Assessments of Soil Parameter Reduction Coefficients from One-Dimensional Ground Response Analyses

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ABSTRACT: Soil parameter reduction coefficient (D_E) has been suggested by Architecture Institute of Japan (AIJ) and Japan Road Association (JRA) for over twenty years. The reduction coefficient denoted as D_E was used to reduce stiffness and/or strength parameters of the soil due to liquefaction caused by earthquakes. This study discusses the observations based on soil parameter reduction coefficients calculated from one-dimensional dynamic responses on artificial ground sites under horizontal earthquakes. A lumped mass model of horizontal sand layers is used. Soil liquefaction is modeled using the UBCSAND model. The factor of safety against liquefaction and the cyclic strength ratio for soils at various depths are calculated from the mechanical analyses to find the reduction factors based on the suggestions from JRA and AIJ. In addition, the ratio between the degraded shear modulus and the initial shear modulus of the soils are computed and compared to the reduction coefficients found varying the influence factors.

KEYWORDS: Liquefaction, Soil parameter reduction coefficient, Ground response Analysis, UBCSAND model.

1. INTRODUCTION

The soil parameter reduction coefficient, D_E has been suggested by Architecture Institute of Japan (AIJ, 1988) and Japan Road Association (JRA, 1996) to reduce the original stiffness and/or strength parameters of the soils due to soil liquefaction caused by earthquakes. Table 1 and Table 2 illustrate the reduction coefficients suggested by JRA and AIJ, respectively. These coefficients can be simply multiplied by the original soil impedance and/or resistance to estimate their residual values after soil liquefaction, the post liquefaction status is therefore inferred.

Table 1 Soil parameter reduction coefficients defined by JRA (1996)

Factor of safety against liquefaction, F_L	Depth of soil, z (m)	Soil parameter reduction coefficients, D_E	
		$R \leq 0.3$	$0.3 < R$
$F_L \leq 1/3$	$0 \leq z \leq 10$	0	1/6
	$10 < z \leq 20$	1/3	1/3
$1/3 < F_L \leq 2/3$	$0 \leq z \leq 10$	1/3	2/3
	$10 < z \leq 20$	2/3	2/3
$2/3 < F_L \leq 1.0$	$0 \leq z \leq 10$	2/3	1
	$10 < z \leq 20$	1	1

Table 2 Soil parameter reduction coefficients defined by AIJ (1988)

Factor of safety against liquefaction, F_L	Depth of soil, z (m)	Soil parameter reduction coefficients, D_E		
		Na ≤ 10	10 < Na ≤ 20	20 < Na
$F_L \leq 0.5$	$0 \leq z \leq 10$	0	0.05	0.1
	$10 < z \leq 20$	0	0.1	0.2
$0.5 < F_L \leq 0.75$	$0 \leq z \leq 10$	0	0.1	0.2
	$10 < z \leq 20$	0.05	0.2	0.5
$0.75 < F_L \leq 1.0$	$0 \leq z \leq 10$	0.05	0.2	0.5
	$10 < z \leq 20$	0.1	0.5	1.0

This paper examines the D_E values using a one-dimensional ground response analysis based on artificial soil profiles. A lumped mass model and UBCSAND Model (Byrne et al., 2004) were used to simulate the nonlinear free-field ground responses under horizontal earthquake motions. To show the influences of earthquake inputs, seismic record data of a number of recent major earthquakes were adopted and calibrated to target Peak Ground Acceleration (PGA) of interest. The influences of the soil stiffness and depth of the ground water table were also monitored. For the validations, ratios of the residual shear modulus of the soil to the original from the ground response analysis were firstly computed (*i.e.*, G/G_{max}). Additionally, the SPRC values were also attempted following the JRA and AIJ's suggestions. The liquefaction potential of the numerical models was also examined through the ground response analysis. Furthermore, the factor of safety F_L, defined as the ratio of the Cyclic Resistance Ratio (CRR, or R) and the Cyclic Stress Ratio (CSR or L), *i.e.* R/L was computed. With the known values of F_L, R, Na and depth z, the SPRC values were thus obtained. Finally, the obtained values were then compared and discussed. Preliminary investigation on the JRA coefficient from the ground response analysis can be found in Chang et al. (2018).

2. SEISMIC GROUND RESPONSE AND LIQUEFACTION

The ground response analysis conducted in this study is based on one-dimensional lumped mass model. The ground soils were characterized as sands by n layers with equal layer thickness h. The lumped mass of each layer can be computed as ρhA where ρ is the mass density of the layer and A the cross section of that layer. Stiffness elements of a single layer, k can be computed as $k = GA/h$ assuming the shear springs, where G is the shear modulus of the layered soil. With the Rayleigh damping model and assuming that the soil damping ratio is known, the equations of motion of the layered system can be solved easily using either an explicit or implicit time-integration method. Modal analysis can also be used to solve the equations. With the acceleration time-history data records, and $a(t)$ presumed at the bedrock, Eq. (1) presents the form of the governing equations for the seismic responses of a layered system, where A can be deleted from the solutions since it appeared in each term of the equation.

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = -[M]\{I\}a(t) \tag{1}$$

In Eq. (1), [M] is a diagonal matrix, [K] is a banded matrix with bandwidth of 3, and [C] is the Rayleigh damping coefficient matrix based on [K] and [M] and the assumed material damping ratio of the soil. $\{\ddot{U}\}$, $\{\dot{U}\}$ and $\{U\}$ are respectively the acceleration, velocity and displacement vectors of the soil to be solved, and {I} is the unit vector. The analysis has been used extensively to solve for the one-dimensional seismic responses of the free-field. Ignoring the differences between surface ground motion and bedrock motions, the ground responses are frequently predicted using seismic data recorded at the ground surface. For a ground site mainly consisting one-dimensional variations of the soils, the above analysis would definitely be applicable in engineering practice. Computer programs such as SHAKE (Schnabel et al., 1972) and its alternate versions are in the categories of such an analysis. The analysis which does not take into account the pore water pressure changes is the total stress analysis. Set of data on the shear modulus and damping of the soils with the variations of shear strains can be implemented in such analysis. If the excess pore water pressure was able to be simulated through rigorous constitutive models, then the analysis is called effective stress analysis. Cyclic 1D analysis (Elgamal et al., 1996) is the one following the effective stress principles.

In order to simulate the nonlinear responses of the layered system where the soils might be liquefied during the earthquake excitations, the one-dimensional analysis needs to be modified with a proper nonlinear soil model. Similar to the ground response analysis suggested by Finn et al. (1977), this study adopts the UBCSAND model suggested by Byrne et al. (2004) to simulate the soil nonlinearities resulting from the earthquake motions. Figure 1 illustrates the computation scheme for the UBCSAND model (Byrne et al., 2004) with the lumped mass analysis. Notice that such an analysis is mainly carried out by approximating the excessive pore pressure influences from calculating the plastic shear strains of the soils. The empirical formula suggested in the UBCSAND model (Byrne et al., 2004) and model parameters used in this study are listed in Table 3. Chang and Lin (2017) have adopted such a model to simulate the experimental data reported by Bay and Sancio (2006). Figure 2 shows the comparisons of the modeling analytically. Although the UBCSAND model is a simplified one-dimensional constitutive model, it can be found that the model behavior can provide rational estimations of the experimental data providing that the power parameters ne and m are varied during the cyclic shearing process.

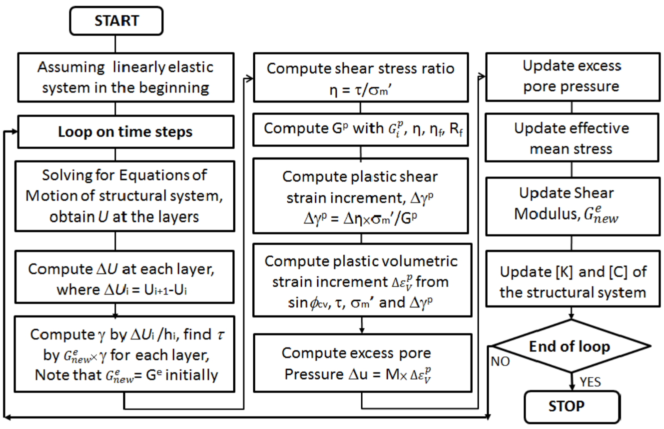

Figure 1 1D GRA using UBCSAND model (after Byrne et al., 2004)

3. CALCULATION OF REDUCTION COEFFICIENTS

From the above mechanical analysis, the shear modulus of the soils at any depth of the profile can be affected by excess pore pressure generations during the seismic excitations. For the convenience of numerical calculations, if the initial liquefaction occurred (which means that the pore pressure is equal to the mean total stress), the shear modulus of the soil would be reduced to 10% of its original value. At any time, ratios denoted by G/G_{max} of the residual value, G and the original shear modulus of the soils, G_{max} can be computed and taken as the reduction coefficients. Such method is called mechanical analysis in this paper

In conducting the liquefaction potential analysis, an alternative method (Method 2) can be achieved by using the suggested concept of the factor of safety F_L against liquefaction during the analysis. Following the estimation of averaged shear stress, the CSR herein is computed by $0.65\tau_{max}/\sigma_m'$, where τ_{max} is the maximum shear stress that occurred during the earthquake excitations, and σ_m' is the mean effective stress exerted upon the soil. To be consistent with the soil liquefaction measurements, CRR can be calculated considering the strength of soil at the time when the maximum pore pressure was generated. The Mohr-Coulomb failure criterion is used to compute the corresponding strength of the soil. The ratio of strength and mean effective stress then gives the value of CRR. From there, the factor of safety can be determined. Subsequently, knowing F_L, *R* and *z*, the correspondent D_E values can be found using Table 1 according to the JRA suggestion.

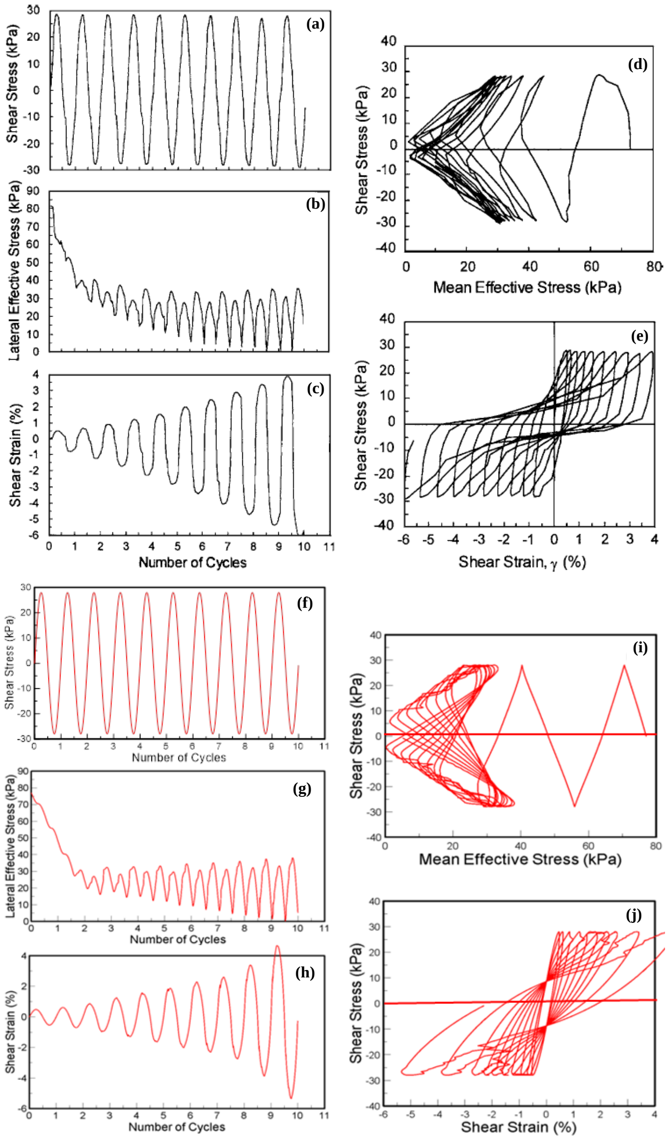

Figure 2 Analytical modeling using UBCSAND model: (a)-(e) experimental data from Bray and Sancio (2006) and (f)-(j) analytical modeling from Chang and Lin (2018)

Similarly, the D_E values following the suggestion of AIJ (Table 2) were also obtained in this study. With the assumption that fine content of the soil is trivial, the value of Na can be calculated by N_1, where the rod energy ratio is ignored temporarily and only the effect of depth is considered. Due to the ground response analysis was performed based on the depth-dependent shear modulus of the soil, the correlations suggested by Imai (1977) between the shear wave velocity (Vs) and SPT-N value (where $Vs = 80.6xN^{0.331}$) were adopted to compute the SPT-N values from the shear wave velocity of the soils. In addition, SPT-N values were calibrated to N_1 using the equation proposed by Liao and Whitman (1986).

The results from these methods are discussed and compared in this study to assess whether the soil parameter reduction coefficients suggested by the JRA and AIJ are agreeable with the ones directly found from the ground response analysis.

Table 3 Formulas of UBCSAND model (Byrne et al., 2004) and model parameters in use

Formulas	**Definition and value of model parameter**
G^e or $G^e_{new} = K^e_G Pa(\frac{\sigma m'}{Pa})^{ne}$	G^e: elastic shear modulus; G^e_{new} : updated shear modulus; K^e_G: elastic modulus constant (500-2000)*; Pa: atmospheric pressure; σ_m': mean effective stress; ne: model parameter (can be taken as 0.5)
$G^P_i \cong 3.7(Dr)^4\ G^e_{new} + Pa$	G^P_i: initial plastic shear modulus; *Dr*: relative density (33%, 45% and 54% were used)
$G^p = G^P_i(1 - \frac{\eta}{\eta f} R_f)^2$	G^p: plastic shear modulus; η_f: shear stress ratio at failure; R_f: calibration factor between 0.7-0.98 (averaged value is used)
$\Delta\varepsilon^p_V = (\sin\phi_{cv} - \tau/\sigma_m')\ \Delta\gamma^p$	$\Delta\varepsilon^p_V$: plastic volumetric strain; ϕ_{cv}: constant volumetric friction angle (between 30°-34° average is used); $\Delta\gamma^p$: plastic shear strain increment
$M = K_m Pa(\sigma_v'/Pa)^m$	M: constraint tangent modulus; K_m: material constant on constraint tangent modulus (taken as 1600); σ_v': effective vertical stress; m: model parameter (suggested as 0.5)

*Note: K^e_G is estimated using the formula $434\times(N_1)_{60}{}^{0.33}$ suggested by Beaty and Byrne (1998)

Table 4 Properties and parameters of soil with geometric information of soil profile

γ_d (kN/m^3)	γ_{sat} (kN/m^3)	Initial $(N_1)_{60}$	K^e_G	Vs (m/sec)	Damping ratio
19	21	5	741	116-239	5%
The thickness of the soil profile is 50 m, and GWT depth is 2 m					
Influence Factors	Assuming that initial $(N_1)_{60}$ = 5, 9 and 13 (where $(N_1)_{60}$ = 5 is the standard one); Water depth = 2 m, 7 m and 12 m (where 2 m is the standard one); PGA = 0.2g, 0.3g and 0.4g (where PGA 0.2g is standard); 1995 Kobe earthquake is the standard seismic record in use.				

4. NUMERICAL MODEL AND SEISMIC RECORDS

The numerical model of the ground site studied is characterized by sands only. The properties and parameters of the sandy soils, and the profile geometry of the model are listed below in Table 4. Note that the empirical formula suggested by Beaty and Byrne (1998) with regards to K^e_G based on an initial value of $(N_1)_{60}$ was used. Therefore, the original shear wave velocity of the soils at different depths was estimated to be 116-239 m/sec for the standard numerical profile. Other model parameters in use can be found in Table 3. Table 5 summarizes the shear wave velocities of the standard soil profile used in this study along with their correspondent SPT-N and N_1 values (N_1=Na where the influence of fine content is assumed trivial). Once the values of Na and F_L for soils at the varying depths are known, D_E can be determined using Table 2.

The standard seismic record for simulations in this study is the 1995 Kobe Earthquake. The corresponding acceleration time-history was calibrated to 0.2g, 0.3g and 0.4g to examine the influences of target PGAs. For simplicity, PGA calibration was adopted. To show the influences of seismic records, the accelerograms of the 1999 Chi Chi Earthquake, 2011 Christchurch Earthquake and 2011 Tohoku Earthquake were also studied for PGA at 0.2g. It should be pointed out that the selected earthquakes were based simply on their popularity at the time. The source of these earthquakes, such as subduction vs. shallow crustal; distant vs. near, etc., were not taken into account in this paper.

Table 5 Vs, SPT-N and N_1 values for the soils of standard profile

Depth (m)	Vs (m/s)	SPT-N	N_1	Depth (m)	Vs (m/s)	SPT-N	N_1
1	116	3	7	11	182	12	10
2	138	5	8	12	185	12	10
3	140	5	8	13	189	13	10
4	148	6	8	14	192	14	11
5	154	7	8	15	195	14	11
6	160	8	9	16	198	15	11
7	165	9	9	17	201	16	11
8	170	10	9	18	203	16	11
9	174	10	10	19	206	17	11
10	178	11	10	20	208	18	11

Note: $Vs = 80\times N^{0.33}$ and $N_1 = N \times C_N$ where $C_N = (1/\sigma_v')^{0.5}$ (σ_v' in kg/cm^2)

Figures 3-6 show the accelerogram and the corresponding Fourier spectrum for each of the seismic records. It should be noted that the seismic force is the most important factor affecting the ground response. Its effects are much more significant than those of the soil parameters and ground profiles (Kramer, 2008). It can be found that regarding the seismic records in use, the predominant periods of the Kobe, Chi Chi, and Tohoku earthquakes are all less than 1 second, while the predominant period of the Christchurch earthquake is nearly 3.5 seconds. Moreover, it was found that the accumulated density of the spectrum is also an important factor. Table 6 lists the predominant periods, accumulated density of the spectrum, and the duration of the seismic records used in this study.

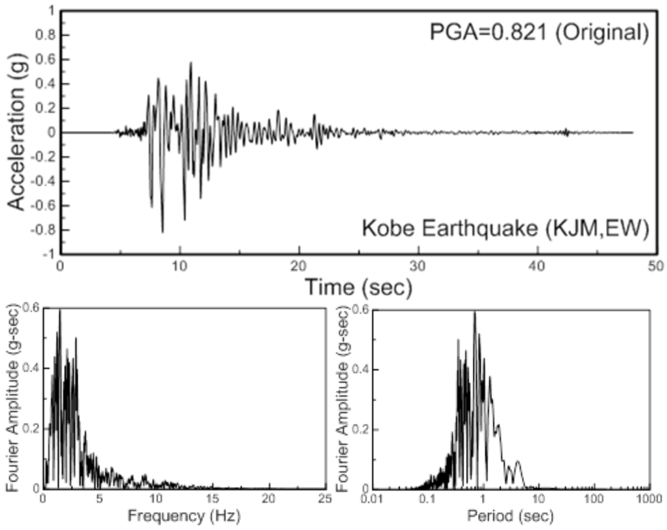

Figure 3 1995 Kobe earthquake record used in this study

Based on the results, the influence order of the accumulated density of the Fourier spectrum among these records is found to be: Tohoku > Chi Chi > Kobe > Christchurch. Furthermore, by eliminating vibrations less than 0.02g after the peak, the order of duration is found to be: Tohoku > Chi Chi > Christchurch > Kobe. Note that the durations estimated herein are simply following the bracketed duration of the record over 0.02g. They are not the same as the significant duration which is used more frequently in the ground motion community (*i.e.*, D5-95 or D5-75), in which the duration needs to be estimated by the time interval over which a specific percentage of the total energy represented by the integral $\int a^2 dt$ is accumulated, where a^2 represents the ground acceleration (corresponding ranges for the accumulated energy are 5%-95% and 5%-75%).

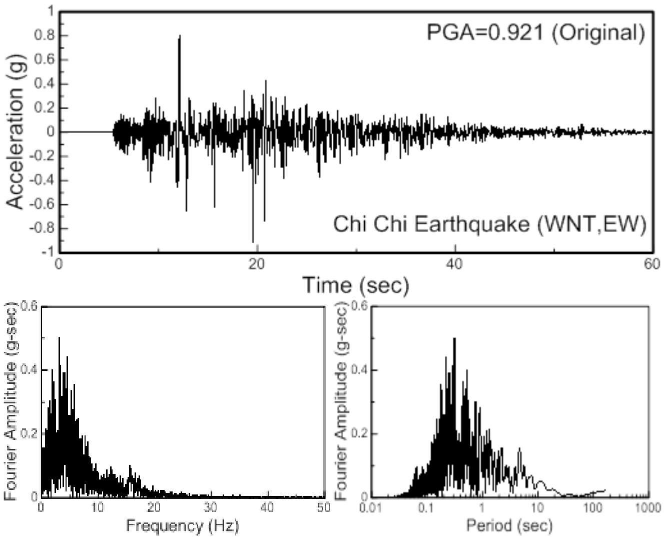

Figure 4 1999 Chi Chi earthquake record used in this study

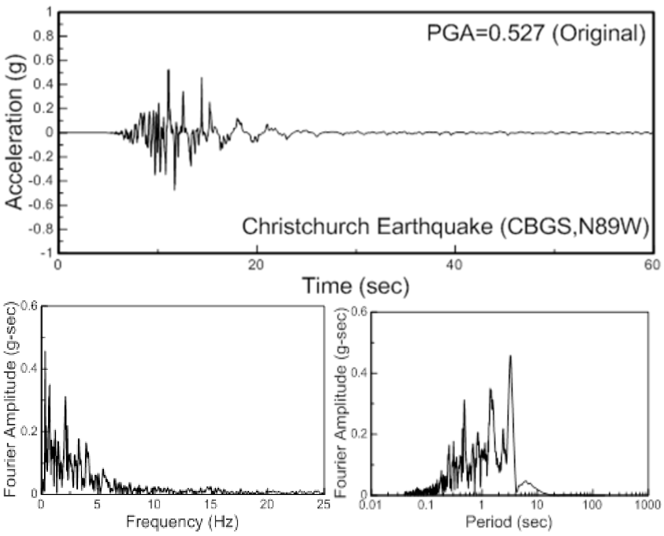

Figure 5 2011 Christchurch earthquake record used in this study

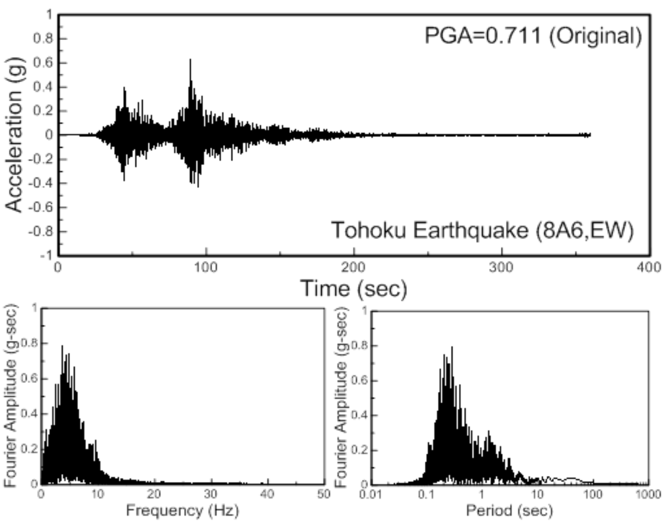

Figure 6 2011 Tohoku earthquake record used in this study

Table 6 Predominant period, accumulated density of Fourier spectrum and duration of the seismic records

Earthquake	1995 Kobe	1999 Chi Chi	2011 Christchurch	2011 Tohoku
Predominant period (sec)	0.75	0.3	3.5	0.28
Accumulated density of spectrum (g)	1.10	1.21	0.75	1.96
Approximate duration* (sec)	24	42	21	180

*Note: vibrations under 0.02g after the peak were not taken into account in the estimations

5. OBSERVATIONS AND DISCUSSIONS

Table 7 depicts the factor of safety against soil liquefaction (F_L), the Cyclic Resistance Ratio (*R*) and the calibrated SPT-N (Na) values for the standard soil profile subjected to seismic input based on the Kobe earthquake with a PGA of 0.2g. Corresponding influence factors on the soil parameter reduction coefficients are subsequently discussed. The time-histories for pore water pressure, mean effective stress and shear modulus of the soils obtained at depths of 5 m, 10 m, 15 m and 20 m are plotted in Figures 7-9. From the figures, it can be seen that the peak ground displacement value decreased as the depth increased. The shear modulus reduction values of the soil are highly dependent on the mean effective stresses exerted. It was found that the values decreased as the depth increased.

Table 7 Factor of safety F_L, cyclic resistance ratio *R* and Na values of standard profile using Kobe earthquake record with PGA = 0.2g

Depth (m)	F_L	*R*	Na	Depth (m)	F_L	*R*	Na
1	1	0.33	7	11	6.21	0.14	10
2	1	0.33	8	12	7.75	0.16	10
3	0.06	0.00	8	13	9.35	0.17	10
4	0.09	0.00	8	14	11.06	0.18	11
5	0.42	0.01	8	15	12.63	0.19	11
6	0.94	0.03	9	16	14.05	0.20	11
7	1.65	0.06	9	17	15.09	0.20	11
8	2.53	0.08	9	18	15.63	0.20	11
9	3.61	0.10	10	19	15.79	0.21	11
10	4.83	0.12	10	20	15.62	0.21	11

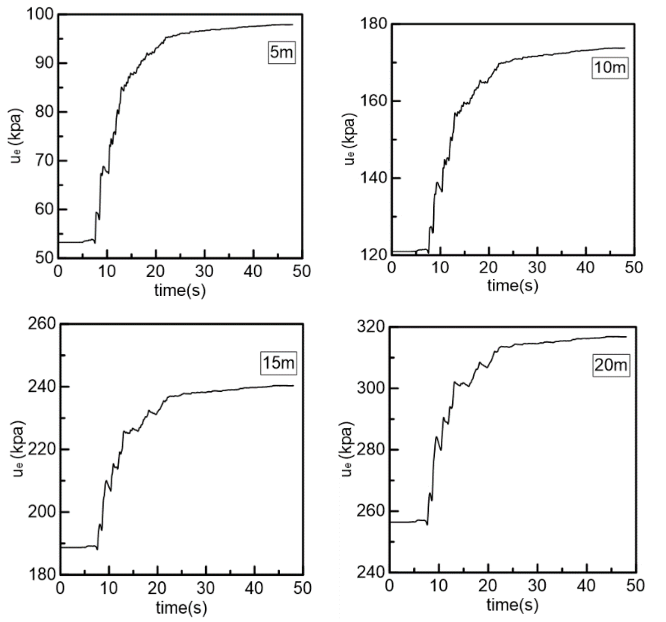

Figure 7 Pore pressure time histories at depths of 5 m, 10 m, 15 m and 20 m for standard soil profile using Kobe earthquake record with PGA = 0.2g

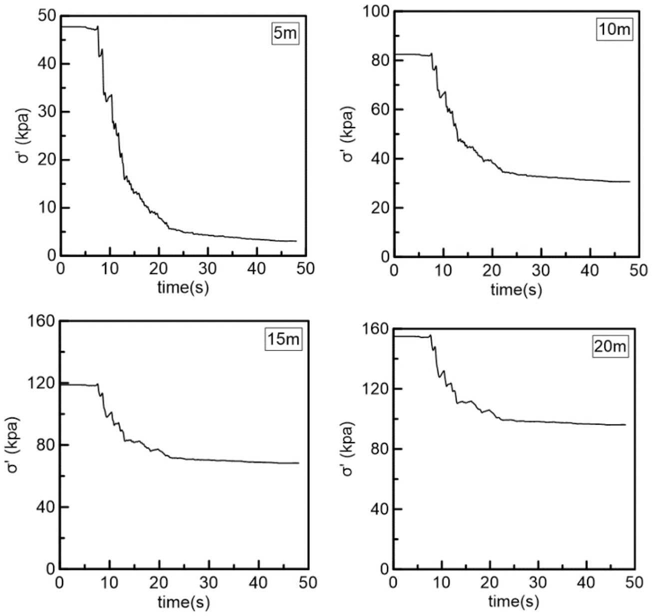

Figure 8 Mean effective stress time histories at depths of 5 m, 10 m, 15 m and 20 m for standard soil profile using Kobe earthquake record with PGA = 0.2g

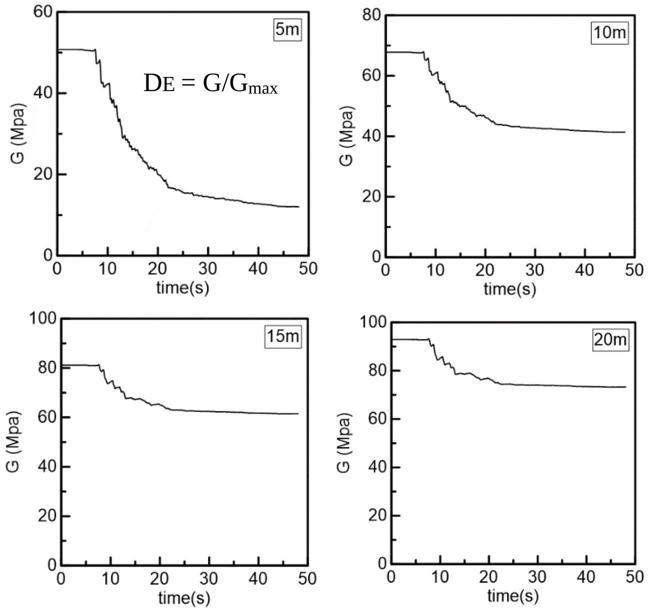

Figure 9 Time dependent shear moduli of the soils at depths of 5 m, 10 m, 15 m and 20 m for standard soil profile using Kobe earthquake record with PGA = 0.2g

5.1 Influences of PGA

Figure 10(a) shows the D_E values computed in this study based on both JRA and AIJ suggestions for the top 20 meters of the standard soil profile based the 1995 Kobe earthquake seismic record, with target PGA = 0.2g. Subsequently, Figures 10(b) and 10(c) illustrate the unique influences of PGA at 0.3g and 0.4g, respectively. It can be seen that as the target PGA increases, the differences between these methods become more apparent for soils at deeper depths. The D_E values obtained from these two methods for the soils at shallow depths (2-4 m) are relatively similar to the G/G_{max} values obtained from the ground response analysis (notice that the reduced shear modulus in this study is limited to values ≥ $0.1G_{max}$). On the other hand, while the JRA and AIJ values are similar in the liquefaction zone, the differences between them became more apparent as the depth increases. It appears that the JRA method provides optimistic estimations for the soils at depths of 6-20 m below the liquefaction zone. However, the AIJ method provides more conservative estimations for the soils at depths of 5-20 m, despite the fact that its predictions were found less sensitive to PGA in this case. The AIJ method suggests that the soils below the water table up to a depth of about 10 m will experience liquefaction. Meanwhile, at a depth of 20 m, the D_E values obtained from the ground response analysis were in the range of 0.6-0.8 for the different target PGA's, while values obtained from the JRA method remained at 1.0. The corresponding D_E values obtained from the AIJ method remained at 0.5 in each case.

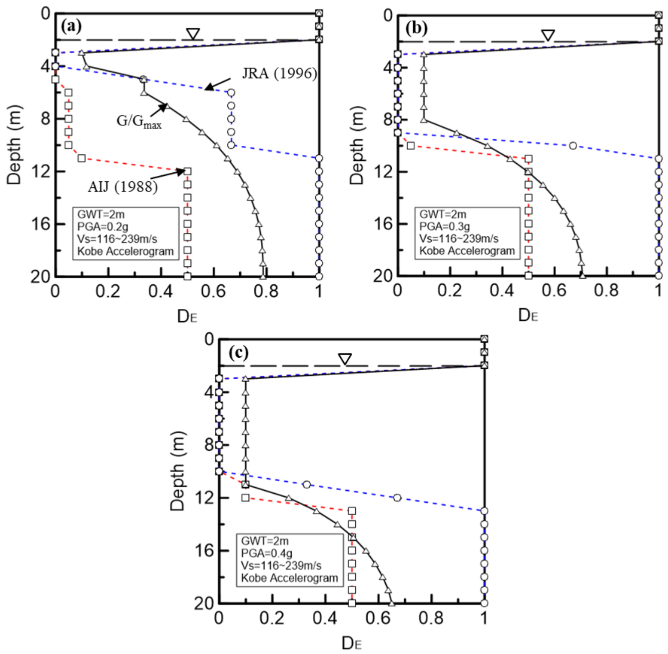

Figure 10 Influences of PGA: (a) 0.2g (b) 0.3g, and (c) 0.4g

5.2 Influences of Water Table Depth

As the ground water table drops, the factor of safety of the site is increased. Such a phenomenon can be observed by comparing Figures 11(a)-11(c). In the case where the ground water depth is at 7 m, it can be seen that for the soils at depths of 7-10 m, the value obtained from the JRA method is a bit more conservative than the values obtained from the G/G_{max} case. However, at depths below 11 m, the JRA method provides less conservative results, while the G/G_{max} case suggests that D_E should be reduced to 0.8^+. Again, when the water depth drops to 12 m, at a depth of 20 m the G/G_{max} analysis provides an approximate reduction of 15%. The D_E values obtained from AIJ were found to be more conservative in these cases. It can be seen that when the ground water table (GWT) is at 7 m, the AIJ method predicts 80-50% reductions. Subsequently, when GWT drops to a depth of 12 m, it suggests a 50% reduction of the soil parameters.

5.3 Influences of Soil Stiffness

By varying the initial value of $(N_1)_{60}$ used in UBCSAND model from 5, 9, and 13, the corresponding shear wave velocities of the soils in the profile are in the range of 116-239 m/s, 129-268 m/s, and 137-286 m/s from the ground surface to the bottom of the soil profile (which is at the depth of 50 m). As the stiffness of the soil increases, a decrease in the influence of soil liquefaction occurs. This can be seen by comparing Figures 12(a)-12(c). Note that JRA method herein provides very conservative estimations compared to the ground response analysis at depths of 2-11 m if the initial $(N_1)_{60}$ value was at 9. However, at a soil depth greater than 11 m, the JRA prediction suggests that no reduction is required. In contrast, the ground response analysis (G/G_{max}) yields reduction coefficients in between 0.8-0.93 at depths of 2-20 m. If the initial $(N_1)_{60}$ value increased to 13, the difference between JRA and the ground response analysis could be neglected, however the JRA would yield more conservative values for soils at a depth of 3 m. Meanwhile, it can be seen that the predictions from AIJ yielded the lowest values among the methods used. Again, it can be seen that the changes in soil stiffness are not sensitive to the predictions suggested by the AIJ method, especially for the soils at depths below 12 m.

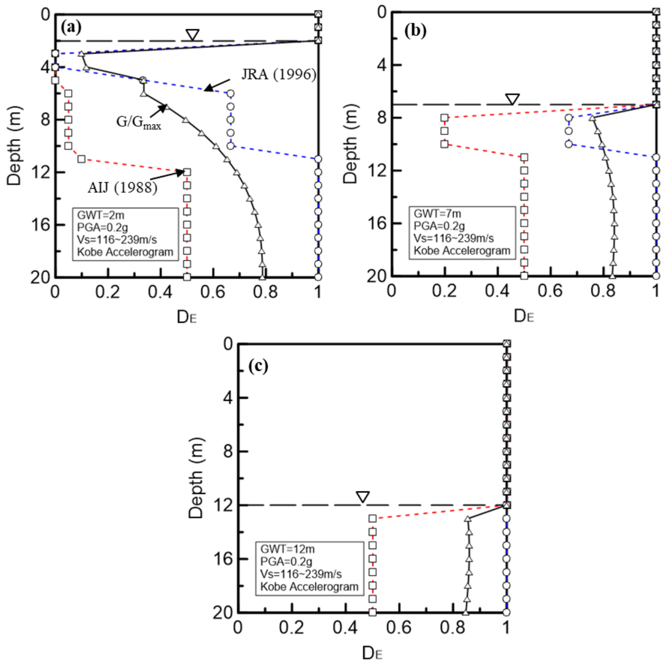

Figure 11 Ground water table influences: (a) 2 m (b) 7 m, and (c) 12 m

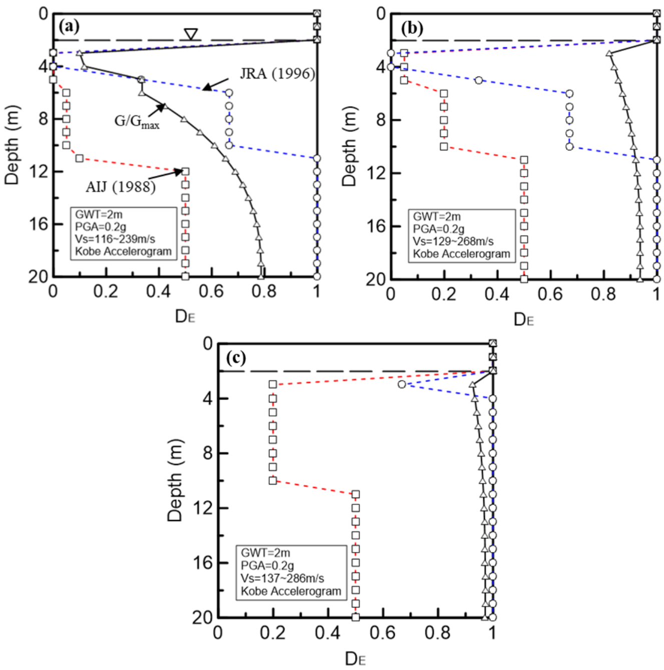

Figure 12 Soil stiffness influences: (a) $(N_1)_{60} = 5$ where Vs = 16-239 m/s, (b) $(N_1)_{60} = 9$ where Vs = 129-268 m/s, and (c) $(N_1)_{60} = 13$ where Vs = 137-286 m/s

5.4 Influences of Seismic Records

Results based on the influence of the seismic records used related to the Chi Chi, Christchurch, and Tohoku earthquakes with PGA = 0.2g on the standard soil profile, are shown in Figures 13(a), 13(b), and 13(c), respectively. In the case of the Chi Chi earthquake, it can be seen that the results obtained from the AIJ and JRA methods suggest that severe reductions ($D_E = 0$) should be made for the soils at depths of 2-11 m. At depths of 11-20 m, the JRA method provides slightly higher D_E values, while the AIJ method results in the same prediction as the ground response analysis. In the case of the Christchurch earthquake, larger reductions values are observed as compared to those in Figure 10(a) at the deeper depths. Deviations in the results obtained among the methods are distinctive for the soils at depths below 10 meters, in this case.

In the case of the Tohoku earthquake, the reduction values obtained from the AIJ method at the deeper depths are similar to those obtained in the case of the Christchurch earthquake. It can also be seen that at depths of 15-20 m, the AIJ method provides higher D_E values than those from the ground response analysis. When comparing the Tohoku and Christchurch cases, similar trends can be found. However, it can be seen that in the case of Tohoku, the severe reduction zone resulting from the ground response (mechanical) analysis increased slightly (2-14 m), while the reduction coefficient at the depth of 20 m increased to 0.5. On the other hand, it can be seen that the JRA method is again too optimistic for the soils at deeper depths, while the AIJ method results in predictions relatively similar to the ground response analysis. Based on the findings, it can be inferred that the resulting predictions for soils at relatively deep depths (say 14-20 m) are influenced greatly by the seismic record used in the analysis.

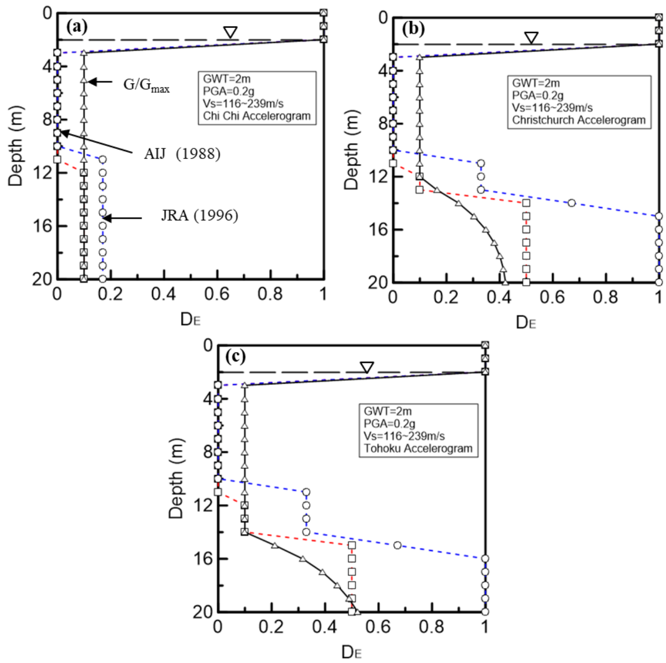

Figure 13 Seismic record influences: (a) Chi Chi EQ, (b) Christchurch EQ, and (c) Tohoku EQ

5.5 Relationships of D_E and $(N_1)_{60}$

The data analyzed for soils at depths of 3 m, 5 m, 7 m, 9 m, 12 m, 16 m and 20 m are plotted in Figures 14(a), 14(b), and 14(c) to reveal the relationship between $(N_1)_{60}$ and D_E. Note that the energy influence is temporarily ignored, therefore $(N_1)_{60}$ should be denoted as N_1. The relationships consist of the influences of PGA, seismic record, ground water table and stiffness of the soil profile. One can find that the reduction coefficients obtained from the JRA method for soils at 0-10 m (Figure 14(b)) are quite consistent to the ones suggested from the mechanical analysis. The AIJ method on the other hand, provides smaller D_E values than the mechanical analysis. For the soils at depths of 10-20 m (Figure 14(c)), again the JRA method results appear to be reasonable except for the case where the suggested $D_E = 1.0$. The AIJ method will again provide D_E values at 0.5 for soils with $(N_1)_{60}$ in the range of 10-20 m. From the mechanical analysis conducted in this study, it appears that for the soils at depths of 0-10 m, any soil of $(N_1)_{60} \leqq 10$ could result in the reduction coefficient D_E less than 0.8 if soil liquefaction has occurred. Similarly, if soil liquefaction was induced, for soils at the depths of 10-20 m, $(N_1)_{60} \leqq 12$ would be the criterion for a 20% reduction of the soil parameters.

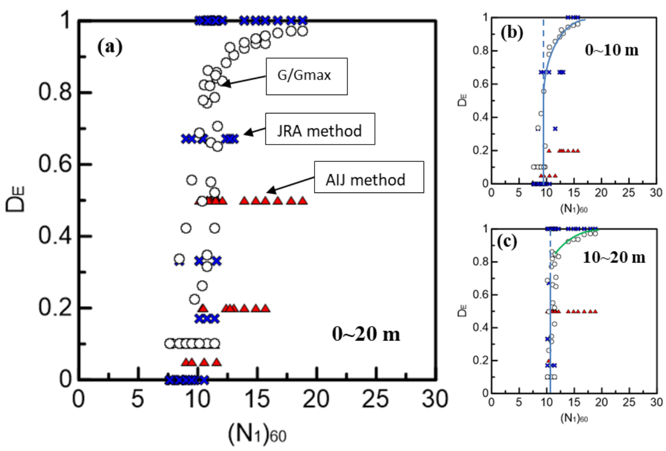

Figure 14 D_E versus $(N_1)_{60}$: (a) 0-20 m (b) 0-10 m, and (c) 10-20 m

It is interesting to compare the data of Figure 14 with the ones reported by Ashford et al. (2011) on the *P*-multiplier, m_p with $(N_1)_{60}$ (see Figure 15). Again, this study is mainly based on N_1 values of 7-11 where the fine content and energy effects are simply neglected. The *P*-multiplier is used to calibrate the load of the *P*-*y* curve subjected to liquefaction. Therefore, the comparisons are made only for observation. With regards to the soil profiles with much lower N_1 values, more reductions are expected. The corresponding data points would shift towards to the left lower corner in Figure 14. For the soil profiles with higher stiffness and lower ground water table, the reductions should become trivial. In such cases, D_E nearly equal to unity will be rational. It is also found that the data points of the AIJ method in Figure 14 seem to match the dashed lines suggested by AIJ in Figure 15. This is typically true for the soils at 0-10 m depth. For the soils at the depth of 10-20 m, the AIJ data from this study showed that D_E is 0.5 for N_1 at 10-18. Further studies are aimed on wider variations of the influence factors. The influences of FC and energy ratio (ER) as well as the magnitude of the earthquake which would affect the magnitude scaling factor (MSF) should be considered.

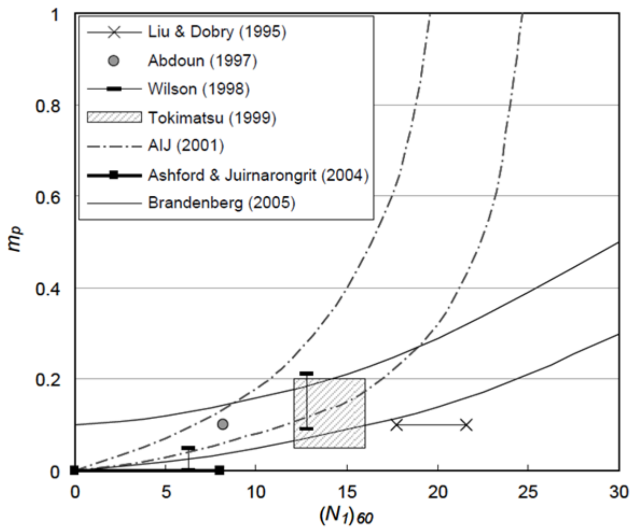

Figure 15 Relationships of m_p vs. $(N_1)_{60}$ (from Ashford et al., 2011)

6. CONCLUSIONS

Soil parameter reduction coefficients, D_E suggested by JRA (1996) and AIJ (1988) for soils at post liquefaction are examined in this study. Dynamic responses of a numerical site characterized by sandy soil layers were computed using the lumped mass analysis while obeying the material model suggested by Byrne et al. (2004). The reduction coefficients were first calculated using G/G_{max} of the soils. The D_E values following the suggestions of JRA and AIJ were also obtained. The differences in results were discussed considering varying influence factors, including PGA, seismic record used, depth of ground water, and soil stiffness. The fine content and rod energy ratio were neglected in this study, and based on the findings, the conclusions drawn are as follows.

The soil parameter reduction coefficients suggested by JRA (1996) were found to be rational for soils in the liquefaction zone. While for the soils at shallow depths underneath the liquefaction zone, the JRA method was found to provide predictions that are a bit conservative. However, for soils at relatively deep depths, the JRA method results obtained seem to be too optimistic.

The influences of PGA were clearly shown for the soils at shallow depths underneath the water table. However, for the soils at deeper depths, the reductions suggested by JRA were found relatively insensitive compared to the values obtained from G/G_{max} from the ground response analysis and the AIJ method. It was also found that, reduction coefficients resulting from JRA suggestions would also be conservative for soils underneath the water table at a medium stiff site.

The seismic record in use significantly influences the results obtained. For the records used in this study, it was found that the analysis based on the Chi Chi seismic record resulted in greater parameter reductions for soils at deep depths. At these depths (say 14-20 m), the AIJ method was found to produce results that suggests its insensitivity to the soil reductions. Except for the case of the Chi Chi earthquake, with regards to the other two cases, the D_E value suggested by the AIJ method at these depths, produced a similar value of about 0.5. If severe soil liquefaction occurred, the AIJ method could provide closer estimations of the D_E values for soils at deep depths. However, if the soil liquefaction is minimal, it was found that AIJ method will over predict the reduction for soils at deeper depths.

Furthermore, the reduction coefficients from the AIJ method appear to be more rational in the case of a large PGA ($\geqq$ 0.3g) or longer time-history of the seismic record being analyzed. Excluding the possible influences of the fine content and energy ratio for $(N_1)_{60}$ which as mentioned before had been neglected in this study, it seems that soils at depths of 0-10 m, where $(N_1)_{60} \leqq 10$ would result in a D_E value less than 0.8. In addition, it should be noted that for soils at depths of 10-20 m, where $(N_1)_{60} \leqq 12$ this method should be approached with caution.

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