Title

Closure to âStandard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potentialâ by K. Onder Cetin, Raymond B. Seed, Armen Der Kiureghian, Kohji Tokimatsu, Leslie F. Harder Jr., Robert E. Kayen, and Rober...

Journal

Journal of Geotechnical and Geoenvironmental Engineering, 132(5)

ISSN

1090-0241 1943-5606

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et al.

Publication Date

2006-05-01

DOI

10.1061/(ASCE)1090-0241(2006)132:5(667.2)

Peer reviewed

DISCUSSIONS AND CLOSURES

Discussion of "Stability of Long Trenches in Sand Supported by Bentonite-Water Slurry" by George M. Filz, Tiffany Adams, and Richard R. Davidson

September 2004, Vol. 130, No. 9, pp. 915–921.
DOI: 10.1061/(ASCE)1090-0241(2004)130:9(915)

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The authors have presented a valuable study on the stability of long slurry-supported trenches in cohesionless soil. Quite by chance, the discusser published a related paper two months ago (Fox 2004). The focus of that paper is the development of analytical solutions for trench stability under various conditions (e.g., two-dimensional (2D) versus three-dimensional (3D), drained versus undrained, c-ϕ strength), whereas the focus of the current paper is largely on understanding stability mechanisms. The two papers overlap in their analysis of 2D global stability of a slurry-supported trench with a fully formed filter cake in drained cohesionless soil. Both papers provide closed-form equations for the factor of safety and orientation of the critical failure plane [Eqs. (26) and (27) in Fox (2004) and Eqs. (2) in the current paper]. The discusser has verified that the values of factor of safety and critical failure plane angle given by these equations are identical.

As stated by the authors, the 2D global stability equations are appropriate for long trenches without significant end effects. Three-dimensional stability effects that occur for shorter panels may yield substantially higher factors of safety. To illustrate, the discusser calculated the global factor of safety, F_S, for various trench lengths L for the hypothetical example presented in Fig. 8 of the current paper using Eqs. (22) and (23) from Fox (2004).

In the notation used by Fox (2004), the input parameters are as follows: $H=H_s=20$ m, $z_w=3$ m, $z_c=0$ (no tension crack), $\gamma=19\ \mathrm{kN/m^3}$, $\gamma_{sat}=20\ \mathrm{kN/m^3}$, $\gamma_s=11.8\ \mathrm{kN/m^3}$, $c_1'=c_2'=0$, $\phi'=37°$, $q=0$, and $K=K_o=1-\sin\phi'$. Fig. 1 presents F_S versus L as obtained from 3D analyses and the corresponding 2D factor of safety (1.47). F_S is strongly affected by trench length, and the 3D values approach the 2D value as L increases. When $L=H$, the calculated 3D F_S is 27% higher than the 2D value. At $L=50$ m and 100 m, the 3D values are 11% and 5% higher, respectively. If we define a 20% underestimate of the F_S as too conservative, the 2D stability analysis is not too conservative for $L>21.7$ m in this example.

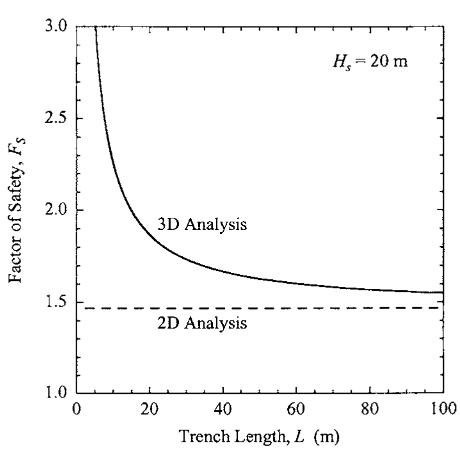

Fig. 1. Variation of 3D global factor of safety with trench length for example problem presented in Fig. 8

The Fox (2004) paper and the current paper are highly complementary and together present a fairly complete picture of limit equilibrium analysis for fully developed filter cake conditions. Unlike the current paper, the Fox paper does not address global stability of slurry trenches in coarse soils without filter cake formation. The development of general analytical solutions for this important case would be a significant and interesting contribution.

Reference

Fox, P. J. (2004). "Analytical solutions for stability of slurry trench," *J. Geotech. Geoenviron. Eng.*, 130(7), 749–758.

Closure to "Stability of Long Trenches in Sand Supported by Bentonite-Water Slurry" by George M. Filz, Tiffany Adams, and Richard R. Davidson

September 2004, Vol. 103, No. 9, pp. 915–921.
DOI: 10.1061/(ASCE)1090-0241(2004)130:9(915)

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The writers appreciate both the discussion by Fox and his previous paper (Fox 2004). Fox makes the important point that three-dimensional (3D) factors of safety are higher than two-dimensional (2D) factors of safety for this type of construction, and he provides useful expressions for taking 3D effects into account. For slurry-supported trenches that are excavated for soil-bentonite cutoff walls, the slurry-supported trench is generally quite long because a cleanout distance is usually provided between the toe of the backfill slope and the excavation face and because the backfill goes into the trench at a relatively shallow slope, which is often within the range of 5H:1V to 10H:1V. For the example analyzed in the paper and by the discusser, if the cleanout distance is 15 m and the backfill slope is 8H:1V, then the average trench length is 95 m. Although the trench length that should be used to evaluate 3D effects for this geometry is not clear at this point, Fig. 1 of the discussion shows that the 3D factor of safety is only about 5% higher than the 2D factor of safety for the average trench length of 95 m. For slurry-supported panels, however, the trench length can be much shorter and 3D effects can become very important, as demonstrated by Fox.

The writers also agree that it would be interesting and useful to develop analytical solutions for factors of safety applicable to cases where filter cakes do not form.

Reference

Fox, P. J. (2004). "Analytical solutions for stability of slurry trench." *J. Geotech. Geoenviron. Eng.*, 130(7), 749–758.

Discussion of "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential" by K. Onder Cetin, Raymond B. Seed, Armen Der Kiureghian, Kohji Tokimatsu, Leslie F. Harder Jr., Robert E. Kayen, and Robert E. S. Moss

December 2004, Vol. 130, No. 12, pp. 1314–1340.
DOI: 10.1061/(ASCE)1090-0241(2004)130:12(1314)

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The writers have presented an excellent and extensive effort to assemble and analyze liquefaction data and arrive at a "new and improved" simplified method of liquefaction evaluation. The discusser, though, is puzzled by the correction for fines content that is expressed by Eq. (14) of the original paper. Eq. (14) is intended to describe the relationship that exists among the three curves shown on Fig. 9(b) of the original paper. The writers state that no correction is applied if the fines content is 5% or less, which is equivalent to a value of 1 for C_{FINES}. However, for a fines content infinitesimally greater than 5%, Eq. (14) yields a value of C_{FINES} ranging from 1.07 to 1.028 for $N_{1,60}$ ranging from 5 to 30. This discontinuity in C_{FINES} at 5% fines content is unnecessary. Further, Eq. (14) does not appear to describe very well the relationship that exists among the three curves shown on Fig. 9(b), especially for fines contents between 5.1 and 10%. Eq. (1) below appears to describe the relationship that exists among the three curves shown on Fig. 9(b) of the original paper better than Eq. (14) does, and it does not invoke a discontinuity at 5% fines content:

$$C_{\mathrm{FINES}} = \left(0.00391 + \frac{0.0382}{N_{1,60}}\right)(\mathrm{FC} - 5) + 1 \tag{1}$$

The meaning of the symbols is the same as in the original paper, and the range of FC is similarly limited to $5 \leqslant \mathrm{FC} \leqslant 35$.

In addition, the definition of C_S in Table 2 in the original paper effectively makes $N_{1,60}$ a function of itself, requiring an iterative solution. Although this can be easily implemented in a spreadsheet, there seems to be little justification for making this refinement, given the apparent rudimentary character of the source of the definition. The discusser believes that C_S could be computed as

$$C_S = 1 + \frac{N_1 \cdot C_R \cdot C_B \cdot C_E \cdot \{1 + 0.01 \cdot \mathrm{MAX}[10, \mathrm{MIN}(30, N_1)]\}}{100} \tag{2}$$

where MIN and MAX are the familiar spreadsheet functions for minimum value and maximum value, respectively, and the meaning of the other symbols is the same as in the original paper. In the preceding equation, the MIN and MAX functions are used to limit the value of the first occurrence of N_1 to the range from 10 to 30. As in the original paper, the final value of C_S should be limited to the range $1.1 \leqslant C_S \leqslant 1.3$.

Closure to "Standard Penetration Test-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential" by K. Onder Cetin, Raymond B. Seed, Armen Der Kiureghian, Kohji Tokimatsu, Leslie F. Harder Jr., Robert E. Kayen, and Robert E. S. Moss

December 2004, Vol. 130, No. 12, pp. 1314–1340.
DOI: 10.1061/(ASCE)1090-0241(2004)130:12(1314)

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The authors appreciate the discusser's interest in our study and his valuable comments. To summarize the discusser has raised three issues:

1. Discontinuity in fines correction, C_{FINES}, for fines content values in the range of 0 to 5%;
2. Difficulties in reproducing Fig. 9(b) from the recommended closed-form solution [i.e., Eq. (20)]; and
3. The iterative nature of SPT nonstandardized sampler configuration correction, C_S; which will be discussed next.

Discontinuous Fines Correction Scheme for $0\% < \mathrm{FC} < 5\%$

The selection of a discontinuous fines correction scheme (i.e., no fines correction is applied on the SPT blow counts if the fines content, FC, is less than 5%), as opposed to a continuous fines correction scheme, has been a well-discussed and well-thought issue among the coauthors and has the following rationale and advantages:

1. The deterministic liquefaction-triggering boundary curve corresponding to FC < 5% has been widely used for assessing the liquefaction-triggering potential of clean sands, which are also consistently defined as sands with 5% or less fines as part of the unified soil classification system and the British soil classification system.
2. Cyclic laboratory test results performed on sands with varying fines content in the range of 0 to 5% have not shown a clear increase trend in cyclic resistance ratio, which could then be attributed to the variations in fines content (e.g., Vaid 1994; Finn et al. 1994; Polito and Martin 2001). Actually, fines content should reach to a threshold value; in our particular case, we that believe the lower and upper limits are 5 and 35%, so that the amount of fines start to play a role in the soil matrix. This observation is also verified with the sand skeleton void-ratio concept discussed by Polito and Martin (2001).
3. The recommended fines correction scheme enables direct comparison of our deterministic seismic soil liquefaction-triggering boundary curves with the widely used curves of Seed et al. (1984, 1985), which have established the state of the practice until the present.
4. Statistical models where fines correction was applied, as presented in our paper and as opposed to a continuous correction scheme, produced better fits (i.e., about 5% greater likelihood values). Similar conclusions were reached by other researchers (e.g., Robertson and Wride 1997) after assessing currently existing liquefaction, nonliquefaction databases.
5. It is consistent with the expert consensus opinion of the NCEER Working Group (1997) and Youd et al. (2001). The fines correction factor recommended by this group, as also discussed in detail by Robertson and Wride (1997), has been presented in a similar and discontinuous form.

Difficulties in Reproducing Fig. 9(b)

After the discusser's comment, CRR values were read from digitized Figs. 9(b) and 14(b) and were also calculated by using the closed-form solution [Eq. (20)], where FC values were entered as 35, 15, and 5%. The comparisons are presented in Table 1. The differences in CRR values read from the corresponding figures and the values calculated by the closed-form solution are insignificantly small (in the range of 1 to 4%) and are completely attributable to errors in the digitization process. However, if a value of 0% for FC is entered, then the differences are slightly higher and are calculated as being in the range of 3–6%. The question then is whether FC should be entered as 0 or 5% in Eq. (20). To be able to answer this question, we would like to discuss the reason behind developing Fig. 9(b). Fig. 9(b) was developed to enable a direct comparison with the Seed et al. (1984) relationship; an FC value of 5% was therefore entered in developing our FC=5% curve so that it would be compatible with the Seed et al. (1984) methodology. Also, Fig. 9(b) cannot be used directly (or even indirectly by applying K_σ corrections, which are developed for a reference vertical effective stress of 1 atm.) to assess the liquefaction initiation potential unless vertical effective stress at the soil layer of interest is exactly equal to 0.65 kPa. Thus, digitizing it for direct use in liquefaction initiation assessments could be misleading. However, we do accept the fact that the CRR curve labels used in Figs. 9(b) and 14(b) for FC less than 5% are misleading to readers. As a conclusion, to eliminate the potential loss of accuracy during digitization and possible confusion regarding K_σ corrections, we recommend direct use of Eq. (20). To clarify, in Eq. (20), if FC is equal to 5%, we recommend that a value of 5% be entered in the expression, as defined by the limits of $5\% \leq \mathrm{FC} \leq 35\%$ defined in the paper. For values of FC less than 5%, a value of 0% should be substituted into the expression.

Table 1. Comparisons between Calculated and Digitized CRR Values

$N_{1,60}$	FC	M_w	$\sigma_{v'}$ (atm.)	P_L	$\mathrm{CRR}_{\mathrm{Eq.20}}$	$\mathrm{CRR}_{\mathrm{Digitized}}$	% difference
17.64	5	7.5	0.65	0.15	0.146	0.148	1.1
19.93	5	7.5	0.65	0.15	0.174	0.175	0.3
21.84	5	7.5	0.65	0.15	0.202	0.201	0.4
23.48	5	7.5	0.65	0.15	0.229	0.230	0.7
28.66	5	7.5	0.65	0.15	0.340	0.338	0.4
29.97	5	7.5	0.65	0.15	0.376	0.373	0.7
30.75	5	7.5	0.65	0.15	0.399	0.398	0.2
31.93	5	7.5	0.65	0.15	0.437	0.433	0.9
33.38	5	7.5	0.65	0.15	0.488	0.483	1.1
17.64	0	7.5	0.65	0.15	0.140	0.148	5.4
19.93	0	7.5	0.65	0.15	0.166	0.175	5.0
21.84	0	7.5	0.65	0.15	0.191	0.201	4.6
23.48	0	7.5	0.65	0.15	0.217	0.230	5.9
28.66	0	7.5	0.65	0.15	0.319	0.338	5.6
29.97	0	7.5	0.65	0.15	0.353	0.373	5.6
30.75	0	7.5	0.65	0.15	0.374	0.398	6.1
31.93	0	7.5	0.65	0.15	0.409	0.433	5.6
33.38	0	7.5	0.65	0.15	0.455	0.483	5.7
11.93	15	7.5	0.65	0.15	0.102	0.106	4.2
13.85	15	7.5	0.65	0.15	0.118	0.124	4.3
16.97	15	7.5	0.65	0.15	0.152	0.156	2.6
18.11	15	7.5	0.65	0.15	0.166	0.171	3.0
20.75	15	7.5	0.65	0.15	0.205	0.208	1.3
22.55	15	7.5	0.65	0.15	0.237	0.239	0.9
23.87	15	7.5	0.65	0.15	0.263	0.263	0.1
27.53	15	7.5	0.65	0.15	0.351	0.350	0.3
28.85	15	7.5	0.65	0.15	0.390	0.390	0.1
30.40	15	7.5	0.65	0.15	0.442	0.439	0.8
31.60	15	7.5	0.65	0.15	0.486	0.483	0.7
33.10	15	7.5	0.65	0.15	0.548	0.543	1.0
34.42	15	7.5	0.65	0.15	0.609	0.600	1.4
5.16	35	7.5	0.65	0.15	0.066	0.066	0.8
7.86	35	7.5	0.65	0.15	0.083	0.086	3.8
10.31	35	7.5	0.65	0.15	0.102	0.106	3.6
11.87	35	7.5	0.65	0.15	0.117	0.121	3.0
16.79	35	7.5	0.65	0.15	0.178	0.182	1.8
19.67	35	7.5	0.65	0.15	0.228	0.229	0.4
22.25	35	7.5	0.65	0.15	0.285	0.283	0.4
24.05	35	7.5	0.65	0.15	0.332	0.329	1.0
25.31	35	7.5	0.65	0.15	0.370	0.369	0.3
26.99	35	7.5	0.65	0.15	0.427	0.426	0.2
28.37	35	7.5	0.65	0.15	0.480	0.478	0.4
29.75	35	7.5	0.65	0.15	0.541	0.535	1.1
31.12	35	7.5	0.65	0.15	0.608	0.599	1.6

Table 2. Comparisons among C_S values

N_1	C_R	C_B	C_E	$C_{S,\text{Luebbers}}$	$C_{S,\text{Cetin etal.}}$	$N_{1,60,\text{Luebbers}}$	$N_{1,60,\text{Cetin etal.}}$
0	1	1	1	1.10	1.10	0.00	0.00
5	1	1	1	1.10	1.10	5.50	5.50
10	1	1	1	1.11	1.11	11.10	11.11
15	1	1	1	1.17	1.18	17.59	17.65
20	1	1	1	1.24	1.25	24.80	25.00
25	1	1	1	1.30	1.30	32.50	32.50
30	1	1	1	1.30	1.30	39.00	39.00
35	1	1	1	1.30	1.30	45.50	45.50
40	1	1	1	1.30	1.30	52.00	52.00
45	1	1	1	1.30	1.30	58.50	58.50
50	1	1	1	1.30	1.30	65.00	65.00

Iterative Nature of SPT Nonstandardized Sampler Configuration Correction, C_S

The discusser's observation regarding the iterative nature of SPT nonstandardized sampler configuration correction, C_S, presented in Eq. (T2) of Table 2, is valid. If a nonstandard sampler (i.e., the sampler with a room for liner without the liner in place) is used during standard penetration testing, which is uncommon, an iterative correction scheme, as presented in Eq. (T2) can be used. As shown in Table 2, the alternative formulation suggested by the discusser also produces similar C_s values as compared with our recommended Eq. (T2) and has the advantage of being noniterative. Thus, it can be conveniently used in the calculation of C_s values by eliminating the inconveniences caused by the iterative nature of our recommended expression without loss of accuracy.

Acknowledgments

Funding for these studies was provided by PG&E, through the PG&E/PEER Center Research Program, under Contract No. Z-19-2-133, and this support is gratefully acknowledged. The writers also wish to thank M. J. Bennett and Drs. J. C. Tinsley, S. Toprak, and T. L. Holzer, all from the U.S. Geological Survey (USGS), for their assistance in collecting and interpreting case histories and for their valuable suggestions. Similarly, the authors are grateful to Dr. T. L. Youd both for his assistance in obtaining and evaluating case history data and for his insight and comments. The writers are also grateful to Dr. Anne Kammerer for useful discussion and checking of the new correlations proposed, as well as assistance in representing these in a tractable form for use by working engineers.

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Discussion of "Discriminant Model for Evaluating Soil Liquefaction Potential Using Cone Penetration Test Data" by Sheng-Yao Lai, Sung-Chi Hsu, and Ming-Jyh Hsieh

December 2004, Vol. 130, No. 12, pp. 1271–1282.
DOI: 10.1061/(ASCE)1090-0241(2004)130:12(1271)

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As the authors point out, liquefaction potential is commonly assessed by assembling a database of field performance case histories from several sites of previous earthquakes and observed cone penetration test (CPT) or standard penetration test (SPT) penetration resistance from these sites. Liquefaction potential is assessed by plotting the penetration resistance from the critical soil layers at these sites against the estimated cyclic stress ratio (*CSR*) related to the earthquakes considered and drawing a threshold separating the data points representing occurrence and nonoccurrence of liquefaction. Carrying out the CPT or the SPT after the earthquake is a common practice, as with the database used by the authors in their paper, with a tacit assumption that the postearthquake in situ penetration resistance reflects the pre-earthquake undrained cyclic soil strength. The assumption may not be appropriate (Chameau et al. 1991).

An alternative procedure for assessing liquefaction potential is based on a correlation between the cyclic undrained soil strength obtained from laboratory testing of undisturbed soil samples and in situ penetration resistance. As the authors suggest, this procedure is of greater technical merit than the framework that is based on field observations. Because of the expense involved in obtaining an undisturbed sample of cohesionless soil, this approach has not been used frequently. Nevertheless, a large amount of cyclic undrained test data from testing undisturbed (frozen) samples has accumulated from research initiatives across the world (e.g., Porcino and Ghionna 2004; Ito et al. 1999; Matsuo and Tsutsumi 1998; Hatanaka and Uchida 1998; Tanaka and Tanaka 1998; Ishihara et al. 1996; Tanaka et al. 1996; Vaid et al. 1996; Hatanaka et al. 1995; Suzuki et al. 1993; Yoshimi et al. 1984), against which the proposed procedure may be benchmarked.

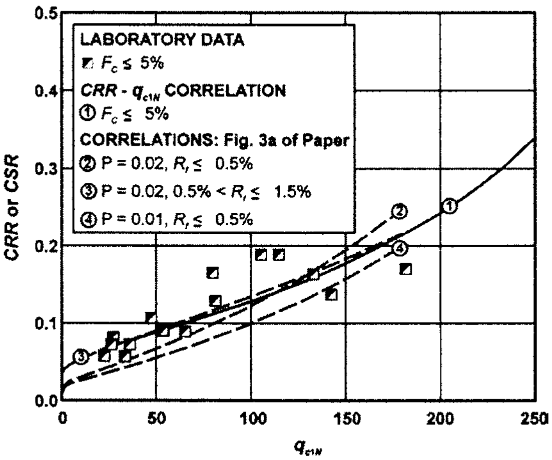

Fig. 1. Correlation between CSR or CRR and q_{ein}

The cyclic resistance ratio (CRR) from tests on undisturbed samples reported in the references cited for soils with fines content of up to 5% corresponding to M7.5 earthquakes is presented in Fig. 1 together with the normalized cone tip resistance, q_{c1}, from near the sampling locations. The friction ratios for these CPT measurements were up to 1.5%. A CRR-q_{c1N} correlation apparent from these data is also plotted in Fig. 1. Comparison of this correlation with the authors' CRR-q_{c1N} relationships (Fig. 3 of the paper) indicates that the correlation obtained from laboratory tests on undisturbed samples compares reasonably with the low "P"-level CSR-q_{c1N} relationships of the authors (Fig. 8). Ideally, the CRR-q_{c1N} correlation should have been comparable with the CSR-q_{c1N} relationship of the authors representing $C(P)=0$. This inconsistency could be attributable to (a) postearthquake penetration resistances not being representative of the pre-earthquake soil strength and (b) scarcity of case histories involving liquefaction at sites characterized by $q_{c1N} \geqslant 120$.

Interestingly, the family of CSR-q_{c1N} relationships proposed by the authors consistently diverges from the deterministic relationships used in practice (e.g., Youd et al. 2001), especially for $q_{c1N} \geqslant 120$. A possible reason for this divergence is the scarcity of field performance data from sites with such high values of q_{c1N}. However, the trend of the authors' CSR-q_{c1N} relationships is in agreement with the trend of the CRR-q_{c1N} correlation obtained from laboratory tests of undisturbed samples.

Further, existing frameworks for assessing liquefaction potential on the basis of q_{c1N} generally suffer from the limitation that they do not account for soil compressibility in an elegant manner, although soil compressibility is one of the key factors that influence liquefaction susceptibility. The procedure developed by the authors is no exception.

Finally, a housekeeping point: If occurrence of liquefaction where nonoccurrence is predicted *and* occurrence of liquefaction where nonoccurrence is predicted are both considered as misclassification, the "P" level of the authors does not represent probability of misclassification. Rather, the "P" level, as it has been implemented in the paper, represents the probability of triggering liquefaction.

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Closure to "Discriminant Model for Evaluating Soil Liquefaction Potential Using Cone Penetration Test Data" by Sheng-Yao Lai, Sung-Chi Hsu, and Ming-Jyh Hsieh

December 2004, Vol. 130, No. 12, pp. 1271–1282.
DOI: 10.1061/(ASCE)1090-0241(2004)130:12(1271)

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The writers appreciate the discusser's interest in the paper, as well as his comments and insightful discussion. The questions raised by the discusser are explained as follows:

1. The selections of the in situ liquefaction and nonliquefaction cases in the paper were based on the assumption that postearthquake in situ penetration resistance reflects the pre-earthquake undrained cyclic strength. However, the discusser did not think that assumption was appropriate and thought that the cyclic strength of the soil might be different during and after an earthquake. The writers believe that the impact on cyclic strength is limited. The soils were disturbed during an earthquake, and their fabric was changed, which may cause the cyclic strength to decrease. However, the relative density and shear strength of the soils may increase after the earthquake because of the effect of compaction and settlement. An increase or decrease of the cyclic strength of the soils will therefore depend on which of the preceding effects has dominated impact. According to Lee et al. (2001), investigation results on Chang-Hua Coastal Industrial Park after the 1999 Chi-Chi earthquake in Taiwan indicated that the loose sand became denser whereas dense sand became looser after the earthquake. However, as reported in the literature, in some cases loose sand may become looser after an earthquake. No conclusion about whether the cyclic strength of the soils will increase or decrease after an earthquake is yet possible. In addition, most researchers who used in situ liquefaction and nonliquefaction cases to build the liquefaction resistance curve did not consider the effect of changes of cyclic strength.
2. The writers also agree that the discusser's opinion about the laboratory testing of undisturbed (frozen) samples is of greater technical merit than the framework that is based on field observations. However, the cyclic shear strength of the soil may change because of soil expansion and change of contacts between particles during freezing and thawing processes. The testing results from several researchers also showed some variation, especially for soils with high relative density, SPT-N value, and CPT-q_c (e.g., Ishihara et al. 1985; Yoshimi et al. 1994; Hatanak et al. 1995). The writers are thankful for the data collected by the discusser, the comparison plot between the laboratory testing data for FC$\leqslant$5%, and the obtained relationship curve of CSR-q_{c1N}. The results show that some of the data did not match the proposed curves very well; however, the data and the correlation obtained from laboratory tests on undisturbed samples compare reasonably with the low "P"-level of the CSR-q_{c1N} relationships suggested in the paper. The trends of the regression curve from the testing results and the relationship curves in Fig. 3 of the paper are very consistent. Thus, the discusser may indicate that including Fig. 1 of the discussion in the paper will implement the paper's integrity. The reason that the tested soils have lower cyclic strength, or CRR, than the suggested values from the relationship curves, especially for soils with $q_{c1N}\geqslant 120$, may be attributable to the expansion consequence during the freezing process. According to the study by Yoshimi et al. (1978), the degrees of volume expansion are proportional to the time of freeze and the relative density of the soil. For these higher-strength soils, therefore, the freezing and thawing processes may have more influence on volume expansion and changes of microfabric.
3. The discusser also mentioned that soil compressibility is one of the key factors that influence liquefaction susceptibility. This is also true for the procedure proposed in the paper, but the analyses did not really consider soil compressibility in the study. In the paper, to investigate the impact of soil types on cyclic strength, the soils were classified into four types for analyses according to the sleeve friction ratio, R_f. The effect of soil compressibility is considered implicitly through the cyclic strength, since soil compressibility is related to the types of the soils. However, the influence of soil plasticity, which has more impact on soil compressibility, on cyclic strength was also not measured in the paper. Only a few studies, e.g., Ishihara (1993), discuss the effect of the plasticity index (PI) of fines content (FC) on cyclic strength. Therefore, not many research results are now available for further study.
4. The question remains about whether if the "P" level in the paper best represents the probability of misclassification or the probability triggering liquefaction. An example of liquefaction assessment is presented in the following.

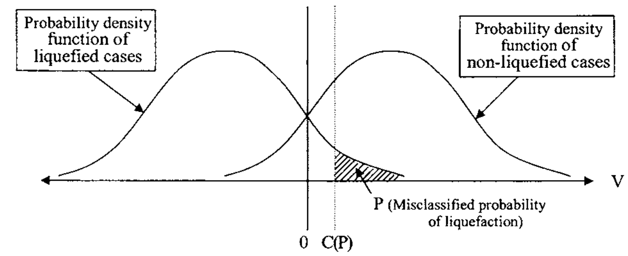

Fig. 1. The schematic diagram of discriminant analysis on liquefaction

Discriminant analysis of liquefaction is performed to decide whether a location can be categorized as a liquefiable or nonliquefiable site. Assuming that both the liquefied and nonliquefied data have multivariate probability distributions, when the misclassified probability of liquefaction is equivalent to the misclassified probability of nonliquefaction, the boundary becomes the best position for a boundary line. The misclassified probability of liquefaction refers to the probability that is misclassified as nonliquefiable when it in fact liquefies. However, the misclassified probability of nonliquefaction means the probability that is misclassified as liquefiable when it in fact does not liquefy.

If the multivariate probability distributions of the liquefied and nonliquefied data could be mapped into two-dimensional space, as shown in Fig. 1, where V=discriminant parameter with $-\infty < V < \infty$, then $V \geqslant 0$ indicates nonliquefaction, whereas $V<0$ indicates liquefaction. P is not the probability of liquefaction, as indicated in the shaded tails in Fig. 1; however, it is the misclassified probability of liquefaction and can be obtained by integrating the $C(P)$ to positive infinity, ∞. $C(P)$, however, located on the positive x-coordinate as shown in Fig. 1, is a more conservative liquefied discriminant value. When $C(P)=0$, the probability of misclassification is equal for the liquefaction and nonliquefaction situations, which is the ideal discriminant boundary.

Errata:

The following corrections should be made to the original paper: Fig. 3b should be switched with Fig. 3c.

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