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Probabilistic Liquefaction Analysis Using Standard Penetration Test

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Abstract: The Youd etal.liquefaction resistancecurves developed in 2001 to characterize the cyclic resistance of soil based on SPT test are the most used in the context of the Seed and Idriss simplified procedure as a deterministic model. These curves were developed from a modified database of Seed etal. in 1985with the assumption thatthe actual peak shear stress (τ_d) induced at depth h is always less than that predicted by the simplified procedure (τ_r) of Seed and Idriss (rd= τ_d/τ_r<1). By using a suite of equivalent linear site response analyses to adjust the dynamic and the simplified shear stress at depth h, Filali and Sbartai showed in 2017that the dynamic peak shear stress for some earthquakes is greater than the simplified peak shear stress (rd>1).As in this case, the assumptionof the simplified procedure is not verified, Filali and Sbartai have proposed a correctorfactor (RC) in the range where r_d>1 to adjust the deformable and rigid body. In this paper, we will present a probabilistic study for the evaluation of the liquefaction potential using a database based on SPT measurementcompiled after the Chi-Chi Taiwan earthquake,in which the cyclic stress ratio is evaluated using the proposed corrector factor. The objective of this study is to present a probabilistic shape of the cyclic resistance ratio (CRR) curves based on the original simplified method of Seed and Idriss and the corrected version and a new formulation for computing the probability of liquefaction.

Keywords: earthquakes; probabilistic hazard analysis; site effects/liquefaction; probability; random variable; wave propagation.

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1 Introduction

To consider the uncertainties in the evaluation of liquefaction resistance, several studies have been conducted based on probabilistic analysis to improve the existing cyclic resistance ratio(CRR) curves proposed in the literature. Juang etal. (2000) have proposed a new approach for developing a liquefaction limit state function related to the Youd etal. (2001) model, which defines a boundary that separates liquefaction from no liquefaction occurrence. In Juang etal. (2009), a procedure for estimating uncertainty of the Youd etal.(2001) method was developed.Goharzay etal.(2017) used gene expression programming (GEP) to evaluate the occurrence of soil liquefaction in terms of liquefaction field performance and factor of safety in logistic regression by using the liquefaction resistance model of Idriss and Boulanger (2010).Sebaaly and Muhsin (2019) have also proposed a procedure to evaluate the uncertainty of the Idriss and Boulanger (2010) models based on Standard Penetration Test and Cone Penetration Test tests. Bagheripour etal. (2012) have performed a reliability analysis based on advanced first-order second-moment (AFOSM) technique associated with genetic algorithm (GA) to estimate the reliability index and the probability of liquefaction using the CRR model of Youd etal. (2001).Based on a probabilistic analysis, Al-Zoubi(2015) suggested a design method based on a predetermined reliability for selecting the coefficients of active and passive lateral earth pressures and their variations under seismic conditions. A reliability analysis of rock slope using soft computing techniques was conducted by Prithvendra etal.(2020) to show that Extreme Learning Machine and Multivariate Adaptive Regression Splines models are well capable of predicting the reliability of slope in terms of the factor of safety of rock slope, considering statistical predictnds.

After the earthquakes of Alaska (1964) and Nigata in Japan (1964), Seed and Idriss (1971) developed a simplified procedure based on insitu tests to evaluate the liquefaction potential, which is defined by a safety factor calculated by the ratio between CRR and the cyclic stress ratio (CRR/CSR). Thereafter, this procedure was modified and improved, in particular, by Seed (1979),Seed and Idriss (1982), Seed etal. (1985), andYoud etal. (1997, 2001). This procedure is based on simplifying the hypothesis by considering the soil column as a rigid body with the assumption that the actual peak shear stress (τ_d) induced at depth h is always less than that predicted by the simplified procedure (τ_r) of Seed and Idriss(1971) (rd= τ_d/ τ_r<1). All the expressions proposed for $r_d$in the literature are based on a many equivalent linear site response analyses in which each site is submitted to one corresponding earthquake. Filali and Sbartai (2017) conducted the same analysis, but by submitting each site to many earthquakes (38 in their study) in order to show the influence of the variation of the input motions on r_d profiles.Thus, Filali and Sbartai (2017), in their study, showed that the dynamic cyclic shear stress (CSRD) can, in many cases, be greater than the simplified shear stress (CSR) according to the used earthquake. Therefore, r_d can be greater than 1;this result (r_d>1) was found in the study conducted by Farrokhzad (2016) for many sites at a significant depth and in the worksreportedby Sun etal.(2020),Cetin and Seed (2004), and Dismuke (2014) at a shallow depth for a few sites.In this case, this procedure cannot be considered as conservative;thus, the simplified procedure of Seed and Idriss (1971) cannot be applied because itis based on the assumption that r_d*<1*, and all the modifications and improvements made in the literature are based on this assumption. For this reason, in order to generalize the use of the simplified procedure, Filali and Sbartai (2017) have proposed a corrector factor to adjustthe simplified CSR in the range where r_d>1, which corresponds to a maximum acceleration of the earthquake less than 0.30*g* (a_{max}≤0.30*g*). In this paper, we will present a probabilistic analysis of liquefaction potential based on the proposed correction (Filali and Sbartai, 2017) in order to define the CRR curves used to characterize the boundary between liquefied and non-liquefied regions. For this purpose, we have used the case history database compiled after the Chi-Chi earthquake, which consists of 287 casesincluding 163 liquefied sets and 124 non-liquefied sets. The liquefaction during this earthquake appeared in several sites such as Nantou, Wufenf, and Yuanlin for soils with low and high fine content (FC), which isthe characteristic unique to this earthquake (Hwang and Yang,2001).

2 Deterministic Model

The approach of Seed and Idriss (1971) is the most widely used procedure in practice for estimating the liquefaction resistance of sandy soils. To represent the ground motions caused by earthquakes with one single parameter, a simplified procedure has been developed by Seed and Idriss (1971) and updated in Youd etal. (2001). The resistance to liquefaction is evaluated by comparing a property index of the soil to the CSR given by the following equation for a magnitude of the earthquake adjusted to 7.5:

$$\mathrm{CSR} = \frac{\tau_{cyc}}{\sigma'_{v0}} = 0.65 \times \left(\frac{a_{max}}{g}\right) \times \left(\frac{\sigma_v}{\sigma'_v}\right) \times r_d \qquad (1)$$

where $\sigma_v$isthe vertical total stress of the soil at the depth studied, $\sigma'_v$the vertical effective stress of the soil at the depth studied, $a_{max}$the peak horizontal ground surface acceleration, *g*the acceleration of gravity, and $r_d$isthe shear stress reduction factor. The variable $r_d$is calculated in accordance with Youd etal. (2001) as follows:

$$\begin{aligned} r_d &= 1 - 0.00765z && z \le 9.15m \\ r_d &= 1.174 - 0.0267z && 9.15 \le z \le 23m \\ r_d &= 0.744 - 0.008z && 23 \le z \le 30m \\ r_d &= 0.5 && z > 30m \end{aligned} \qquad (2)$$

The various worksexisting in the literature, such as Cetin and Seed (2004), Lasleyetal. (2016), and others, related to $r_d$factor use a suite of equivalent linear site response analyses to adjust the dynamic and simplified results (deformable and rigid body). AfterFilali and Sbartai (2017), as the assumption r_d<1 is verified only when a_{max}>0.30*g*, in other words, when a_{max}<0.30*g*, which corresponds to r_d>1. Then, the deformable and rigid body is not adjusted in accordance with the assumption on which is based the simplified procedure.Also,in order to generalize the use of the simplified method by adjusting the deformable and rigid body whatever the used earthquake is, the authors have proposed a new earthquake corrector factor, RC, in the range where a_{max}≤0.30*g* in order to adjust the dynamic and simplified results when r_d>1 and ensure the reliability of the simplified method by giving the most conservative case for all earthquakes. The proposed correction (Filali and Sbartai,2017) is defined by an earthquake corrector factor, RC, which is the ratio between the dynamic and the simplified shear stress, expressed as follows:

$$\begin{cases} \mathrm{RC} = 0.696 \left(\frac{a_{max}}{g}\right)^{-0.577} & \text{if } a_{max} \le 0.30g \\ \mathrm{RC} = 1 & \text{if } a_{max} > 0.30g \end{cases} \qquad (3)$$

This correction can be applied only when a_{max}≤0.30g;otherwise, Eq.(1) is kept without correction (RC=1).

Then, by applying this correction, the original form of CSR (Eq.1) can be rewritten in accordance with the following expression (Filali and Sbartai,2017):

$$\begin{cases} \mathrm{CSR} = 0.65 \times \left(\frac{a_{\max}}{g}\right) \times \left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) \times r_d & \text{if } a_{\max} > 0.30g \\ \mathrm{CSR} = 0.65 \times \left(\frac{a_{\max}}{g}\right) \times \left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) \times r_d \times \mathrm{RC} & \text{if } a_{\max} \le 0.30g \end{cases} \quad (4)$$

2.1 Cyclic resistance ratio

The empirical graph for evaluating liquefaction resistance based on SPT test developed by Seed etal.(1984) hasbeen in the first term approximated by an equation proposed by Rauch(1997) based on the corrected blow count N_{160}. To consider the effect of FC, Youd etal.(2001) have introduced the corrected blow counts for cleansands and giventhis equation by the following expression:

$$\mathrm{CRR}_{7.5} = \frac{1}{34 - N_{160\mathrm{cs}}} + \frac{N_{160cs}}{135} + \frac{50}{(10N_{160\mathrm{cs}} + 45)^2} - 1/200 \quad (5)$$

where N_{160cs} is the corrected blow count for clean sands expressed as:

$$N_{160\mathrm{cs}} = a + bN_{160} \quad (6)$$

where a and b are two constant parameters introduced to account the effect of FC and are both functions of FC. The coefficients a and b are given by the following equations:

$$\begin{cases} \mathrm{a} = 0 & \text{for FC} \le 5\% \\ \mathrm{a} = \exp\left(1.76 - \frac{190}{\mathrm{FC}^2}\right) & \text{for } 5\% < \mathrm{FC} < 35\% \\ \mathrm{a} = 5 & \text{for FC} \ge 35\% \end{cases} \quad (7)$$

$$\begin{cases} \mathrm{b} = 1 & \text{for FC} \le 5\% \\ \mathrm{b} = 0.99 + \frac{\mathrm{FC}^{1.5}}{1000} & \text{for } 5\% < \mathrm{FC} < 35\% \\ \mathrm{b} = 1.2 & \text{for FC} \ge 35\% \end{cases} \quad (8)$$

Also,N_{160} is the corrected blow counts expressed as

$$N_{160} = N_m C_N C_E C_B C_R C_S \quad (9)$$

where N_m is the measured standard penetration resistance,$C_E$the correction for hammer energy ratio, C_B the correction factor for borehole diameter,$C_R$the correction factor for rod length,$C_S$the correction for samplers with or without liners, and C_N is the factor to normalize N_m to a common reference effective overburden stress expressed as (Youd etal.,2001; Liao and Whitman,1986a)

$$C_N = \left(\frac{P_a}{\sigma'_{v0}}\right)^{0.5} \le 1.7 \quad (10)$$

The $\mathrm{CRR}_{7.5}$ should be corrected for the earthquake magnitude, overburden pressure, and static shear stress (Seed and Idriss,1982,1983;Boulanger and Idriss,2004) as follows:

$$\mathrm{CRR}_{M_w} = \mathrm{CRR}_{7.5cs}(\mathrm{MSF})K_\sigma K_\alpha \quad (11)$$

where MSFis the magnitude scaling factor and K_σ and K_α are the factors for overburden and initial static stress ratio corrections, respectively. These factorsare calculated with the formulae recommended by Boulanger and Idriss (2004).

2.1.1 Magnitude scaling factor

Several equations have been proposed for the assessment of MSF according to the earthquake moment magnitude (Seed and Idriss, 1982; Idriss,1999). Idriss (1999) proposed the MSF as

$$\mathrm{MSF} = 6.9\exp(-M_w/4) - 0.058 \le 1.8 \quad (12)$$

2.1.2 Overburden correction factor K_σ

The overburden correction factor K_σ can be estimated by the relationship proposed by Boulanger and Idriss (2004) as follows:

$$K_\sigma = 1 - C_\sigma \ln(\sigma'_{v0}/P_a) \le 1.1 \quad (13a)$$

where the coefficient $C_\sigma$can be expressed in terms of corrected shear wave velocity as follows:

$$C_\sigma = 1/(18.9 - 2.55(N_1)_{60}) \le 0.3 \quad (13b)$$

2.1.3 Static shear stress correction factor k_α

To take into account the influence of static shear stresses on CRR, Seed etal. (1983) have proposed a correction factor K_α to correct the CRR. Several studies were conducted by Idriss and Boulanger (2003a,2003b). The author believes that these results can be used. As the soil layers are considered horizontal, the value of K_α in this study is kept as equal to 1.

3 Bayesian Mapping Function (BMF)

Since the deterministic safety factor (*Fs*) is the most widely used in geotechnical practice, it is interesting to relate it to the probability of liquefaction in order to facilitate the use of the probabilistic approach for engineers to take a correctdecision. Juang etal. (1999) have proposed a mapping function approach which linked the deterministic *Fs* to the probability of liquefaction; this approach has been refined by Juang etal. (2000a,2000b). In this approach, the conditional probability of liquefaction for a given site is deduced from the information contained in the case history database (Juang etal., 2000b,2002) according to the following equation:

$$P_L = \frac{f_L(Fs)}{f_L(Fs)+f_{NL}(Fs)} \tag{14}$$

where $f_L(Fs)$ and $f_{NL}(Fs)$ are the probability densityfunctions of the calculated F_S for the sets of liquefied cases and non-liquefied cases, respectively. Based on the obtained Eq.(14), the probability of liquefaction is calculated for each of the287 cases in the database using the original and the corrected versions of the simplified procedure.

3.1 Original procedure of Seed and Idriss (1971)

The variation of the probability of liquefaction against the deterministic safety factor (*Fs*) calculated using the original version of the simplified procedure (Seed and Idriss, 1971) is plotted in Fig.1. The set of the 287 points can be fitted in terms of mapping function, which linked P_L to *Fs*by the following equation:

$$P_L = \frac{1}{1+\left(\frac{F_S}{0.9674}\right)^{7.558}} \tag{15}$$

The deterministic curve model is defined by *Fs*=1. Thus, the Youdetal. (2001) curve can be characterized with a probability of liquefaction of 45% based on Bayesian mapping model.This result is very close to that obtained by Juang etal. (2000). From Eq.(15), we can plot for a given

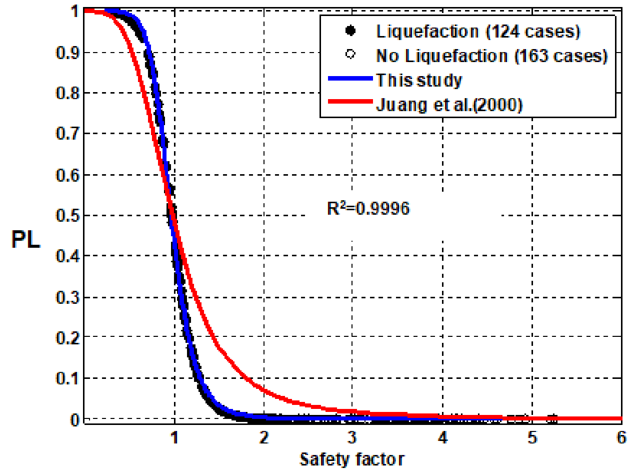

Figure 1: Relationship between P_L and *Fs* based on Bayesian mapping function using the original version of the simplified procedure. PL = probability of liquefaction.

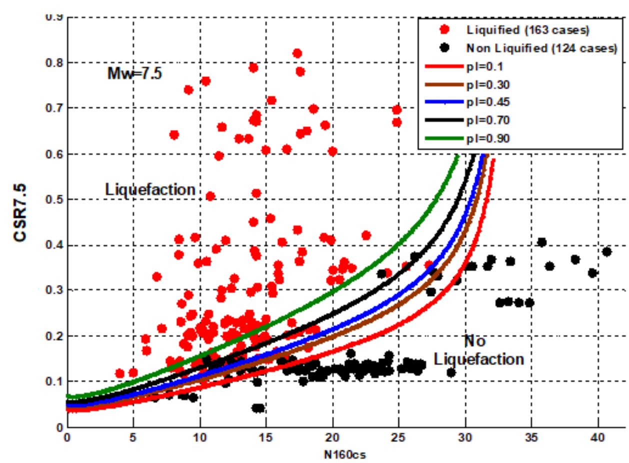

Figure 2: Bayesian mapping function along the case history databaseusing the original version of the simplified procedure. CSR = cyclic stress ratio.

value of P_L the CRR boundary curves presented in Fig. 2. This figure shows that the value of N_{160CS} converges to 32 for high values of CSR.

3.2 Corrected version of the simplified procedure (Filali and Sbartai, 2017)

The safety factor calculated using Eq.(4) for the CSR and Eq.(5) for the CRR is used to recalculate the probability of liquefaction. By fitting the set of points presented inFig.3, the mapping function can be expressed by the relationship below:

$$P_L = \frac{1}{1+\left(\frac{F_S}{0.7585}\right)^{5.076}} \quad (16)$$

In this equation, a value of *Fs*=1 corresponds to the deterministic curve model. Therefore, for this case, the Youdetal. (2001) curves can be characterized with a probability of liquefaction of 20% based on Bayesian mapping model. From Eq.(16), we can plot for a given value of P_L,the CRR boundary curves presented in Fig. 4. This figure shows that the value of N_{160CS} converges to 32 for high values of CSR.

The figure also shows that the Youd etal. (2001) boundary curve is characterized by a P_L=0.20.In accordancewith the corrected version of the simplified procedure, it is not conservative because it cannot be considered as a boundary curve, which separates the liquefied and non-liquefied cases and must be adjusted to the curve corresponding to P_L=0.40, which is very close to the true boundary between the two zones.

By fitting the true boundary between the liquefied and non-liquefied sets using the same shape of the Youd etal. (2001) model, the CRR, $CRR_{7.5}$, can be expressed by the following equation:

$$\mathrm{CRR}_{7.5} = \frac{1}{34-N_{160cs}} + \frac{N_{160cs}}{96.83} + \frac{344.1}{21.43N_{160cs}+87.33} - 1/100 \quad (17)$$

By comparing this equation with that proposed by Youd etal.(2001), we can say that only the curve-fitting parameters have changed. This result is reasonable because according to the corrected version of the simplified procedure, the values of the CSRhave changed;therefore, the boundary between the liquefied and non-liquefied cases may also change and the curve-fitting parameters must be adjusted to the new position of the boundary. Since the mathematical modelof the true boundary is defined, we must recalculate the safety factor and the probability of liquefaction for all cases in the database using Eqs(4) and (17). Inthe same manner, the mapping function deduced by fitting the set of points presented in Fig.5 can be expressed as follows:

$$P_L = \frac{1}{1+\left(\frac{F_S}{0.8976}\right)^{6.271}} \quad (18)$$

This equation shows that the deterministic boundary curve, which correspond to *Fs*=1 is characterized by a probability of liquefaction of 35% instead of40%.The set of probabilistic boundary curves deduced from Eqs(17) and (18) are plotted in Fig.6.The deterministic design decision is always made based on the safety factor, which indicates that the liquefaction occurs or not according to a reference value by choosing the most conservative case. The liquefaction boundaries plotted in Fig.6 show that the Youd etal.(2001) CRR curve is characterized by a probability of 15% using the Bayesian mapping function with the deterministic model given by Eq.(17) based on the corrected simplified method, which corresponds to a deterministic **safety** factor (*Fs*) of 1.20, while the adjusted model proposed in this study shown in Eq.(17) is related to a probability of 35%, which corresponds to *Fs*=1. Then, according to these results, the more conservative case is always given by the corrected simplified method. Thus,

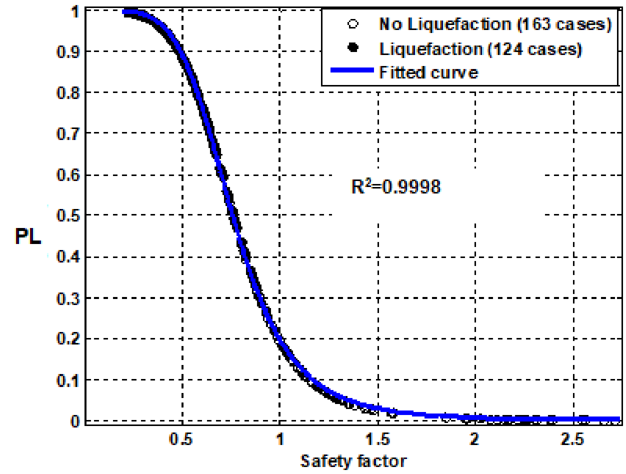

Figure 3: Relationship between P_L and *Fs*based on Bayesian mapping function using the corrected version of the simplified procedure. PL = probability of liquefaction.

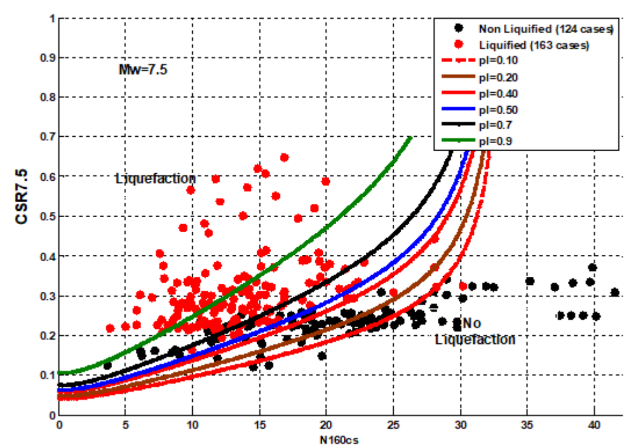

Figure 4: Bayesian mapping function along the case history databaseusing the corrected version of the simplified procedure. CSR = cyclic stress ratio.

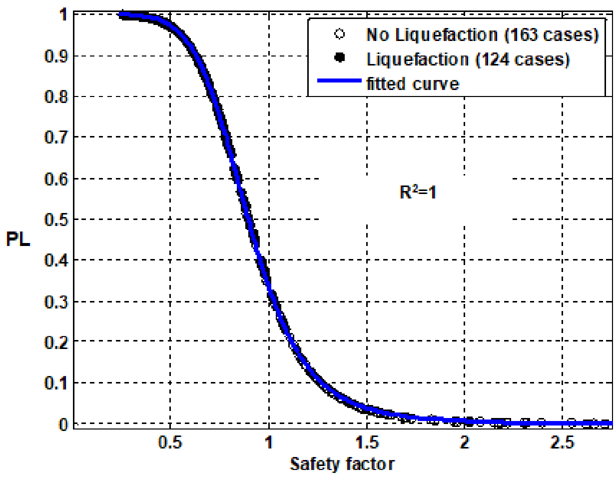

Figure 5: Relationship between P_L and *Fs* based on Bayesian mapping function using the corrected version of the simplified procedure. PL = probability of liquefaction.

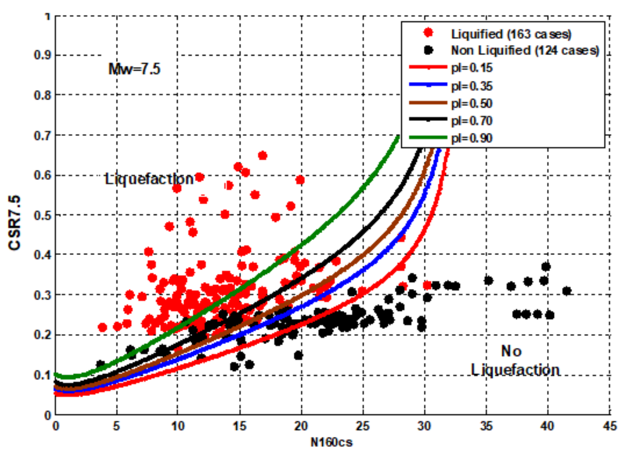

Figure 6: Bayesian mapping function along the case history databaseusing the corrected version of the simplified procedure with Eq.(17). CSR = cyclic stress ratio.

the set of curves shown in Fig.6 indicates liquefaction for the zone above the boundary curve of PL=90% and no liquefaction for the zone below the boundary curve of PL=15%. The zone between PL=15% and PL=90% is an intermediate zone, in which 15% and 35% represent the lower and marginal probabilities, respectively, and above 35%, the risk of liquefaction increases with the probability of liquefaction. To define the severity of the liquefaction potential using a probabilistic analysis, Juang etal. (2001) have proposed a liquefaction likelihood classification thatcan be used for probabilistic design decision using the corrected version of the simplified method.

4 Comparison with Other Curves

The liquefaction resistance correlation based on standard penetration test has been studied by several authors. Based on an updated case history database used to develop the Idriss and Boulanger (2008) and the Boulanger and Idriss (2004) liquefactioncorrelation for cohesionless soils, Idriss and Boulanger (2010) have developed a new liquefaction resistance correlation based on the standard penetration test. Hwang etal. (2012), based on a case history database collected after the Chi-Chi Taiwan earthquake 2001, have proposed a hyperbolic model to express a liquefaction resistance correlation based on an SPT test. Other comprehensive studies have been performed by the geotechnical experts in order to develop liquefaction resistance correlations (Youd etal.,2001; Seed etal.,1984, 1985; Cetin etal.,2016).A comparison between these liquefaction resistance correlations and the adjusted model proposed in this study is presentedin Fig.7. This figure shows clearly that the best fit is given by the corrected version of the simplified method, which materializes the true liquefaction boundary expressed by Eq.(17).The other correlations in Fig.7 cannot be considered as boundary curves because according to the corrected version of the simplified method, the values of CSR for all cases in the database for which a_{max}≤0.30*g* are adjusted through a corrector factor,RC, defined by Eq.(3). Therefore, the plotted set of points of the case history database is translated upward, which leads to move the boundary curve to a new position different to that defined by the original simplified method. Then, in this figure, we have kept the original position of the plotted curves in order to show the effect of the proposed correction on these curves.

5 Validation

Validation of the obtained results will be performed through two case studies for Yuanlin and Coastal road Skikda sites and by using the case history database based on SPT test of Cetin etal. (2016).

5.1 YualinTaiwan site

Nantou site is located approximately 0–5 km from the fault rupture within the Taichung basin. The Chi-Chi, Taiwan earthquake of 09/25/1999 caused significant damage in the village of Yuanlin;for example, liquefaction, landslide,

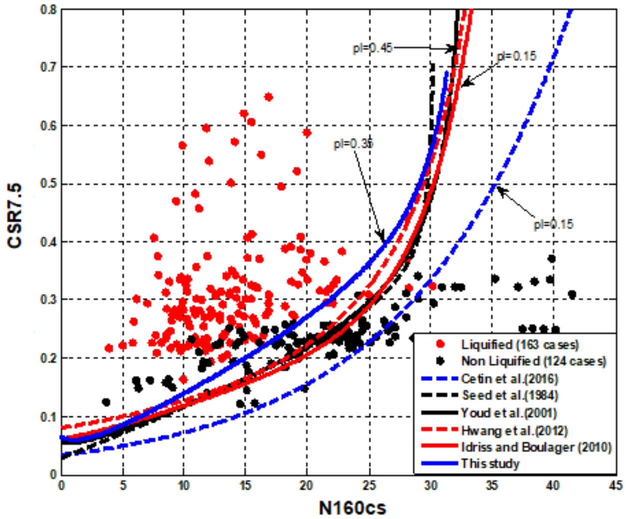

Figure7: Comparison of the original and the adjusted CRR-N_{160cs} curves. CRR = cyclic resistance ratio.

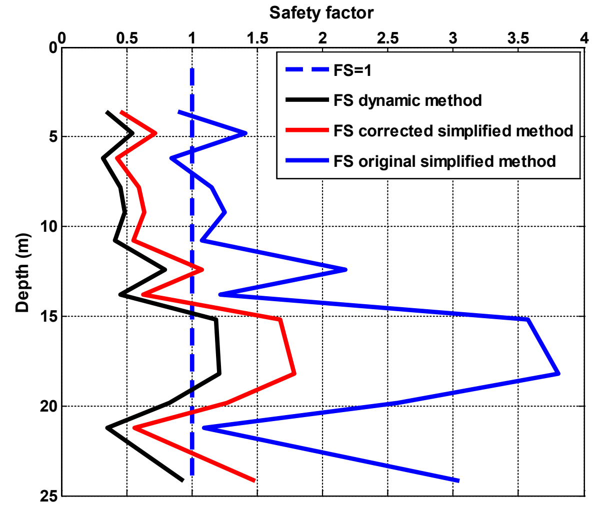

Figure 9: Safety factor according to the depth computed by the original and corrected versions of the simplified method (Nantou site).

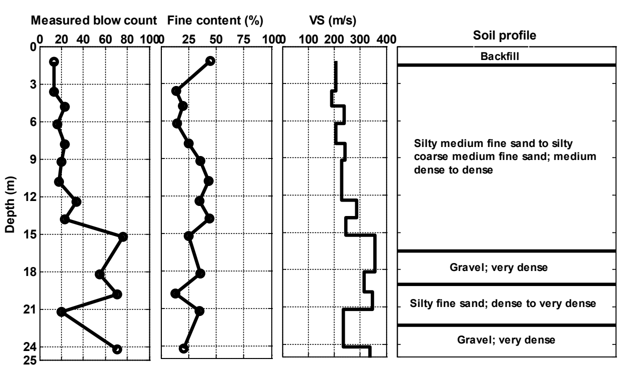

Figure 8: Profile of soil according to the depth (Nantou site).

and major faults appeared on the surface. The geological environment of Nantou is in the form of youngalluvial sediments with shallow groundwater (within about 0.5–5 m of the surface). The National Center for Research on Earthquake Engineering (NCREE) has conducted several investigative programs based on the in situ CPT, SPT, and shear wave velocity (VS) testing. The soil stratigraphy is generally silty medium to fine sand interspersed with very dense layers of small gravelwith a percentage of fines of 7%–45%. The profile of the soil along the SPT boring MAA-BH6 is shown in Fig. 8 (NCREE, 2001).

In this example, we will evaluate the liquefaction potential with the original andcorrected versions of the simplified procedure in order to define which of the two methods gives the more conservative case. Then, the CSR is calculated by using both Eqs (1) and (4); for the estimation of the CRR, we will use Eq.(5)adjusted to FC ≤5%and Eq.(17).The peak ground acceleration value, a_{max}, used for the calculation of CSR is taken to be equal to 0.1687*g*. The depth ofthe groundwater table is kept at 1.2m relative to the ground surface. The average value of the unit weight is taken to be equal to 18.3kN/m^3 above the water table and 20.35kN/m^3 below the water table. In Fig.9 are shown the profiles of safety factor according to the depth computed by the original and corrected versions of the simplified procedure;the profile of the dynamic *F*sis deduced from a dynamic analysis conducted with Shake91_input software(Idriss and Sun,1992) by using Eq.(17) to estimate the CRR.

Fig. 9 shows that the more conservative case is given by the corrected version of the simplified procedure and the profile of the corrected safety factor is very close to the dynamic profile. These results indicate that the maximum shear stress given by the corrected version, which is very close to that computed from a dynamic analysis, is always for this case greater than the shear stress estimated by the original simplified method, which implies that the stress corrector factor, r_d, is greater than 1.To confirm this, we have conducted a dynamic analysis using Shake91_input software (Idriss and Sun,1992), in which the Chi-Chi Taiwan 2001 earthquake is simulated by the TCU075 accelerogram applied at the bottom of the soil profile. In this analysis, we have calculated the maximum shear stress for soil profile using Shake91_input and the simplified method with the original and corrected versions using the maximum acceleration of the TCU075 accelerogram, which is 0.1687*g*. The results are presented in Fig.10. This figure shows clearly that the maximum

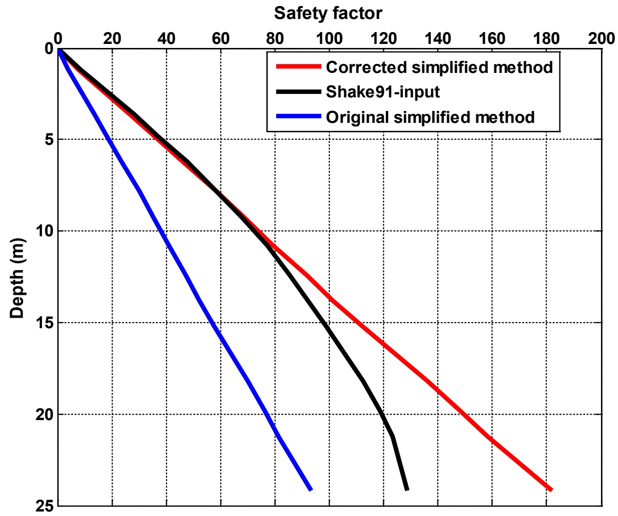

Figure 10: Maximum shear stress according to depth with dynamic and simplified analysis (Yuanlin site).

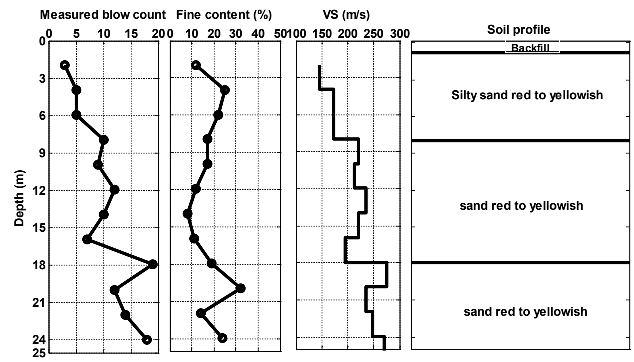

Figure 11: Profile of soil and shear wave velocity according to the depth (petrochemical zone site).

shear stress computed by the original simplified method is less than that given by the dynamic analysis conducted using Shake91_input (r_d>1, a_{max}<0.30g),while the corrected version of the simplified method gives values greater than or equal to those of the dynamic method (r_d≤1).

Then, for this site, the liquefaction potential evaluation must be conducted using the corrected version of the simplified method because the original version cannot be applied since $r_d$is greater than 1.

5.2 Coastal road Skikda site (Algeria)

Based on the request of the National Petroleum Refining Company of Skikda department (NAFTEC), the laboratory has performed a geophysical investigation with three downhole tests. The study site is located within the industrial zone of Skikda;it has a flat topography. The downhole test SC02 detected the presence of a sandy horizon, reddish to brownish, which extendedup to depth 20 m and was saturated with a mean diameter D_{50} varying between 0.11 and 1 mm. The average value of the unit saturated weight is taken to be between19.6 and 20.5kN/m^3. The water table is assumed to be on the ground surface. The magnitude of the earthquake is 6.8, and the maximum acceleration at the surface is equal to 0.122g. The site is classified aszone II according to the Algerian earthquake code RPA 2003.The profile of soil and shear wave velocity chosen in this study are shown in Fig. 11.

In Fig.12 are shown the profiles of safety factor according to the depth computed by the original and corrected versions of the simplified procedure.The profile of the dynamic FS is deduced from a dynamic analysis performed with Shake_input software (Idriss and Sun,1992),in which the dynamic cyclic stress ratio (CSRD) was expressed as the ratio of the maximum shear stress and the vertical effective stress.

For this site, the conclusion is the same as for the Treasure Island site. To confirm this, we have conducted a dynamic analysis using Shake91_input software (Idriss and Sun,1992), in which the Boumerdes earthquake of 21/05/2003 is simulated by the Azazga station accelerogram EW component applied at the bottom of the soil profile. In this analysis, we have calculated the maximum shear stress for soil profile using Shake91_input and the simplified method with the original and corrected versions using the maximum acceleration of the used accelerogram, which is 0.122g. The results are presented in Fig.13. This figure shows clearly that the maximum shear stress computed by the original simplified method is less than that given by the dynamic analysis conducted using Shake91_input (r_d>1, a_{max}<0.30g),while the corrected version of the simplified method gives values greater than or equal to those of the dynamic method (r_d≤1).

5.3 Case history database

From a case history database of SPT liquefaction (Cetin etal.,2016) including 210 cases,we have retained20liquefied cases with a maximum acceleration less than 0.30g and a safety factor computed by the original simplified method greater than 1. For each case in the database, we have computed the safety factor using the proposed correction and the original simplified method; the results are presented in Table 1 shown in Appendix A. By examining the results, we can conclude that the proposed correction indicates that allcases are liquefied (FS_{SMC}<1), while the

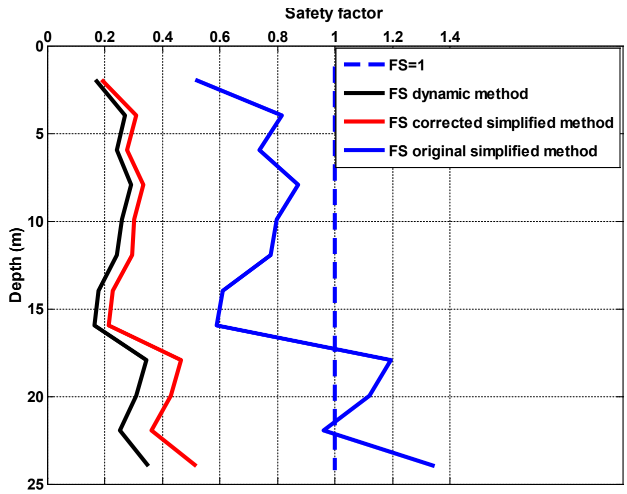

Figure 12: Safety factor according to the depth computed by the original and corrected versions of the simplified method (Coastal roadSkikda site).

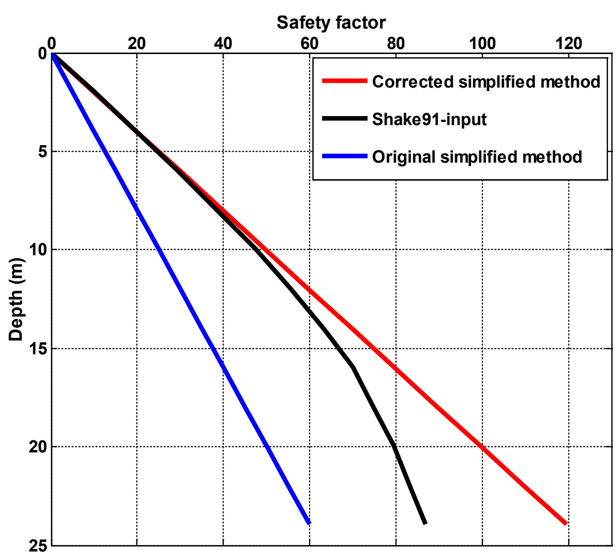

Figure 13: Maximum shear stress according to the depth with dynamic and simplified analysis (Coastal road Skikda site).

original simplified method indicates otherwise (FS_{SM}>1 for 18 cases). Then, it is clearly visible that the database confirms the results of the proposed correction.

6 Conclusion

A probabilistic analysis has been conducted in this paper based on the original simplified method (Seed and Idriss,1971) and the corrected version of this method (Filali and Sbartai,2017) by using a Bayesian mapping function based on standard penetration test. The results show the following:

1. The boundary curve is characterized on one hand by P_L=0.45, which corresponds to FS=1, by using the original simplified method, and on theother hand by P_L=0.40, which corresponds to FS=0.82, by using the corrected version of this method with the Youd etal. (2001) shape of the CRR expressed by Eq.(7).
2. Then, the proposed model for the CRR curve of Youd etal.(2001) must be adjusted to the new boundary in accordance with the corrected version of the simplified method, which corresponds toP_L=0.40, because the boundary curve is obtained by plotting CSR against N_{160cs} from the case history data;also, as the CSR changes for all sites in the database where a_{max}<0.30g, the boundary curve must also change and may be readjusted.
3. According to the corrected version of the simplified method, the boundary between liquefied and non-liquefied zones is readjusted using the proposed Eq.(17). By using the shape of the proposed CRR curve (Eq.17), the probability of liquefaction, which corresponds to a deterministic FS=1, becomes 0.35 instead of 0.40.
4. Then, the proposed model of CRR curve is characterized by a probability of 0.35.This correction is only valid for clean sand (FC<5%).Forother sands e (FC>5%) an adjustment to clean sand may be made according to $N160$cs in order to be able to use the proposed correction.

References

[1] Al-Zoubi, M. S. (2015). Reliability-based determination of the coefficients of lateral earth pressure on retaining walls subjected to seismic loading. *Jordan Journal of Civil Engineering*, *9*(4), 421–434. https://doi.org/10.14525/jjce.9.4.3115

[2] Bagheripour, M. H., I. Shooshpasha, and M. Afzalirad. (2012). “A Genetic Algorithm Approach for Assessing Soil Liquefaction Potential Based on Reliability Method.” *Journal of Earth System Science* 121 (1): 45–62. doi:10.1007/s12040-012-0137-2.

[3] Boulanger, R., & Idriss, I. (2004). Evaluating the potential for liquefaction or cyclic failure of silts and clays. *Neuroscience Letters*, *339*(December), 123–126. https://doi.org/UCD/CGM-04/01

[4] Cetin, K. O., & Seed, R. B. (2004). Nonlinear shear mass participation factor (rd) for cyclic shear stress ratio evaluation. *Soil Dynamics and Earthquake Engineering*, *24*(2), 103–113. http://doi.org/10.1016/j.soildyn.2003.10.008

[5] Cetin, K. Onder, Raymond B Seed, Robert E Kayen, Robb E. S Moss, H. Tolga Bilge, MakbuleIlgac, and Khaled Chowdhury.

(2016). *Summary of SPT Based Field Case History Data of CETIN (2016) Database*. Ankara: METU / GTENG 08/16-01 Middle East Technical University. https://pubs.er.usgs.gov/publication/70184187.

[6] Dismuke, J. N. (2014). Nonlinear shear stress reduction factor (rd) for assessment of liquefaction potential in christchurch central business district. *Bulletin of the New Zealand Society for Earthquake Engineering*, *47*(1), 1–14. http://doi.org/10.5459/bnzsee.47.1.1-14

[7] F, Rauch A. 1997. "Soil Liquefaction in Earthquakes." university of Texas. http:/%0AScholarlib.vt.edu/theses/available/etd-219182249741411/%0Aunrestricted/chp02.pdf.

[8] Farrokhzad, F. (2016). Depth reduction factor assessment for evaluation of cyclic stress ratio based on site response analysis. *Advances in Systems Science and Applications*, *16*(3), 33–51.

[9] Filali, K., & Sbartai, B. (2017). A comparative study between simplified and nonlinear dynamic methods for estimating liquefaction potential. *Journal of Rock Mechanics and Geotechnical Engineering*, *9*(5), 955–966. https://doi.org/10.1016/j.jrmge.2017.05.008

[10] Goharzay, Maral, Ali Noorzad, AhmadrezaMahboubiArdakani, and Mostafa Jalal. (2017). "A Worldwide SPT-Based Soil Liquefaction Triggering Analysis Utilizing Gene Expression Programming and Bayesian Probabilistic Method." *Journal of Rock Mechanics and Geotechnical Engineering* 9 (4): 683–693. doi:10.1016/j.jrmge.2017.03.011.

[11] Hwang, J. H., C. H. Chen, and C. H Juang. 2012. "Calibrating the Model Uncertainty of the HBF Simplified Method for Assessing Liquefaction Potential of Soils." *Sino-Geotechnics* 133: 77–86. https://scholar.google.com/scholar_lookup?title=Calibrating the model uncertainty of the HBF simplified method for assessing liquefaction potential of soils&journal=Sino-geotechnics&volume=133&pages=77-86&publication_year=2012&author=Hwang%2CJH&author=Che.

[12] Hwang, Jin Hung, and Chin Wen Yang. (2001). "Verification of Critical Cyclic Strength Curve by Taiwan Chi-Chi Earthquake Data." *Soil Dynamics and Earthquake Engineering* 21 (3): 237–257. doi:10.1016/S0267-7261(01)00002-1.

[13] Idriss, I M, and R W Boulanger. (2010). "Spt-Based Liquification Triggering Procedures." *Report UCD/CGM-10/02*, no. December: 259. https://faculty.engineering.ucdavis.edu/boulanger/wp-content/uploads/sites/71/2014/09/Idriss_Boulanger_SPT_Liquefaction_CGM-10-02.pdf.

[14] Idriss, I M, and Ross W. Boulanger. (2008). *Soil Liquefaction during Earthquakes*. Oakland, California: Earthquake Engineering Research Institute. http://b-ok.org/dl/1129142/46a2fd.

[15] Idriss, I M, Joseph. I. S. (1992). *User's Manual for SHAKE91. Center for Geotechnical Modeling* (p. 75). Department of Civil Engineering, University of California, Davis.

[16] Idriss, I. M. (1999). An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential. *Proc., TRB Worshop on New Approaches to Liquefaction, Pubbl. n. FHWA-RD-99-165*.

[17] Idriss, I. M., & Boulanger, R. W. (2003a). Estimating Kα for use in evaluating cyclic resistance of sloping ground. *8th US–Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Liquefaction, Report MCEER-03-0003, MCEER*, 449–468.

[18] Idriss, I. M., & Boulanger, R. W. (2003b). Estimating Kα for use in evaluating cyclic resistance of sloping ground. *8th US–Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Liquefaction, Report MCEER-03-0003, MCEER*, 449–468.

[19] Juang, C. H., Chen, C. J., Rosowsky, D. V., & Tang, W. H. (2000b). CPT-based liquefaction analysis, Part 2: Reliability for design. *Geotechnique*. https://doi.org/10.1680/geot.2000.50.5.593

[20] Juang, C. H., Chen, C. J., Tang, W. H., &Rosowsky, D. V. (2000a). CPT-based liquefaction analysis, Part 1: Determination of limit state function. *Géotechnique*, *50*(5), 583–592. https://doi.org/10.1680/geot.2000.50.5.583

[21] Juang, C. H., Rosowsky, D. V., & Tang, W. H. (1999). Reliability-based method for assessing liquefaction potential of soils. *Journal of Geotechnical and Geoenvironmental Engineering*. https://doi.org/10.1061/(ASCE)1090-0241(1999)125:8(684)

[22] Juang, C. Hsein, Caroline J. Chen, Tao Jiang, and Ronald D. Andrus. (2000). "Risk-Based Liquefaction Potential Evaluation Using Standard Penetration Tests." *Canadian Geotechnical Journal* 37 (6): 1195–1208. doi:10.1139/cgj-37-6-1195.

[23] Juang, C. Hsein, Sunny Ye Fang, Wilson H. Tang, Eng Hui Khor, Gordon Tung Chin Kung, and Jie Zhang. (2009). "Evaluating Model Uncertainty of an Spt-Based Simplified Method for Reliability Analysis for Probability of Liquefaction." *Soils and Foundations* 49 (1): 135–152. doi:10.3208/sandf.49.135.

[24] Juang, C., Andrus, R., Jiang, T., & Chen, C. (2001). Probability-based liquefaction evaluation using shear wave velocity measurements. *Proc., 4th Int. Conf. Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, 26–31.

[25] Juang, C., Jiang, T., & Andrus, R. D. (2002). Assessing probability-based methods for liquefaction potential evaluation. *Journal of Geotechnical and Geoenvironmental Engineering*, *128*(7), 580–589. http://ascelibrary.org/doi/abs/10.1061/(ASCE)1090-0241(2002)128:7(580)

[26] Lasley, S. J., Green, R. A., & Rodriguez-Marek, A. (2016). New stress reduction coefficient relationship for liquefaction triggering analyses. *Journal of Geotechnical and Geoenvironmental Engineering*, *142*(11). http://doi.org/10.1061/(ASCE)GT.1943-5606.0001530

[27] Liao, Samson S C, and Robert V Whitman. (1986). "Overburden Correction Factors for SPT in Sand." *Journal of Geotechnical Engineering* 112 (3): 373–377. doi:10.1061/(ASCE)0733-9410(1986)112:3(373).

[28] National Center for Research on Earthquake Engineering (NCREE), National Advanced Project in Hazard Mitigation (NAPHM), and Taiwan Geotechnical Society (GST). (2001). *Geotechnical Reconnaissance Report Ofthe 921 Ji-Ji Earthquake, Taiwan, 1999*.

[29] Singh, P., Kumar, D., & Samui, P. (2020). Reliability analysis of rock slope using soft computing techniques. *Jordan Journal of Civil Engineering*, *14*(1), 2020.

[30] Sebaaly, Graziella T., and Muhsin E. Rahhal. (2019). "Probabilistic Analysis of Soil Liquefaction Based on CPT and SPT Results." In *COMPDYN Proceedings*, 1:141–150. doi:10.7712/120119.6908.19549.

[31] Seed, H B, K Tokimatsu, L F Harder, and R M Chung. (1985). "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations." *Journal of Geotechnical Engineering* 111(12):

1425–45. http://ascelibrary.org/doi/abs/10.1061/(ASCE)0733-9410(1985)111:12(1425).

[32] Seed, H. B. (1979). Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. *Journal of Geotechnical and Geoenvironmental Engineering*, *105*(GT2), 201–255. http://worldcat.org/oclc/3519342

[33] Seed, H. B., & Idriss, I. M. (1971). Simplified procedure for evaluating soil liquefaction potential. *Journal of the Soil Mechanics and Foundations Division*, *97*(9), 1249–1273.

[34] Seed, H. B., & Idriss, I. M. (1982). *Ground motions and soil liquefaction during earthquakes*. Earthquake Engineering Research Institute.

[35] Seed, H. B., Idriss, I. M., & Arango, I. (1983). Evaluation of Liquefaction Potential Using Field Performance Data. *Journal of Geotechnical Engineering*, *109*(3), 458–482. https://doi.org/10.1061/(ASCE)0733-9410(1983)109:3(458)

[36] Seed, H. Bolton. (1984). *The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations*. Report No. UCB/EERC-84/15. Berkeley: University of California,Earthquake Engineering Research Center. http://www.worldcat.org/title/influence-of-spt-procedures-in-soil-liquefaction-resistance-evaluations/oclc/11804853.

[37] Sun, R., Wang, K., & Yuan, X. (2020). Influencing Factors and New Calculation Formulae for the Stress Reduction Coefficient. *Journal of Earthquake Engineering*. http://doi.org/10.1080/13632469.2020.1739172

[38] Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., … Ii, K. H. S. (2001). Liquefaction Resistance of Soils : Summary R Eport From the 1996 Nceer and 1998 Nceer / Nsf Workshops on Evaluation. *Journal of Geotechnical and Geoenvironmental Engineering*, *127*(10), 817–833. https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817)

[39] Youd, T. L., & Noble, S. K. (1997). Magnitude scaling factors. *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, 149–166. https://trid.trb.org/view.aspx?id=542970

Appendix A

Case history data based on SPT test

Table 1: Summary of updated Cetin etal.'s (2016) field performance case history parameters (20 cases retained for validation).

Case	Earth-quake	Site	Liq?	Depth (m)	GWT (m)	σ_v (kPa)	σ'_v (kPa)	a_{max} (g)	CSR	M_w	MSF	K_σ	FC %	N	C_s	c_R	C_B	C_E	C_N	N_{160CS}	CRR_Youd for ml=7,5σ'v =1atm	CRR_{SM}	FS_{SM}	CRR_Adj. for ml=7,5 sigpv0 = 1atm	CRR_{SMC}	FS_{SMC}
6	1964 Niigata	Arayamo-tomachi	Yes	3.3	1	56	34	0.09	0.09	7.6	0.967	1.31	5	4.4	1	0.86	1	1.22	1.72	8.4	0.099	0.126	**1.397**	0.121	0.153	**0.607**
24	1975 Haicheng	Panjin Ch. F. P.	Yes	8	1.5	148	85	0.13	0.13	7	1.193	1.041	67	8.1	1	1	1	0.83	1.09	10.9	0.121	0.151	**1.158**	0.149	0.185	**0.631**
25	1975 Haicheng	Ying Kou G. F. P.	Yes	7	1.5	130	76	0.2	0.2	7	1.193	1.071	48	12.4	1	0.98	1	1	1.16	17.9	0.191	0.244	**1.218**	0.239	0.305	**0.864**
26	1975 Haicheng	Ying Kou P. P.	Yes	7.5	1.5	139	80	0.2	0.18	7	1.193	1.057	20	10.3	1	0.99	1	1	1.12	13.6	0.146	0.185	**1.025**	0.182	0.229	**0.723**
30	1976 Tangshan	Coastal Region	Yes	4.5	1.1	83	50	0.13	0.13	7.6	0.967	1.189	12	9.45	1	0.9	1	1	1.43	13.5	0.145	0.167	**1.285**	0.181	0.208	**0.706**
47	1978 Miyagiken-Oki	Nakamura 4	Yes	4	0.5	75	40	0.12	0.14	6.5	1.442	1.257	5	5.6	1	0.89	1	1	1.59	8.4	0.099	0.180	**1.286**	0.121	0.219	**0.660**
58	1978 Miyagiken-Oki	Hiyori-18	Yes	3.3	2.4	57	49	0.24	0.18	7.7	0.935	1.195	20	9.1	1	0.86	1	1.09	1.43	14.4	0.154	0.172	**0.956**	0.192	0.214	**0.750**
70	1978 Miyagiken-Oki	Yuriage Br-2	Yes	2.4	1.3	43	32	0.24	0.21	7.7	0.935	1.33	7	11.4	1	0.82	1	1.12	1.79	19.5	0.209	0.260	**1.239**	0.262	0.325	**0.976**
81	1979 Imperia' Valley	Radio Tower B1	Yes	4.3	2	72	50	0.18	0.16	6.53	1.426	1.189	43.5	4.45	1	0.86	1	1.13	1.42	9.7	0.110	0.187	**1.171**	0.135	0.229	**0.765**
83	1979 Imperia' Valley	River Park A	Yes	1.1	0.3	18	10	0.16	0.18	6.53	1.426	1.778	91	2.65	1	0.66	1	1.13	2	7.3	0.090	0.228	**1.269**	0.109	0.275	**0.763**
95	1983 Nihonkai-Chubu	Takeda Ele.Sch.	Yes	4.5	0.4	80	40	0.12	0.13	7.1	1.151	1.257	0	7.85	1	0.9	1	1.22	1.6	14.3	0.153	0.222	**1.704**	0.191	0.276	**0.896**
97	1983 Nihonkai-Chubu	Aomori Station	Yes	5.8	0	108	52	0.12	0.14	7.7	0.935	1.178	3	9	1	0.94	1	1.22	1.4	15	0.160	0.176	**1.259**	0.200	0.220	**0.663**

122	1987 Superstition Hills	Wildlife B	Yes	4.7	0.9	86	49	0.2	0.19	6.54	1.42	1.195	26.2	7.85	1	0.88	1	1.13	1.44	14.1	0.151	0.257	**1.350**	0.188	0.319	**0.953**
132	1989 Loma Prieta	P007-2	Yes	6.2	3	118	88	0.22	0.17	6.93	1.224	1.032	3	12.7	1	0.93	1	0.92	1.08	13.3	0.143	0.181	**1.066**	0.178	0.225	**0.794**
134	1989 Loma Prieta	POR-2&3&4	Yes	4.9	3.5	79	65	0.15	0.09	6.93	1.224	1.114	50	3.15	1	0.89	1	0.92	1.24	6.8	0.086	0.117	**1.304**	0.103	0.141	**0.752**
135	1989 Lama Prieta	Sandholdt UC-B10	Yes	3.2	1.7	58	43	0.26	0.22	6.93	1.224	1.235	2	9.15	1	0.81	1	1.25	1.53	14.8	0.158	0.239	**1.086**	0.197	0.298	**0.893**
139	1989 Lama Prieta	Treasure Island	Yes	5.3	1.5	91	55	0.18	0.17	6.93	1.224	1.161	20	5.05	1	0.9	1	1.13	1.36	9.8	0.111	0.158	**0.931**	0.136	0.194	**0.609**
140	1989 Lama Prieta	Wood Marine UC-B4	Yes	1.8	1	31	24	0.25	0.21	6.93	1.224	1.429	35	5.75	1	0.72	1	1	2	11.9	0.130	0.228	**1.085**	0.161	0.282	**0.865**
143	1989 Lama Prieta	Marine Laboratory UC-B2 Yes	Yes	3.5	2.5	63	53	0.26	0.2	6.93	1.224	1.172	3	13	1	0.83	1	1	1.38	15.4	0.164	0.235	**1.177**	0.205	0.294	**0.969**
210	1995 Hyogo ken-Na nbu	Torishima Dike	Yes	4.8	0	86	39	0.25	0.29	6.9	1.238	1.265	20	8.5	1	0.91	1	1.22	1.61	17.5	0.186	0.292	**1.006**	0.233	0.365	**0.812**

CRR_{SM}=CRR_{Youd}*MSF*K_σ; CRR_{SMC}=CRR_{Adj}*MSF*K_σ

CRR_{Youd}: cyclic resistance ratio computed by Youd etal.(2001)

CRR_{Adj}: cyclic resistance ratio computed by the proposed model in this study

FS_{SM}: safety factor computed by the simplified method

FS_{SMC}: safety factor computed by the corrected simplified method

Liq?:Yes= liquefaction; No= no liquefaction