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SELECTION OF AN APPROPRIATE a_{max} FOR LIQUEFACTION ANALYSES FROM ONE-DIMENSIONAL SITE RESPONSE ANALYSES

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ABSTRACT

This paper is an attempt to clarify a possible confusion regarding which maximum ground acceleration (a_{max}) should be used when performing a site-specific liquefaction analysis. Usually, one-dimensional free-field site response analysis is performed to estimate a_{max} at the foundation elevation and the strain-compatible soil parameters within the soil profile. From the soil-structure interaction (SSI) analysis perspective, this calculation is repeated for the best-estimate, lower-bound, and upper-bound soil profiles. For this, the shear moduli are adjusted using a coefficient of variation (c_v) to account for the spatial variation in the soil properties and the uncertainties in SSI calculations. The procedure is explained in ASCE 4-98. On the other hand, establishing a design a_{max} is open to interpretation in current guidelines and procedures that discuss the liquefaction analysis. The simplified cyclic stress ratio (CSR) procedure is an empirical method that uses a depth dependent stress reduction factor (r_d). In the CSR procedure, a_{max} corresponds to the magnitude of an earthquake that is assumed to occur at the site. The question is, should the a_{max} from the one-dimensional response analysis using the best estimate (or representative) soil profile be used in liquefaction analysis? Or, should the average or possibly the least favorable a_{max} be used? If the least favorable a_{max} is used, then the corresponding soil profile should also be used in liquefaction analyses. Historically, r_d values are based on studies done using different earthquake time histories and average soil profiles. In this study, a small scale parametric study is conducted to show that the average a_{max} from the one-dimensional response analyses with best estimate soil profile is appropriate to use in simplified liquefaction analyses.

INTRODUCTION

Liquefaction is an earthquake hazard explained by the shear strength loss due to excess pore pressure induced by earthquakes in saturated mostly cohesionless soils. Small or large displacements, sand boiling at the ground surface, and landslides can be observed in soils that liquefied.

When amplification of seismic loads are anticipated at a site, one dimensional (1D) site or ground response studies are performed. This analysis provides the strain-compatible soil properties of the soils beneath the site for use in soil-structure interaction analyses and sometimes can be used to determine the maximum ground acceleration to be used in site specific liquefaction analysis.

In more than one instance, when preparing liquefaction and 1D ground response calculations for the same projects, the author encountered reviewers' comments demanding the use of the upper-bound maximum surface acceleration estimated by 1D analyses. The study presented here is an attempt to explain why that upper-bound maximum ground acceleration (a_{max}) cannot be used directly in the simplified liquefaction calculations and why that approach is most likely not a conservative one, even if used correctly.

The study is applicable to soils that are accepted as liquefiable. The question of which soils are liquefiable and the criteria used in the decision process is out of scope of this study, but discussed and summarized in detail in many other studies including Youd et al. (2001) and Boulanger and Idriss (2006)

Earthquakes are usually quantified by ground motion parameters: magnitude, design response spectrum, maximum ground acceleration, and time histories. The seismic loading can be expressed in terms of cyclic shear stresses because the earthquakes produce stress waves that travel through the rocks and soils. If a site specific response analysis is performed using an earthquake time history, the loading can be obtained from the time history of the shear stress that developed during the shaking. The loading can also be estimated using an empirical simplified approach.

The Simplified Procedure first developed by Seed and Idriss (1971) for liquefaction potential evaluation is still the most widely accepted empirical method based on field test results. With the accumulation of more data and knowledge and advanced computer use, more studies have been performed since then to improve the accuracy of the method. In 1996, a group of experts gathered for a workshop sponsored by National Center for Earthquake Engineering Research (NCEER) to review the developments and gain concensus on the evaluation of liquefaction resistance. A technical report and a paper were published in 1997 and 2000, respectively. Both the report and the paper are basically the same; however, there are a couple of differences in some figures and equations. Therefore, in this document, both of them are cited as NCEER (1997) and Youd et al. (2001).

The factor of safety against liquefaction is defined as the ratio of the Cyclic Resistance Ratio (CRR) to the Cyclic Stress Ratio (CSR). These terms are explained below in the context of the simplified procedure.

SIMPLIFIED PROCEDURE

The procedure characterizes the earthquake loading by the amplitude of an equivalent uniform cyclic stress and the liquefaction resistance by the amplitude of the uniform cyclic stress required to produce liquefaction in the same number of cycles. The simplified procedure utilizes empirical relations using field test results such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), and shear wave velocity (V_s) measurements to obtain the CRR.

The liquefaction potential is evaluated by comparing the earthquake load with the liquefaction resistance at various depths in the soil profile. The earthquake load is expressed in terms of a cyclic stress ratio (CSR), as first formulated by Seed and Idriss (1971) and re-arranged by Youd et al. (2001a) as:

$$CSR = \tau_{ave}/\sigma'_{vo} = 0.65\,(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (1)$$

where:

τ_{ave} = average cyclic shear stress

a_{max} = the maximum horizontal acceleration at ground surface

g = acceleration due to gravity

σ_{vo}, σ'_{vo}= total and effective vertical (overburden) stresses

r_d = depth (z) dependent stress reduction factor.

Because of the deformable nature of soils, the actual shear stress that may be obtained by site specific analyses should be smaller than the calculated shear stress at a depth z. This reduction in stress is accounted for by the use of an r_d factor which ranges between 0.5 and 1. In NCEER (1997), approximations of r_d for use in non-critical projects are given as follows.

$$r_d = 1.0 - 0.00765\ z \text{ for } z \le 9.15 \text{ m} \qquad (2a)$$

$$r_d = 1.174 - 0.0267\ z \text{ for } 9.15 \text{ m} < z \le 23 \text{ m} \qquad (2b)$$

$$r_d = 0.744 - 0.008\ z \text{ for } 23 < z \le 30 \text{ m} \qquad (2c)$$

$$r_d = 0.50 \text{ For } z > 30 \text{ m} \qquad (2d)$$

The stress reduction curves presented by Seed and Iddriss (1971) and the mean value line from the equations 2a through 2d are shown in Fig. 1.

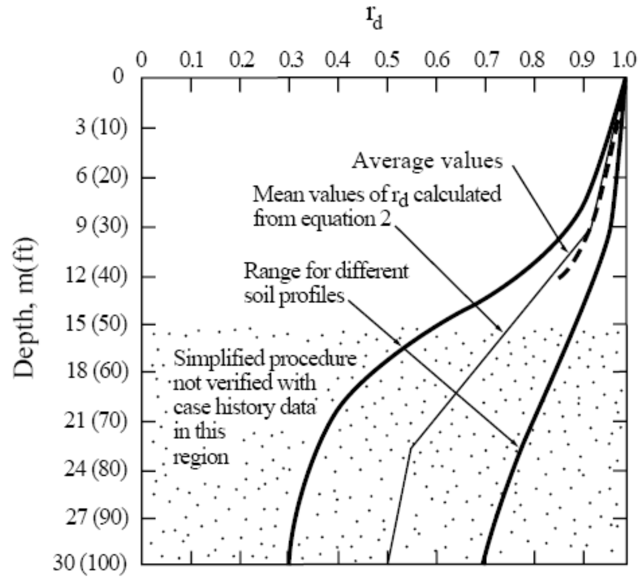

Fig. 1. r_d versus depth curves developed by Seed and Idriss (1971), with added mean value line from equation 2 (from Youd et al. 2001b).

In determining CRR, primarily a Vs based procedure has been selected in order to be consistent with the one dimensional ground response analysis provided later in this document. In other words, a site response analysis involves uncertainties. These uncertainties, especially in dynamic material properties, may be accommodated by parametric variations deterministically with best estimate, lower-, and upper-bound shear moduli of the soil (or shear wave velocity).

As a second method of estimated CRR, the SPT based simplified procedure has been used. A correlation between corrected standard penetration N value ($(N_1)_{60}$) and shear wave velocity has been used to estimate and represent the lower- and upper-bound soil states.

Vs Approach in Determining CRR

The procedure established by Andrus and Stokoe (2000) has been used in this study. Further studies have been conducted by other researchers, especially with respect to probabilistic liquefaction resistance evaluation using Vs; however, at the moment, those studies are not incorporated in the Youd (2001) mean value curve and for the purposes of this particular study, however limited they are, the empirical relationships presented by Andrus and Stokoe (2000) are considered sufficient.

Shear-wave velocities can be measured in the field by several seismic tests including cross hole, downhole, seismic cone penetrometer, suspension logger, and the spectral-analysis-of-surface-waves techniques. CRR is given as:

$$CRR = \left\{ a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*}\right) \right\} MSF \qquad (3)$$

where $V_{s1}=$ overburden stress-corrected shear wave velocity; $V_{s1}^*=$ limiting upper value of V_{s1} for cyclic liquefaction occurrence; a and b = curve fitting parameters (0.022 and 2.8, respectively); and MSF = magnitude scaling factor to account for the effect of earthquake magnitude. The magnitude scaling factor is traditionally applied to CRR, rather than the cyclic loading parameter CSR, and equals 1.0 for earthquakes with a magnitude of 7.5. In Andrus and Stokoe (2000) revised average r_d values by Idriss (1999) are presented along with the Seed and Idriss average r_d curve and presented in Fig. 2 here. Unlike the original r_d values, these revised r_d values are magnitude dependent. As shown in Fig. 2, the revised r_d curve for moment magnitude of 7.5 is almost identical to the average curve published by Seed and Idriss (1971).

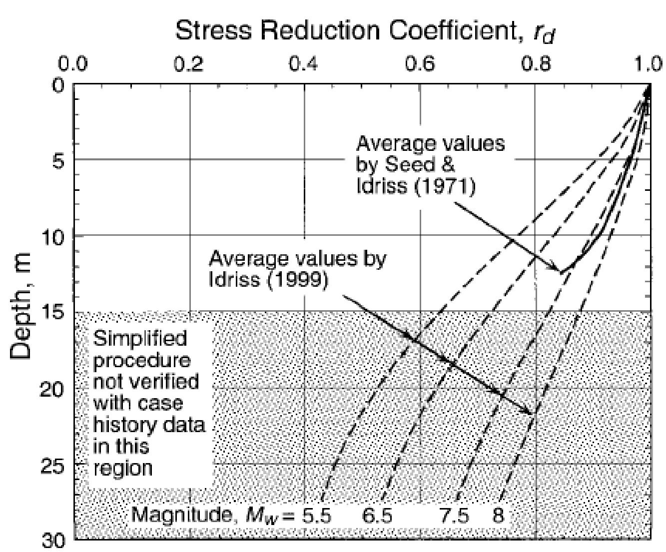

Fig. 2. Average r_d versus depth curves developed by Seed and Idriss (1971) and Idriss (1999) for various earthquake magnitudes (Andrus and Stokoe, 2000).

In Andrus and Stokoe (2000), the relationship between the limiting shear wave velocity, V_{s1}^*, and fines content is expressed by

$$V_{s1}^* = 215 \text{ m/s, for sands with } FC \le 5\% \qquad (4a)$$

$$V_{s1}^* = 215\text{-}0.5(FC\text{-}5) \text{ m/s, for sands with } 5\% < FC < 35 \qquad (4b)$$

$$V_{s1}^* = 200 \text{ m/s, for sands and silts with } FC \ge 35\% \qquad (4c)$$

where FC = average fines content in percent by mass.

Overburden stress-corrected shear wave velocity is calculated as follow.

$$V_{s1} = V_s \left(P_a / \sigma_v'\right)^{0.25} \qquad (5)$$

where V_s = measured shear wave velocity, P_a = reference stress of 100 kPa or about atmospheric pressure; and σ_v' = initial effective overburden stress (kPa).

<u>SPT Approach in Determining CRR</u>

Based on Youd et al. (2001), CRR may be calculated as follows:

$$CRR = \left\{ \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{\left[10(N_1)_{60}+45\right]^2} - \frac{1}{200} \right\} MSF \qquad (6)$$

where $(N_1)_{60}$ is the energy corrected SPT N value.

Normally, when the SPT based simplified liquefaction procedure is followed, there are various corrections involved to estimate the $(N_1)_{60}$. In addition to those corrections, another correction for fines in sand is applied to obtain the "clean sand" $(N_1)_{60cs}$ value and the $(N_1)_{60cs}$ is used in equation (6). However, in this study, fine content is less than 5%, therefore there is no need for fine correction, and since $(N_1)_{60}$ is obtained through a V_s correlation, the other corrections are not used. Thus, the whole SPT procedure is not explained here. The interested reader can find the detailed information in NCEER (1997) and Youd et al. (2001). The correlation for $(N_1)_{60}$ is provided below.

$$(N_1)_{60} = \left(V_{s1} / B_1\right)^{1/B_2} \qquad (7)$$

where B_1 = 93.2 ± 6.5 and B_2 = 0.231 ± 0.022 for soils with fines content <10% and with V_{s1} in meters per second and $(N_1)_{60}$ in blows/0.3 m (blows/ 1 ft).

$(N_1)_{60}$ has been calculated as described above because, when the lower-, upper-, and best estimate maximum ground accelerations from one dimensional analyses – as explained in the following section – are desired to be incorporated in the liquefaction analyses, proper adjustments to the soil properties are also necessary. In one dimensional site response analyses, the shear wave velocity or shear modulus is used to vary the soil strength. Therefore, a solution that involved shear wave velocity to obtain modified $(N_1)_{60}$ values seemed reasonable to solve this problem.

ONE DIMENSIONAL GROUND RESPONSE ANALYSIS

The objective of a one dimensional (1D) ground response analysis is to obtain the ground surface motions at a site and to obtain the strain-compatible soil properties of the soils beneath the site for use in soil-structure interaction analyses. This analysis sometimes can be used to determine the maximum ground acceleration to be used in site specific liquefaction analysis.

Vertically propagating shear waves often are the dominant contributors to free-field ground motions at a site; therefore, a one-dimensional equivalent-linear analysis is usually accepted as appropriate (RG 1.208, 2007). Proshake is a computer program for seismic ground response analysis of horizontally layered soil deposits. In this program, the geo-material mass is represented by a 1D soil column, and the soil nonlinear and hysteretic stress-strain characteristics are simulated by an equivalent shear modulus and an equivalent viscous damping factor both of which vary with strain level through iteration.

An effective shear strain representative of the average shear strain level of each layer during the earthquake is computed based on the assumption that it is equal to 65 percent of the peak shear strain value. The dynamic shear modulus and damping to be used for each layer in the next iteration are evaluated based on their compatibility with the effective shear strain in the layer. The variation of shear modulus (normalized with respect to the low-strain value) as a function of effective shear strain is defined by the shear modulus degradation curve for each soil and rock type. The variation of damping ratio with effective shear strain is defined by the damping ratio curve for each soil and rock type.

Dealing with Material Uncertainties

In accordance with recommendations in ASCE 4-98 (ASCE, 2000) for performance of soil-structure interaction (SSI) analyses, upper- and lower-bound values of dynamic soil properties are also required. Therefore, upper- and lower-bound low-strain shear modulus and shear wave velocity profiles were prepared for the site response analyses, which provide the required ranges of soil dynamic properties for SSI analyses. According to Section 3.3.1.7 in ASCE 4-98:

"Low strain soil shear modulus shall be varied between the best estimate value times (1 + C_v) and the best estimate value divided by (1 + C_v), where C_v is a factor that accounts for uncertainties in the SSI analysis and soil properties. If sufficient, adequate soil investigation data are available, the mean and standard deviation of the low strain shear moduli shall be established for every soil layer. The C_v shall then be established so that it will cover the mean plus or minus one standard deviation for every layer. The minimum value of C_v shall be 0.5. When insufficient data are available to address uncertainties in soil properties, C_v shall be taken as no less than 1.0."

These requirements indicate that:

$$G_{UB} = G_{BE} \times (1 + C_v) \qquad \text{Eq. (8a)}$$

$$G_{LB} = G_{BE} / (1 + C_v) \qquad \text{Eq. (8b)}$$

where G_{UB}= Upper-bound low strain shear moduli, G_{BE} = Best-estimate shear moduli, G_{LB}= Lower-bound low strain shear moduli. The relationship between G and V_s is given as follows:

$$V_s = (G / \rho)^{0.5} \qquad \text{Eq. (9)}$$

where V_s = low-strain shear wave velocity, G = low-strain shear modulus, and ρ = soil mass density.

CURRENT STUDY

Soil Profile

An idealized soil profile consisting of 40-ft deep liquefiable clean sand stratum (FC<5%) and a rock layer beneath the sand stratum has been selected at a hypothetical site. Different responses of different soil properties to the same motions are realized by the author; however, in order to reduce the number of variables, only one soil profile has been used in the current study. Assuming that statistically enough shear wave velocity measurements were done at the hypothetical site and based on the explanation given in the previous section, the lower- and upper-bound properties have been obtained assuming that C_v=0.5. Low strain shear wave velocities and shear moduli are presented in Table 1. The unit weight of the sand and the bedrock have been assumed 120 psf and 140 psf, respectively. Depth to groundwater has been assumed 5 ft.

Earthquake Data

Earthquake time histories are real earthquake records from the Kocaeli (1999) earthquake and they have been obtained from the Pacific Earthquake Engineering Research Center (PEER) online earthquake database. Recordings from four different stations (GBZ, GYN, IZN, IZT) have been selected. Two horizontal components of each earthquake have been included in the analysis. Therefore, a total of eight different time histories have been used. The peak ground accelerations of these histories have been scaled to

0.15g before assigning them as outcrop motions at the top of rock elevation.

An earthquake magnitude of 7.5 has been selected, because the starting point of this study was the Seed and Idriss (1971) r_d curves. The updated average r_d values that should be used in accordance with Andrus and Stokoe (2000) match with the original Seed and Idriss (1971) average r_d values for this magnitude. Therefore, the use of the original r_d values does not create an inconsistency.

Modulus Degradation and Damping Curves

Modulus degradation curves are used in ProShake to simulate the strain dependent reduction in the shear moduli of the soils. In the sand stratum, lower-bound, average, and upper-bound Seed and Idriss modulus degradation curves provided in ProShake have been used for lower-bound, best estimate, and upper-bound soil profiles, respectively. For the rock stratum, modulus degradation by Idriss as provided by ProShake has been used for all cases.

Damping curves are used to simulate the damping characteristics of soils with strain level. In the sand stratum, upper-bound, average, and lower-bound Seed and Idriss damping curves provided in ProShake have been selected. Use of upper-bound results in more damping with increased strain rate. For rock, the damping curve by Idriss as provided in ProShake has been used for all cases.

Table 1 Idealized soil profiles at a hypothetical site.

Stratum	Depth (ft)	Lower-Bound V_s (fps)	Lower-Bound G (psf)	Best-Estimate V_s (fps)	Best-Estimate G (psf)	Upper-Bound V_s (fps)	Upper-Bound G (psf)
Sand	5	408	622	500	932	613	1399
	10	408	622	500	932	613	1399
	15	408	622	500	932	613	1399
	20	408	622	500	932	613	1399
	25	408	622	500	932	613	1399
	30	408	622	500	932	613	1399
	35	408	622	500	932	613	1399
	40	408	622	500	932	613	1399
Bedrock	40 and below	1633	11599	2000	17405	2450	26119

Analyses

First, 1D analyses using ProShake software have been carried out to find the a_{max} at the ground surface for lower-, upper-bound, and best estimate soil profiles. Then the a_{max} values have been used in the liquefaction calculations. CSR profiles have also been estimated from the 1D analyses.

Liquefaction calculations have been carried out by 1) directly following the V_s based approach proposed by Andrus and Stokoe (2000) explained in the Simplified Procedure section; 2) using the CSR values obtained from the ProShake analyses in the V_s based approach; 3) following the SPT based approach except that the corrected N values have been obtained from a correlation with shear wave velocity given by Andrus and Stokoe (2000). In these analyses, the lower bound of the Seed and Idriss (1971) r_d curves has been used for the lower-bound soil profile, the average curve or the mean values of the r_d have been used for the best estimate soil profile, and the upper bound rd values have been used for the upper-bound profile. The approximations for the lower- and upper-bound Seed and Iddriss (1971) curves are provided below.

Lower-bound

$$r_d = 1.0 - 0.0133\, z \text{ for } z \leq 7.5 \text{ m} \quad (10a)$$

$$r_d = 0.9 - 0.0311\, (z-7.5) \text{ for } 7.5 \text{ m} < z \leq 12 \text{ m} \quad (10b)$$

$$r_d = 0.76 - 0.0483\, (z-12) \text{ for } 12 \text{ m} < z \leq 18 \text{ m} \quad (10c)$$

$$r_d = 0.47 - 0.0244\, (z-18) \text{ for } 18 \text{ m} < z \leq 22.5 \text{ m} \quad (10d)$$

$$r_d = 0.36 - 0.008\, (z-22.5) \text{ for } 22.5 \text{ m} < z \leq 30 \text{ m} \quad (10e)$$

$$r_d = 0.3 \text{ for } z > 30 \text{ m} \quad (10f)$$

where z is the depth measured from the level ground surface.

Upper-bound

$$r_d = 1.0 - 0.0044\, z \text{ for } z \leq 9 \text{ m} \quad (11a)$$

$$r_d = 0.96 - 0.013\, (z-9) \text{ for } 9 \text{ m} < z \leq 30 \text{ m} \quad (11b)$$

$$r_d = 0.7 \text{ for } z > 30 \text{ m} \quad (11c)$$

It should be realized however, that these lower- and upper-bound approximations may not correspond to the "lower- and upper-bound" soil profiles at hand. Some intermediate r_d might be more suitable depending on the soil properties. In actuality, it is almost an impossible task to determine the best r_d values. Therefore, a prudent approach is to use the average values.

RESULTS

The peak ground acceleration (PGA) profiles obtained from the ProShake analyses are presented in Fig. 3, Fig. 4, and Fig. 5 for lower-bound, best estimate, and upper-bound soil profiles, respectively. For the soil profiles and earthquake time histories studied here, the upper bound profile has produced the largest maximum ground surface accelerations. Table 2 shows the obtained average maximum surface accelerations corresponding to the individual earthquake time histories.

The CSR profiles obtained from the ProShake analyses are presented in Fig. 6, Fig. 7, and Fig. 8 for lower-bound, best estimate, and upper-bound soil profiles, respectively.

Comparison of factors of safety obtained by using the CSR from 1D ProShake analyses and CSR from Simplified Procedure is presented in Fig. 9. In this figure, non-liquefiable regions (when V_{s1}^{*} >215 m/s) were assigned a FS_L of 5 in order to indicate that the regions were considered in the study. Only within 12 to 20 ft depth region, shear wave velocity based calculation with lower-bound soil profile and a_{max} has given somewhat lower factors of safety than the factors of safety calculated with best estimate soil profile and best estimate a_{max}.

Fig. 9 indicates that the FS_L obtained by using the simplified method with the best estimate soil profile and the best estimate maximum ground acceleration provides a quite conservative approximation.

Table 2. Average a_{max} from lower-bound, best-estimate, and upper-bound 1D analyses.

Earthquake time history Station ID	**a_{max} (g)**		
	LB	**BE**	**UB**
GBZ000	0.1169	0.1663	0.2123
GBZ270	0.1088	0.1578	0.2931
GYN000	0.1145	0.1716	0.2453
GYN090	0.0964	0.1662	0.2707
IZN090	0.1270	0.2289	0.2272
IZN180	0.1261	0.1793	0.2678
IZT090	0.1146	0.1638	0.2496
IZT180	0.1146	0.1908	0.2310
Average	***0.12***	***0.18***	***0.25***

CONCLUSIONS

When a site specific one dimensional ground response study is carried out at a site with lower-bound, best estimate, and upper-bound soil profile properties, a different a_{max} at the ground surface is obtained for each profile. Practitioners sometimes have a dilemma in deciding which one of the maximum surface accelerations they should use in their liquefaction analyses. Before deciding on using the greatest of the three different maximum ground acceleration numbers, first, the soil conditions in which those surface acceleration values obtained should be considered. Then, it should be kept in mind that the simplified liquefaction procedure describes the loading as equivalent cyclic stresses induced by an earthquake and CSR is calculated based on mean values of the depth dependent stress reduction factor (r_d). In reality, there is a range for this number. The uncertainty of CSR increases with depth when mean r_d values are used to simplify calculations.

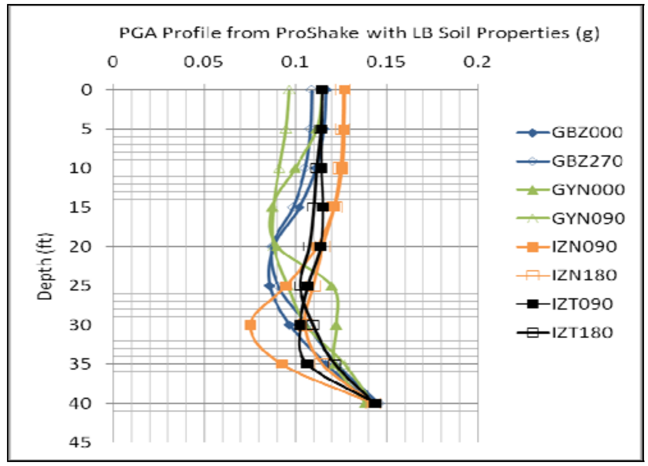

Fig. 3. Peak ground acceleration profiles for the earthquake time histories using the lower-bound soil profile.

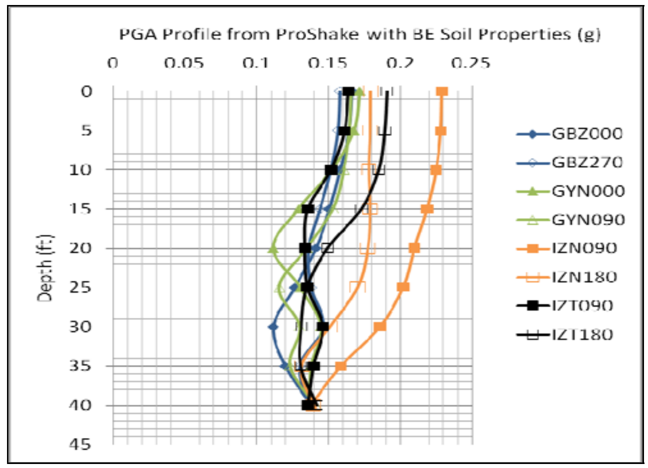

Fig. 4. Peak ground acceleration profiles for the earthquake time histories using the best estimate soil profile.

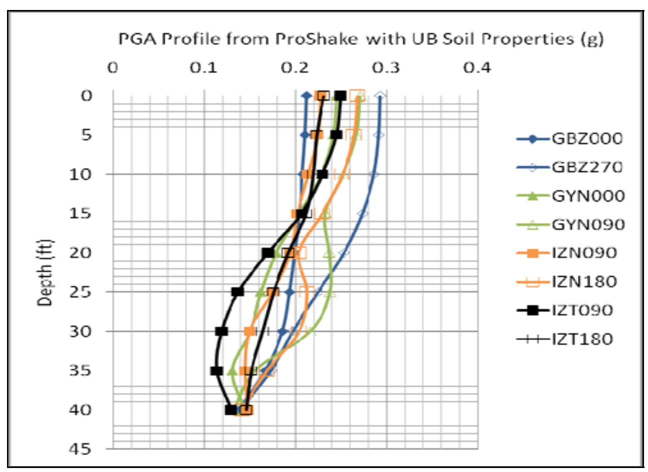

Fig. 5. Peak ground acceleration profiles for the earthquake time histories using the upper-bound soil profile.

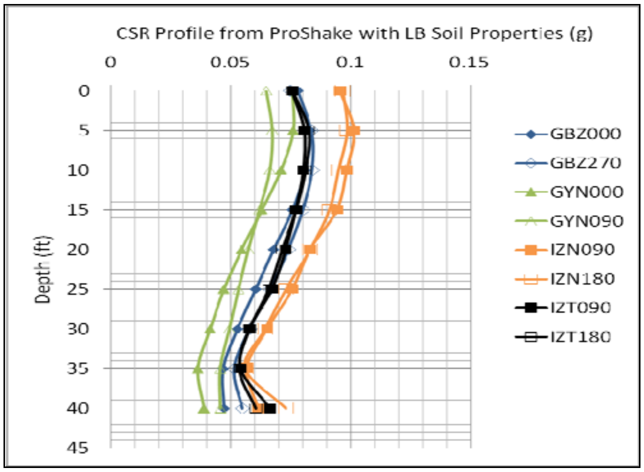

Fig. 6. Cyclic stress ratio profiles for the earthquake time histories using the lower-bound soil profile.

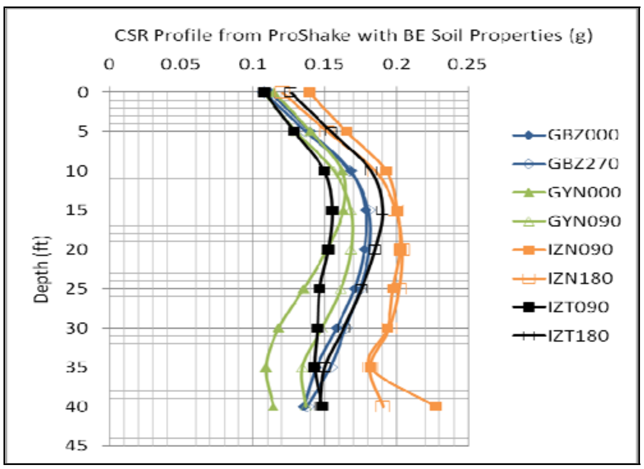

Fig. 7. Cyclic stress ratio profiles for the earthquake time histories using the best estimate soil profile.

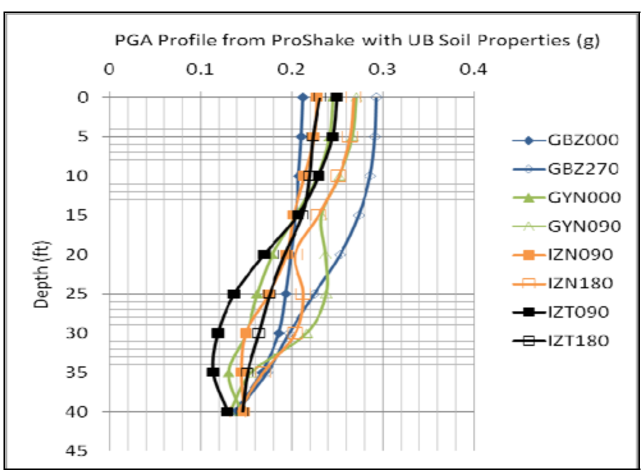

Fig. 8. Cyclic stress ratio profiles for the earthquake time histories using the upper-bound soil profile.

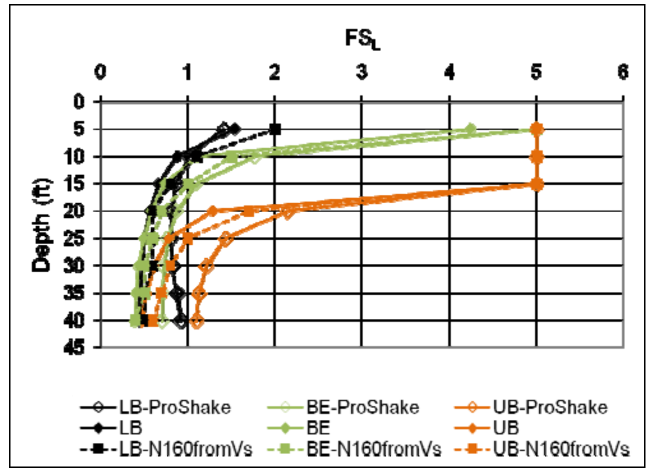

Fig. 9. Factors of safety against liquefaction calculated using empirical relationships with V_s and using CSR estimated by ProShake analyses.

It should be realized that, although in the calculations presented here lower- and upper-bound r_d factors have been incorporated, these lower- and upper-bound approximations may not correspond to the "lower- and upper-bound" soil profiles at hand. Some intermediate r_d might be more suitable depending on the soil properties. In actuality, it is almost an impossible task to determine the best r_d values. Therefore, it is really a good idea to use the mean r_d values, regardless of the soil profile used.

For the cases analyzed here, the simplified approach is almost always more conservative than using the CSR values from ProShake; therefore, for non-critical projects, if the method chosen for liquefaction analysis is the simplified method, using an a_{max} from the 1D analysis with the best estimate soil profile is considered accurate. Thus, in general, when the simplified procedure is used for the liquefaction analysis, using the best estimate a_{max} is considered appropriate because of the use of the average reduction factor r_d and the average soil properties.

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