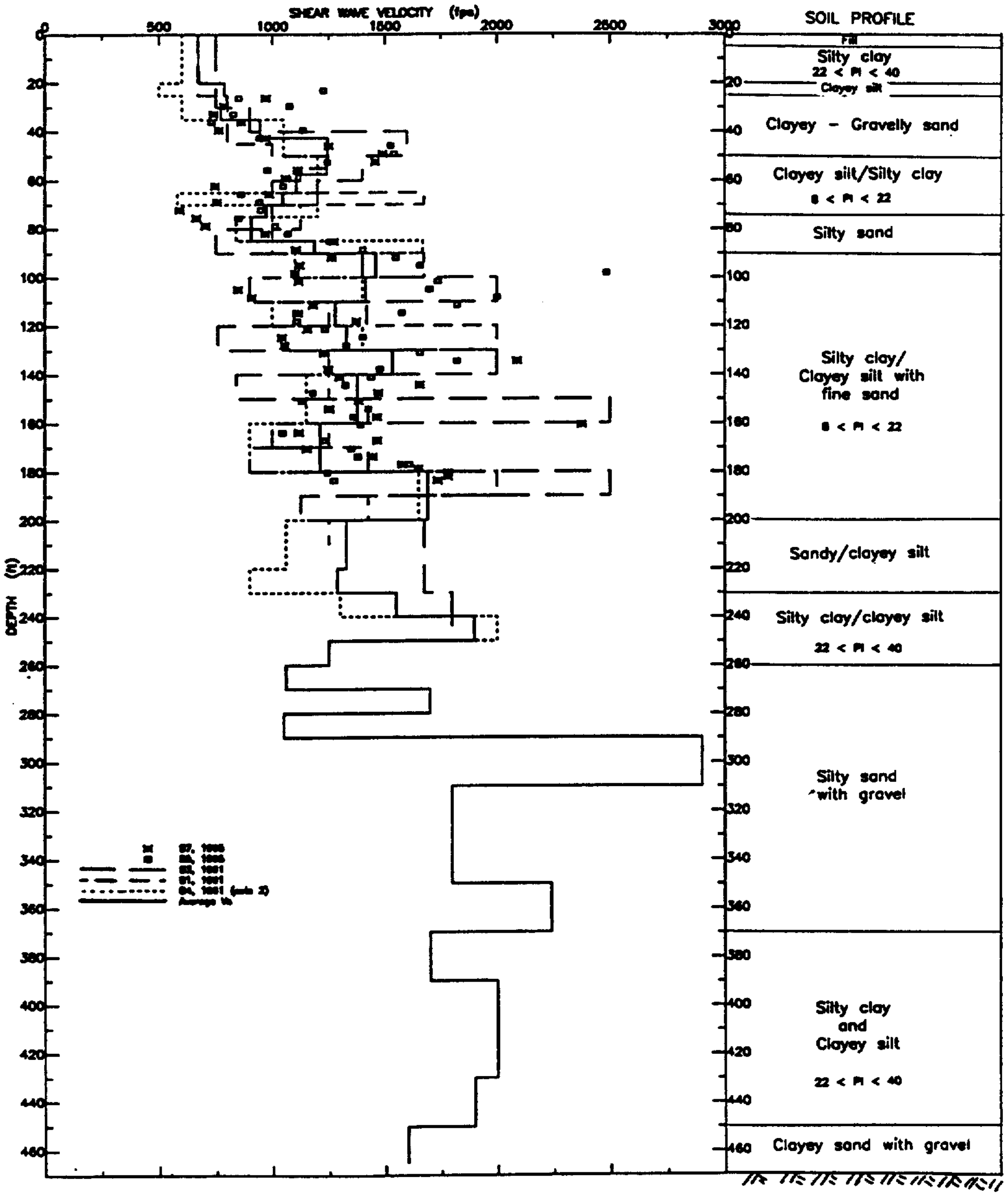
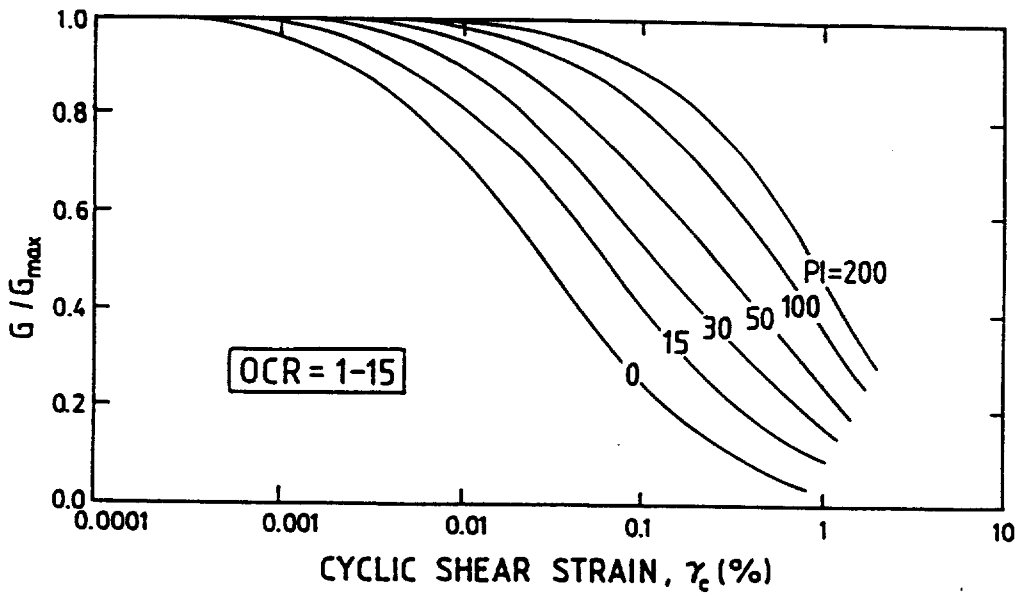


Fig. 2-5 **Measured shear wave velocities from three boreholes in 1991 and two boreholes in 1995 as well as schematic site model**



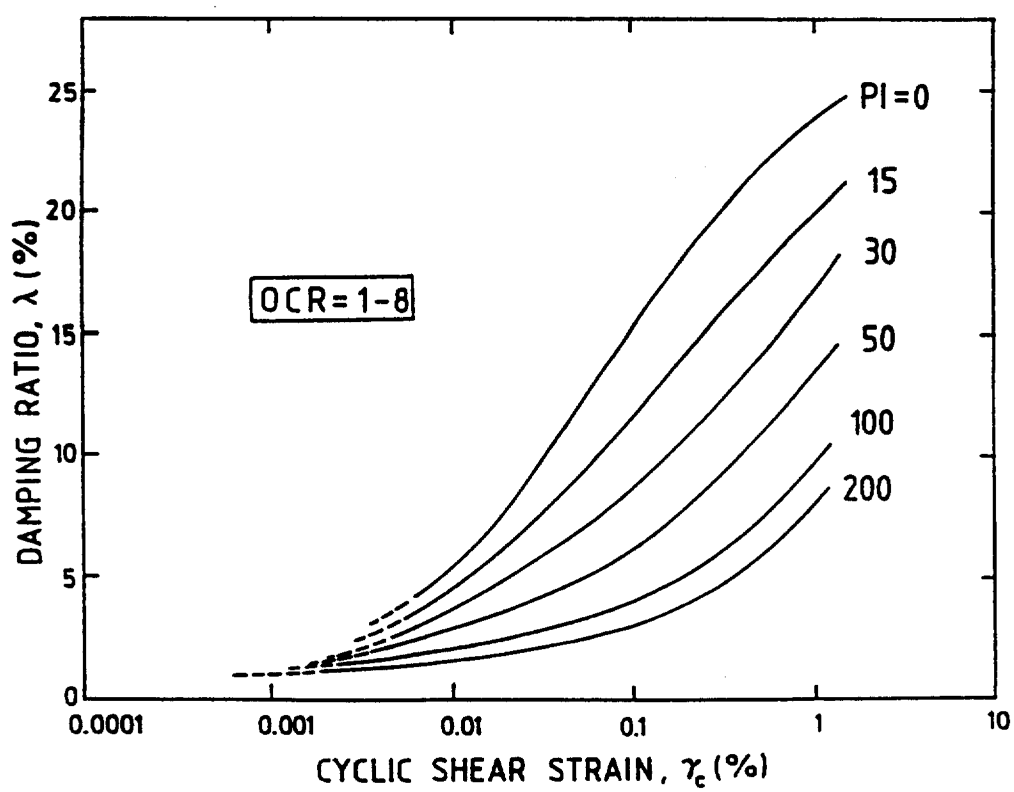


Fig. 2-6 **Normalized shear modulus (A) and damping ratio (B) versus shear strains for clay with various PIs, (after Vucetic and Dobry, 1991)**

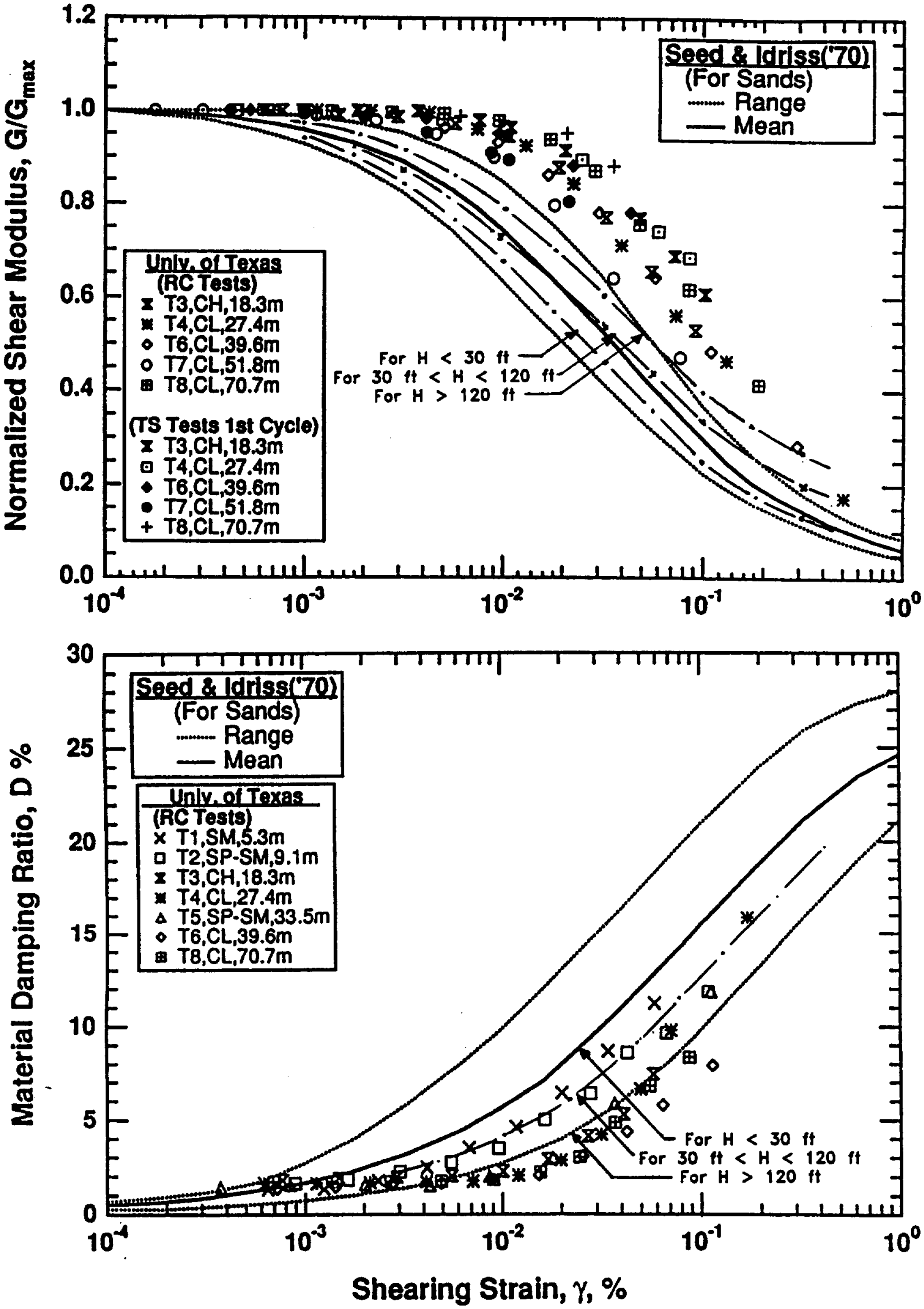


Fig. 2-7 Normalized shear modulus (A) and damping ratio (B) versus shear strains for sandy and gravely soils. Data from Seed et al. (1970), Seed et al. (1984), Sun et al. (1988), EPRI (1993)

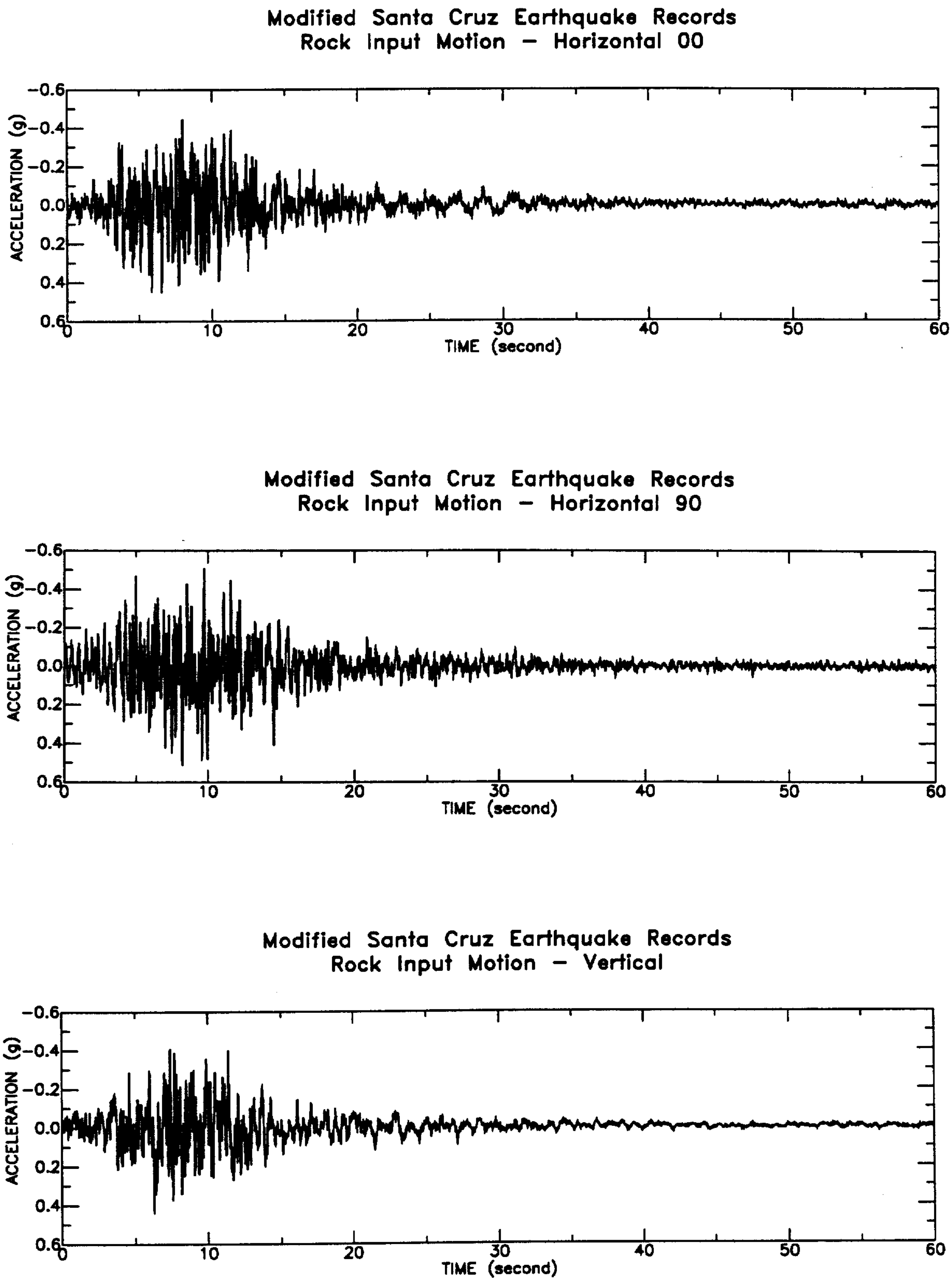


Fig. 3-1 Acceleration time histories of modified Santa Cruz earthquake records

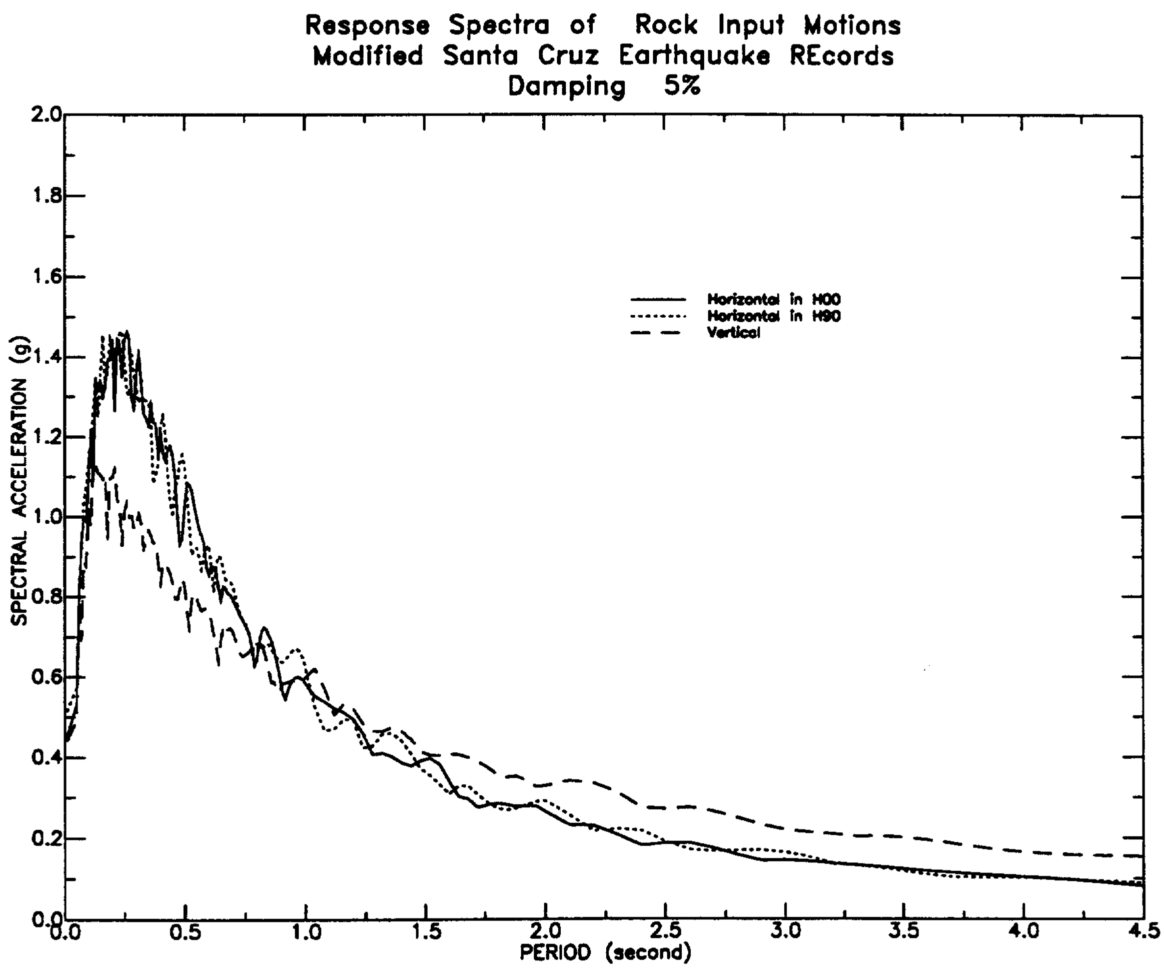
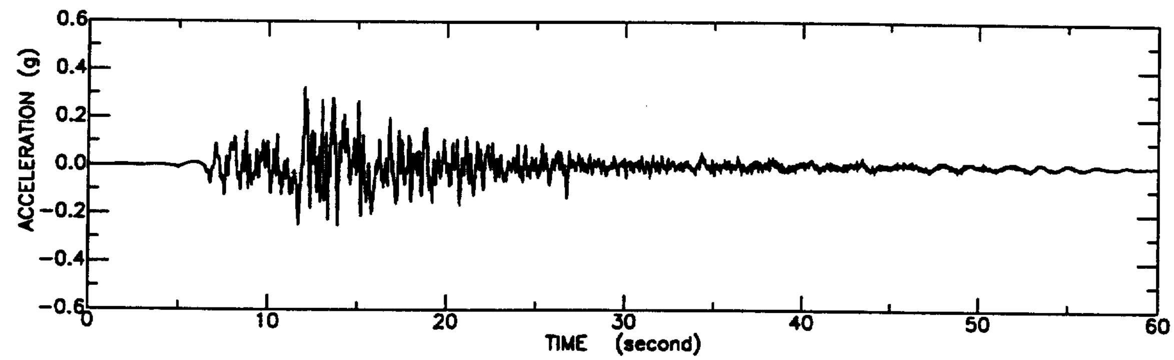
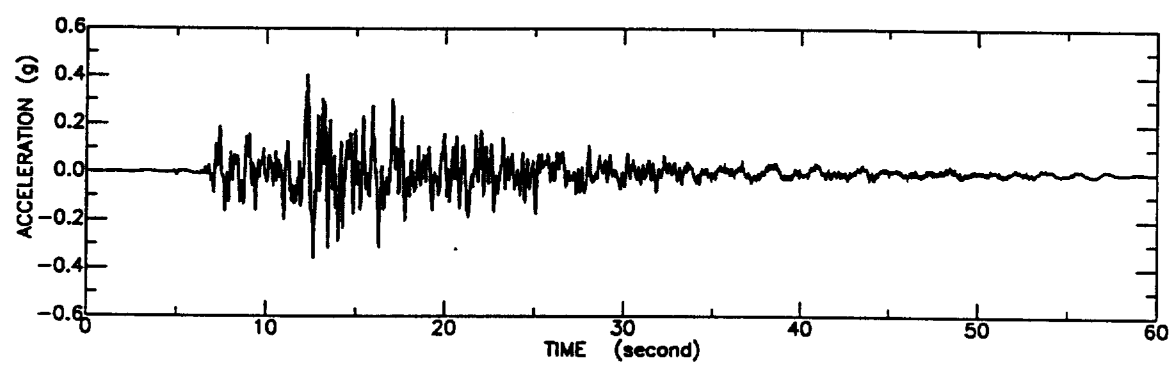


Fig. 3-2 Horizontal and vertical response spectra of modified Santa Cruz earthquake records

Synthetic Rock Outcrop Motion, LLNL Median HA.145
I-24/580/980 Interchange, Oakland



Sythetic Rock Outcrop Motion, LLNL Median HA.235
I-24/580/980 Interchange, Oakland



Synthetic Rock Outcrop Motion, LLNL Median HA.zzz
I-24/580/980 Interchange, Oakland

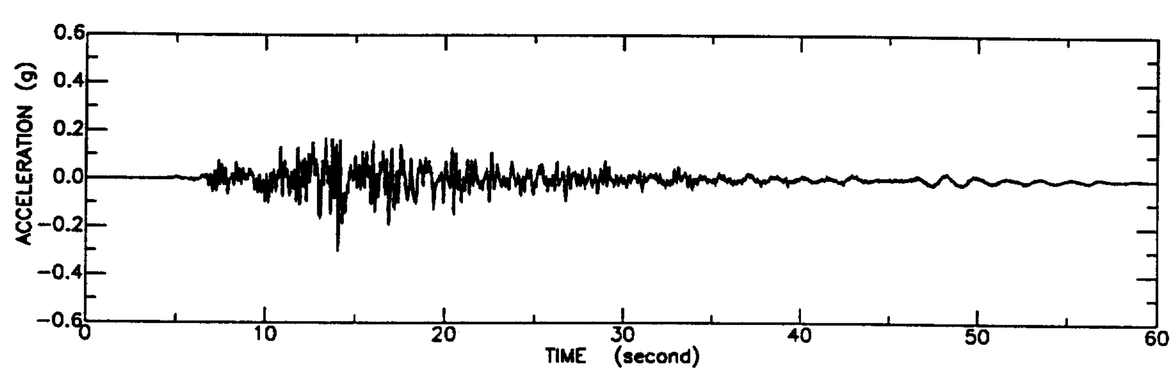


Fig. 3-3 LLNL Median Synthetic Motions: Horizontal and vertical acceleration time histories

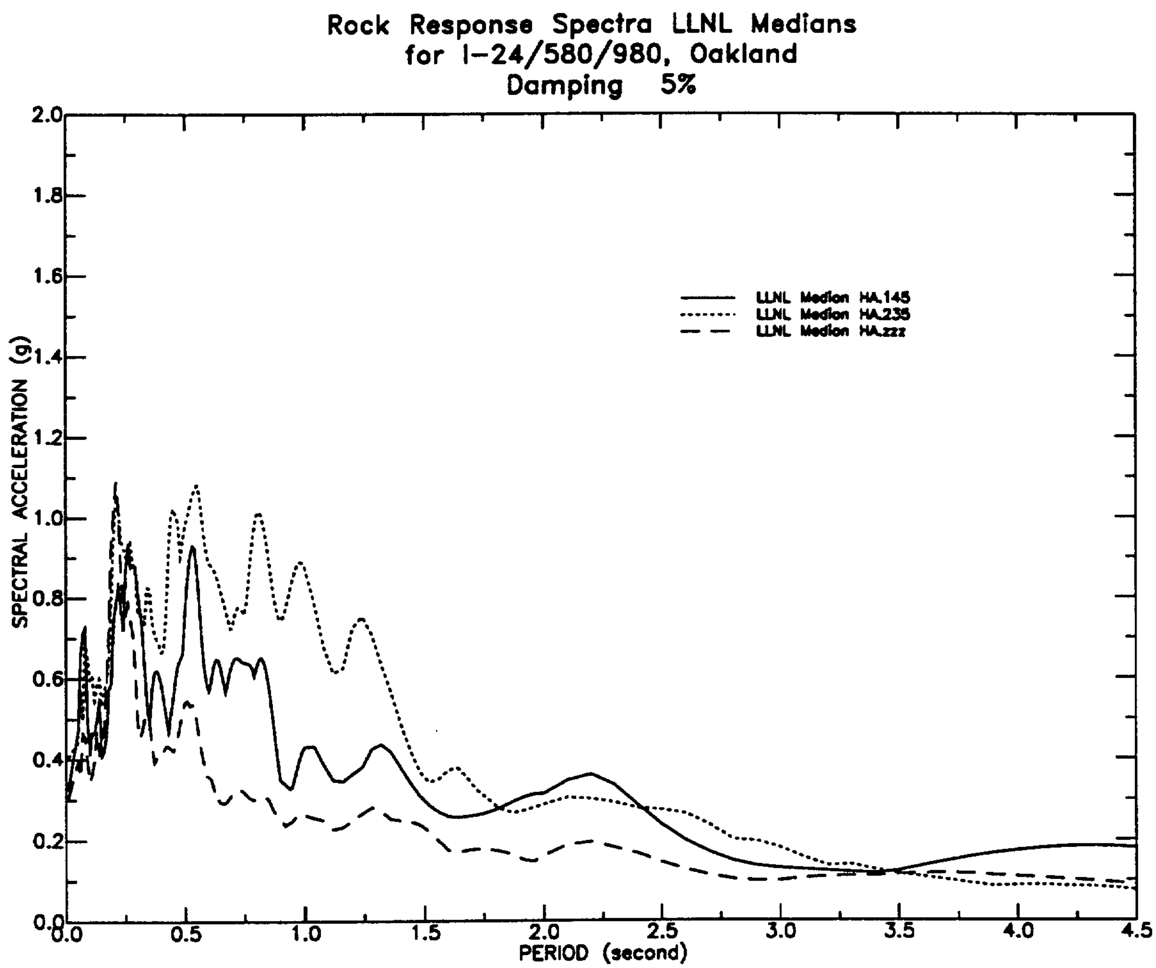
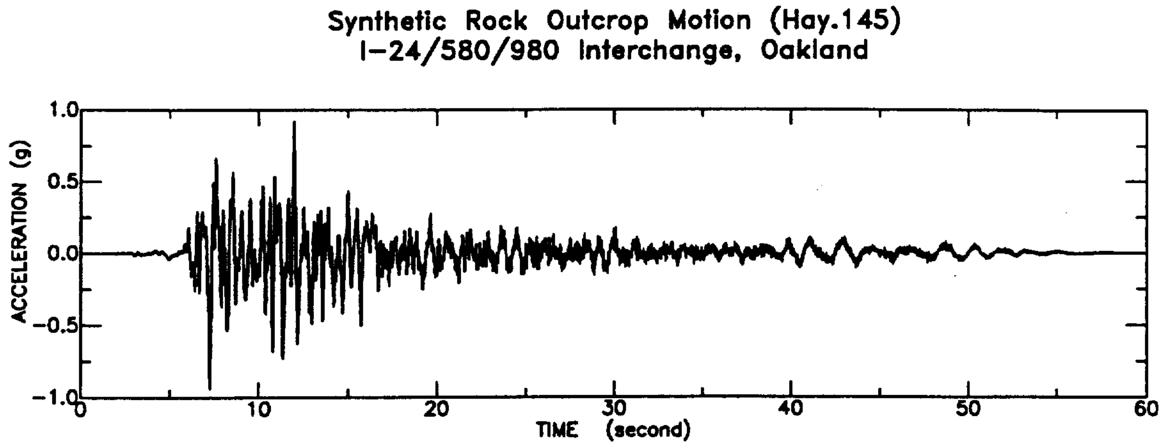
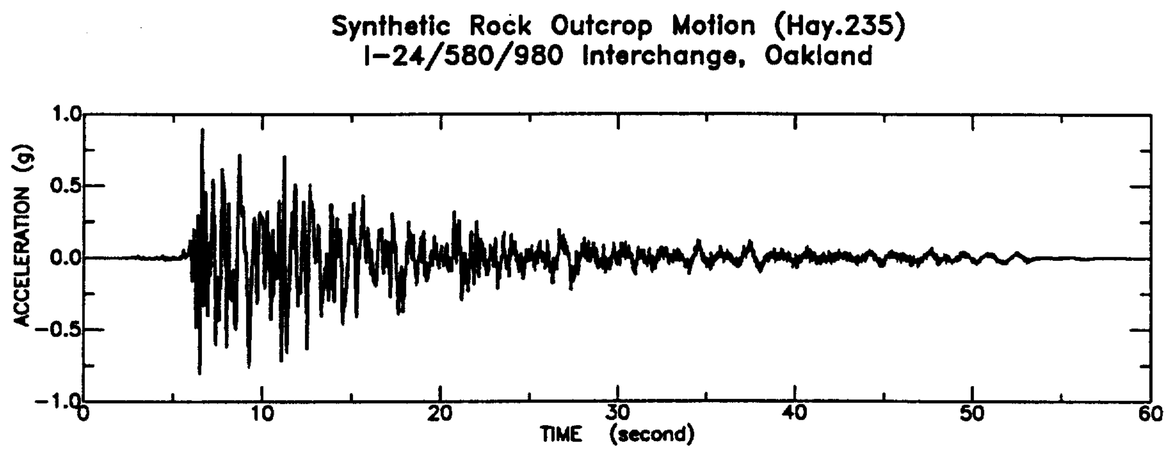


Fig. 3-4 LLNL Median Synthetic Motions: Horizontal and vertical response spectra with 5% damping





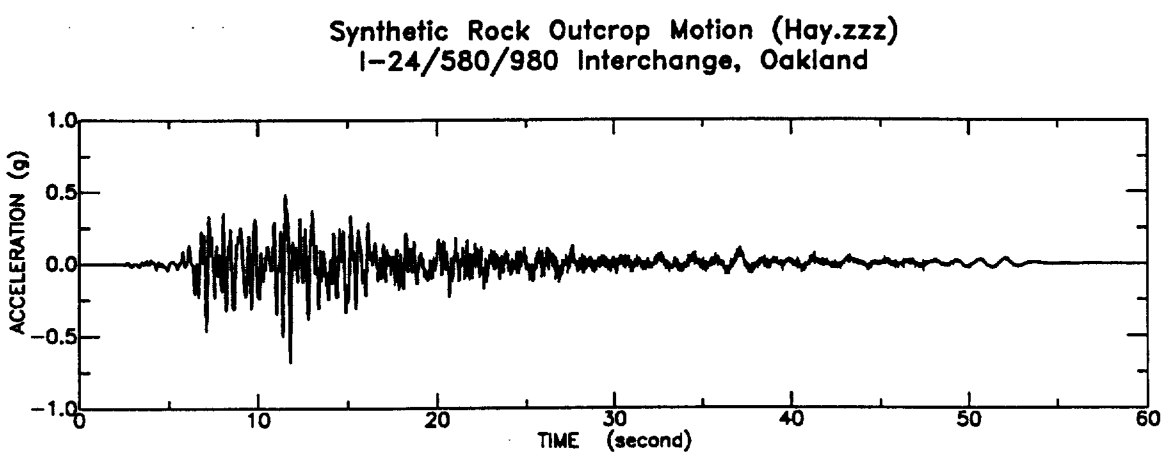


Fig. 3-5 LLNL 84th Percentile Synthetic Motions: Horizontal and vertical acceleration time histories

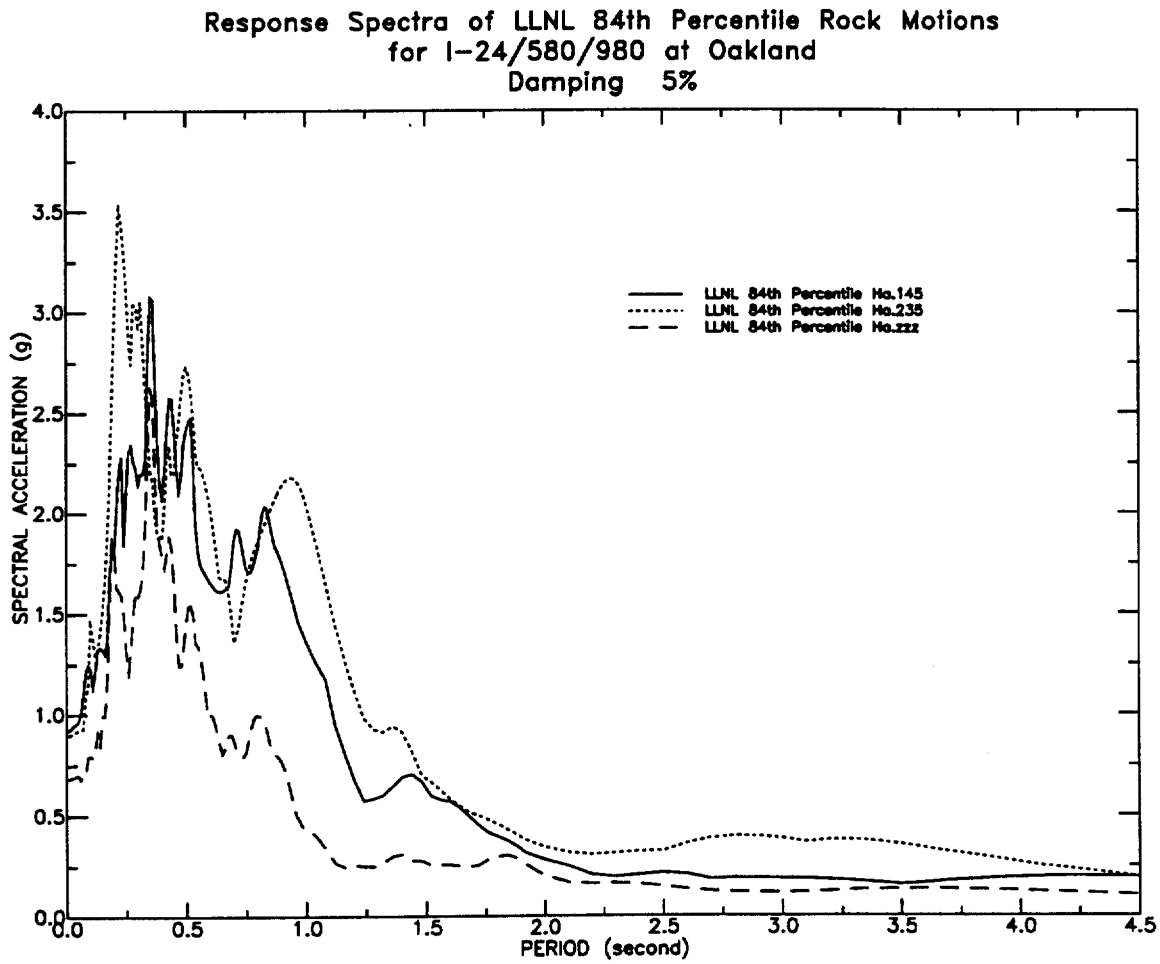


Fig. 3-6 LLNL 84th Percentile Synthetic Motions: Horizontal and vertical response spectra with 5% damping

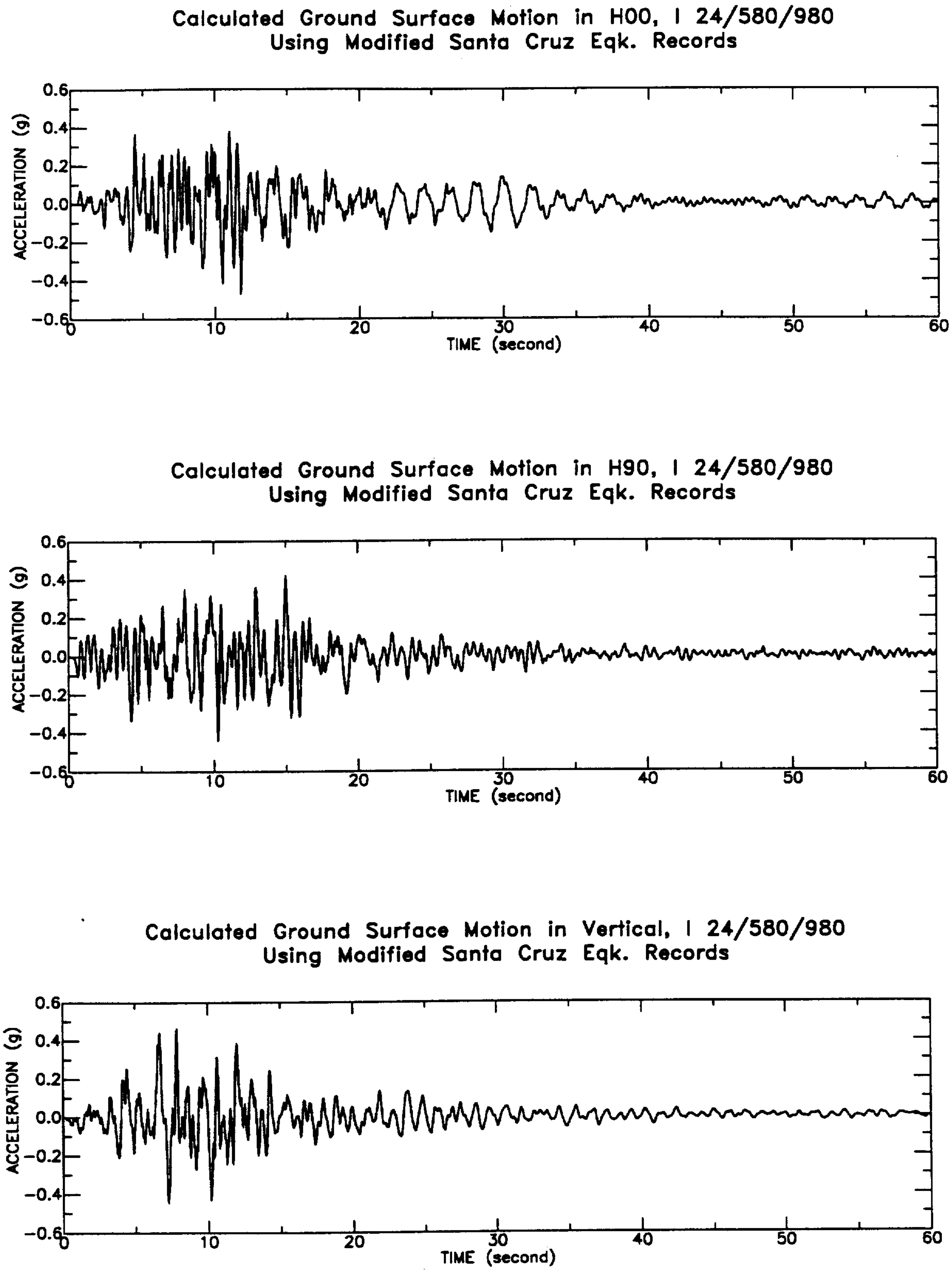


Fig. 4-1 **Computed ground acceleration time histories on the interchange site using modified Santa Cruz earthquake records**

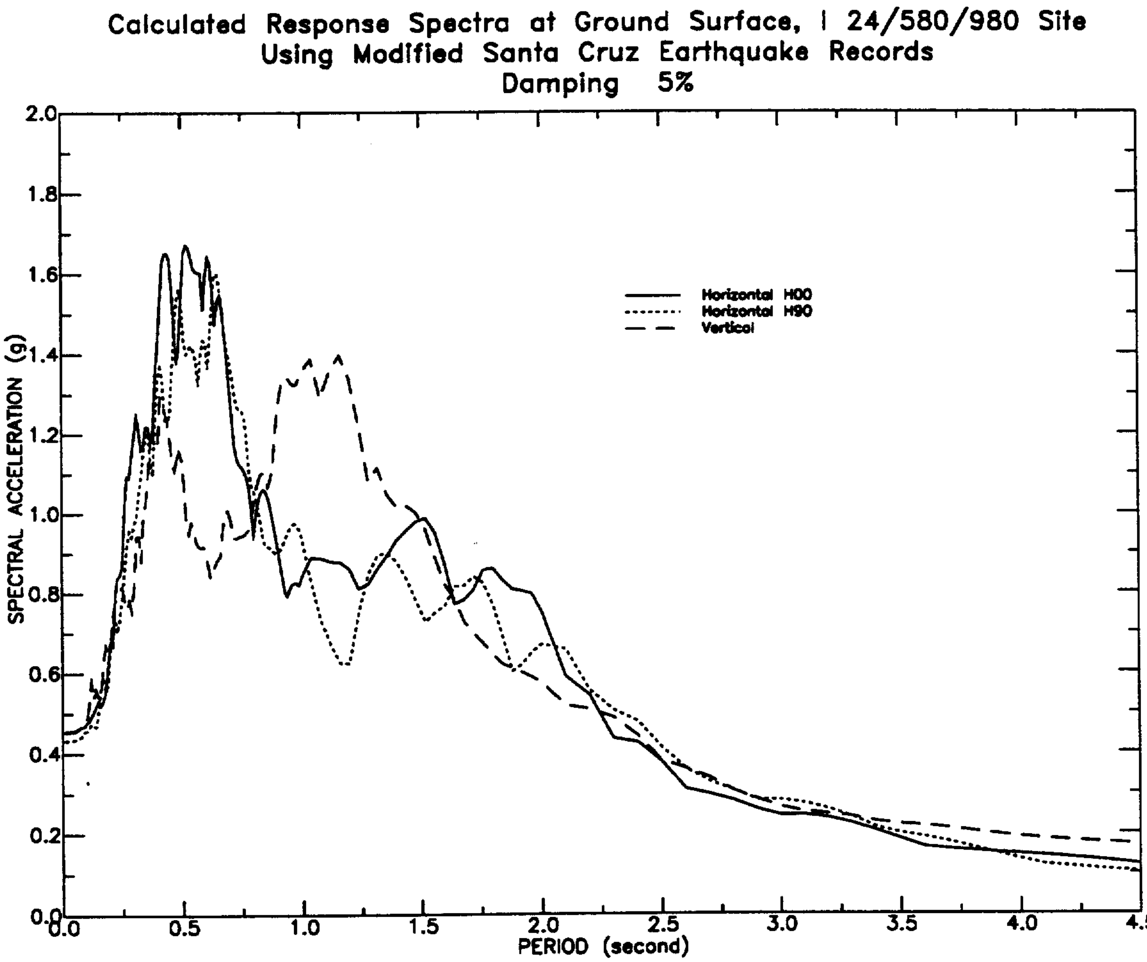


Fig. 4-2 Computed response spectra on the interchange site using modified Santa Cruz earthquake records

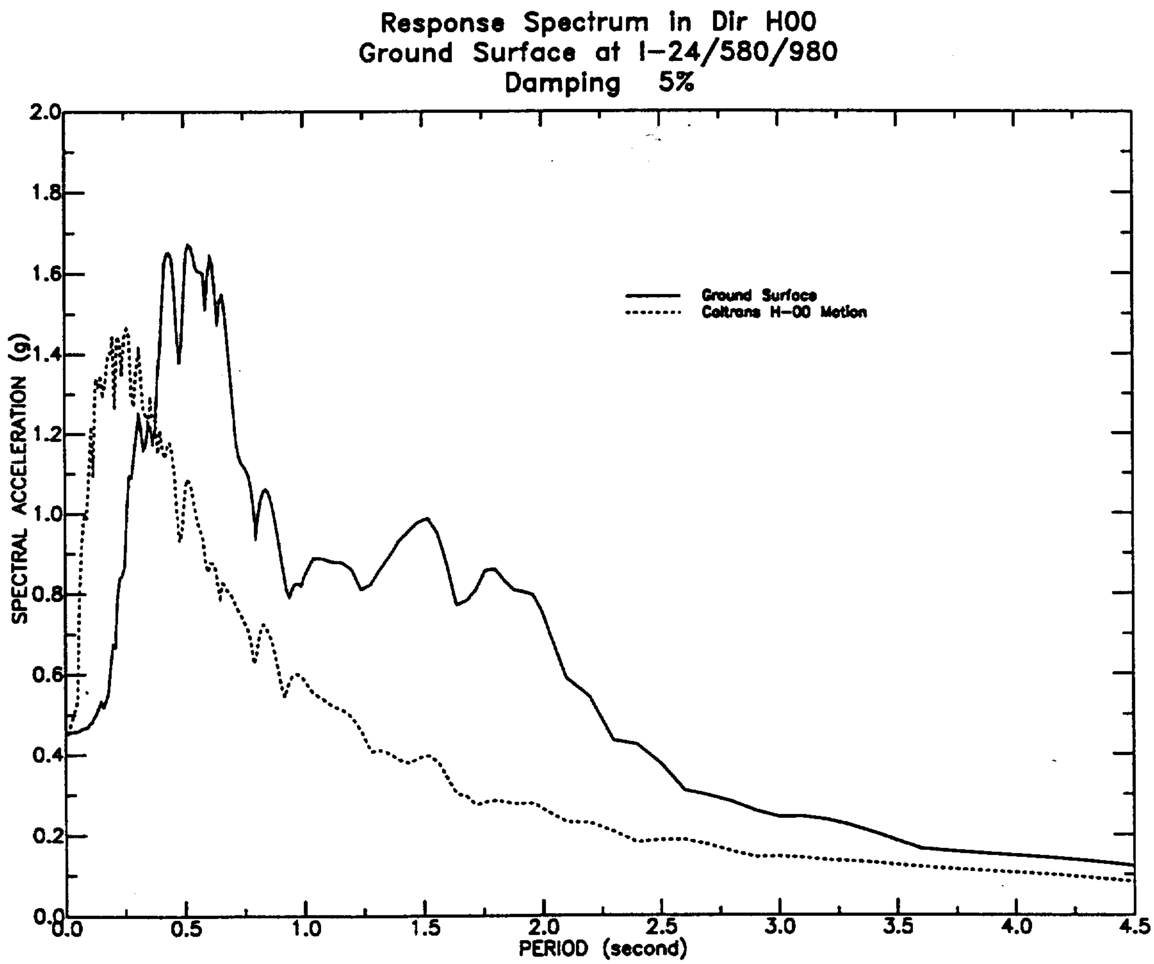


Fig. 4-3 **Comparison of input and calculated response spectra in horizontal direction of H00, modified Santa Cruz earthquake record**

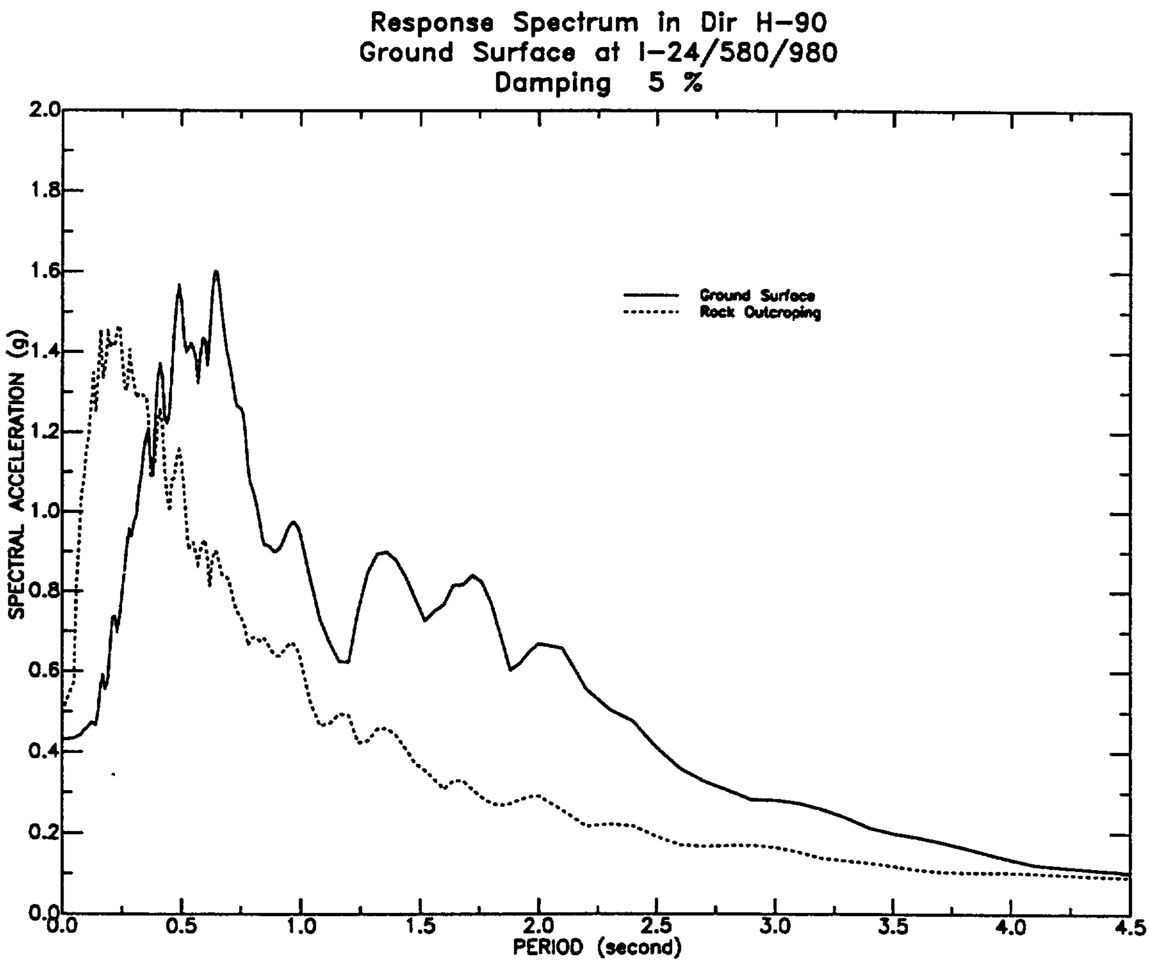


Fig. 4-4 **Comparison of input and calculated response spectra in horizontal direction of H90, modified Santa Cruz earthquake record**

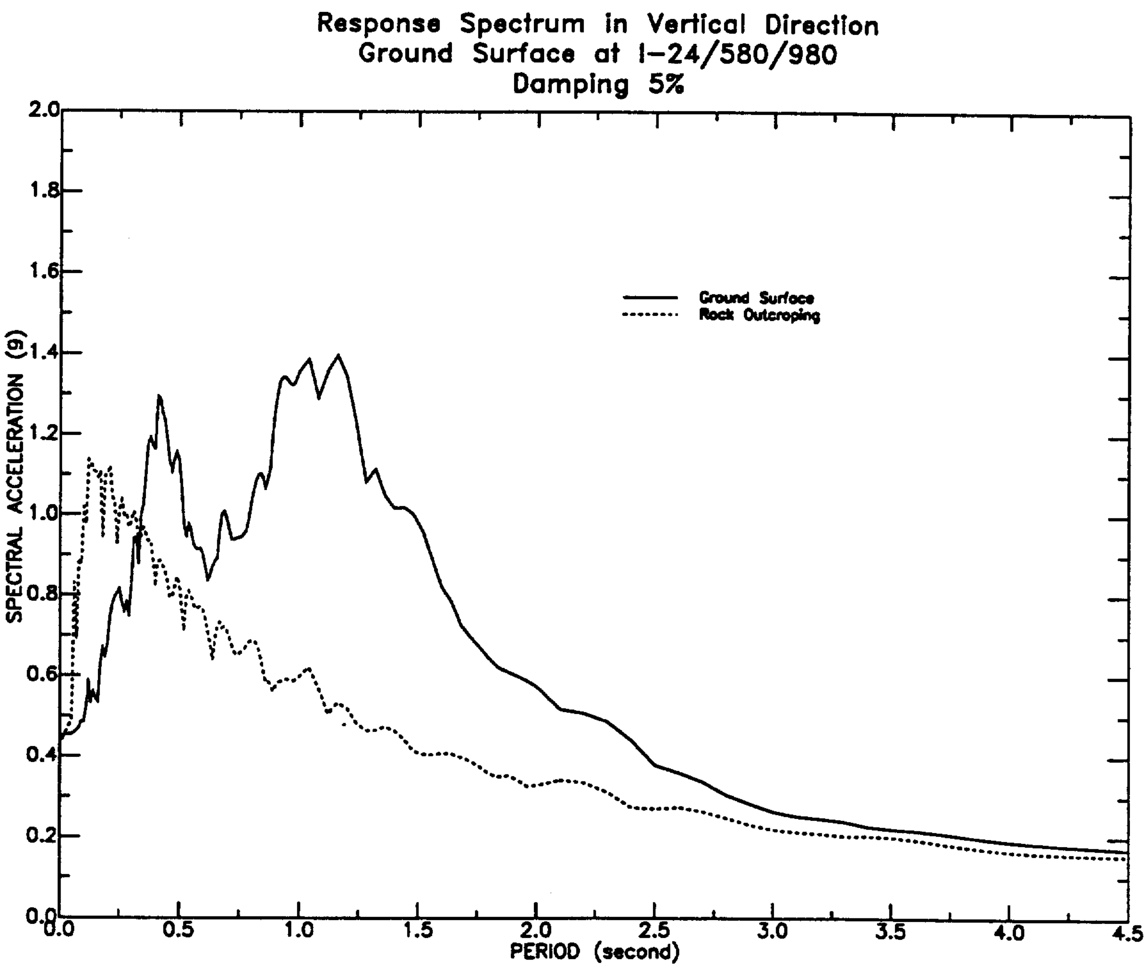


Fig. 4-5 Comparison of input and calculated response spectra in vertical direction, modified Santa Cruz earthquake record

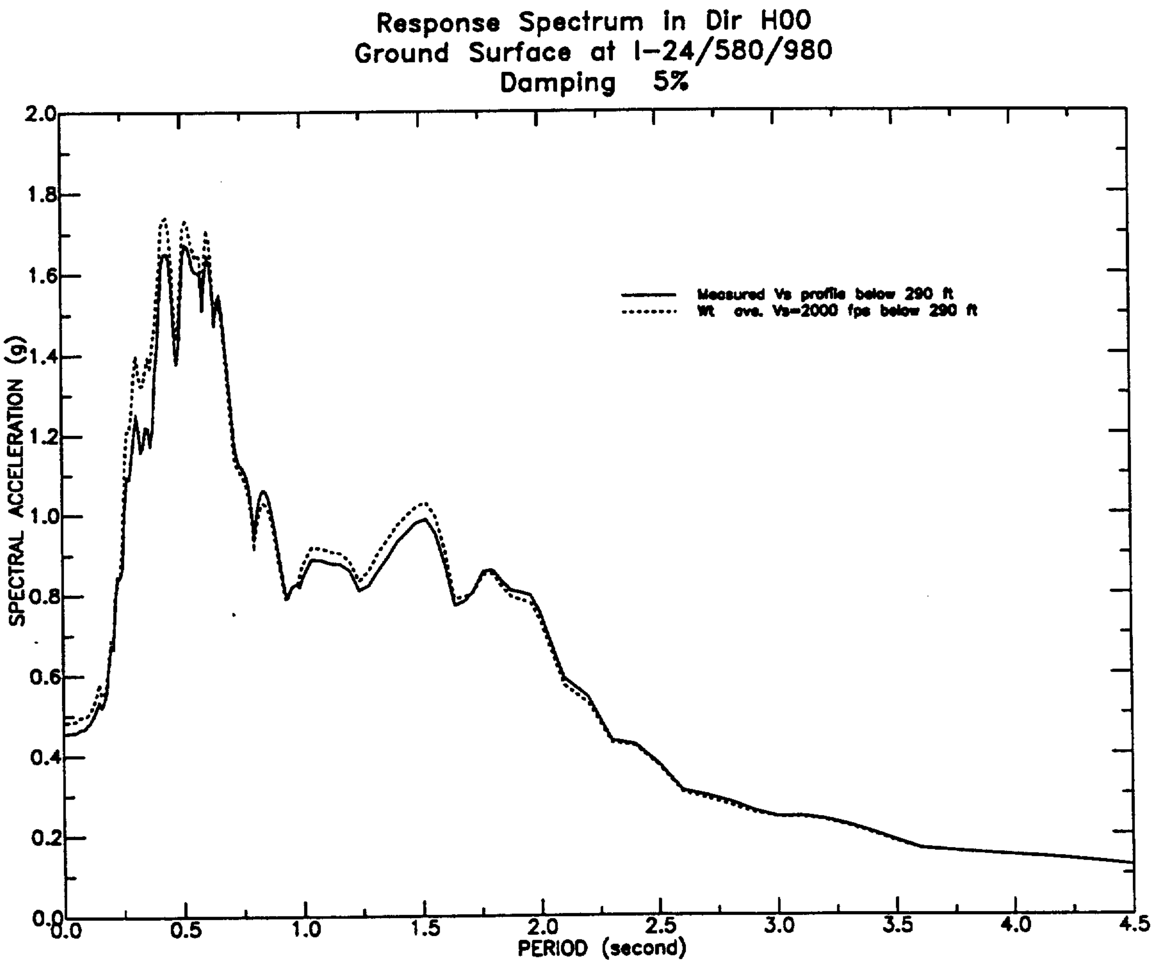
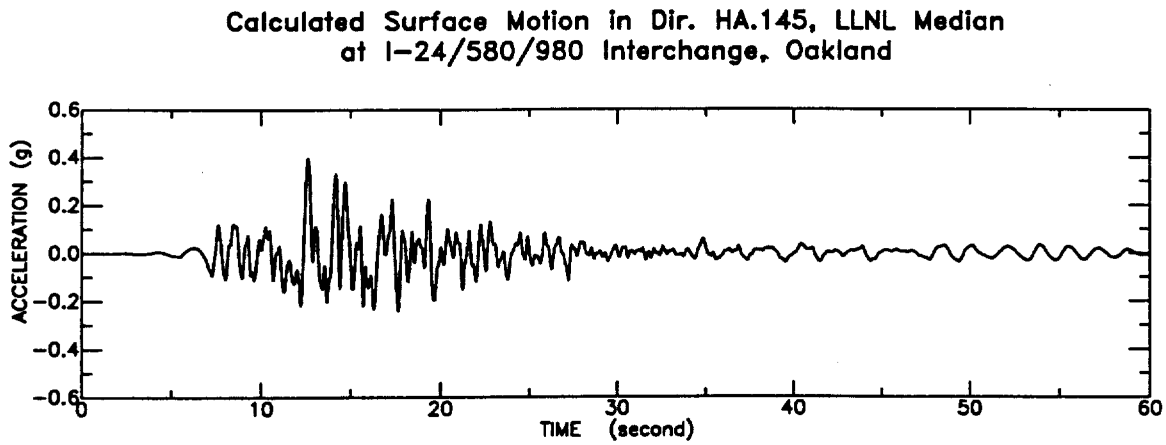
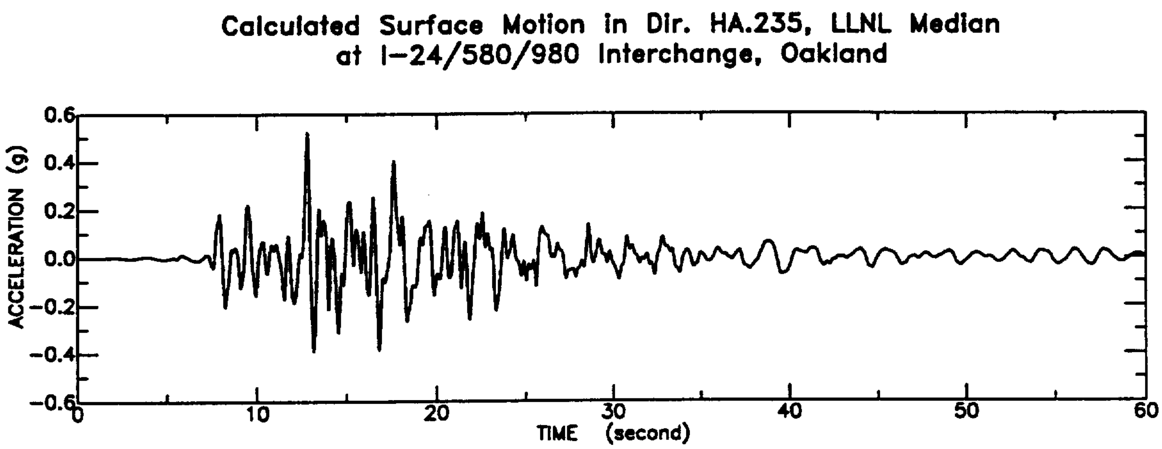


Fig. 4-6 **Sensitivity on calculated response spectra due to variation of shear wave velocity below 290 ft from ground surface**





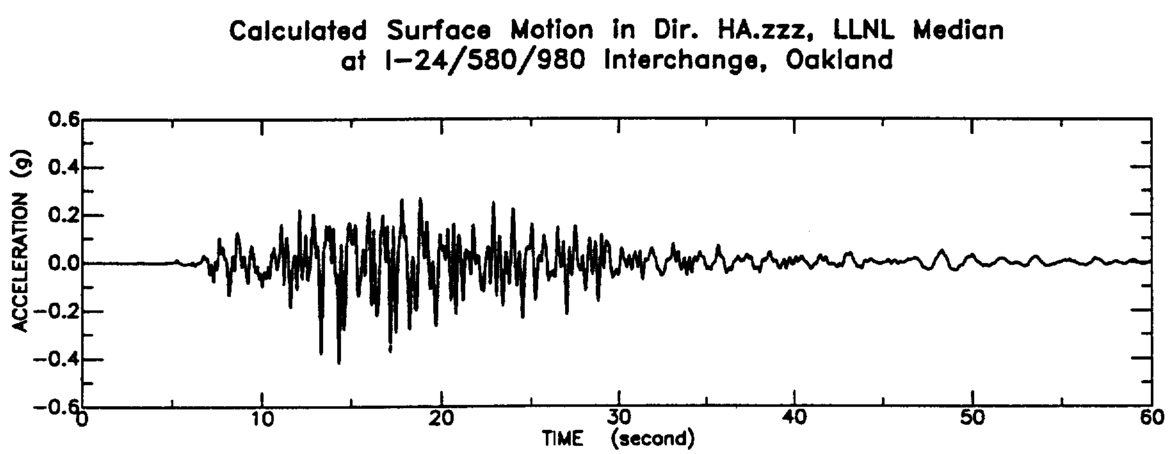


Fig. 4-7 Computed ground acceleration time histories on the surface of interchange site using LLNL median synthetic motions

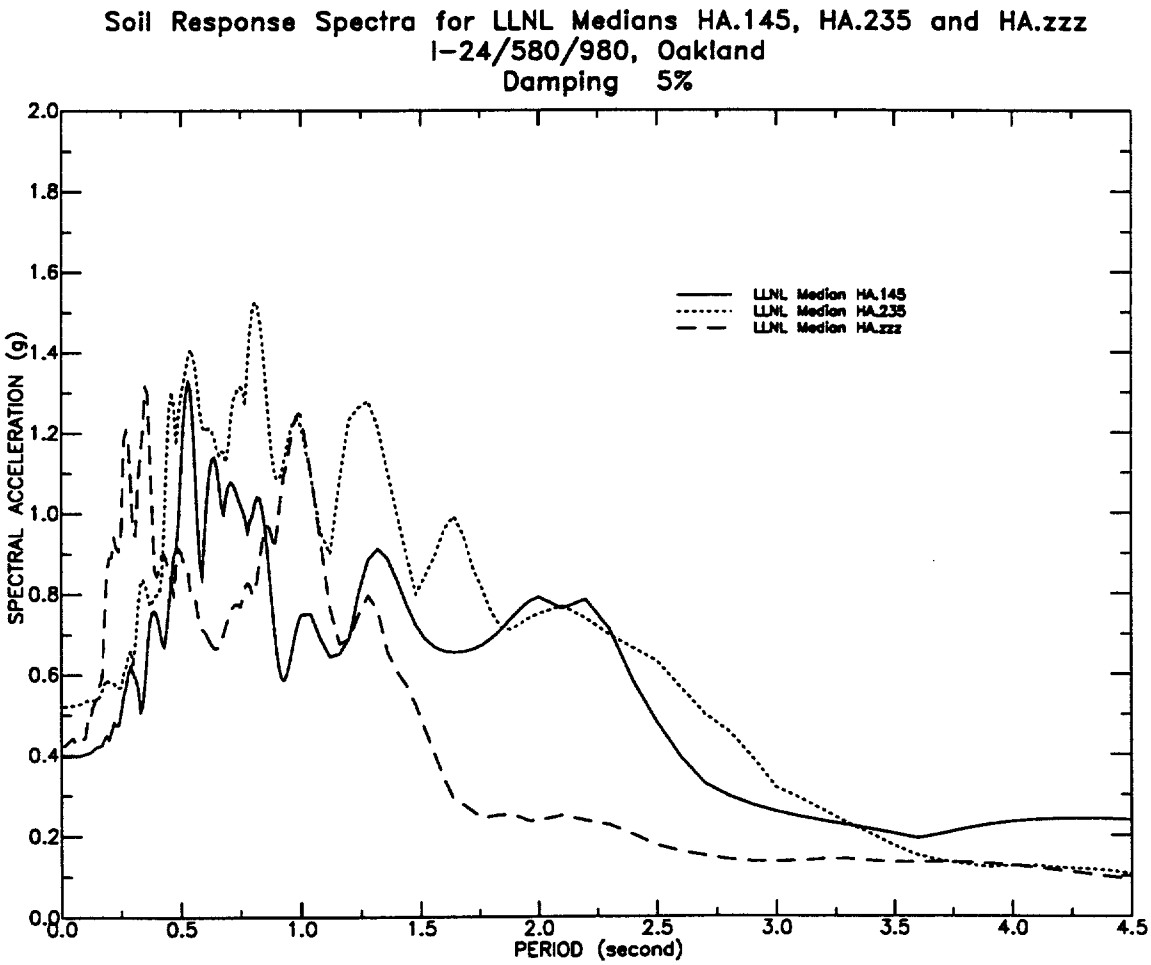


Fig. 4-8 **Computed response spectra on the interchange site using LLNL median synthetic motions**

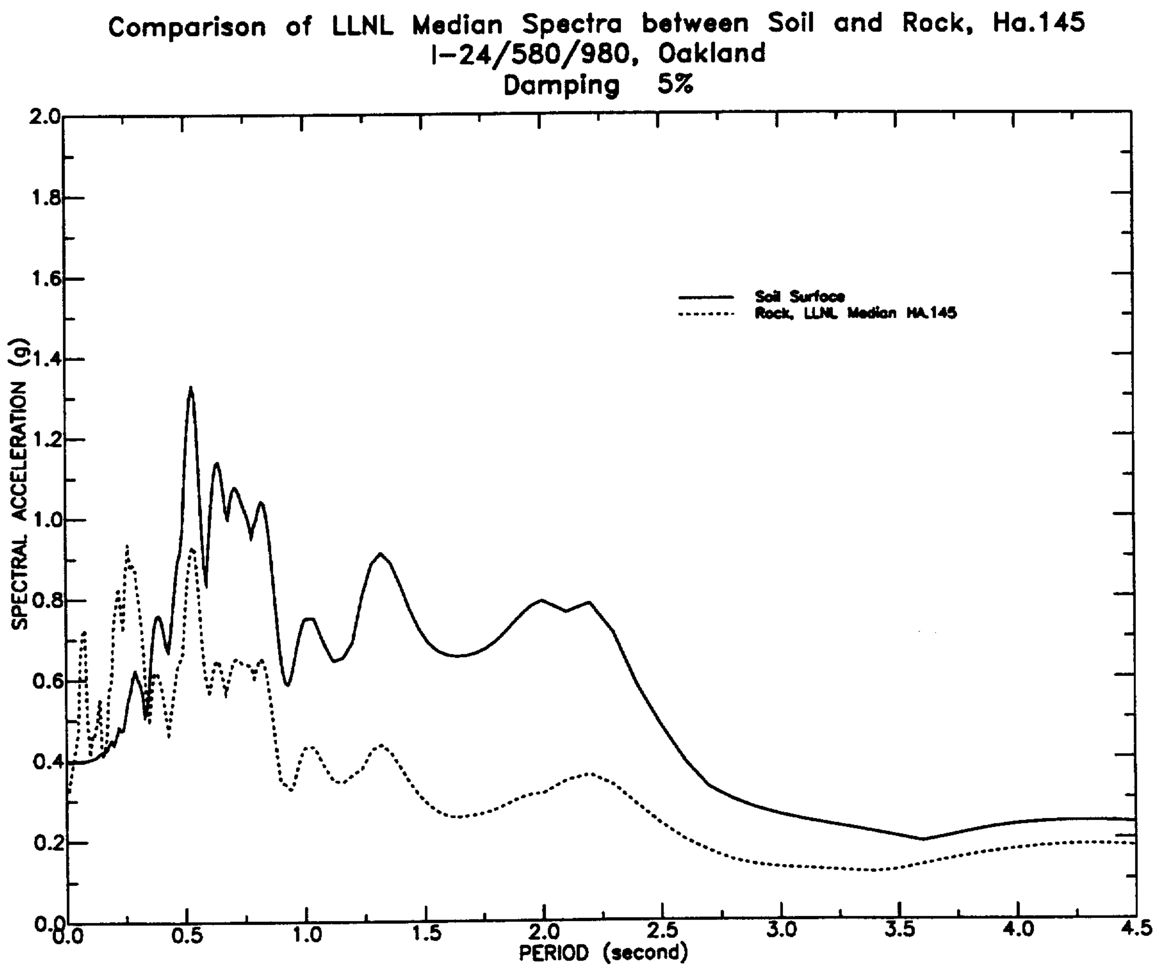


Fig. 4-9 **Comparison of input and calculated response spectra in horizontal direction of HA.145 (Median)**

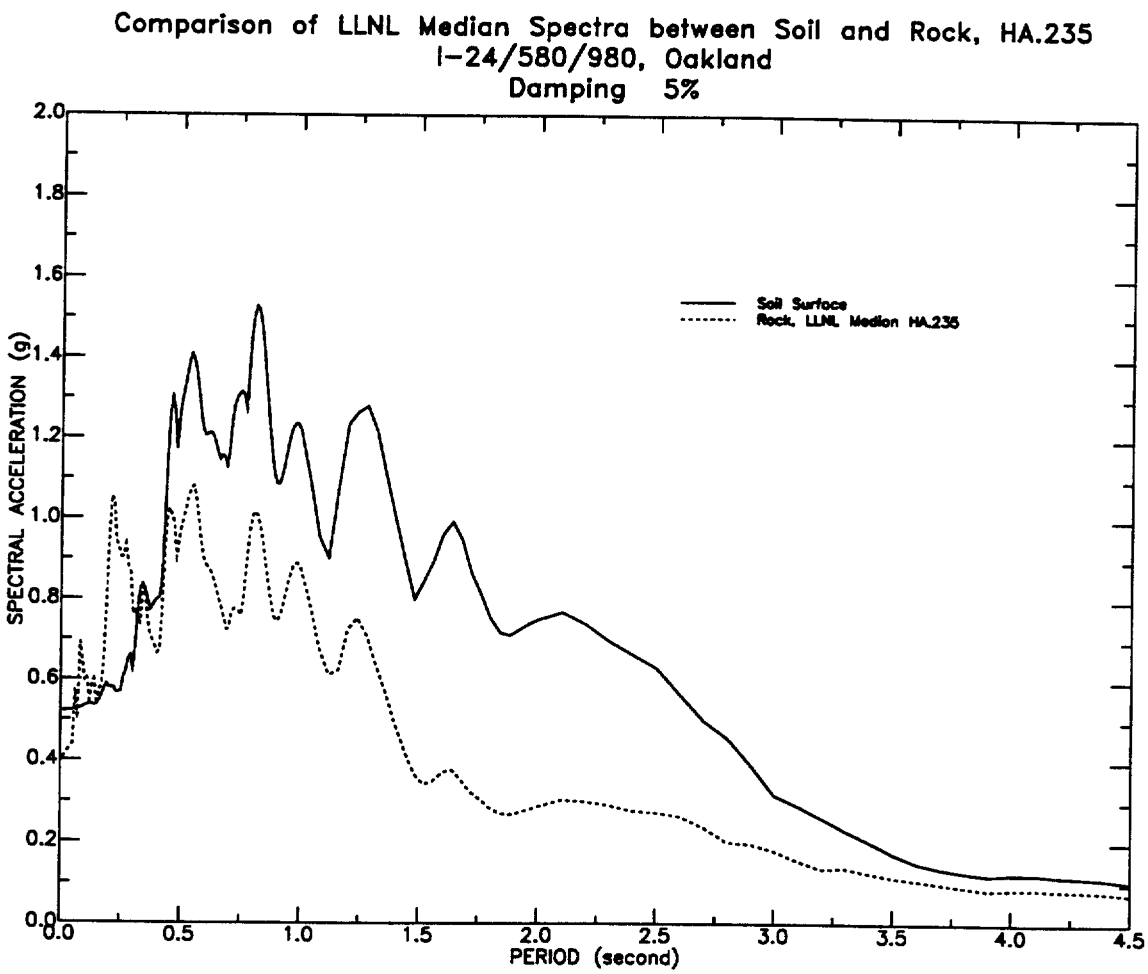


Fig. 4-10 Comparison of input and calculated response spectra in horizontal direction of HA.235 (Median)

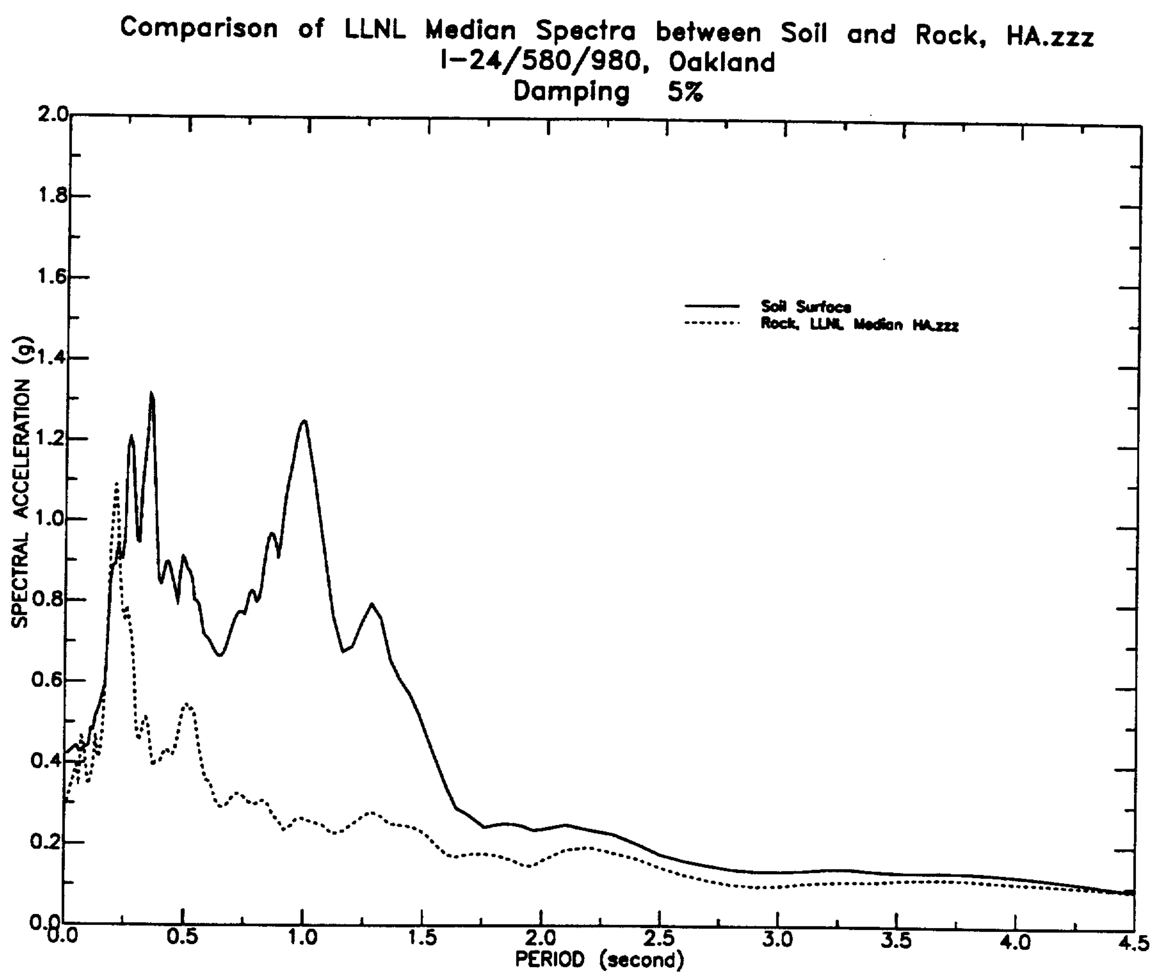


Fig. 4-11 Comparison of input and calculated response spectra in vertical direction (median)

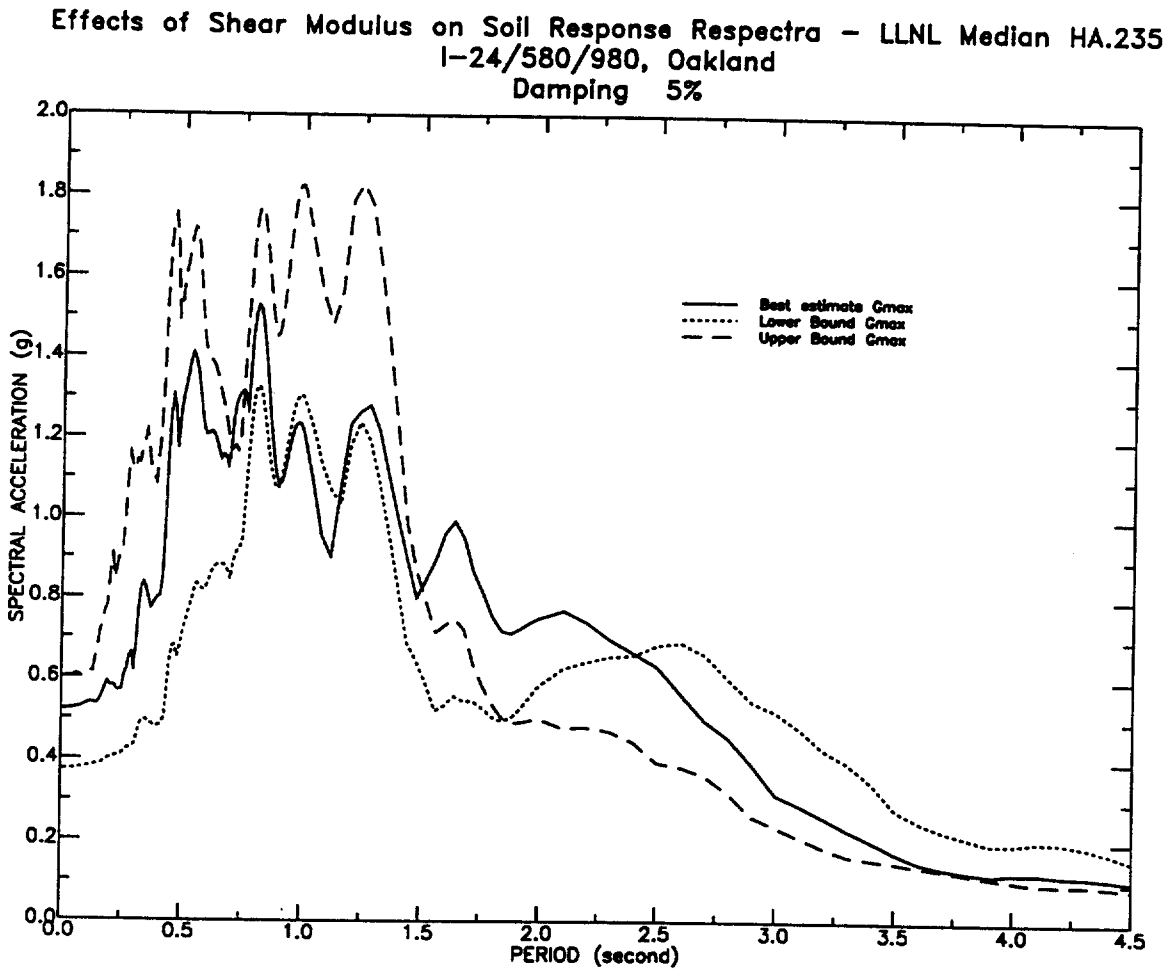


Fig. 4-12 **Computed response spectra using low bound, best estimate and upper bound low strain shear moduli of each layer**

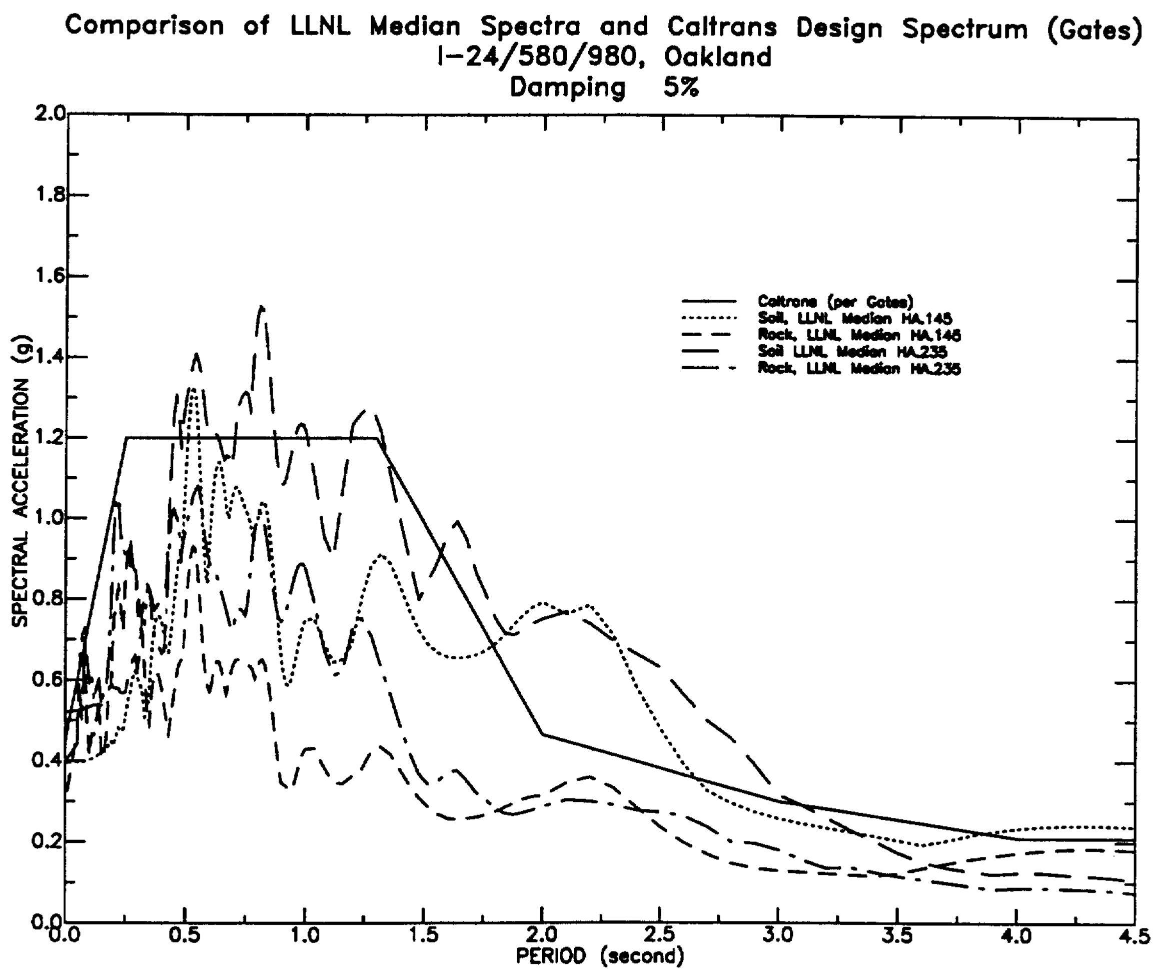
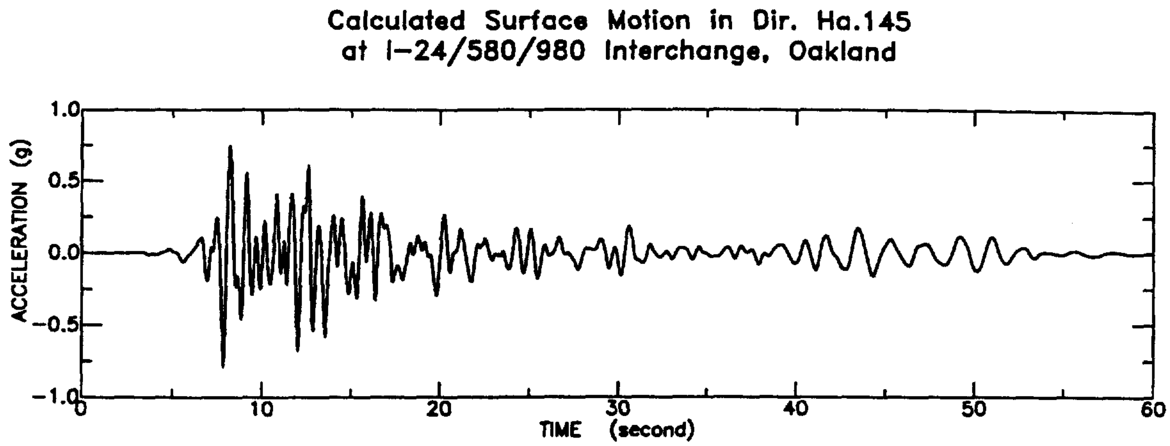
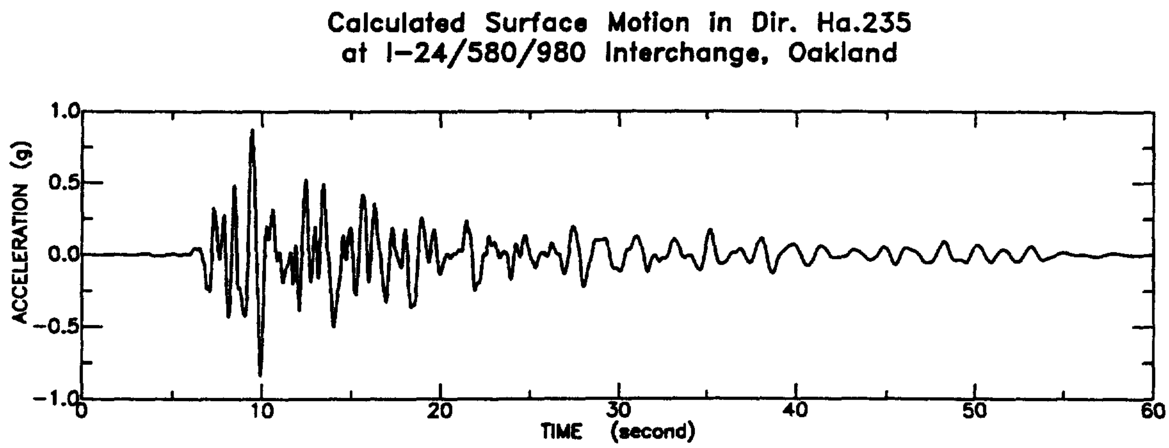


Fig. 4-13 Comparison of LLNL median rock and soil spectra as well as Caltrans design/evaluation spectrum





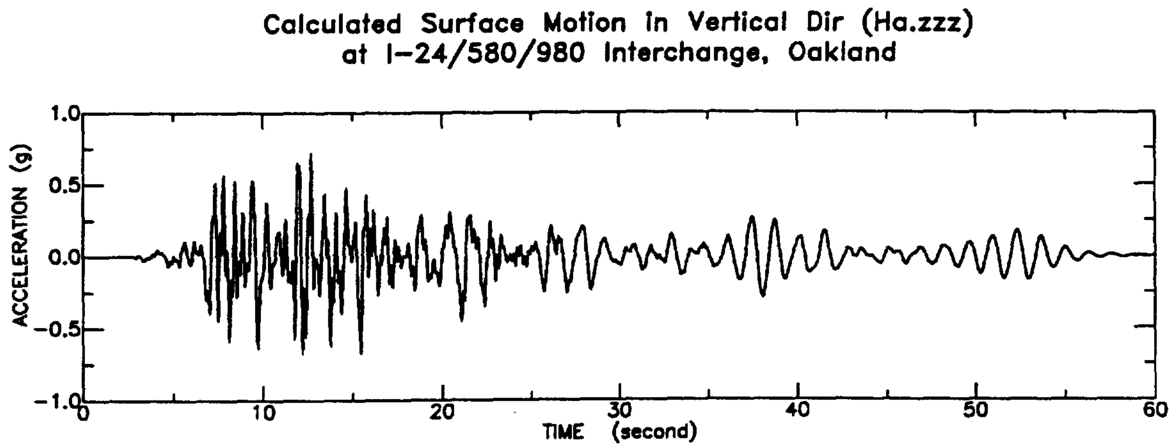


Fig. 4-14 **Computed ground acceleration time histories on the interchange site using LLNL 84th percentile synthetic motions**

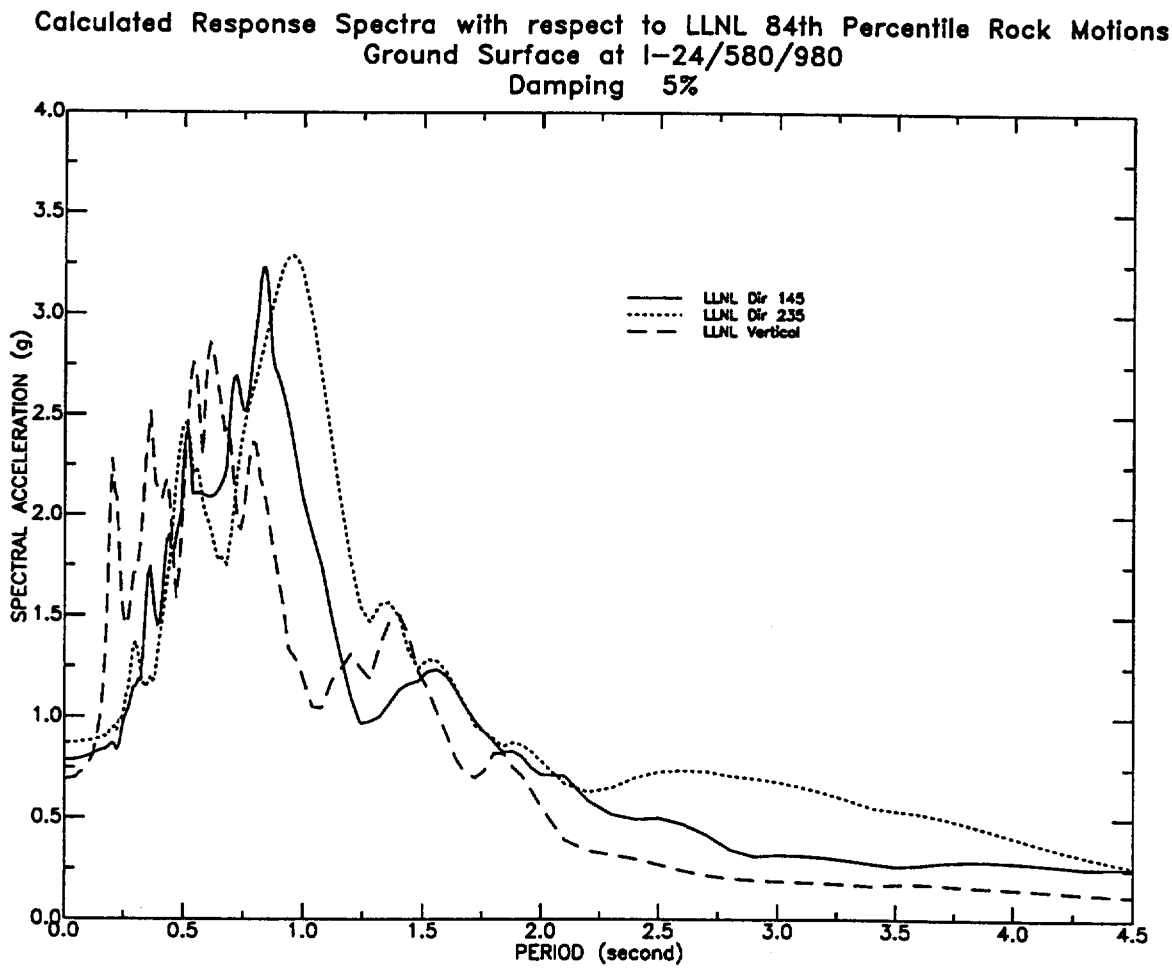


Fig. 4-15 **Computed response spectra on the interchange site using LLNL 84th percentile motions**

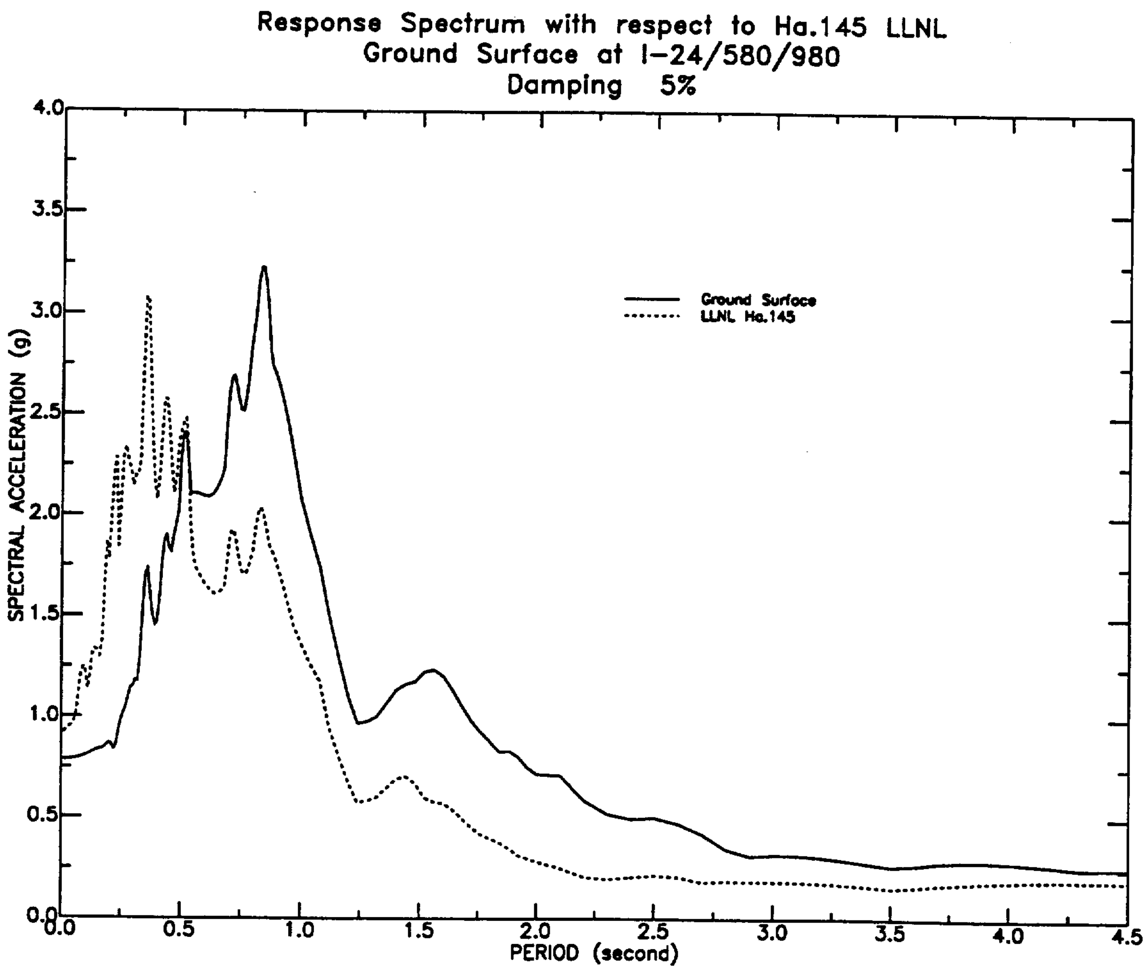


Fig. 4-16 Comparison of input and calculated response spectra in horizontal direction of HA.145 (84^{th} percentile)

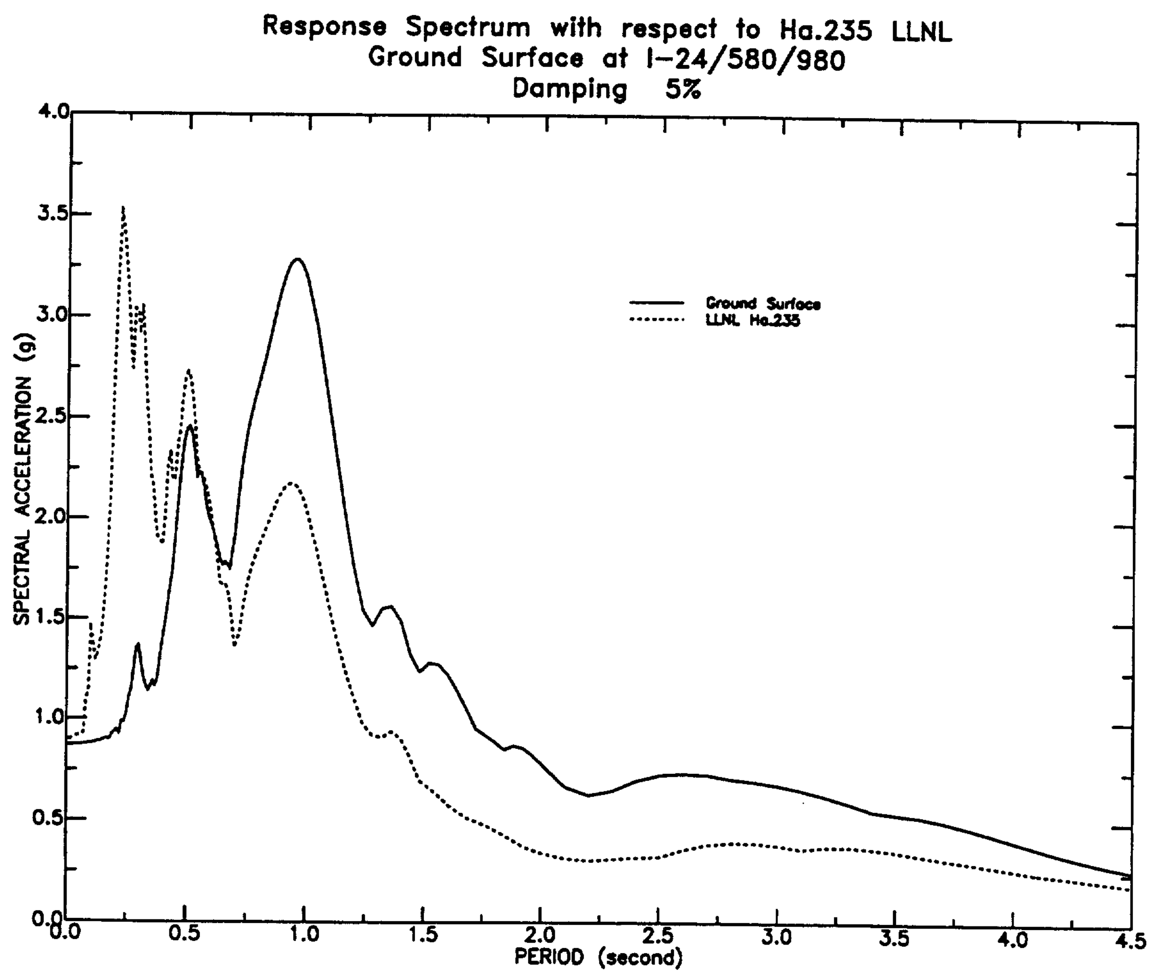


Fig. 4-17 Comparison of input and calculated response spectra in horizontal directions of HA.235 (84th percentile)

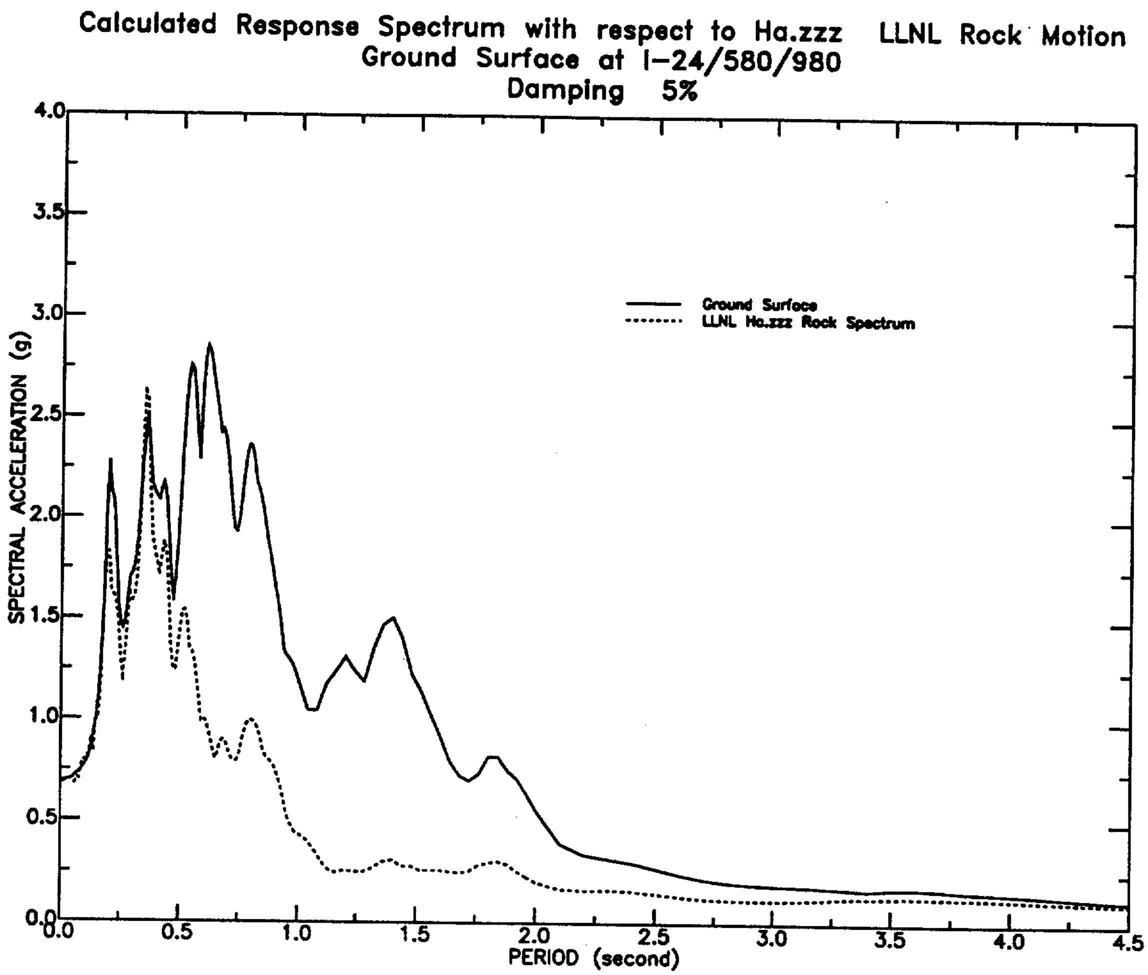


Fig. 4-18 Comparison of input and calculated response spectra in vertical direction (84th percentile)

Appendix A

SHAKE input files for Site Response Analysis using LLNL median synthetic motions

1. Shkin-t1: for shear wave analysis

2. Shkin-zzz: for final p-wave analysis

```
8192  0.5
  8     read strain dependent soil properties
  7    1   10  100.     data obtained from similar materials
 11 100. Modulus reduction curve #1: PI=0 (by Dobry)
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     1.000     .9700     .8800     .7100     .5000      .260     .1000
   .0500      .020     .0200
 11  1.0 damping :  PI=0
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.30      1.90      3.10      5.50      9.80     15.20     20.00
   23.70     26.00     26.00
 11 100. Modulus reduction curve #2: PI=15
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     1.000     .9900     .9500     .8300     .6400      .410     .2100
   .0900      .050      .050
 11  1.0 damping :  PI=15
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.20      1.40      2.60      4.60      7.60     11.60     16.00
   20.00     22.66     22.66
 11 100. Modulus reduction curve #3: PI=30
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     1.000     1.000     .9800     .9100     .7400      .550     .3500
   .1700      .080      .080
 11  1.0 damping :  PI=30
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.00      1.30      2.10      3.90      6.00      8.70     12.30
   16.90     20.30     20.30
 11 100. Modulus reduction curve #4: PI=50
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.0000    3.1600    10.00
   1.000     1.000     1.000     .9900     .9500     .8400      .680     .4700
   .2700      .120      .120
 11  1.0 damping :  PI=50
 .000100   .000316   .001000   .003160   .010000   .031600    100000   .316000
1.000000  3.160000 10.00
    1.00      1.00      1.30      1.90      3.00      4.30      6.20      9.30
   13.40     17.10     17.10
 11 100. Modulus reduction curve for sand with CP<1.0 KSC,(30 ft), S1, Curve #5
 .000100   .000316   .001000   .003160   .010000   .031600    100000   .316000
1.000000  3.160000 10.00
   1.000     0.978      .934      .838      .672      .463      .253      .140
    .090      .070      .070
 11  1.0 Damping for gravelly soils & sand, Depth 0 to 30 ft, Seed 1984,mean curve
 .000100   .000316   .001000   .003160   .010000   .031600    100000   .316000
1.000000  3.160000 10.00
    0.70      1.00      1.70      3.12      5.60      9.80     15.50     21.00
   25.00     25.00     25.00
 11 100. Modulus reduction curve for sand with CP=1 to 3 KSC, (30 to 120 ft)S2, Curve #6
 .000100   .000316   .001000   .003160   .010000   .031600    100000   .316000
1.000000  3.160000 10.00
   1.000     0.985      .952      .873      .724      .532      .332      .200
    .114      .100      .100
 11  1.0 Damping for gravelly soils, & sands depth 30 - 120 ft, Seed 1984,ave of m&lb
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    0.60      0.80      1.25      2.25      4.00      7.50     12.80     18.40
   23.10     23.10     23.10
```

```
 11 100. Modulus reduction curve for sand with CP>3.0 KSC, (>120 ft),S3, Curve #7
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     0.991      .969      .908      .782      .602      .393      .266
    .183      .163      .163
 11  1.0 Damping for gravelly soils, & sands depth > 120 ft,Seed 1984, lower bound
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
 1.00000  3.16000  10.00
    0.50      0.60      0.80      1.40      2.40      5.20     10.10     15.80
   21.20     21.20     21.20
  2     read soil profile
  1   41    5   run for making data file for P-wave analysis (LLNL Median Ham.145)
  1    1    1       5.0    -0.333      0.0480      0.128      673.0     1800.0 1.00
  2    3    1       5.0    -0.38       0.0480      0.130      673.0     1829.0 1.00
  3    3    1       5.0    -0.38       0.0480      0.126      673.0     1772.0 1.00
  4    3    1       5.0    -0.38       0.0480      0.126      675.0     1783.0 1.00
  5    2    1       5.0    -0.42       0.0480      0.126      787.0     2424.0 1.00
  6    6    1       5.0    -0.42       0.0480      0.129      799.0     2558.0 1.00
  7    6    1       5.0    -0.42       0.0480      0.129      773.0     2394.0 1.00
  8    6    1       7.5    -0.44       0.0430      0.127      946.0     3530.0 1.00
  9    6    1       7.5    -0.44       0.0430      0.127     1245.0     6113.0 1.00
 10    2    1       7.5    -0.44       0.0430      0.125     1241.0     5979.0 1.00
 11    2    1       7.5    -0.44       0.0430      0.125     1107.0     4757.0 1.00
 12    2    1       5.0    -0.44       0.0380      0.125     1046.0     4247.0 1.00
 13    2    1       5.0    -0.44       0.0380      0.129      974.0     3801.0 1.00
 14    6    1      10.0    -0.38       0.0380      0.127      907.0     3245.0 1.00
 15    6    1       5.0    -0.38       0.0380      0.132     1187.0     5776.0 1.00
 16    2    1      10.0    -0.44       0.0350      0.127     1460.0     8407.0 1.00
 17    2    1      10.0    -0.44       0.0350      0.127     1414.0     7886.0 1.00
 18    2    1      10.0    -0.44       0.0330      0.127     1279.0     6452.0 1.00
 19    2    1      10.0    -0.44       0.0330      0.127     1329.0     6966.0 1.00
 20    2    1      10.0    -0.44       0.0330      0.127     1535.0     9293.0 1.00
 21    2    1      20.0    -0.44       0.0330      0.127     1378.0     7489.0 1.00
 22    2    1      20.0    -0.44       0.0330      0.127     1211.0     5784.0 1.00
 23    2    1      20.0    -0.44       0.0330      0.128     1690.0    11353.0 1.00
 24    7    1      20.0    -0.38       0.0300      0.128     1328.0     7011.0 1.00
 25    7    1      10.0    -0.38       0.0300      0.128     1288.0     6595.0 1.00
 26    3    1      10.0    -0.42       0.0300      0.128     1550.0     9550.0 1.00
 27    3    1      10.0    -0.42       0.0300      0.128     1900.0    14350.0 1.00
 28    3    1      10.0    -0.42       0.0300      0.128     1250.0     6221.0 1.00
 29    7    1      10.0    -0.38       0.0300      0.128     1060.0     4466.0 1.00
 30    7    1      10.0    -0.38       0.0300      0.128     1700.0    11488.0 1.00
 31    7    1      10.0    -0.38       0.0280      0.128     1050.0     4383.0 1.00
 32    7    1      20.0    -0.38       0.0280      0.130     2900.0    33953.0 1.00
 33    7    1      20.0    -0.38       0.0240      0.130     1800.0    13081.0 1.00
 34    7    1      20.0    -0.38       0.0240      0.130     1800.0    13081.0 1.00
 35    7    1      20.0    -0.38       0.0240      0.130     2240.0    20257.0 1.00
 36    3    1      20.0    -0.42       0.0200      0.130     1700.0    11668.0 1.00
 37    3    1      20.0    -0.42       0.0200      0.130     2000.0    16149.0 1.00
 38    3    1      20.0    -0.42       0.0200      0.130     2000.0    16149.0 1.00
 39    3    1      20.0    -0.42       0.0200      0.130     1900.0    14575.0 1.00
 40    7    1      15.0    -0.38       0.0200      0.130     1600.0    10335.0 1.00
 41                                               0.150     5000.0
  1     read new input motion >>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>
  3     assign object  motion to a specefied layer
 41    0
  4     obtain strain compatible soil ppties
  3    7  10.0       0.65
  5     compute new motions
  1    2    3    4    5    6    7    8    9   10   11   12   13   14   15
  0    1    1    1    1    1    1    1    1    1    1    1    1    1    1
  2    0    0    0    0    0    0    0    0    0    0    0    0    0    0
  5     compute new motions
```

```
      16    17    18    19    20    21    22    23    24    25    26    27    28    29    30
       1     1     1     1     1     1     1     1     1     1     1     1     1     1     1
       0     0     0     0     0     0     0     0     0     0     0     0     0     0     0
       5      compute new motions
      31    32    33    34    35    36    37    38    39    40    41    41
       1     1     1     1     1     1     1     1     1     1     1     0
       0     0     0     0     0     0     0     0     0     0     0     0
       9
       1     0
       1     1     1     1     2
   0.05
   0
```

```
8192  0.5
  8     read strain dependent soil properties
  7    1   10  100.     data obtained from similar materials
  11 100. Modulus reduction curve #1: PI=0 (by Dobry)
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     1.000     .9700     .8800     .7100     .5000      .260     .1000
   .0500      .020     .0200
  11  1.0 damping :  PI=0
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.30      1.90      3.10      5.50      9.80     15.20     20.00
   23.70     26.00     26.00
  11 100. Modulus reduction curve #2: PI=15
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     1.000     .9900     .9500     .8300     .6400      .410     .2100
   .0900      .050      .050
  11  1.0 damping :  PI=15
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.20      1.40      2.60      4.60      7.60     11.60     16.00
   20.00     22.66     22.66
  11 100. Modulus reduction curve #3: PI=30
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     1.000     1.000     .9800     .9100     .7400      .550     .3500
   .1700      .080      .080
  11  1.0 damping :  PI=30
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.00      1.30      2.10      3.90      6.00      8.70     12.30
   16.90     20.30     20.30
  11 100. Modulus reduction curve #4: PI=50
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.0000    3.1600    10.00
   1.000     1.000     1.000     .9900     .9500     .8400      .680     .4700
   .2700      .120      .120
  11  1.0 damping :  PI=50
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    1.00      1.00      1.30      1.90      3.00      4.30      6.20      9.30
   13.40     17.10     17.10
  11 100. Modulus reduction curve for sand with CP<1.0 KSC,(30 ft), S1, Curve #5
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     0.978      .934      .838      .672      .463      .253      .140
    .090      .070      .070
  11  1.0 Damping for gravelly soils & sand, Depth 0 to 30 ft, Seed 1984,mean curve
 .000100   .000316   .001000   .003160   .010000   .031600    100000   .316000
1.000000  3.160000 10.00
    0.70      1.00      1.70      3.12      5.60      9.80     15.50     21.00
   25.00     25.00     25.00
  11 100. Modulus reduction curve for sand with CP=1 to 3 KSC, (30 to 120 ft)S2, Curve #6
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     0.985      .952      .873      .724      .532      .332      .200
    .114      .100      .100
  11  1.0 Damping for gravelly soils, & sands depth 30 - 120 ft, Seed 1984,ave of m&lb
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
    0.60      0.80      1.25      2.25      4.00      7.50     12.80     18.40
   23.10     23.10     23.10
```

```
 11 100. Modulus reduction curve for sand with CP>3.0 KSC, (>120 ft),S3, Curve #7
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
1.000000  3.160000 10.00
   1.000     0.991      .969      .908      .782      .602      .393      .266
    .183      .163      .163
 11  1.0 Damping for gravelly soils, & sands depth > 120 ft,Seed 1984, lower bound
 .000100   .000316   .001000   .003160   .010000   .031600   .100000   .316000
 1.00000  3.16000  10.00
    0.50      0.60      0.80      1.40      2.40      5.20     10.10     15.80
   21.20     21.20     21.20
  2     read soil profile
  1   41    5    for LLNL  Median  Hm.zzz, P-wave analysis 3/2/96
  1    0    1       5.00    5453.9    0.0072     0.128        0.50    5714.8 1.00
  2    0    1       5.00    7840.9    0.0050     0.130        0.50    7970.5 1.00
  3    0    1       5.00    6499.7    0.0074     0.126        0.50    6835.2 1.00
  4    0    1       5.00    5782.3    0.0091     0.126        0.50    6257.9 1.00
  5    0    1       5.00    8120.7    0.0074     0.126        0.50    9401.2 1.00
  6    0    1       5.00    4067.1    0.0105     0.129        0.50    6463.4 1.00
  7    0    1       5.00    2817.2    0.0135     0.129        0.50    5196.5 1.00
  8    0    1       7.50    7836.0    0.0067     0.127        0.50   11419.2 1.00
  9    0    1       7.50   20745.8    0.0044     0.127        0.50   25900.3 1.00
 10    0    1       7.50   28387.4    0.0046     0.125        0.50   31142.7 1.00
 11    0    1       7.50   16930.9    0.0062     0.125        0.50   20005.3 1.00
 12    0    1       5.00   12065.2    0.0076     0.125        0.50   15351.1 1.00
 13    0    1       5.00    8593.7    0.0090     0.129        0.50   11927.9 1.00
 14    0    1      10.00     899.3    0.0409     0.127        0.50    3457.4 1.00
 15    0    1       5.00    4410.8    0.0231     0.132        0.50    9053.3 1.00
 16    0    1      10.00   37413.0    0.0048     0.127        0.50   41372.4 1.00
 17    0    1      10.00   31946.8    0.0052     0.127        0.50   36038.6 1.00
 18    0    1      10.00   20525.3    0.0064     0.127        0.50   24514.6 1.00
 19    0    1      10.00   22957.0    0.0062     0.127        0.50   27211.2 1.00
 20    0    1      10.00   38748.4    0.0051     0.127        0.50   43481.0 1.00
 21    0    1      20.00   24345.8    0.0065     0.127        0.50   29225.4 1.00
 22    0    1      20.00   12801.2    0.0092     0.127        0.50   17989.8 1.00
 23    0    1      20.00   51161.9    0.0052     0.128        0.50   57612.0 1.00
 24    0    1      20.00    7477.2    0.0164     0.128        0.50   13518.2 1.00
 25    0    1      10.00    6291.6    0.0184     0.128        0.50   12104.4 1.00
 26    0    1      10.00   41256.7    0.0064     0.128        0.50   44584.2 1.00
 27    0    1      10.00   72306.9    0.0048     0.128        0.50   75364.4 1.00
 28    0    1      10.00   19806.1    0.0089     0.128        0.50   23732.2 1.00
 29    0    1      10.00    2084.5    0.0321     0.128        0.50    6107.6 1.00
 30    0    1      10.00   17413.4    0.0105     0.128        0.50   25771.7 1.00
 31    0    1      10.00    1914.9    0.0336     0.128        0.50    5831.1 1.00
 32    0    1      20.00  109423.1    0.0037     0.130        0.50  120898.2 1.00
 33    0    1      20.00   20879.1    0.0098     0.130        0.50   30203.0 1.00
 34    0    1      20.00   20108.8    0.0105     0.130        0.50   29761.2 1.00
 35    0    1      20.00   45996.2    0.0061     0.130        0.50   58566.0 1.00
 36    0    1      20.00   48005.8    0.0072     0.130        0.50   52847.0 1.00
 37    0    1      20.00   74880.9    0.0062     0.130        0.50   80393.6 1.00
 38    0    1      20.00   73447.2    0.0064     0.130        0.50   79306.3 1.00
 39    0    1      20.00   62249.1    0.0070     0.130        0.50   68214.0 1.00
 40    0    1      15.00    8537.4    0.0221     0.130        0.50   18608.6 1.00
 41         1             349378.9    0.0044    0.1500
  1     read new input motion >>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>>
  3     assign object  motion to a specefied layer
 41    0
  4     obtain strain compatible soil ppties
  1    1  10.0      0.65
  5     compute new motions
  1    2    3    4    5    6    7    8    9   10   11   12   13   14   15
  0    1    1    1    1    1    1    1    1    1    1    1    1    1    1
  2    0    0    0    0    0    0    0    0    0    0    0    0    0    0
  5     compute new motions
```

```
   16   17   18   19   20   21   22   23   24   25   26   27   28   29   30
    1    1    1    1    1    1    1    1    1    1    1    1    1    1    1
    0    0    0    0    0    0    0    0    0    0    0    0    0    0    0
    5      compute new motions
   31   32   33   34   35   36   37   38   39   40   41   41
    1    1    1    1    1    1    1    1    1    1    1    0
    0    0    0    0    0    0    0    0    0    0    0    0
    9
    1    0
    1    1    1    1    2
0.05
0
```

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