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Liquefaction Resistance of Silty Sand and Silt from Cyclic Direct Simple Shear Tests

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ABSTRACT

The current standard of practice to evaluate the cyclic resistance of granular soils to liquefaction is based on empirical procedures developed from seismic case history data. The method is based on the interpretation of in situ penetration test data in sand with fines contents primarily up to about 35%. In clean sands, granular soils with fines contents less than 10%, the validity of the methods are generally well documented. For evaluating the cyclic resistance as the fines content increases, the in-situ penetration test data are corrected to an equivalent clean sand value, based on the amount of fines content. However, historical evidence of liquefaction for soils with fines contents larger than 35%, such as silty sand and silt, is limited and the calibration information is considered insufficient to characterize the liquefaction resistance of these soils with the existing empirical procedures. Furthermore, where the fines content of soil is higher than 50%, the plasticity characteristics of the fines fraction, and the overconsolidation ratio, will also affect the cyclic resistances of these soils. Neither of these aspects can be determined based on the normal procedures using in situ penetration resistance.

This paper presents the results of a series of cyclic direct simple shear tests on silty sand and silt samples collected from the Pacific Northwest. The test results were used to assess the effect of fines content, plasticity, and overconsolidation ratio on the cyclic resistance of the soils with higher fines fractions. It has been observed through testing that the resistance to liquefaction in granular soils with fines contents above 15% is typically higher than that predicted by in-situ penetration test data. The laboratory results would suggest that the fines correction applied to the in-situ penetration test data may be a lower bound adjustment. Some comments are also provided on the issue of soil disturbance during sampling of these high fines content soils.

Keywords: Liquefaction, cyclic resistance, fines content, overconsolidation ratio, cyclic direct simple shear test.

INTRODUCTION

The liquefaction triggering assessment in granular soils is commonly based on in-situ penetration test data and the empirical procedures derived from case history information. The method proposed by Seed et al. [1] compares the cyclic resistance ratio (CRR) of the soil with the cyclic stress ratio (CSR) induced by the earthquake ground motions. The cyclic resistance ratio (CRR) is estimated based on the standard penetration test (SPT) blow count (N value) or the cone tip resistance (q_c) from the cone penetration test (CPT). The original chart presented by Seed et al. [1] is based primarily on the evaluation of case histories of liquefaction for clean sands. The chart presents a dividing line between cases where liquefaction was recorded and results where no liquefaction was evidenced. The dividing line is taken to be applicable to clean sands with fines contents less than 5%. Subsequently, lines corresponding to 15% and 35% fines content were added to the chart, as shown on Figure 1a. Figure 1b shows the liquefaction trigger curves for CPT q_{c1N} suggested by Robertson & Wride [2]. Figure 1c and Figure 1d present the latest liquefaction triggering curves recommended by Boulanger & Idriss [3]. As indicated on Figure 1 (a to c) the cyclic resistances increase with fines content (up to 35%), at a constant N_{160} or q_{c1N}. The CPT-based liquefaction triggering correlation recommended by Boulanger & Idriss [3] considers a correction for fines contents of up to 70%.

The measured penetration resistance in granular soil is affected by the fines content of the soil. For a sand of given relative density, as the fines content of the sand increases, the penetration resistance decreases. For a sand with constant relative density (or void ratio), the measured penetration resistance needs to be corrected to compensate for the reduction due to the effect of the fines content (Figure 2). The correction is directly related to the fines content, but the plastic component of the fines may also be a contributing factor. Ishihara [4] suggests that the correction for fines content and plasticity would be of the form:

$$(N_1)_{FC+PI} = N_1 + \Delta N_{FC} + \Delta N_{PI} \tag{1}$$

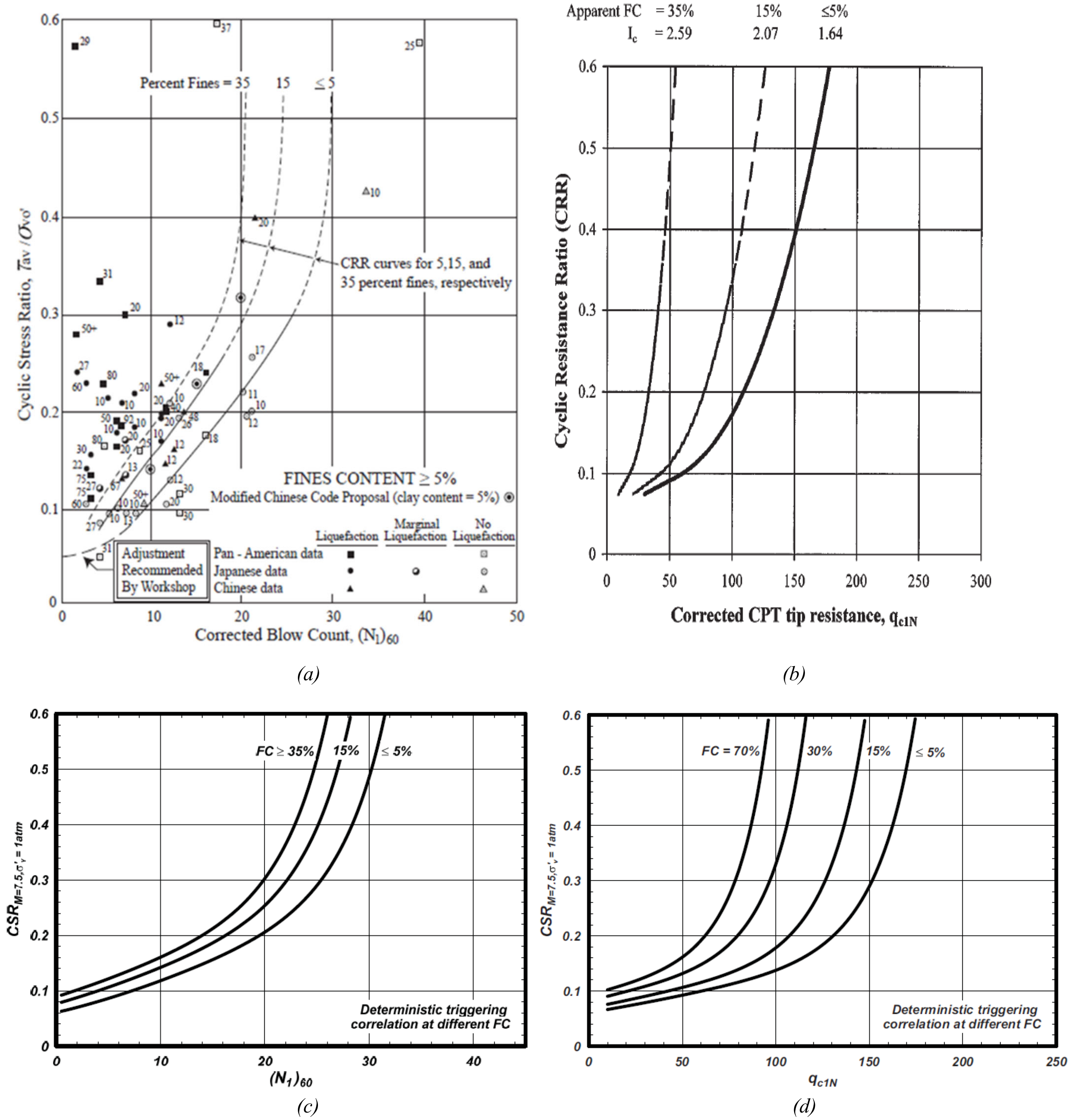


Figure 1. Liquefaction Triggering Curves: (a) SPT data from empirical liquefaction data (Youd et al. [4], modified from Seed et al. [1]), (b) CPT-based liquefaction curves for various values of soil behavior index (Robertson & Wride, [2]), (c) SPT-based triggering correlation (Boulanger & Idriss [3]), (d) CPT-based triggering correlation (Boulanger & Idriss [3]).

Figure 2 indicates the correction required for a sand of equal relative density where the penetration resistance is affected by the content and plasticity of the fines. A possible correction factor for the effect of plasticity suggested by Ishihara [4] is indicated on Figure 3.

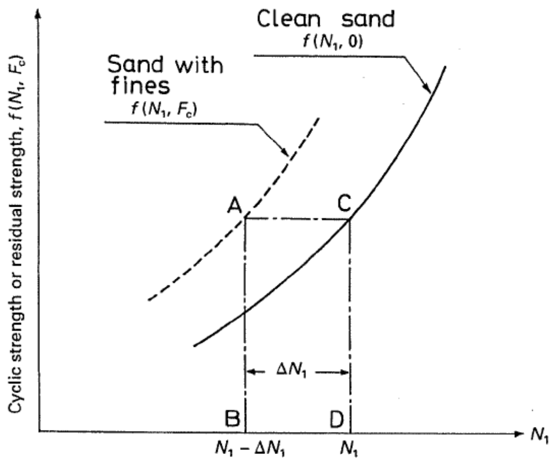


Figure 2. Definition of ΔN_{1FC} for sands of equal relative density but differing fines contents (Ishihara [4])

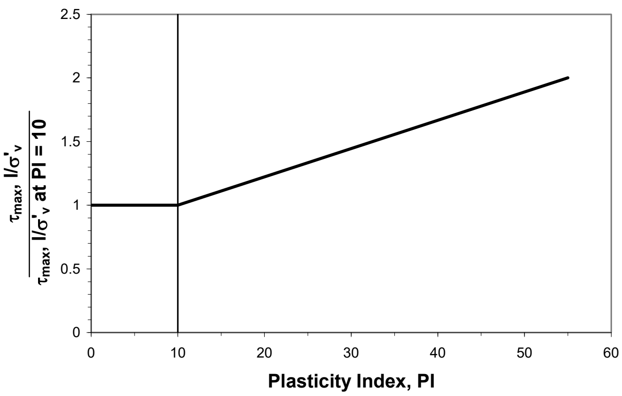


Figure 3. Modification of cyclic strength allowing for the effect of plasticity index (Ishihara [4])

A similar fines content correction also exists for the use of CPT data in fine-grained soils. Robertson and Wride [2] suggest a correction for fines content based on the soil behavior type index, I_c:

$$(q_{c1n})_{cs} = K_c\,(q_{c1n}) \qquad (2)$$

where K_c is a function of I_c.

As indicated by Boulanger and Idriss [3], there are large uncertainties in the I_c and fines content relationship, which include the unknown influence of the plasticity. Various approaches have been proposed for adjusting the penetration resistance (N or q_c) to account for the influence of the fines content of the soils:

- Idriss and Boulanger [3][6]
- Seed et al. [7]
- Stark and Olson [8]
- NCEER [5]

Typical correction values for both N_1 and q_{c1} as a function of fines content are indicated on Figure 4. As shown on Figure 4 for the SPT-based procedure, the correction was limited to a fines content of up to 40%. For the CPT-based procedure proposed by Boulanger & Idriss [3] the correction starts to plateau for fines contents greater than 70%.

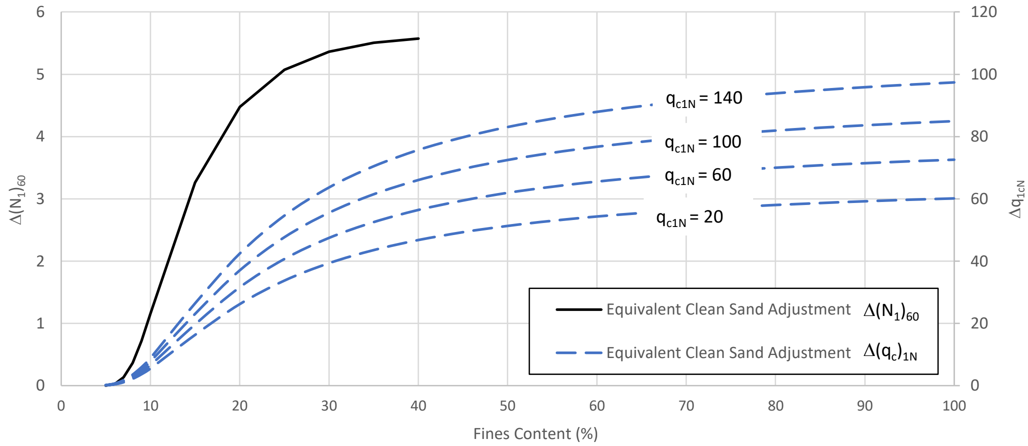


Figure 4. Equivalent clean sand adjustments for SPT- and CPT-based liquefaction triggering procedures (Boulanger & Idriss [3])

However, it is commonly thought that the SPT/CPT corrections for fines content underestimate the real impact of the fines on the penetration resistance (ATC/MCEER [9], Sully et al. [17]). As a result, it is generally recommended that laboratory tests be performed on soils where the fines content may be large. Based on data published in the literature and our own experience, we consider that this should be the case wherever the fines content of a granular soil is larger than about 15%. Furthermore, we believe that laboratory testing is one of the most reliable ways of determining the liquefaction resistance of silty sand and silt, subject to being able to obtain good quality 'undisturbed' samples.

Empirical approaches have also been developed to evaluate the liquefaction susceptibility of fine-grained soils. Similar to the SPT approach for sands, the Chinese Criteria were developed from observations after earthquakes in China. The Chinese Criteria suggest that fine-grained soils may be susceptible to significant strength loss if they satisfy the following conditions:

- 15% of the particles are finer than 0.005 mm
- liquid limit, LL < 35%
- ratio of water content (w) to liquid limit, w/LL > 0.9

Various modifications to the Chinese Criteria were subsequently proposed by other researchers. The Chinese Criteria were modified by Bray et al. [10] for lightly overconsolidated silt based on observations following the Kocaeli earthquake in Turkey. The basic conditions proposed by Bray et al. [10] are summarized on Figure 5.

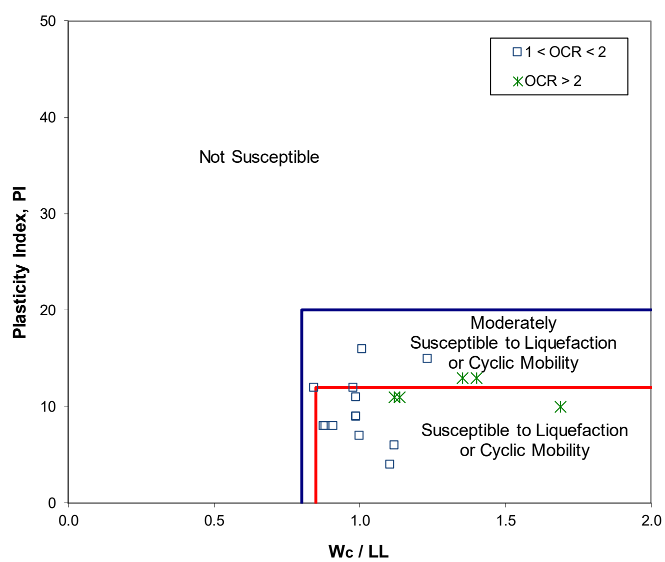


Figure 5. Application of the Bray et al. [10] criteria for liquefaction assessment for overconsolidated silt.

Boulanger and Idriss [11] suggest that if the plasticity index (PI) of the soil is less than 7, then the soil can be analyzed as if it were sand, using the penetration-based approach. If the PI>7, then the assessment should be based on shear strength considerations.

Based on recent results presented by Bray and Sancio [13] and Sanin and Wijewickreme [14], the methodology recommended in the Greater Vancouver Task force report [15] and Canadian Highway Bridge Design Code (CHBDC) S6-19 [16] suggests the following approach be followed:

- for PI<7, assume the material behaves like a sand and apply the penetration-based method to determine the cyclic resistance, or undertake a specific laboratory test program on good quality samples;
- for 7<PI<12, strain accumulation and post-seismic settlement may be the primary concern and the material is considered less likely to liquefy. The post-seismic residual strength can be approximated by 80% of the static Su;
- for PI>12, the material is considered to behave as a cohesive material and the cyclic effects on stiffness and strength may be limited. Post-seismic strength reductions are generally limited, and Su is used for design.

However, the task force urges caution in the application of these guidelines for sensitive and overconsolidated soils.

A technical paper entitled “Liquefaction of Fine-Grained Soils from Cyclic DSS Testing” [17] which discussed the results of cyclic direct simple shear (CycDSS) tests on soils with varying fines contents was presented in the 2011 Pan-Am Canadian Geotechnical Society Geotechnical Conference. This paper includes additional laboratory test results and provides an update on the liquefaction resistance of silty sand and silt.

CYCLIC DIRECT SIMPLE SHEAR TEST PROCEDURES

Limited studies have been performed to assess the cyclic resistance of granular soils with important fines contents (silty sands and silts). However, the consensus is that these soils are more resistant to initial liquefaction than inferred from field-based approaches.

Laboratory testing approaches can be used to assess the liquefaction resistance of fine-grained soils based on element tests on undisturbed or reconstituted samples. It is considered feasible to obtain undisturbed samples of acceptable quality in most granular soils where significant fines contents are present. Where the fines content is sufficient to allow the recovery of good quality samples, it is generally possible to handle and prepare these samples for testing in the laboratory.

A series of cyclic DSS tests have been performed on samples of silty sands and silts as part of projects performed mainly in the Pacific Northwest region. Only a portion of the results are presented herein.

The testing was performed in the Tetra Tech Geotechnical Laboratory in Richmond, BC. Tetra Tech has two cyclic DSS testing machines from GDS Instruments in the UK. Both machines are rated to a testing frequency of 5 Hz, but cyclic tests are generally performed at 1 Hz. One machine has a vertical load capacity of 10 kN while the other is a 5 kN machine. The soil samples are contained in a series of low-friction rings during consolidation and shear, which is performed under constant-volume conditions.

Samples tested have generally been recovered by thin-walled Shelby tube sampling. The Shelby tubes are scanned using gamma or X-rays to determine the areas of the sample in the tube that have been most affected by the sampling process. Samples for testing are only cut from parts of the tube where disturbance effects are not visible on the digital versions of the scans.

Once the sample intervals to be tested have been defined, the sample tube is cut manually using a rotary tube cutter. Minimal pressure is applied to the tube to avoid deformations to the cross-section that may affect the sample. The sharpened disc cutter is rotated over the tube circumference to slowly cut into the wall, thus minimizing any vibration. The tube is cut into sections about 75-100 mm long which permit the preparation of two or three hockey-puck sized samples for DSS testing. Once the tube is cut, the burrs (if any) at the ends of the cut section are removed and the sample is extruded in the same direction as the sample enters the tube during sample recovery in the field.

Sample preparation is performed using a thin cheese wire to trim the sample height. The shear rings and internal membrane of the DDS equipment allow the 73 mm diameter sample to be placed directly into the membrane and rings with no trimming of the diameter required. Final sample dimensions are 25 mm high with a diameter of 73 mm. The cyclic DSS tests were performed in accordance with the general procedures outlined in the American Society for Testing and Materials standard ASTM D8296-19.

The testing program performed is generally broad and examines factors such as stress history and fines content on the response of fine-grained soils to cyclic loading. Since stress history is considered to be an important factor in characterizing the cyclic response, one-dimensional consolidation tests are usually performed on samples to determine the stress history of the deposit to be tested. To cover the range of conditions, tests have been performed on ko-consolidated samples without a static bias. In addition, the response after different loading histories (after initially identical stress conditions) has also been studied.

All tests have been performed under constant volume conditions whereby the vertical load on the sample is adjusted to maintain no overall change in volume of the sample during shear. Under undrained conditions, the change in the vertical load is synonymous with the change in pore pressure in the sample. Cyclic loading has been performed under stress-controlled conditions. Shear wave velocity measurements can be made at any time during the test by means of bender elements seated in the top and bottom caps of the DSS equipment. Once the cyclic loading is complete, either post-cyclic static shear strength or volume change characteristics can be measured. (The shear wave velocity measured in the laboratory is compared to that measured in situ as one approach to assessing the quality of the DSS sample to be tested.)

The results presented in this paper are primarily from tests performed on good quality samples recovered in the field by thin-walled piston/Shelby sampling. The highest quality sections of the Shelby tube have been identified using gamma scanning and only these samples have been tested. An additional two tests are on sand samples prepared by moist tamping with another two samples prepared by dry tamping.

CYCLIC DIRECT SIMPLE SHEAR TEST RESULTS

The results of the cyclic DSS tests presented here cover three aspects of the test program completed:

- the effect of fines content on the response of silty sand to cyclic loading,
- the effect of plasticity on the response of silt to the cyclic resistance, and
- the effect of the overconsolidation ratio on the resistance to liquefaction.

Figure 6 presents the laboratory test results in terms of the variation of fines content, plasticity and the overconsolidation ratio. The results on Figure 6 are for failure defined by initial liquefaction or 5% single amplitude strain. Discussion on each of the influence factors to cyclic resistance will be discussed in the following sections.

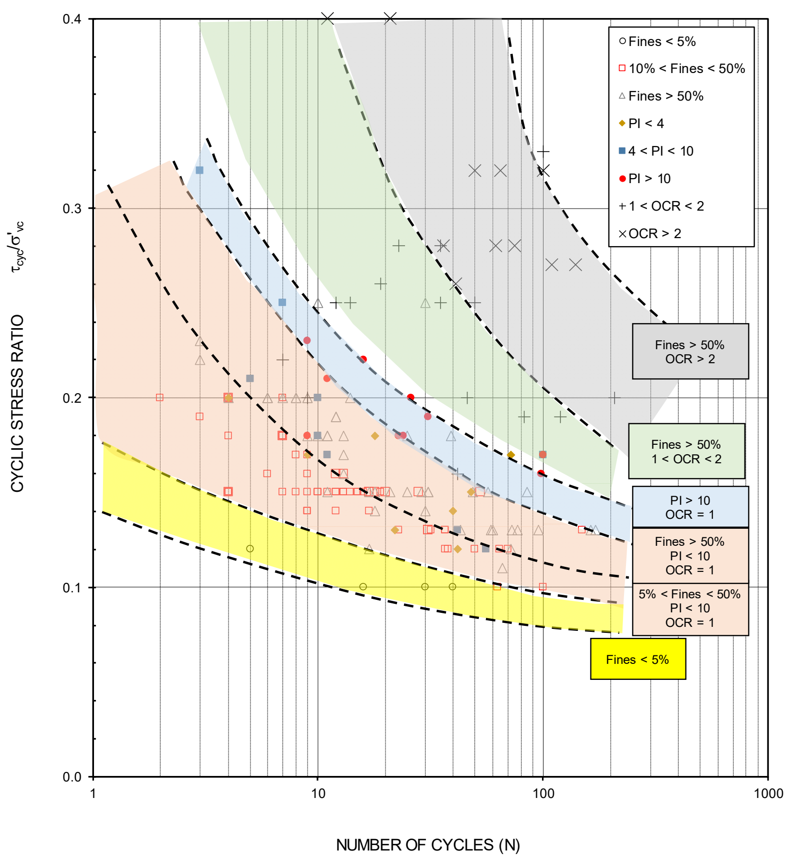


Figure 6. Results of cyclic (stress-controlled) DSS tests on sands with different fines contents and silts, with varying plasticity and overconsolidation ratio.

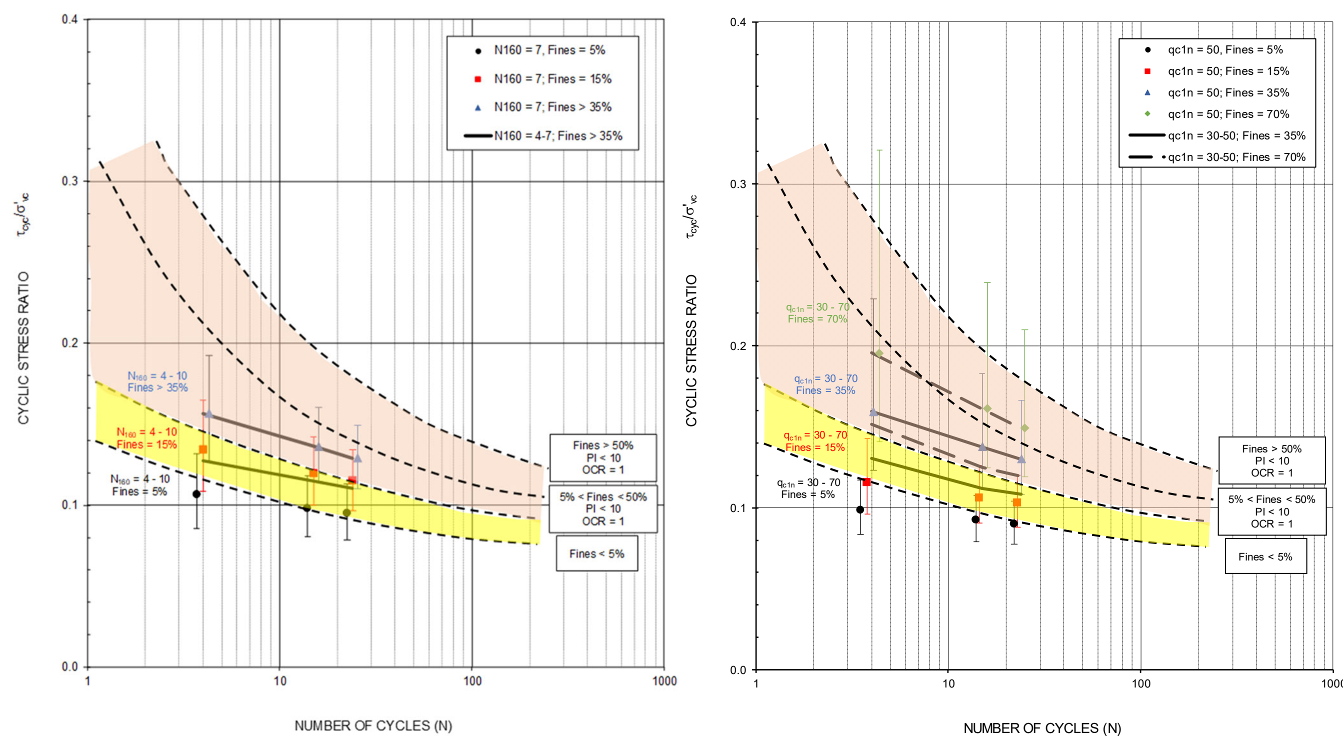
Fines Content

The effect of fines content for the samples tested is presented on Figure 6. All the normally consolidated soils tested were consolidated to pressures slightly above those present in the field condition to ensure that no effect of overconsolidation history was present. Results presented are for soils that vary from clean sand with less than 5% fines content, sandy silt to silty sand with fines content between 5% and 50%, and silt with over 50% fines. The relative density of the clean sand samples was reasonably constant at around 40-60%. The PI of the silt samples indicate the soils to be essentially non-plastic.

The results on Figure 6 clearly indicate the increased resistance to liquefaction that occurs as the fines content (and OCR) of the sample increases. For the case where number of cycles, N=15, the cyclic resistance ratio, given by the mid-point for each of the ranges, varies from about 0.1 for clean sand to just under 0.2 for silt (with PI < 10). At any specific level of cyclic stress ratio, the increase in the resistance to liquefaction is clearly a function of the fines content (Figure 6).

The trend in the increased resistance can be visualized by comparing the results of the cyclic DSS testing with the SPT- and CPT-based liquefaction triggering procedures recommended by Boulanger & Idriss [3] for the differing fines contents. Figure 1 is drawn for an earthquake magnitude of 7.5, which corresponds to 15 equivalent cycles of shaking. To generate the appropriate ordinates for other numbers of cycles, the magnitude scaling factors (MSF) proposed by Boulanger & Idriss [3] have been applied to Figure 1.

The comparison between the cyclic DSS test results and the penetration-based liquefaction triggering procedures are indicated on Figure 7. Table 1 summarizes the variations of penetration resistances $(N_1)_{60}$ and q_{c1N} values, earthquake magnitude and the equivalent number of cycles. Figure 7 presents average $(N_1)_{60}$ values of 7 and q_{c1N} of 50 with the lower and upper ranges corresponding to a variation in $(N_1)_{60}$ of 4-10 and in q_{c1N} of 30-70. These $(N_1)_{60}$ and q_{c1N} values are the typical values for sand with high fines content, as shown on Figure 1 (a and b).



(a) *(b)*

Figure 7. Comparison between cyclic DSS results and cyclic resistance (a) SPT-based and (b) CPT-based recommended by Boulanger and Idriss [6].

Overall, the laboratory and field liquefaction resistance curves are in reasonable agreement. For the clean sand (fines content less than 5%), the CRR from the laboratory tests are slightly higher than the average data (N_{160}=7 and q_{c1N}=50) provided by Boulanger and Idriss [6]. Given the relative densities of the clean sand samples are between 40% and 55%, the upper bound penetration data (N_{160}=10, q_{c1N}=70) should be considered, which gives a better agreement between the field and laboratory test results. Note that the fines content correction is limited at 40% for the SPT-based liquefaction triggering, the CRR for soils with high fines content (>35%), the field data is generally underestimated the cyclic resistances as compared to the laboratory test results.

Table 1. Summary of parameters used for liquefaction triggering.

	Penetration Resistance	Fines Content	Earthquake Magnitude (M) & Equivalent Number of Cycles (N)
SPT	$(N_1)_{60}$ = 4, 7, 10	5%, 15%, 35%	M = 6.0 (N = 4) M = 8.2 (N = 24)
CPT	q_{c1N} = 30, 50, 70	5%, 15%, 35%, 70%	M = 6.0 (N = 4) M = 8.2 (N = 24)

As shown on the case history data on Figure 1, the measured penetration resistances for the liquefied soils are generally less than 7 for SPT $(N_1)_{60}$ and are less than 50 for CPT q_{c1N}. Comparing the lower bound penetration resistances (as indicated by the solid or dashed lines on Figure 7) for the field liquefaction curves with the laboratory test results, the CRR is underestimated for soils with high fines content using the field data. As indicated by the cyclic DSS results presented herein, the fines corrections applied for SPT and CPT data in fine-grained soils do not fully account for the effect of fines content.

Plasticity Index

The effect of the plasticity for the samples tested is also presented on Figure 6. Different ranges of plasticity have been reviewed to assess the impact of the plasticity. The results on Figure 6 indicate the increased resistance to liquefaction that occurs as the plasticity of the soils is larger than about 10. For the case where the number of cycles N=15, the cyclic resistance ratio could increase from 0.20 to 0.22 for PI > 10. The results are consistent with the recommendations from Ishihara [4] where the cyclic strength could be modified when the PI is larger than 10.

The effect of the plasticity on the cyclic resistance was not considered in the liquefaction triggering procedures by Boulanger & Idriss [6] given that when the plasticity is larger than 7, the cyclic response of these soils is considered to be representative of clay-like behavior. The recommendation provided by Greater Vancouver Task force report [15] and CHBDC S6-19 [16] suggest a transition zone for the soils with PI between 7% and 12%, which are considered less sand-like and the soils are likely to develop cyclic mobility instead of liquefaction. This latter recommendation would seem more appropriate to consider the effect of changes in plasticity index greater than about 10%.

Overconsolidation Ratio

Sanin and Wijewickreme [14] have demonstrated the effect of overconsolidation on the resistance to liquefaction in silt. The stress-strain loops for a typical sequence of tests are presented on Figure 8 for the silt sample that has an OCR that varies from 1.0 to 2.4. Testing was performed in stress-controlled mode. All samples were saturated prior to testing. Since the saturation cannot be measured in the test, saturation was assumed to be complete after passing two pore volumes of distilled/deaired water through the sample under a nominal backpressure.

The change in the vertical stress ratio (equivalent in the constant volume DSS test to the pore pressure induced during cyclic loading) is presented for the same three tests on Figure 9. All samples indicate a similar final pore pressure ratio of about 70% but at widely differing numbers of cycles. The number of cycles to failure increases dramatically as the OCR of the sample increases.

For OCR=1, the sample achieved 5% strain very quickly after only 4 cycles. The numbers of cycles increased to 26 and 100 as the OCR increased to 1.7 and 2.4, respectively. The shape of the stress-strain curves also changed noticeably (Figure 8). As indicated on Figure 9, the excess pore pressure is much higher for OCR=1 at the first cycle. In addition, the rate of pore pressure generation with number of cycles is similar for OCR=1 and 1.7 but decreases significantly for OCR=2.4.

A comparison of the results obtained for samples with differing OCR values is presented on Figure 10. The three ranges correspond to soils with fines contents greater than 50%, plasticity indices in the range from 4% to 16% and overconsolidation ratio increasing from OCR=1.0 to OCR=3.0. For the clean sand, at N=15 the average cyclic resistance ratio is about 0.11. This increases to 0.18 for PI<10 as the fines content increases to more than 50% (consistent with Figure 6). With the OCR between 1 and 2, for N=15 the CRR increases to about 0.30, while for OCR larger than 2 the CRR is larger than 0.40.

The cyclic strength of clay-like fine-grained soils are also estimated based on the empirical approach recommended by Idriss & Boulanger [6]:

$$CRR_{M=7.5} = 0.80 * S * OCR^{m} \tag{3}$$

where S is the value of S_u/σ'_v for the normally consolidated condition (OCR = 1), and m is the slope of the S_u/σ'_v versus OCR on a log-log plot. The value of m is typically between 0.8 and 1.0. The CRR is estimated using S = 0.22 and OCR of 1.0, 1.5

and 2.0 with m = 1. The MSF proposed by Boulanger & Idriss [6][18] for clay and sand has been applied to generate the appropriate ordinates for other numbers of cycles. As suggested by Boulanger & Idriss [18], the relationship between the CRR and the number of cycles is flatter for clays (and plastic silts) than sands. The comparison between the cyclic DSS tests and the empirical approach is indicated on Figure 10 for different OCR values.

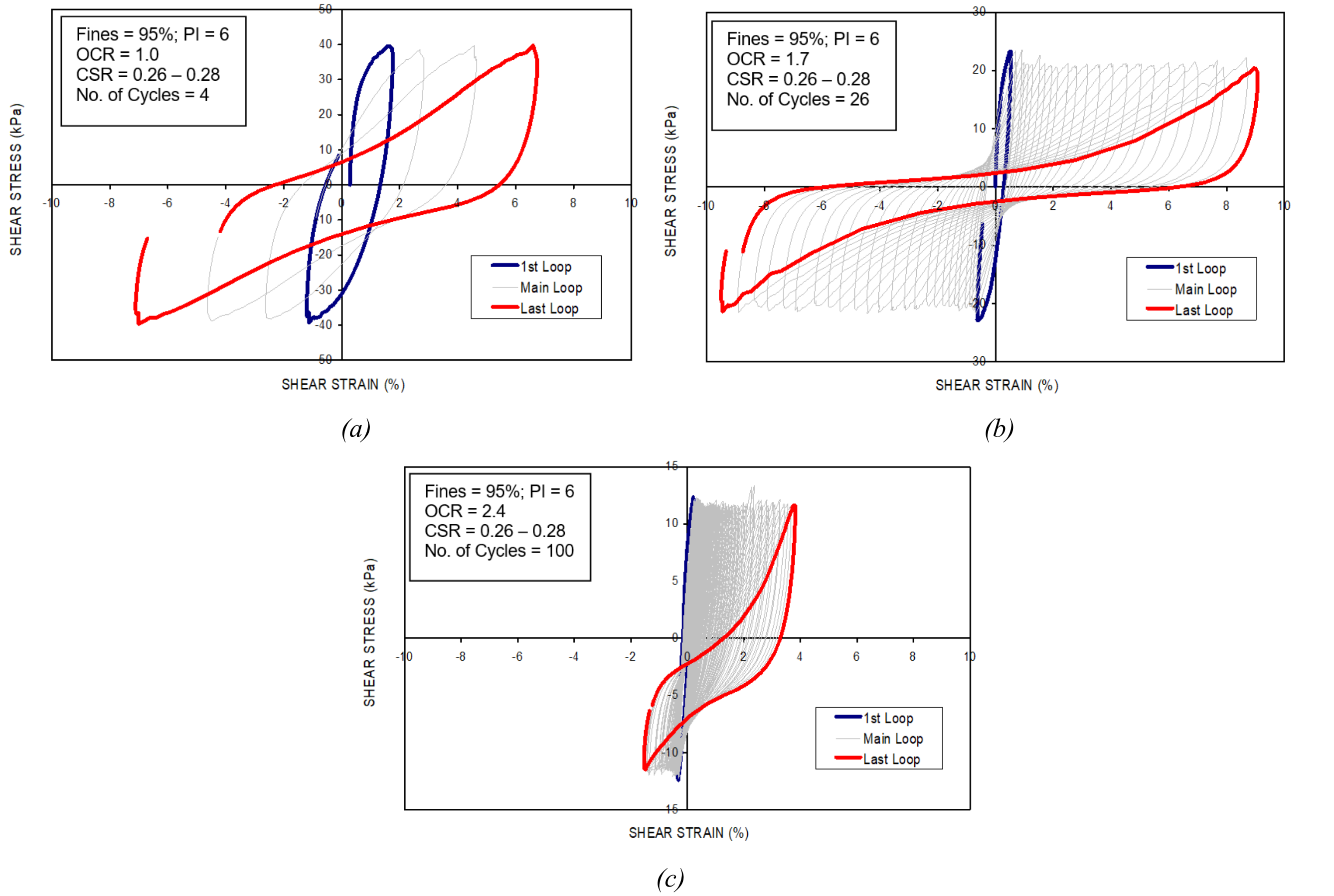


Figure 8. Stress-strain loops for silt sample consolidated to different OCR values: (a) OCR = 1.0, (b) OCR = 1.7 and (c) OCR=2.4.

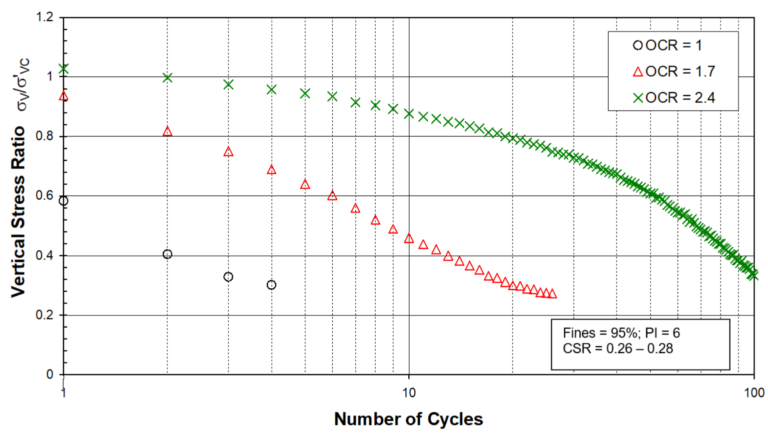


Figure 9. Variation of vertical stress ratio with number of cycles for silt sample with different OCR values (reduction of vertical stress is equivalent to increase in excess pore pressure during the constant volume (undrained) cyclic DSS test).

The laboratory test results and the empirical approach are in reasonable agreement to estimate the impact of overconsolidation ratio to cyclic resistances. For the CRR and number of cycles relationship, the results of cyclic DSS testing indicate that the plastic silts exhibit sand-like behavior. Comparing one clay (CL) sample with the highest PI of 16 and OCR of 1.3, and one silt

(ML) sample with PI of 6 and OCR of 1.7, the cyclic DSS results indicate the increases in CRR with higher OCR and the results also indicate the clay sample has a flatter MSF relationship than the silt sample (Figure 10).

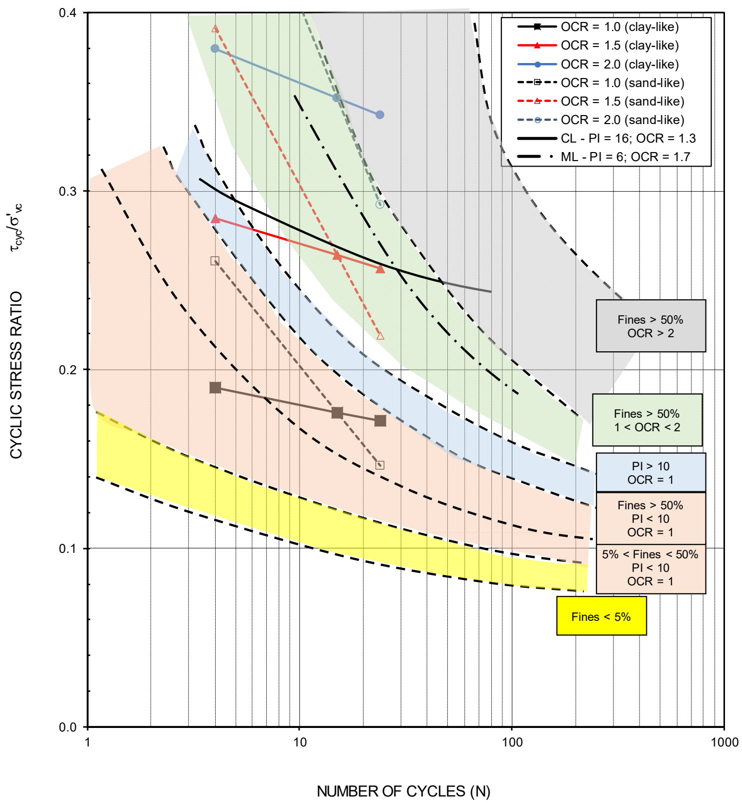


Figure 10. Comparison between cyclic DSS results and cyclic strengths recommended by Idriss & Boulanger [6].

It is also interesting to compare the results from the cyclic DSS laboratory tests with the method proposed by Bray et al. (2004). The index parameters for the overconsolidated silt samples are plotted on the Bray et al. (2004) chart indicated on Figure 5. The Bray et al. Chart would suggest that these samples are potentially susceptible to liquefaction or cyclic mobility. The results of the cyclic DSS tests would suggest that cyclic mobility or strain accumulation is more likely for a soil with these characteristics. The post-cyclic response would also appear to be more representative of a material with a higher PI, since the post-cyclic strength loss was determined to be negligible for the soils tested.

CONCLUSIONS

The results presented above clearly indicate the increased resistance to liquefaction caused by an increase in fines content of sand and the beneficial effect of OCR for soils with high fines content. The cyclic resistance also increases rapidly as the OCR increases. The effect of OCR would appear to be more significant than the effect of fines content. The effect of plasticity is small (about 10%) based on the limited test results presented herein. However, the laboratory tests are consistent with Ishihara [4] that the cyclic resistance will slightly increase with higher plasticity (PI > 10).

Overall, the laboratory and field liquefaction resistance curves are in reasonable agreement, especially for sands with low fines contents. This is not surprising since the field curves are based on measured/interpreted response during/after earthquake events. This does not change the earlier comment that the fines corrections applied for SPT and CPT data in fine-grained soils do not fully account for the effect of fines content. At numerous sites, we have consistently witnessed that the $(N_1)_{60}$ values in sands with silt contents of 15-30% or more, indicate higher susceptibilities to liquefaction than suggested by the results of cyclic DSS tests performed on recovered samples.

The CRR and number of cycles relationship (i.e. MSF) between sand-like and clay-like behaviors are in good agreement with plasticity at about 7-10%, as suggested by Boulanger & Idriss [6][18].

This conclusion would suggest laboratory testing such as cyclic DSS of representative undisturbed samples is recommended to determine the cyclic resistances of fine-grained soils. This conclusion is also consistent with the recommendations by Canadian Highway Bridge Design Code CSA S6-19 and Idriss & Boulanger [6]. However, it should be noted that the increased resistances to liquefaction due to fines content, OCR and plasticity may only provide beneficial effects at low to moderate earthquakes, since the significant increase in cyclic stress ratios generated by large earthquakes may dominate the soil response.

ACKNOWLEDGMENTS

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