Ground Sampling and Laboratory Testing on Low Plasticity Clays

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SUMMARY:
A comprehensive field and laboratory testing program was carried out to characterize cyclic stress strain behavior of various low plasticity clay layers. Undisturbed samples were collected using the fixed piston sampling technique. Consolidation, monotonic and cyclic direct simple shear tests were performed on select soil samples. Cone Penetration Testing was used for site characterization as well as delineating different subsurface soil layers.
The low plasticity clays tested in this site showed a "clay-like" behavior. The Cone Penetration Test proved useful in characterizing the soil profile but necessary for identifying sampling depths. The over-consolidation ratio was shown to be a helpful index for characterizing monotonic and cyclic response of the low plasticity clays in this study. The materials were observed to develop significant excess pore water pressures and large shear strains under cyclic loading. The post cyclic monotonic tests suggested dilative behavior and strength gain at the cost of additional straining.

Keywords: Liquefaction, low plasticity, clay, silt, laboratory testing

1. INTRODUCTION

Loss of strength and development of strains for saturated cohesionless soils such as clean sands or non-plastic silts under cyclic loading is commonly referred to as liquefaction. For cohesive soils such as clays and plastic silts, the loss of strength and development of strains under cyclic loading is generally referred to as cyclic softening. Evaluation of the seismic behavior of low plasticity clays remains a challenge in practice because the fundamental behavior of the material is not completely understood.

The first step in assessing the seismic behavior of low plasticity clays is to assess the likelihood of strength loss and development of strains during cyclic loading using soil index properties like plasticity index (Seed and Idriss, 1982; Koester, 1992; Andrews and Martin, 2000; Bray and Sancio, 2006; and Idriss and Boulanger, 2008). The next step is to evaluate the triggering potential for strength loss during seismic loading by estimating the cyclic resistance ratio of the soil. The cyclic resistance ratio for low plasticity clays can be estimated using case history-based liquefaction correlations of standard penetration tests (Idriss and Boulanger, 2008), cone penetration tests (Robertson and Wride, 1998; Suzuki et al., 1997; and Idriss and Boulanger, 2008), and in situ shear wave velocity measurements (Andrus and Stokoe, 2000). However, direct measurement in the laboratory (e.g., triaxial, direct simple shear, or torsional shear) provides the best estimates of the cyclic resistance ratio provided the effects of sample disturbance are managed (Dahl, 2011).

This paper summarizes a case study carried out on a hydropower project site located in a high seismicity area in British Columbia, Canada. The study involved comprehensive field and laboratory investigation programs to locate, characterize and assess post-cyclic stress strain behavior of low plasticity clay layers at the foundation of the structures sensitive to differential settlements and lateral deformations. State of the art sampling and testing techniques were employed and the results were used to evaluate residual deformation of the foundation.

2. SITE SEISMISITY

The project site was located in a high seismicity area on the west coast of British Columbia. Based on the Canadian Dam Association guidelines (CDA, 2007), the project was classified as a “High” consequence structure. For this consequence category, the recommended minimum requirement is that the structure should be assessed for probabilistic maximum design earthquake ground motions with an annual exceedance frequency of 1 in 2,475 years. The horizontal peak ground acceleration corresponding to this return period for the project site was assessed to reach 0.48 g. Based on the project performance criteria, the structure must return to service after 72 hours following the design earthquake event.

3. GENERAL SITE GEOLOGY

Sediments at the project site were deposited during the Fraser glaciation and postglacial periods between 30,000 to 13,000 years ago. The glacial deposits generally consist of a very dense layer of sand, known as Quadra Sand, overlain by a glacial till blanket over the Basalt bedrock. The glacial units were not considered to be a concern in this case study and will not be discussed further. The postglacial deposits at the project site consisted of the following three major units from bottom to top:

Lower Clay: An up to about 20 m thick deposit of grey low plasticity clay deposited in relatively deep water on the coastal lowlands with occasional seashells which attest to its marine origin.

Interbedded Clay and Sand: This unit was deposited in a very complex sedimentary sequence because of changing sea levels, isostatic rebound, water discharge, waves, tides and other factors. The interbedded clays and sands are generally clay-dominant ranging between thinly laminated beds separated by very thin, fine sand seams to massive layers up to 10 m in thickness.

Deltaic Sands: This deposit, ranges from silty fine sand with sand and silt seams near the bottom of the layer, and coarse gravelly sands near the surface.

4. FIELD INVESTIGATION PROGRAM

4.1. Site Characterization

The proposed field investigation program was mainly focused on two areas near two natural slopes within the project site. In this paper, these areas are referred to as test zones 1 and 2. A mud-rotary drillhole was initially drilled at each test zone and Standard Penetration Tests (SPT) were carried out in order to obtain a general understanding of the foundation material under the proposed structures. A Cone Penetration Test (CPT) was then carried out at a distance of about 3.0 m from the SPT drillholes. CPT results provided a continuous soil profile and sample depths were located using the CPT logs. A careful sample depth selection procedure helped the authors target the required layers with high precision and avoid damaging the sampler tubes in dense or coarse-grained soil layers. A mud rotary drillhole was finally advanced for piston sampling; forming an equilateral triangle at about 3.0 m spacing between the three test holes (i.e., SPT hole, CPT hole and sample hole) at both test zones. In test zones 1 and 2, the overburden material would be excavated and the proposed structures would be founded at depths of about 18.0 m and 5.0 m below the existing ground surface, respectively. Table 1 shows the sequence of the material encountered at each test zone.

Table 1. Soil Profiles in Test Zones 1 and 2 (above Glacial Till)

Test Zone 1		Test Zone 2	
Depth (m)	**Soil Description**	**Depth (m)**	**Soil Description**
18.0 - 26.0	Sand	5.0 – 12.6	Intermixed Silt and Sand
26.0 - 35.2	Interlayered Clay and Sand	12.6 – 18.5	Interlayered Clay and Sand
35.2 - 45.2	Interlayered/Interbedded Clay and Sand	18.5 – 22.5	Stiff to Hard Clay (Hard Clay)
45.2 - 49.0	Fine-Grained Sand	22.5 – 30.5	Marine Clay (Lower Clay)
49.0 - 59.5	Marine Clay (Lower Clay)		

The CPT results and the layers at both test zones are shown in Figure 1. The hydrostatic water pressure obtained from standpipe piezometers and CPT pore water pressure dissipation tests is also included on the pore water pressure plot. The identified clay and interlayered/interbedded layers were targeted for piston sampling. The sampling program was planned to provide enough samples for laboratory testing in order to characterize each fine-grained soil layer.

4.2. Fixed Piston Sampling

Mechanically operated 76 mm fixed piston samplers with stainless steel tubes were used for sampling. The tubes were 76 cm long, with a nominal inside diameter of 73 mm and wall thickness of 1.6 mm. Half of the tubes were crimped inwards at the tip to create a positive inside clearance. The rest of the tubes were machined to have sharp ends at 5 degrees without clearance. A total of 39 tubes were used for sampling.

Sampling was carried out in a cased drillhole which was advanced carefully with tricone and drilling mud to the sample target depth. The casing followed with a few meters distance to assure easy cleaning of the bottom of the hole without causing additional disturbance. The depth of the sampling was measured to the closest centimeter to accurately identify the location of the sampled soil and assure a

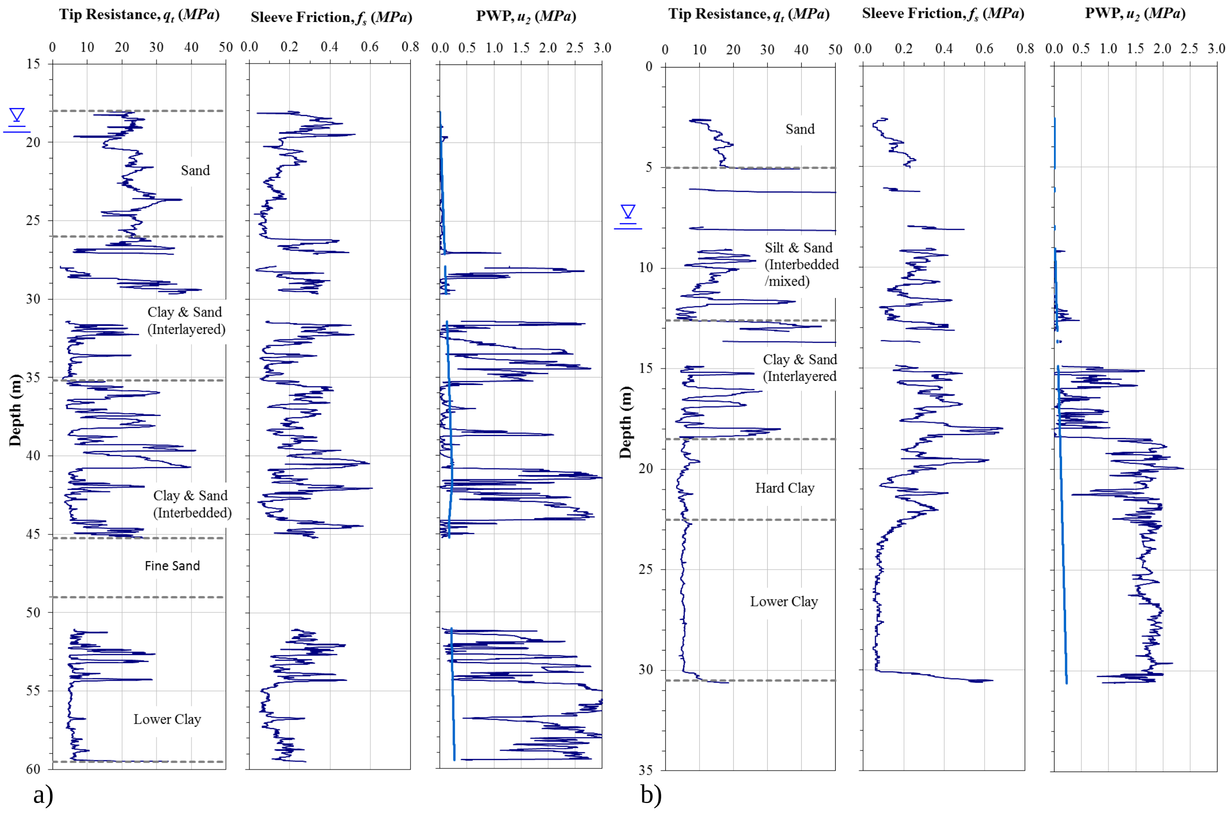


Figure 1. Soil profiles and CPT results (tip resistance, sleeve friction, and pore water pressure) at a) Test zone 1; b) Test zone 2

clean drillhole bottom. The piston sampler had two independent components including a piston that was fixed to the drill rig and a tube that was pushed into the soil. The fixed piston stayed on top of the sampled soil and resisted the suction applied to the sample during extraction. Movements of the tube and the minor movements of the piston were monitored to the closest millimeter using surveying equipment. The tubes were pushed at a constant rate of 20 mm/sec and the maximum pressure applied to the tubes was recorded. The pressure was correlated with the damage observed in the tubes upon extraction and was used as a reference to minimize the damage in subsequent attempts. The samples were kept in a temperature and moisture controlled environment for two weeks during the program and then shipped to the soil testing laboratories in isolated containers. The 250 km distance between the project site and the laboratories was equally split between slow driving and a ferry ride. The maximum accelerations applied to the sample boxes were recorded as part of the quality assurance program.

5. LABORATORY TESTING PROGRAM

5.1. Preliminary Sample Disturbance Assessment

The first stage of assessment of sample disturbance was carried out by visual inspection of the tubes after they were extracted from the drillhole. Any damage to the tubes or any obvious loss of sample inside the tubes was recorded. The interbedded denser sand seams were a particular challenge causing excessive resistance and damage to the tubes in many cases. Both the length of the tube push and the length of the recovered sample were accurately measured. The ratio of the two measurements was expressed in percentage and referred to as sample recovery. A total of 17 tubes with no physical damage and 100% recovery were selected for testing on samples collected from test zones 1 and 2.

5.2. Sample Extrusion

Gamma-Ray imaging was used to identify any potential defects inside the tubes prior to sample extrusion. This technique proved to be effective in identifying the voids and cracks existing in soil samples inside the tube. In many cases the fabric of the coarser materials and the interbedded seams could also be identified. After carefully selecting the exact testing locations, the tubes were cut using manual rotary tube cutters by applying very mild pressures. A 3 cm space between the tested samples and the cut was allowed for all samples. The soil was then extruded using a hydraulic piston extrusion device at maximum 15 cm long sections. The samples were then trimmed to the appropriate lengths and diameters using a wire cutter.

5.3. Soil Testing Scope

A total of 223 tests were performed by two soil testing laboratories. Table 2 provides a list of all the tests completed. Atterberg limits were measured for every tested sample to establish a material type index. In cases where significant material variability was obvious in the soil specimen within a tube, some of the trimmings were also tested for Atterberg limits and moisture contents. The plasticity charts for the two test zones are shown in Figure 2. The cohesive materials in this site were characterized as low plasticity clays. The marine lower clays were more plastic than the deltaic interlayered materials. All the materials fell above the transition zone for liquefaction assessment purposes and were assumed to show a “clay-like” behavior and thus the SHANSEP framework (Ladd and Foott, 1974) was used to plan the testing.

The testing program focused on cyclic simple shear tests on clay samples. The purpose was to assess the cyclic behavior of these materials under seismic loading. The Cyclic Resistance Ratio (CRR) and the post cyclic residual strengths were the main parameters of interest in these tests. Monotonic simple shear tests were performed to estimate the static characteristics of the materials. Consolidation tests (conventional and constant rate of strain) were performed to establish the level of over-consolidation and estimate the compressibility of the materials.

Table 2. Summary of laboratory tests done on clay samples

Test	Number Completed
Conventional Consolidation on Clays	8
CRS Consolidation on Clays	8
Cyclic-DSS on Clays	41
Monotonic-DSS on Clays	19
Plasticity Index	88
Sieve Analysis (including sand samples)	44
Hydrometer	15
Total	223

5.4. Conventional Consolidation Tests

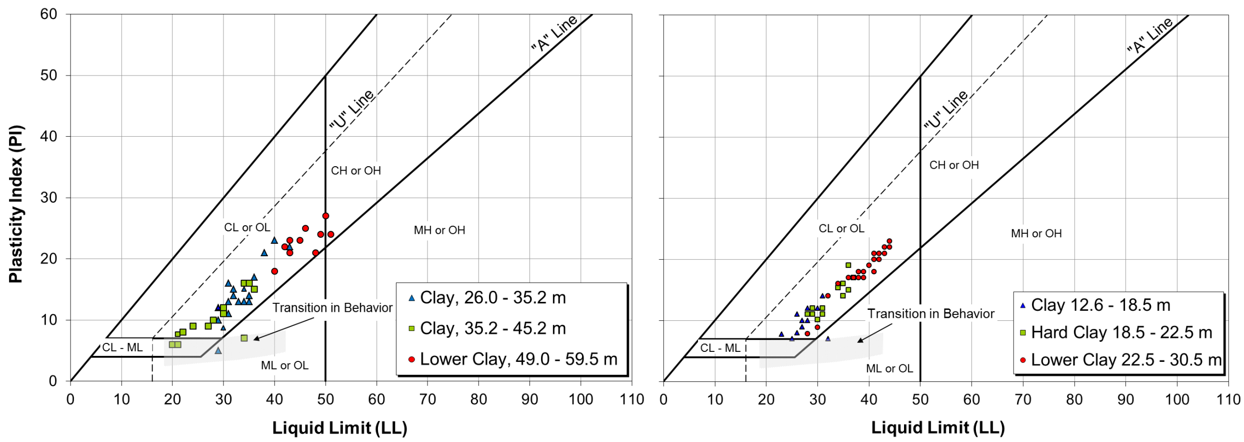
The testing program started with eight conventional Oedometer tests. The tests were selected so that at least one test was performed on each clay material (there were a total of six layers/materials identified). After a seating pressure of 5 kPa was applied, the vertical stress was increased to the in-situ stress level with a load increment ratio (LIR) of 2. The vertical load was then increased to a multiple of the in-situ stress which was smaller than the maximum stress allowed by the equipment capacity. The vertical stress was then reduced back to the in-situ stress to estimate the rebound parameters. Finally the sample was loaded back to the maximum stress level before it was removed from the Oedometer.

5.5. Constant Rate of Strain (CRS) Consolidation Tests

CRS consolidation tests provide a continuous compression curve which can be used to identify the pre-consolidation stress of a soil sample. Eight CRS tests were performed. The samples were chosen from the tubes which were not selected for conventional consolidation tests. The same loading path applied to the conventional consolidation tests was applied to the CRS tests. The samples were unloaded back to the in-situ stress at the end of the tests.

5.6. Monotonic Direct Simple Shear Tests

Monotonic direct simple shear tests were performed on one sample from each tube which was considered undisturbed. A specific sequence of vertical loads was applied to the sample prior to shearing. A seating pressure of 5 kPa was applied for one hour. The stress was then increased to the prescribed OCR level, estimated based on the preceding consolidation test results, and maintained for six hours. The vertical stress was then lowered to the in-situ level and maintained for another six hours.



a) b)

Figure 2. USCS classification charts: Atterberg limits and the range of transition in liquefaction response (after Idriss and Boulanger, 2008) a) Test zone 1; b) Test zone 2

The OCRs selected for testing were within the range of or larger than the best estimate of the OCR in the layer. For example, the OCR for the interlayered clay and sand between depths of 26.0 m and 35.2 m in test zone 1 was estimated to range between 1.7 and 2.3; and the monotonic direct simple shear tests were performed at OCRs of 1.7, 2.0 and 2.5. The intent was to establish an over-consolidation level slightly above the in-situ conditions to minimize the effects of sample disturbance. The soil strength parameters were estimated by back calculating to the estimated OCR once a trend was established using the SHANSEP approach.

After completion of the vertical loading sequence, the samples were sheared under constant volume conditions to 20% strain at a strain rate of 1% per hour. At the end of shearing, the vertical stress was recorded for 15 minutes under constant volume conditions. This was done to identify any stress or pore water pressure redistribution after the test was complete. After this stage the sample was consolidated under its in-situ vertical stress for 6 hours.

5.7. Cyclic Direct Simple Shear Tests

Cyclic direct simple shear tests were performed on several samples from each layer. Different levels of cyclic loading were applied to different samples to establish a trend between the CSR and the number of cycles to cyclic failure. A similar sequence of vertical loading applied in the monotonic tests was used to establish the required OCR. Two OCR levels were chosen such that a range of OCRs became available for each material. The intent was to estimate cyclic response of the sample by back calculating to the estimated OCR once a trend between the OCR and the parameters (e.g. CRR) was established.

Load controlled constant volume cyclic shear was applied with a frequency of 0.5 Hz until one of the following conditions was met: 5% single amplitude strain was reached with 30 cycles or more, 15% single amplitude strain was reached, or 150 cycles were applied to the sample. In the latter case, the sample was reconsolidated to the in-situ vertical stress and the test was repeated with a higher CSR.

After completion of each cyclic loading test, the shear stress was brought back to zero and the vertical stress was recorded for 15 minutes under constant volume conditions. The sample was then monotonically sheared to 20% strain at a rate of 5% per hour in the direction of the post cyclic residual strain. The vertical stress was recorded again for 15 minutes under constant volume conditions. The sample was then brought back to zero shear strain and consolidated under the in-situ stress for 6 hours.

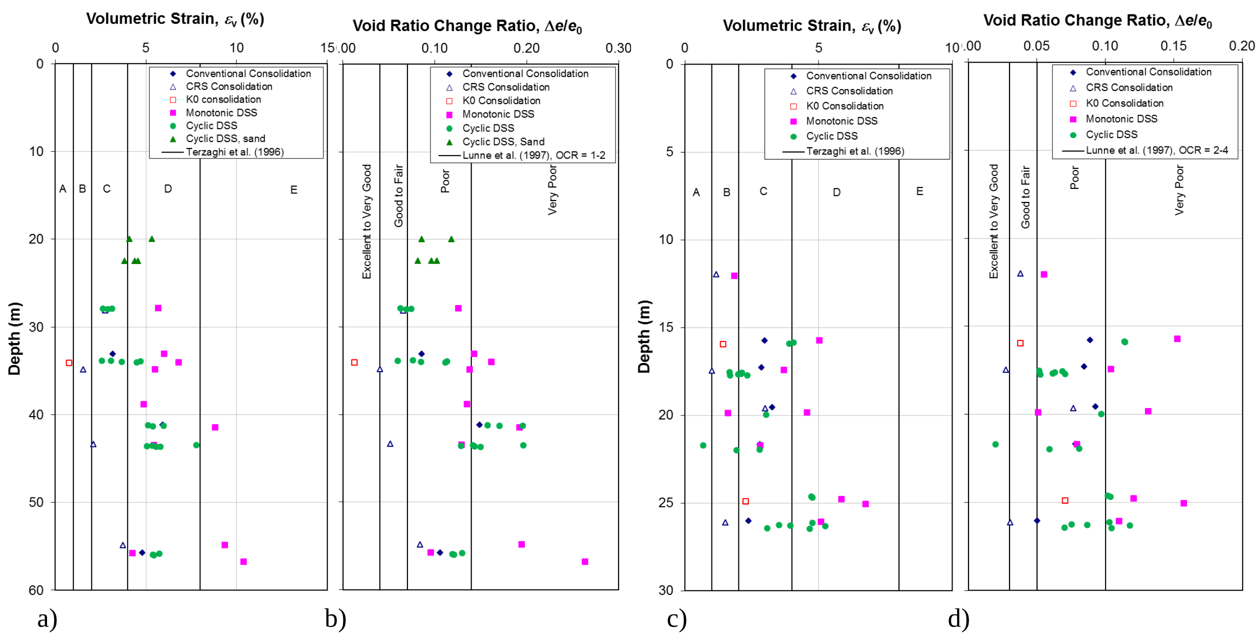


Figure 3. Disturbance indexes at test zone 1: a) Terzaghi et al. (1996); b) Lunne et al. (1997); and test zone 2: c) Terzaghi et al. (1996); d) Lunne et al. (1997)

6. LABORATORY TEST RESULTS

6.1. Sample Disturbance

The level of sample disturbance for the soil samples used in consolidation, monotonic and cyclic simple shear tests were evaluated using the methods proposed by Terzaghi et al. (1996) and Lunne et al. (1997). Both methods are based on the observations made by Andersen and Kolstad (1979) that in fine grained soils, an increase in sample disturbance should result in an increase in the volumetric strain when the samples are re-consolidated to the in-situ vertical effective stress.

Terzaghi et al. (1996) defined sample quality using the Sample Quality Designation (SQD), which varies from A (best) to E (worst) based on the amount of the volumetric strain that occurs during re-consolidation to σ'_{v0}. Lunne et al. (1997) used the change in void ratio ($\Delta e / e_0$) that occurs during re-consolidation to σ'_{v0} and OCR to rate the sample quality from excellent to poor. Lunne et al. (1997) considered $\Delta e / e_0$ a better index for sample quality since the change in void ratio is more damaging in soils with lower void ratios. They also suggest that for good quality samples, the change in void ratio should be smaller for higher OCR samples. The sample quality assessment information from both methods is plotted in Figure 3. Both methods suggest a wide range of sample qualities ranging between good to very poor (Lunne et al., 1997) or B to D (Terzaghi et al., 1996). An increasing level of disturbance was detected with depth at both test zones, with test zone 1 offering a clearer trend.

6.2. Consolidation Tests Results

Pre-consolidation stresses were estimated using the methods proposed by Casagrande (1936) and Becker et al. (1987). The well-known Casagrande method uses the point of maximum curvature on the e – log p' plot for estimating the pre-consolidation stress. The Becker method considers the work done per unit volume of the sample for obtaining the pre-consolidation stress. Both methods were used to process all the consolidation test results as shown in Figure 4. The two methods returned fairly consistent estimates of OCR. The geological history of the site did not point to a mechanism of over-consolidation that could explain the observed trends. The apparent over-consolidation was likely linked to aging, cementation, desiccation, and other physiochemical processes.

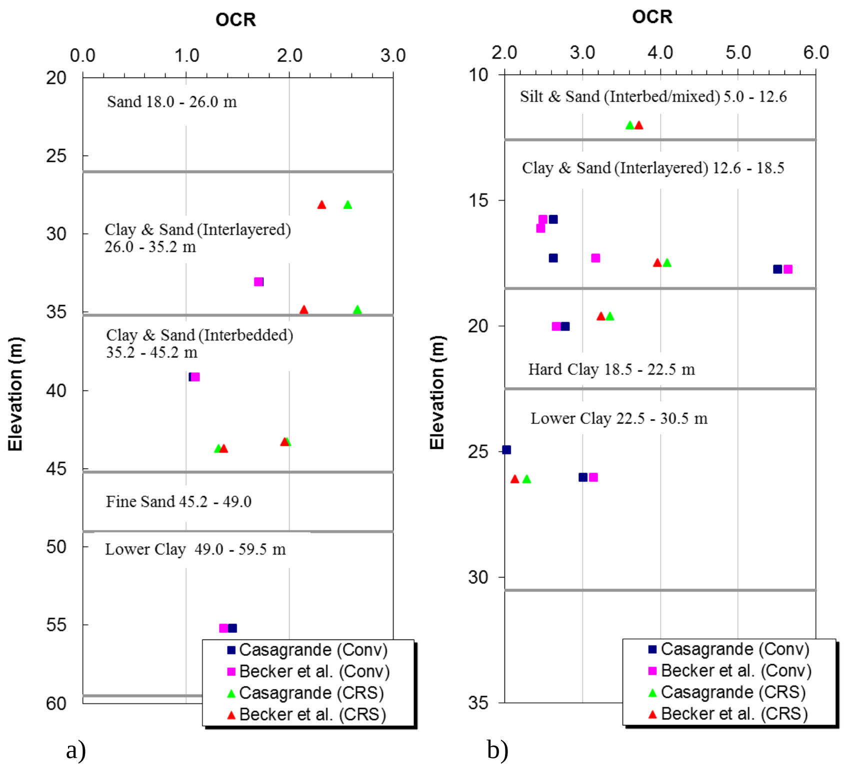


Figure 4. Summary of OCR values against depth for a) Test zone 1; b) Test zone 2

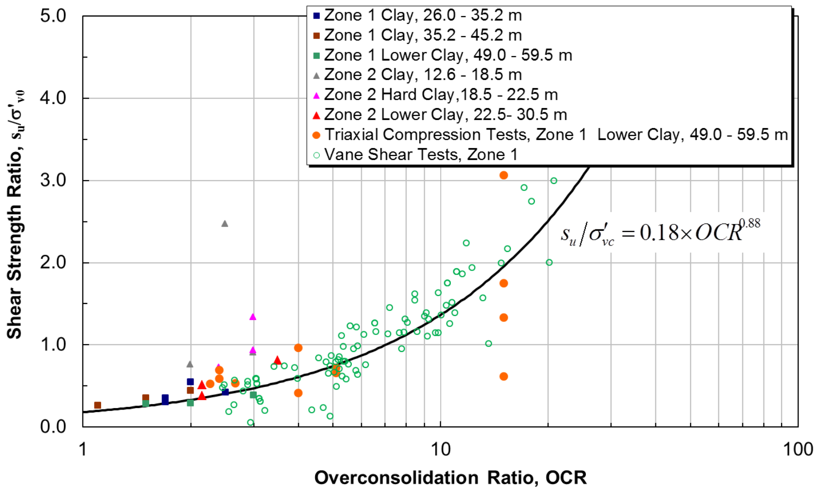


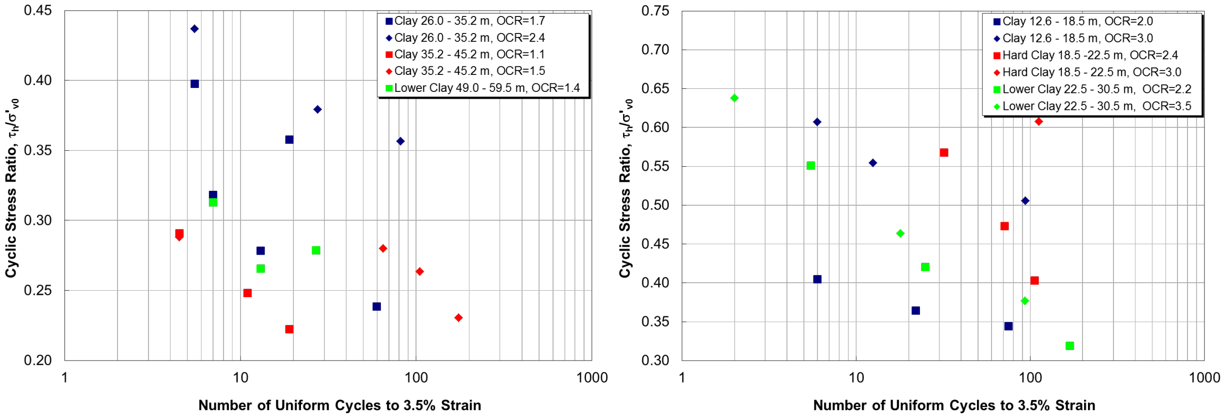
Figure 5. Variation of shear strength ratios versus OCR from monotonic direct simple shear, triaxial compression and field vane shear tests

6.3. Monotonic Direct Simple Shear Test Results

The results of the monotonic direct simple shear tests were compiled for each material type and are plotted as undrained shear strength ratio (s_u / σ'_{v0}) versus over-consolidation ratios in Figure 5. Triaxial test results from previous investigations are also included in the figure. These tests were carried out on samples collected from conventional Shelby tubes in the past. The OCR values for these tests were obtained from the estimated values for the lower clay layer as reported earlier in the paper. These tests were directly consolidated to their target stress level, so the OCR levels were not re-established in the laboratory. Consequently, for samples that were consolidated to lower stress levels, a higher OCR was assumed. This factor and a cruder sampling technique have likely contributed to the scatter in these data. Field vane shear test results from previous investigations are also plotted in this figure using the estimated OCR for the associated layers. Despite the scatter in the data, a single trend is established against OCR reconfirming the applicability of the SHANSEP concept to these materials. The curve is represented by the SHANSEP equation $s_u/\sigma'_{vc} = S \times OCR^m$ with coefficients S = 0.18 and m = 0.88. The triaxial and field vane data conform to the results obtained from monotonic direct simple shear tests despite the different modes of shearing.

6.4. Cyclic Direct Simple Shear Test Results

The results of the cyclic direct simple shear tests on the materials from the two zones are plotted in



a) b)

Figure 6. Liquefaction (cyclic softening) triggering charts; Cyclic Stress Ratio versus number of cycles to 3.5% single amplitude shear strain; a) Test zone 1; b) Test zone 2

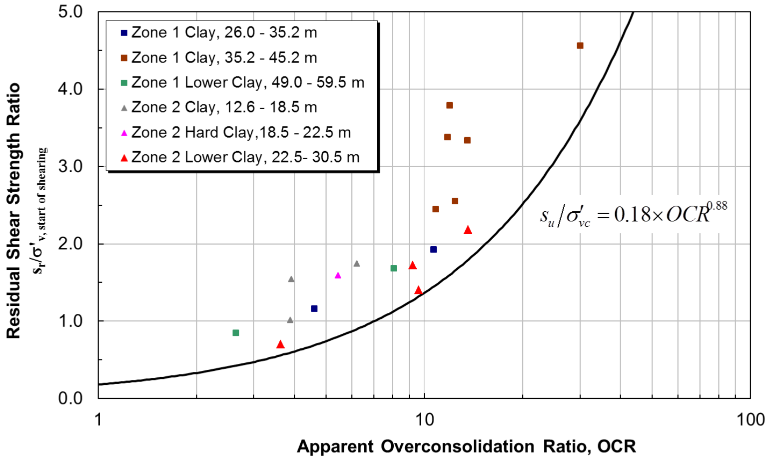


Figure 7. Variation of residual shear strength ratios versus apparent OCR from post-cyclic monotonic direct simple shear tests

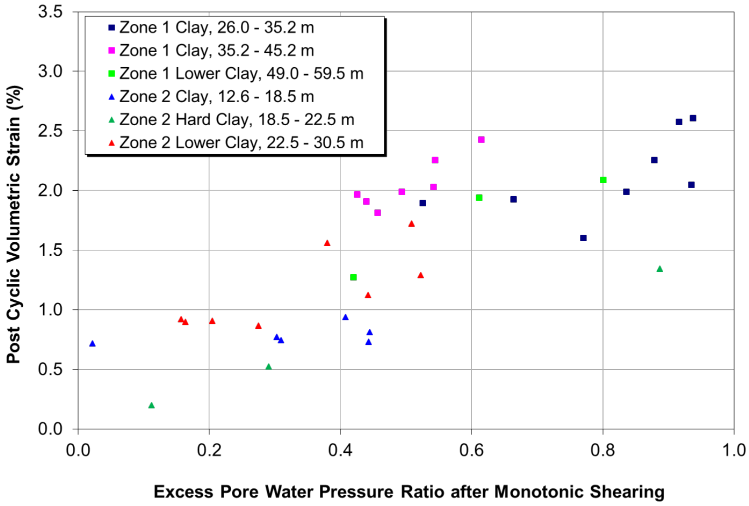


Figure 8. Post cyclic volumetric strain versus maximum excess pore water pressure experienced by the sample

Figure 6. The Cyclic Stress Ratio τ_h / σ'_{v0} which was applied to each sample is plotted against the number of cycles to cyclic failure defined as 3.5% single amplitude shear strain. The Cyclic resistances increase with the applied OCR for each material. The materials showed a largely dilative behavior in post-cyclic monotonic shearing with peak strengths reached around 15% strain. It was found that a similar trend to that of Figure 5 exists for post cyclic monotonic tests if the ratio of the residual shear strength to the vertical effective stress at the beginning of monotonic shearing $s_r/\sigma'_{v,\text{start of shearing}}$ is plotted against the apparent OCR defined as the ratio of the maximum vertical stress the sample has ever experienced to the vertical effective stress at the beginning of monotonic shearing. The trend is compared to that established for the shear strength ratios of monotonic tests in Figure 7. The post cyclic volumetric strain measured in consolidation is plotted against the pore water pressure ratio at the end of monotonic shearing in Figure 8. A direct relation is evident.

7. CONCLUSIONS

Cone Penetration Testing results provided a valuable reference to select sample depths and target specific soil layers. Ultimate care was taken to minimize sample disturbance.

All the samples selected for laboratory testing had 100% recovery; however, samples exhibited volume reduction during the initial consolidation stage of the tests. The disturbance levels of the soil samples were evaluated as “good” to “very poor”.

The low plasticity clays tested for monotonic shear strengths conformed to the SHANSEP framework. Available triaxial compression test results and field vane shear test data followed the same trend.

The over-consolidation ratio appeared to have a controlling effect on the resistance of these materials to cyclic softening. In general, their cyclic response was similar to that observed for other natural soils with similar characteristics.

The materials developed significant excess pore water pressures and large shear strains under cyclic loading. The post cyclic monotonic tests suggested dilative behavior and strength gain at the cost of additional straining.

The apparent over-consolidation ratio defined with respect to the effective stress at the beginning of post cyclic monotonic shearing was shown to correlate well with the residual shear strength ratios.

Volumetric strains accumulated during the post-cyclic consolidation phase were in the range of 1 to 2 percent and showed a direct relation with the pore water pressure developed during cyclic loading.

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