COMPARISON OF PGA DETERMINATION METHODS FOR LIQUEFACTION ANALYSIS

Fu-Kuo Huang [1] and Grace S. Wang [2]

ABSTRACT

Seismic-induced soil liquefaction causes damages at soft soil sites. Therefore, it is an important issue to determine peak ground acceleration (PGA) appropriately for liquefaction analysis considering site effects. These subjects are examined in this paper in detail, including PGA attenuation law, empirical relationships and amplification coefficients in the seismic design specifications, and site-specific response analysis. Especially, a frequency-dependent equivalent linearized technique, FDEL, of site-specific dynamic ground response analysis is considered. A case study of Chianan Plain in Taiwan is implemented with the aim to explore the difference of liquefaction potential evaluated by different PGA determination methods. PGA and liquefaction potential estimated from Design Earthquake and Maximum Considered Earthquake (MCE) according to newly-revised building specifications in Taiwan are compared to those from seismic hazard analysis (SHA) and ground response analysis (GRA). It is shown that site effects estimated from codes may be un-conservative or over-conservative, which will depend on seismic intensity at reference-base outcropping. Accordingly, performing a site-specific GRA accompanied with SHA is the best way to determine PGA on the surface at soft sites for liquefaction analysis.

KEY WORDS

soil liquefaction, site effects, design earthquake, maximum considered earthquake.

INTRODUCTION

Over the past several decades, seismic-induced soil liquefaction has become an important topic in aseismic design. The procedure for evaluating liquefaction potential of soils widely used throughout much of the world is so-called *simplified procedure* (Youd et al. 2001). One way to quantify the potential for liquefaction is in terms of factor of safety; a second way is by probability. Factor of safety, *Fs*, is defined by the ratio of cyclic resistance ratio (CRR), the capacity of the soil to resist liquefaction, to the cyclic stress ratio (CSR), the seismic demand on a soil layer. Yet liquefaction probability is a function of CRR and CSR. Cyclic resistance ratio is usually represented by some index of in situ test, including blow counts *N* of the Standard Penetration Test (SPT), tip resistance q_c and friction ratio f_s of the Cone Penetration Test (CPT), and velocity *Vs* of small-strain shear-wave velocity measurements, etc. On the other hand, cyclic stress ratio is determined by peak ground acceleration (PGA) at soft sites.

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There are many researches on the study of cyclic resistance ratio according to different index of field tests, and thus several different discriminant criteria for triggering liquefaction based on these index have been developed, e.g. Seed et al. (1985), and Robertson and Wride (1998), etc. Nevertheless, these criteria are all using the same formula of cyclic stress ratio suggested by Seed et al. (1985). The most important parameter in the CSR formula suggested by Seed et al. (1985) is peak ground acceleration (PGA) at soft sites. It is a challenging work to determine an appropriate PGA value for liquefaction evaluation because of sites effects. Seldom study is explored to compare the differences of liquefaction potential evaluated from different PGA determination methods. In engineering practice, a little difference of requirement of factor of safety against liquefaction, e.g. 1.00 vs. 1.05, or probability of liquefaction is examined precisely for the reason of economical consideration. Therefore, it is an important topic to understand the relative conservatism of the effects of PGA determination methods on the liquefaction potential.

This paper will first discuss three common used methods for determination of peak ground acceleration (PGA) at soft sites. Following, a frequency-dependent equivalent linearized technique, FDEL, of site-specific dynamic ground response analysis is introduced. And then, a case study of Chianan Plain in Taiwan is implemented. PGA and liquefaction potential evaluated from Design Earthquake and Maximum Considered Earthquake (MCE) according to newly-revised building codes in Taiwan are compared to those from seismic hazard analysis (SHA) and ground response analysis (GRA) in detail. Finally, summaries and some suggestions are given.

METHODS FOR DETERMINING PEAK GROUND ACCELERATION

In the summary report written by Youd et al. (2001): “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”, it lists the following methods for determining peak ground acceleration, a_{max}, at potentially liquefiable sites, in order of preference:

(1) Using empirical attenuation correlations of a_{max} with earthquake magnitude, distance from the seismic energy source, and local site conditions.
(2) Performing local site response analyses for soft sites and soil profiles that are not compatible with available attenuation relationships. A suite of plausible earthquake records from earthquakes with similar magnitudes, source distances, etc. should be used in the analysis.
(3) Using empirical amplification ratios, such as those developed by Idriss (1990) and Seed et al. (1994). This method use a multiplier or ratio by which bedrock outcrop motions are amplified to estimate surface motions at soil sites.

When examine the above three methods, although using empirical attenuation correlations is the first priority, well accepted attenuation relationships at soft soils is seldom found up to now in Taiwan. Thus the first method is impractical in engineering practice. The second choice to estimate a_{max} at soil sites is to perform local site response analyses using computer programs such as SHAKE (Schnabel et al. 1972). But it needs sufficient soil profile data to set up the soil model, including the dynamic characteristics of the soils. In general, not every site can meet the requirements for performing site response analyses. Some assumptions and empirical curves that describe the variation of dynamic characteristics of soils with shear strain (e.g., shear modulus reduction curves, and damping ratio curves) are usually adopted by engineering judgment. In view of the complication and cost consideration, site response analyses are usually performed only for critical projects. The third method for estimating a_{max} is using empirical amplification ratios, which is always in term of site coefficients for corresponding site class in structural provisions or codes according to research results, such as those developed by Idriss (1990) and Seed et al. (1994). Because of its convenience, the final method is the most popular choice among those mentioned above for estimating a_{max} in engineering practice. But the site coefficients in provisions or codes may be too simplified to reflect the site effects. Engineers must pay attention to it when using this method.

FREQUENCY-DEPENDENT EQUIVALENT LINEARIZED TECHNIQUE FOR GROUND RESPONSE ANALYSIS

If there are sufficient soil profile data, performing ground response analyses (GRA) is the direct way to estimate a_{max} for soft sites. Computer program such as SHAKE (Schnabel et al. 1972), based on the multi-reflection theory and taking the equivalent linearized technique to account for the non-linear characteristics of soils, is the most common used software for analysis of horizontally layered ground. If bedrock under the soft site can be defined definitely, a_{max} at soft site surface (PGA_S) can be obtained through convolution analysis by SHAKE easily. But it is not the case for Chianan Plain in Taiwan, where the thickness of alluvium is very deep so that the depth of bedrock of the soft sites is generally unknown. Under this situation, de-convolution analyses are required first to obtain the reference-base motion. And then, convolution analyses are followed to estimate a_{max} at soft site surface. The procedures of ground response analysis used in this study are shown in Figure 1 as below:

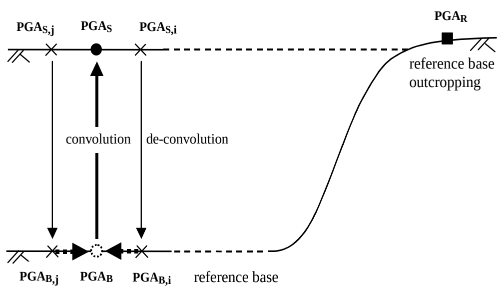

Figure 1: Procedures of Ground Response Analysis

Unfortunately, de-convolution analysis in SHAKE may cause a numerical problem, as a result of damping ratio of high frequency component is estimated exceedingly large, in the case of soft and deep sites when strain level is large. A cut-off frequency, f_c, is adopted to overcome the problem. But no rule can be followed about what value of f_c is appropriate. In order to resolve this problem, Furumoto et al. (2000) introduce a frequency-dependent equivalent linearized technique, FDEL, for site-specific dynamic ground response analysis. FDEL technique can resolves the unrealistic amplification effects or divergence phenomena over the high frequency range efficiently when using SHAKE model for de-convolution analysis on soft and deep sites (Huang and Wang 2005). It provides a feasible technique to perform ground response analyses and statistical analyses directly for evaluating ground motions at soft sites using geological data and neighboring earthquake records.

FDEL technique assumes that the shear modulus and damping ratio of soils, been modeled as a function of equivalent mean shear strain in SHAKE, are frequency-dependent. Thus the corresponding equivalent strain is also frequency-dependent, and is defined by the following equation:

$$\gamma_f(\omega) = C\gamma_{\max}\frac{F_\gamma(\omega)}{F_{\gamma_{\max}}} \tag{1}$$

where C = constant, $\gamma_{\max}$ = maximum shear strain, $F_\gamma(\omega)$ = Fourier spectrum of shear strain, and $F_{\gamma_{\max}}$ represents the maximum of $F_\gamma(\omega)$. It is shown that the equivalent strain $\gamma_f(\omega)$, which controls equivalent shear modulus and damping ratio, is given in proportional to the spectral amplitude of shear strain in frequency domain. The constant C controls the level of equivalent strain uniformly along the frequency axis. The conditions of $F_\gamma(\omega)/F_{\gamma_{\max}}$ =1.0 and C =0.65 give the same situations as SHAKE program. The constant C in this study is given as a function of earthquake magnitude, M, by the following equation according to suggestion of Idriss & Sun(1992). It differs from Furumoto et al. (2000), and some modifications are made:

$$C = \frac{M-1}{10} \tag{2}$$

The numerical calculations of equivalent linearized technique in FDEL are also an iterative process like SHAKE. Iterations are carried out by comparing the equivalent shear modulus and damping ratio for each layer, corresponding to equivalent strain given by equation (1), with that given in the previous calculation until the deviation in the two consecutive value is converged into some given level, e.g., 5%. In FDEL, the convergence judgment is performed individually in three frequency regions: (a) low frequency region (1 Hz or lower), (b) middle frequency region (1 to 5 Hz), and (3) high frequency region (5 Hz or higher). The average of deviation in each frequency region is calculated and compared with previous value for each layer. The case of response analyses in the following, the numbers of iterations in FDEL are generally below 10 times, which is similar to those of SHAKE.

CASE STUDY

To illustrate the methods for determining peak ground acceleration, and its influence on liquefaction potential, a case study of Chianan Plain in Taiwan is implemented.

SITE CONDITION

The test site located at Chianan Plain is near a river. The ground water table at the time of the field investigations was located at a depth of about 1.6m. The majority of the layers are alluvium and the thickness of the alluvium is very large, where the depth of bedrock is unknown. As shown in Figure 2(a), the simplified profile of the upper 90 m of soil at the site consists of clays with plastic index (PI) less than 20, silty sands, or non-plastic silts.

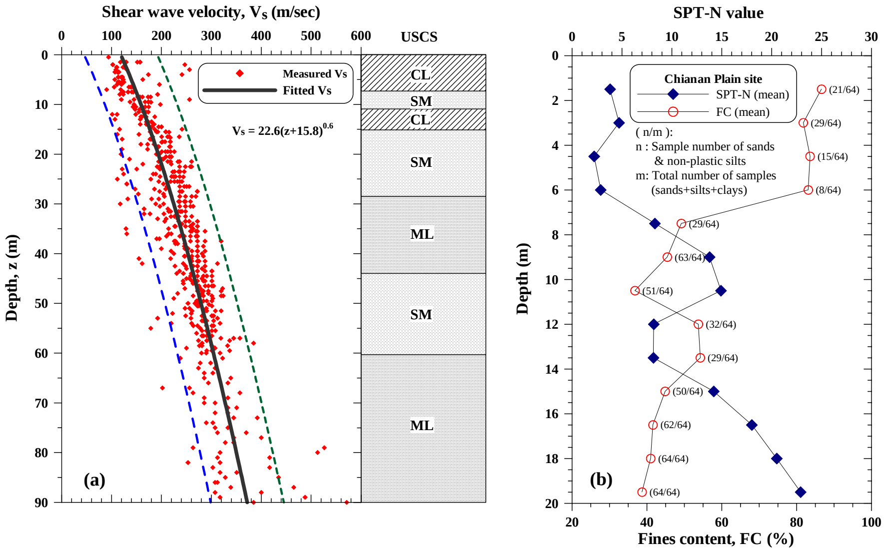

Figure 2: (a) Shear Wave Velocity and Simplified Soil Profile; and (b) SPT-N Values and Fines Contents (FC) at Test Site

There are 9 in-situ seismic tests, including 5 of SCPT testing, 2 of cross-hole testing, and 2 of suspension logger testing, in this site. The shear wave velocity profile is defined by the following equation through regression analysis:

$$Vs = 22.6(z + 15.8)^{0.6} \tag{3}$$

where Vs is the shear wave velocity in units of m/sec, and z is the depth below ground surface in units of meter. Shown in Figure 2(a), the values of Vs increase from about 120 m/sec at surface to about 370 m/sec at the depth of 90 m incrementally.

On the other hand, the total numbers of 64 boring holes were implemented in site investigation. Mean blow counts N of the standard penetration test and fines content of the upper 20 m of soil at the site are shown in Figure 2(b). The soils at depths in the range of 8.0 m - 10.9 m and 15.1 m - 20 m are mostly the silty sands, or non-plastic silts with mean SPT-N values less than 30, below the bound that may occur liquefaction according to Youd et al. (2001).

Ground Response Analysis

The frequency-dependent equivalent linearized technique, FDEL, is used to perform site-specific ground response analysis (GRA) here. FDEL model is the same as SHAKE model except that the equivalent shear strain is frequency-dependent defined by equation (1) and the convergence criteria of iterations are performed individually in three frequency regions. The profile of Figure 2(a) to depth of 90 m is used as the soil profile model in GRA. At the depth of 90 m, the Vs value is near the site condition, so-called ‘stiff soil’, that the seismic hazard analysis (SHA) is based in Taiwan currently. Thus, the depth of 90 m can be served as the reference base in this study. Variations of shear modulus and damping ratio of soils with shear strain are defined according to dynamic test results. Vs defined by equation (3), together with these modulus reduction curves and damping curves, are used to represent the dynamic characteristics of the soil at this site.

The procedures of GRA in Figure 1 are followed to perform ground response analysis. Acceleration time histories that are representative of horizontal surface motions near the site are prepared as input to the soil model. There are 3 seismological stations, whose site condition is similar to the one of the site under study, in the neighborhood of the Chianan Plain site at a distance less than 5 km. A total set of 27 acceleration time histories, with EW and NS components for each set, are selected. The earthquake magnitude of those records is between 5.03 and 7.30, and the peak ground acceleration (PGA) is in the range of 0.02g to 0.30g, including small to large, shallow to deep, and near to far earthquake. These accelerations are input to the surface of the soil model, de-convolution analyses using FDEL technique are performed to obtain the responses at depth of the reference base. And then, each response acceleration is scaled to a PGA = 0.05g, 0.1g, 0.2g, 0.3g, 0.4g, 0.5g, 0.6g, and 0.7g, respectively. A convolution analysis is followed to get the surface response, and the site effect is estimated. The soil amplification of PGA between the surface at the site and the reference base could be evaluated through statistical analysis easily. Assumed that the ratio of PGA at reference base to those at reference-base outcropping is 0.8 according to the site condition and the researches by Chen (1995), a median relationship of the site amplification coefficient, F_A, with reference-base outcropping peak ground acceleration, PGA_R, is obtained shown as the thick dash line in Figure 3, and could be expressed as:

$$F_A = 0.124 + \frac{0.402}{PGA_R + 0.117} \tag{4}$$

From Figure 3, it is apparent that the critical intensity of PGA_R is about 0.34g. That is, PGA_R = 0.34 g is corresponding to F_A =1.0, and the smaller the PGA_R, the larger the F_A is.

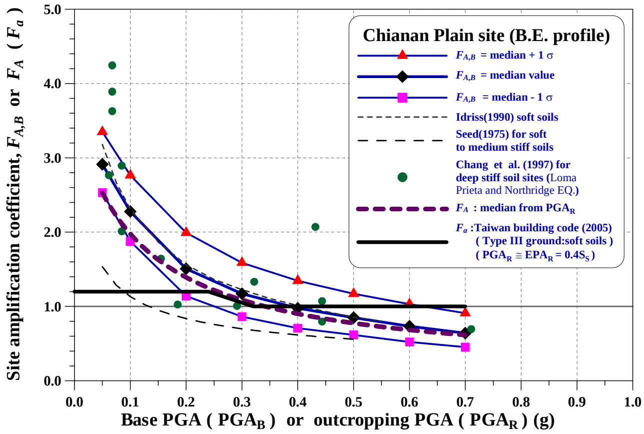

Figure 3: Amplification Coefficients $F_{A,B}$ or F_A (F_a) with Respect to PGA_B or PGA_R of Reference Base or Reference-base outcropping

According to the seismic hazard analysis (SHA), peak ground acceleration at the reference-base outcropping, PGA_R, is 0.275g and 0.395g, where F_A value is 1.150 and 0.909 shown in Figure 3, corresponding to return period *Tr* =475 years and 2500 years, respectively.

Assumed that the important factor I of facilities located at the site is 1.5, the peak ground acceleration for liquefaction analysis on this soft site, represented by A here, is equal to 0.474g and 0.539g, respectively. The associated parameters in above calculations are shown in Table 1, where A values will be used to the following liquefaction analysis for this site.

Table 1: Peak Ground Acceleration Determined by Seismic Hazard Analysis (SHA) and Ground Response Analysis (GRA)

Parameter / Return Period	PGA_R (g)	F_A	PGA_S (g) ($=F_A \cdot PGA_R$)	I	$A=PGA_S I$ (g)
475 year	0.275	1.150	0.316	1.5	0.474
2500 year	0.395	0.909	0.359	1.5	0.539

PGA FROM TAIWAN BUILDING SPECIFICATIONS

The newly-revised version of building specifications in Taiwan stipulates that the liquefaction potential of sites located at sandy soils shall be evaluated based on three PGA values (represented by A) of different level earthquake, where A=0.4$S_{DS}I$g and A=0.4$S_{MS}I$g are used for Design Earthquake (DE) and Maximum Considered Earthquake (MCE) that corresponding to return period Tr =475 years and 2500 years, respectively. The symbol I is also the important factor of the facilities located at the site. S_{DS} and S_{MS} represent the horizontally spectral acceleration coefficients, in units of g, at short period and can be expressed as below:

$$S_{DS} = F_a S_S^D \tag{5a}$$

$$S_{MS} = F_a S_S^M \tag{5b}$$

where F_a, shown in Table 2, is the site coefficient that depends on site class defined by mean shear wave velocity $\overline{V_S}$ in the upper 30 m of the site profile. S_S^D and S_S^M are the mapped, 5-percent-damped, horizontally spectral acceleration parameter at short period (0.3 sec) corresponding to DE and MCE, respectively, for sites located at stiff soils.

Table 2: Values of Site Coefficient F_a in Taiwan (Ministry of the Interior 2005)

Site Class	$\overline{V_S}$ (m/sec)	Mapped Horizontally Spectral Acceleration Parameter at Short Period S_S (S_S^D or S_S^M)				
		$S_S \le 0.5$	$S_S = 0.6$	$S_S = 0.7$	$S_S = 0.8$	$S_S \ge 0.9$
Type I (stiff soil)	$\overline{V_S} > 360$	1.0	1.0	1.0	1.0	1.0
Type II (medium stiff soil)	$180 \le \overline{V_S} \le 360$	1.1	1.1	1.0	1.0	1.0
Type III (soft soil)	$\overline{V_S} < 180$	1.2	1.2	1.1	1.0	1.0

According to the shear wave velocity profile shown in Figure 2, mean shear wave velocity $\overline{V_S}$ in the upper 30 m of the site profile is 172 m/sec. Thus the site belongs to Type III site class in Table 2, i.e., the site is a soft soil site. The requirements of S_S^D and S_S^M in

Specifications at this site are 0.7 and 0.9, respectively. Therefore, we can find site coefficient values F_a of 1.1 and 1.0 from Table 2 at once. Also assumed that the important factor I is 1.5, the peak ground acceleration for liquefaction analysis on this soft site, A, is equal to 0.462g and 0.540g corresponding to DE and MCE, respectively. The associated parameters to obtain A values are shown in Table 3. Comparing Table 1 with Table 3, we can find that A value from *Specifications* for Design Earthquake is less than that from SHA+GRA for earthquake with return period Tr =475 years; however, the trend is conversely for Maximum Considered Earthquake to earthquake with return period Tr =2500 years.

Table 3: Peak Ground Acceleration Determined by Seismic Design Specifications

Parameter	S_S^D	F_a	$S_{DS}\left(=F_a S_S^D\right)$	$0.4S_{DS}$	I	Design Earthquake (Tr = 475 yr.)
						$A=0.4S_{DS}I\ (g)$
	0.7	1.1	0.77	0.308	1.5	0.462

Parameter	S_S^M	F_a	$S_{MS}\left(=F_a S_S^M\right)$	$0.4S_{MS}$	I	Max. Considered Earthquake (Tr = 2500 yr.)
						$A=0.4S_{MS}I\ (g)$
	0.9	1.0	0.90	0.360	1.5	0.540

EVALUATION OF LIQUEFACTION POTENTIAL

Because of probability of liquefaction, P_L, is required information for making risk-based design decisions, the following P_L expression derived by Huang (2004) using logistic regression is used to evaluate the liquefaction potential in this study:

$$P_L=\frac{1}{1+\exp[-(10.097-0.245(N_1)_{60cs}+3.757\ln(CSRN))]} \tag{6a}$$

where

$$CSRN=CSR/MSF=0.65\cdot\left(\frac{A}{g}\right)\cdot\left(\frac{\sigma_v}{\sigma_v^{'}}\right)\cdot r_d \Bigg/ \left(\frac{M_w}{7.5}\right)^{-2.56} \tag{6b}$$

in which, $(N_1)_{60cs}$ is the clean sand equivalence of the corrected SPT blow count defined in Youd et al. (2001). *CSRN* is the normalized cyclic stress ratio (*CSR*) shown in Equation (6b) where *MSF*=magnitude scaling factor; *A*=peak horizontal ground surface acceleration; *g*=acceleration due to gravity; σ_v =total vertical stress in question; $\sigma_v^{'}$ =effective vertical stress at the same depth; r_d =stress reduction factor defined in Youd et al. (2001); and M_w =moment magnitude.

In order to represent the overall liquefaction potential in the upper 20 m of the site profile, the liquefaction probability index, P_W, is defined as below:

$$P_W=\int_0^{20}P_L(z)W(z)dz \Big/ \int_0^{20}W(z)dz \tag{7}$$

where $P_L(z)$ is the P_L value at the depth of z, and the weighting function $W(z)$=10-0.5z.

Based on the PGA values determined from SHA+GRA and building specifications, and the boring data of site investigation, liquefaction probability P_L and liquefaction probability index P_W of this site are evaluated for all 64 boreholes. The mean probability of liquefaction in the upper 20 m of the site profile, and the mean liquefaction probability index for 64 boreholes are shown in Figure 4. It can be seen that the mean P_L for every depth of the site are almost larger than 0.4 except that at depths between 4 m and 6 m, where clayey soils are in the majority in these depth range. The depths at 8 m to 10 m are considered susceptible to liquefaction in particular due to high P_L value. From Figure 4(b), it can also be learned that the site has medium possibility to occur liquefaction, with P_W larger than 0.45. The P_W determined from Design Earthquake are a little un-conservative with respect to that from SHA+GRA, while P_W determined from Maximum Considered Earthquake are a little over-conservative with respect to that from SHA+GRA. Therefore, performing a site-specific GRA accompanied with SHA is the best way to determine PGA on soft ground for liquefaction analysis.

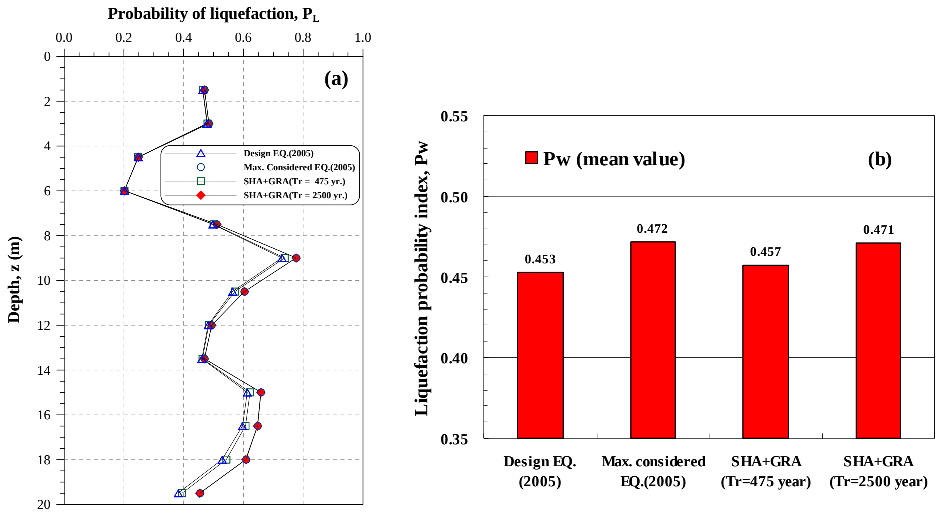

Figure 4: (a) Mean Probability of Liquefaction P_L; and (b) Liquefaction Probability Index P_W Corresponding to Design Earthquake, Maximum Considered Earthquake, 475 and 2500 Years Return Period Earthquake from SHA, Respectively.

CONCLUSIONS

1. The procedures proposed here for ground response analysis at deep soft site, where the depth of bedrock is unknown, is feasible to estimate site effects reasonably. The frequency-dependent equivalent linearized technique, FDEL, is effective that can resolves some numerical divergence phenomena for de-convolution analysis, and is recommended for using in engineering practice.
2. Based on case study in this paper, it is evidently that site effects estimated from codes or specifications may be un-conservative or over-conservative, which will depend on seismic intensity at reference-base outcropping. Accordingly, performing a site-specific GRA accompanied with SHA is the best way to determine PGA appropriately on the surface at soft sites for liquefaction analysis.

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