DAMAGE POTENTIAL INDEX MAPPING FOR SALT LAKE VALLEY, UTAH

by
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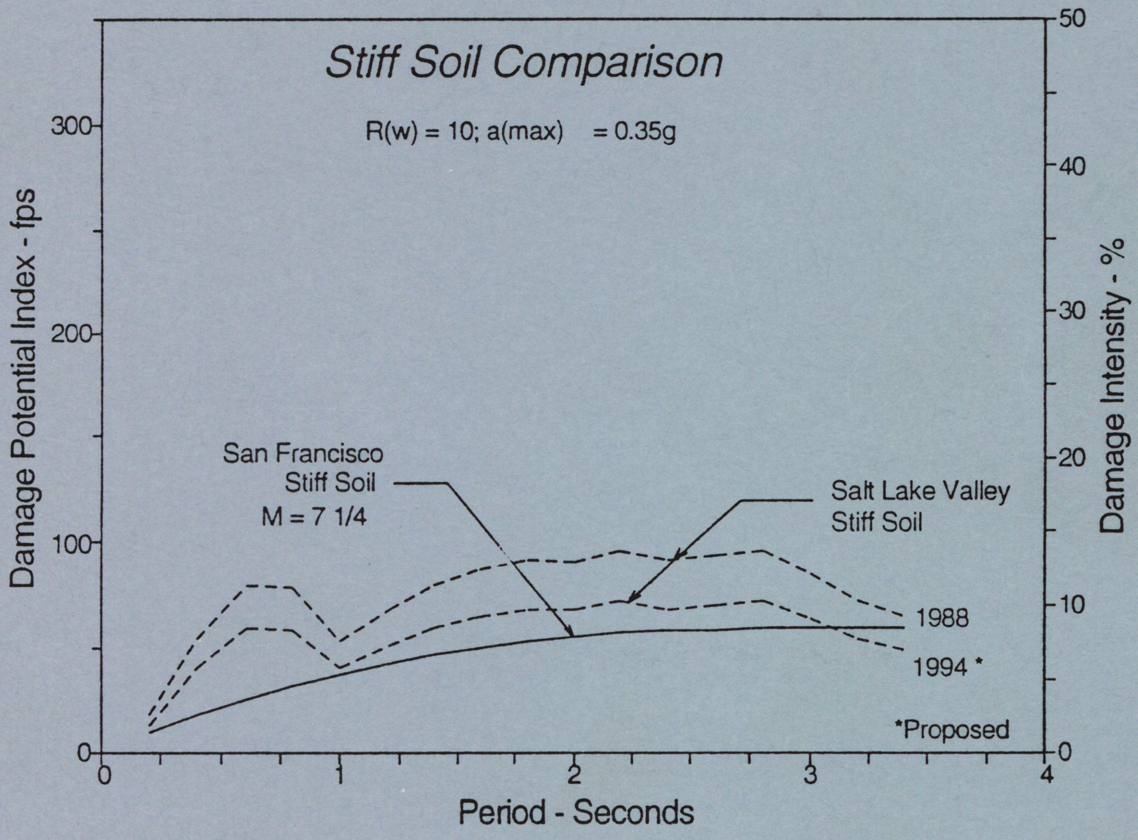

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Adan and Rollins DAMAGE POTENTIAL INDEX MAPPING FOR SALT LAKE VALLEY, UTAH UGS Miscellaneous Publication 93-4

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ABSTRACT

Damage potential index (DPI) calculations have been made for the Salt Lake Valley based on ground response analyses at 13 sites where soil and shear wave velocity profiles had been previously defined by the U.S. Geological Survey. The damage potential index is proportional to the ratio of the earthquake induced loads (demand) to the lateral resisting force required by the seismic code. Based on soil conditions, geology and low-strain amplification measurements, the valley was divided into four zones ranging from stiff shallow sites to deep soft sites. Ground response analyses were performed for three to four sites within each zone using the computer program SHAKE. Each site was subjected to eight M 7.0-7.25 rock input motions with peak accelerations of 0.35 g and 0.70 g. These accelerations levels have a 10% chance of being exceeded in 50 years and 250 years respectively. Based on the ground response studies, representative acceleration response spectra were developed for each of the four zones for the two acceleration levels. For building periods less than about 1.0 second, spectral accelerations were highest for stiff shallow soil profiles, however, for building periods greater than about 1.0 second spectral accelerations were highest for soft deep soil profiles.

The earthquake demand determined from the response analyses was divided by the equivalent lateral force coefficient specified by past, present and proposed Uniform Building Code seismic requirements. The damage potential index has been correlated to structural damage based on data obtained in Mexico City. This allowed an estimation of the damage percentage for buildings with various periods constructed according to each building code. For the earlier codes, the damage potential was much higher for soft soils than for stiff soils but for more recent codes, which include a soil coefficient, the damage potential values are less dependent on soil type. Damage percentages for buildings constructed under the earlier codes approach that observed in Mexico City, however, the current code appears to provide reasonable protection for the entire valley for the 50 year acceleration level. At the 250 year acceleration level, the potential for damage is significantly greater with damage intensities as high as 20% over large period ranges. Amplification in Salt Lake Valley soft soil appears to be less than for San Francisco soft soil, however, because Salt Lake City is in seismic zone 3 rather than zone 4, the damage intensity would be about the same for a M 7.0 earthquake. High damage potentials were computed for stiff shallow soils overlying bedrock in comparison with similar soil sites in San Francisco. This poses a significant hazard to low-rise structures located on stiff shallow soils around the edge of the valley. A change from zone 3 to 4 would reduce the damage intensity in the event of an earthquake by 25%.

INTRODUCTION

The fact that local soil conditions play a major role in determining the intensity of earthquake shaking has been recognized by seismologists and engineers for many years. Recently, a number of major earthquakes have provided excellent opportunities to study the relationship between local soil conditions, earthquake shaking intensities, and building damage. For example, the 1985 Mexico City and the 1989 Loma Prieta earthquakes provided unmistakable evidence that local soil conditions can significantly alter ground motions in comparison with rock motions.

During these earthquakes, substantial differences in recorded maximum accelerations were observed at soil sites in relation to rock sites. Soil amplification in Mexico City increased peak acceleration levels from 0.04 g on rock to 0.2 g on soft soil deposits. In San Francisco, ground motions increased from 0.09 g on rock to 0.25 g on soft soil deposits at comparable distances (EERI, 1989). During both earthquakes, the amplification in deep soft soils accounted for much of the damage at distant sites.

Conditions in some areas of the Salt Lake Valley appear to have similarities with locations near San Francisco where soil amplification was observed during the 1989 Loma Prieta earthquake. The potential for soil amplification of lacustrine deposits in the Salt Lake Valley has been recognized for some time based on low-amplitude ground motions measured after nuclear detonations at the Nevada test site. Spectral amplification ratios (spectral acceleration on soil divided by spectral acceleration on rock) recorded by U.S. Geological Survey researchers following these detonations are between 5 to 10 in large areas of the valley (Tinsley et al, 1991).

Differences in sediment properties and thicknesses in the valley may produce substantial geographical variation in the intensity of ground motions. The potentially damaging levels of soil amplification can be mitigated by the development of special building code provisions. One method for assessing the degree of protection provided by building code provisions is the damage potential index (DPI) developed by Seed and Sun (1989). The DPI is the ratio of the forces exerted by an earthquake to the lateral resisting force prescribed by the building code and is a function of building period. Because this index includes both building resistance and earthquake forces, the DPI can be important for evaluating the effects of soil amplification on structural damage.

The purpose of this study is to examine the possibility of soil amplification in selected areas of the Salt Lake Valley, estimate expected ground motion characteristics, and provide an evaluation of the damage potential index for these areas. Based on research performed by Seed and Sun (1989) following the 1985 Mexico City earthquake, we will then relate these index values to expected damage intensities. Finally, we will comment on recommendations regarding possible code revisions to reduce potential earthquake damage.

SCOPE AND METHODOLOGY

Over the past ten years, personnel from the U.S. Geological Survey (USGS) drilled approximately 40 bore holes throughout the Salt Lake Valley to depths of 150 to 300 feet. At each of these test sites, measurements of low-amplitude ground motions were recorded, shear wave velocities were measured, and a limited amount of soil testing was performed. Based on the USGS measurements of low-amplitude ground motions at each site, contours of average spectral ratios in the period range from 0.2 to 0.7 seconds were drawn for areas around the valley (King et al., 1983; Tinsley et al, 1991). These contours are shown on Figure 1 with respect to urban and topographical features, and significant faults.

While it is impossible to argue with the measured spectral ratio data provided by USGS studies for small strain events, it is unclear whether these spectral ratios are directly applicable for strong ground shaking. This uncertainty results from consideration of at least three factors.

First, the frequency content of the incoming rock motions is significantly different for distant blasts compared to strong ground shaking. Response spectra recorded on rock in Salt Lake Valley due to nuclear detonations in Nevada tend to have fundamental periods between 1.5 and 2.5 seconds (King et al, 1983). However, earthquake motions produced by the Wasatch fault would likely have fundamental periods between 0.25 and 0.4 seconds (Seed, Ugas, and Lysmer, 1974; Joyner and Boore, 1991; Sadigh, 1990). Longer period motions would be more likely to resonate with the deep alluvial basins. As a result, data from distant nuclear blasts tend to overestimate amplification in the deeper basins and underestimate amplification on shallow deposits.

Secondly, seismic waves from nuclear detonations travel from the west to the east, while waves from an earthquake on the Wasatch fault would likley generated nearly vertically propagating waves or wave moving from east to west. Studies performed by Murphy (1989) and Hill et al (1990) indicate that amplification due to the basin shape is several times greater for waves moving into the basin from the west than from the vertically propagating waves because of reflections off the steeply dipping fault surface.

Finally, the amplitude of waves from nuclear blasts are several orders of magnitude smaller than waves expected from earthquakes on the Wasatch fault. As the level of ground shaking increases, soils undergo greater cyclic shear strain which leads to greater soil damping and a reduction in shear modulus. As a result of the higher damping and lower shear modulus, soils may be unable to produce the same amplification of ground motions at high acceleration levels that they do at low acceleration levels.

Because of these three factors: (1) differences in frequency content, (2) differences in travel path and (3) soil non-linearity, amplification ratios for earthquake motions are expected to be different from those measured for small strain events. Thus, while the spectral ratios recorde by the USGS separate the valley into zones which should behave in a similar manner, we believe additional analyses are required to estimate likely response to earthquake motions. Comparisons between computed and measured ground motions in Mexico City and San Francisco (Seed et al, 1987; Idriss, 1990) suggest that reasonable estimations of ground response can be made using the equivalent linear method employed in the computer program SHAKE. SHAKE was developed to compute the ground response of a system of homogeneous, viscoelastic soil layers of infinite horizontal extent subjected to vertically traveling shear waves generated by a given earthquake (Schnabel, 1972).

In order to perform these SHAKE analyses, 13 representative USGS test sites were chosen for study and the locations of these sites are also shown on Figure 1. For the purposes of this study, the contour levels were divided into four zones having spectral ratios of 1 to 3, 3 to 5, 5 to 8, and 8 to 11. Three to four USGS test sites are located within each amplification zone. Utilizing the data collected by the USGS personnel, ground response analyses were peformed at each test site using the computer program SHAKE.

As input motions for the ground response analyses, we selected nine rock outcrop records, including many near field records from the 1989 Loma Prieta earthquake. While earthquakes on the Wasatch fault may produce a wide variety of ground accelerations, our study concentrated on rock outcrop accelerations having a 90% probability of not being exceeded in 50 and 250 years.

Based on the ground response analyses, representative spectral acceleration shapes were determined for each amplification zone identified by Tinsley et al (1991). Using the representative acceleration values, and the historical building code provisions for Salt Lake City, we calculated the DPI values for structures in each amplification zone. As previously indicated, the DPI can be a measure of the earthquake forces exerted on a structure divided by the resisting capacity of that structure as prescribed by building code requirements. Finally, we compared the DPI with damage intensities expected for structures in the period range of 0.3 to 4 seconds.

The results of this report allow a rational evaluation of expected earthquake damage within the Salt Lake Valley under various building code provisions and provide important information regarding the behavior of different Salt Lake Valley soils when subjected to earthquake shaking. In addition, the calculated DPI values may be valuable for assessing the desirability of changing building code provisions.

DAMAGE POTENTIAL MAPPING

Building damage increases when the soil column amplifies the earthquake motions at the period of the building. Because of variations in the depth and shear wave velocity of soil deposits, soil response can cause significant variations in ground motions across a valley and lead to major variations in damage patterns in the same city during the same earthquake. The influence of soil conditions on induced base shear force may reach 500% in many cases in comparison with forces on rock (Seed, 1987).

In a detailed study of damage in the 1967 Caracas, Venezuela earthquake (M=6.4?) researchers discovered that building damage was associated with the depth of the soil deposit below the building. The damage intensity for tall buildings was greatest when they were located on deep soil deposits while the damage intensity for short buildings was greatest when they were located on shallow soil deposits (Seed et al, 1972). A comprehensive study of the reasons for these observations led to the conclusion that the effects were related to resonance between the soil deposits and the buildings. In other words, short buildings with a natural period of 0.4 to 0.5 seconds suffered the greatest damage when they were located on soil deposits having a period of about 0.5 seconds, while tall buildings having a period of vibration of between 1.2 and 1.4 seconds received the most damage on soil deposits having a period of about 1.3 seconds. In addition, Seed et al (1972), indicated that the building damage intensity could be related to the ratio of the spectral acceleration, S_a, divided by the lateral force coefficient, k, for which the building was designed.

Subsequent to the 1985 Mexico City earthquake, similar studies were undertaken in an effort to relate damage to soil conditions. Correlations between the damage potential index (DPI) and actual damage intensities were established for the heavy damage zone of Mexico City. Then techniques for applying these correlations to other areas were developed. Since the DPI relates induced forces to resisting forces and includes the effects of soil response, it appears to provide a reasonable indication of the damage which might be produced by earthquake shaking. The following section will briefly review the procedure developed by Seed and Sun (1989) in the aftermath of the Mexico City earthquake and provide background for comparisons with San Francisco Bay and Salt Lake Valley areas.

Development of Damage Potential Index

In the approach proposed by Seed and Sun (1989) for ductile buildings, of the type which suffered major damage in Mexico City, the damage potential index is evaluated as follows:

$$DPI = \frac{Induced\ Force \times Duration\ of\ Force}{Design\ Resistance} \qquad \text{(Eq. 1)}$$

The force induced on a building by an earthquake can be evaluated from the expression:

$$\text{Induced Force} = W \times S_a/g \qquad \text{(Eq. 2)}$$

where W is the building weight, S_a is the spectral acceleration, and g is the acceleration of gravity. The spectral acceleration is determined from the ground surface response spectrum. The duration of force is considered to be proportional to the period of the building, T, and the number of load cycles induced by the earthquake:

$$\text{Force Duration} \propto \text{Period} \times \text{No. of Cycles} \qquad \text{(Eq. 3)}$$

Seed and Sun (1989) replaced the no. of cycles with a term called the duration weighting factor (DWF), which reflects the number of load cycles induced by the earthquake, and is a function of the earthquake magnitude.

The design resistance (capacity) is determined by the load combinations and the allowable stresses prescribed by the building code, and is expressed as follows:

$$\text{Design Resistance} = W \times k \times R_f \qquad \text{(Eq. 4)}$$

AS indicated previously, the k factor is the design lateral force coefficient used to design the structure for the effects of earthquake shaking. The k value is generally determined from building code seismic provisions. The structural resistance factor, R_f, expresses the relative design resistances as they are affected by allowable stresses, load combinations, and construction quality. In any one city this factor may be the same for all structures, but in comparing structures in different cities, this factor changes because of varying code requirements and construction standards.

The damage potential index may now be expressed in the following manner:

$$DPI \ \alpha \ \left(\frac{W\times\frac{S_a}{g}\times T}{W\times k\times R_f}\right)\times DWF \qquad \text{(Eq. 5)}$$

Because spectral velocity, S_v, is related to spectral acceleration as,

$$S_v = \frac{S_a}{\omega} = \frac{T \times S_a}{2\pi} \qquad \text{(Eq. 6)}$$

the proportionality in equation 5 can also be written as,

$$DPI = \left(\frac{S_v}{k\times R_f}\right)\times DWF \qquad \text{(Eq. 7)}$$

if the constants (g and 2π) are neglected. The units for DPI then become ft/second or velocity.

In Mexico City, R_f was assigned a value of one by Seed and Sun (1989) for the sake of convenience. For other areas appropriate values can be assessed by knowledgeable structural engineers familiar with code requirements in Mexico City and local design codes. In San Francisco, for example, the structural resistance factor was assigned a value of 1.3 based on recommendations from Dr. V. V. Bertero (Univ. of Calif., Berkeley). In comparison, the Salt Lake Valley was assigned a value of 1.2 based on recommendations from Drs. L. Reaveley (Reaveley Engineers) and R. Goodwin (Brigham Young Univ.).

Damage Intensity in Heavy Damage Area of Mexico City

Following the Mexico City earthquake, a detailed survey was made of the damage intensity to different classes of structures in different parts of the city (Borja-Navarrete et al, 1986). The damage intensity was defined as the ratio of the number of structures in any given category which suffered major damage divided by the total number of structures in that category existing in the heavy damage zone. The statistics for the heavy damage zone in Mexico city are summarized in Table 1. Structures suffering the highest damage intensities were those with fundamental periods in the range of 1.5 to 2.5 seconds.

Number of Stories	Building Period (sec)	No. Bldgs. w/Serious Damage	Total Number of Buildings	Damage Intensity (%)
1-2	0.250	297	15,000	2
3-5	0.667	154	5400	3
6-8	1.167	117	650	18
9-12	1.750	62	215	29
> 12	2.670	21	92	23

Table 1. Statistics for heavy damage zone of Mexico City earthquake (After Borja-Navarrete et al, 1986).

Damage Potential Index for Heavy Damage Zone in Mexico City

In order to develop correlations with the damage intensity values for the heavy damage zone of Mexico City, damage potential index values were computed for this area. This effort required that ground response analyses be performed to study the ground motions developed in the lake bed areas of the heavy damage zone during the 1985 earthquake. In terms of acceleration response spectra, the recorded motions at the SCT recording station (near the heavy damage zone), together with the spectra for computed motions likely to have been developed for soil depths typical of the heavy damage zone (25 to 45 meters) are all shown in Figure 2 (Seed et al., 1987). Based on these results, a representative average response spectrum was determined to represent the general characteristics of the earthquake motions in the heavy damage zone. The average spectral acceleration values are listed in Table 2. The design lateral force coefficients were calculated from the 1976 Mexico City building code.

Building Period	Spectral Accel.	Lateral Force Coefficient (K = 0.8)	Damage Potential Index
T (sec)	S_a (g)	k	DPI (fps)
0.00	0.15	0.030	0.00
0.50	0.23	0.045	26.19
1.00	0.31	0.060	52.95
1.50	0.50	0.060	128.10
2.00	0.65	0.060	222.05
2.50	0.60	0.060	256.21
3.00	0.30	0.060	153.72

Table 2. Lateral coefficient and Damage Potential Index values for heavy damage zone of Mexico City earthquake.

The computed DPI values in Table 2 are plotted together with the damage intensity values from Table 1 for comparison in Figure 3. The DPI values correspond well with the damage intensity values and indicate that buildings which have natural periods in the neighborhood of two seconds would exhibit the highest damage potentials. At a period of two seconds, a DPI of 200 fps corresponds to an observed damage intensity of approximately 30% for the damage developed in the Mexico City earthquake.

Damage Potential Index for San Francisco Bay Sites

In view of the relationship observed between the DPI and the actual damage intensities for Mexico City, Seed and Sun (1989), examined the significance of these results for areas of the San Francisco Bay. Three bayshore sites underlain by soft Bay mud were chosen for the purpose of studying the ground motions that are likely to develop on such sites in the event of M $7^1/_4$ and 8+ earthquakes. The characteristics of the two earthquake motions which were used in the analyses are shown in Table 3. In addition to the soft soil sites, Seed and Sun selected several stiff soil sites to analyze for comparison.

Magnitude M	Distance to Fault	Peak Ground Acceleration	Duration	Duration Weighting Factor
$7^1/_4$	6 miles	0.45g	32 sec	1
8+	6 miles	0.55g	75 sec	1.35

Table 3. Characteristics of rock outcrop motions (Seed and Sun, 1989).

The acceleration response spectra for the M $7^1/_4$ earthquake at the three soft soil sites are shown in Figure 4. A representative average spectral shape is also presented on Figure 4. Seed and Sun (1989) also developed a representative soft soil spectrum for the M 8+ earthquake using the same three sites. The representative spectrum shapes for the heavy damage area of Mexico City and the three bayshore sites in San Francisco are compared in Figure 5. The spectral shapes for the bayshore sites are higher than that for Mexico City, primarily because of the close proximity of the San Andreas fault.

While the spectral accelerations anticipated on San Francisco Bayshore sites are higher than those recorded in Mexico City, conclusions regarding damage also require consideration of the higher seismic design provisions employed in San Francisco. Using the recommendations of the 1988 Uniform Building Code, the lateral force coefficient, k, was computed for the three bayshore sites. The computed DPI values for the M $7^1/_4$ and 8+ earthquakes are presented graphically in Figures 6 and 7, respectively. For comparison, the DPI values for both San Francisco stiff soil sites and the heavy damage zone of Mexico City are also shown on Figures 6 and 7. While the computed DPI on bayshore sites is relatively low for the M 7 $^1/_4$ event, the DPI for the M 8+ event is very similar to that computed for the heavy damage zone of Mexico City. The influence of local soil characteristics on damage potential is clearly seen in Figures 6 and 7, where damage intensity on soft soils is predicted to be twice as high as that of stiff soils.

At the three bayshore sites that were analyzed, buildings which have natural periods in the range of 1.5 to 2.5 seconds exhibit the highest damage potentials. Within this range, the peak DPI values are about 200 fps or about 30% damage intensity for mid-rise (10 to 20 stories) buildings.

Using the relationship between the DPI and the observed damage intensity for the heavy damage zone of Mexico City, it is now of interest to examine the significance of these results to areas of the Salt Lake Valley, where the potential for soil amplification has been recognized for some time. The remainder of this report will concentrate on the calculation of DPI values for selected areas of the Salt Lake Valley.

SEISMIC ENVIRONMENT OF SALT LAKE VALLEY

The Salt Lake Valley is situated near the eastern edge of the Basin and Range Province, a region of north-south trending mountains and valleys bounded on the west by the Sierra Nevada Mountains in California and on the east by the Wasatch Mountains in Utah. This region, which is undergoing extensional tectonic stresses, is characterized by diffuse shallow seismicity and Holocene normal faulting. Most of the mountain ranges in the region are bounded by major high angle normal faults (45 to 65 degrees). Typically, the valleys are filled with relatively thick sequences of Tertiary and Quaternary alluvial and lacustrine settlements. The Great Salt Lake occupies the lowest spot in the Great Basin. A succession of lakes, including ancient Lake Bonneville, have occupied the area since late Miocene time (over 12 millions years ago).

Central Utah lies in the Intermountain seismic belt, a belt of earthquake activity that extends north-south through Utah along the eastern margin of the Basin and Range Province, and northward through eastern Idaho and Western Wyoming into Montana. Since 1850, at least 23 independent earthquakes of M 6.0 or greater have occurred in this belt which gives an average recurrence interval of 6 years. Only two earthquakes of M 7.0 or greater have occurred within the belt over the last 85 years and this prevents a meaningful estimate of recurrence for this magnitude event (Smith and Arabasz, 1991). Earthquakes within the belt have been classified as shallow because their foci have typically been at depths less than 20 km.

Major fault zones in the Salt Lake Valley area consist of the Wasatch fault zone and the Oquirrh Mountain fault zone. The East Great Salt Lake fault may be considered as a part of the Oquirrh Mountain fault zone. In addition to these major fault zones, a smaller zone, the West Valley fault zone, lies within the Salt Lake Valley. The Wasatch fault zone consists of at least eight independent segments. Each segment is thought to be capable of producing a M 6.5 to 7.5 earthquake. The segmentation of the fault into smaller lengths is generally thought to preclude a M 8+ event. (Machette et al, 1991). The largest historic earthquakes in the Utah region have been the M 6.6 Hansel Valley earthquake of 1934 and an earthquake of similar magnitude near Richfield in 1901. No earthquakes have generated surface rupture along the Wasatch fault in historic time.

Estimates of the recurrence for large earthquakes ($M > 7.0$) for most segments on the Wasatch fault range from 1500 to 3000 years. The last rupture on the Salt Lake City segment occurred about 1500 ± 250 years ago. Slip rates are typically 0.3 to 1 mm/year and vertical displacements per event are about 2 m for the various segments. Geologic interpretation of trenching studies suggests that each segment ruptures with a "characteristic" earthquake having about the same magnitude, rupture length, and displacement every time the fault breaks (Machette et al, 1987).

Probabilistic Estimates of Ground Motions

A useful way to quantify the ground-shaking hazard from earthquakes at any given location is to calculate probabilistic estimates of peak accelerations. As part of a national study, Algermissen et al (1978, 1982) determined peak horizontal ground accelerations on rock in Utah with a 90% probability of not being exceeded in 10, 50, and 250 years. Algermissen et al (1982) estimated the peak ground accelerations on rock with a 90% probability of not being exceeded in 50 and 250 years as about 0.28 and 0.70 g respectively, within the Salt Lake Valley. The 50 year acceleration represents moderate earthquake shaking and is closely associated with earthquake code design provisions. The 250 year acceleration comes close to that expected from the maximum credible earthquake on the Wasatch fault for many locations in the valley using attenuation relations developed by Seed and Idriss (1982) as well as Joyner and Boore (1988).

Recently, Youngs et al (1987) completed a probabilistic study of the Wasatch Front Corridor and developed estimates of ground shaking which included earthquake recurrence intervals determined from trenching studies of the Wasatch fault. Figure 8 shows the location of contours developed by Youngs et al (1987) to show peak ground accelerations on soil sites. The probabilistic ground accelerations on rock sites were found to be approximately 20% higher than those shown for soil. Youngs et al (1987), indicate that the higher levels of acceleration are primarily associated with large magnitude events on the Wasatch fault.

The probabilistic accelerations predicted by Algermissen et al (1982) and Youngs et al (1987) are summarized in Table 4. In both cases, the values reflect the maximum expected horizontal accelerations on rock. While Youngs et al (1987) predict somewhat higher values, the two studies are in reasonable agreement. Values of peak horizontal accelerations on rock used in this study are also shown in Table 4 and were selected to represent reasonable average values across the valley based primarily on the results of the study by Youngs et al (1987). While it may be argued that acceleration should decrease with distance from the fault, it should also be noted that basin shape effects have been found to increase ground motions near the center of the valley (Murphy, 1989; Hill et al, 1990). As a compromise and in an effort to facilitate comparisons, a constant acceleration has been used across the valley for the 50 yr and 250 yr events.

Source	10% Exceedence Probability	
	50 Year	250 Year
Youngs et al.(1987)	0.30-0.36g	0.60-0.84g
Algermissen et al.(1982)	0.28g	0.60-0.7g
Accelerations used in study	0.35g	0.70g

Table 4. Predicted maximum horizontal accelerations on rock for the Salt Lake Valley.

Characteristics of Rock Outcrop Motions in Salt Lake Valley

In order to evaluate ground response in the Salt Lake Valley, it is necessary to select rock motion records which would be typical of earthquakes along the Wasatch fault. Because of the lack of recorded ground motions along the fault, this selection requires some judgement. A number of investigators (Sadigh et al, 1986; Joyner and Boore, 1988; and Idriss, 1985) have proposed mean spectral acceleration curves for rock motions and these curves are presented in Figure 9 for the case of a M 7.5 earthquake producing a peak acceleration of 0.35 g. In general, the proposed spectral shapes are in very good agreement. Also shown in Figure 9 is the mean spectral shape for rock sites specified by the 1988 UBC code for a peak acceleration of 0.35 g. The code shape generally envelopes the curves proposed by various investigators. Because of the general concensus which appears to exist with regard to the rock spectrum shape, we have selected rock motions for use in the ground response analysis which have response spectra close to the mean rock spectra shown in Figure 9a.

For example, the first rock motion used in this study was a synthetic recording developed by Seed and Sun (1989) to represent a M 7 $^1/_4$ earthquake. This record has a spectrum shape very similar to the shape proposed by Idriss (1985). In addition to the Seed and Sun (1989) record, eight near field rock motion records from the 1989 Loma Prieta earthquake were also selected. These records have spectral shapes which are within one standard deviation of mean rock spectral shape. The response spectra for the nine rock outcrop records are presented in Figure 9b. These nine records represent a reasonable range of ground motions which might be expected due to a rupture on the nearby Wasatch fault. Characteristics of each record prior to any modification are presented in Table 5.

The rock input motions were scaled to produce peak acceleration values of 0.35 g and 0.70 g. In general, the scaling factors ranged from 0.7 to 1.4, which in our opinion, should not lead to excessive error in relation to the other uncertainties involved. It should be noted that inappropriate scaling can lead to an inappropriate frequency content for the acceleration levels involved.

Record	Event	Distance to Rupture (km)	a(max.) (g)	Approx. Mag. M
Gilroy #1 (E-W)	Loma Prieta	15	0.50	7.1
Gilroy #1 (N-S)	Loma Prieta	15	0.43	7.1
Capitola (E-W)	Loma Prieta	14	0.47	7.1
Capitola (N-S)	Loma Prieta	14	0.54	7.1
Corrilitos (E-W)	Loma Prieta	1	0.50	7.1
Corrilitos (N-S)	Loma Prieta	1	0.64	7.1
Santa Cruz (E-W)	Loma Prieta	16	0.44	7.1
Santa Cruz (N-S)	Loma Prieta	16	0.47	7.1
Seed & Sun (1989)	Synthetic	10	0.45	7.25

Table 5. Original rock outcrop motion records and characteristics.

SOIL CONDITIONS IN THE SALT LAKE VALLEY

The northward-trending Salt Lake Valley is bounded on the north by the Great Salt Lake, on the east by the Wasatch Mountains, and on the west by the Oquirrh Mountains. The valley is crossed in its long dimension by the Jordan River. Near the mountain fronts, soil deposits tend to consist of shallow, coarse alluvial deposits overlying bedrock. The four USGS test sites located in this area, exhibit soil profiles dominated by layers of coarse sand and gravel. The average depth to the underlying rock or rock-like layer (V_s=2500 ft/sec) is between 90 and 100 feet. A typical example is the profile at Laird Park (LAI044) shown in Figure A.3 in Appendix A.

Towards the center of the valley, soil profiles contain deep, relatively soft lacustrine sediments deposited by ancient Lake Bonneville. Test site soil profiles in this area predominantly consist of silts, sands, and clays. A typical example is the soil profile at the Blackhawk Duckclub site (DUC053), which is shown in Figure A.13. The highest spectral ratio measured by Tinsley et al (1991), was recorded at this site. The soil profiles for the other test sites analyzed in this report are also shown in Appendix A.

The locations and designations of all the test sites are listed in Table 6. In addition to site locations and designations, the USGS low amplitude spectral ratios are also listed in the same table. The spectral ratios were recorded in the period band of 0.2-0.7 seconds. At most of the test sites, the USGS drilled to 200 feet. However, the average depth to rock at the softer sites is well over 200 feet. In fact, depth to bedrock at the softer sites extends to 4000 feet or more based on studies by Fox (1983). The depth of these deposits as shown in Figure 10, increase towards the center of the valley. It may be observed that the spectral ratio contours also increase as the depth to bedrock increases (see Figure 1).

Site	Design-ation	Location	Recorded Spectral Ratio
SLC006	UGS006	Utah Geologic Survey	2.5
SLC015	AVH015	Alta View Hospital	6.2
SLC017	EAP017	SLC Airport East	8.0
SLC019	WAP019	SLC Airport West (Fire Station)	11.0
SLC029	RIV029	4060 S. 725 W. Jordan River Site	11.6
SLC042	SUN042	Sun. Trng. Ctr., 2675 Michigan	2.3
SLC043	BON043	Bonneville Golf Course	4.5
SLC044	LAI044	Laird Park	1.3
SLC046	ROO046	Roosevelt School	4.6
SLC051	MAG051	Magna Water District, Wellfield	5.5
SLC052	TMP052	Temple Square/Utah Power & Light	4.1
SLC053	DUC053	Blackhawk Duck Club	15.0
SLC056	CCB056	City and County Building	n.a.

Table 6. Designations, locations, and recorded spectral ratios of selected USGS sites (Tinsley et al, 1991).

The response of the deep, soft soil deposits under seismic loading is of concern to many seismologists and engineers. While Salt Lake Valley clays have little in common with Mexico City clay, soil properties within some areas of the valley have similarities with San Francisco Bay muds, where soil amplification was observed in the 1989 Loma Prieta earthquake (see Table 7). Soil property comparisons

Property	San Francisco Young Bay mud	Salt Lake Valley Clay
Sat. Unit Weight	90 - 110 pcf	95 - 120 pcf
Natural Water Content	35 - 90%	25 - 65%
Liquid Limit	40 - 90%	25 - 65%
Plastic Index	20 - 40%	10 - 30%
Undrained Shear Strength	$s_u=0.30\sigma_o'$(N.C.)	$s_u=0.26\sigma_o'$(N.C.)
Compression Index	0.30 - 0.50	0.15 - 0.40
Sensitivity	Low (3 - 4)	Low (2 - 3)

Table 7. Soil properties of San Francisco Bay mud and Salt Lake Valley clay (Rollins, 1991; Seed and Sun, 1989).

indicate that the young San Francisco Bay muds are generally more plastic and more compressible than the Salt Lake Valley clays. However, many of the properties listed in Table 7 have significant overlaps.

Shear Wave Velocity Profiles for Salt Lake Valley Soils

At each of the selected test sites, shear wave velocity measurements were made as reported by Tinsley et al (1991). The results of these measurements are plotted adjacent to the soil profiles in Appendix A. Mean shear wave velocity profiles for sites in the four zones are shown in Figure 11. There is a clear trend of decreasing shear wave velocity from the stiff, shallow sites to the soft, deep soil sites. The average shear wave velocity for soft soil sites in the San Francisco Bay area is also presented in Figure 11 and it may be seen that measured shear wave velocities for the soft Salt Lake soil sites are very similar to those measured at San Francisco soft soil sites.

Site Categorization

While the 13 selected test sites all exhibit different characteristics, properties such as soil consistency, soil depth, shear wave velocity, and recorded USGS low amplitude spectral ratios, can be used to categorize each test site. For example, sites located near the mountain fronts have stiff, shallow soils with high shear wave velocities. Sites closer to the valley center have soft, deep soils and lower shear wave velocities. Correspondingly, the USGS contours of spectral ratios, divide the sites into four zones having spectral ratios of 1 to 3, 3 to 5, 5 to 8, and 8 to 11 (see Figure 1). Three to four of the selected USGS test sites lie within each zone. The four USGS contour zones with the proposed classification system are presented in Table 8.

Shear Modulus and Damping Relationships for Salt Lake Valley Soils

A reasonable estimate of ground response requires that the nonlinear dynamic characteristics of the various soil strata be correctly modeled. As shear strain increases, the shear modulus decreases and the soil damping ratio increases. For the equivalent linear method which is employed in the computer program SHAKE an iterative process is used to ensure that the shear modulus and damping ratio are compatible with the computed shear strains in each soil strata.

USGS Contour Zone	1-3 Zone	3-5 Zone	5-8 Zone	8-11 Zone
Average Depth to Rock (ft)	93 (Shallow)	140 (Deep)	>200 (Deep)	>200 (Deep)
Shear Wave Velocity Range (ft/sec)	600-3000	700-2600	500-1600	450-1400
Soil Consistency	Stiff	Medium Stiff	Medium Soft	Soft
Test Sites within each Zone	UGS006 SUN042 LAI044	BON043 ROO046 TMP052	MAG051 AVH015 CCB056	EAP017 WAP019 RIV029 DUC053

Table 8. Site categorization of USGS test sites.

The shear modulus at small strain (10^{-4}%), G_{max}, was computed from the shear wave velocity profiles measured at each station. The degradation of the shear modulus with shear strain for clay was determined using correlations with the plasticity index (PI) for the soil developed by Sun et al (1988). These correlations are based on dynamic testing of over 70 soils throughout the world and are summarized in Figure 12. The modulus reduction curve used for each soil layer is shown on the boring logs for each site. Modulus reduction curves for sands and gravels were based on correlations proposed by Seed et al (1986). The damping ratio vs cyclic shear strain relationships were also based on standard curves for sand, gravel and clay as proposed Seed et al (1986) and Seed and Idriss (1970).

GROUND RESPONSE ANALYSES FOR SALT LAKE VALLEY SITES

Using the information provided by Tinsley et al (1991), the computer program SHAKE was employed to compute the ground response at each test site. At several of the test sites, soil profiles extended to bedrock, while at others, the shear wave velocity at depth approached or exceeded 2500 ft/sec and could be considered rock-like. However, at some soft soil test sites the shear wave velocity did not reach 2500 ft/sec even at depths of 200 feet or more. In these cases, a number of analyses were performed using various hypothetical depths to bedrock and various assumptions regarding the soil layering below 200 feet. These sensitivity studies demonstrated that the ground surface response was not significantly affected by variations in assumptions about the soil profile below 200 feet. Similar results have been reported by other investigators (Hays, 1982).

The nine rock outcrop records were scaled to 0.35g and 0.70g to represent the 50 and 250 year rock acceleration level respectively. In all, 234 ground motion response spectra were computed. The ground surface response spectra computed for each site and event are shown in Appendix B on Figures B.1-B.13. Solid lines represent ground motion response spectra for each rock motion record.

For each site the mean acceleration response spectrum was calculated based on the computed response spectra for the nine input records. The computed mean acceleration response spectrum for the UGS site (UGM006) is presented in Figure 13. The computed mean spectra for the two additional stiff soil sites in this zone are also shown in Figure 13. Based on the mean spectra for these three sites, a representative response spectrum was drawn to characterize the general shape of earthquake motions in stiff soils (1-3 Zone). A similar approach has been taken for each zone for both rock acceleration levels. The representative spectra for each zone and the computed the computed mean spectra for each site in this zone is presented in Appendix C on Figures C.1-C.4

The representative spectrum has been drawn to envelope the mean response spectra for the various sites. In some cases, the representative spectrum has been drawn somewhat higher than the mean shape computed by SHAKE based on the judgement of the authors. This was done because the representative spectrum must account for variations in the site characteristics and the rock input motions within the zone. It must also account for the inaccuracies which are inherent in the SHAKE program due to the equivalent linear method. SHAKE computations tend to overestimate peak response at the fundamental period of the soil and overdamp response at other periods.

The representative spectra in each of the four soil zones subjected to the 50 and 250 year acceleration levels are shown in Figures 14 and 15. In the short period range, ($T < 0.8$), spectral accelerations are much higher for stiff soils than for soft soils. At a period of 0.4 seconds, for example, spectral accelerations on stiff soils are twice that on soft soils for the 50 year acceleration level. This results from the fact that stiff soil sites have natural periods of about 0.40 seconds. At longer periods, (T > 1.0), the trend reverses and spectral accelerations are significantly greater on soft soils than on stiff soils. This results from the fact that the soft soils investigated in this study have site periods of around 1.8 seconds.

Overall, the stiff sites have the highest spectral accelerations but these high values occur over narrow period bands. The spectral acceleration peaks for the soft soils, while lower, occur over much wider period bands. The contrast between stiff and soft soil response is even greater for the 250 year acceleration case. The peak spectral accelerations also occur at period ranges between 0.1 and 0.9 seconds. It should be recognized that variations from these general trends may occur at specific sites due to variations in soil profiles and soil characteristics. Although the developement of these representative spectra shapes does not eliminate the need for site-specific investigations for more important structures, it does provide some basis for comparison for future ground response studies in the valley.

Peak Ground Accelerations

A summary of computed peak ground accelerations, and site periods is presented in Table 9 for each site in the study. In general, the peak accelerations are highest for the stiff sites and decrease as the soil becomes softer. The natural site period increases from an average of 0.44 seconds for the stiff sites to an average of 1.88 for the soft sites. The peak ground accelerations on soil with relation to peak ground accelerations on rock are plotted in Figure 16 for the stiff (1-3 Zone) and soft (8-11 Zone) soils.

Zone and Site Designation	50 Year Acceleration		250 Year Acceleration	
	a_{max} (avg) g	Site Period sec	a_{max} (avg) g	Site Period sec
1-3 ZONE	0.66	0.44	1.19	0.44
UGS006	0.58	0.55	1.13	0.55
SUN042	0.69	0.51	1.17	0.51
LAI044	0.71	0.26	1.29	0.26
3-5 ZONE	0.44	1.05	0.71	1.05
BON043	0.54	0.88	0.89	0.88
ROO046	0.41	1.16	0.73	1.16
TMP052	0.36	1.12	0.51	1.12
5-8 ZONE	0.40	1.35	0.65	1.35
AVH015	0.46	2.06	0.67	2.06
MAG051	0.34	1.23	0.38	1.23
CCB056	0.39	0.77	0.90	0.77
8-11 ZONE	0.33	1.83	0.45	1.83
EAP017	0.33	1.83	0.43	1.83
WAP109	0.29	1.72	0.43	1.72
RIV029	0.40	1.66	0.48	1.66
DUC053	0.31	2.09	0.45	2.09

Table 9. Summary of ground surface accelerations for 50 and 250 year rock acceleration levels.

In the soft soils, the maximum ground accelerations were within the range predicted by Idriss (1990). However, the maximum accelerations on the stiff soils were unexpectedly higher than those anticipated. These higher ground acceleration levels occur when the natural period of the soil is close to the predominant period of the incoming rock motions. The resulting resonance leads to significant increase of the spectral acceleration in the low period range. Amplification of stiff, shallow soils is not without precedent. During the 1976 Friuli earthquake, peak accelerations on stiff, shallow soil profiles were significantly higher than on rock (Muzzi and Vallini, 1977). Similar results were also obtained using ground response calculations (Seed, 1987).

EARTHQUAKE-RESISTANT DESIGN PROVISIONS

The fundamental aim of earthquake engineering is to provide structures and facilities that are safe for public use even if some damage occurs to the building during a seismic event. The primary function of a building code, according to the Structural Engineers Association of California (SEAOC), is to provide minimum standards to assure public safety and prevent loss of life. Damage limitation is to be sought but is not in itself a code objective. The aim of the code is to design structures that can resist minor earthquakes undamaged, resist moderate earthquakes without significant structural damage even though incurring nonstructural damage, and resist severe earthquakes without collapse (SEAOC, 1991).

Historical Seismic Design Provisions for Structures in Utah

The building code governing construction in the Salt Lake Valley and Utah is the Uniform Building Code developed by the International Conference of Building Officials. In 1927, seismic design provisions appeared in the first edition of this building code. However, the provisions were contained only in an appendix and adherence was not mandatory. It was not until 1961 when the Uniform Building Code adopted seismic design provisions developed by SEOAC, that earthquake-resistant design provisions were mandatory for structures constructed in Utah. Previous to the adoption of the 1961 code, the concept of using lateral earthquake forces on structures was applied mainly to buildings designed for use by the federal government.

In the 1961 Uniform Building Code adopted by Salt Lake City, the seismic design provisions recommended by SEAOC stipulated that the design lateral force for a structure, V, was related to the natural period of the structure, T, the horizontal force factor, K, a seismic zoning factor, Z, and the mass or weight of the structure, W, by the following equation:

$$V = 0.05 \times \frac{Z \times K \times W}{T^{\frac{1}{3}}} \qquad \text{(Eq. 8)}$$

The horizontal force factor is dependent on the type or arrangement of resisting elements of a building.

The SEAOC seismic design recommendations within the Uniform Building Code have been constantly under revision since 1961. In the 1970 UBC Code for example, a new seismic risk map reclassified the Salt Lake Valley and much of the State of Utah as seismic Zone No. 3, equal to that of much of California.

In light of damage caused by the 1971 San Fernando earthquake, major modifications were introduced in the SEAOC code. The recommendations at that time introduced several new ideas into the design provisions, including the concept of soil-structure response interaction and a categorization of the importance of structures. In addition, the seismic zoning map of the United States was modified to include a Zone 4 in part of California. This new zone was composed of areas within Zone 3 which were in close proximity to certain major fault systems, such as the San Andreas. Under this revised map, the Salt Lake Valley and much of Utah remained in seismic Zone 3. The seismic zone factor value is listed in Table 10. The SEAOC recommendations were adopted into the 1976 Uniform Building Code and subsequently the 1976 code was adopted by Salt Lake City.

Under the 1976 UBC recommendations, the design lateral force for a structure was determined by the expression:

$$V = \frac{Z \times S \times I \times K \times W}{T^{\frac{1}{2}} \times 15} \qquad \text{Eq. 8}$$

The soil-structure interaction factor, S, increased from 1.0 to 1.5 when the building period, T, corresponded with the period of the soil, T_s. The importance factor, I, was 1.5 for essential buildings and 1.0 for most common structures.

Current Seismic Design Provisions for Structures in Utah

In 1988, the Seismology Committee of SEAOC recommended that the soil-structure interaction factor, S, in the present code be replaced by a site coefficient, S. This was done because of the difficulty and cost encountered in accurately estimating the site period as well as the effective building period. The site coefficient is determined by the soil characteristics at the site and ranges in value from 1.0 for rock or stiff sites (S_1) to 2.0 for sites containing thick layers of soft clay (S_4). The last soil category, S_4, was added in recognition of the effects observed in the heavy damage zone of the Mexico City where the soft Mexico City clay greatly amplified the earthquake motions in 1985.

In addition to other revisions, the horizontal force factor, K, was replaced by the structural system factor, R_w. The design lateral force, V, is determined by the following expression:

$$V = 1.25 \times \frac{Z \times S \times I \times W}{R_w \times T^{\frac{2}{3}}} \qquad \text{(Eq. 9)}$$

The seismic zone factor, Z, is listed in Table 10.

Code Year (Proposed*)	Design Later Force Equation	Seismic Zone/Zone$_{(max)}$	Z/Z$_{(max)}$ Value
1961	Eq. 6	2/3	0.5/1.0
1970	Eq. 7	3/3	1.0/1.0
1976	Eq. 8	3/4	0.75/1.0
1988	Eq. 9	3/4	0.3/0.4
1994*	Eq. 9	4/4	0.4/0.4

Table 10. Past, current, and proposed seismic design provisions for the Salt Lake Valley.

These recommendations were adopted into the 1988 Uniform Building Code which subsequently was adopted by Salt Lake City. The current 1991 UBC seismic design provisions have not been modified since 1988, however, the UBC Commission in Utah recommended revising seismic zones to establish a zone 4 along the Wasatch fault. Under the proposed changes, the UBC seismic design provisions for the Salt Lake Valley would have become the same as prescribed for San Francisco, and most of coastal California. During the course of this investigation, the zone 4 proposal was voted down and while there are presently no plans to revive this effort it may come up again in the future. Table 10 summarizes the history of past, current, and proposed UBC seismic design requirements for the Salt Lake Valley.

The historical design lateral force coefficients determined by Equations 7, 8, and 9 for Salt Lake Valley soft and stiff soils are shown in Figures 17 and 18. The design lateral force coefficient varies with the natural period of structure, the soil type, and the building structural system. At longer periods, the design requirements for soft soils are generally more stringent than those for stiff soils to account for soil amplification. The design lateral force requirements for Mexico city, enforced prior to the 1985 earthquake, are also shown in Figure 17.

Historically, the design lateral force requirements for Salt Lake Valley soils have increased with the adoption of each new code. However, the 1988 requirements for stiff soils decreased from previous code levels, even falling below the 1970 code at periods longer than 0.75 sec. Considering the levels of computed peak acceleration in shallow stiff soil, this lower level of design will increase the damage potential of low- and mid-rise structures, particularly at the 250 year rock acceleration level.

Using the lateral force coefficient, the damage potential index can now be evaluated for the four soil zones within the Salt Lake Valley. This report will include calculations of the damage potential index for the 1970, 1976, 1988 and the proposed 1994 editions of the Uniform Building Code.

COMPUTATION OF DPI VALUES FOR SALT LAKE VALLEY

Some indication of the structural vulnerability in the Salt Lake area to earthquake shaking can be obtained by comparing the computed ground motion spectra for soil conditions throughout the valley. However, since seismic code requirements also vary depending on the soil conditions, one must also consider the structural resistance in estimating the potential for earthquake damage. The damage potential index (DPI) combines the effects of differences in earthquake shaking intensity, code requirements, and soil conditions and allows a more rational evaluation of the earthquake hazard. Levels of damage intensity can also be correlated with the DPI values as discussed previously. The damage intensity is the percentage of structures of a given type (period) which would sustain significant damage due to earthquake shaking.

In calculating the DPI, a value of 1.0 was assigned for the duration weighting factor (DWF). This represents the expected duration of strong ground shaking for a M 7.0-7.5 earthquake on the Wasatch fault. As mentioned previously, the building resistance factor, R_f, was assigned a value of 1.2. This factor relates the varying code requirements and construction standards of Salt Lake, San Francisco, and Mexico City. Graphical representation of computed DPI values are shown in Figures 19 to 26 for the various codes in force from 1970 through 1992.

Damage Potential Index, 1970 UBC

The computed DPI values for the 1970 Uniform Building Code are shown in Figures 19 and 20 for the 50 year and 250 year acceleration levels respectively. Using correlations developed by Seed and Sun (1989), the damage intensity associated with various DPI values is also presented in these figures.

In the 1970 code, no soil factor was incorporated into computation of the design lateral force coefficient. Accordingly, the DPI values for the soft to medium soft soils are significantly higher than on the stiff to medium stiff soils at the longer periods ($T > 1.0$), indicating the greater vulnerability of multi-story structures constructed on soft, deep deposits in comparison with those on the stiffer alluvium. At the 250 year level, DPI values in soft soils are comparable to those developed in the heavy damage area of Mexico City. The expected damage intensity for long-period structures ($T > 1.0$) constructed over these softer deposits is approximately 30 percent.

Damage Potential Index, 1976 UBC

In the 1976 edition of the Uniform Building Code, the introduction of the soil/site coefficient greatly reduced the disparity between the soft and stiff sites. As a result, the computed DPI levels shown on Figures 21 and 22 were moderately less than values computed for the 1970 UBC. In the long period range ($T > 1.0$) the maximum damage intensity values on soft soils are on the order of 15 and 20 percent for the 50 and 250 year accelerations respectively. This represents a reduction in damage intensity of approximately 20 to 25 percent over the 1970 UBC requirements. Even with the addition of the soil/site coefficient, the DPI is still somewhat higher for soft soils than stiff soils. Ideally, the building code should equalize the potential for damage on all types of soils.

Damage Potential Index, 1988 UBC

The requirements of the 1988 Uniform Building Code tend to equalize the DPI values for structures with natural periods greater than 0.8 seconds for 50 year rock acceleration levels (see Figure 23). However, this trend does not continue at higher levels of ground shaking. As shown in Figure 24, DPI values for stiff soil exceed that of soft soils in the entire period band.

In a previous section, calculation of the lateral force coefficients showed that the stiff soil design requirements in the 1988 UBC were below those of the 1970 UBC at periods greater than 1.0 second. At higher levels of ground shaking the lower lateral force requirement escalates the damage intensity to levels even greater than computed with 1970 UBC code provisions.

The damage intensity values computed for structures on stiff soil sites are approximately 13 and 25 percent in the long period range (1.5 to 2.5 seconds) for 50 and 250 year rock accelerations respectively. Although these damage intensities are less than those corresponding to the heavy damage area of Mexico City in 1985, they demonstrate the need to examine code requirements for stiff soils in Salt Lake Valley. In the case of Mexico City, damage potential index values reached levels of 250 fps and nearly 30 percent of the mid-rise 9- to 12-story buildings (typically with periods between about 1.5 to 2.5 seconds) located inside the heavy-damage zone either collapsed or suffered major damage.

The damage potentials for soft soil conditions appear to be significantly lower than those for the 1985 Mexico City earthquake, indicating a much lower degree of vulnerability than that exhibited by buildings in Mexico City.

Damage Potential Index, Proposed 1994 Zone Change

As mentioned previously, a proposal was recently made to upgrade the Salt Lake Valley and areas near the Wasatch fault to seismic Zone 4. While this proposal was eventually rejected, it is still of interest to evaluate the consequences of such a change. The calculated DPI values for the higher seismic zone are shown in Figures 25 and 26 for the 50 and 250 year rock acceleration levels respectively. Under the proposed code, peak damage potential index values drop to 70 and 110 fps for the 50 and 250 year rock accelerations respectively. The 25 percent increase in the lateral force coefficient due to the change in seismic zone reduces the damage intensity by about 5 percentage points over the entire period range.

It is not the purpose of this report to suggest necessary actions for implementation of building codes, this can only be done by professional organizations. However, an economic assessment of expected increases in construction costs for upgrading the Salt Lake Valley to zone 4, compared to the savings in the event that a moderate to large earthquake were to occur along the Wasatch fault, could provide additional insight. The information provided in this study would be useful in making an assessment of this kind. Other non-technical factors, such as the ability of the Utah economy to recover from a major earthquake might also need to be examined in making recommendations for code revisions.

COMPARISON OF SOFT SOIL RESPONSE IN SALT LAKE AND SAN FRANCISCO

Based on the USGS low-strain measurements, amplification of earthquake motions by deep soil sites in the Salt Lake Valley is a significant concern. These studies suggest that soft soils in the valley might act similarly to soils in San Francisco and Mexico City where distant earthquake motions amplified ground motions and increased damage intensities.

Soft soil comparisons for Mexico City, San Francisco, and Salt Lake are shown on Figures 27 and 28. As shown in Figure 27, the DPI values for 50 year rock acceleration levels in Salt Lake are about equal to that computed for soft soils in San Francisco. While the comparisons represent response to roughly similar earthquake events (M $7^1/_4$, $a_{max}=0.35$ g), it should be recognized that the earthquake recurrence intervals for Utah are much longer than for San Francisco. DPI values for both Salt Lake and San Francisco are significantly lower than for Mexico City. The change from seismic zone 3 to 4 would bring the Salt Lake damage potential level below that computed for San Francisco.

A comparison of DPI values for maximum credible earthquake events in San Francisco and Salt Lake City is presented in Figure 28. For San Francisco the maximum credible earthquake is a M 8+ earthquake on the San Andreas fault, while for Utah it would be a M 7.5 earthquake on the Wasatch fault. The damage intensity on soft Salt Lake Valley soils would only be 50 to 60% of that predicted for soft soils in San Francisco or observed in Mexico City. Therefore, while many soft soils in the Salt Lake Valley have properties similar to those found in San Francisco, they do not appear to produce the same degree of damage when subjected to large earthquake motions. This is a reflection of the lower plasticity and greater non-linearity of soft soils in Salt Lake in comparison with soft soils in San Francisco.

Finally, it should be noted that this investigation has concentrated on soil response and has not considered the influence of the basin shape on the input rock motions. If preliminary studies regarding amplification due to basin shape are confirmed, base input motions for the soft soil sites will be significantly higher than input motions used in this study. This could lead to significant increases in the DPI values at soft soil sites.

COMPARISON OF STIFF SOIL RESPONSE IN SALT LAKE AND SAN FRANCISCO

Stiff soil comparisons for San Francisco and Salt Lake are shown in Figures 29 and 30. Ground response calculations indicated that stiff, shallow soils in Salt Lake City exhibited high spectral accelerations in the short period range. Figure 29 compares the damage potential index for stiff soils found in the Salt Lake Valley with those found in San Francisco for roughly equivalent earthquake events (M $7^{1}/_{4}$, a_{max}=0.35 g).

The damage potential index values for stiff sites in Salt Lake City are somewhat higher than for similar soils in San Francisco. The differences are particularly large at periods around 0.5 seconds where the damage potential index is nearly twice as high for Salt Lake soils as for San Francisco soils. A comparison of stiff soil response due to the maximum credible earthquake is presented in Figure 30. In this case, the damage potential index values are substantially higher for Salt Lake City soil in comparison with San Francisco. Under the 1988 code provisions, the computed damage intensity approaches that observed for the Mexico City earthquake. If Salt Lake were to adopt seismic zone 4 requirements the damage intensity values would decrease significantly but they would still be much higher than for San Francisco sites. These results suggest that structures constructed on stiff, shallow soils in Salt Lake City will be subjected to increased risks of damage over a relatively large period range. Some justification for a move from seismic zone 3 to zone 4 may be warranted based on these results.

SUMMARY

It has been recognized for some time that local ground conditions can significantly influence the characteristics of ground motions throughout an area. Recently, amplification of earthquake shaking due to soft soils has been observed in a number of earthquakes. As a result, seismologists and engineers have become concerned about the effect of soft soils in the Salt Lake Valley. Previously, research related to soil amplification in the Salt Lake Valley was generally limited to measurements of low-amplitude ground motions after nuclear detonations in Nevada.

The research contained in this study examined the extent of soil amplification in four geologically similar areas of the Salt Lake Valley, and estimated ground motion characteristics using computer models. Analyses were performed for 13 sites where relatively good information was available regarding soil profile, shear wave velocity, and basic soil properties to significant depths ($\approx$200 ft). The 13 sites provided an excellent cross-section of the conditions across the valley. Representative response spectra were then developed for each of the four areas.

In order to evaluate the significance of the various response spectra in relation to potential structural damage, the Damage Potential Index (DPI) was computed for each area using techniques developed by Seed and Sun (1989) based on their studies of the 1985 Mexico City earthquake. The DPI relates earthquake induced forces to the resisting forces prescribed by code provisions. The DPI value has also been correlated with the damage intensity so that approximate estimates of the percentage of buildings which would be damaged could be made based on the DPI value. With the use of the DPI values it has been possible to make comparisons of the relative protection provided by various seismic codes employed in Salt Lake over the past 30 years as well as those recently proposed for use. Finally, our study allows a comparison between DPI values computed for areas of San Francisco and Mexico City so that some perspective on the nature of the problems in Salt Lake Valley can be obtained.

CONCLUSIONS

1. The Salt Lake Valley can be roughly divided into four zones based on geology, soil conditions, and probable ground response. These four areas have been designated as (1) stiff, shallow soils (USGS zone 1-3), (2) medium stiff, deep soils (USGS zone 3-5), (3) medium soft, deep soils (USGS zone 5-8), and (4) soft deep soils.

2. Representative spectral shapes have been computed to characterize ground response in each of the four zones. At the 50 year acceleration level, response spectra shapes for stiff sites have the highest spectral acceleration, but these high values occur over narrow period bands. The spectral accelerations for the soft soils, while being lower, occur over much wider period bands. The contrast between stiff and soft soil response is even greater for the 250 year acceleration level.

3. Peak accelerations are highest for the stiff sites (1-3 Zone) and decrease as the soil becomes softer. These higher ground acceleration levels occur when the natural period of the soil is close to that of the incoming rock motions. The resulting resonance leads to significant increases of the spectral acceleration in the low period range. In the soft soils, the maximum ground accelerations were within the range predicted by Idriss (1990).

4. Damage potential index values computed for the 1970 Uniform Building Code indicate a greater vulnerability of multi-story structures constructed on soft, deep soils due to the lack of soil factors in the design lateral force coefficient equation. Existing multi-story structures built under the 1970 Code in soft soil should be examined and retro-fitted as required.

5. Damage potential index values computed for the 1976 Uniform Building Code demonstrated the advantages of a soil/site coefficient which somewhat reduced disparities between damage intensity levels in stiff and soft soils.

6. The current, 1988, Uniform Building Code appears to provide reasonable protection for the entire valley for the 50 year rock acceleration level. Damage potential index values for all soil types are similar to the level which would be expected in San Francisco for similar earthquake motions. For the 250 year acceleration levels, the potential for damage is significantly greater and much damage would be expected throughout the valley. While damage intensities on soft soils would be less than for Mexico City, damage on stiff soils would approach that in the Mexico City.

   The large increase in damage associated with the 250 year acceleration levels is not entirely unexpected. Earthquake codes are generally formulated to prevent structural damage only for moderate levels of shaking. For maximum credible earthquake conditions, the goal is to prevent building collapse since it becomes uneconomical to prevent structural damage. Thus, a significant degree of structural damage would have to be accepted for the 250 year acceleration levels. At 250 year acceleration levels DPI values for stiff soil exceed that of soft soils in the entire period range. This indicates the need to reevaluate code requirements for stiff soils in the Salt Lake Valley.

7. The proposed 1994 Uniform Building Code would upgrade the Salt Lake Valley to seismic Zone 4. Damage potential index values in soft soil would be lower than those of San Francisco soft soils. However, in the case of stiff soils, DPI values are much higher than those expected on San Francisco stiff soils. This provides some justification for change from Zone 3 to Zone 4. An economic assessment of expected increases in construction costs for changing to Zone 4 compared

to the savings in the event that a moderate to large earthquake were to occur along the Wasatch fault, could provide additional insight on the desirability of the change.

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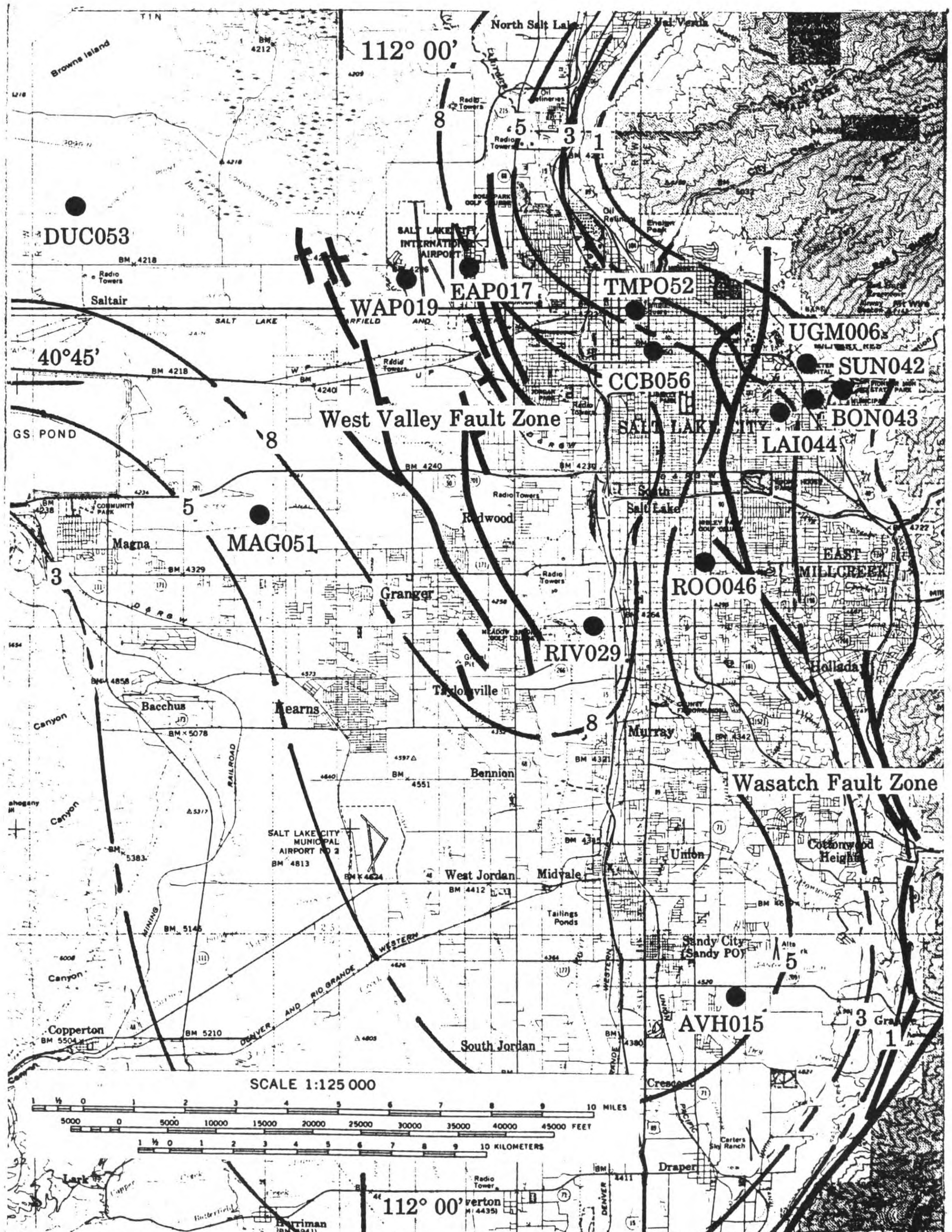

Figure 1. Test site locations, major fault systems, and contours of constant average spectral ratios on alluvium relative to bedrock in the Salt Lake Valley (after Tinsley et al, 1991).

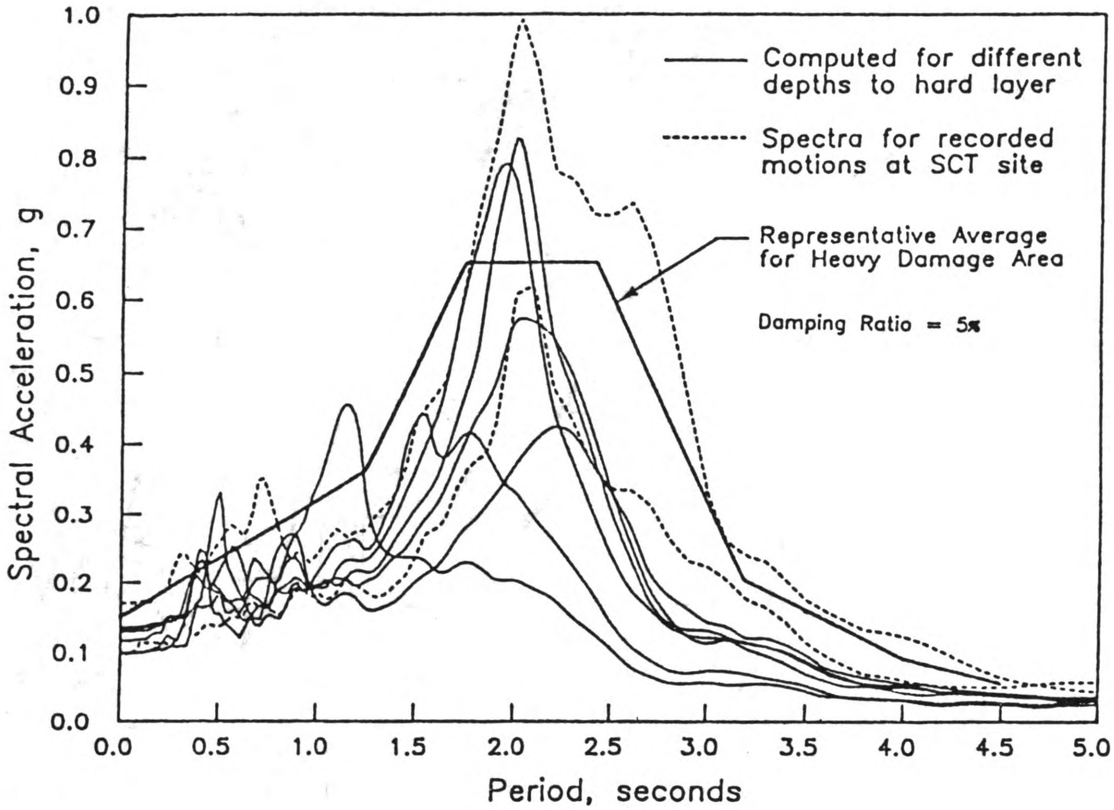

Figure 2. Computed and recorded spectra for heavy damage zone of Mexico City for soil depths ranging from 25 to 45 meters (Seed et al., 1987).

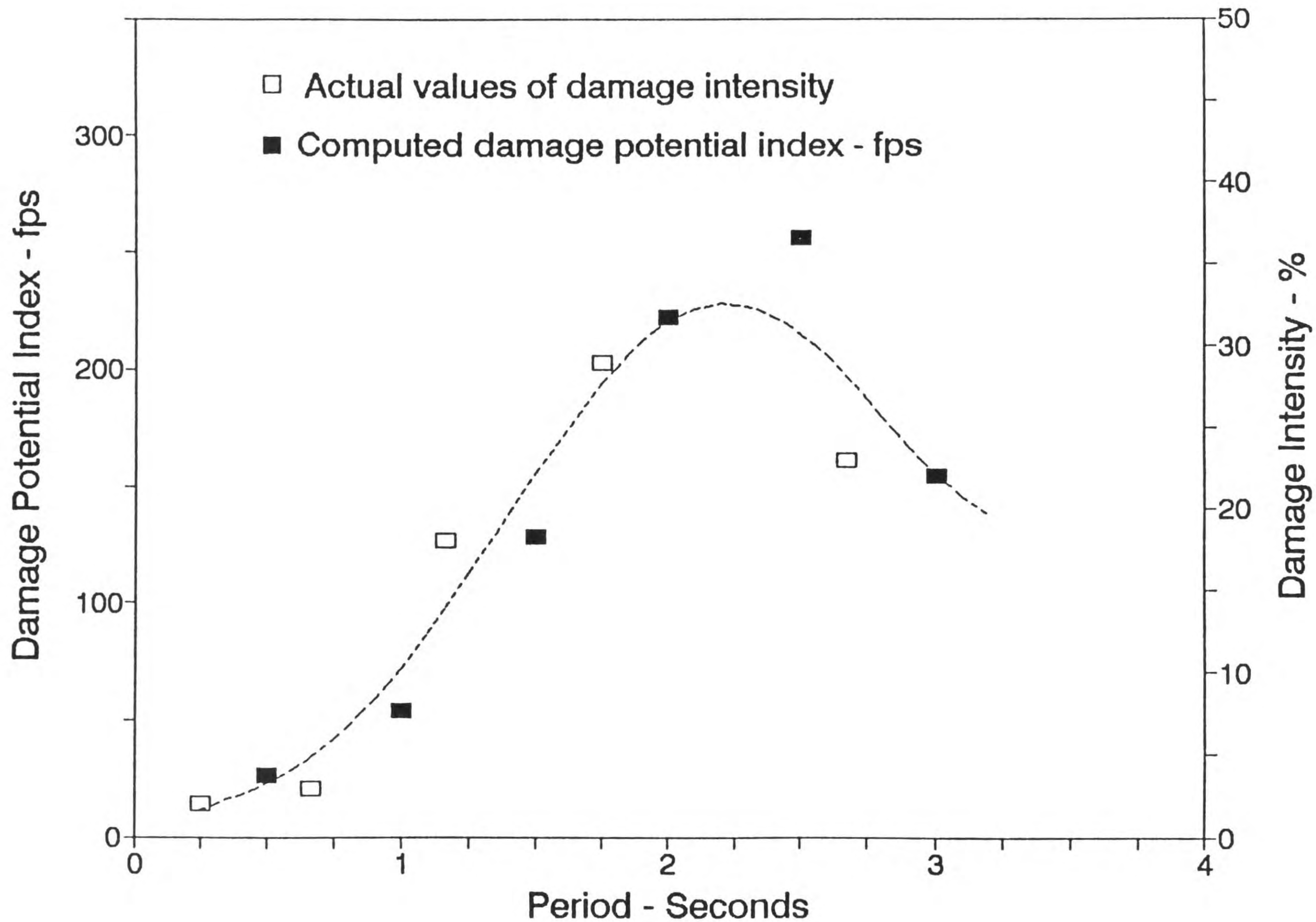

Figure 3. Damage intensity and calculated damage potential index values for heavy damage zone in Mexico City (after Seed and Sun, 1989)

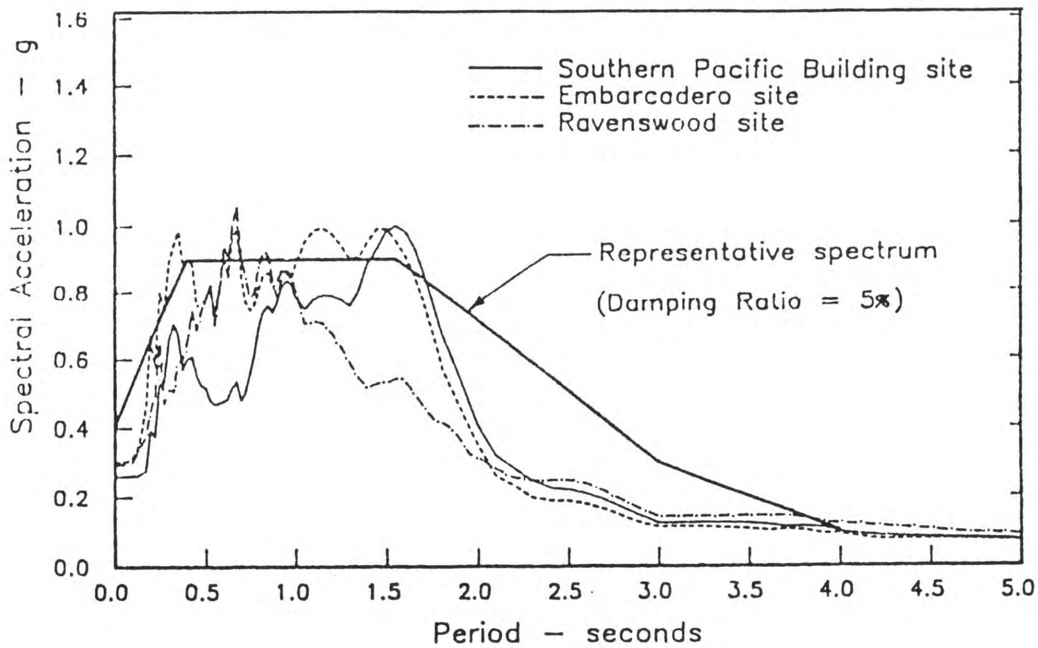

Figure 4. Acceleration response spectra for three San Francisco bayshore sites for a magnitude 7.25 earthquake (Seed and Sun, 1989).

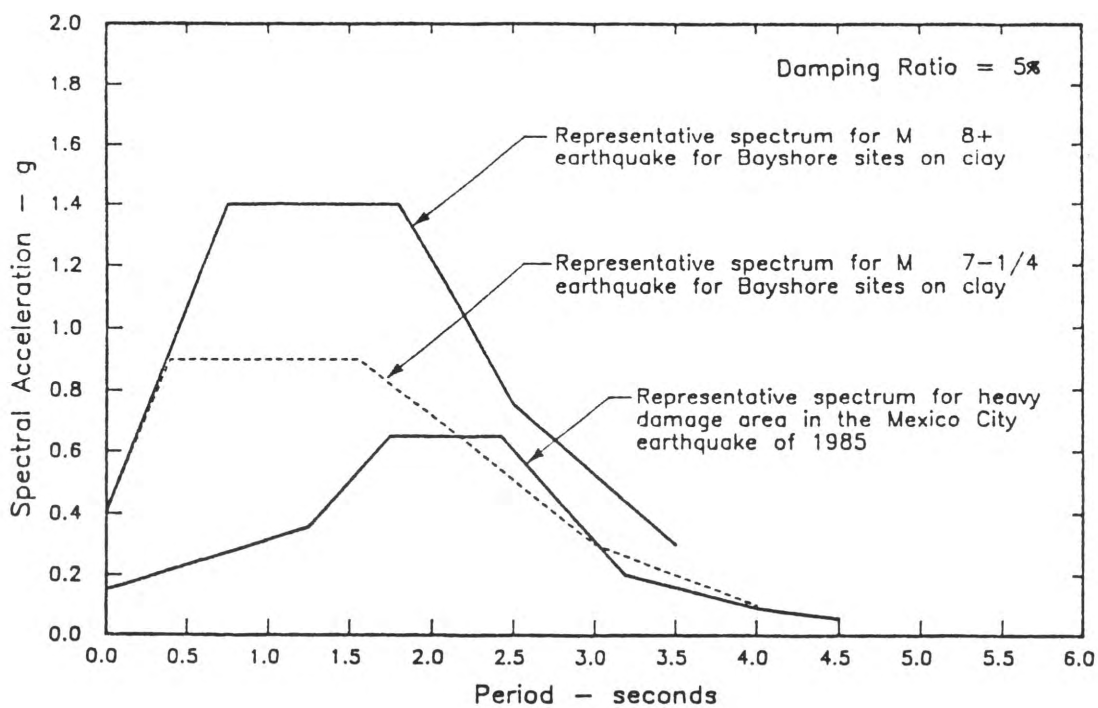

Figure 5. Comparison of representative spectrum for heavy damage zone of Mexico City and San Francisco Bayshore sites for magnitude 7.25 and 8+ earthquakes (Seed and Sun, 1989).

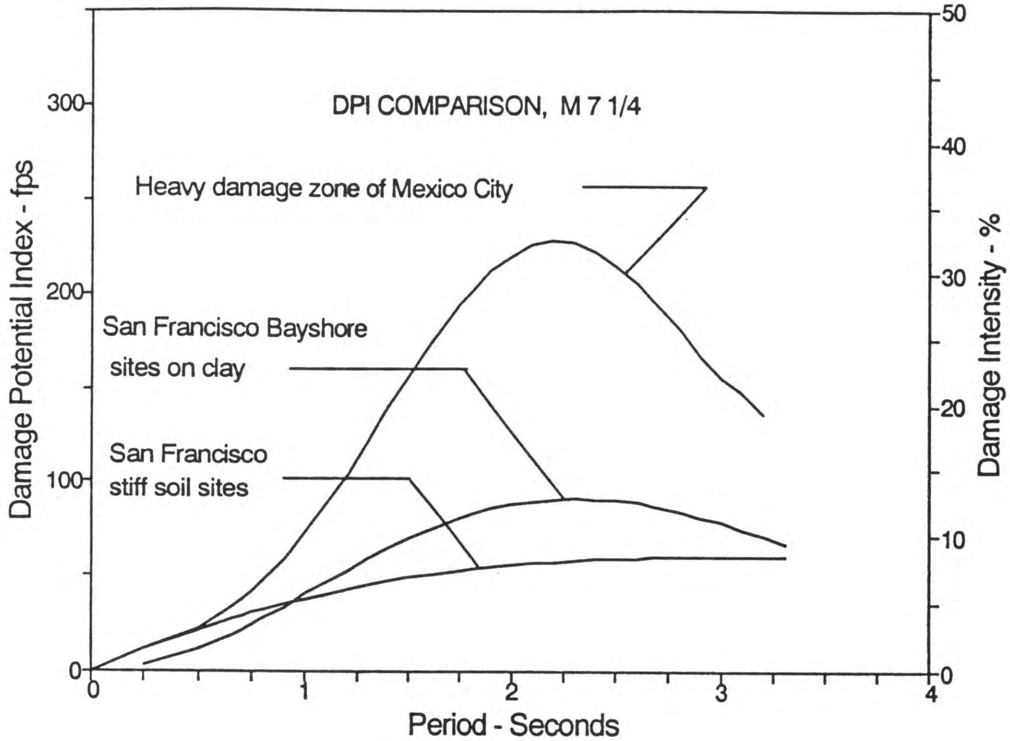

Figure 6. Comparison of damage potential index values for heavy damage area of Mexico City and San Francisco sites for M 7.25 earthquake (after Seed and Sun, 1989).

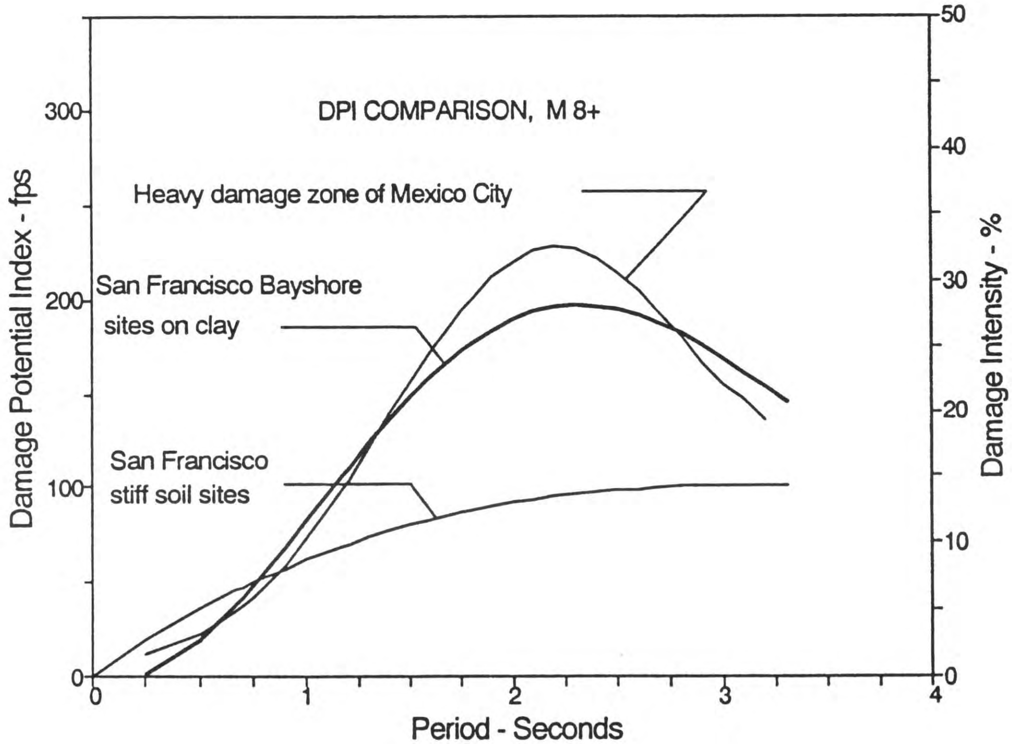

Figure 7. Comparison of damage potential index values for heavy damage area of Mexico City and San Francisco sites for M 8+ earthquake (after Seed and Sun, 1989).

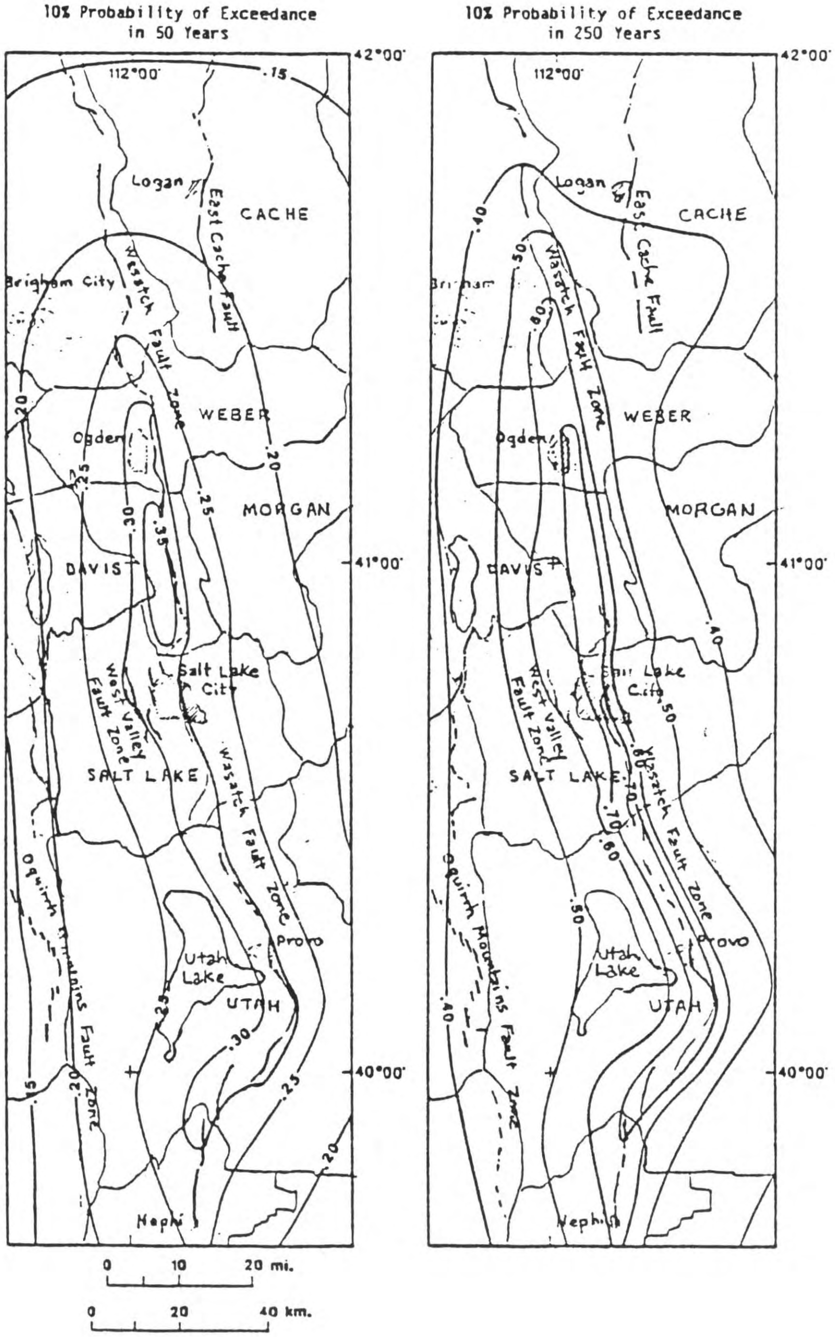

Figure 8. Peak ground acceleration contours on soil sites with a 90% probability of not being exceeded in 50 and 250 years (modified from Youngs et al., 1987).

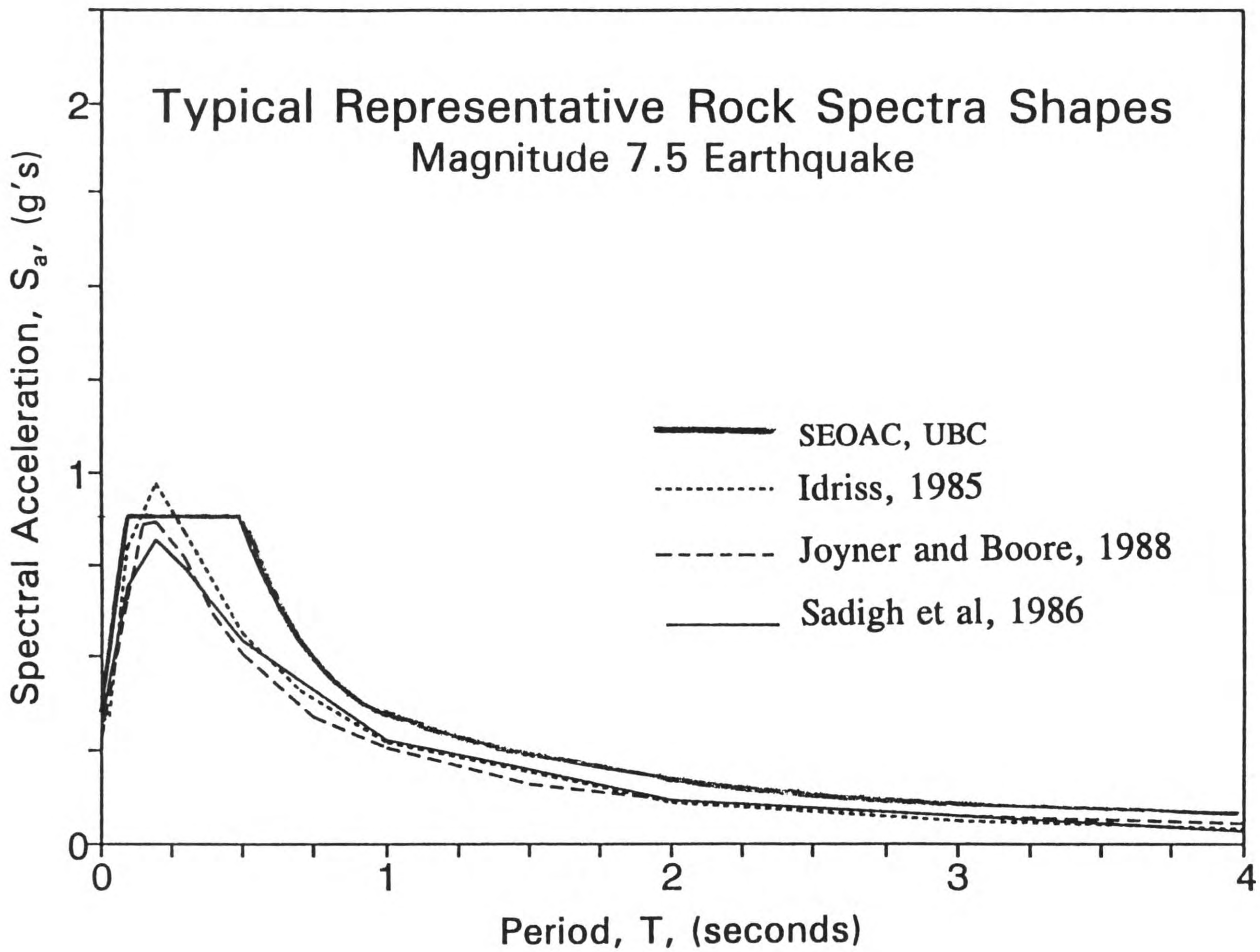

Figure 9a. Typical representative spectra for rock outcrop motion due to a M 7.5 earthquake with a peak acceleration of 0.50 g. (Idriss, 1982: Joyner and Boore, 1988; Sadigh et al, 1986; UBC, 1988)

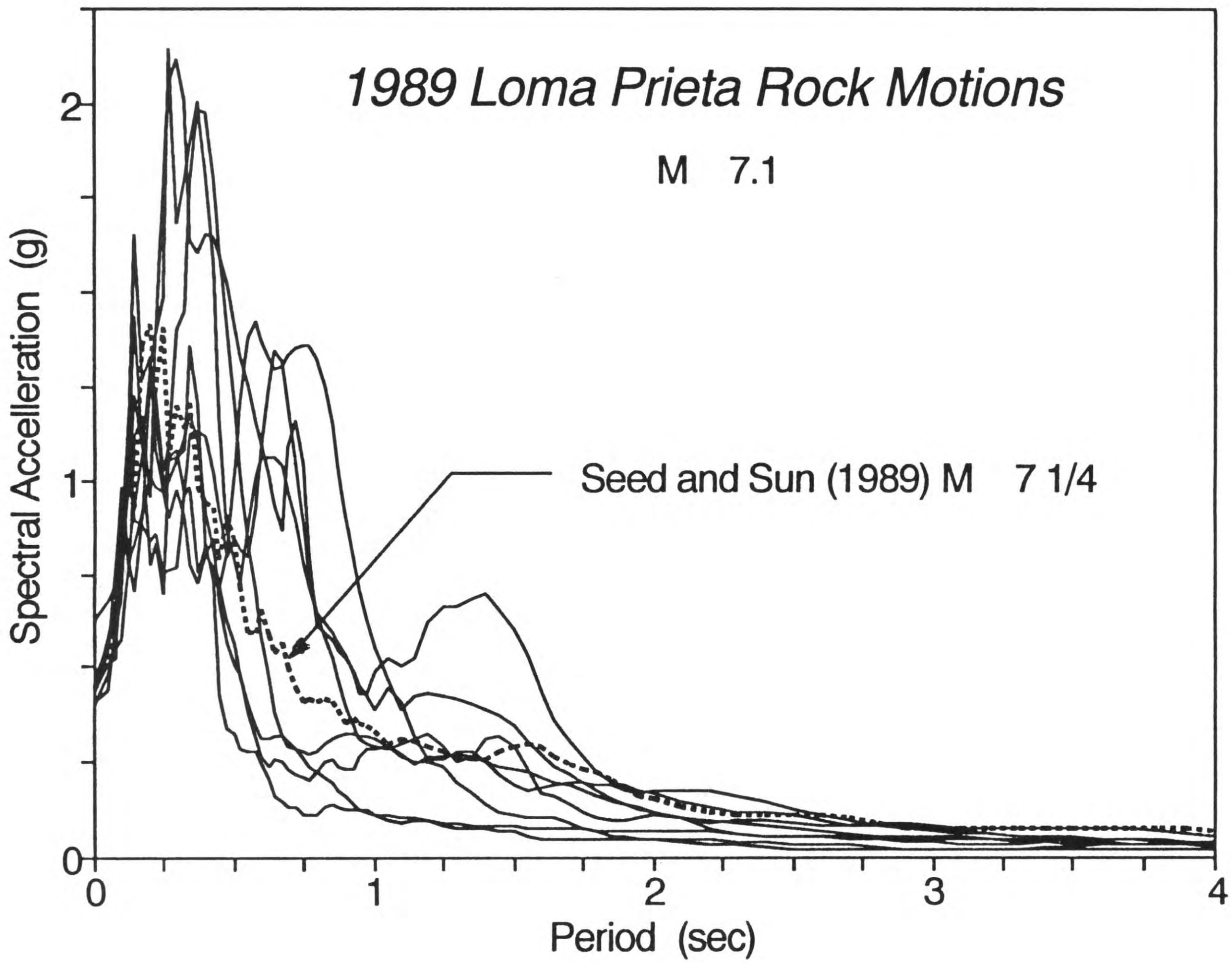

Figure 9b. Acceleration response spectra for eight Loma Prieta rock motions and Seed-Sun M 7.25 rock motion used as input motions in this study.

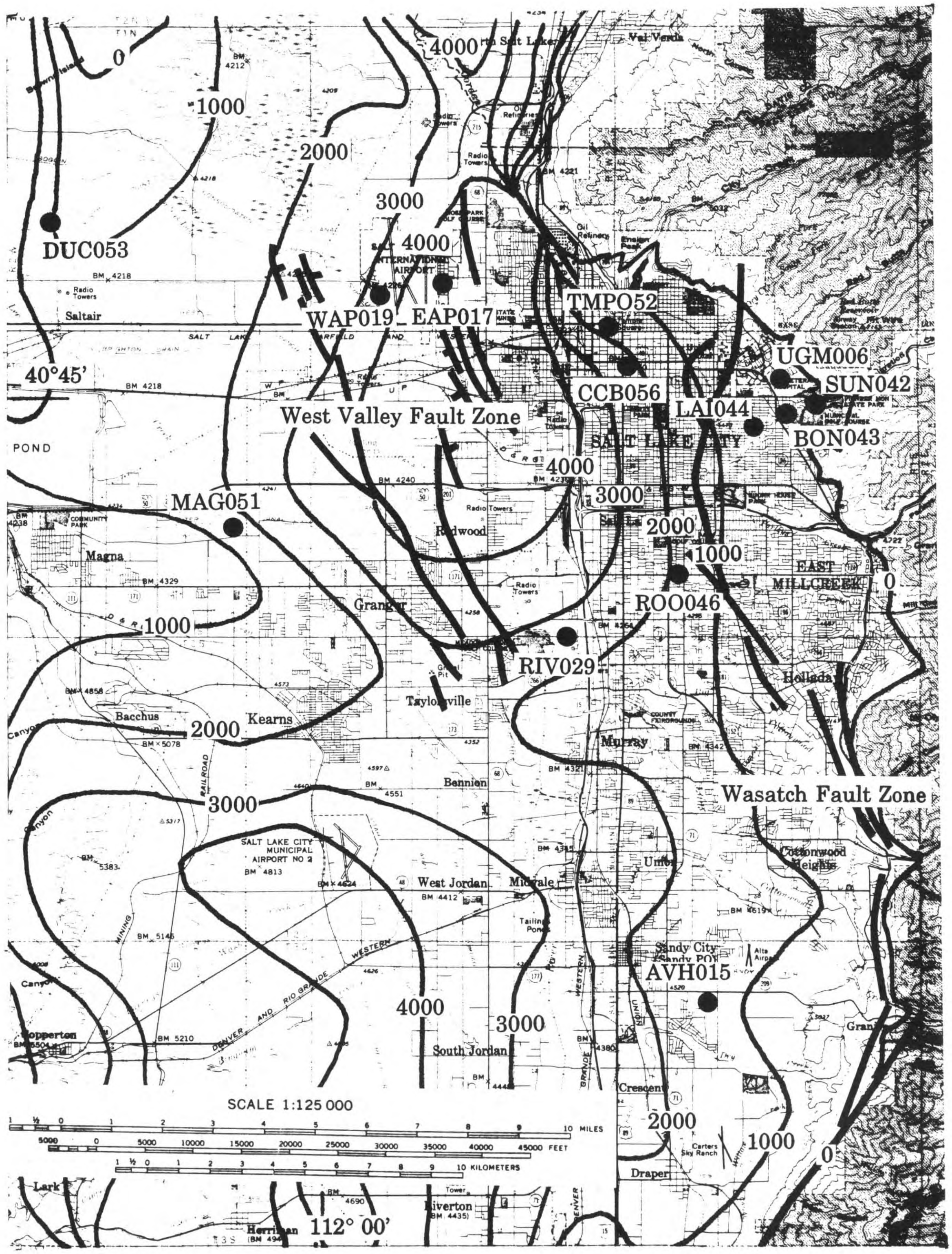

Figure 10. Approximate depth to bedrock in Salt Lake Valley in feet with relation to surrounding geography, instrument stations, and local fault systems (modified from Fox, 1983)

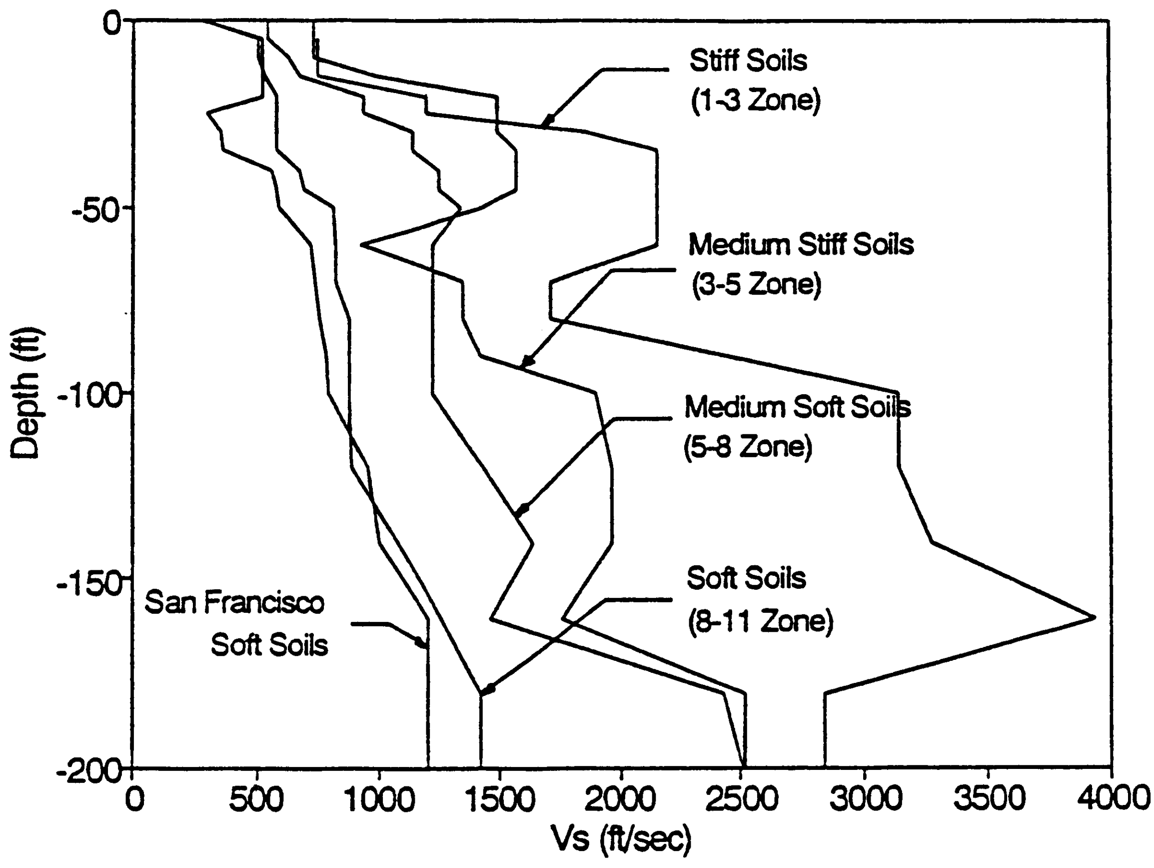

Figure 11. Average shear wave velocity profiles for Salt Lake Valley site and soft soil sites near San Francisco Bay (Seed and Sun, 1989; Tinsley, et al, 1991).

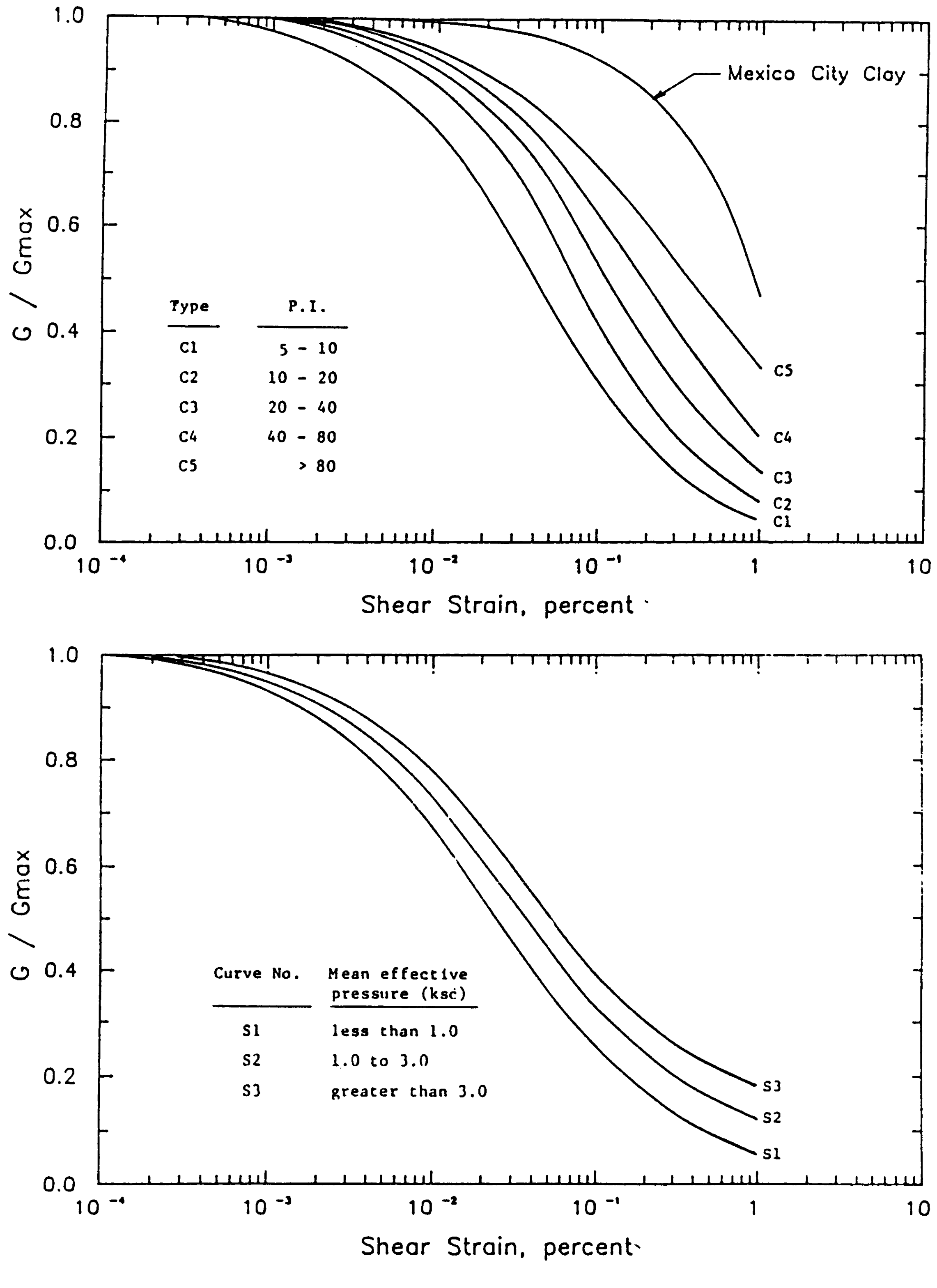

Figure 12. Modulus reduction relationships for clays with different plasticity indices and sands with different mean effective pressures (Sun et al, 1988; Seed et al, 1986)

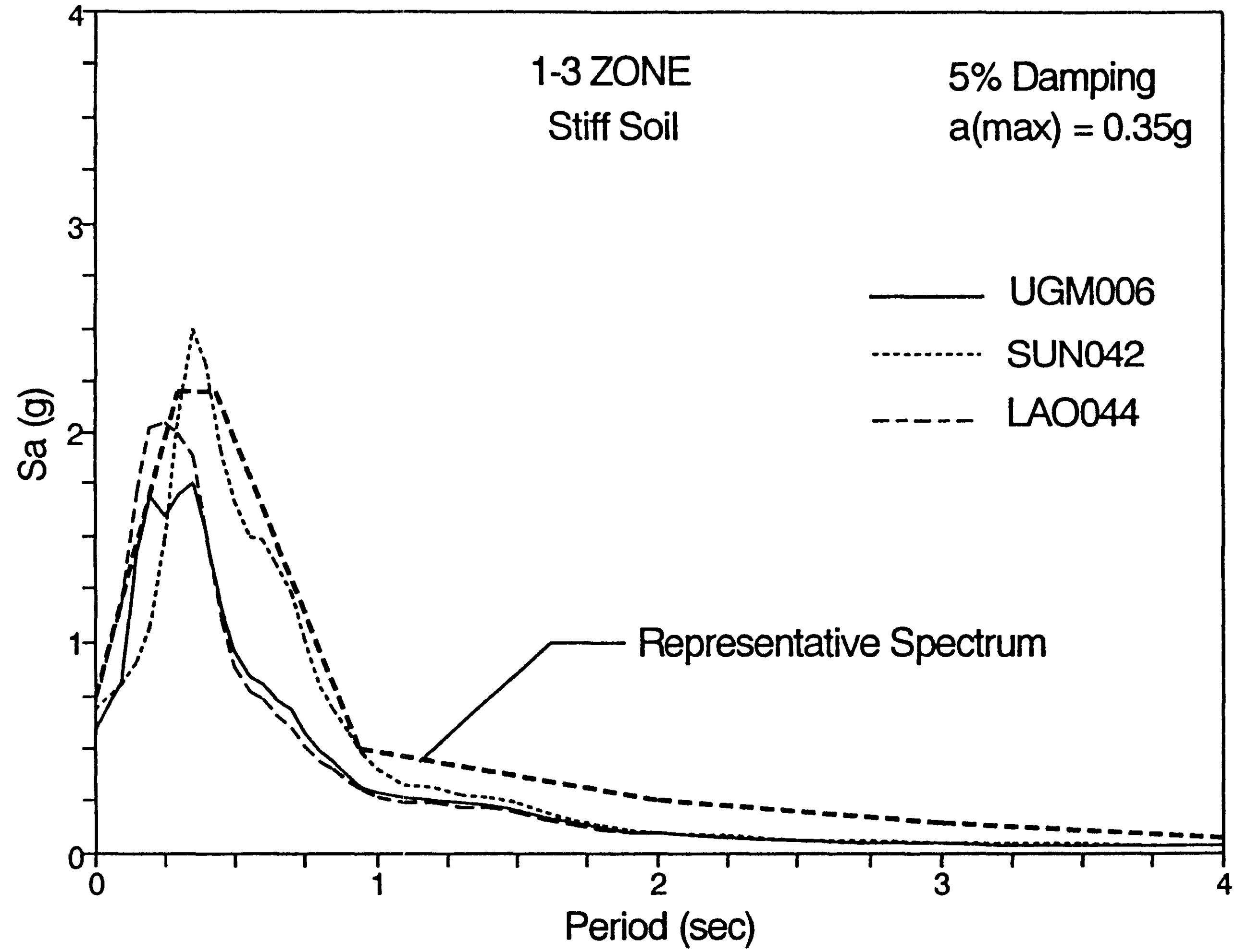

Figure 13. Mean acceleration response spectra and representative spectrum for stiff soil (1-3 Zone) at the 50 year rock acceleration, a_{max}(rock)=0.35g.

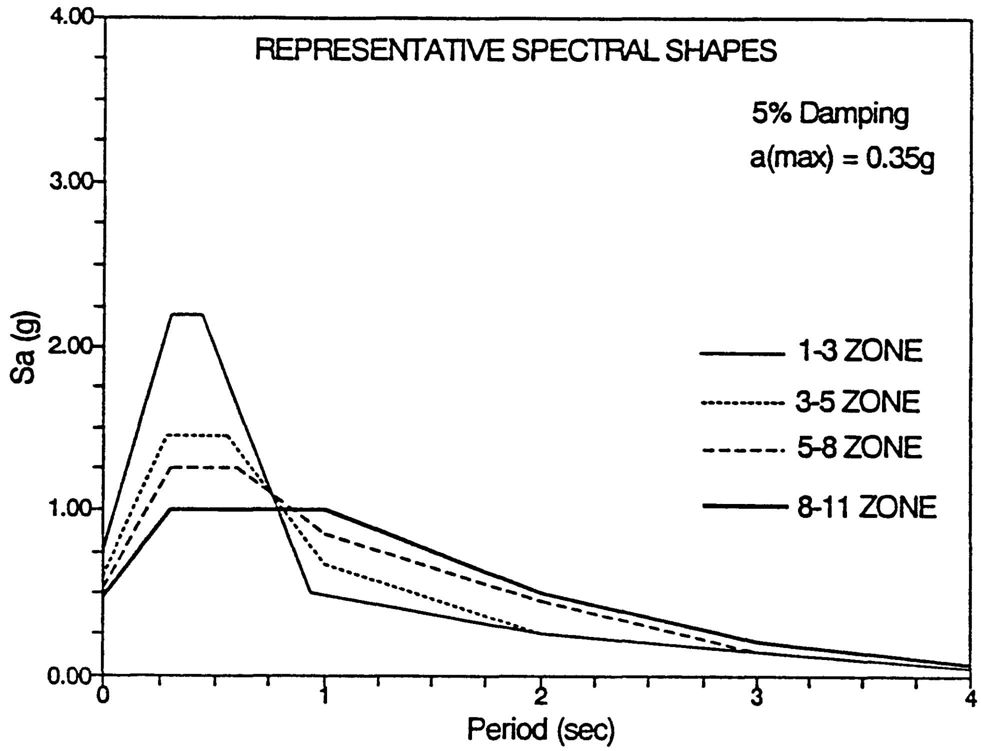

Figure 14. Representative spectral shapes computed using the 50 year rock acceleration with a 10% probability of exceedance (0.35g) as input.

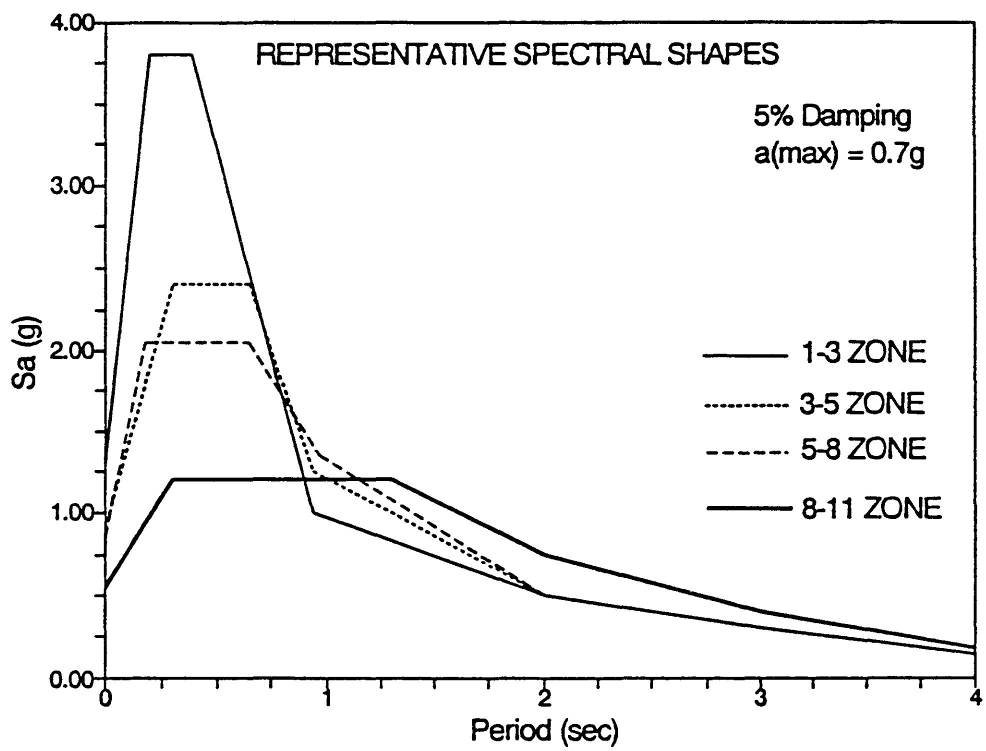

Figure 15. Representative spectral shapes computed using the 250 year rock acceleration with a 10% probability of exceedance (0.70g) as input.

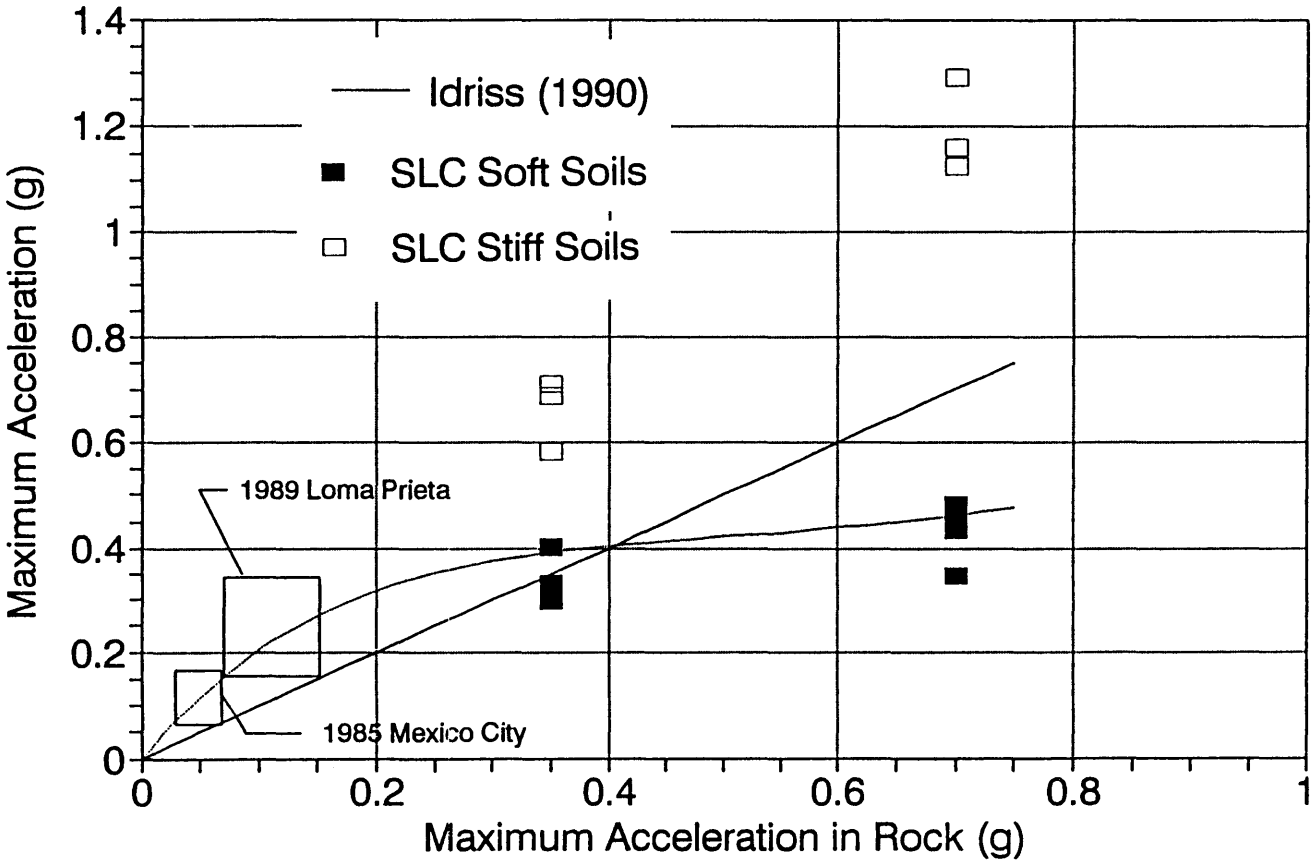

Figure 16. Maximum acceleration on rock vs. maximum acceleration on soil at ground surface.

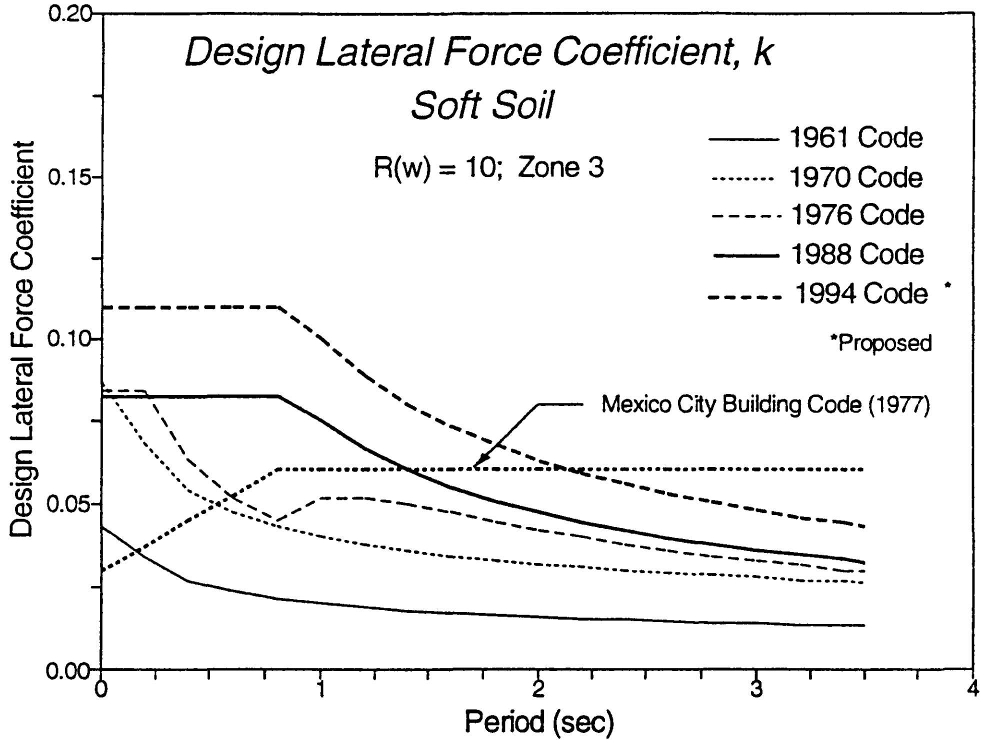

Figure 17. Historical design lateral force coefficients required for Salt Lake Valley soft soil and Mexico City (1977) soft soil.

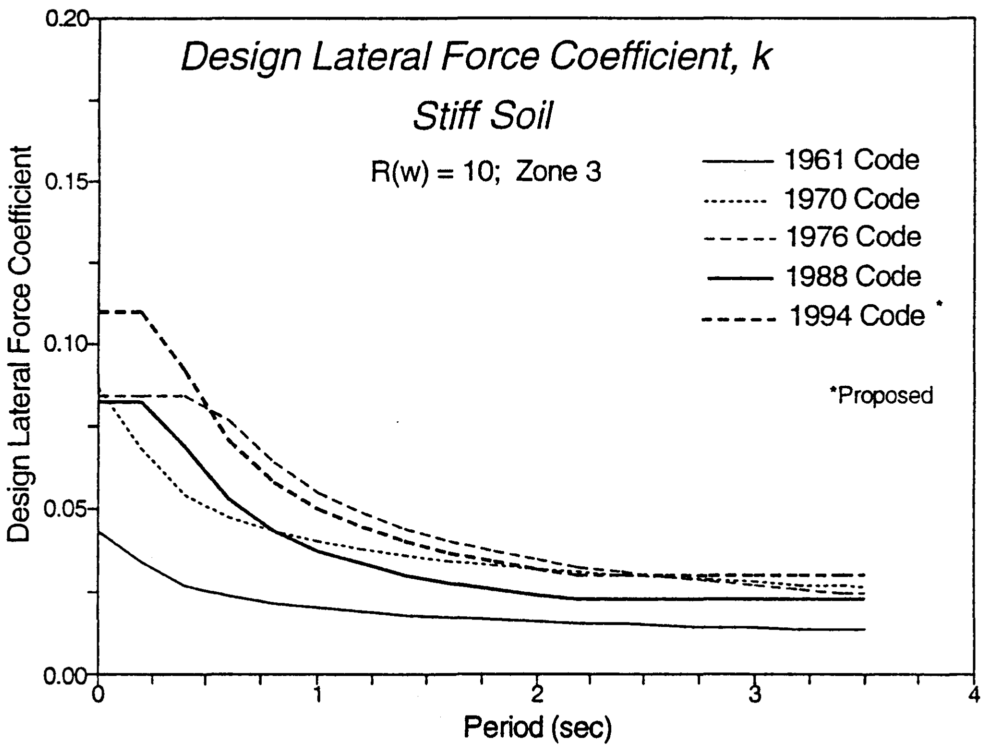

Figure 18. Historical design lateral force coefficients required for Salt Lake Valley stiff soils.

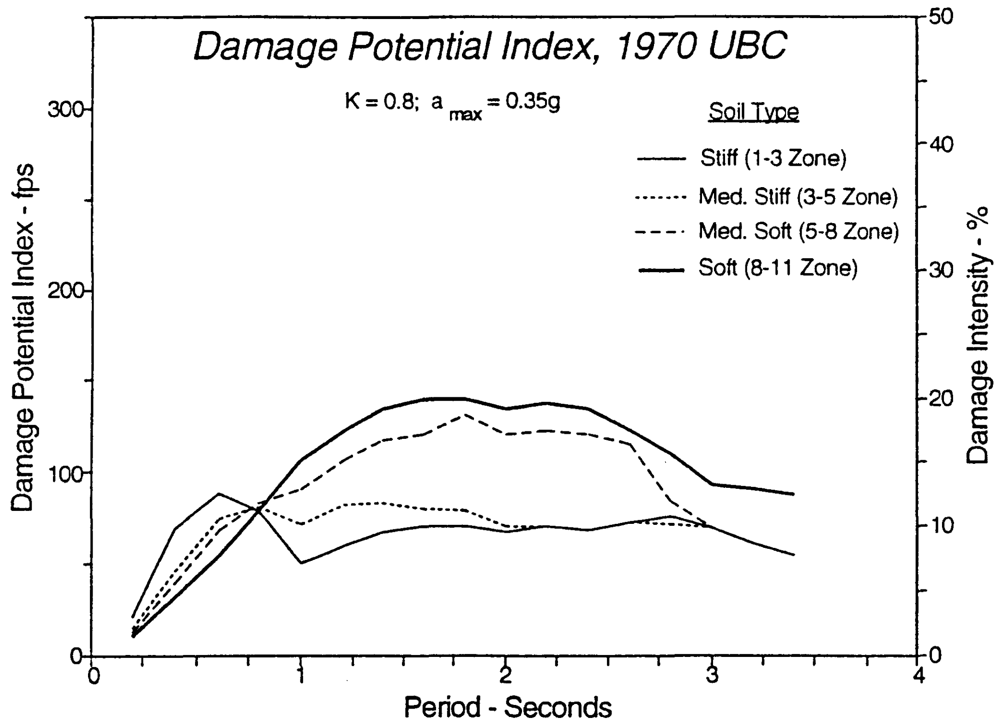

Figure 19. Comparison of DPI values for Salt Lake Valley soils at 50 year accelerations in accordance with 1970 UBC requirements.

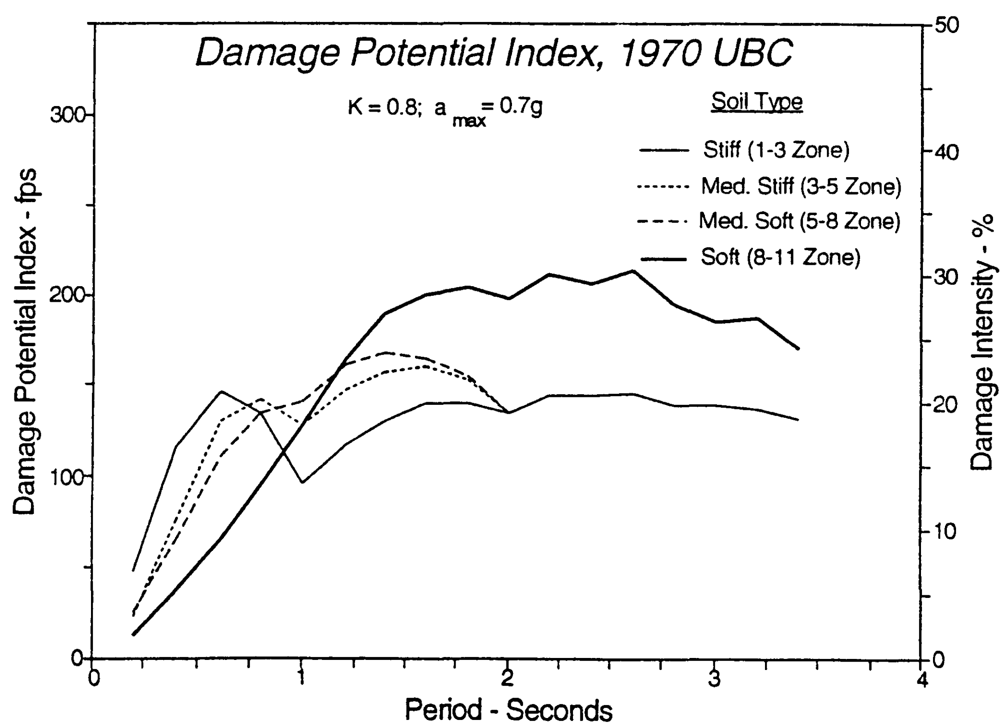

Figure 20. Comparison of DPI values for Salt Lake Valley soils at 250 year accelerations in accordance with 1970 UBC requirements.

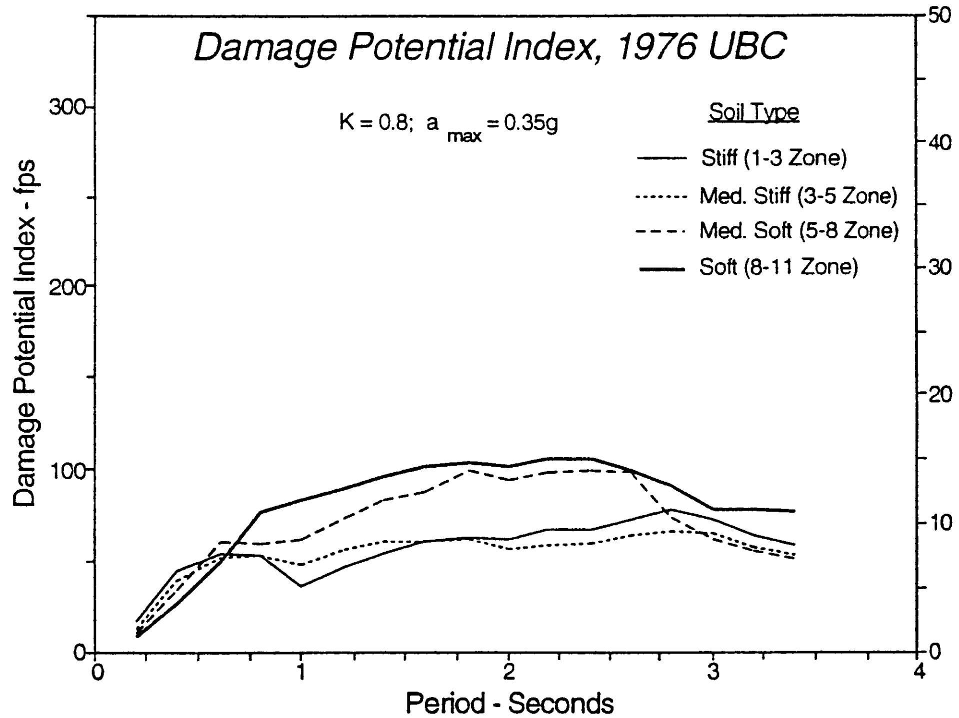

Figure 21. Comparison of DPI values for Salt Lake Valley soils at 50 year accelerations in accordance with 1976 UBC requirements.

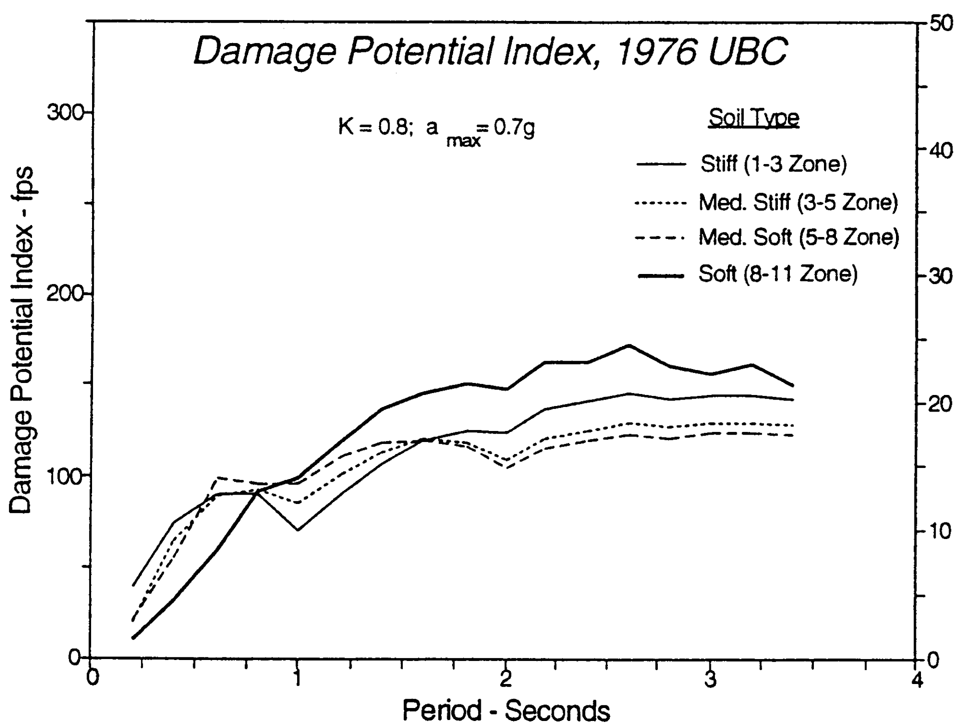

Figure 22. Comparison of DPI values for Salt Lake Valley soils at 250 year accelerations in accordance with 1976 UBC requirements.

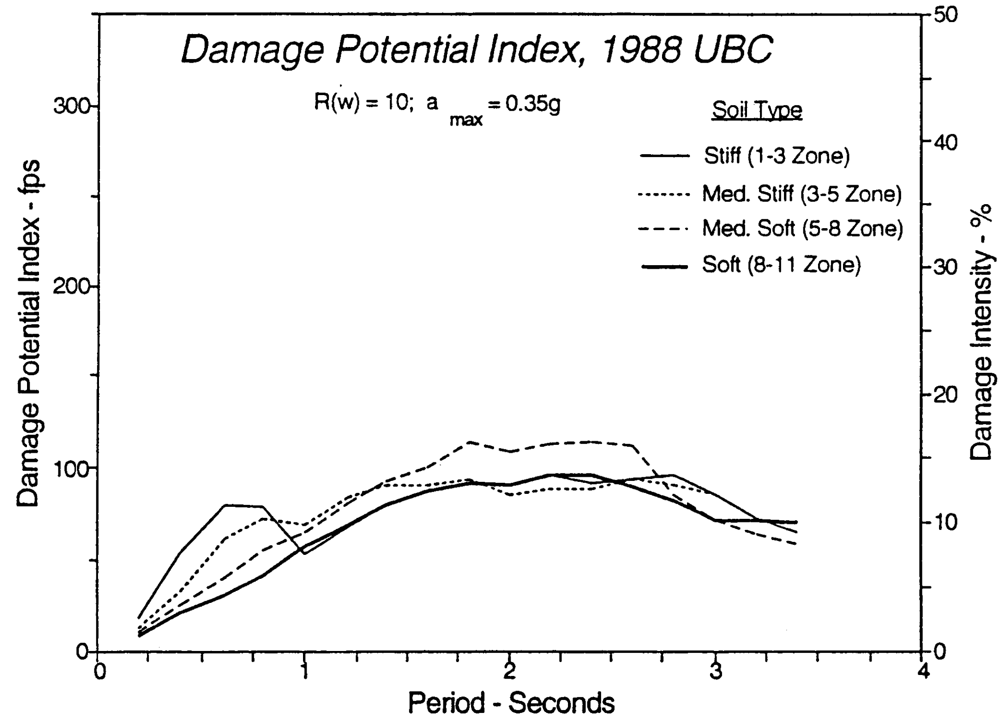

Figure 23. Comparison of DPI values for Salt Lake Valley soils at 50 year accelerations in accordance with 1988 UBC requirements.

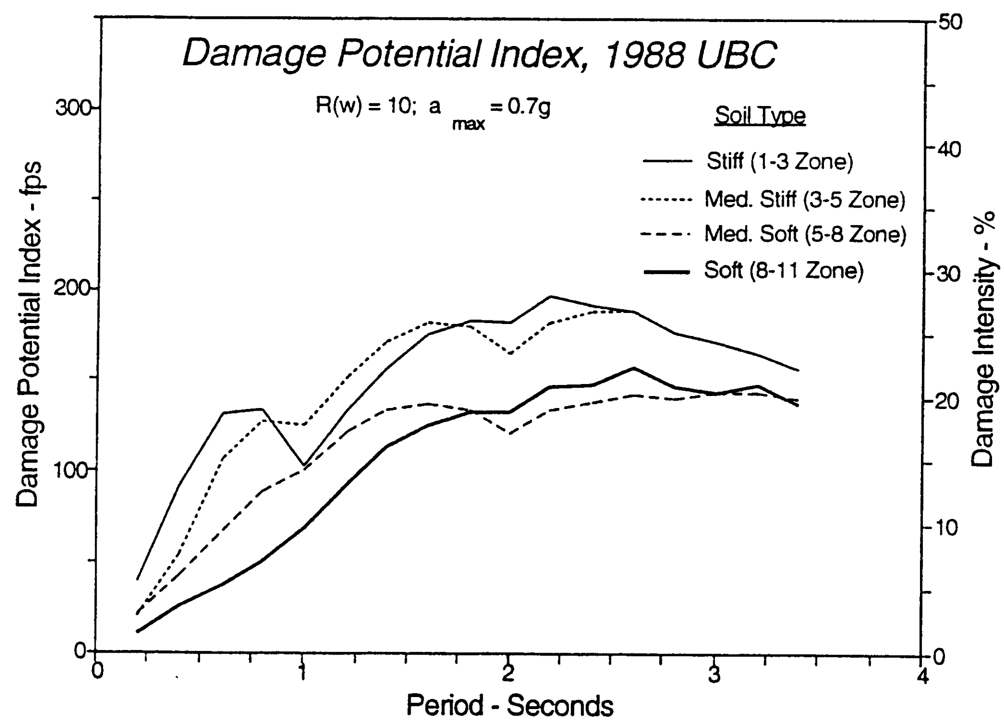

Figure 24. Comparison of DPI values for Salt Lake Valley soils at 250 year accelerations in accordance with 1988 UBC requirements.

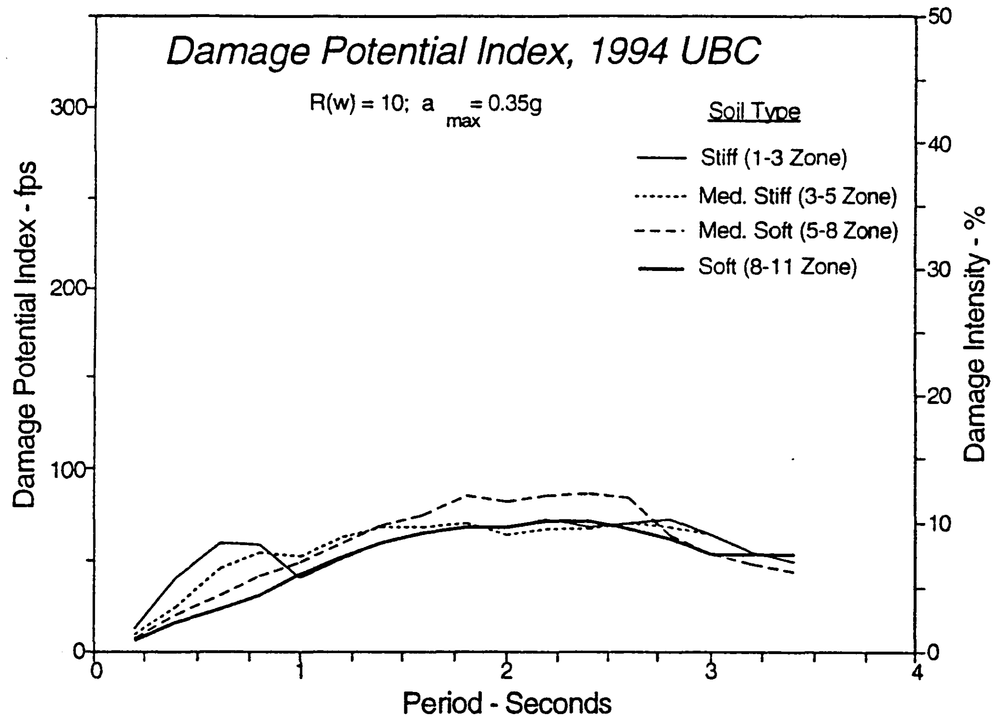

Figure 25. Comparison of DPI values for Salt Lake Valley soils at 50 year accelerations in accordance with proposal for 1994 UBC requirements.

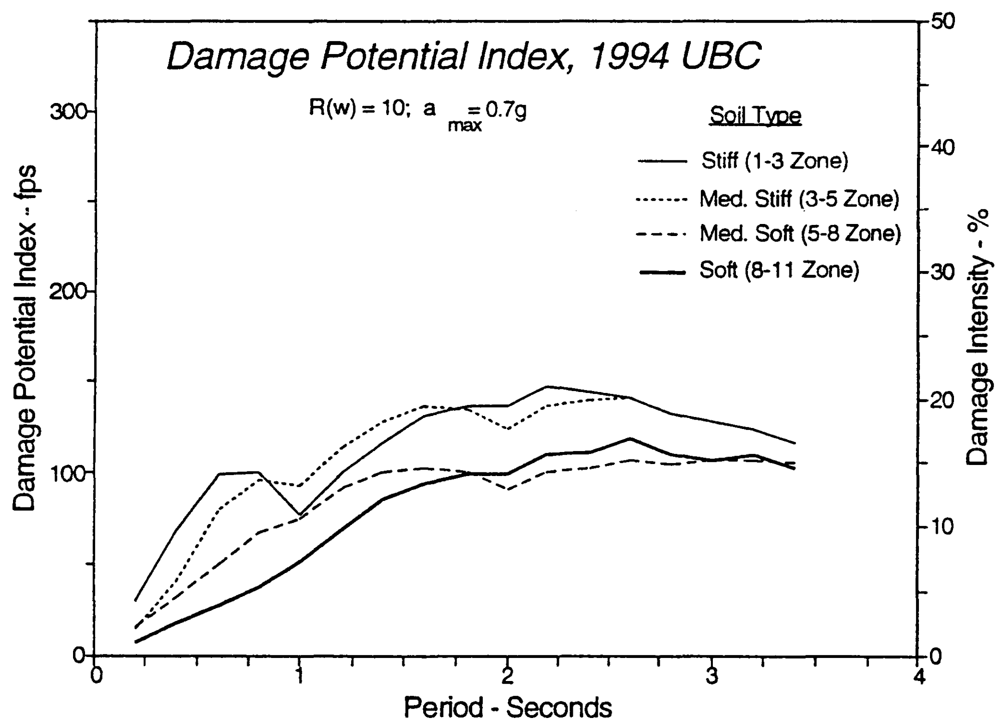

Figure 26. Comparison of DPI values for Salt Lake Valley soils at 250 year accelerations in accordance with proposal for 1994 UBC requirements.

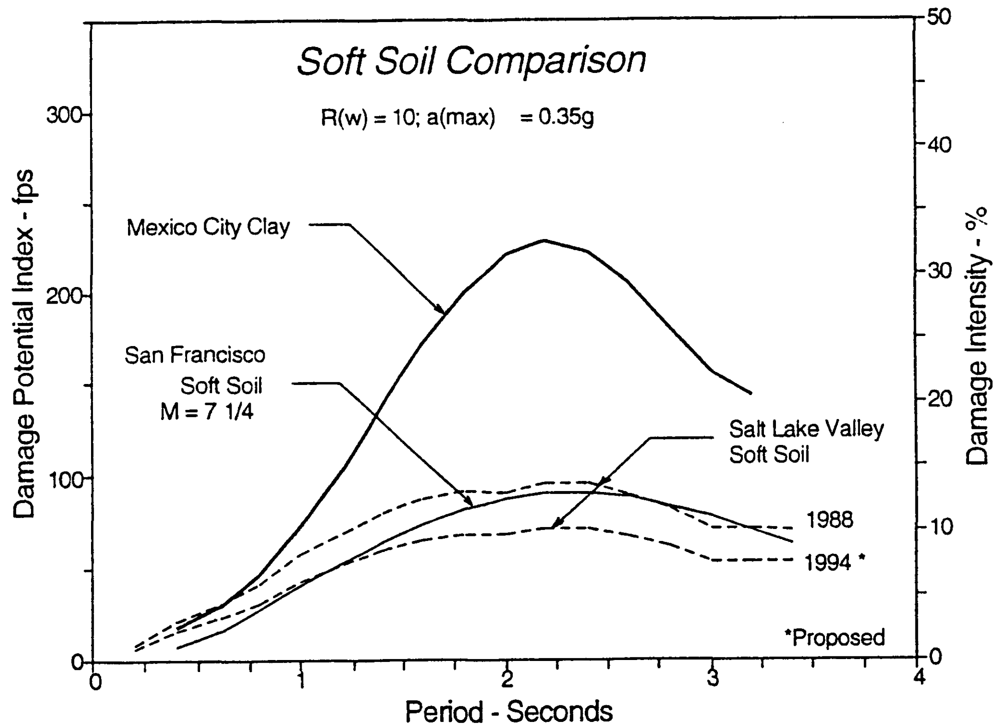

Figure 27. Comparison of computed DPI values for soft soil sites in Salt Lake at 50 year acceleration levels with response of soft soils in San Francisco and Mexico City.

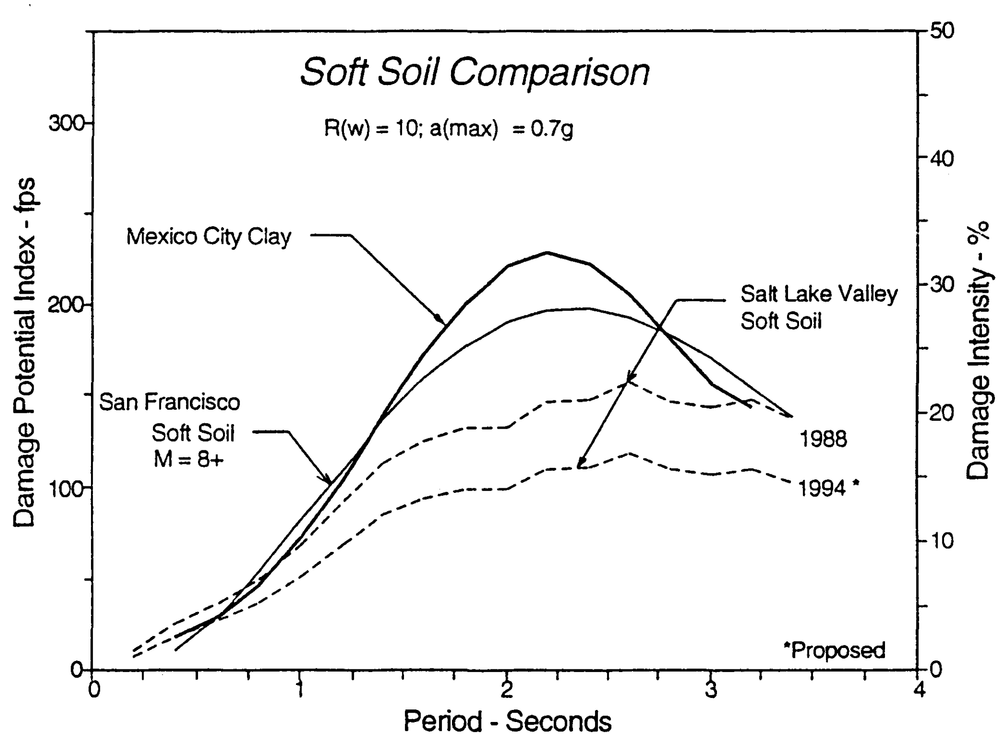

Figure 28. Comparison of computed DPI values for soft soil sites in Salt Lake at 250 year acceleration levels with response of soft soils in San Francisco and Mexico City.

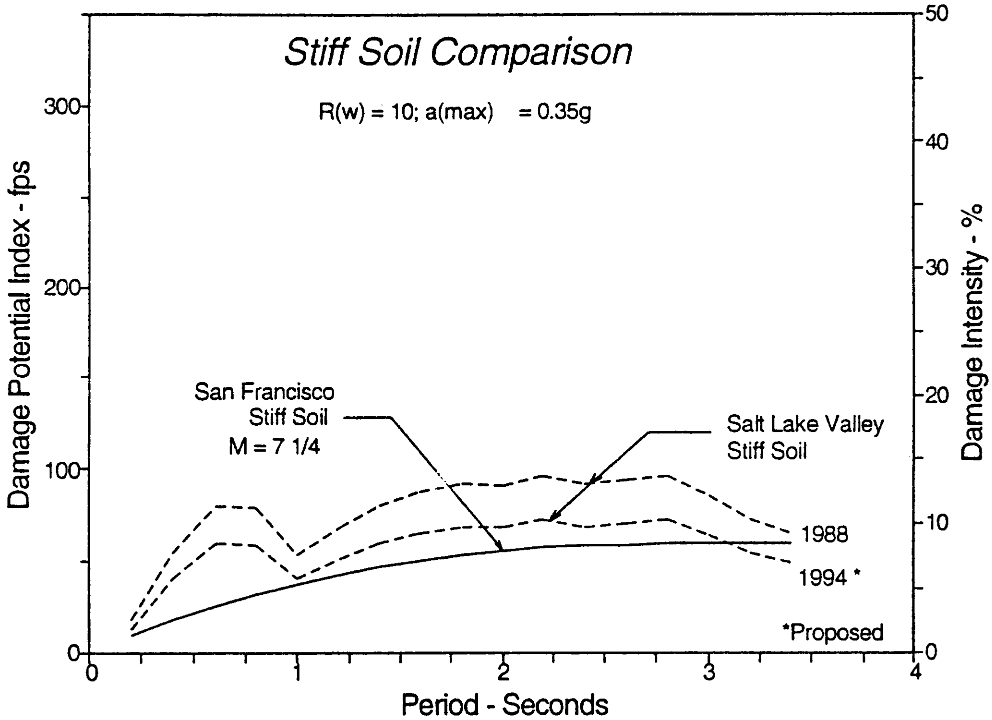

Figure 29. Comparison of computed DPI values for stiff soil sites in Salt Lake at 50 year acceleration levels with response of stiff soils in San Francisco and Mexico City.

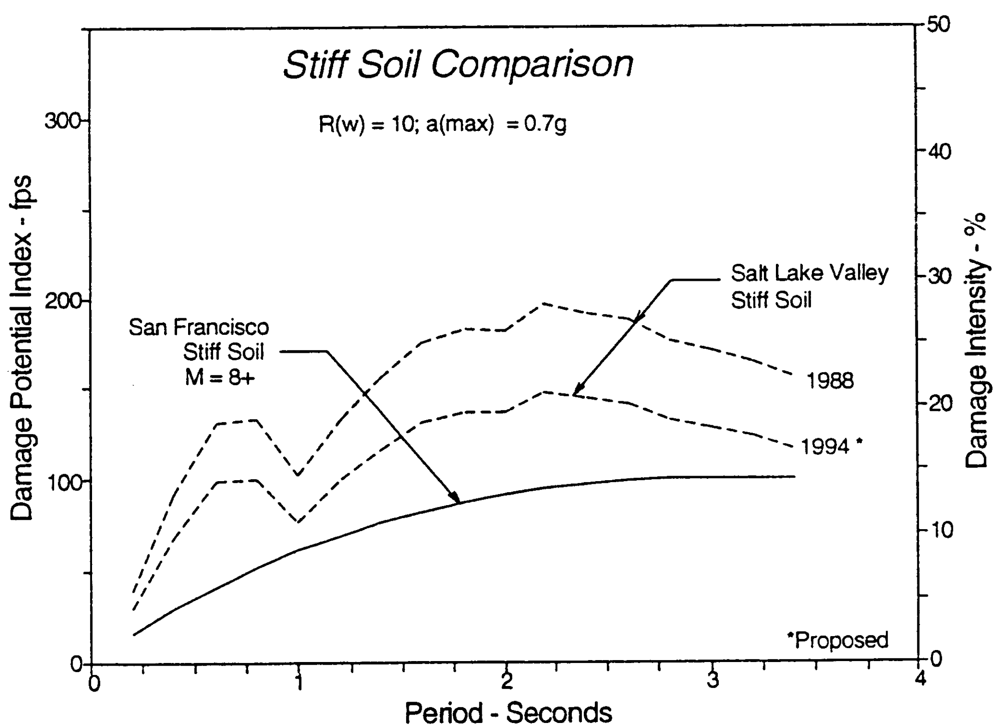

Figure 30. Comparison of computed DPI values for stiff soil sites in Salt Lake at 250 year acceleration levels with response of stiff soils in San Francisco and Mexico City.

APPENDIX A

SOIL AND SHEAR WAVE VELOCITY PROFILES AT SELECTED USGS RECORDING SITES

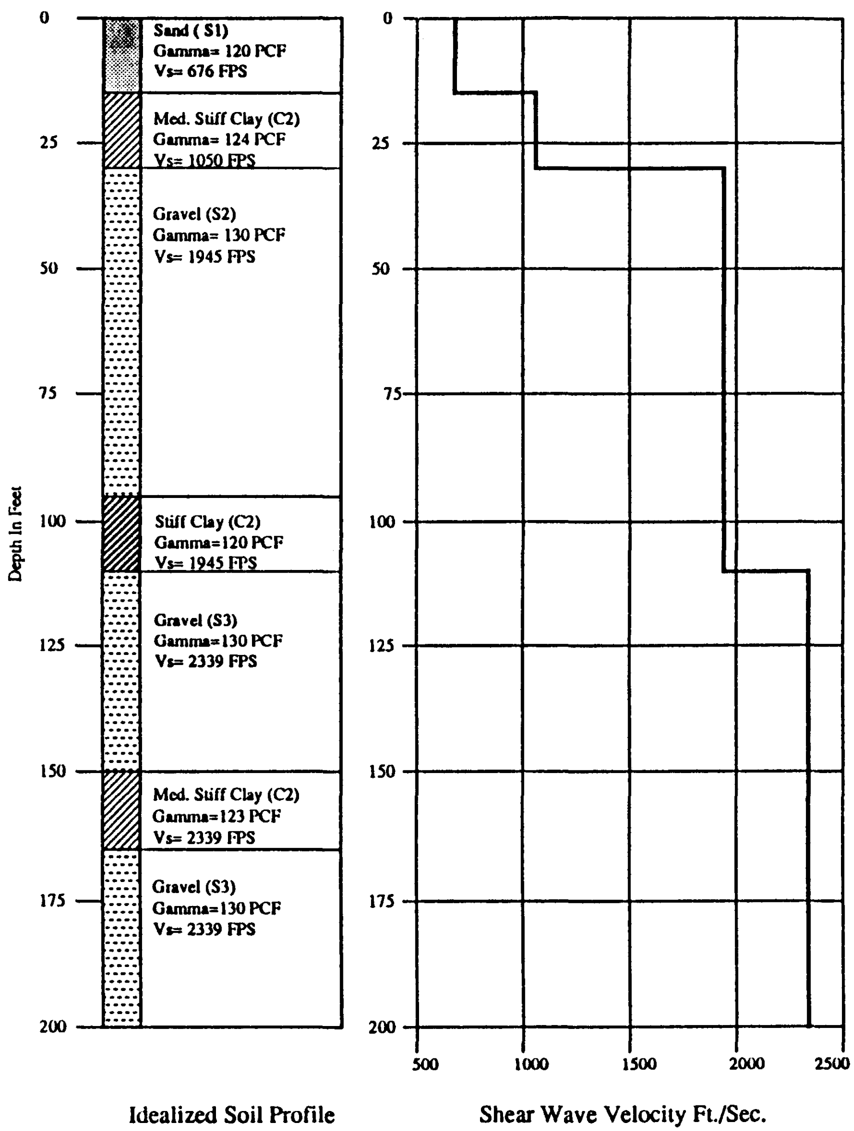

Figure A.1. Soil and shear wave velocity profiles at the Utah Geologic Survey site (UGS006), 1-3 Zone (Tinsley et al 1991).

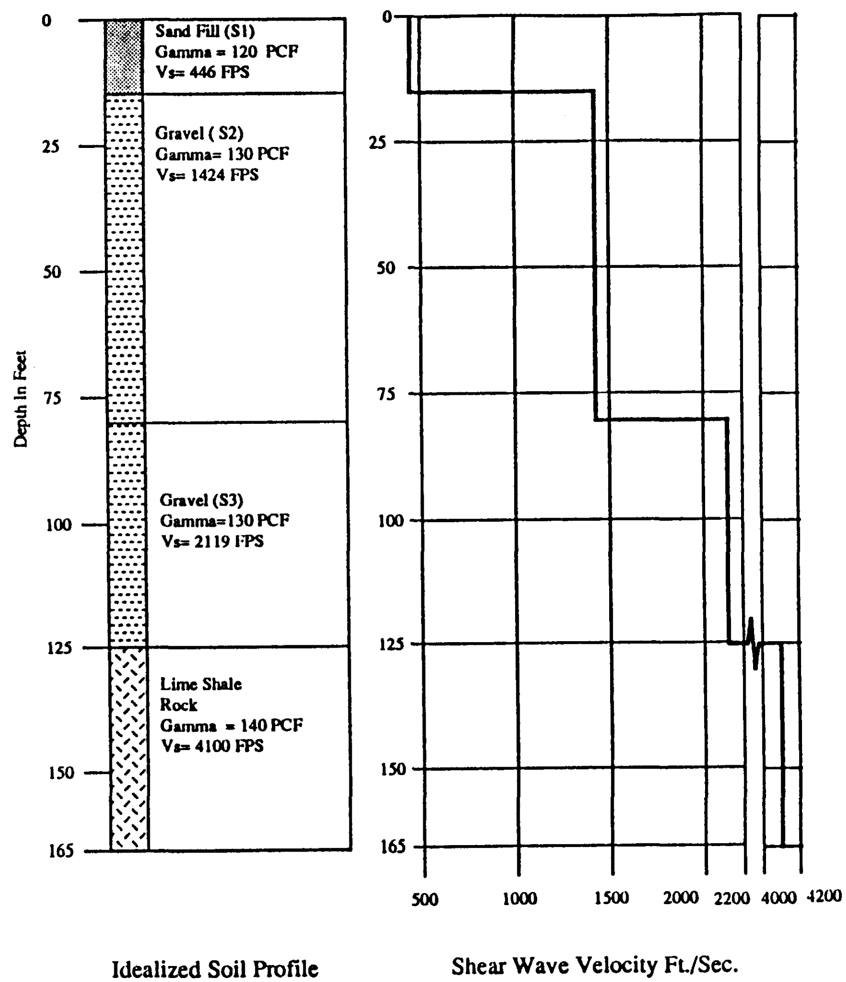

Figure A.2. Soil and shear wave velocity profiles at the Sunnyside Trng. Center site (SUN042), 1-3 Zone (Tinsley et al 1991).

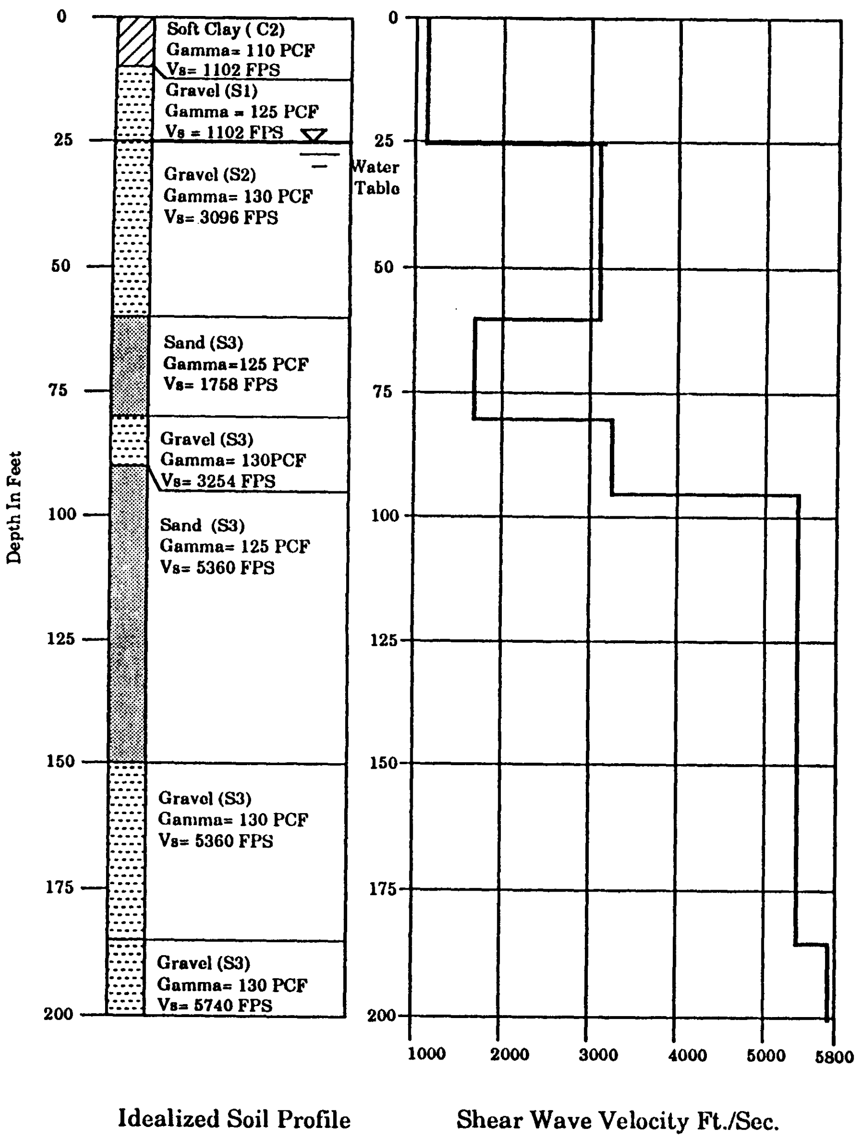

Figure A.3. Soil and shear wave velocity profiles at the Laird Park site (LAI044), 1-3 Zone (Tinsley et al 1991).

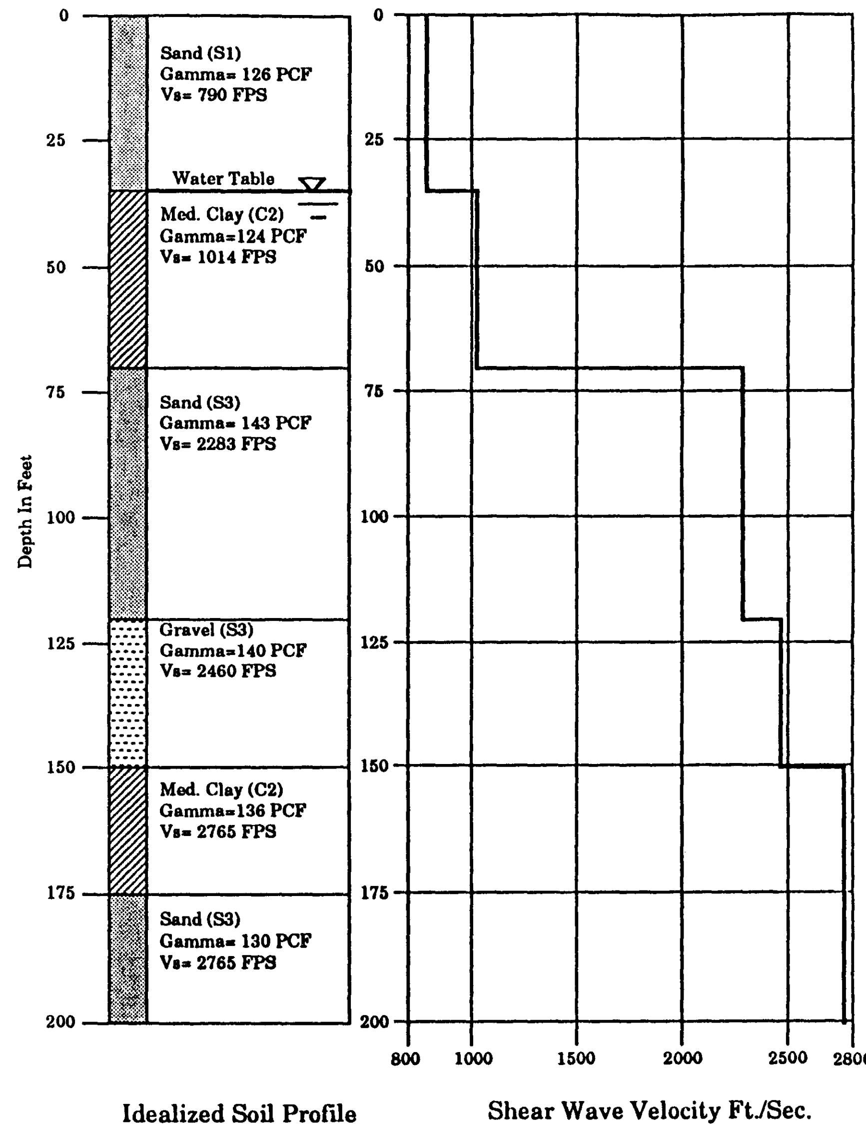

Figure A.4. Soil and shear wave velocity profiles at the Bonneville Golf Course site (BON043), 3-5 Zone (Tinsley et al 1991).

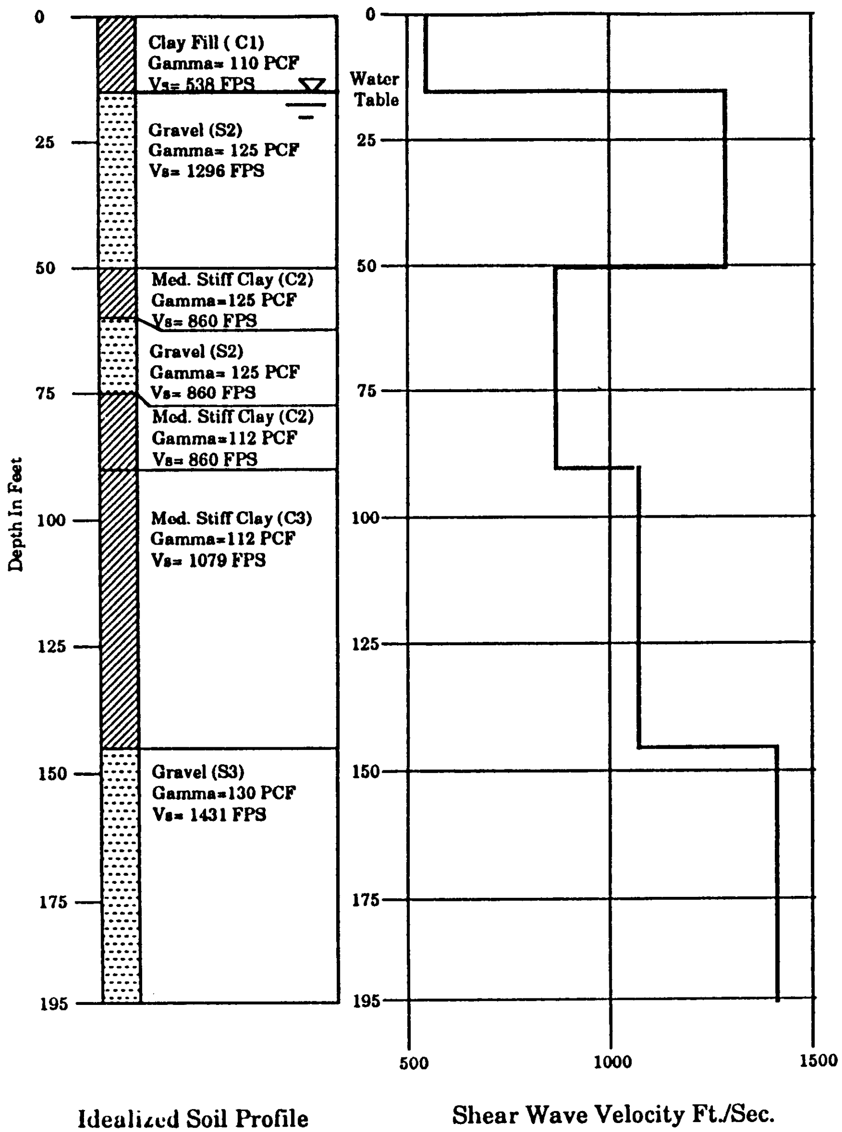

Figure A.5. Soil and shear wave velocity profiles at the Roosevelt School (ROO046), 3-5 Zone (Tinsley et al 1991).

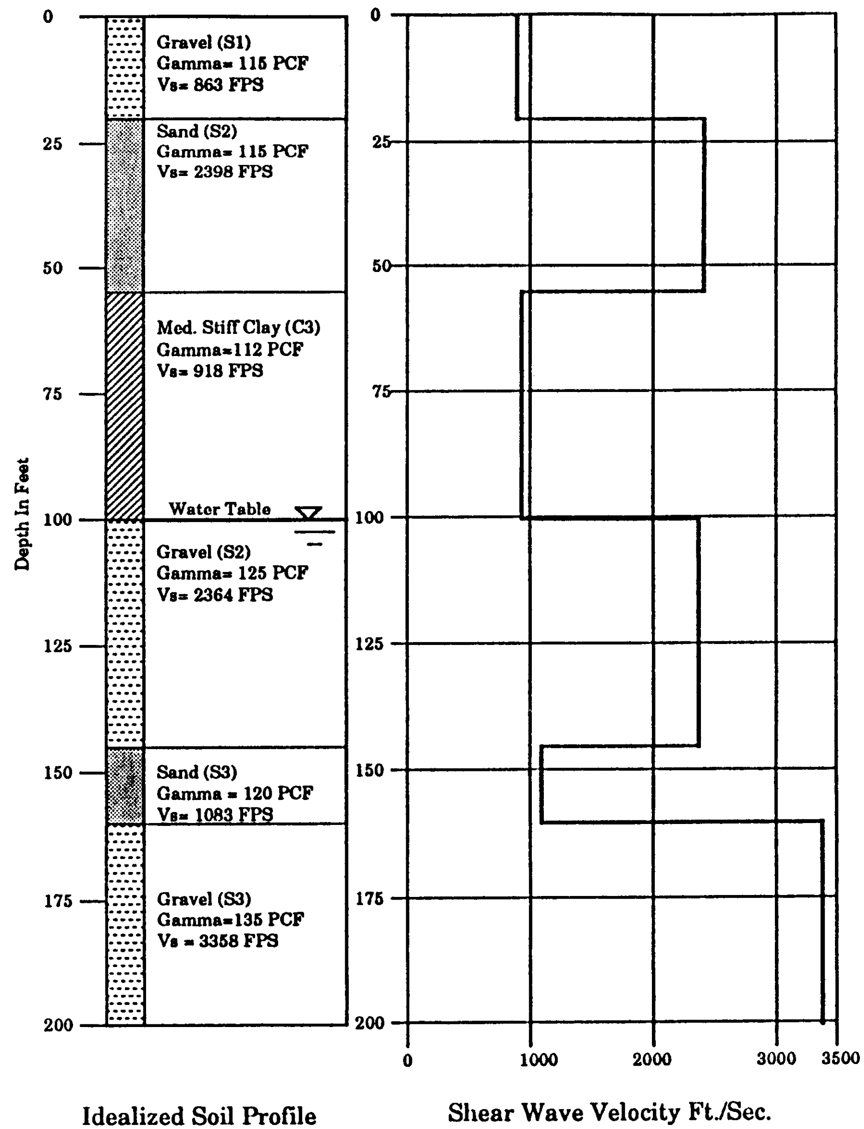

Figure A.6. Soil and shear wave velocity profiles at the Temple Square site (TMP052), 3-5 Zone (Tinsley et al 1991).

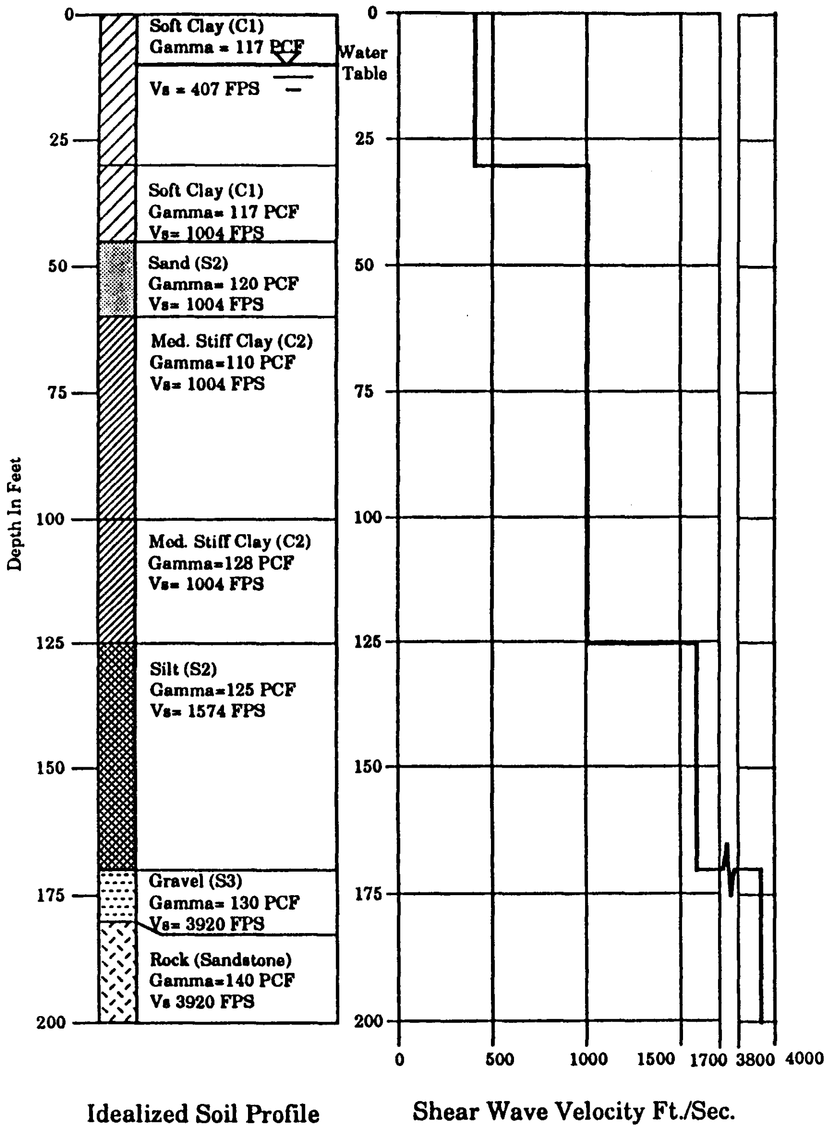

Figure A.7. Soil and shear wave velocity profiles at the Magna Water District (MAG051), 5-8 Zone (Tinsley et al 1991).

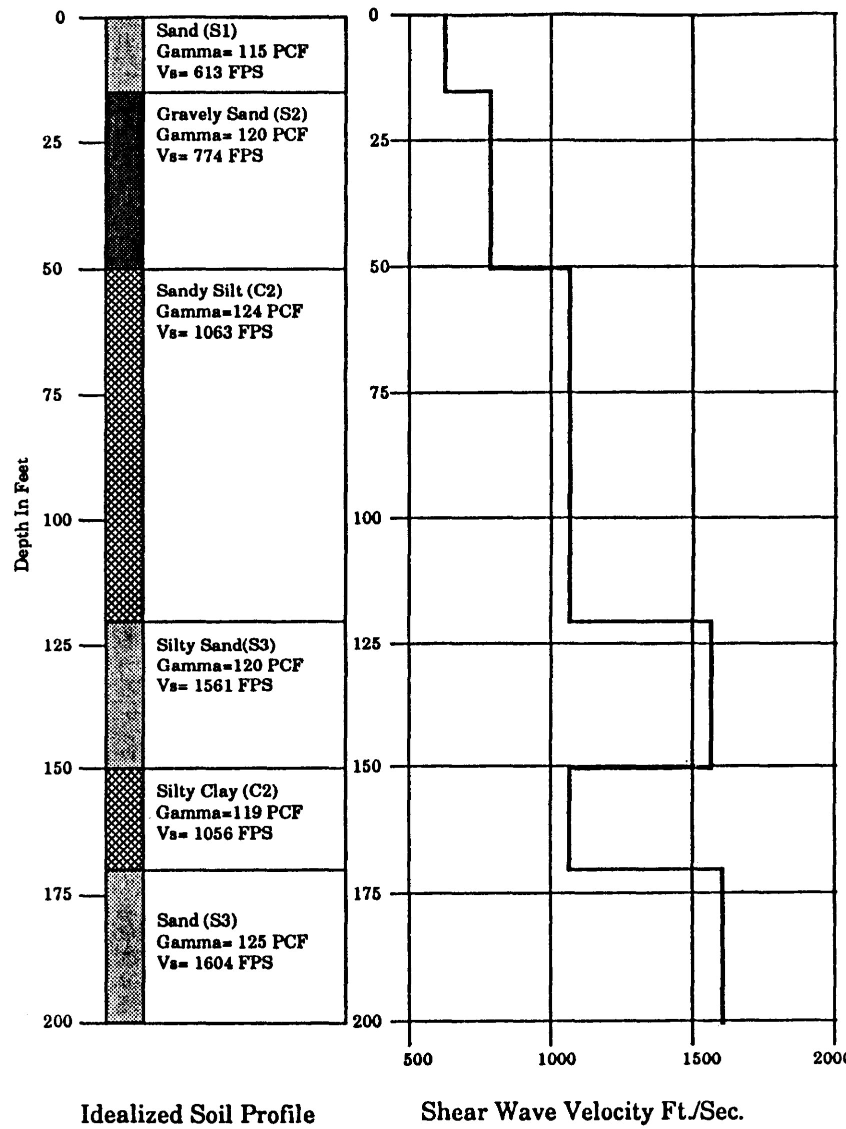

Figure A.8. Soil and shear wave velocity profiles at the Alta View Hospital Site, 5-8 Zone (Tinsley et al 1991).

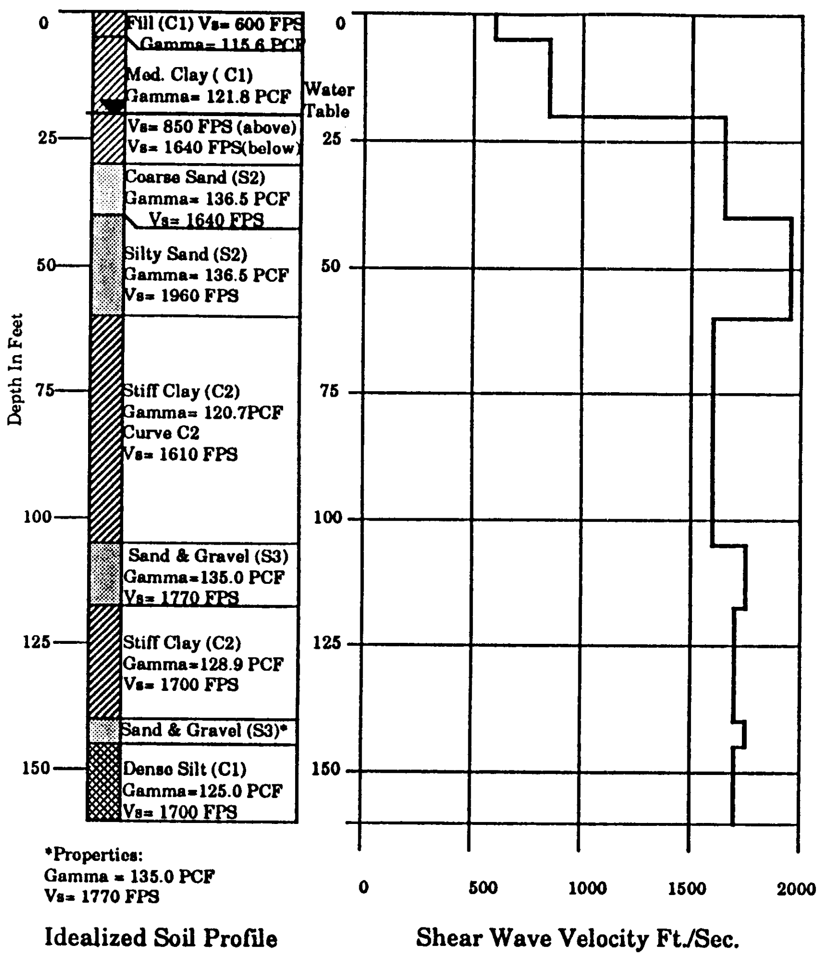

Figure A.9. Soil and shear wave velocity profiles at the City and County Building site (CCB056), 5-8 Zone (Tinsley et al 1991).

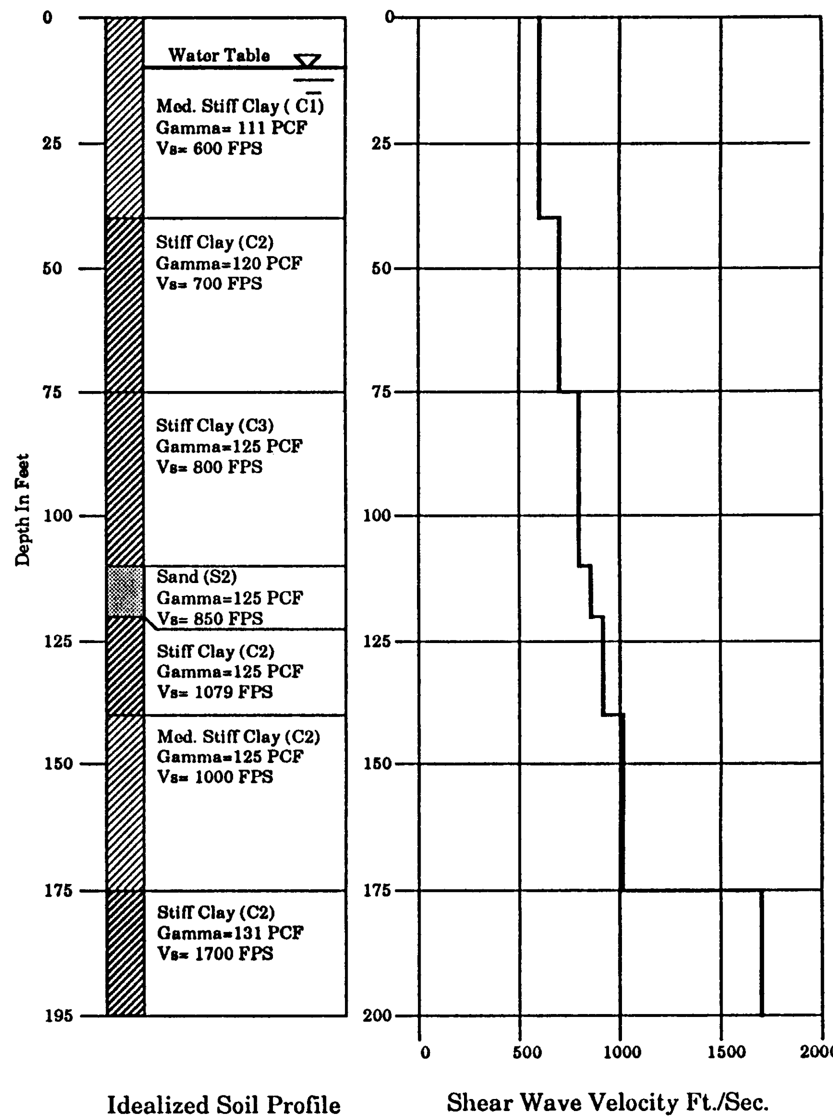

Figure A.10. Soil and shear wave velocity profiles at the East Airport site (EAP017), 8-11 Zone (Tinsley et al 1991).

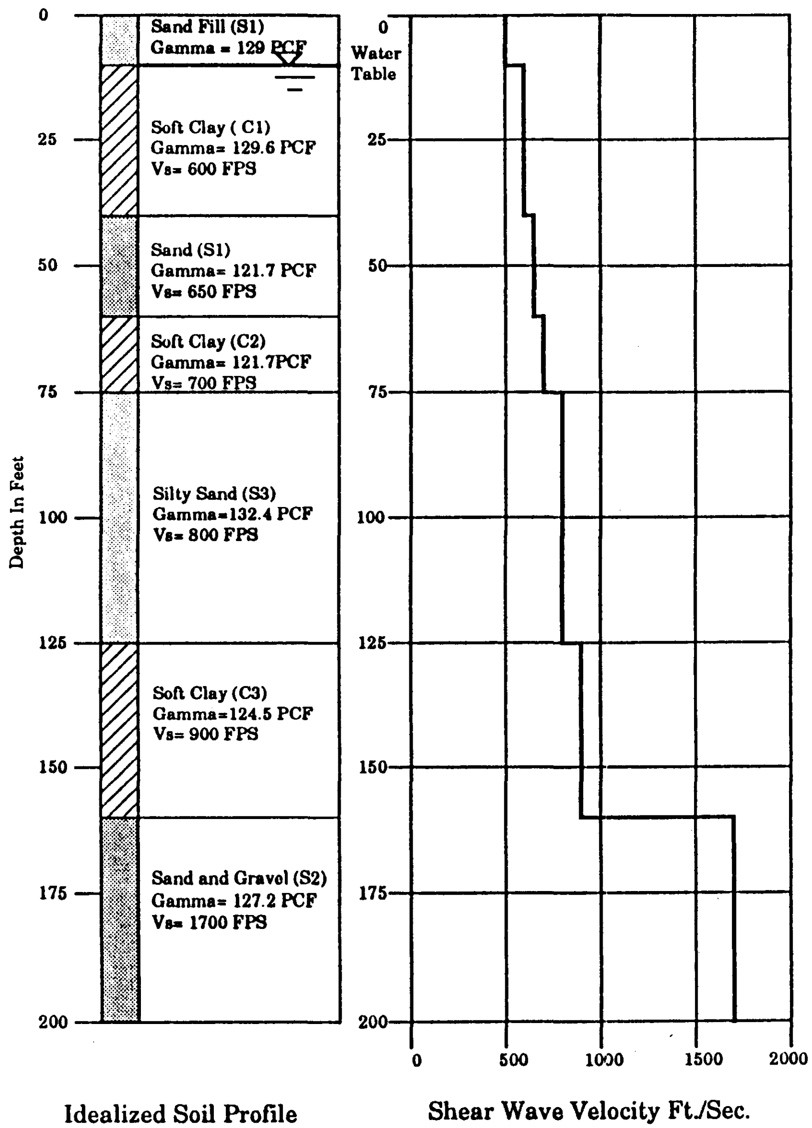

Figure A.11. Soil and shear wave velocity profiles at the West Airport site (WAP019), 8-11 Zone (Tinsley et al 1991).

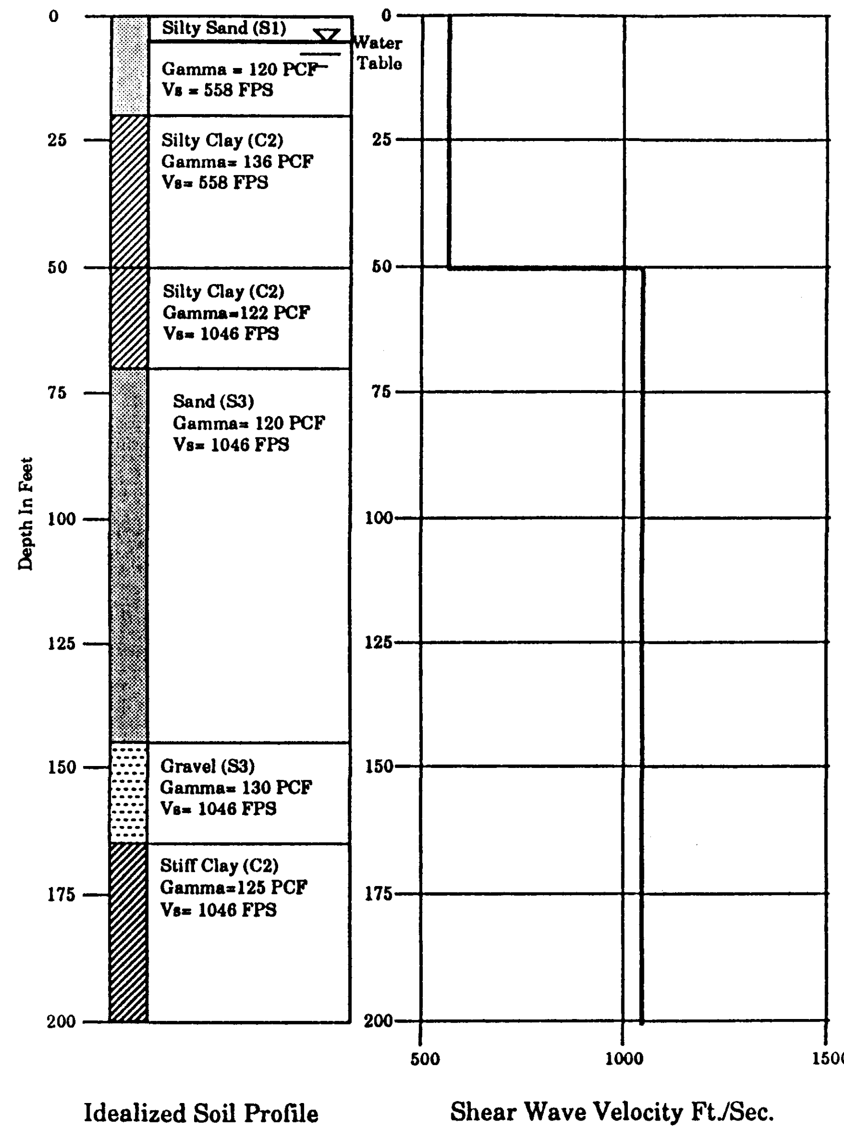

Figure A.12. Soil and shear wave velocity profiles at the Jordan River Station (RIV029), 8-11 Zone (Tinsley et al 1991).

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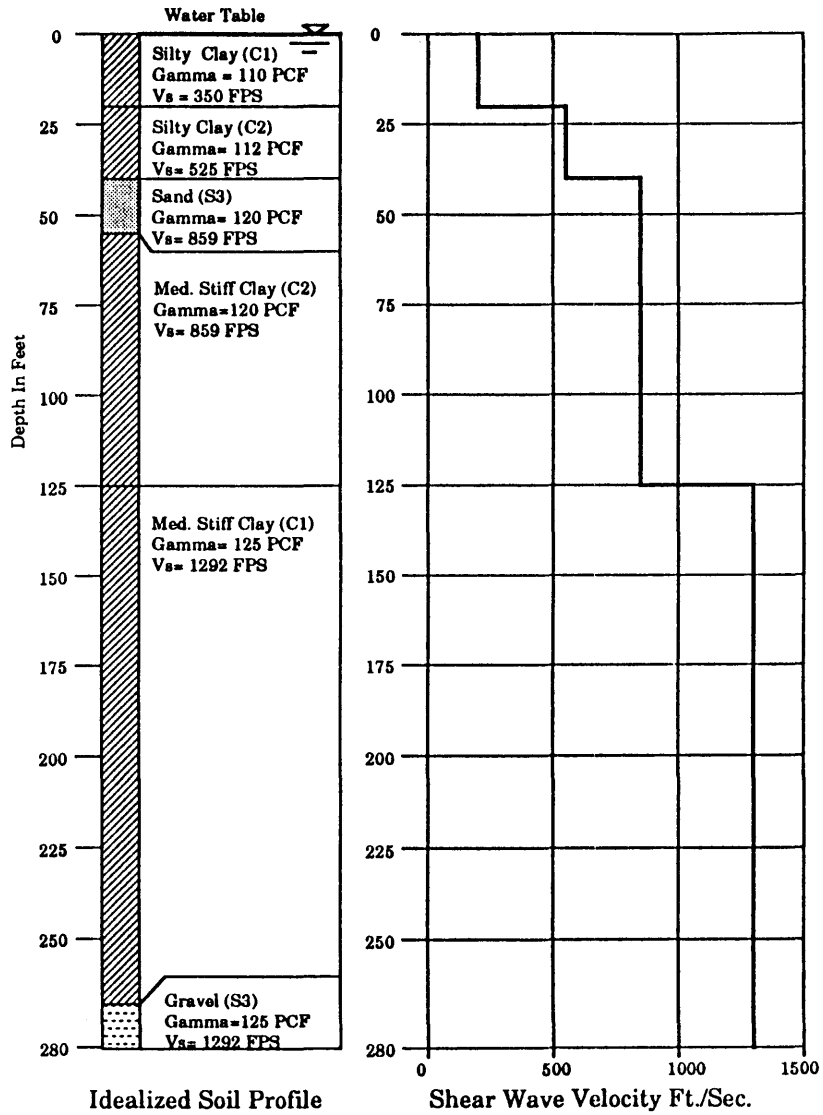

Figure A.13. Soil and shear wave velocity profiles at the Blackhawk Duck Club (DUC053), 8-11 Zone (Tinsley et al 1991).

APPENDIX B

ACCELERATION RESPONSE SPECTRA AT EACH USGS RECORDING SITE DUE TO ROCK INPUT RECORDS REPRESENTING 50 AND 250 YR ACCELERATION LEVELS

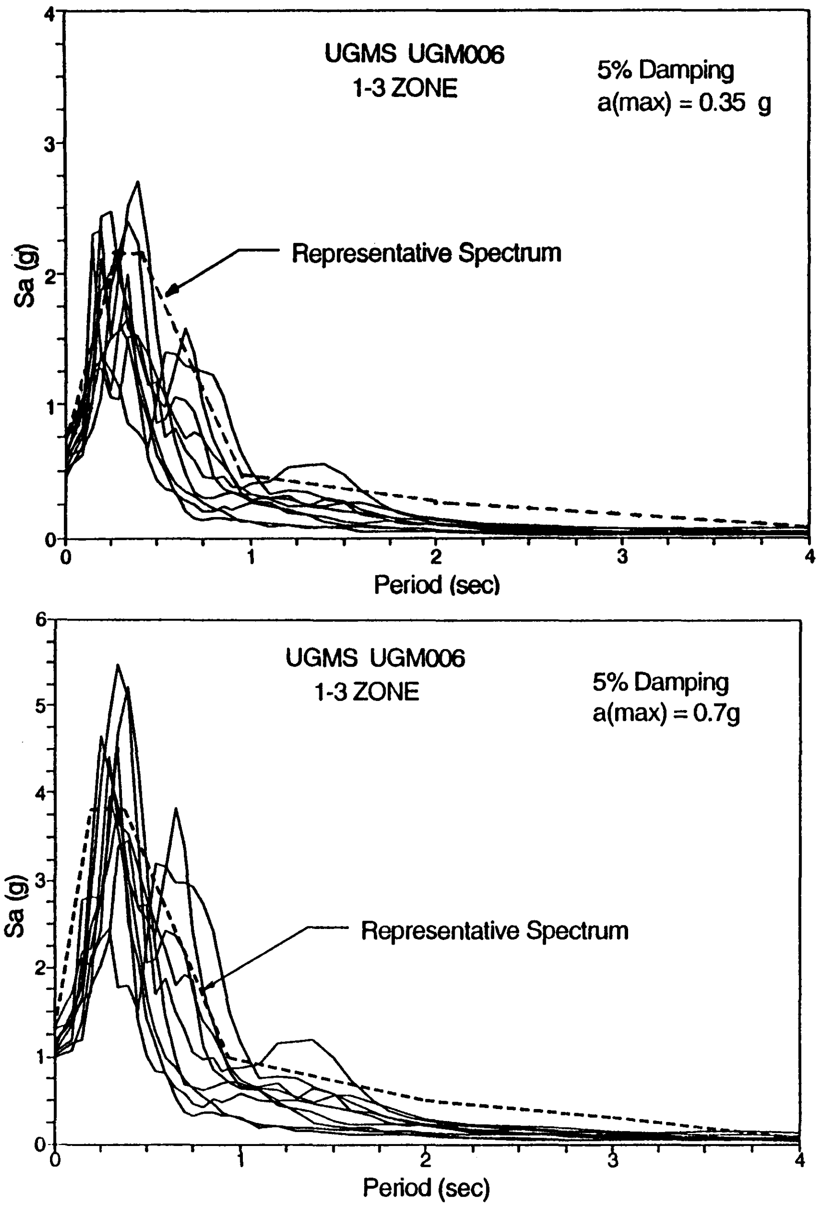

Figure B.1. Computed acceleration response spectra for Utah Geological Survey (UGM006) at 50 (top) and 250 (bottom) year rock acceleration levels.

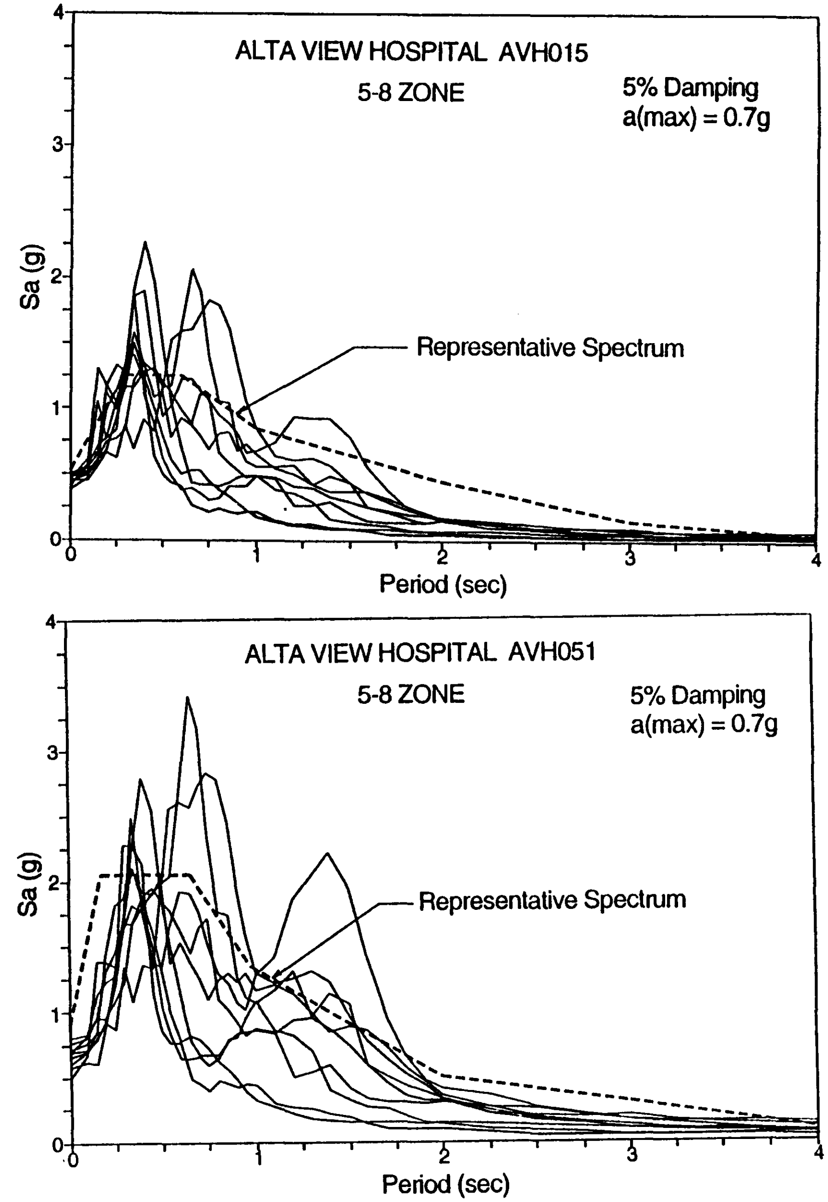

Figure B.2. Computed acceleration response spectra for Alta View Hospital (AVH015) at 50 (top) and 250 (bottom) year rock acceleration levels.

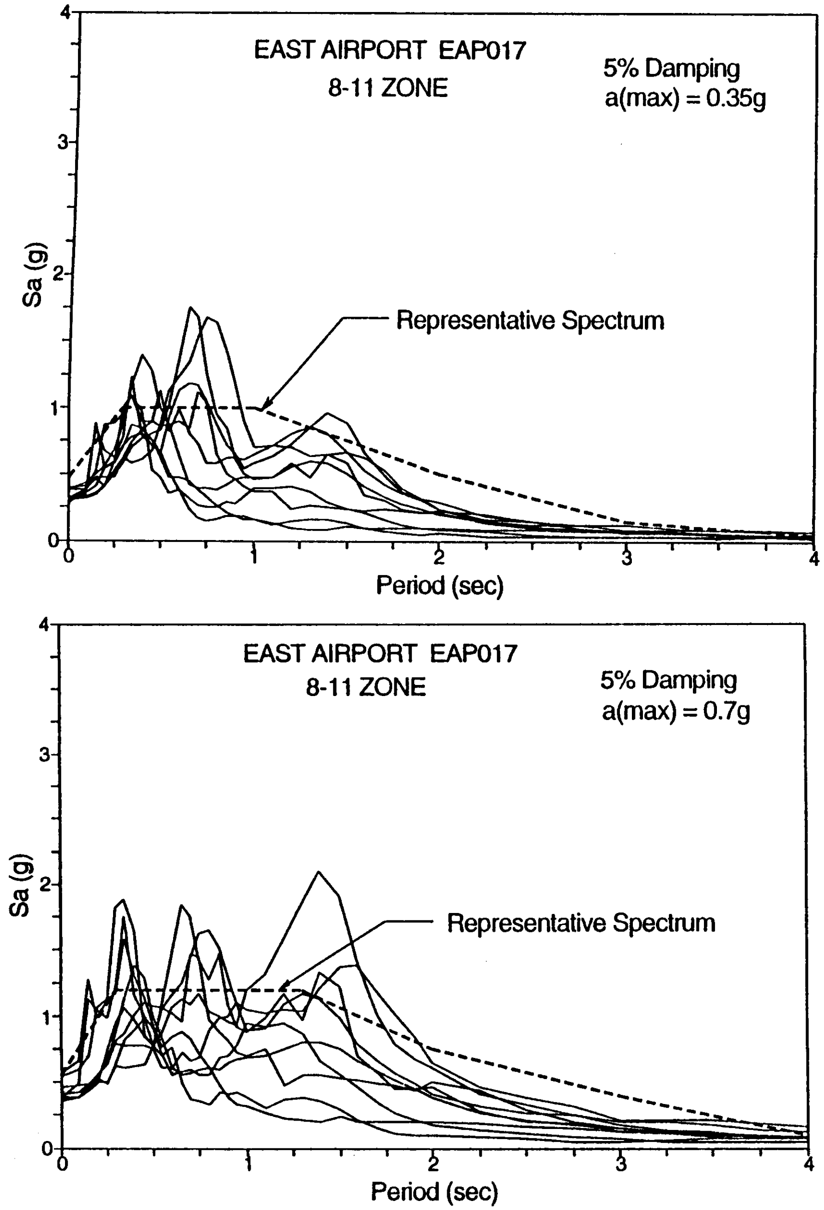

Figure B.3. Computed acceleration response spectra for Salt Lake East Airport (EAP017) at 50 (top) and 250 (bottom) year rock acceleration levels.

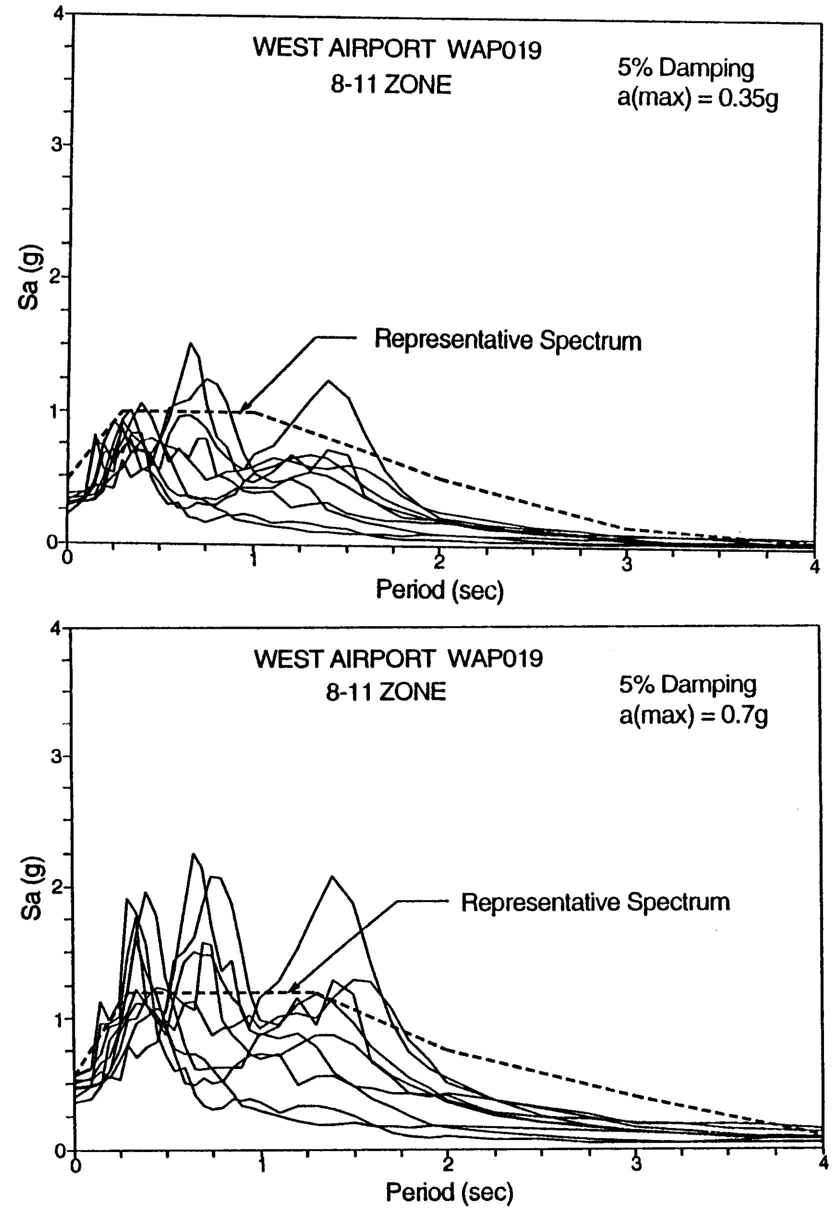

Figure B.4. Computed acceleration response spectra for Salt Lake West Airport (WAP019) at 50 (top) and 250 (bottom) year rock acceleration levels.

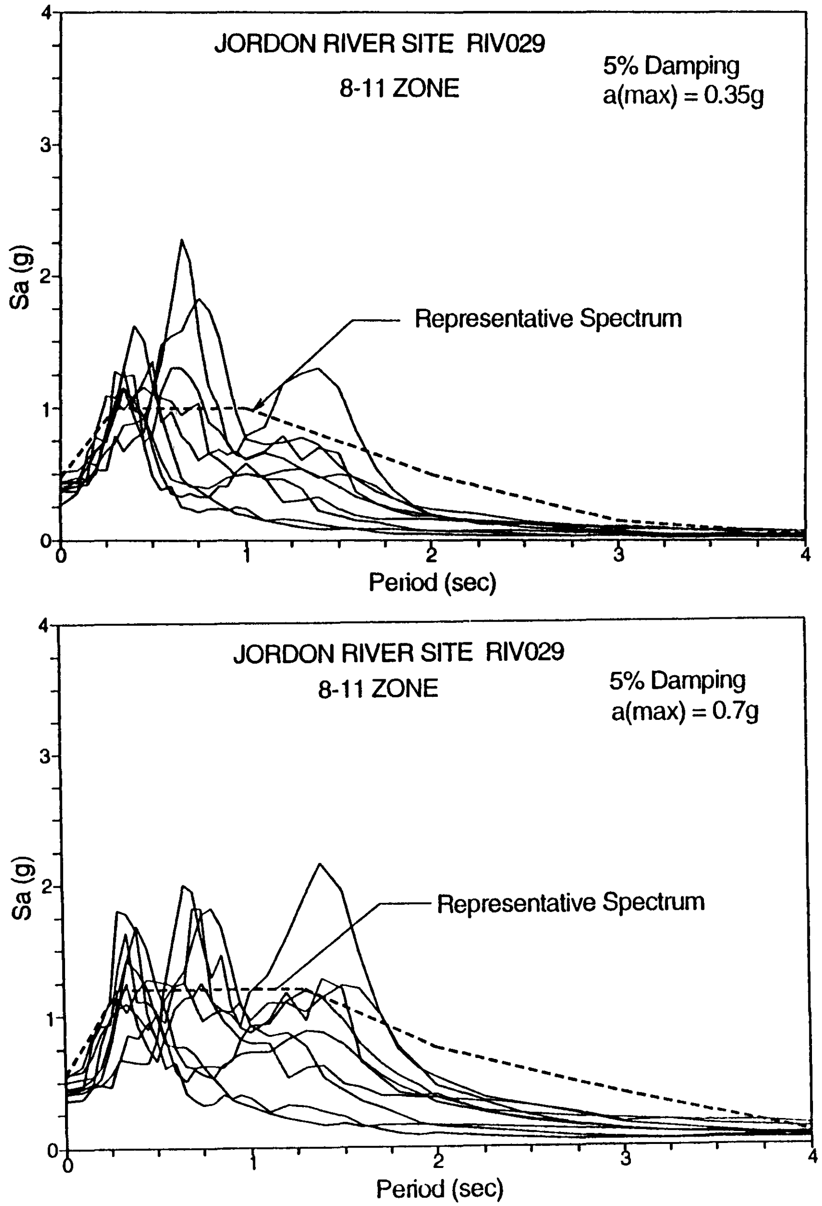

Figure B.5. Computed acceleration response spectra for Jordan River Site (RIV029) at 50 (top) and 250 (bottom) year rock acceleration levels.

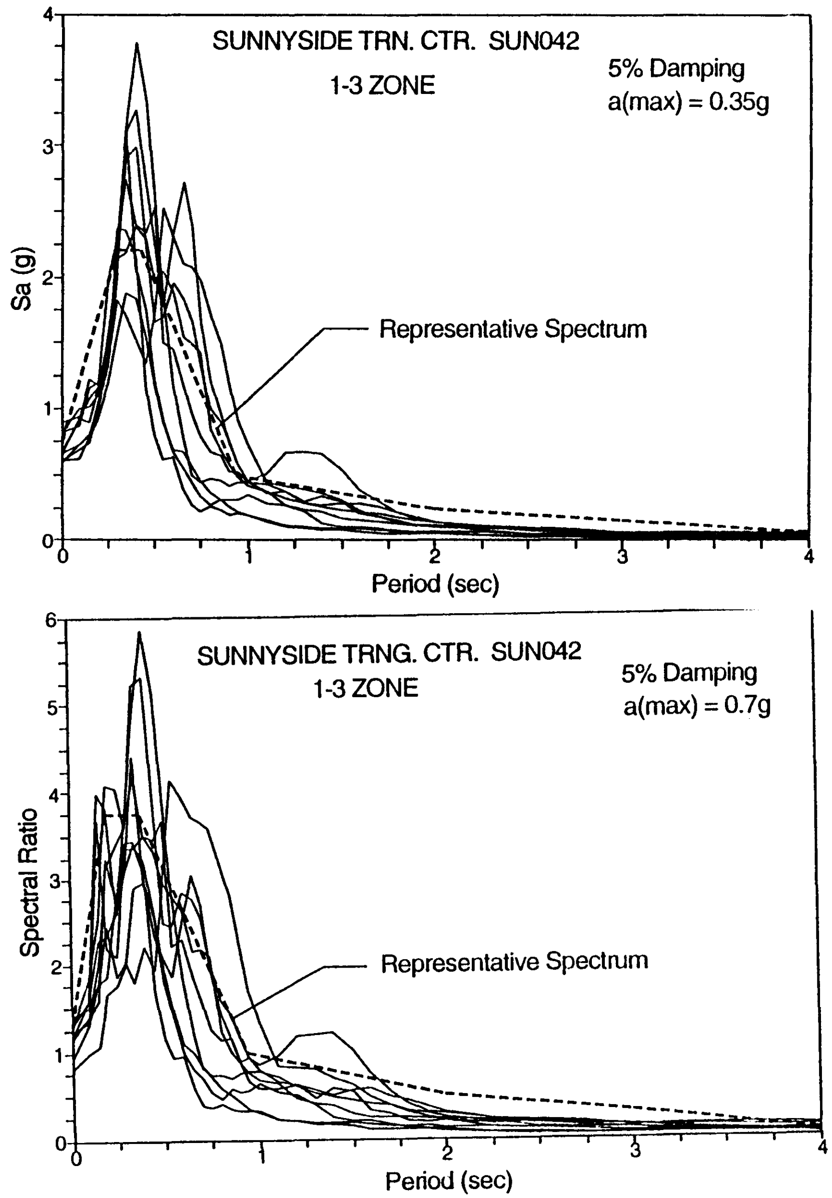

Figure B.6. Computed acceleration response spectra for Sunnyside Trg. Ctr. (SUN042) at 50 (top) and 250 (bottom) year rock acceleration levels.

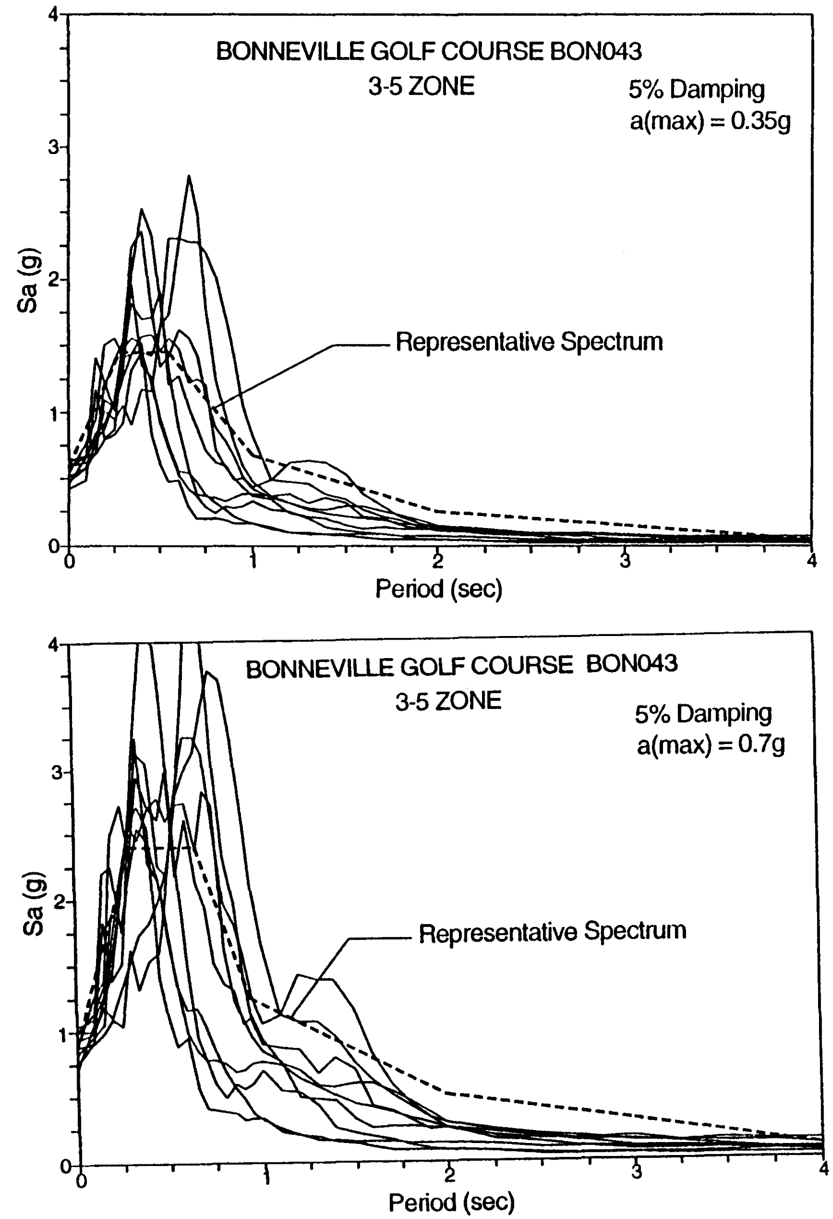

Figure B.7. Computed acceleration response spectra for Bonneville Golf Course (BON043) at 50 (top) and 250 (bottom) year rock acceleration levels.

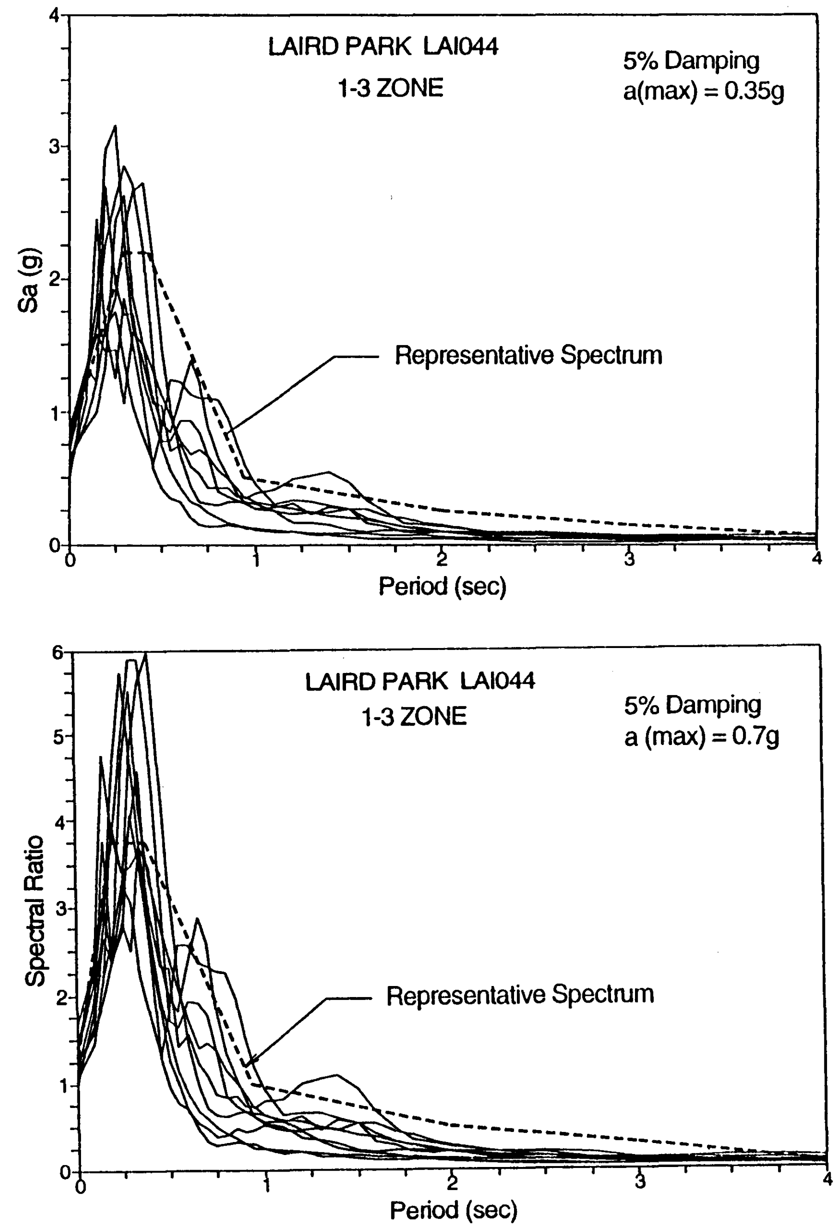

Figure B.8. Computed acceleration response spectra for Laird Park (LAI044) at 50 (top) and 250 (bottom) year rock acceleration levels.

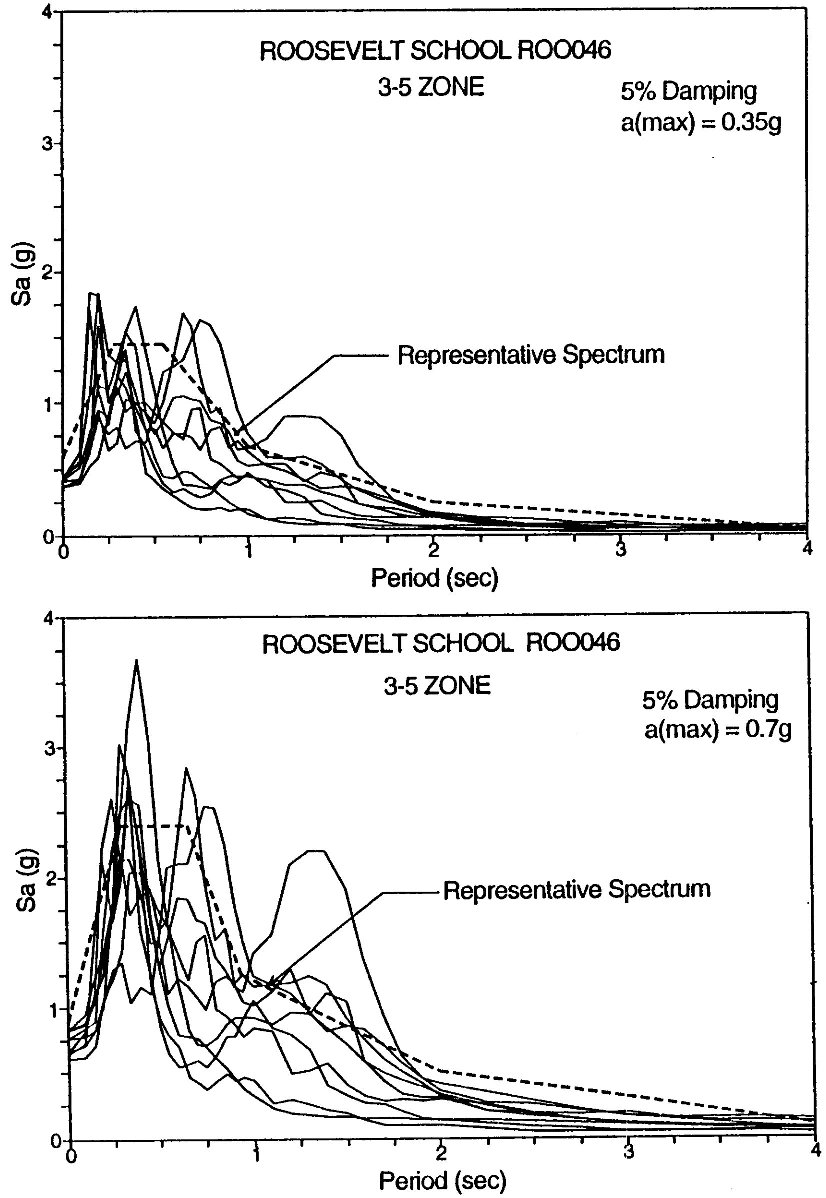

Figure B.9. Computed acceleration response spectra for Roosevelt School (ROO046) at 50 (top) and 250 (bottom) year rock acceleration levels.

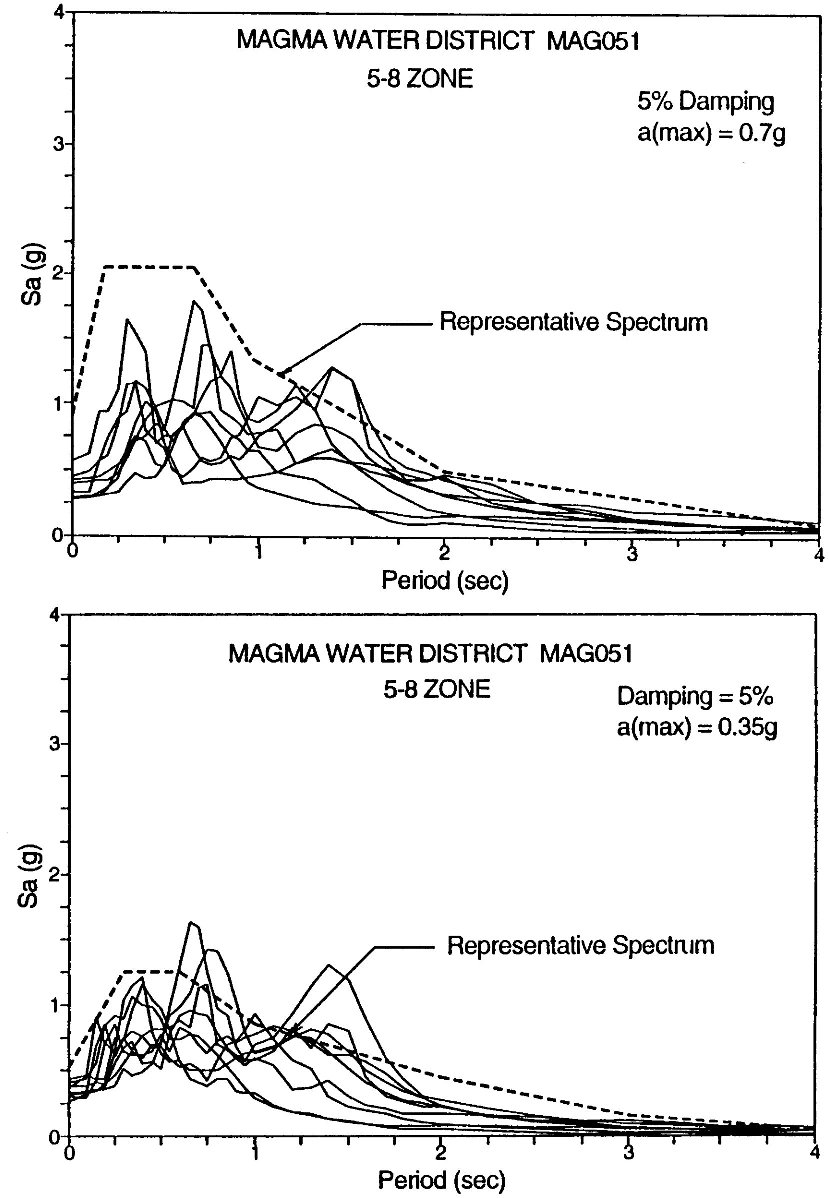

Figure B.10. Computed acceleration response spectra for Magna Water District (MAG051) at 50 (top) and 250 (bottom) year rock acceleration levels.

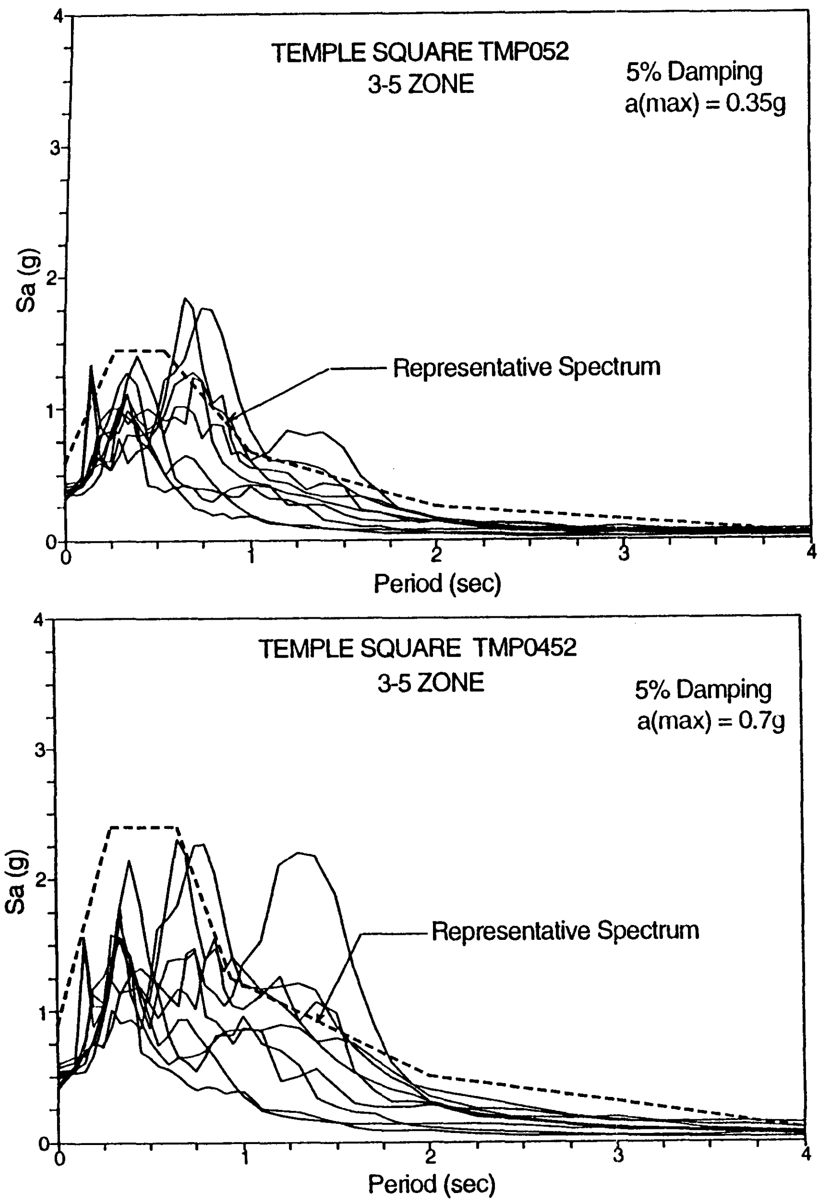

Figure B.11. Computed acceleration response spectra for Temple Square (TMP052) at 50 (top) and 250 (bottom) year rock acceleration levels.

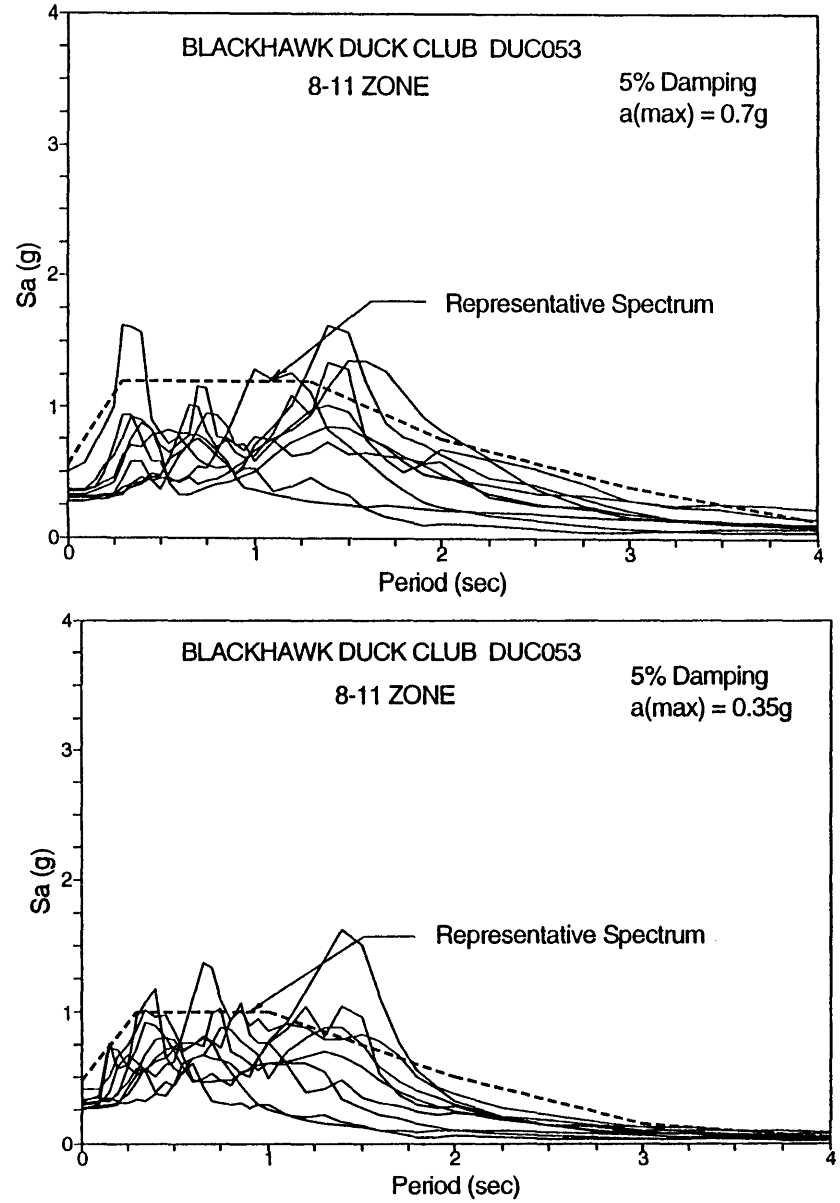

Figure B.12. Computed acceleration response spectra for Blackhawk Duck Club (DUC053) at 50 (top) and 250 (bottom) year rock acceleration levels.

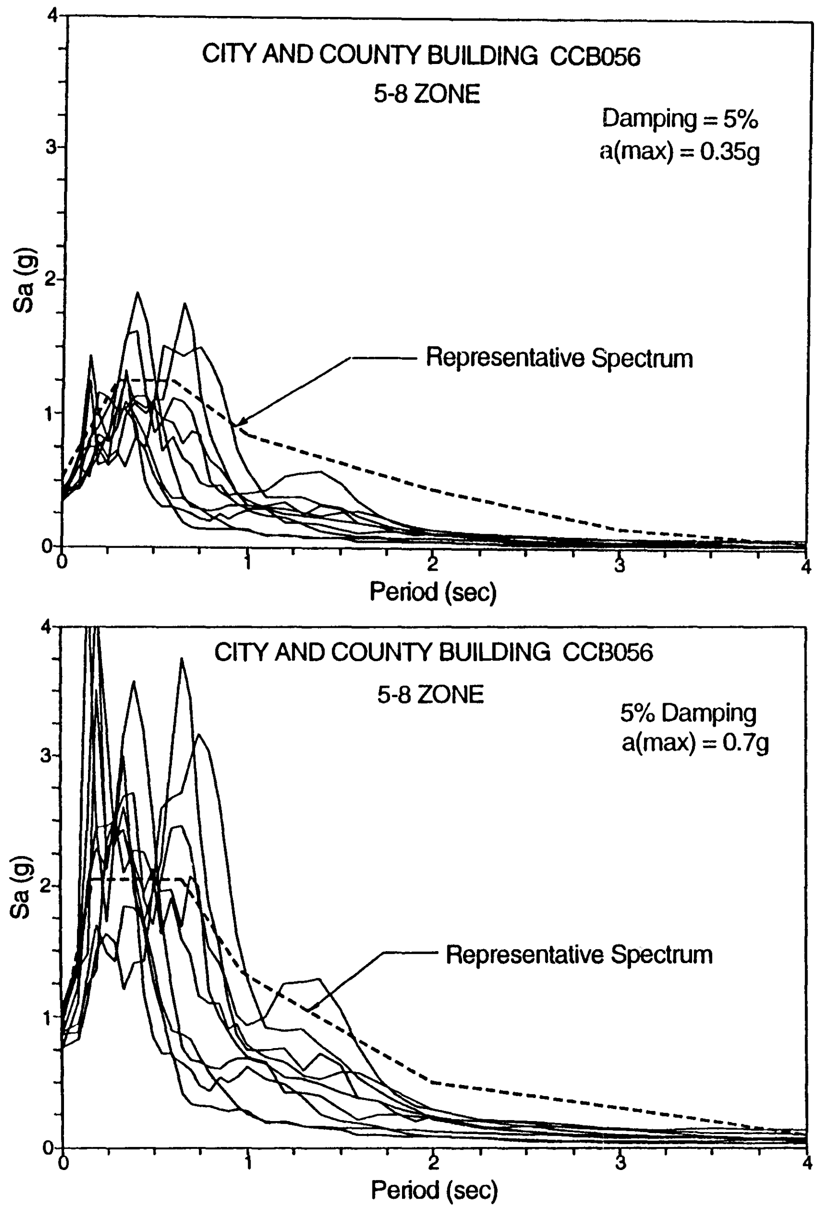

Figure B.13. Computed acceleration response spectra for City and County Building (CCB056) at 50 (top) and 250 (bottom) year rock acceleration levels.

APPENDIX C

MEAN ACCELERATION RESPONSE SPECTRA FOR USGS RECORDING SITES AND REPRESENTATIVE SPECTRUM SHAPES IN EACH AMPLIFICATION ZONE

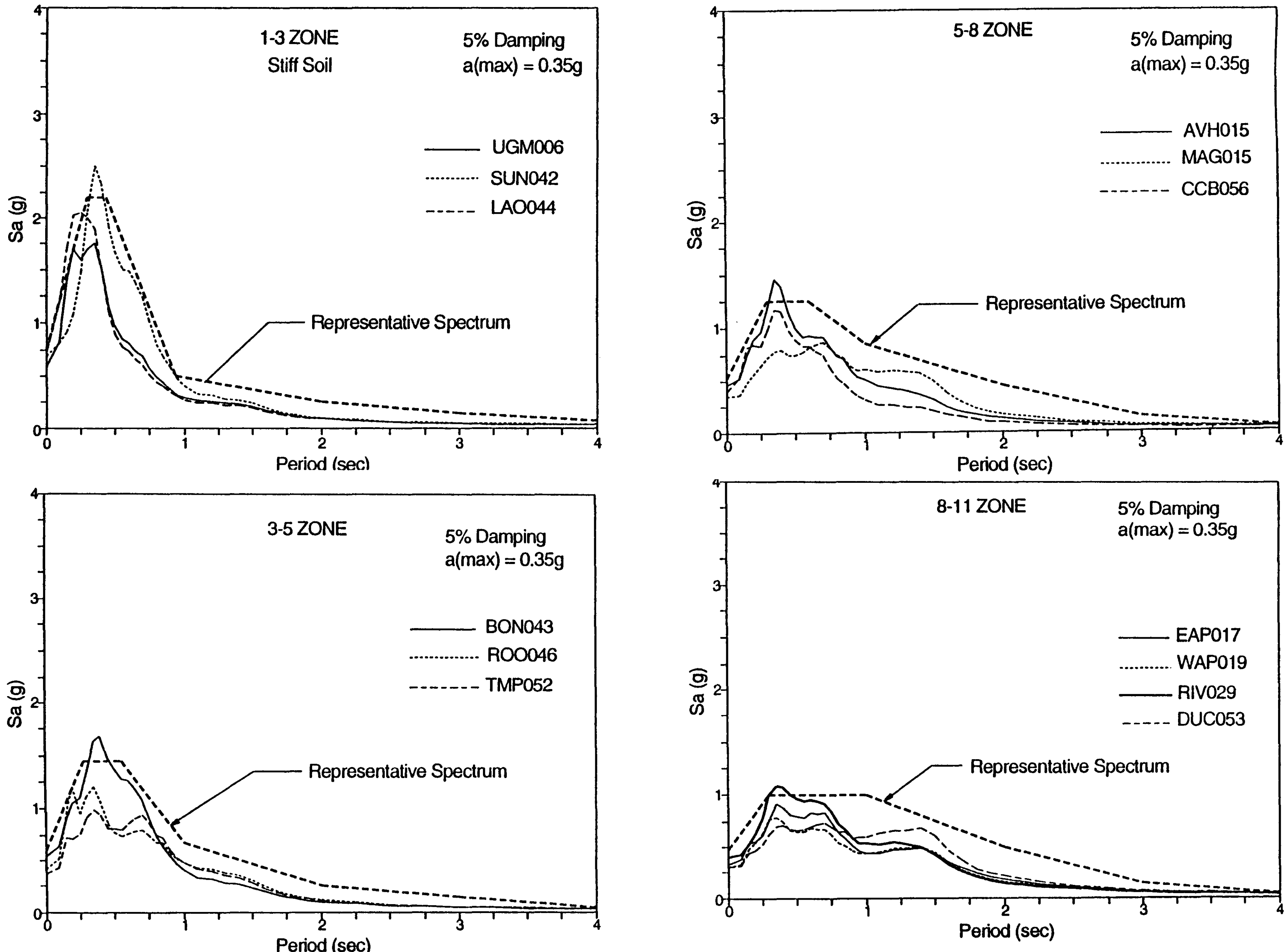

Figure C.1. Mean acceleration response spectra and representative spectrum for stiff soil (1-3 Zone), medium stiff soil (3-5 Zone), medium soft soil (5-8 Zone), and soft soil (8-11 Zone) at the 50 year acceleration a_{max} (rock) = 0.35g.

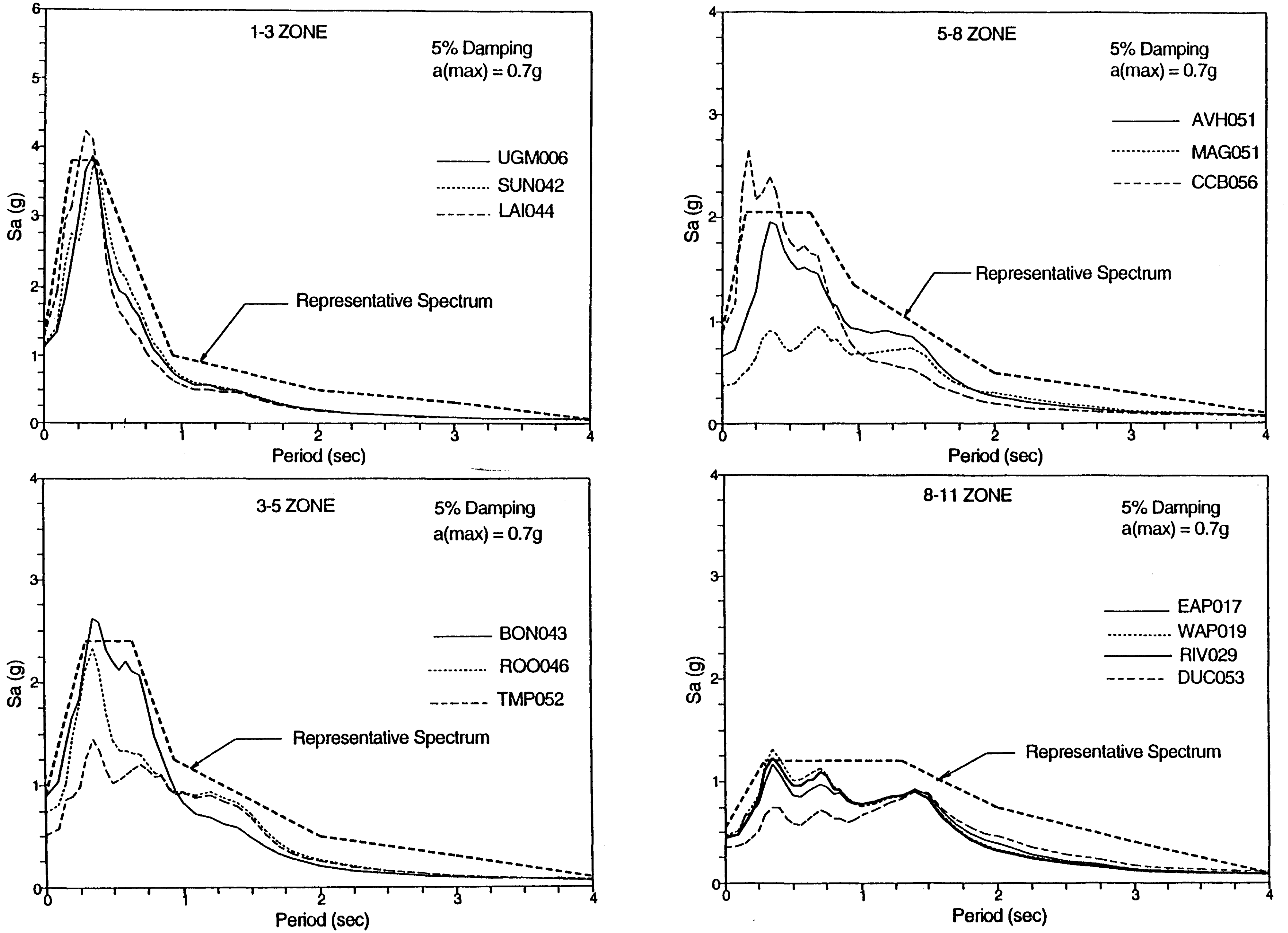

Figure C.2. Mean acceleration response spectra and representative spectrum for stiff soil (1-3 Zone), medium stiff soil (3-5 Zone), medium soft soil (5-8 Zone), and soft soil (8-11 Zone) at the 250 year acceleration a_{max} (rock) = 0.7g.