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SHORT COMMUNICATION

Reexamination of effect of plasticity on liquefaction resistance of low-plasticity fine-grained soils and its potential application

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Abstract The paper summarizes a compilation of existing cyclic experimental data on reconstituted and undisturbed specimens of low-plasticity fine-grained soils to assess liquefaction resistance. The authors normalized the data to reduce the effect of other relevant factors such as shear mode, density, effective confining stress and cyclic loading frequency. It is indicated that liquefaction resistance of the specimens reconstituted using slurry consolidation approach is lower than that of the undisturbed specimens. The liquefaction resistance for undisturbed specimens decreases with an increase in the plasticity index up to 4–5 and then increases with a further increase in plasticity index. A new correction factor K_{PI} to estimate the effect of plasticity index on cyclic resistance ratio is proposed for design purposes and added into the framework of liquefaction evaluation of claylike fine-grained soils with PI of 7–18 (change to 5–18, if ML–CL) on the base of the approach of Boulanger and Idriss. Because the effect of plasticity index on liquefaction resistance is slight when the plasticity index is <7, it is suggested that the liquefaction evaluation of sandlike fine-grained soils with PI of 0–7 (changed to 0–5, if ML–CL) follows the framework of simplified procedures using SPT and CPT data.

Keywords Laboratory data · Liquefaction resistance · Low-plasticity fine-grained soil · Plasticity index

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1 Introduction

Liquefaction of low-plasticity fine-grained soils is a common phenomenon during earthquake events [2, 4, 6]. Initially, liquefaction potential was evaluated using clay content according to the Chinese criteria [22]. As pointed out by Seed et al. [23] and Bray et al. [6, 7], the use of percentage of "clay-size" particles in the Chinese criteria is misleading and the key factor is the percentage and type of clay minerals present in the soil.

The variation of liquefaction resistance with plasticity index (PI) of soil has been studied by many researchers. Puri [17] reported that cyclic strength of undisturbed and reconstituted silt from Memphis, TN, increased with an increase in plasticity index from 10 to 20 by conducting cyclic triaxial tests. Conversely, Sandoval [20] observed that the silt from East Saint Louis, IL, had a decrease in cyclic strength when the PI increased from 1.7 to 3.4; but with a PI of 12, the silt had higher cyclic strength than that with a PI of 3.4. Izadi [16] added kaolinite to a silt from Collinsville, IL, to form soil mixtures having 5 and 10 % kaolinite. The tests indicated that the cyclic strength decreased with an increase in clay content, due to a decrease in hydraulic conductivity with no apparent increase in plasticity index. Guo and Prakash [11] reported that the liquefaction resistance decreases with an increase in PI up to 4, while it increases with an increase in PI above 10. Beroya et al. [1] studied the effect of mineralogy on the cyclic strength of silt–clay mixtures and concluded that the relationships of % clay fraction, % clay mineral and PI with cyclic strength are not unidirectional, because the PI does not adequately encapsulate the effects of clay mineralogy on the cyclic strength of soils.

So far, the effect of plasticity on liquefaction resistance of low-plasticity fine-grained soils is not monotonic. To reexamine the effect of PI on the liquefaction resistance of low-plasticity fine-grained soils, this paper collects existing cyclic experimental data and normalizes them to reduce the effect of other relevant factors such as shear mode, density, effective confining stress and cyclic loading frequency. Then, a new correction factor K_{PI} to estimate the effect of plasticity index on CRR is proposed for design purposes.

2 Reexamination of laboratory data

2.1 Summary of laboratory data collected in this study

Table 1 summarizes the laboratory data from some researchers, who conducted cyclic triaxial (CTX) or cyclic direct simple shear (CDSS) tests to evaluate liquefaction resistance of low-plasticity fine-grained soils under different testing conditions. Table 1 only lists the testing data when the cyclic failure was defined using double-axial strain of 5 %, shear strain of 3.75 % or single-axial strain of 3 %. They were considered to yield comparable testing results [1, 4, 6]. Cyclic failure defined by initial liquefaction ($R_u = 1$) can produce different results. For example, Puri [17] found for loessial soils (PI = 10) that the 5 and 10 % double-amplitude axial strains developed before initial liquefaction, but the 20 % double-amplitude axial strain happened after that. Thus, to avoid the confusion from the criteria to define cyclic failure, the laboratory data of soils showing initial liquefaction were not considered here. In Table 1, the CSR is the cyclic stress ratio, defined as $\Delta\sigma/2\sigma'_c$ or $\tau/2\sigma'_{v0}$ ($\Delta\sigma$—deviator stress; effective consolidation pressure for cyclic triaxial tests; τ—shear stress; σ'_{v0}—vertical overburden pressure for cyclic simple shear tests) in the cyclic triaxial test and cyclic DSS test, respectively.

Besides the plasticity index, the factors influencing liquefaction resistance may include the following: shear mode, specimen preparation method, loading frequency, effective confining pressure, density, initial shear stress and overconsolidation ratio, among others. To consider the effect of plasticity index on liquefaction resistance based on laboratory data, the cyclic strength of soils having different testing conditions needs to be normalized to that with the same testing conditions.

1. Shear mode: CTX or CDSS tests are commonly conducted to investigate liquefaction resistance of soils. The stress path during the CDSS tests better simulates cyclic rotation of principal stresses during earthquake loading [26]. Seed [21] used the term C_r to relate CDSS and CTX testing results as $CSR_{CDSS} = C_r \times CSR_{CTX}$. Boulanger et al. [4] recommended using $C_r = 0.7$ for several fine-grained soils under normal consolidation. Bray and Sancio [6] suggested a C_r value of 0.8 for Adapazari silt. Hence, the use of a C_r value of 0.7–0.8 for fine-grained soils appears to be reasonable. In the current paper, a C_r of 0.75 was used to normalize CSR values obtained from CTX tests to equivalent CSR values based on CDSS tests.
2. Specimen preparation: Laboratory tests can be performed on undisturbed specimens or specimens reconstituted using techniques such as slurry consolidation and moist tamping, among others. Bradshaw and Baxter [5] reported that there is consensus that the specimens reconstituted by slurry consolidation (or deposition) approach have the most representative fabric for fluvial soils. Therefore, the slurry consolidation approach is best option to reconstitute specimens for laboratory tests, if undisturbed soil specimens are not available. Thus, the laboratory data in Table 1 were collected only from tests on undisturbed specimens and reconstituted specimens prepared using slurry consolidation approach.
3. Effective confining pressure: The cyclic strength normally increases with a decrease in effective confining pressure [6, 14, 16, 18, 19]. Based on Hynes and Olsen [14] and Bray and Sancio [6], the factor of $K_\sigma = \left(\sigma'_{V0}/P_a\right)^{f-1}$ can be used to adjust the cyclic strength to that at effective confining pressure of 100 kPa (1 atm). The cyclic stress ratio (CSR) obtained from cyclic test at any effective confining pressure is multiplied with the K_σ to get the CSR at effective confining pressure of 100 kPa. The f values are about 0.8, 0.7 and 0.6, respectively, for relatively loose, medium dense and dense or slightly overconsolidated deposits. Bray and Sancio [6] found that the curve from the equation with $f = 0.7$ fitted well with laboratory data points of fine-grained soils in Adapazari, Turkey. In this study, the effect of effective confining pressure on cyclic strength was considered using the Hynes and Olsen's equation with $f = 0.7$.
4. Density: The decrease in void ratio induces an increase in cyclic strength. Guo and Prakash [11] assumed the CSR was inversely proportional to the void ratio. No other definitive equation has been presented to consider the effect of void ratio on the liquefaction resistance of low-plasticity fine-grained soil. Thus, in

Table 1 Collection of laboratory data of cyclic tests on low-plasticity fine-grained soils

Soil material	Soil type	LL	PI	Fines content (%)	Spec. prep.	Test type	f (Hz)	σ'_c or σ'_{v0} (kPa)	e	Cyclic failure criteria	Number of tests	CSR	References
Adapazari soil	ML, CL	Close to A line, PI < 12		>35	UD	CTX	1	40	0.75–1.00	S.A. $\varepsilon = 3$ %	10	0.24–0.59	[6]
								50			16	0.28–0.595	
		Close to A line, PI = 12–18						40			6	0.275–0.595	
								50			16	0.276–0.487	
loessial soil	ML	34	10	93–98	UD	CTX	1	69	0.738–0.818	D.A. $\varepsilon = 5$ %	8	0.137–0.481	[17]
								103	0.734–0.799		8	0.139–0.475	
								138	0.734–0.799		8	0.145–0.481	
Laterite Tailing	CL	34	12	100	UD	CDSS	0.1	100	1.324–1.451	$\gamma = 3.75$ %	5	0.112–0.245	[26]
Copper–gold tailing	ML	27	0	8.5–97.0				100	0.556		4	0.13–0.25	
Copper–gold–zinc tailing	ML	19	2	62.7–100.0				100–352	0.493–1.180		20	0.107–0.137	
		21	1	97.6				100, 250	0.840–0.870		3	0.115–0.19	
Clay silt	ML		0	16.50	UD	CTX	0.2	150	0.640–0.647	D.A. $\varepsilon = 5$ %	3	0.25–0.4	[25]
		22	5	51				2	0.474–0.482		2	0.3–0.35	
		32	8	88				300	0.543–0.558		3	0.2–0.3	
		34	9	74				200	0.650–0.657		2	0.65–0.657	
		30	6.5	100				200	0.597–0.603		2	0.597–0.603	
Fraser River Delta silt	ML	30.4	4	90	UD	CDSS	0.1	84.9–200	0.855–0.985	$\gamma = 3.75$ %	11	0.1–0.29	[8]
Mississippi River Valley silt	ML	30	6		SC	CTX	0.1	35–105	0.751–0.824	D.A. $\varepsilon = 5$ %	13	0.3–0.45	[16]
Fraser River Delta silt	ML	30.4	4	90	SC	CDSS	0.1	97.3–103	0.843–0.866		4	0.1–0.14	[8]
Quartz rock powder	ML		0	~94.5	SC			95.9–99.7	0.790–0.788		4	0.12	
Lime-stone power	ML	24	6	~88	SC	CTX	0.1	100	0.600	D.A. $\varepsilon = 5$ %	7	0.109–0.221	[13]
Mississippi River Valley silt	ML	28.1	5.8	94.5	SC	CTX	0.1	90	0.665–0.682	D.A. $\varepsilon = 5$ %	7	0.18–0.35	[24]
		28.9	6.2	94.6					0.629–0.677		4	0.18–0.35	
	CL	32.7	9.4	94.8					0.685–0.690		3	0.18–0.35	

UD, undisturbed sampling; SC, slurry consolidation; CTX, cyclic triaxial test; CDSS, direct simple shear test; D.A. ε, double-axial strain; S. A. ε, single-axial strain; γ, shear strain; f, frequency; σ_c', effective consolidation pressure for cyclic triaxial tests; σ_{v0}', vertical overburden pressure for cyclic simple shear tests; e, void ratio after consolidation; N, number of loading cycles at the cyclic failure; Raw CSR, cyclic stress ratio before correction; CSR, cyclic stress ratio after being normalized this study, the CSR for liquefaction evaluation was also considered to be inversely proportional to the void ratio, following the recommendation of Guo and Prakash [11]. For showing the effect of density on cyclic strength, it is best to normalize results by relative density. However, the maximum and minimum void ratios used for determining relative densities were not available for majority of studies in Table 1. Thus, all CSRs were converted to those at the void ratio of 0.600. As shown in the latter, this chosen void ratio will not influence the conclusions drawn in this study.

5. Initial shear stress: Hyde et al. [12] found that cyclic strength reduced with an increase in initial shear stress ratio for stress reversal; but for stress nonreversal, the cyclic strength reduced with an increase in the ratio up to 0.50–0.60 and kept increasing when the ratio exceeds 0.50–0.60. This paper only considers level ground conditions where there is no effect of initial shear stress, and so the CTX tests with no initial shear stress were considered.
6. Overconsolidation ratio: Puri [17], Sandoval [20] and Izadi [16] found that the increase in OCR increased the liquefaction resistance (cyclic strength) of the low-plasticity silt. The current work only collected the data from normally consolidated tests, since the data obtained from tests on overconsolidated specimens were very limited.
7. Loading frequency: The increase in loading rate causes the increase in the cyclic resistance of the silt [19]. The CSRs were adjusted to be 1 Hz to more representative of earthquake loading and were normalized to include the effect of the strain rate using an average of 9 % increase in CSR per log cycle increase in rate, following the recommendation of Boulanger and Idriss [3].

Table 1 collects testing data of the undisturbed specimens [6, 8, 17, 25, 26] and the specimens reconstituted using slurry consolidation approach [8, 13, 16, 24]. The PIs are in the ranges of 0–18 and 0–9.4, respectively, for the undisturbed and reconstituted specimens. Because of too many testing data, Table 1 does not list them but only includes the range of each data range if available. As an example, Table 2 is given to show the raw CSRs and the CSRs after being normalized for Mississippi River Valley (MRV) silt.

2.2 Variation of liquefaction resistance with plasticity index

The cyclic stress ratios after being normalized were obtained and plotted with number of cycle (N_{cyc}) in Fig. 1. Some issues need to be explained here. There were no big differences in the curves of CSR versus N_{cyc} for the MRV silt and its mixtures with various percentage of kaolinite tested by Izadi [16], because they had the same plasticity index of six regardless of different percentages of kaolinite. Thus, only one curve of CSR versus N_{cyc} was produced to fit all data points of the MRV silt and its mixtures [16]. For the Adapazari soils investigated by Bray and Sancio [6], there were two ranges of plasticity index: PI < 12 and $12 <$ PI < 18. To plot the curve of CSR versus plasticity index, the specific values need to be selected. The figure showing the Atterberg limits of the Adapazari soils by Bray and Sancio [6] was reexamined. For PI < 12, the range was separated into two small ranges. The PI in the low range may be represented by 0, and in the high range the PI is scattered but relatively concentrates around 10. Thus, the range of PI < 12 was represented by two points of PI $= 0$ and 10. For the range of $12 <$ PI < 18, an average value of 15 was used.

Figure 1 indicates that the curves (dash) of CSR versus N_{cyc} for undisturbed soils are generally higher than those (solid) for reconstituted soils using slurry consolidation approach. Looking more closely, however, it can be found that the CSR for undisturbed soils was higher than that for reconstituted soils even at the same PI and N_{cyc}. This can be seen clearly in Fig. 2, which shows the values of CSRs required to induce cyclic failure at 30 loading cycles versus plasticity index. The cyclic shear strength of a natural deposit is often referred to an earthquake of moment magnitude, $M_{\mathrm{w}} = 7.5$, which is represented by 30 equivalent uniform loading cycles, N_{cyc} [3]. As shown in Fig. 2, the undisturbed specimens have higher CSR than the reconstituted ones at the same PI. Thus, although the specimen reconstituted using slurry consolidation approach best indicates soil fabric of undisturbed sample of low-plasticity soil [5], the current study based on lots of laboratory data shows that the tests on soil specimens reconstituted using slurring consolidation approach still underestimate liquefaction resistance. Because of this, from here on, the study focuses on the undisturbed specimen results.

The data points of the undisturbed specimens were best-fitted using a parabola (the fitting accuracy $R = 0.761$). The effect of plasticity index on the CSR can be represented by the following equation:

$$\mathrm{CSR}_{(N=30)} = 0.0010\mathrm{PI}^2 - 0.0096\mathrm{PI} + 0.2752 \quad (1)$$

When the PI is 4.8, the $\mathrm{CSR}_{(N=15)}$ reaches the lowest value. With an increase in the PI up to 4.8, the $\mathrm{CSR}_{(N=30)}$ decreases. With a further increase in the PI value larger than 4.8, the $\mathrm{CSR}_{(N=30)}$ increases. When the CSR is normalized to other void ratios rather than 0.6, the best-fitted curves of $\mathrm{CSR}_{(N=30)}$ versus PI are plotted in Fig. 3. It

Table 2 A study example including the raw CSRs and the CSRs after being normalized

Soil material	Soil type	LL	PI	Fines content (%)	Spec. prep.	Test type	f (Hz)	σ'_c or σ'_{v0} (kPa)	e	Cyclic failure criteria	N	Raw CSR	CSR	Reference
Mississippi River Valley silt	ML	28.1	5.8	94.5	SC	TX	0.1	90	0.682	D.A. $\varepsilon = 5\ \%$	0.78	0.35	0.315	[24]
								90	0.676		0.75	0.35	0.312	
								90	0.680		1.78	0.25	0.224	
								90	0.681		32.77	0.18	0.162	
								90	0.661		30.2	0.18	0.157	
								90	0.669		25.2	0.18	0.159	
								90	0.665		29.7	0.18	0.158	
		28.9	6.2	94.6				90	0.660		158	0.18	0.157	
								90	0.677		121	0.18	0.161	
								90	0.648		4.1	0.25	0.214	
								90	0.629		0.7	0.35	0.291	
	CL	32.7	9.4	94.8				90	0.690		405.2	0.18	0.164	
								90	0.688		10.3	0.25	0.227	
								90	0.685		0.8	0.35	0.316	

can be found that the PIs for lowest CSRs $_{(N=30)}$ are 4.8, 4.8, 4.6, 3.8 and 4.6, respectively, when the void ratios used to normalize CSRs are 0.5, 0.6, 0.7, 0.8 and 0.9. Thus, low-plasticity fine-grained soil has lowest liquefaction resistance when its PI is about 4–5. The above finding obtained from a comprehensive investigation using the collection of the extensive laboratory data happened to be similar to that by Guo and Prakash [11] and Gratchev et al. [9, 10] who reported that the plasticity index for the minimum CSR was also about 4.

3 Discussion

For the claylike fine-grained soil with a plasticity index larger than 7 (reduced to 5, if CL–ML), Boulanger and Idriss [3] presented Eq. (2) to evaluate liquefaction resistance, based on the finding that the cyclic strength can be expressed as a ratio of the soil's undrained shear strength (S_u)

$$\mathrm{CRR_M} = C_{2\mathrm{D}}\left(\frac{\tau_{\mathrm{cyc}}}{S_{\mathrm{u}}}\right)_{M=7.5}\frac{S_{\mathrm{u}}}{\sigma'_{\mathrm{vc}}}\mathrm{MSFK}_{\alpha} \tag{2}$$

where $\mathrm{CRR_M}$ is the cyclic resistance ratio at an earthquake with a magnitude of M; $C_{2\mathrm{D}}$ is a correction for two-dimensional versus one-dimensional cyclic loading; $(\tau_{\mathrm{cyc}}/S_{\mathrm{u}})_{M=7.5}$ (or $(\tau_{\mathrm{cyc}}/S_{\mathrm{u}})_{N=30}$) is the ratio of cyclic shear stress (τ_{cyc}) to S_{u} for 30 equivalent uniform cycles representative of an $M_{\mathrm{w}} = 7.5$ earthquake; MSF is the magnitude scaling factor to approximately account for the correlation between earthquake magnitude and number of equivalent uniform loading cycles; K_α is the initial shear stress ratio correction factor. For specific number of loading cycles to evaluate liquefaction initiation, cyclic resistance ratio (CRR) is used instead in this section.

Although $\left(\tau_{\mathrm{cyc}}/S_{\mathrm{u}}\right)_{M=7.5} = 0.83$ was used for the clay-like materials by Boulanger and Idriss [12], they suggested that the continued compilation of laboratory test data can lead to future refinement. Actually, with a further investigation by Boulanger and Idriss [3], the natural silt (ML) has a lower $\left(\tau_{\mathrm{cyc}}/S_{\mathrm{u}}\right)_{M=7.5}$ than natural clays (CL and CH). The difference in $(\tau_{\mathrm{cyc}}/S_{\mathrm{u}})_{M=7.5}$ between them was about 0.1. As stated by Boulanger and Idriss [3], the available data were not sufficient to define the effect of the plasticity index, age, soil type, OCR and test type on the value of $\tau_{\mathrm{cyc}}/S_{\mathrm{u}}$ at that time; therefore, the $\left(\tau_{\mathrm{cyc}}/S_{\mathrm{u}}\right)_{M=7.5} = 0.83$ was used in their work.

Until now, there has not been available chart to consider the effect of plasticity index on cyclic resistance ratio of low-plasticity fine-grained soils for engineering application. Equation (1) was presented based on extensive laboratory data of undisturbed specimens. It may be used to consider the effect of plasticity index when estimating the CRR for liquefaction resistance in engineering application.

Equation (1) is based on the fine-grained soils under the conditions of $\mathrm{OCR} = 1$, $\sigma'_{\mathrm{v}} = 100\,\mathrm{kPa}$ and $e = 0.6$. Although some researchers found that cyclic resistance ratio increased with an increase in OCR [16, 17, 19, 20], no definitive relationship has been presented to consider the effect of OCR on the CSR. Thus, it will be impossible to predict the CRR of overconsolidated soils directly using

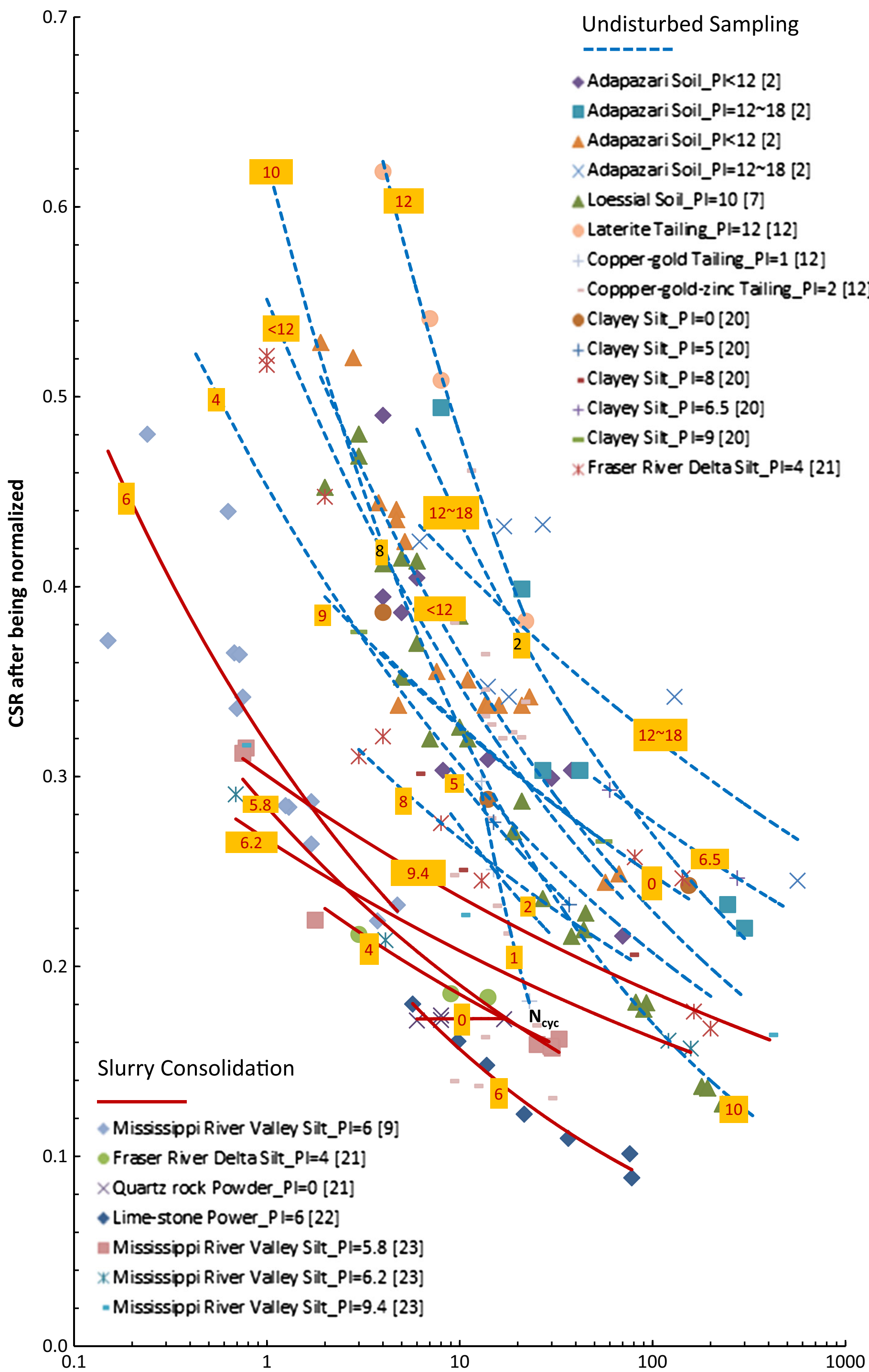

Fig. 1 Summary of relationships between CSR after being normalized and number of loading cycles of low-plasticity fine-grained soils (the numbers on the *curves* represent the values of plasticity index)

Eq. (1). Equation (2) has been used by Boulanger and Idriss [3] to do an evaluation of liquefaction resistance for Carrefour Shopping Center in Turkey during the 1999 Kocaeli earthquake. Therefore, it is recommended that the liquefaction evaluation be done using Eq. (2) with a consideration of the effect of plasticity index on liquefaction resistance shown in Eq. (1).

Following the recommendations of Boulanger and Idriss [3], the low-plasticity fine-grained soils were divided into the claylike and sandlike soils to evaluate their liquefaction

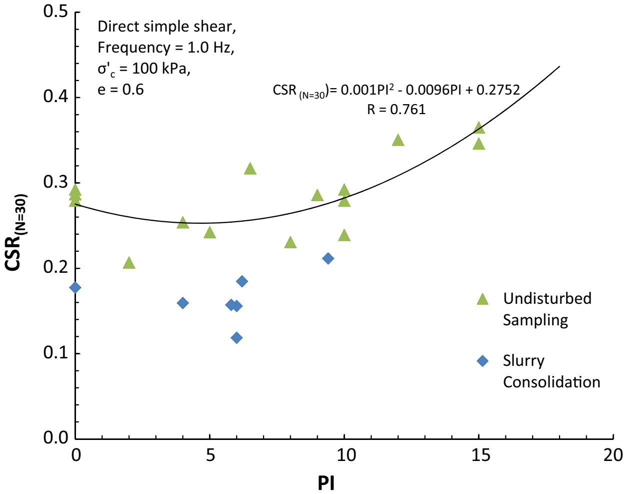

Fig. 2 Variation of $\mathrm{CSR}_{(N=30)}$ with plasticity index for undisturbed and reconstituted specimens of low-plasticity fine-grained soils

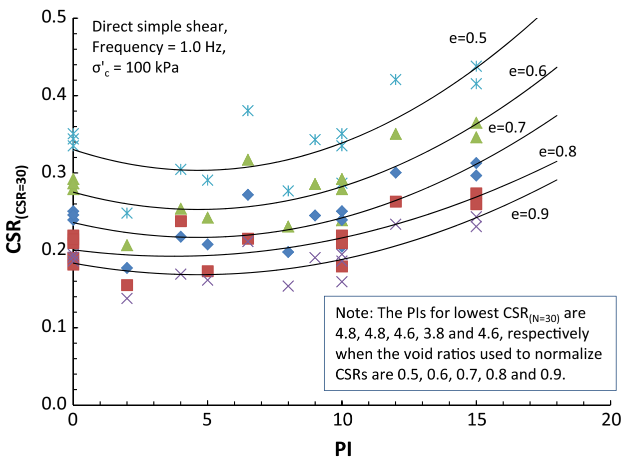

Fig. 3 Variation of $\mathrm{CSR}_{(N=30)}$ with plasticity index for undisturbed specimens of low-plasticity fine-grained soils with different void ratios used for normalizing CSR data

resistance. For the claylike fine-grained soil with a plasticity index larger than 7 (reduced to 5, if CL–ML), Boulanger and Idriss [3] presented the following equation:

$$\mathrm{CRR}_{M=7.5} = 0.18 C_{2\mathrm{D}} \mathrm{OCR}^{0.8} K_{\alpha} \tag{3}$$

based on Eq. (2) to calculate the $\mathrm{CRR}_{M=7.5}$ by considering the effect of OCR on the monotonic undrained shear strength with $S_\mathrm{u}/\sigma'_\mathrm{vc} = S \times \mathrm{OCR}^m$ ($S = 0.22$ and $m = 0.8$) and adopting $\left(\tau_\mathrm{cyc}/S_\mathrm{u}\right)_{M=7.5} = 0.83$. It is noted that the effect of effective confining pressure on CRR can be included in the $S_\mathrm{u}/\sigma'_\mathrm{vc}$.

To consider the effect of plasticity index, a coefficient of correction K_PI for claylike materials is proposed in this paper and added into the equation as follows:

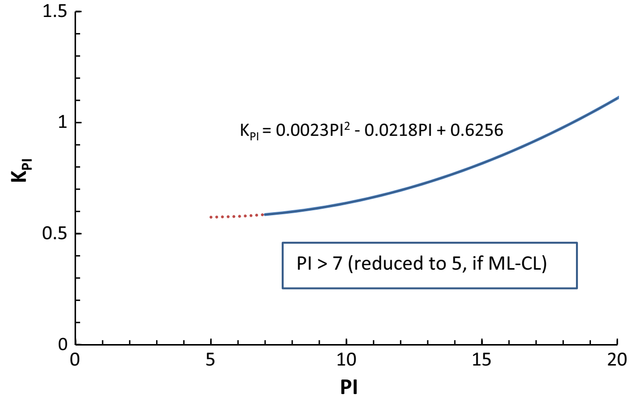

Fig. 4 Variation of K_PI against PI for claylike materials

$$\mathrm{CRR}_{M=7.5} = 0.18 C_{2\mathrm{D}} \mathrm{OCR}^{0.8} K_{\alpha} K_\mathrm{PI} \tag{4}$$

Because Eq. (3) was deduced by Boulanger and Idriss [3] based on the natural silt (ML) and natural clays (CL and CH) with the PI values in the range of 10–27, the central value equal to 18.5 in the range was used to formulate the correction factor K_PI as follows

$$\begin{aligned} K_\mathrm{PI} &= \frac{\mathrm{CRR}}{\mathrm{CRR}_{\mathrm{PI}=18.5}} = \frac{0.0010\mathrm{PI}^2 - 0.0096\mathrm{PI} + 0.2752}{0.4399} \\ &= 0.0023\mathrm{PI}^2 - 0.0218\mathrm{PI} + 0.6256 - \text{Claylike materials} \end{aligned} \tag{5}$$

where the $\mathrm{CRR}_{\mathrm{PI}=18.5}$ is the cyclic resistance ratio at the PI of 18.5 in Eq. (1). One of benefits of Eq. (5) is that the effect of void ratio on K_PI can be removed, because both the numerator (CRR) and the denominator ($\mathrm{CRR}_{\mathrm{PI}=18.5}$) in the equation should be multiplied by the same coefficient considering the effect of void ratio on CRR. Thus, Eq. (5) can be used for any testing conditions. The equation is plotted in Fig. 4, the K_PI increases with an increase in PI for claylike material with a PI higher than 7 (reduced to 5, if ML–CL). Since Eq. (1) was obtained based on the data of low-plasticity soil specimens with PI equal to 18 at maximum, it is required that Eq. (5) should only be used for the soil materials with PIs >18 to determine K_PI for considering the effect of plasticity index on cyclic strength.

For sandlike materials with PI up to 7 (reduced to 5, if ML–CL), the ratio of maximum CRR to minimum CRR at 30 loading cycles was calculated to be 1.02, according to Eq. (1). Thus, the effect of the PI up to 7 (reduced to 5, if ML–CL) on cyclic strength is slight and ignored when the frameworks of existing standard penetration test (SPT) and cone penetration test (CPT) are used for liquefaction evaluation. This is a little different with the suggestion by Ishihara [15] for sandy soils, who stated that the plasticity index indicated little influence on liquefaction resistance at the plasticity index <10.

4 Summary and conclusions

With a collection of extensive laboratory data, the CSRs obtained from different cyclic tests were corrected to the same testing conditions, and then the variation of CSR against number of loading cycles was plotted for different plasticity indexes. It was shown that the liquefaction resistance of the specimens reconstituted using slurry consolidation approach was lower than that of the undisturbed specimens. An equation was presented to show the effect of plasticity index on the liquefaction resistance of the low-plasticity fine-grained soils based on the laboratory data. The liquefaction resistance decreased with an increase in plasticity index <4–5 regardless of void ratio of test specimens. Beyond 4–5, it increased with a further increase in plasticity index.

Following the approach of Boulanger and Idriss [3], the low-plasticity fine-grained soils were divided into the two types: claylike and sandlike materials. For the claylike materials with PI of 7–18 (change to 5–18, if ML–CL), the effect of plasticity index on cyclic stress ratio shown in Eq. (1) was combined with Eq. (3) for liquefaction evaluation. A correction factor K_{PI} was proposed to consider the effect of plasticity on liquefaction resistance. For sandlike materials with PI of 0–7 (changed to 0–5, if ML–CL), the frameworks of existing standard penetration test (SPT) and cone penetration test (CPT) based on liquefaction correlations can be used without considering the effect of plasticity index on cyclic strength, since the change in liquefaction resistance is slight when PI is up to 7 (reduced to 5, if ML–CL).

The proposed approach considers the effect of plasticity index on cyclic stress ratio was not verified, because no testing data are available to do that. However, so far, this idea is presented for a reasonable communication.

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