Liquefaction of fine-grained soils from cyclic dss testing

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ABSTRACT
Generally accepted methods used to evaluate the potential for liquefaction have been developed primarily for clean sands or silty sands with up to about 35% fines content. As the fines content increases in sand, the in situ penetration test data is corrected in an attempt to compensate for the effect of fines content on the penetration resistance of the sand. Correction methods have been proposed for both SPT and CPT data. Where the silt content of sand is higher than 35%, correction factors do not appear to adequately reflect the increased resistance to liquefaction. Under these conditions, the generally accepted procedure is to base the assessment on laboratory tests. The paper presents the results of a series of cyclic simple shear tests performed on silty sands and silts from profiles in the Lower Mainland of British Columbia. The test results confirm that the increased resistance to liquefaction may not be adequately represented by the correction methods proposed for in situ penetration data.

RESUMEN
El enfoque normal para evaluar la resistencia a la licuacion se basa en la ejecucion de ensayos de penetration en campo. Sin embargo, se ha demostrado que la resistencia a la penetracion es sensible al porcentaje de finos en los suelos granular. Incrementos en la cantidad de finos causan una reduccion en la resistencia a la penetracion y una reduccion en la resistencia a la licuacion – particularmente si el porcentaje de finos es mayor a 35%. La ejecucion de ensayos dinamicos de laboratorio es el metodo preferido para evaluar la resistencia a la licuacion para suelos finos. El articulo presenta resultados de ensayos dinamicos (DSS) realizados con arenas limosas y limos de la zona de Vancouver, British Columbia y la comparacion de resultados con el grafico basado en la resistencia a la penetracion.

1 INTRODUCTION

The evaluation of the susceptibility of granular soils to liquefaction is commonly assessed based on in situ penetration test data. The method proposed by Seed et al. (1985) 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 is defined based on either the N value from the standard penetration test or the tip resistance (q_c) from a CPT sounding. The original chart presented by Seed et al (1985) 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 figure (Figure 1).

The measured penetration resistance in granular soil is affected by the fines content of the soil. As the fines content of sand increases, the penetration resistance is decreased. 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. Considering that the soil cyclic resistance or strength is defined based on the penetration resistance, then the correction to the penetration resistance also applies to the soil cyclic resistance (Figure 2). The correction is directly related to the fines content, but the plastic component of the fines may also be a contributing factor (Figure 3). Ishihara (1996) suggests that the correction for fines content and plasticity would be of the form:

$$(N_1)_{FC+PI} = N_1 + \Delta N_{FC} + \Delta N_{PI} \quad [1]$$

A similar correction also exists for the use of CPT data in fine-grained soils. Figure 2 indicates the correction required for a sand of equal relative density where the penetration resistance is affected by the fines content. A possible correction factor on the cyclic strength of fine-grained soils for the effect of plasticity, suggested by Ishihara (1996) is indicated on Figure 3.

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 (2004)
- Seed et al. (2003)
- Robertson and Wride (1998)
- Stark and Olson (1995)
- NCEER (1996 and 1998)
- Seed and De Alba (1986)

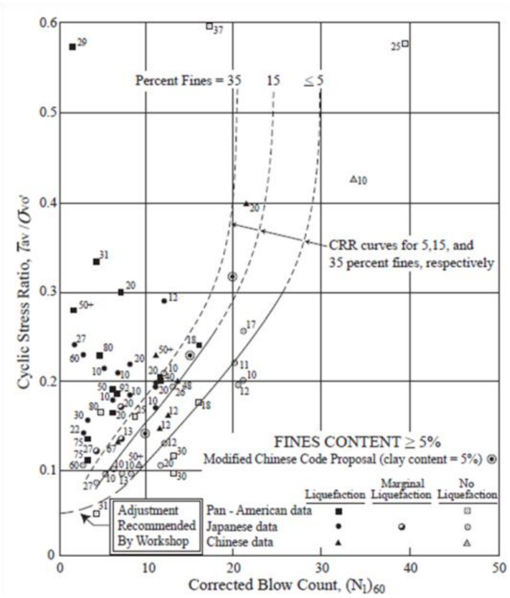

Figure 1. Evaluation of CRR from SPT data from empirical liquefaction data (Youd et al. 2001).

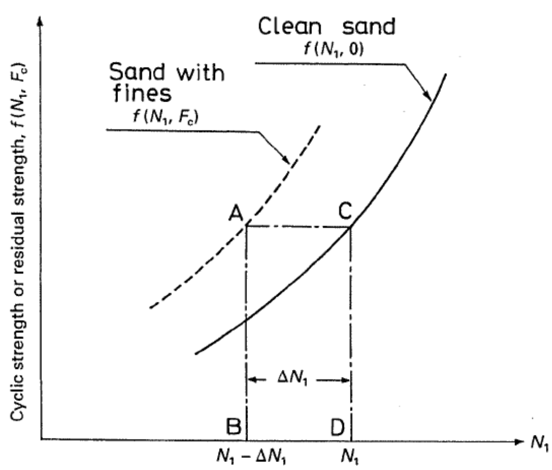

Figure 2. Definition of ΔN_{1FC} for sands of equal relative density but differing fines contents (Ishihara, 1996).

Typical correction values for both N_1 and q_{c1} as a function of fines content are indicated on Figure 4.

However, it is commonly thought that the correction for fines content underestimates the real impact of the fines on the penetration resistance (ATC/MCEER 2006). 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 10-15%. Furthermore, we believe that laboratory testing is the most reliable way of determining the liquefaction resistance of silt.

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 was 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

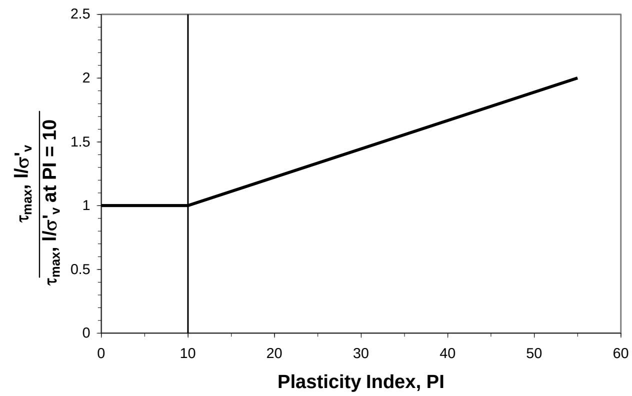

Figure 3. Modification of cyclic strength of fine-grained soils allowing for the effect of plasticity index (Ishihara, 1996).

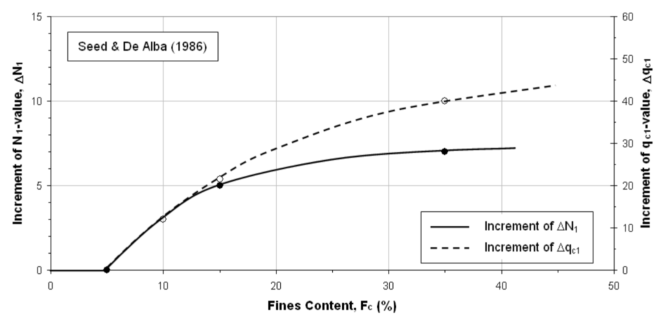

Figure 4. ΔN_1 and Δq_{c1} values as a function of fines content (Seed and De Alba, 1986).

Various modifications to the Chinese Criteria were subsequently proposed by other researchers. The Chinese Criteria were modified by Bray et al. (2004) based on observations following the Kocaeli earthquake in Turkey. The basic conditions proposed by Bray et al. (2004) are summarized on Figure 5.

Boulanger and Idriss (2004) 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 Boulanger and Idriss (2006), Bray and Sancio (2004) and Sanin and Wijewickreme (2006), the methodology recommended in the Greater Vancouver Task force report (2007) 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 to sensitive and overconsolidated soils.

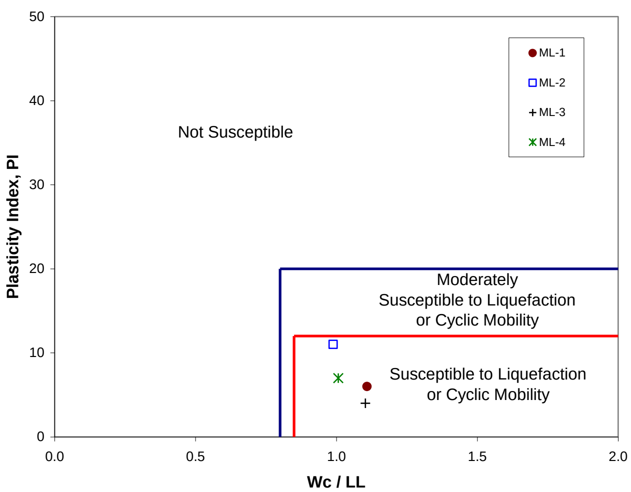

Figure 5. Application of the Bray et al. (2004) criteria for liquefaction assessment for silt.

2 TEST PROGRAM

Limited studies have been performed to assess the cyclic shear behaviour of fine grained soils (silty sands and silts). However, we believe 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 fine-grained deposits. 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 in the Lower Mainland of British Columbia and other locations in North and South America. Only a portion of the results are presented herein.

The testing was performed in the MEG Consulting Technical Services (MTS) Geotechnical Laboratory in Richmond, BC. MTS 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 using a rotating 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. The tube is cut into sections about 75-100 mm long which permits 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 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 were performed on selected 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 automatically adjusted (computer controlled) to maintain no overall change in volume of the sample during shear. Cyclic loading for the set of tests presented in this paper has been performed under stress controlled conditions. Shear wave velocity measurements were made on selected samples at different times during the test by means of bender elements seated in the top and bottom caps. Once the cyclic loading was completed, either post-cyclic static shear or volume change were measured on selected samples. However, the information presented in this paper covers only the cyclic resistance results.

The results presented in this paper are primarily from tests performed on good quality samples recovered in the field by thin-walled 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 samples prepared by moist tamping with another two samples prepared by dry tamping.

The results presented here cover two aspects of the test program completed:

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

3 TEST RESULTS

A summary of the samples considered in this paper and specific testing characteristics is presented on Table 1.

3.1 Fines Content

The effect of fines content for the samples tested is presented on Figure 6. The test results presented on Figure 6 are from samples consolidated to pressures 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 to silt with over 50% fines. The relative density of the sand samples was reasonably constant at around 45-55% to ensure similar field penetration resistances. The PI of the silt samples varied from non-plastic to a plasticity index of less than 12%.

The results on Figure 6 are for failure defined by initial liquefaction or 3.75% single amplitude strain. The results on Figure 6 clearly indicate the increased resistance to liquefaction that occurs in samples with similar field penetration resistances as the fines content of the sample increases.

For the case where 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. The increase in the resistance to liquefaction is clearly a function of the fines content, but would also appear to be dependent on the level of the cyclic stress ratio being applied (Figure 6).

3.2 Overconsolidation Ratio

Sanin and Wijewickreme (2006) have demonstrated the effect of overconsolidation on the resistance to liquefaction in silt. Testing was performed to verify the OCR effect for the design of ground improvement projects in the Lower Mainland.

The testing program considers the effect of the above parameters separately and in combination. The stress-strain loops (for up to 100 cycles) for a typical sequence of tests are presented on Figure 7 for the silt sample ML-1 tested under OCR values in the range from 1.0 to 2.4. Testing was performed in the 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/de-aired water through the sample under a nominal backpressure and conditions of constant volume.

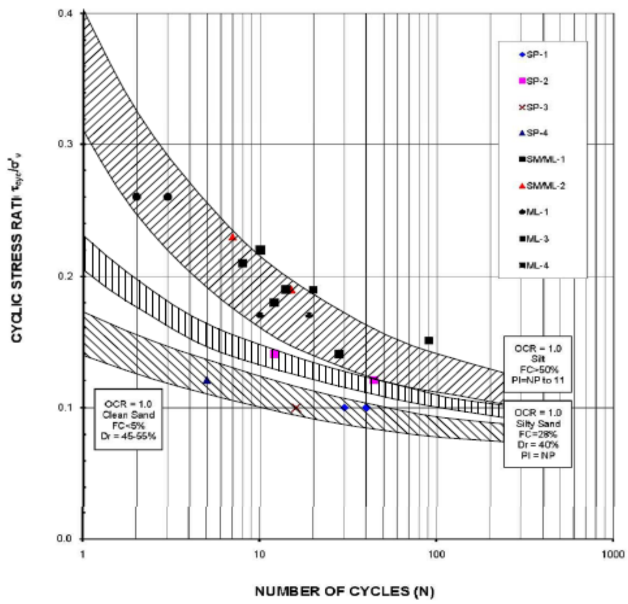

Figure 6. Results of cyclic stress-controlled DSS tests on sands with differing fines contents (F_c).

The cyclic stress ratio for all tests is 0.26-0.28; the first and last loops are highlighted on the figure. The change in the vertical stress ratio (equivalent to the pore pressure induced during cyclic loading) is presented for the same three tests on Figure 8. 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 an 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 7).

As indicated on Figure 8, after completing the first cycle, the rate of pore pressure generation with number of cycles is approximately equal 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 9. The lowest curve corresponds to normally consolidated clean sands with similar field penetration resistances. The remaining three ranges correspond to soils with fines contents greater than 50%, plasticity indices in the range non-plastic to 11% and overconsolidation ratio increasing from OCR=1 to OCR=2.4. for the clean sand, at N=15 cycles the average cyclic resistance ratio is about 0.11. This increases to 0.18 as the fines content increases to more than 50% (consistent with Figure 6). As the OCR increases to 1.7, for N=15 the CRR increases to 0.35, while for OCR=2.4 the range is asymptotic to a value of about N=30. Figure 9 confirms the significant effect that the OCR may have on the soil resistance to liquefaction.

The results on Figure 9 are for failure defined by initial liquefaction or 3.75% single amplitude strain.

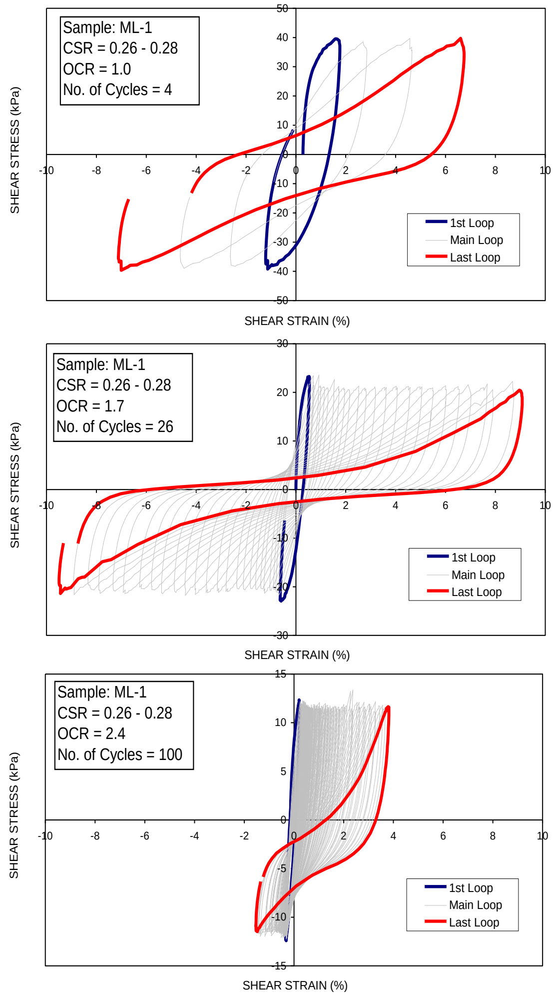

Figure 7. Stress-strain loops for sample ML-1 consolidated to different OCR values.

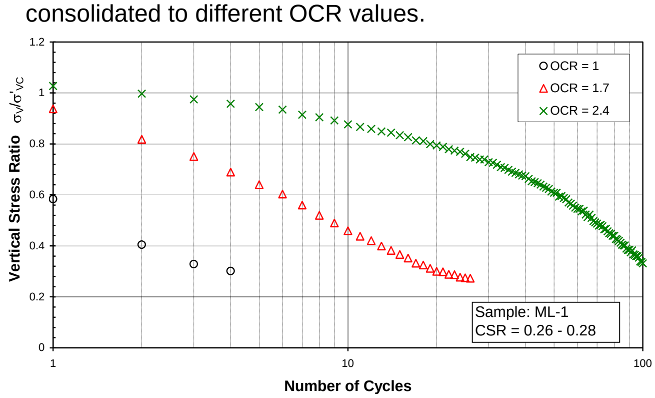

Figure 8. Variation of vertical stress ratio with number of cycles for sample ML-1 with different OCR values

It is 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 silt samples ML-1, ML-2, ML-4 and ML-4 are plotted on the Bray et al. (2004) chart indicated on Figure 4. The index properties are summarized in Table 1 below. 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.

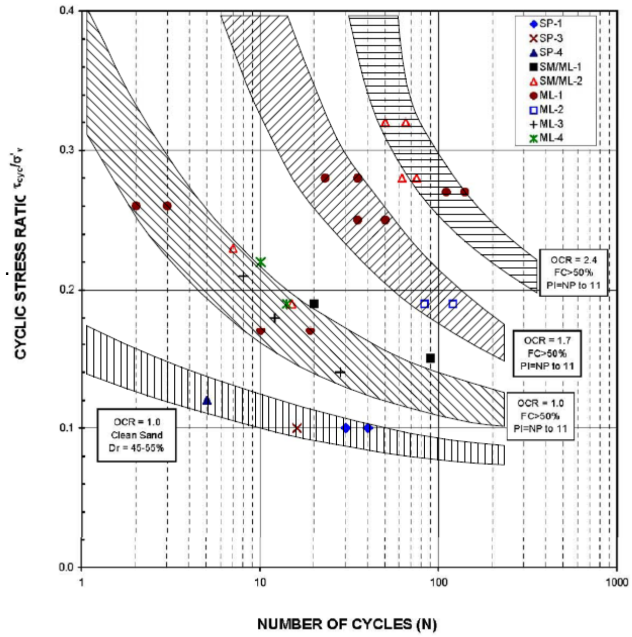

Figure 9. Results of the DSS tests on the cyclic strength of fine-grained soils under different overconsolidation ratio (OCR)

4 CONCLUDING COMMENTS

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, even if non-plastic or of low PI.

The trend in the increased resistance can be visualized by comparing the results of the cyclic DSS testing with the data provided by the Youd et al. (2001) liquefaction resistance curves (Figure 1) for the differing fines contents. Figure 1 is drawn for a M=7.5 earthquake which corresponds to 15 equivalent cycles of shaking. In order to generate the appropriate ordinates for other numbers of cycles, the magnitude scaling factors proposed by Idriss (2009) have been applied to Figure 1.

The comparison is indicated on Figure 10. The results of the cyclic DSS tests are indicated as solid or dashed lines, depending on the fines contents. The shaded areas on Figure 10 correspond to the three fines contents considered on the Youd et al (2001) chart (Figure 1). For the clean sand shaded area at the bottom of Figure 10, the lower and upper ranges correspond to a variation in $(N_1)_{60}$ of 5-10. The numbers of cycles, N, vary from 4 for a M=6 earthquake to 24 for an M=8.2. The Youd et al. (2001) chart on Figure 1 is for M=7.5 with N=15.

Over the range from N=4 to N=24, the CRR from the laboratory tests on clean sand (Dr = 45-55%) agree reasonably well with the field data. At low numbers of cycles, the DSS data are slightly lower than the data provided by Youd et al., although the DSS data provide progressively higher resistances as the number of cycles of shaking increases. The differences in response are likely due to the contractive nature of the moist-tamped sample preparation method. Overall, the comparison between the field and laboratory results for clean sand is good.

At the upper end of the scale, the cyclic DSS results for the normally consolidated silt coincide with the upper limit for the Youd et al. fines content adjusted field curve. However, notably the cyclic resistance increases rapidly as the OCR increases. The effect of OCR would appear to be significant than the effect of fines content.

Overall, the laboratory and field liquefaction resistance curves are in reasonable agreement. 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 adequately account for the effect of fines content. At numerous sites, we have consistently witnessed that the (N1)60cs values in sands with silt contents of 15-30% or more, indicate higher susceptibilities to liquefaction than indicated by cyclic DSS tests performed on recovered samples. We are presently evaluating the results to pursue a more appropriate fines content correction. This conclusion would suggest that the ΔN_1 that is determined from the Seed et al. curve could be considered as a lower bound value for the correction of $(N_1)_{60}$ to $(N_1)_{60cs}$ in order to compensate for the effect of the fines content.

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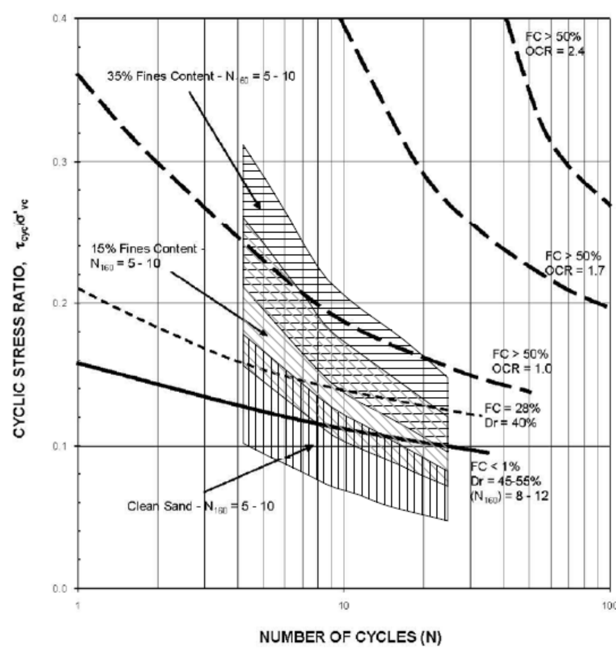

Figure 10. Comparison between DSS results and cyclic resistance curves recommended by Youd et al. (2001)

Table 1. Summary of properties for DSS tests

Soil Type	Description	Depth	σ'_{vo}	In-situ OCR (1)	PI	σ'_v test	OCR test (2)	Fines (%)	Relative Density, Dr (%)	Sample preparation method
SP - 1	Fine SAND with mica	16.5	165	-	-	165	-	0	55	Dry deposition, tamping
SP - 2	Fine silty SAND	11.2	125	-	-	139	-	28	40	Dry deposition, tamping
SP - 3	Fine SAND	17.7	155	-	-	155	-	1	53	Moist tamping
SP - 4	Fine SAND	35.3	321	-	-	321	-	1	45	Moist tamping
SM/ML - 1	Sandy SILT	13.4	135	-	Non-Plastic	148	-	61	-	undisturbed sampling
SM/ML - 2	Interbedded sandy SILT to silty sand	10.3	90	8	Non-Plastic	864	1	85	-	undisturbed sampling
						360	2.4			
ML - 1	SILT with some clay and trace sand	4.8	44	2	6	150	1	95	-	undisturbed sampling
						77	1.7			
						44	2.4			
ML - 2	Clayey SILT with some sand	8.8	86	1.9	11	105	1.7	67	-	undisturbed sampling
ML - 3	Clayey SILT with organics	4.8	50	4.8	4	288	1	81	-	undisturbed sampling
ML - 4	SILT with some sand	13.5	112	4	7	812	1	75	-	undisturbed sampling

Notes:
(1) OCR interpreted from field and laboratory test results
(2) OCR imposed on cyclic DSS samples