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SITE-SPECIFIC EARTHQUAKE RESPONSE ANALYSIS, CASE STUDY: EL MERCADO LIBERTAD

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ABSTRACT

A site-specific response analysis using non-linear soil properties was conducted for the Mercado Libertad located in Guadalajara, Jalisco, Mexico. The analyses have been conducted using partially site specific historical data-based and fully site specific inputs. All input data were generated by local site characterization studies and lab testing of samples from the site. To assess the input earthquake motion, both a deterministic (quasi-probabilistic) and a probabilistic seismic hazard approach were considered. Three of the largest faults in the general vicinity of the site were first identified. A maximum earthquake magnitude was provided for each one of these three sources. A controlling earthquake (the earthquake that is expected to produce the strongest level of shaking at the site) was selected using published attenuation curves of maximum acceleration versus distance from causative fault for the specific fault characteristics. The bedrock motion was analytically propagated up through the soil profile to the base of the structure. Liquefaction analysis was performed and the factor of safety against liquefaction was determined for this site. An amplification of the earthquake induced rock accelerations was observed for all of the calculated configurations. This amplification factor ranged from about 1.2 to 1.5 times the estimated base rock maximum acceleration. The recommended design value for the peak spectral acceleration is about 0.42g at a period of about 0.2 seconds. It is definitely required to improve the soil conditions in some layers in order to reduce or eliminate the possibility of heavy damage due to soil liquefaction during an earthquake.

Keywords: Spectrum, Earthquake, Liquefaction, SHAKE

INTRODUCTION

Ground response analyses are used to predict ground surface motions for development of design spectra, to evaluate dynamic stresses and strains for evaluation of liquefaction hazards, and to determine the earthquake-induced forces that can lead to instability of earth and earth retaining structures. One-dimensional ground response analyses are based on the assumption that all boundaries are horizontal and that the response of a soil deposit is predominantly cause by horizontal shear waves (SH-waves) propagating vertically from the underlying bedrock.

ONE-DIMENSION WAVE PROPAGATION

The input motions, at the base of a structure, can be generated in a variety of ways. Case I - Code-based analysis, Case II - Partially site specific historical data-based and Case III - Fully site specific.

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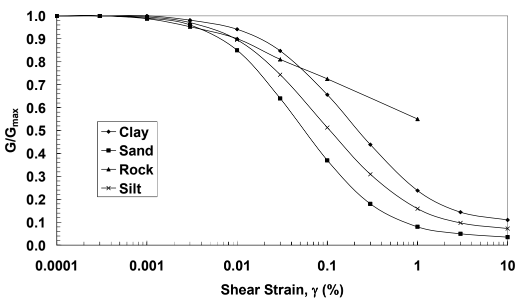
All input data were generated by local site characterization studies and lab testing of samples from the site. The propagation of stress waves is most easily understood by first considering an unbound, or "infinite," medium. A simple, one-dimensional idealization of an unbound medium is that of an infinitely long rod or bar. Using the basic requirements of equilibrium of forces and compatibility of displacements, and using strain-displacement and stress-strain relationships, a one dimensional wave equation be derived and solved. The one-dimensional wave equation can be written as follows (Idriss and Seed, 1968),

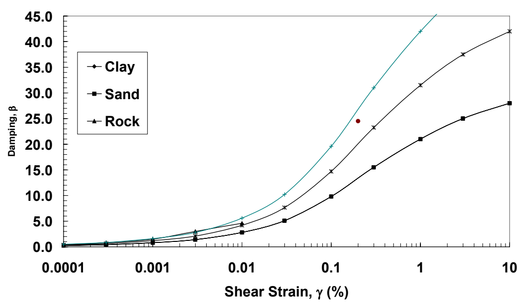
$$\frac{\partial^2 u}{\partial t^2} = v_p^2 \frac{\partial^2 u}{\partial x^2} \tag{1}$$

where v_p is the wave propagation velocity, u is the soil displacement, t is the time, and x is the travel distance. The selection of the bedrock motion may be "probabilistic" or "quasi-probabilistic". Using the "quasi-probabilistic" approach the attenuation of shaking severity as the disturbance travels from fault to site will be modeled and calculated (Campbell and Bozorgina, 1994). The propagation of the bedrock motion up through the soil profile to the base of the structure can be done with any of several computer codes, including 2-D or 3-D models. However, by far the most commonly used code is program SHAKE, a 1-D wave propagation code by Schnabel et al. (1972) and modified by Idriss and Sun (1992). Especially under the influence of moderate to large earthquakes, most soil deposits are non-linear in their response. This means, quite simply, that the appropriate modulus and damping for a given layer depends on the shear strain level, which varies with time throughout the earthquake. However, through research, development, and calibration, Schnabel et al. (1972) have managed to make SHAKE capable of achieving good answers (i.e., good amplification factors) using an equivalent linear analysis. However, the nonlinear relationship between modulus, damping, and strain must be input to SHAKE for each layer in order for SHAKE to be able to iterate to an appropriate "effective" modulus and damping for each layer. Thus, the actual nonlinear hysteretic stress-strain behavior of cyclically loaded soils can be approximated by equivalent linear soil properties. The equivalent linear shear modulus, G, is generally taken as a secant shear modulus and the equivalent linear damping, β, as the damping ratio that produces the same energy loss in a single cycle as the actual hysteresis loop.

Since the computed strain level depends on the values of the equivalent linear properties, an iterative procedure is required to ensure that the properties used in the analysis are compatible with the strain levels in all the layers. This iterative process is programmed in SHAKE. The nonlinear relationship between modulus, damping, and strain can be determined from laboratory dynamic testing of samples from each layer at the site. However, for a Case II analysis, reliance would be placed on dynamic test data from the literature, of which there is much. These test data from the literature have been placed into three groups: Rock, Sand, and Clay. The ratio G/G_{max}, where G = shear modulus and G_{max} = maximum shear modulus corresponding to very low shear strain of 10^{-5} to 10^{-4} percent, is plotted versus shear strain, γ, for each material type. These three plots are embedded in SHAKE and referred to as the "Standard Curves" as shown in Figure 1(a) and Figure 1(b). These standard curves can be replaced by curves derived from site-specific testing if desired, or they can be used "as is" by simply designating each layer as rock, sand, or clay.



(a) G/Gmax



(b) Damping, β

Figure 1. Standard Curves in Program SHAKE (Idriss and Sun 1992)

However, SHAKE must also be supplied with a means of determining G_{max}. The value of G_{max} for each layer can be determined from field measurements of shear velocity, V_s, for each layer. If these values of V_s are input to SHAKE, it will calculate G_{max} internally. With only slight loss of accuracy, V_p can be measured in the field and estimated values of Poisson's ratio, ν, can be used to get V_s. As an alternative to field measurements, resonant column or torsional shear lab tests can be run to get low-strain G values. However, very "undisturbed" specimens are needed for these tests. For the sand and Sandy layers a correlation between standard penetration test (SPT) N-vlues and G_{max} is used. For clays and clayey layers a correlation between undrained shear strength, S_u, and G_{max} is used.

LIQUEFACTION ANALYSIS

The term liquefaction has historically been used in conjunction with a variety of phenomena that involve soil deformations caused by monotonic, transient, or repeated disturbance of saturated cohesionless soils under undrained conditions. The generation of excess pore pressure under undrained loading conditions is the hallmark of all liquefaction phenomena. When cohesionless soils are saturated and subjected to rapid loading, conditions are typically undrained. Therefore, the tendency for densification causes excess pore pressures to develop and effective stresses to decrease. The factor of safety against liquefaction, FS, can be determined using the following relation (Castro, 1975),

$$\mathrm{FS} = \frac{(\mathrm{CSR})_\mathrm{L}}{(\mathrm{CSR})_\mathrm{R}} \tag{2}$$

where $(CSR)_L$ is the cyclic stress ratio required to cause liquefaction and $(CSR)_R$ is the cyclic stress ratio applied by the earthquake. The $(CSR)_R$ is related to the exciting motion and can be calculated either using SHAKE program (Schnabel et al., 1972), or using the Seed and Idriss (1971) simplified approach,.

$$(CSR)_R = 0.65 \frac{a_{\max}}{g} \frac{\sigma_v}{\sigma_v^{`}} r_d \tag{3}$$

where a_{max} is the maximum ground acceleration, σ_v is the total vertical stress at a certain point, $\sigma_v^{`}$ is the effective overburden pressure at the same point, r_d is a depth reduction factor (Kramer, 1996), and g is gravity acceleration. The $(CSR)_L$ is related to the soil properties (Standard Penetration Test) be calculated using Seed et al., (1985) relation (Figure 2). This figure shows the relation between $(N_1)_{60}$ and $(CSR)_L$ for Earthquake Magnitude, M=7.5 and for different percentages of fines. Where,

$$(N_1)_{60} = C_N \times N \tag{4}$$

where N is the uncorrected SPT N-Value for 60 % efficiency, $(N_1)_{60}$ is the N-Value at 60% efficiency correlated to an overburden pressure of 1 tsf (Seed et al., 1983), and C_N is the correction factor for overburden stress and can be calculated as follows (Day 2002)

$$C_N = \left(\sigma_v^{`}\right)^{-0.496} \quad (\sigma_v^{`} \text{ in tsf.}) \tag{5}$$

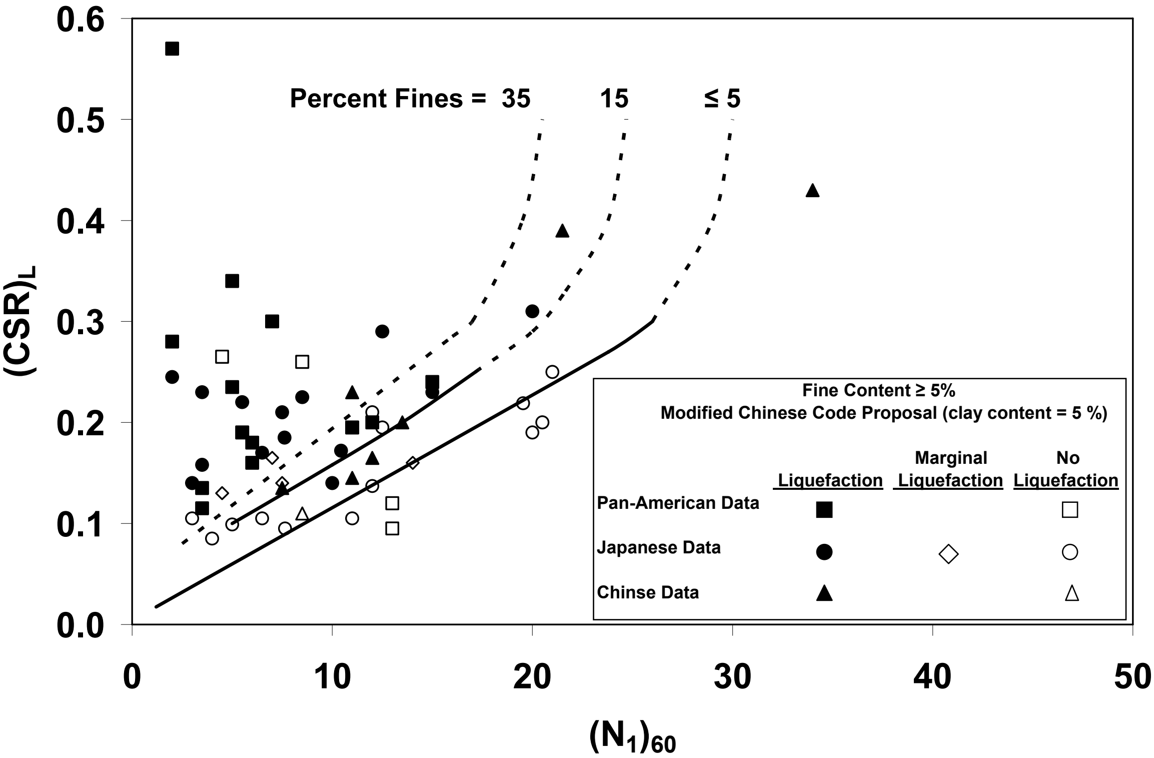


Figure 2. Relationship Between $(CSR)_L$ and $(N_1)_{60}$ (after Seed et al., 1985)

CASE STUDY: EL MERCADO LIBERTAD

A site-specific response analysis using non-linear soil properties was conducted for the Mercado Libertad located in Guadalajara, Jalisco, Mexico. The risk of seismic activity has been evaluated in cities like the metropolitan zone of Guadalajara. It has been evaluated from the scientific point of view to assess and locate the local and regional structures geologically active to determine the probability of earthquakes occurrence. The estimation of the seismic risk within a zone depends on the knowledge and potential characteristics of the sources that trigger the earthquakes. This knowledge allows the assessment of the so called "local seismicity" that constitutes the first step, fundamental and unavoidable to the process that ends up in specifications of the design spectra for civil structures (Seed et al., 1976). Local seismicity is understood as the degree of seismic activity within a volume of the earth surface and the quantification is based on the magnitude that can be reached by the earthquake, the number of earthquakes that can occur within an interval of time, the delivered energy or any other similar measure (Arboleda and Ordaz, 1997).

Mexico Occident Tectonic

The "Jalisco" state is part of the North America tectonics that interacts with plates as Rivera and Cocos. This interaction is produced by means of a process that is known as "subduction" (a process wherein a plate plunges beneath another one). Within the North America plate there are three micro-structures which converge and that correspond to the tectonic Fosas or Graben, known as "Tepic-Zocoalco, Colima and Chapala" (Dominguez et al., 1997). The first two isolate another block which is denominated "Bloque Jalisco". (Figure 3)

Seismicity History

Jalisco is located within a zone of great seismic activity which dates since 1542 like a testimony of the events that had occurred in the state, some of the most representatives earthquakes -because of their characteristics and their consequences- were registered in 1875, 1911, 1912, and 1932 as shown in Figure 4 (Lazcano, 2001).

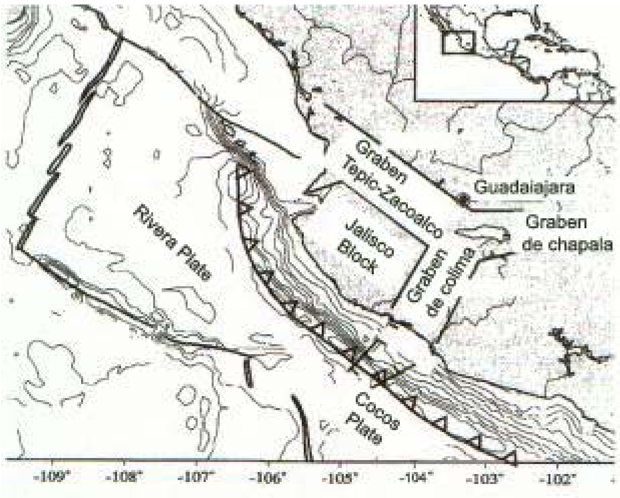


Figure 3. Mexico Occident Tectonic (after Lazcano, 2001)

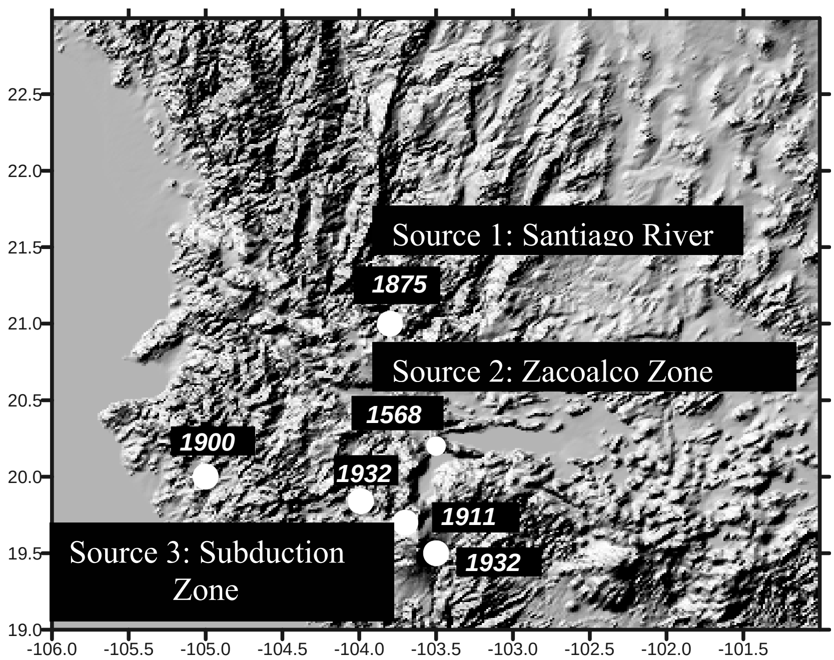


Figure 4. Historic Earthquakes and Main Seismogenetic zones in Jalisco State (Lazcano, 2001)

From the historic catalogs and the current instrumental activity, it is concluded that there are three major seismic genetic zones that can generate earthquakes that affect the Metropolitan Zone of Guadalajara and they are The Santiago River Zone, The Zacoalco Zone, and The Subduction Zone (Waitz and Urbina, 1919). The Santiago River is located approximately 5 km from the metropolitan zone. Possibly it was the origin of the 1875 earthquake (even though currently it is attributed to the subduction zone) and the 1912 earthquake which was also originated in this seism genetic zone according to the geologic evidences and the distribution of the epicenters (Figure 4). The Zacoalco zone is to the southwest of the Chapala lake, approximately 50 km in a straight line to the Guadalajara City (Figure 4). The Subduction Zone is in the Mexican Pacific in front of the Jalisco seashore and Colima (Figure 4). Table 1 shows the maximum theoretical acceleration generated in site due to three of the nearer sources.

Table 1: Maximum Theoretical Acceleration of the Sources Near the Mercado Libertad

			Maximum Theoretical Acceleration (g)							
	M	Distance (Km.)	Campbell (1981)	Singh	Ordaz (1989)	Esteva y Villaverde (1973)	McGuire (1978)	Bufaliza (1984)	Atkin	Ohno
Source 1	7.5	61	0.112	0.204	0.111	0.226	0.179	0.131	0.18	0.127
Source 2	6.0	50	0.042	0.836	0.052	0.086	0.082	0.051	0.11	0.054
Source 3	8.4	161	0.089	0.276	0.038	0.117	0.116	0.109	0.06	0.064

Site Response Analyses

With the discussion of cases I, II, and III introduced previously, it should be noted that the approach adopted by the authors for El Mercado Libertad is intermediate between Cases II and III. Because the laboratory dynamic test program was not concluded, substantial reliance was placed on the standard curves in SHAKE. Because the site boring logs indicated a dominance of sands and silts, a G/G_{max} curve intermediate between the standard curves for sand and clay was chosen. For the G_{max} values, two sets of analyses were performed. First, the V_s values as reported for the site were used without modification. These V_s values were then used by SHAKE to compute G_{max} values internally. A second series of analyses was completed using Ishihara's correlation between SPT N-values (from the boring logs) and V_s (Ishihara and Towhata, 1985). These two series of runs were made as a part of a parametric study in an effort to envelop the probable site response at the structure foundation.

No existing earthquake acceleration-time histories recorded for the region were available for the performance of this analysis. Two earthquake records used for the final analyses were synthetically created and made to match the bedrock seismological data reported for the area. The first earthquake record is based on the Taft Earthquake acceleration-time history scaled for a maximum acceleration of 0.11g and a predominant period of 0.35 seconds. Results using this record are identified as $T_p = 0.35$. Scaling and filtering the original Taft acceleration-time history to match approximately the Uniform Risk Spectrum for a return period of 500 years reported for the site created the second earthquake record used to calculate the average spectra. Several trials were required in order scale the original Taft record to an acceleration-time history that matches this spectrum to the degree as shown in Figure 5. Trial-2 was used to represent the smoothed response spectrum as shown in Figure 5. Results using this second record matching are identified as $T_p = 0.2$.

Response Spectra Using Reported Vs

As part of our parametric studies, two different approaches for obtaining the dynamic soil properties were used. The first approach was to use the reported shear wave velocities as shown in Figure 6, soil densities, and soil classification for the four boreholes, without modification. The shear wave velocity together with the soil density was used to calculate the maximum shear modulus, G_{max} (at very small strains).

The damping function was obtained using the average curves reported in the literature for the same soil types. Spectra resulting from use of these site-specific data are identified as DATA on the graphs. Calculated spectral curves for the four different profiles were identified as S2, S4, S5, and S7 corresponding the boreholes. Figures 7 shows the acceleration response spectra at the ground surface, S_a, for the four soil profile using the data reported for shear wave velocities, based on the Taft Earthquake record with maximum ground acceleration, a_g=0.11g, and a predominant period, T_p=0.35 seconds for damping value of 5%.

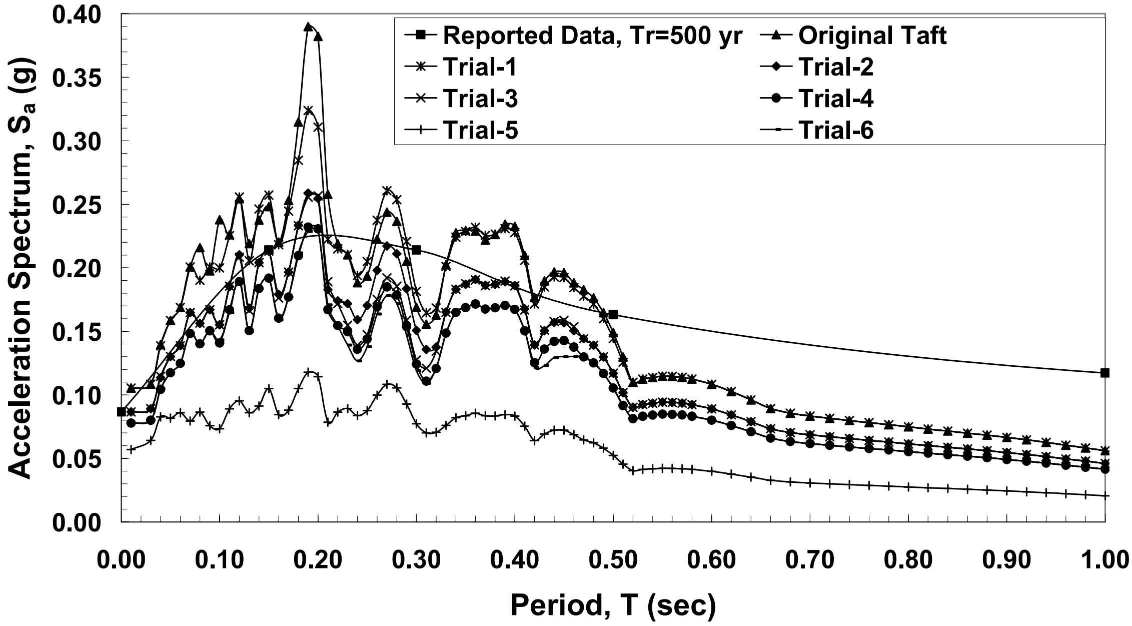


Figure 5. Trials to Match Reported Acceleration Response Spectrum For Bedrock Motion

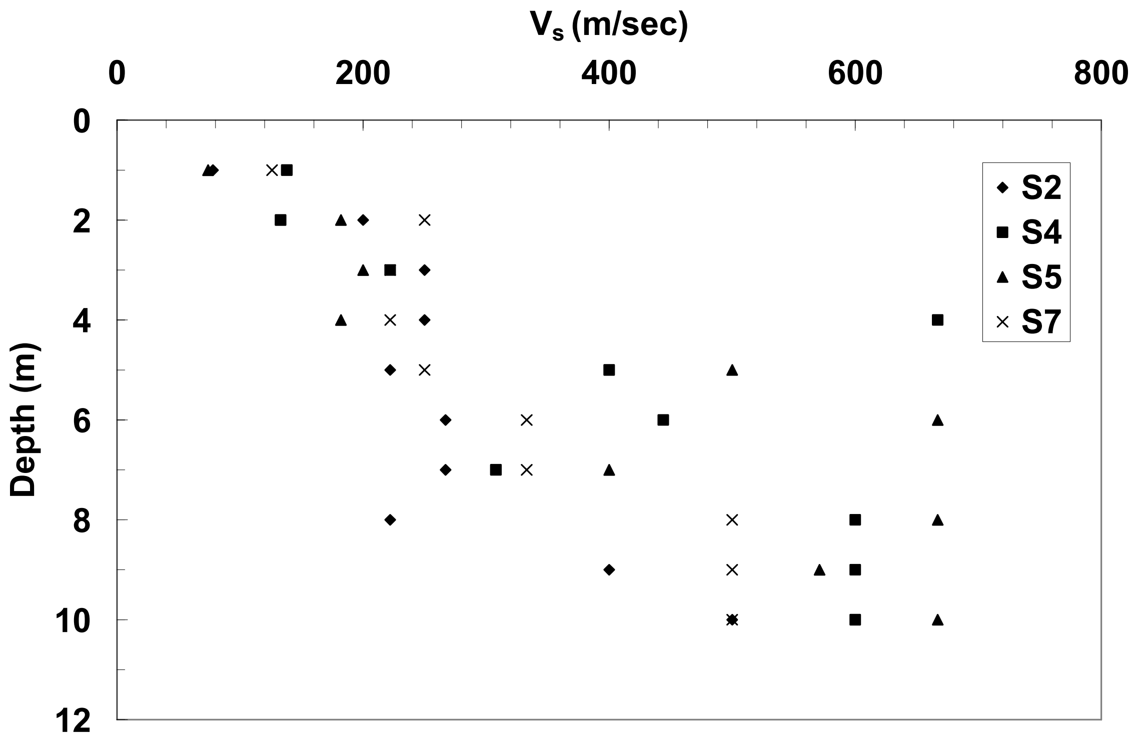


Figure 6. Shear Wave Velocity, V_s, for Soil Profiles, S2, S4, S5, S7

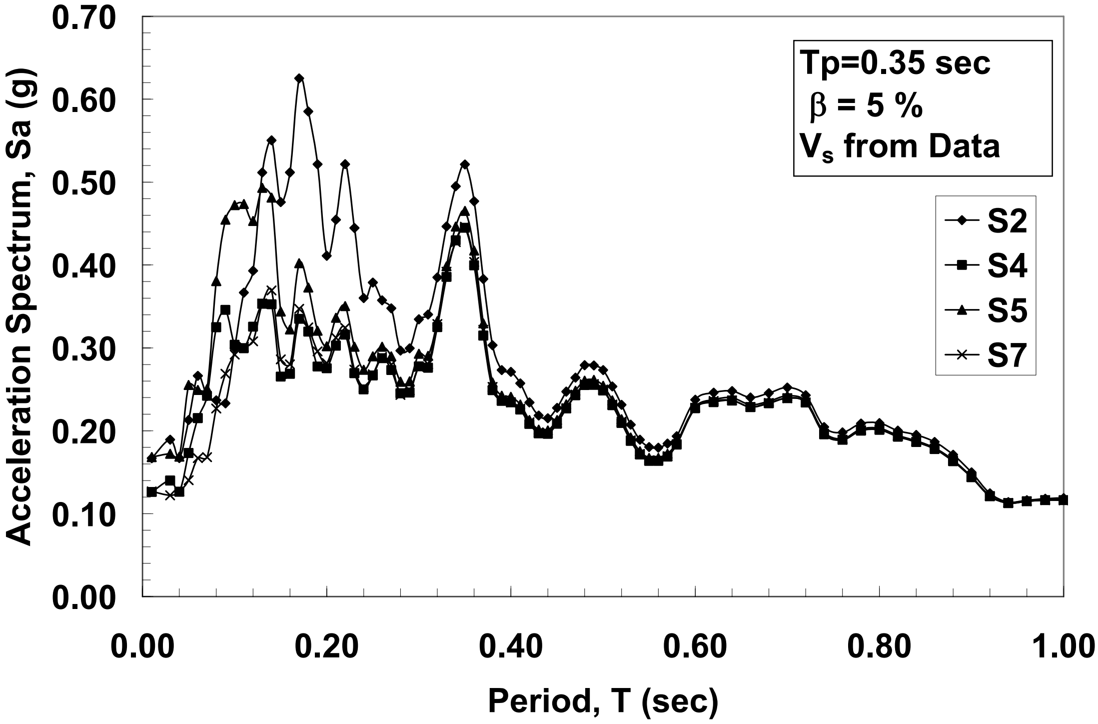


Figure 7. Acceleration Response Spectrum using V_s from Reported Data

Response Spectra Using Reported SPT, N Values

The second method of obtaining the dynamic soil properties was similar to the method described above but different shear wave velocities were used. For this second approach, the velocity was calculated using Ishihara's correlation between shear wave velocity, V_s, and Standard Penetration Test (N Values) as shown in Figure 8 instead of the V_s values reported for the site. The maximum shear modulus was used to fix the shear modulus degradation curve starting point in SHAKE. Spectra resulting from this second approach are identified as ISH, indicating Ishihara.

Figure 9 shows the acceleration response spectra at the ground surface, S_a, for the four soil profiles using shear wave velocities from the Ishihara correlation (ISH) and Taft Earthquake (T_p=0.35 sec) for 5% damping value.

Discussion of Results

A total of 16 different spectral curves were obtained from the SHAKE program analyses for output at the ground surface. These 16 curves arise from 4 soil profiles, 2 bedrock input motions, and 2 methods for getting V_s. For each set of 16 curves some averaging and smoothing were done. Figure 10 shows the calculated spectra identifying each individual profile and data composition. Figure 11 shows upper, minimum, and average smoothed envelopes of the resulting spectra superimposed by the resulting response spectra points.

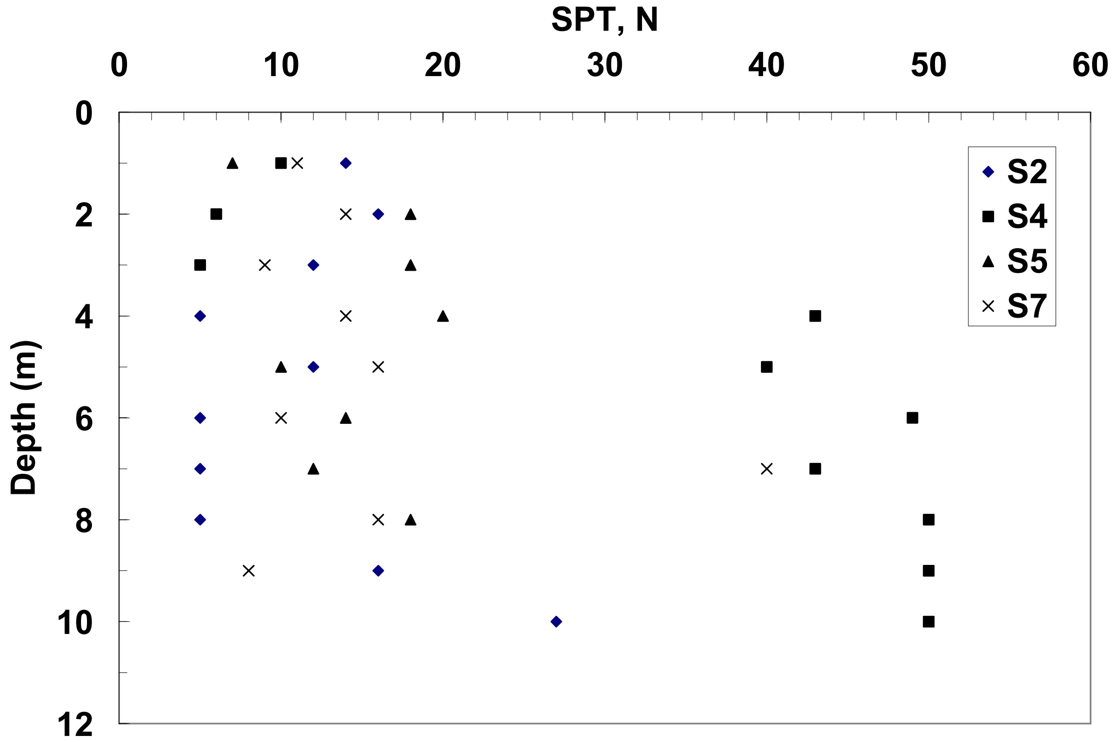


Figure 8. SPT Values for four Soil Profiles, S2, S4, S5, and S7

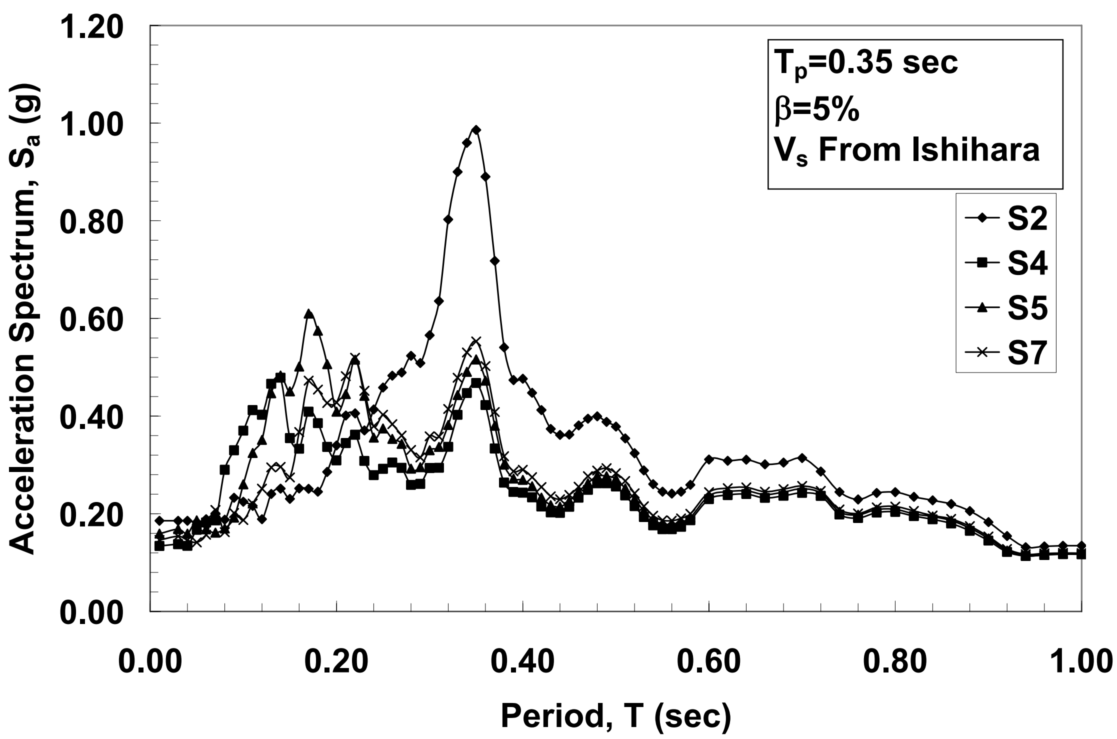


Figure 9. Acceleration Response Spectrum using Vs from Ishihara

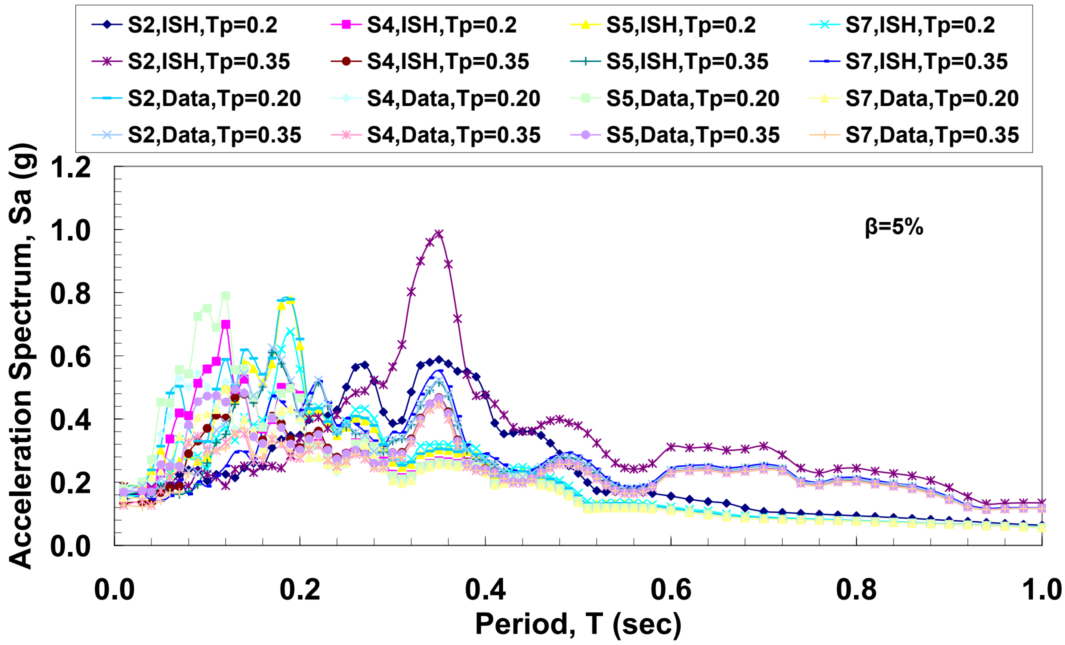


Figure 10. Response Spectra for the all Sixteen Runs (Four profiles, Tp=0.35, 0.2 sec and a 5% Damping Ratio)

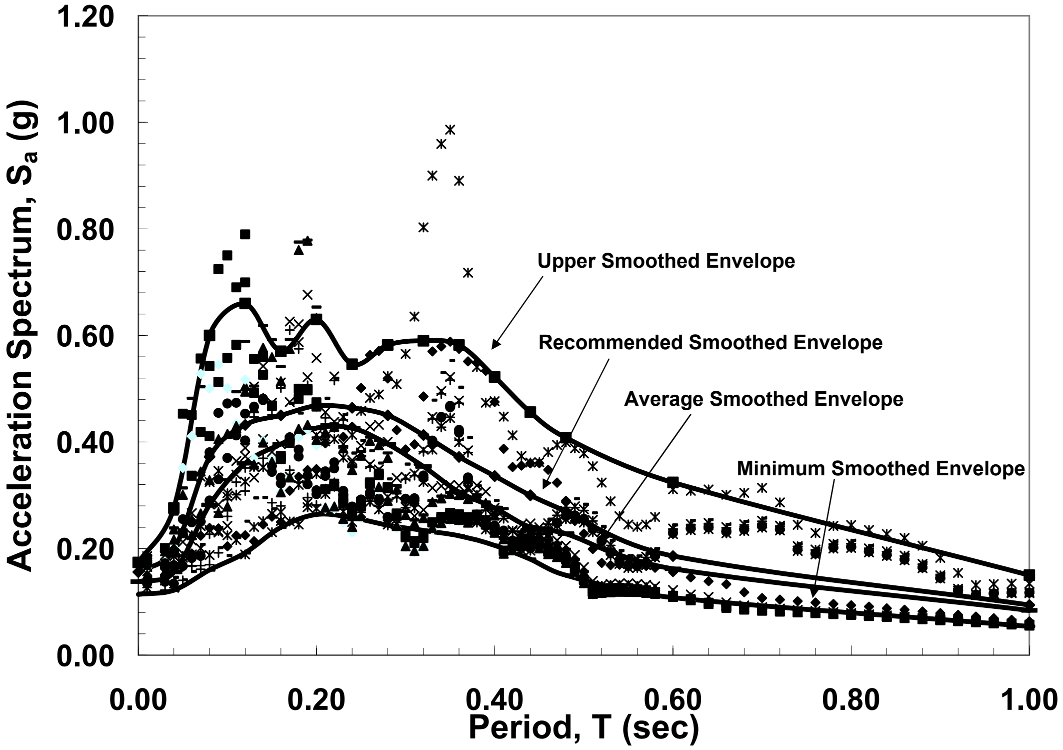


Figure 11. Upper, Lower Smoothed Envelopes Superimposed by the Resulted Data Points

Liquefaction Assessment

The factor of safety against liquefaction was determined for the four site profiles using the procedure introduced earlier. Table 2 shows a sample spreadsheet which outlines the calculation of the factor of safety for soil profile S2. As shown in Table 2, the factor of safety against liquefaction in some layers went below one, which indicates that there is a possibility of liquefaction.

T able 2: Factor of Safety against Liquefaction for Soil Profile S2

Layer	γ (Kg/ m^3)	*Thick* (m)	r_d	σ_v	$\sigma`_v$	a_{max}	CSR	N	C_N	N_1	% Fine	CSR %Fine	M (Corr)	Corr. CSR	F.S.
1	1631	0.6	0.99	100.2	100.2	0.17	0.109	14	4.37	61.2	33	6	1.087	0.6522	5.96
2	1489	0.6	0.99	291.9	291.9	0.17	0.109	16	2.58	41.3	30	6	1.087	0.6522	5.96
3	1465	0.6	0.99	473.4	473.4	0.17	0.109	7	2.03	14.2	25	22	1.087	0.2391	2.2
4	1697	0.6	0.98	667.7	667.7	0.17	0.108	12	1.71	20.6	50	6	1.087	0.6522	6.0
5	1584	0.6	0.98	869.3	746.5	0.17	0.126	5	1.62	8.13	72	28	1.087	0.3044	2.41
6	1701	0.6	0.98	1071.1	825.5	0.17	0.140	1	1.54	1.55	50	11	1.087	0.1196	0.85
7	1692	0.6	0.98	1279.6	911.1	0.17	0.152	1	1.47	1.47	32	05	1.087	0.0544	0.36
8	1684	0.6	0.98	1487.0	995.7	0.17	0.162	12	1.41	16.9	35	3	1.087	0.3261	2.01
9	1516	0.6	0.97	1683.7	1069.5	0.17	0.168	5	1.36	6.81	96	3	1.087	0.3261	1.93
10	1333	0.6	0.97	1858.7	1121.7	0.17	0.177	1	1.33	1.33	51	11	1.087	0.1196	0.67
11	1400	0.6	0.97	2026.7	1166.8	0.17	0.186		1.30				1.087		
12	1400	0.6	0.97	2198.7	1216.0	0.17	0.194		1.28				1.087		
13	1395	0.6	0.97	2370.4	1264.9	0.17	0.201		1.25				1.087		
14	1410	0.6	0.96	2542.8	1314.4	0.17	0.205	16	1.23	19.7	47	6	1.087	0.6522	3.18
15	1525	0.6	0.96	2723.1	1371.9	0.17	0.210	27	1.20	32.5	40	6	1.087	0.6522	3.09
16	1498	0.6	0.96	2908.8	1434.9	0.17	0.215	50	1.18	58.9	17	6	1.087	0.6522	3.03

CONCLUSIONS AND RECOMMENDATIONS

A site-specific response analysis using non-linear soil properties was calculated for the Mercado Libertad in Guadalajara, Jalisco, Mexico. A recommended average response spectrum is presented. Peaks for some of the calculated spectra were smoothed out as they are considered to be possible but not probable occurrences. In general, an amplification of the earthquake induced rock accelerations was observed for all of the calculated configurations. That is, the bedrock motion was amplified in route to the ground surface. This amplification factor ranged from about 1.2 to 1.5 times the estimated base rock maximum acceleration. Thus the recommended design value for the ground surface maximum acceleration is about 0.16g. The recommended design value for the peak spectral acceleration is about 0.42g at a period of about 0.2 seconds (Figure 11). Thus, this spectral acceleration value of 0.42g compared to the zero-period value of 0.16g corresponds to a spectral acceleration amplification factor of about 2.6

Liquefaction analyses were performed at the four different soil profiles. It is definitely required to improve the soil conditions for some layers in order to reduce or eliminate the possibility of heavy damage due to soil liquefaction during an earthquake. Three possible methods are recommended but an economic analysis should provide the best solution. Given that the structure is already in place, the first recommended method is the injection of a low viscosity grout in the layers with the lowest factor of safety against liquefaction. The second recommended solution is the installation of drilled shafts at selected locations and install them all the way down to the bedrock. The third and last possible solution is a combination of the two methods recommended above.

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