

The strength behavior of resedimented Presumpscot Clay and a comparison with intact behavior

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ABSTRACT: Laboratory consolidation and triaxial shear tests were performed on resedimented samples of Presumpscot Clay to establish the variation in void ratio, coefficient of lateral earth pressure (K_0), permeability, undrained strength and friction angle over the effective stress range of 0.2 MPa to 100 MPa. The value of K_0 remains within the range of 0.46 – 0.51 during normal compression for stresses up to 10 MPa. The undrained strength ratio of the clay varies remarkably little, between 0.30 – 0.32 at an overconsolidation ratio of one. The results of tests performed on intact samples compare very favorably with those from the corresponding resedimented material, demonstrating that the strength properties of intact Presumpscot clay can be closely replicated using laboratory resedimentation. When compared to seven other soils (liquid limits from 26 – 79 %), Presumpscot clay displays the least variation in strength properties and K_0 across a very wide range of effective stresses.

1 INTRODUCTION

Although the mechanical behavior of fine-grained soils is now relatively well understood for the range of stresses conventionally encountered in geotechnical engineering practice, below about 1 MPa, behavior at higher stresses is less well understood. On the other hand, the field of rock mechanics has traditionally been associated with the study of lithified materials, often at pressures in excess of 100 MPa. As a result, the mechanical behavior of intermediary materials, which include hard clays and unlithified or weakly lithified soft clay-shales, is far less well understood. Such materials are the most abundant in the uppermost 5 km of the Earth's crust (Petley 1999). In recent years a desire to gain a much better understanding of the behavior of these materials has been driven primarily by the petroleum industry for applications in hydrocarbon reservoir development.

This paper examines the consolidation and shear behavior of Presumpscot Clay over a very wide range of effective stresses, from 0.2 MPa up to 100 MPa (14.5 psi to 14,500 psi). Presumpscot clay was chosen as an analogue test material to

represent low plasticity clay sediments which have an appreciable quantity of silt and a clay mineralogy dominated by illite. In particular, the work investigates the evolution of shear strength properties (normalized undrained strength, friction angle) of Presumpscot Clay as a function of effective stress level and compares this normalized behavior to that of other fine-grained sediments. It is necessary to prepare samples using laboratory resedimentation for this purpose, as it would not be possible to perform the research using intact samples obtained from the field e.g. due to the natural variability inherent in intact samples. However, a limited number of consolidation and shear tests have also been performed on intact samples of Presumpscot Clay for the purpose of comparison with the corresponding resedimented material. The results of these tests on intact samples are used to demonstrate the validity of applying the resedimentation procedure to Presumpscot Clay. The overall goal of the research is to increase understanding of the consolidation and shearing behavior of fine-grained sediments at stresses traditionally encountered in geotechnical

engineering practice but also at much higher stresses. The research aims to make a consistent link between the mechanical behavior of soft soils and unlithified soft rock.

2 TEST MATERIALS AND METHODOLOGY

Presumpscot clay is a glacio-marine clay of low plasticity from coastal Maine and along major inland waterways of the state. The clay is generally blue-grey in color and has a similar geologic origin to Boston Blue Clay. The resedimentation technique was used to produce samples of resedimented Presumpscot Clay (RPC) for testing. RPC is derived from a large number of Shelby tube samples taken from a site in the area of Norridgewock, ME. These samples were first broken up and air-dried, after which they were combined and mixed together to create a uniform composition for resedimentation. The mixed and broken-up core was then ground into a fine powder using a commercial disc-style grinder (Casey 2014). RPC has a low plasticity ($w_L = 33.1\%$) and a clay fraction of 37%, where clay fraction is defined as the percentage of particles by mass with an equivalent diameter $< 2\ \mu\text{m}$. The particle size distribution of RPC as determined by sedimentation, i.e. the hydrometer method (ASTM D422) is presented in Figure 1. For comparison, Figure 1 also includes the particle size distribution of resedimented Boston Blue Clay (RBBC), a glacio-marine medium plasticity clay from the vicinity of Boston, Massachusetts. The mineralogical clay fraction of

RPC was determined from X-ray diffraction to be just 22.8%, the dominant clay minerals being illite and chlorite. A more precise mineralogy of the clay is given in Casey et al. (2015). Intact samples of Presumpscot clay were also tested in this work and were obtained from the same site as the cores used for resedimentation. X-ray photography was used to select the highest quality test specimens, with no visible signs of disturbance, from the Shelby samples.

In addition to the considerable practical benefits associated with resedimentation (e.g. significantly lower cost, no sample disturbance), resedimentation was also a technical necessity for this research. The work involves the isolation of various factors influencing mechanical behavior (e.g. effective stress level, stress history, composition) so that these individual factors can then be systematically quantified. This is not possible with the use of intact samples, since no two intact samples, even of the same sediment, will possess an identical composition and stress history. Resedimentation also eliminates sample variability and produces uniform specimens with one-dimensional consolidation histories and complete saturation. For materials which have not been significantly affected by diagenesis or lithification, a large amount of experimental work has demonstrated that the mechanical behavior of intact samples can be satisfactorily reproduced by the laboratory resedimented material (e.g. Berman 1993, Betts 2014).

The resedimentation procedure has been progressively developed since the 1960's. The basic procedure involves incrementally loading a deaired slurry of the homogenized natural material in a tube called a consolidometer. The sample is consolidated to a desired preconsolidation pressure over a range of weeks or months, the duration depending highly on the consolidation properties of the particular material. Upon reaching the preconsolidation pressure the sample is swelled to an overconsolidated state prior to extrusion from the consolidometer, and then mounted in a test device. A detailed description of the resedimentation procedure is given in Casey (2014).

One-dimensional consolidation properties were measured using constant rate of strain (CRS) consolidation tests. The CRS test data were analyzed using standard linear theory (Wissa et al. 1971, ASTM D4186). Several

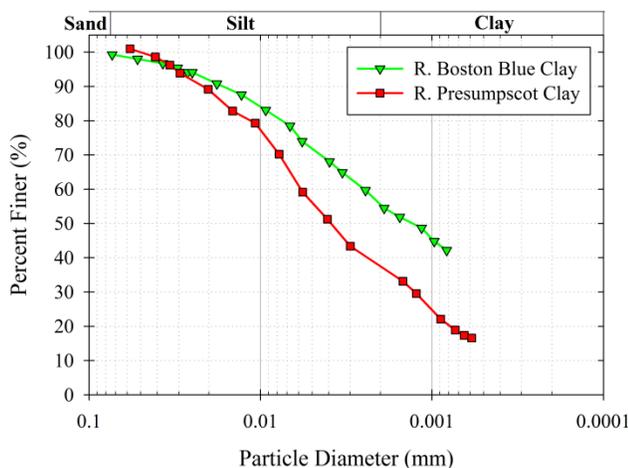


Figure 1. Particle size distribution of RPC and RBBC as determined by sedimentation.

different CRS cells capable of testing specimens from 3.4 to 6.4 cm in diameter were utilized. A Trautwein CRS cell which tests 3.4 cm diameter specimens was used for vertical effective stresses up to 40 MPa. Unlike incremental loading, CRS testing allows one to measure hydraulic conductivity on a continuous basis with very small intervals between measurements. All hydraulic conductivity measurements were made on homogenous specimens in the direction of major principal loading, i.e. it is a bedding perpendicular hydraulic conductivity (k_v).

Triaxial testing was performed using automated triaxial systems which possess an internal load cell for axial load measurement as well as the ability to perform K_0 (i.e. one-dimensional)-consolidation. Essentially, each triaxial system operates optimally over approximately one order of magnitude in stress. A low pressure system was used in tests where specimens were consolidated to a maximum preconsolidation stress (σ'_p) of 2 MPa, a medium pressure system for tests up to 10 MPa, while a high pressure system was developed for tests from 10 MPa to 100 MPa (Casey and Germaine 2014). Each system tests standard sized specimens of 3.6 cm diameter and 8.1 cm height. The K_0 -consolidation algorithm used to control the triaxial system applies a constant axial rate of strain and ensures zero radial strain of the specimen by continuously adjusting cell pressure to keep volumetric and axial strains equal. A continuous measurement of K_0 can therefore be obtained as the specimen consolidates.

During triaxial K_0 -consolidation, specimens were consolidated to at least 1.5 times the stress level applied during resedimentation, as per the standard SHANSEP method of laboratory reconsolidation (Ladd 1991). An axial strain rate of between 0.15 – 0.20 %/hr was chosen to minimize the magnitude of excess pore pressures developed within the specimen. At the end of K_0 normal consolidation, all specimens were allowed at least 24 hrs of [drained] secondary compression prior to undrained shearing. Shearing was typically carried out in compression mode of shear at a standard axial strain rate of 0.5 %/hr. This rate is sufficiently slow to ensure that pore pressures generated during undrained shearing are distributed uniformly throughout the specimen. A limited number of triaxial extension tests were also performed on intact samples of

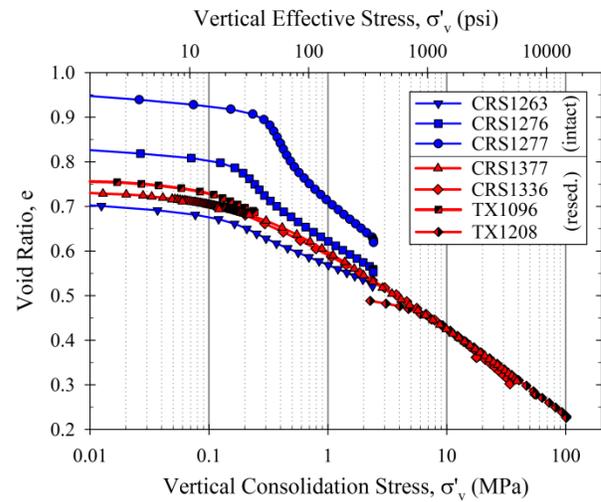


Figure 2. One-dimensional compression behavior of intact and resedimented Presumpscot Clay as determined from CRS tests and the consolidation phase of triaxial tests.

Presumpscot Clay (though none on RPC). Shearing was carried out to axial strains greater than 12 % in each test, by which point a steady critical state friction angle (constant volume friction angle) had been reached.

3 CONSOLIDATION BEHAVIOR

Figure 2 shows the one-dimensional compression behavior of both intact and resedimented samples of Presumpscot Clay in a plot of void ratio (e) versus logarithm of vertical effective stress (σ'_v). The results were derived from CRS consolidation tests as well as from the K_0 -consolidation phase of triaxial tests. A preconsolidation stress can be interpreted for each of the intact specimens, and clear transitions between recompression and normal/virgin compression behavior are observable. Each of the resedimented specimens had a σ'_p of approximately 0.15 MPa (with the exception of TX1208 which had a σ'_p of approximately 5 MPa applied during resedimentation). The virgin compression behavior of RPC shows excellent repeatability over the full range of stresses tested, and is not affected by the type of test device used. On the other hand, intact specimens of the clay display considerable scatter in their compression behavior, this likely being due to variations in composition as a function of depth as well as changes in fabric associated with the environmental conditions in which the clay was

deposited. The scatter may also be influenced by varying amounts of sampling disturbance that the intact samples possessed.

The change in the hydraulic conductivity of Presumpscot clay was also measured during the course of consolidation testing (Fig. 3). The measurements were determined by either one of two well established methods. The first involved the application of Terzaghi’s one-dimensional consolidation theory (Terzaghi 1943) to load increments during resedimentation. In this approach permeability is determined at discrete intervals corresponding to each individual load increment. Hydraulic conductivity can be measured at very high porosities approaching 0.6 ($e = 1.5$). The second and primary method of measuring permeability was through the use of CRS consolidation tests performed on both intact and resedimented samples.

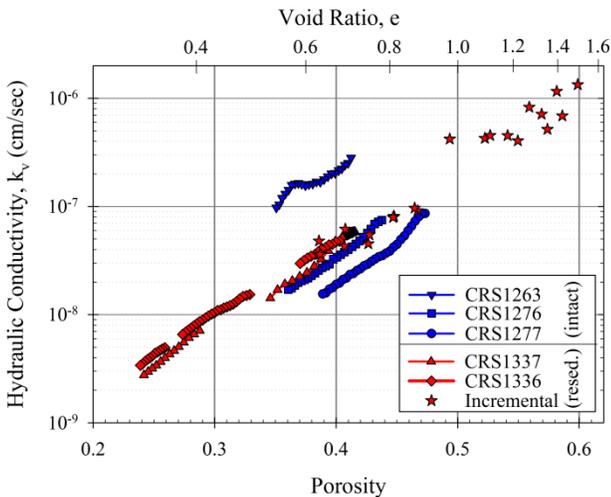


Figure 3. Change in the [vertical] hydraulic conductivity of intact and resedimented Presumpscot Clay as a function of porosity. A log-linear relationship is observed between hydraulic conductivity and porosity.

Similar to one-dimensional compression behavior, the k_v of RPC shows good consistency between tests as well as between the two types of measurement procedure used. The hydraulic conductivity of RPC reduces in a log-linear fashion as a function of porosity, decreasing from about 1×10^{-6} cm/s at very high porosities of 0.6 (corresponding to the early stages of the resedimentation process) to values as low as 3×10^{-9} cm/s at a porosity of 0.25 (corresponding to $\sigma'_v > 10$ MPa). The hydraulic conductivity of the intact specimens also displays an essentially

log-linear dependence on porosity. However, there is much greater variation between the intact tests, with k_v varying by over an order of magnitude at a given porosity. These variations in k_v for the intact specimens are consistent with the compression behavior presented in Figure 2 and can be attributed to variations in the composition of different intact samples. For example, an increasing proportion of silt in the clay would be expected to produce both a higher permeability at a given porosity as well as a lower void ratio at low effective stresses.

The variations in the coefficient of lateral earth pressure at rest (K_0) observed during typical triaxial tests performed on resedimented and intact samples of Presumpscot Clay are presented in Figure 4. The change in K_0 during consolidation follows the same trend for each specimen, and is consistent with the behavior reported by previous researchers (e.g. Mesri & Hayat 1993). During the initial pressure-up and back-pressure saturation phases of a test, the overconsolidated specimen is subjected to almost isotropic stress conditions. The value of K_0 is therefore close to unity at the beginning of consolidation. It decreases rapidly during recompression to σ'_p , before reaching a stable value during normal consolidation (K_{0NC}). Tests performed on intact Presumpscot Clay often display a significant drop in K_0 just prior to reaching σ'_p , an example of this behavior being

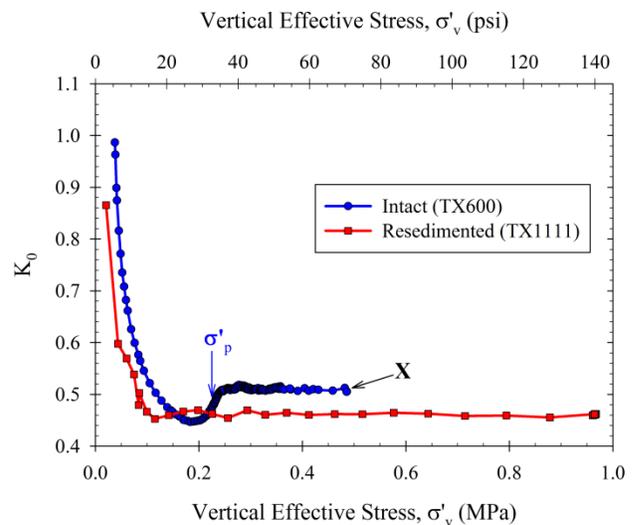


Figure 4. The K_0 of intact and resedimented Presumpscot Clay as measured during virgin consolidation. The value of K_{0NC} remains fairly constant for effective stresses up to 10 MPa.

shown in Figure 4. Such behavior is generally not observed for resedimented samples, the precise reason for this being unknown.

To illustrate the variation in K_{0NC} from multiple triaxial tests over three orders of magnitude of effective stress more clearly, stress level is plotted on a logarithmic scale in Figure 5. Each data point in Figure 5 represents the value of K_{0NC} measured at the end of the virgin consolidation portion of a test (e.g. point X in Figure 4). The value of K_{0NC} for RPC remains remarkably constant for stresses up to 10 MPa, remaining within the range 0.46-0.51, with an average value of 0.48. The measured values of K_{0NC} for intact Presumpscot Clay are very similar, producing an average value of 0.51 (standard deviation of 0.02).

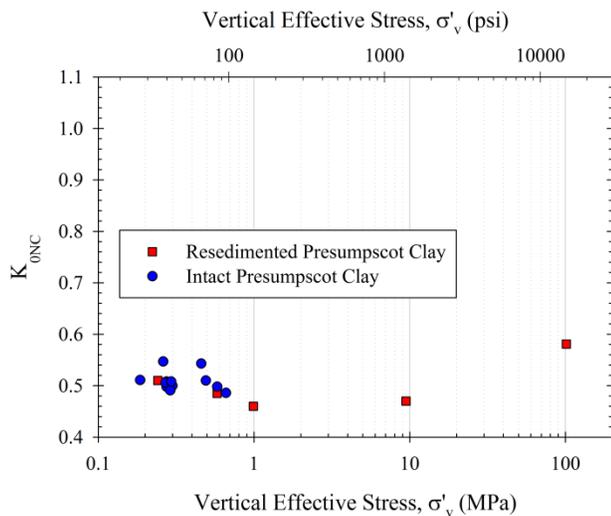


Figure 5. The variation in K_{0NC} of intact and resedimented Presumpscot Clay as a function of stress level. Each data point represents the value of K_{0NC} measured at the end of the virgin consolidation portion of a test (e.g. point X in Figure 4).

Due to experimental challenges, an accurate measurement of K_0 can be more difficult during unload-reload cycles of laboratory consolidation tests. Figure 6 shows the variation in K_0 as a function of overconsolidation ratio (OCR) for one test on RPC. As expected, there is significant hysteresis in the value of K_0 between swelling (unloading) and recompression (reloading). The value of K_0 during the swelling phase (K_{0S}) can be approximated by a function originally proposed by Schmidt (1966):

$$K_{0S} = K_{0NC}(\text{OCR})^a \quad (1)$$

where a is a constant and equal to 0.39 for RPC. This form of equation has since been widely accepted to describe the variation in K_0 during swelling for both clays and sands (e.g. Mayne & Kulhawy 1982, Mesri & Hayat 1993). During swelling, an isotropic stress state is achieved at an OCR of approximately six for RPC. Figure 6 also includes the swelling portion of a test performed on an intact specimen. The change in K_0 during swelling of this specimen does not follow the trend predicted by Equation 1 and is significantly lower than the value of K_0 for RPC at the same OCR. Lacking confirmation by further testing, the swelling portion of this particular test performed on intact Presumpscot clay could be considered unrepresentative.

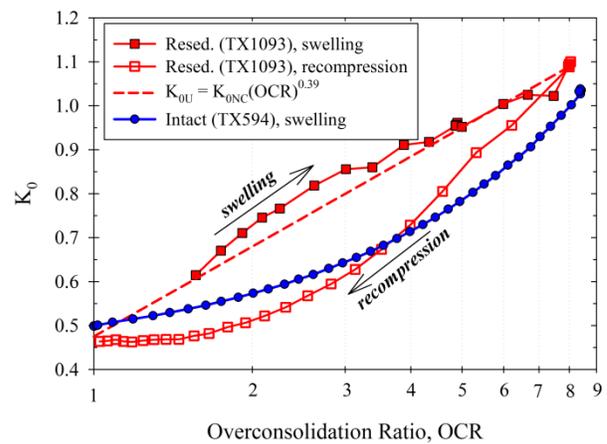


Figure 6. The variation in K_0 of intact and resedimented Presumpscot Clay as a function of OCR. Significant hysteresis is observed in the value of K_0 between unloading and reloading.

4 SHEAR BEHAVIOR

4.1 Undrained Strength

Figure 7 shows the normalized shear stress-strain responses measured during undrained shearing for 5 tests performed on RPC at consolidation stresses ranging from 0.24 MPa to 101 MPa. All of these tests correspond to $\text{OCR} = 1$ and triaxial compression (TC) mode of shear. The undrained strength ratio (i.e. the peak shear strength measured during undrained shearing, s_u , normalized with respect to the pre-shear vertical consolidation stress, σ'_{vc}) of RPC varies

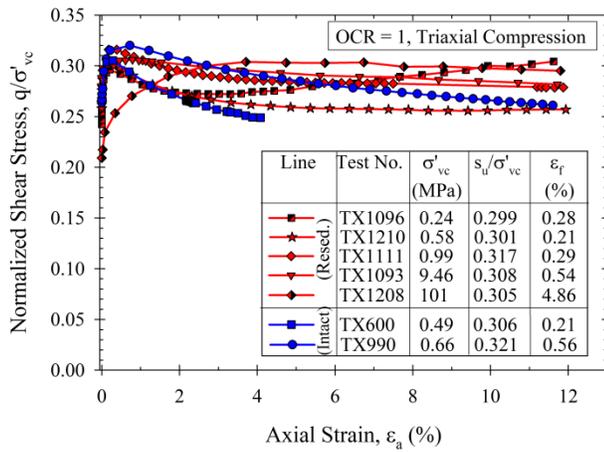


Figure 7. Stress-strain responses measured during undrained shearing of intact and resedimented Presumpscot Clay. Increasing consolidation stress leads to a more ductile stress-strain response, but has no effect on normalized undrained strength.

remarkably little, between 0.299 and 0.317, over the entire stress range. Small differences in s_u/σ'_{vc} values between triaxial tests are likely the result of experimental non-repeatability. The form of the stress-strain curve displays significant stress level dependence, however, as the behavior of RPC becomes much more ductile with increasing consolidation stress. There is a consistent and dramatic increase in axial strain to failure (ϵ_f) with increasing stress level, with ϵ_f increasing from just 0.28 % at 0.24 MPa to almost 5 % at 101 MPa. The amount of post-peak strain softening also generally reduces with increasing stress, with almost no strain softening observed at 101 MPa. However, at the lowest stress level of 0.24 MPa the soil exhibits significant strain hardening following an initial peak stress, and the highest shear stress mobilized in the test actually occurs at large strains (although the quoted values of $s_u/\sigma'_{vc} = 0.299$ and $\epsilon_f = 0.28$ % correspond to the initial peak in the stress-strain curve).

For comparison, Figure 7 also includes the results for two representative tests performed on intact samples of Presumpscot Clay at OCR = 1. The undrained strengths and ϵ_f of the intact samples are very similar to those of the resedimented material, although the intact samples do display slightly increased post-peak strain softening. This comparison demonstrates that the shearing behavior of intact Presumpscot Clay can be satisfactorily reproduced using laboratory resedimentation.

The variation in undrained strength ratio is demonstrated more clearly in Figure 8, which plots values of s_u/σ'_{vc} measured in each of the triaxial tests. Stress level is defined in terms of σ'_p (which is equivalent to σ'_{vc} at OCR = 1). For comparison, the figure also includes the results of triaxial extension (TE) tests performed on intact samples. The strength of the clay is substantially lower in extension mode of shear, being approximately one third of the compressive strength at OCR = 1. The experimental compression data for RPC shown in Figure 8 is fitted using a power law function of the form:

$$s_u/\sigma'_{vc} = S_1(1000\sigma'_p [\text{MPa}])^T \quad (2)$$

where S_1 and T are fitting parameters, equal to 0.30 and 0.001 for RPC respectively. The parameter S_1 is an apparent intercept value and corresponds to the undrained strength ratio at $\sigma'_{vc} = 1$ kPa (the parameter is therefore an extrapolated value and may not have real physical meaning). The T parameter describes the variation in the undrained strength of a soil as a function of effective stress level, so that a lower value of T indicates a faster reduction in undrained strength ratio with increasing stress. The fact that the value of T is essentially zero for RPC indicates that the soil displays ideal normalized behavior, at least for TC mode of shear.

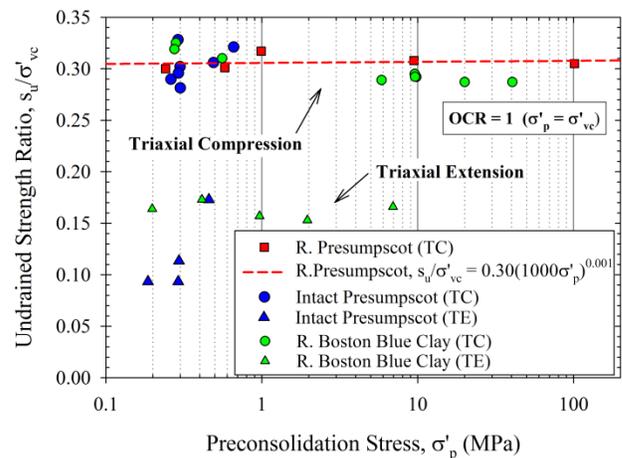


Figure 8. Undrained strength ratios of RPC, RBBC and intact Presumpscot clay at OCR = 1 as a function of preconsolidation stress level. Results for both compression and extension modes of shear are presented.

For comparison, Figure 8 also includes data for RBBC. The undrained strength of RBBC is very similar to that of RPC for both TC and TE modes of shear. In TC mode, however, the strength ratio of RBBC does display a slight decrease with increasing consolidation stress, a behavior which is demonstrated more conclusively in Casey and Germaine (2013).

A procedure similar to that described above has also been performed on the measured undrained strength ratios for several other fine-grained soils (Casey & Germaine 2013). In contrast to RPC, these other soils displayed consistent variations in s_u/σ'_{vc} with stress level. The direction and rate to which the normalized strength of a soil varies as a function of effective stress level is closely related to its liquid limit, where liquid limit is a reflection of soil composition. Figures 9 and 10 show the correlations between the parameters S_1 and T with liquid limit respectively. The r^2 values of 0.98 and 0.93 indicate that the correlations are of high quality. Given that a lower value of T implies a faster reduction in strength ratio with increasing stress level, the relationship presented in Figure 10 indicates that high plasticity soils display a more rapid reduction in normalized undrained strength with increasing stress. Equivalently, high plasticity soils show a greater sensitivity of normalized undrained strength to effective stress level. For example, a value of $T = -0.15$ as measured for resedimented Eugene Island Clay from the US Gulf of Mexico Clay

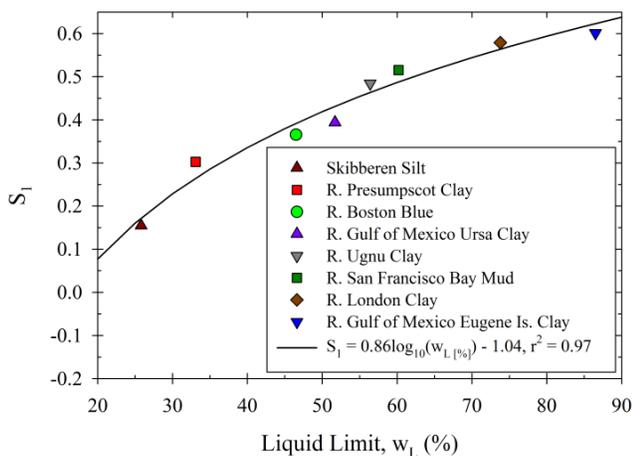


Figure 9. Correlation between the strength parameter S_1 and liquid limit.

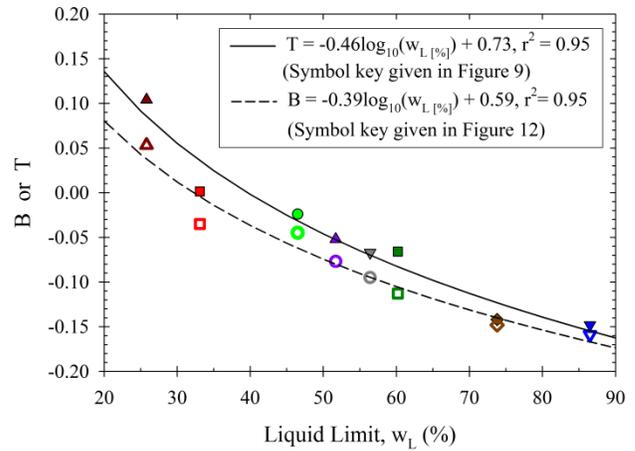


Figure 10. Correlations between liquid limit and the strength parameters B and T.

corresponds to a reduction in undrained strength ratio (at $OCR = 1$) from 0.30 at $\sigma'_{vc} = 0.1$ MPa to 0.22 at $\sigma'_{vc} = 1$ MPa. Of all the soils which have tested, Presumpscot Clay is the only soil which displays almost perfect normalized behavior over a very wide range of effective stresses.

With regard to the effect of overconsolidation, triaxial tests have not been performed on overconsolidated RPC in this work. However, triaxial tests have been performed on some of the other soils presented in Figures 9 and 10 in an overconsolidated state. The results of these tests indicate that the parameter T is independent of OCR, which implies that undrained strength ratio decreases at a similar rate at each OCR (Casey & Germaine 2013). Furthermore, the value of the parameter S_1 can be accurately described using a SHANSEP type equation (Ladd and Foott 1974) with a measured or assumed value of OCR corresponding to the depth of interest.

4.2 Friction Angle

In contrast to what was observed for undrained strength ratio, the [secant] critical state friction angle (ϕ'_{cs}) of RPC displays a consistent, though relatively gradual, decrease from 36.7° at 0.24 MPa to 28.7° at 101 MPa (Figure 11). For comparison, the critical state friction angles measured on intact samples for both compression and extension modes of shear are also presented. With regard to compression, the ϕ'_{cs} values of the intact samples display more scatter than those of the resedimented samples, although the average values are very similar. The increased scatter for

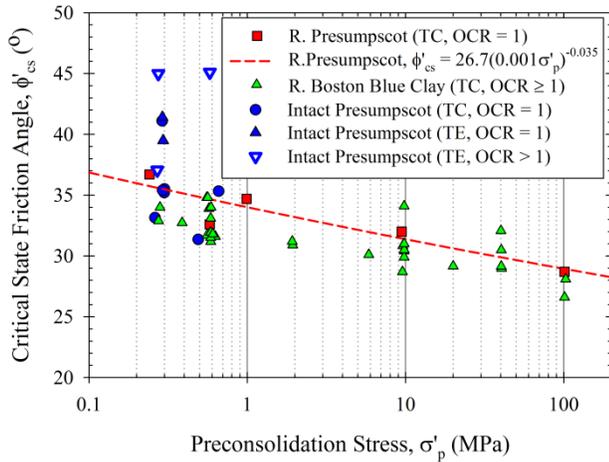


Figure 11. Critical State friction angles of RPC, RBBC and intact Presumpscot Clay as a function of preconsolidation stress level for both compression and extension modes of shear.

the intact tests is expected and can be explained by variations in composition between the samples, a variation which does not exist in the resedimented samples. The higher critical state friction angle measured for intact Presumpscot Clay in extension is consistent with what has been observed for other similar soils (e.g. RBBC) and may be expected to approach a value similar to that for compression mode of shear at higher stresses (Casey & Germaine 2013). Two outlier values in excess of 60° measured in extension tests (not shown in Figure 11) are likely to be unreliable and may have been caused by experimental difficulties sometimes encountered in TE tests. It can also be seen from Figure 11 that the friction angle of RBBC is very similar to that of RPC for both normally and overconsolidated soil, and shows a similar consistent decrease with increasing stress.

The variation in ϕ'_{cs} of RPC as a function of preconsolidation stress can be described by a power-law function of the form:

$$\phi'_{cs} = A(0.001\sigma'_p [\text{MPa}])^B \quad (3)$$

where A and B are fitting parameters equal to 26.7 and -0.035 for RPC respectively. The A parameter is an apparent intercept value produced by extrapolating the regression to $\sigma'_v = 1$ GPa (the parameter A is therefore an extrapolated value and may not have real physical meaning). The parameter B describes the change in friction angle with stress, so that a lower value of B (i.e. more negative) indicates a faster reduction in

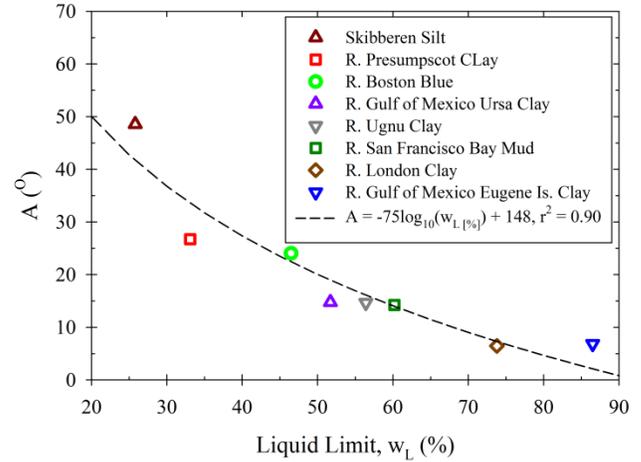


Figure 12. Correlation between the strength parameter A and liquid limit.

friction angle with increasing stress. Consistent variations in friction angle have also been observed for other soils that have been tested. Figures 10 and 12 present the correlations between the parameters B and A with liquid limit respectively. Similar to what was observed for undrained strength, Figure 12 indicates that high plasticity soils display a more rapid reduction in friction angle with increasing effective stress. For example, a value of $B = -0.16$ as measured for resedimented Eugene Island Clay corresponds to a reduction in ϕ'_{cs} from 30° at $\sigma'_p = 0.1$ MPa to 21° at $\sigma'_p = 1$ MPa.

With regard to the effect of overconsolidation, Equation 3 has been found to describe behavior at all OCRs when stress level is defined in terms of σ'_p . Furthermore, the results of tests performed on other resedimented soils at OCRs ranging from 1 to 8 indicate that the values of A and B are independent of OCR (Casey & Germaine 2013).

5 CONCLUSIONS

K_0 -consolidated triaxial tests CRS consolidation tests were performed on Resedimented Presumpscot Clay to determine the variation in compression behavior, hydraulic conductivity, K_0 , undrained strength, and friction angle over the effective stress range of 0.2 MPa to 100 MPa. It was necessary to use the resedimentation technique to prepare samples for testing over such a wide range of stresses. In terms of virgin compression behavior, the void ratio – effective stress relationship of RPC remains essentially

log-linear up to at least 40 MPa, and exhibits highly consistent behavior between tests. As expected, the compression behavior of the intact clay displays more scatter between tests, and the virgin compression line of the resedimented clay falls within the range of this scatter. Similar behavior is observed for hydraulic conductivity, where variations in the composition of intact specimens cause k_v to vary by over an order of magnitude at a given porosity. The hydraulic conductivity of RPC displays a log-linear dependence on porosity over a very wide porosity range of 0.25 – 0.60. The value of K_0 remains within the range of 0.46 – 0.51 during normal compression for stresses up to 10 MPa. During swelling K_0 increases substantially with OCR, reaching a value of one at OCR = 6.

The undrained strength ratio of RPC changes remarkably little over the full range of stresses investigated, remaining in the range of 0.30 to 0.32 at OCR = 1. This ideal normalized behavior appears to be unique among fine-grained soils, with other soils displaying consistent variations in strength ratio as a function of stress level and composition. For example, higher plasticity soils have been found to display a more rapid decrease in normalized undrained strength with increasing stress level. With regard to friction angle, the critical state friction angle of RPC displays a gradual decrease with increasing stress, remaining in the range of 33° - 37° for σ'_p below 1 MPa. The variation in ϕ'_{cs} of RPC within this stress range can likely be ignored for most practical applications. Once again, however, higher plasticity soils have been found to display greater variations in ϕ'_{cs} as a function of stress level when compared to RPC, and these variations are such that they may need to be accounted for in conventional geotechnical engineering design. The results of triaxial shear tests performed on intact samples of Presumpscot Clay compare very favorably with the results for the corresponding resedimented material. This demonstrates that the strength properties of intact Presumpscot clay can be closely replicated using laboratory resedimentation, thereby reducing the need to obtain expensive high quality field samples for laboratory shear testing.

Acknowledgements

This work was funded by the UT GeoFluids Consortium, comprising Anadarko Petroleum, BP, BHP Billiton, Chevron, ConocoPhillips, ExxonMobil, Hess, Murphy Oil Corporation, Repsol, Schlumberger, Shell, Statoil and Total. The authors are very grateful to Waste Management for the permission to publish information on intact core samples of Presumpscot Clay.

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