Comparison of liquefaction triggering curves with laboratory and in-situ tests

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ABSTRACT
Liquefaction of saturated loose cohesionless soil deposits has often been the culprit for ground failures and damage to built structures during past earthquakes. Liquefaction analysis and design against liquefaction occurring involves comparing cyclic shear stress imposed during an earthquake with the cyclic shearing resistance of a saturated soil deposit. Many difficulties and the enormous cost associated with undisturbed sampling of saturated cohesionless soils generally prohibit the using of laboratory experiments for measuring soil cyclic shear strength. Accordingly, in engineering practice cyclic resistance of in-situ soil deposits are indirectly determined from liquefaction triggering curves with Standard Penetration (SPT) or Cone Penetration (CPT) resistance. The liquefaction triggering curves are established as boundary lines between field cases where evidences of liquefaction were observed and those where liquefaction did not appear on the ground surface. In this study, a database of SPT and CPT penetration resistances are collected at soil depths from which high-quality frozen samples were retrieved for cyclic triaxial testing. Cyclic resistances of soil specimens are corrected for the effects of differences in stress anisotropy, multidirectional shearing and mode of shearing between laboratory triaxial testing and field conditions. The comparison of these data with field-based liquefaction triggering boundaries suggests that the current approach used in engineering practice might overestimate the cyclic resistance of clean sands and silty sands using SPT blow counts, or overestimate those of silty sands using cone tip resistance.

1 INTRODUCTION

Liquefaction of saturated conhesionless soils is a typical example of strength loss caused by the application of cyclic shear stresses. Assessment of soil liquefaction potential requires the determination of soil cyclic resistance to liquefaction. This is often defined by cyclic resistance of soil (τ) normalized by the initial effective vertical stress (σ'_{vo}). Liquefaction could occur at a given depth when the cyclic stress applied by an earthquake exceeds cyclic resistance of soil.

Due to the difficulty in sampling and recovering undisturbed samples of conhesionless soils, resistance to triggering liquefaction is commonly determined from in-situ penetration resistance tests together with field experience during past earthquakes. This indirect approach was originally developed by Whitman (1971). Seed et al. (1985) extended this method as a liquefaction triggering chart for an earthquake magnitude of 7.5 using surficial observations (e.g., sand boils and ground cracks) of liquefaction occurring during past earthquakes where SPT values were available. The field data have been primarily for Holocene-age, clean sand deposits of level or gently sloping ground. As demonstrated in Figure 1, the line separating liquefied data from non-liquefied sites is used as a measure of the cyclic resistance ratio (CRR) of a soil. The cone penetration test (CPT) offers several advantages over the standard penetration test (SPT) for liquefaction assessment as it is more economical to perform, and provides a nearly continuous record of more reproducible penetration resistance throughout a soil deposit and a hence better description of soil variability. Thus, CPT has received increasing interest in recent years, and many empirical correlations using CPT data have been developed for clean sands as well as for silty sands (Ishihara 1985; Shibata and Teparaska 1988; Robertson and Wride, 1998, Seed and De Alba, 1986, Stark and Olson, 1995).

Despite the aforementioned developments in identifying a liquefaction triggering boundary from the evidences of liquefaction occurrence, there can be a significant uncertainty in using this approach, including the assessment of whether a soil deposit has liquefied, extent of liquefaction, the specific zone within a soil deposit where liquefaction occurred, and the estimation of cyclic shear stress applied by an earthquake. Many of the existing CPT-based liquefaction triggering charts are developed by using empirical correlations to convert SPT blow counts to CPT resistances (Seed et al. 1983; Seed and De Alba 1986; Robertson and Campanella 1985; Robertson and Wride 1998). Such correlations often involve wide scatter which would produce large inaccuracy in developing CPT-based liquefaction correlations.

Cyclic resistance can be alternatively determined by carrying out cyclic shear tests on saturated sand samples. However, sample preparation method and soil fabric play an important role on the cyclic behavior and liquefaction resistance of cohesionless soils. Several studies have indicated that the liquefaction resistance of undisturbed specimens taken by ground freezing can be significantly different than that of reconstituted specimens at similar densities (Hatanaka, et al., 1995, Ishihara, 1993, Tokimatsu and Hosaka, 1986). Sample disturbance using a conventional tube sampler could also loosen a dense sand due to dilation or densify a loose sand by consolidation and destroy interparticle contacts, resulting in an overall increase of the cyclic strength for loose sands (Ladd, 1974, Mulilis, et al., 1975, Singh, et al., 1979, Stark, et al., 2011). Therefore, high-quality undisturbed samples collected by ground freezing, which best represent in-situ soil fabric need to be used.

The objective of this study is to examine correlations between CRR and SPT or CPT resistance, based on in-situ tests performed at sites where in-situ freezing sampling is made. The liquefaction resistance data from in-situ frozen samples are compared with empirical liquefaction triggering curves.

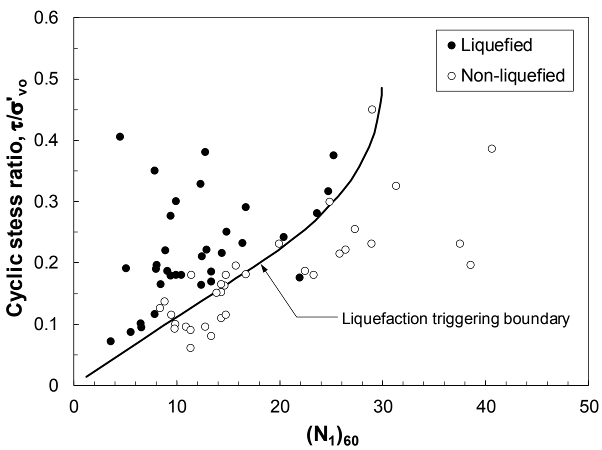


Figure 1. Liquefaction assessment chart of Seed et al. (1985) for M = 7.5 earthquakes, σ'_{vo} = 100 kPa, and FC ≤ 5%

2 DATA COLLECTION AND INTERPRETATION

Tables 1 and 2 summarize the experimental data collected in this study from previous laboratory investigations (Chen and You, 2003, Hatanaka, et al., 1985, Huang, et al., 1999, Huang, et al., 2009, Huang, et al., 2004, Ishihara and Koga, 1981, Kokusho, et al., 2011, Mimura, 2003, Mimura and Suzaki, 2001, Mori and Ishihara, 1979, Morioka, 1999, Suzuki, et al., 1995, Taylor, et al., 2015, Yoshimi, et al., 1989, Yoshimi, et al., 1994). In order to preserve soil fabric, cyclic strength of soil samples is determined from laboratory cyclic triaxial tests on high-quality frozen samples of saturated cohesionless soil obtained from below groundwater. In these experiments, after thawing the specimens were isotropically consolidated to an effective confining stress of equivalent to the in-situ overburden pressure and cyclic triaxial tests were carried out under undrained conditions. For the cyclic triaxial tests collected here, liquefaction is determined as the time at which the pore water pressure becomes equal to the initial effective confining pressure or when a sample experiences a double-amplitude axial strain level of 5% (Ishihara, 1993). Cyclic resistance ratio (CRR) is defined as one-half of the ratio of deviatoric stress to the initial effective confining pressure (σ'_c) required to produce liquefaction in 15 cycles of uniform loading, which corresponds to an equivalent earthquake magnitude of 7.5. CRR for any other earthquake magnitude can be estimated using $CRR_M = MSF \times CRR_{7.5}$, where MSF is a magnitude scaling factor (Robertson and Wride, 1998).

Table 1. Summary of data from in-situ SPTs and cyclic triaxial tests on undisturbed samples

FC (%)	σ'_{vo} (kPa)	D_r (%)	$(N_1)_{60}$ (bpf)	CRR	Refs*
0	20	40	13.9	0.204	(2)
0	33	58	16.6	0.22	(2)
0	78	54	23	0.288	(4)
0	78	54	23.1	0.37	(4)
0	80	30	7.4	0.138	(1)
0	84	64	17.4	0.201	(3)
0	85	50	17.1	0.204	(3)
0	92	52	18.4	0.202	(3)
0	98	64	23.6	0.27	(4)
0	98	64	23.6	0.281	(4)
0	110	56	14.9	0.179	(3)
0	118	52	24.3	0.394	(5)
0	120	51	13.9	0.218	(3)
1	88	N/A	20.6	0.325	(6)
1	92	N/A	21.5	0.353	(6)
1	92	26	4.1	0.13	(4)
1	95	N/A	21.3	0.325	(6)
2	47	70	14.4	0.247	(2)
2	98	58	23.2	0.392	(5)
4	80.4	69	25.5	0.458	(5)
13	89	63	21.8	0.369	(7)
14	89	55	20.3	0.36	(7)
16	123	N/A	20.1	0.322	(8)
19	95	70	16.4	0.321	(7)
30	102	N/A	10.5	0.236	(6)
30	103	N/A	9.7	0.247	(6)
30	112	N/A	6.4	0.227	(6)
51	236	N/A	15.2	0.266	(8)
52	183	N/A	16.5	0.205	(8)
54	210	N/A	16.2	0.21	(8)
57	248	N/A	14	0.181	(8)
0	20	40	13.9	0.204	(2)
0	33	58	16.6	0.22	(2)

** (1) Mimura and Suzaki (2001); (2) Mori and Ishihara (1979); (3) Ishihara and Koga (1981); (4) Yoshimi et al. (1989, 1994); (5) Mimura (2003); (6) Suzuki et al. (1995); (7) Hatanaka et al. (1985); (8) Chen and You (2003)*

Laboratory measurements indicate that CRR decreases non-linearly with increasing effective confining pressure (Seed and Harder, 1990, Vaid, et al., 1985). For soil elements at depths with σ'_{vo} different than about 100 kPa, an empirical correction factor (K_σ) is commonly applied to account for the changes in CRR (Seed and Harder, 1990). However, this could be complicated by the changes in relative density (D_r), fines content (FC), aging, stress history, and soil fabric (Hynes and Olsen, 1999). Therefore, to minimize uncertainty and limit bias regarding the effect of σ'_{vc} on CRR and K_σ, laboratory tests on specimens consolidated to σ'_{vc} = 75 – 125 kPa are only considered in this study.

Table 2. Summary of data from in-situ CPTs and cyclic triaxial tests on undisturbed samples

FC (%)	σ'vo (kPa)	Dr (%)	qc1 (MPa)	CRR	Refs*
0	80	30	4.6	0.138	(2)
0	84	64	7.1	0.201	(3)
0	85	50	6.9	0.204	(3)
0	92	52	6.9	0.202	(3)
0	98	30	3.1	0.124	(1)
0	98	50	4.6	0.162	(1)
0	98	70	7.5	0.206	(1)
0	110	56	5.2	0.179	(3)
0	118	52	12.6	0.394	(4)
0	120	50	7.5	0.218	(3)
1	88	N/A	12.2	0.325	(5)
1	95	N/A	10.9	0.325	(5)
1	100	50	5.1	0.157	(6)
1	100	40	5.5	0.147	(6)
2	98	58	13.4	0.392	(4)
4	80	69	13.7	0.458	(4)
5	98	30	1.4	0.1	(1)
5	98	50	3.5	0.135	(1)
5	98	70	5.9	0.169	(1)
10	98	50	2.3	0.115	(1)
10	98	70	2.3	0.101	(1)
18	111	83	7	0.295	(7)
20	98	30	0.4	0.093	(1)
20	98	50	0.7	0.096	(1)
20	98	70	1.2	0.103	(1)
20	N/A	70	6.5	0.235	(8)
30	98	50	0.6	0.092	(1)
30	102	N/A	5.9	0.236	(5)
30	103	N/A	5.9	0.247	(5)
30	112	N/A	5.6	0.227	(5)
43	47	81	7.1	0.269	(7)

** (1) Kokusho et al. (2011), (2) Mimura and Suzaki (2001); (3) Ishihara and Koga (1981); (4) Mimura (2003); (5) Suzuki et al. (1995); (6) Morioka (1999); (7) Huang et al. (1999, 2004, 2009); (8) Taylor et al. (2015)*

Both SPT and CPT measurements are normalized for the effect of overburden pressure (Boulanger, 2003, Boulanger and Idriss, 2014). SPT blow counts are also converted to an energy level of 60% of the theoretical free-fall hammer energy on the drill stem (N_{60}). Note that in Japan, SPT is often carried out using the trip (tonbi) method with a donut hammer. Since no direct energy measurements were available, the rod energy ratio for this method was assumed to be 78% (Seed, et al., 1985). Hence, SPT blow counts from Japanese studies are increased by 30% (i.e. energy ratio of 78/60) to represent N_{60}.

3 RESULTS

Figure 2 presents SPT and CPT data versus CRR_{txc} from the cyclic triaxial tests of Tables 1 and 2. The data are grouped into different ranges of fines contents (FC). Despite some scatter, these data display distinct exponential trends of CRR-$(N_1)_{60}$ and CRR-q_{c1} for clean (FC < 5%) and silty sands (10% < FC < 50%). These are shown by the solid and dashed lines respectively for FC < 5% and 10% < FC < 50% in Figure 2. The amount of scatter is probably associated with the variability of SPT (compared to CPT), and disregarding the fundamental effect of cyclic strain in producing liquefaction. Furthermore, dynamic driving of the split-spoon sampler or cone penetration drives the in-situ soil to large deformations, whereas liquefaction is a medium strain (≤ 5%) phenomenon. Therefore, a simple correlation may not necessarily exist between these two measurements.

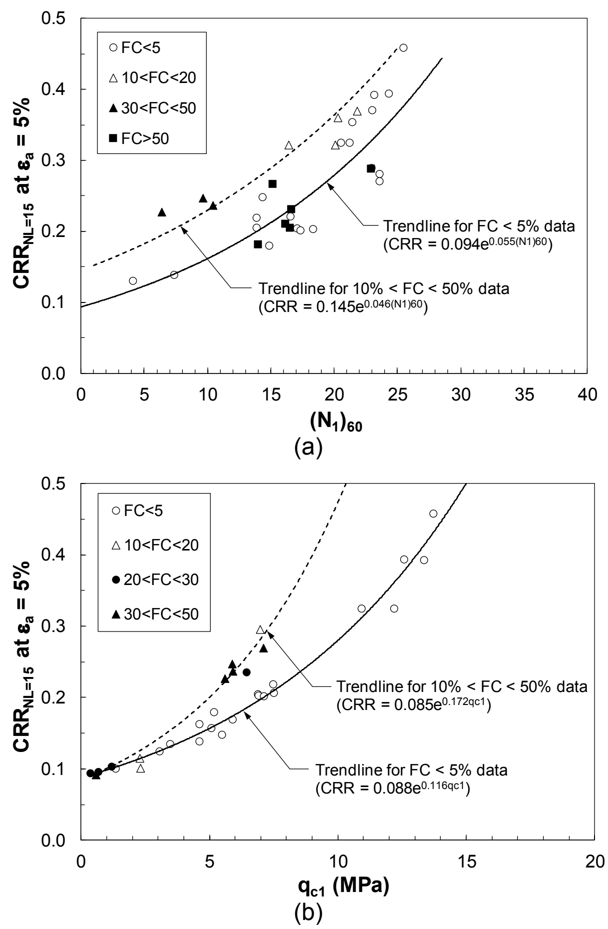


Figure 2. Plots of cyclic resistance ratio for liquefaction triggering in 15 uniform cycles from the cyclic triaxial tests of Tables 1 and 2 versus (a) $(N_1)_{60}$, and (b) q_{c1}

Per Figure 2, CRR is generally the lowest for clean sands (containing less than 5% fines passing No. 200 sieve) for a given SPT or CPT resistance, indicating clean sands' high susceptibility to liquefaction. CRR increases with increasing FC up to a certain limit (at about 40 - 50%) after which the effect of FC > 50% is reversed for sandy silts in Figure 2a. Similar effect of fines content is also reported by other investigators (Polito and Martin 2001; Green et al. 2006). For the same CRR, SPT or CPT penetration resistance in silty sands is also smaller because of the greater compressibility and lower permeability of silty sands. By impeding free drainage (i.e. partial drainage), fine particles reduce $(N_1)_{60}$ or q_{c1} and hence shift the CRR correlations to the left. The reversing effect of FC (in Fig. 2a) likely results from the gradual change in soil fabric with increasing silt content and the dominant effect of fine particles as FC exceeds 50%. This is not observed in Figure 2b as the collected database lacks CPT measurement in soils with FC > 50%.

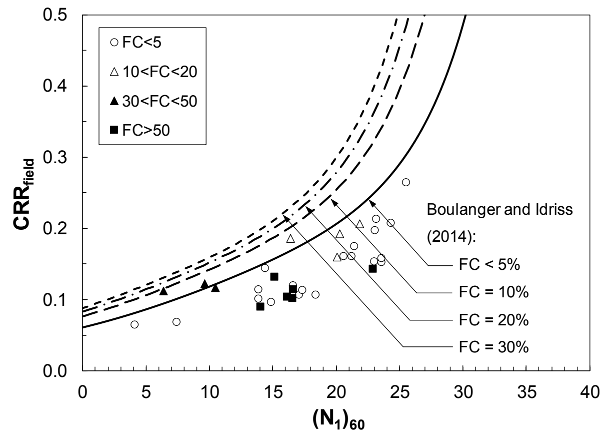
4 COMPARISON WITH LIQUEFACTION TRIGGERING BOUNDARIES

In order to compare CRR from the cyclic triaxial tests of Tables 1 and 2 with field-based liquefaction triggering boundaries, the effects of differences in mode of shearing, stress anisotropy, multidirectional shearing, and equipment limitations need to be considered. The initial stress anisotropy (K_c) can have a large impact on the cyclic resistance and liquefaction behavior of saturated sands (Finn, et al., 1971, Ishibashi and Sherif, 1974, Seed and Peacock 1971). Undrained shearing response of undisturbed samples is also influenced by the direction of loading, with triaxial compression being considerably stronger than triaxial extension. Simple shear response is generally intermediate between compression and extension responses (Hanzawa, 1980, Vaid and Sivathayalan, 1996, Yoshimine, 1996) and it provides a better replicate of the in-situ mode of shearing (Yoshimine, et al., 1998). The effects of these phenomena are accounted for by correcting CRR_{txc} from cyclic triaxial tests as below:

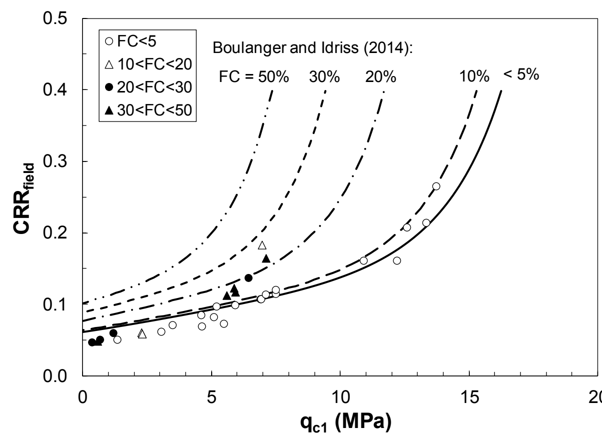
$$CRR_{field} = C_b C_r CRR_{txc} \quad [1]$$

where CRR_{field} represents the cyclic stress ratio for an in-situ soil under level ground conditions (K_c = 0.4) and at the same relative density as CRR_{txc}. The conversion factor (C_r) for converting CRR from cyclic triaxial shear to cyclic simple shearing conditions tends to increase with increasing relative density from 0.55 at D_r = 40% to 0.77 at D_r = 85% (Seed and Peacock 1971). A reduction factor of C_b = 0.9 is also considered to correct for the greater pore pressure developed under multidirectional shearing (Ishihara and Yamazaki, 1980, Seed, et al., 1978).

Figure 3 compares CRR_{field} computed from Equation 1 for the cyclic triaxial tests of Tables 1 and 2 with field-based liquefaction triggering boundaries (Boulanger and Idriss, 2014). According to these plots, cyclic resistance determined from the triaxial tests agree well only with the CPT-based liquefaction triggering curve for clean sands (FC < 5). However, the liquefaction triggering boundaries from SPT blow counts or those from CPT measurements for silty sands (FC = 10%, 20%, 30%, and 50%) are located above the corrected CRR from the cyclic triaxial tests on undisturbed samples. In other words, using the current liquefaction triggering boundaries one would overestimate the liquefaction resistance of sands and silty sands from $(N_1)_{60}$ or the liquefaction resistance of silty sands from q_{c1} measurements. As discussed earlier, the empirical liquefaction triggering boundaries were originally established from the manifestations of liquefaction at the ground surface (e.g. sand boils, lateral spreading, fissures, ground surface disruptions, etc.). Liquefaction resulting from a deep cohesionless soil layer, a medium-dense sand (with high SPT or CPT resistance), or a weak ground motion (low CSR) may not produce any surface manifestations. Therefore, it is plausible that the occurrence of liquefaction may have not been correctly captured in some case histories used in determining the liquefaction triggering boundaries, shifting the liquefaction curve to higher CRR and lower penetration resistances.



(a)



(b)

Figure 3. Comparisons of corrected (a) CRR-$(N_1)_{60}$, and (b) CRR-q_{c1} data collected in this study with liquefaction triggering boundaries established from case histories of liquefaction occurrence (Boulanger and Idriss 2014).

It is finally noted that laboratory cyclic triaxial tests could also involve several uncertainties, system compliances (e.g., membrane penetration and end friction) and sample disturbance. The liquefaction triggering data determined from the laboratory experiments may be also biased at low densities (low SPT or CPT resistance) as loose samples may have become disturbed and compacted during laboratory thawing and reconsolidation prior to cyclic testing. Additional uncertainty is introduced by correction factors (C_r and C_b) used to convert CRR from cyclic triaxial tests to multidirectional cyclic simple shearing condition, as well as associating a 7.5 earthquake magnitude with 15 uniform shear stress cycles applied in laboratory tests irrespective of fines content.

5 CONCLUSIONS

This study presents a database of cyclic triaxial tests on high-quality frozen samples, as well as SPT and CPT penetration resistances at the corresponding specimen depths. The evaluation of these data indicates that field penetration resistance tends to decrease with increasing the amount of fine particles up to a fines content of about 40 – 50%. As a result, silty sands appear to exhibit higher resistance against cyclic liquefaction than clean sands. Laboratory triaxial tests show lower cyclic resistances than those predicted from field-based liquefaction triggering curves for clean and silty sands using SPT blow count, or for silty sands using CPT tip resistance. Further research is required to understand the uncertainty and limitations of estimating the in-situ CRR from cyclic triaxial test results.

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