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Estimating Undrained Strength of Clays from Direct Shear Testing at Fast Displacement Rates

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ABSTRACT: When the direct shear test is performed in accordance with ASTM guidelines, the measured shear stresses at failure estimate drained strength parameters. We investigate the possibility of estimating undrained strength using direct shear testing at variable shear displacement rates on specimens composed of various combinations of kaolinite and bentonite. Even at fast displacement rates, constant volume conditions are not achieved in the direct shear device because of changes in specimen height that are large relative to allowable ASTM thresholds for constant volume simple shear testing. However, undrained strengths established by constant volume simple shear testing at slow strain rates are well approximated by direct shear tests conducted at fast shear displacement rates (time to failure < t_{50}/8, where t_{50}=time to 50% consolidation in a conventional oedometer test). Because of the simplicity of direct shear testing, such estimates of undrained strength may be useful in engineering practice when access to a simple shear device is limited. Nevertheless, fast direct shear tests have shortcomings, including lack of control of rate effects, and constant volume testing is recommended for critical projects.

INTRODUCTION

The direct shear test is one of the most commonly used soil shear strength tests due to its operational simplicity. Testing performed in accordance with ASTM standard D 3080 (American Society for Testing and Materials, 2007) is performed at speeds slow enough to allow the dissipation of most excess water pressures. As a result, direct shear tests per standard ASTM D 3080 can be used to measure only drained strength parameters. Because neither change in water content nor axial deformation of the specimen is controlled in a conventional direct shear test, truly undrained conditions cannot be achieved.

A number of textbooks report that undrained strengths can be approximated for saturated specimens using the direct shear test performed at rates on the order of 1.3 mm/min (0.05 in/min) (e.g., Bowles, 1992; Carter, 1983; Lambe, 1951; Liu and Evett, 1997). These investigators recognize the inability to control drainage during the direct shear test, but postulate that if direct shear tests are run on soils of low hydraulic conductivity at sufficiently fast shear displacement rates, saturated specimens can be sheared to failure without significant volume change. If true, the results of such a test would approximate the undrained soil behavior. It is noteworthy that none of the aforementioned references cite a laboratory study with direct shear testing to support their recommendations.

Drained strength parameters are obtained from the direct shear test by shearing the soil at sufficiently slow displacement rates that negligible pore pressure generation occurs inside the specimen. ASTM testing procedures specify a time to failure, $t_f > 50 \cdot t_{50}$. where t_{50} is the time to 50% of ultimate consolidation in a conventional oedometer test. This recommendation is based on a laboratory testing program by Gibson and Henkel (1954).

While the standard direct shear apparatus is intended to measure drained shear strength parameters, previous researchers have sought to modify the direct shear apparatus to simulate undrained conditions by maintaining a constant specimen volume during shearing. This is generally accomplished by modifying the normal load during shearing either through an active control system (which adjusts the normal load on the specimen during shear in response to a measurement of specimen height – e.g., Taylor, 1952; O'Neil, 1962) or passive control system (which mechanically restrains the specimen height and measures changes in normal load with a load cell – Takada, 1993). Direct shear devices with these modifications have been shown to provide reasonable estimates of undrained strength from constant volume simple shear tests (e.g., Hanzawa et al., 2007). However, the emphasis of the present paper is on the relatively pragmatic question of whether an unmodified, conventional direct shear device can be operated to provide reasonable estimates of undrained strength. Specifically, our objective is to evaluate whether direct shear testing performed at a sufficiently fast displacement rate can provide strength estimates that approximate shear strengths from constant volume direct simple shear tests (ASTM D 6528). The practical need for work of this type has been identified in engineering guidelines documents for practicing engineers (e.g., Blake et al., 2002).

We recognize that other simplified tests for estimating undrained shear strength exist (torvane, fall cone, laboratory miniature vane [ASTM D 4648]), however each has separate limitations. These tests either do not directly measure shear strength (fall cone), or test a limited zone of soil (torvane and laboratory miniature vane). Our objective is not to promote fast direct shear tests as superior to these other techniques, but merely to investigate its potential feasibility, which is of significant practical interest because of the widespread use of direct shear testing in engineering practice.

SOIL SPECIMEN PREPARATION

Soil Materials

We utilized pure clay specimens prepared in the laboratory by means of slurry consolidation. Source clay minerals included kaolinite and sodium bentonite. The kaolinite was air-floated EPK Kaolin, manufactured by The Feldspar Corporation.

According to the manufacturer's product data sheet, the kaolinite mineral comprises 97% of the material by weight. The bentonite used was Big Horn FND 200, manufactured by Wyo-Ben, Inc. Two clay minerologies were considered in the present study, the first being 100% kaolinite and the second being 80% kaolinite and 20% bentonite.

Index testing was performed to evaluate Atterberg Limits (ASTM D 4318) for the two clays. The results of the Atterberg Limits tests as well as other index properties for the two materials are summarized in Table 1.

Table 1. Composition and average index properties of clays used in this study

Composition	USCS Classification	Atterberg Limits (%)			Coefficient of Consolidation, c_v (m^2/sec)*
		LL	PL	PI	
100% Kaolinite	CH/MH	53	33	20	1.1×10^{-7}
80% Kaolinite / 20% Bentonite	CH	129	30	99	4.3×10^{-9}

*Measured at Vertical Consolidation Stress (σ'_c) = 77 kPa, OCR=1

Sample Preparation in Consolidation Tank

Clay specimens were prepared by means of one-dimensional slurry consolidation. The slurry consisted of a mixture of dry clay powder (kaolinite or a combination of kaolinite and bentonite) and water. The slurry was prepared at a water content corresponding to approximately twice the liquid limit. To produce uniform slurry and to mimic the natural sedimentation process, oven-dried clay powder was slowly air pluviated into a large basin containing a predetermined volume of water. The slurry was periodically mixed during sedimentation to eliminate minor clods that formed on the surface. Once the entire amount of dry soil was sieved into the basin, the material was thoroughly blended with an electric mixer to produce a uniform slurry. Special care was taken to limit air entrapment during mixing. The slurry was then transferred to a consolidation tank with an inside height of 47 cm and inside diameter of 26 cm. Additional details on the consolidation tank are provided by Bro (2007). The slurry was then slowly stirred again to release air bubbles that became entrapped during the transfer process. The distance from the top of the consolidation tank to the surface of the slurry was measured to determine the initial height of the soil column.

The slurry/soil in the tank was consolidated through vertical pressure applied from the top by a piston assembly sealed against the inside walls of the tank. Water drained from the tank through the base. Friction between the piston assembly and inside tank walls was quantified so that the pressures applied to the soil could be evaluated from the load applied to the piston assembly. To minimize friction, a thin layer of petroleum jelly was applied to the inside walls of the consolidation tank prior to adding the slurry. A net consolidation pressure of 38.3 kPa was applied to batches of pure kaolinite. A slightly higher pressure of 47.9 kPa was used for the kaolinite-bentonite mixture to account for the softer consistency of the material and create samples that could be more readily trimmed for shear and consolidation testing.

Vertical compression of the soil column was measured during consolidation and loading was continued until approximately 90% of the theoretical ultimate consolidation was completed. Near the end of the consolidation process, vertical strains were on the order of 45 to 50%.

Sampling of Test Specimens

Upon completion of consolidation, the piston assembly was removed and replaced with a sampling cap. Seven thin-walled sampling tubes were mounted onto the sampling cap. The sharpened brass tubes were 152 mm long and had an inside diameter of 61.4 mm, an outside diameter of 63.5 mm, and a 35° cutting edge. The area ratio, A_r of the sampling tubes was 7%, which is significantly less than the 20% maximum recommended by Terzaghi et al. (1996) for relatively undisturbed soil sampling. The minimum clearance between the outside edge of the perimeter tubes and the tank wall was 25 mm, which was intended to minimize potential disturbance caused by soil shearing near the edge of the tank. Bro (2007) discussed in greater detail the mechanics of the sampling process.

One-dimensional consolidation tests (ASTM D 2435) were performed and preconsolidation pressures (σ'_p) were estimated using the Casagrande construction technique. For the pure kaolinite batches, values of σ'_p were generally 75-85% of the net pressure applied to the soil in the consolidation tank. For the kaolinite-bentonite batches, values of σ'_p were 42-70% of the tank pressure. Measured values of σ'_p that are less than the tank pressure are expected because the soil was sampled in an under-consolidated state. Prior to shearing, most of the test specimens were consolidated to the virgin compression line to ensure a normally consolidated condition.

RESULTS OF SHEAR STRENGTH TESTING

Two test sequences were performed on specimens of the two clay materials. The first consisted of monotonic direct simple shear tests that were intended to provide a baseline set of undrained shear strength ratios (i.e., ratio of undrained strength to pre-shear vertical effective stress, as per Ladd and Foott, 1974). Our use of the term "undrained strength" in the remainder of the article implies constant volume simple shear testing. The second set consisted of direct shear tests performed at various displacement rates intended to produce varying degrees of partial drainage. In the following sections, we present the baseline simple shear results, which are followed by the direct shear results, and discussion of the principal findings.

Undrained Shear Strength from Constant Volume Simple Shear Testing

Monotonic direct simple shear (DSS) tests were performed in accordance with ASTM D 6528, in which undrained conditions are simulated by maintaining constant volume of the test specimen. The shear strain rates in these tests was 11%/hr for the pure kaolinite specimens and 1%/hr for the kaolinite-bentonite mixtures. The shear strain rate was sufficiently slow such that pore pressure buildup is not expected. Changes in vertical stress during shear were measured using a load cell mounted between the top of the specimen and the fixed loading frame. The measured change in vertical stress is assumed to correspond to the pore pressure that would have been generated in a conventional undrained test where the total stress is kept constant (Dyvik, et al., 1987). Further description of the test apparatus is provided in Bro (2007).

As shown in Table 2, ten direct simple shear tests were performed, five for each material. All specimens were consolidated past the preconsolidation pressure to vertical consolidation stresses of approximately σ'_c=50, 100, and 185 kPa. As discussed subsequently, the specimens consolidated to the lowest confining pressures did not reach the virgin compression line and are slightly overconsolidated, whereas the specimens at higher consolidation stresses are normally consolidated. Samples were sheared monotonically to ultimate strains typically in the range of 11-15%. Peak stresses typically occurred at shear strains between 6 and 9%.

Table 2. Results of direct simple shear testing on kaolinite and kaolinite-bentonite mixtures

Test No.	Consol-idation Stress, σ'_c (kPa)	Shear Strain at Failure (%)	Peak Shear Stress, τ_{peak} (kPa)	Largest Shear Strain Tested (%)	Shear Stress at Largest Strain (kPa)	Strength Ratio (τ_{peak}/σ'_c)	Decrease in Normal Stress at Failure, $\Delta\sigma_n$ (kPa)	$\Delta\sigma_n/\sigma'_c$	Corrected Undrained Strength Ratio
DSSK100 3a	48.2*	8.7	14.8	20	12.0	0.31	23.7	0.49	0.31
DSSK100 3b	95.6	6.9	21.7	11	20.6	0.23	47.4	0.50	0.23
DSSK100 3c	177.8	6.7	42.3	11	39.9	0.24	73.8	0.42	0.22
DSSK100 3d	51.6*	6.1	17.1	15	17.1	0.33	16.8	0.32	0.31
DSSK100 3e	184.5	8.6	47.2	15	44.3	0.26	78.8	0.43	0.24
DSSK8B2 5c	51.1*	8.4	15.3	15	14.7	0.30	10.3	0.20	0.28
DSSK8B2 5d	104.5	6.7	18.9	11	17.7	0.18	35.8	0.34	0.17
DSSK8B2 5e	106.6	9.0	19.4	15	18.3	0.18	37.5	0.35	0.17
DSSK8B2 5f	185.0	9.4	37.5	15	35.2	0.20	61.1	0.33	0.19
DSSK8B2 5g	184.9	10.5	37.2	15	34.9	0.20	58.0	0.31	0.19

*OCR=1-1.5. For all other specimens, OCR=1

In Table 2, results for pure kaolinite samples are indicated as DSSK100 whereas those for the kaolinite/bentonite mix are indicated as DSSK8B2. Test results are summarized in Figures 1 and 2 for the two materials in terms of stress-strain relations, volume change-strain relations, change in vertical stress-strain relations, and stress paths. Note that the applied shear stress (τ) and change in normal stress ($\Delta\sigma$) are normalized by the pre-shear vertical consolidation stress (σ'_c). For consistency with ASTM reporting procedures, stress paths were not normalized.

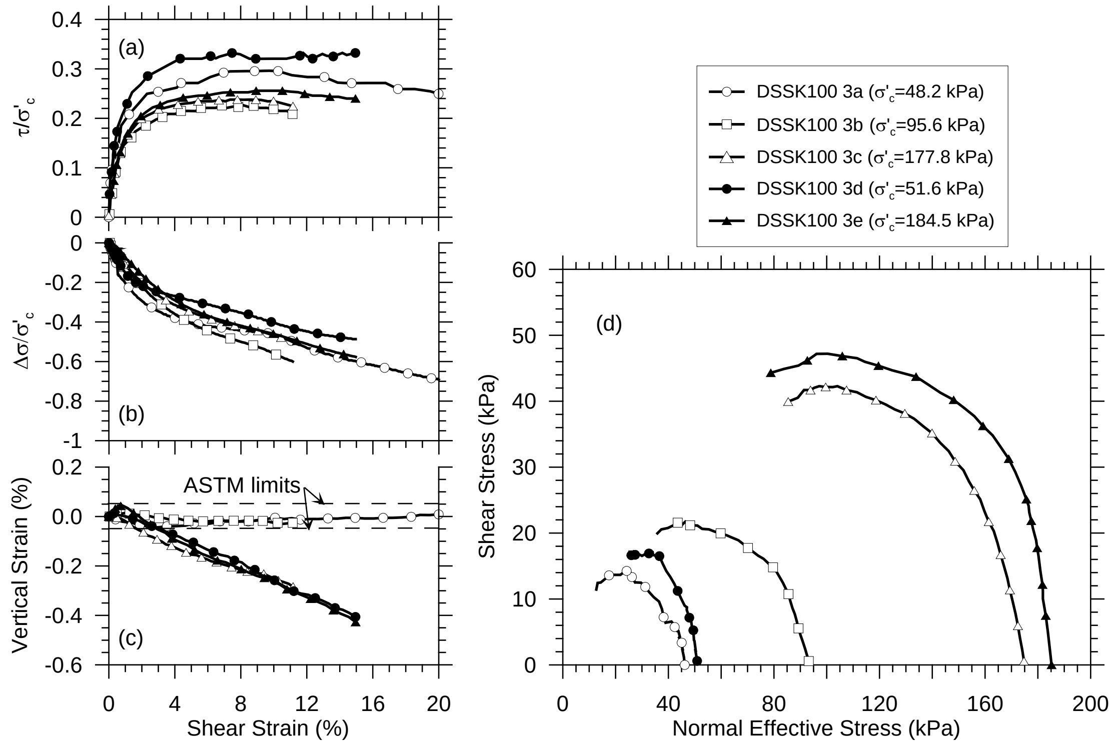

FIG. 1. Direct simple shear test results for kaolinite specimens

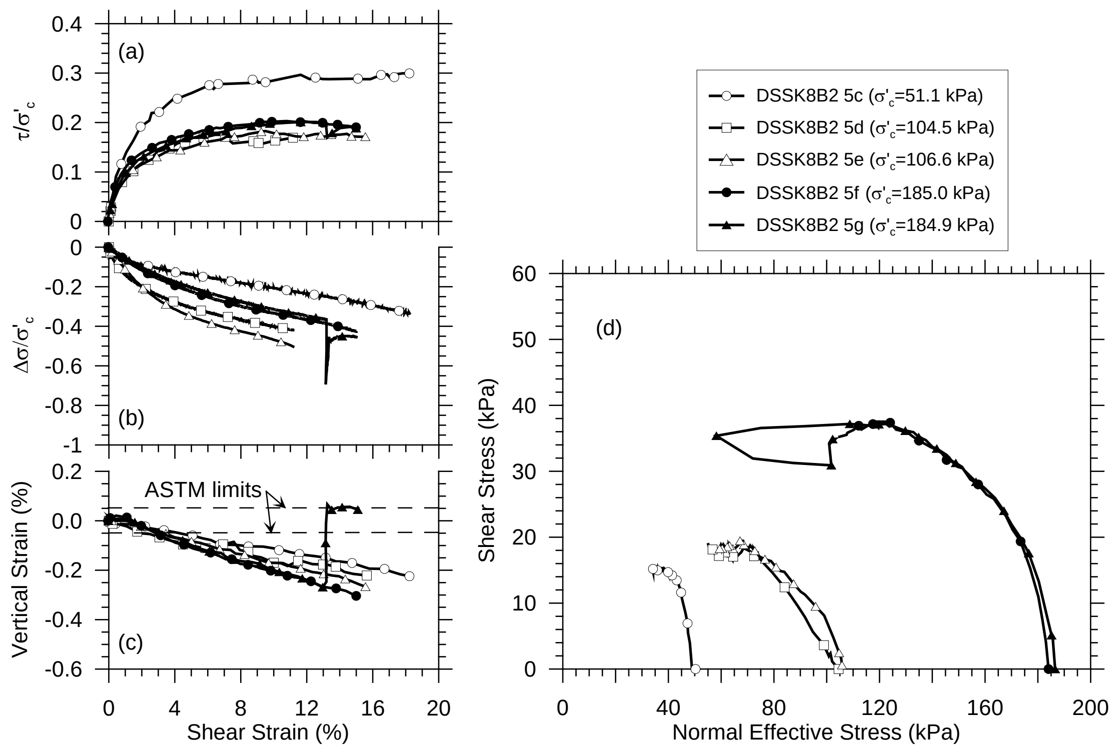

FIG. 2. Direct simple shear test results for kaolinite-bentonite specimens

As shown in Figures 1-2, the specimens consolidated to $\sigma'_c \approx 50$ kPa typically exhibit higher normalized shear strengths, lower change in normal stress, and reduced vertical strain relative to specimens consolidated to higher stresses. This affects specimens 3a and 3d (kaolinite) as well as 5c (kaolinite-bentonite). This is attributed to a small degree of overconsolidation. We believe those specimens were overconsolidated because σ'_c was only 110 to 150% of the consolidation pressure applied to the sample block in the consolidation tank. Accordingly, those specimens were likely not re-consolidated to the virgin compression line (according to Ladd (1991) specimens should be consolidated to 1.5-2.0 times their preconsolidation pressure to ensure that their state is on the virgin compression line).

Specimens with $\sigma'_c \approx 100$ or 185 kPa are normally consolidated, and exhibit excellent normalization of shear stress-strain and change in normal stress-strain relationships. For the kaolinite specimens, strength ratios (defined as ratio of peak shear stress to pre-shear vertical effective stress, τ_{peak}/σ'_c) for these normally consolidated specimens range from 0.23 to 0.26. The corresponding range for the normally consolidated kaolinite-bentonite mixture is 0.18 to 0.20.

ASTM standards for constant volume simple shear testing (ASTM D 6528) state that for passive height control systems such as the one used in the current study, the change in the specimen height after accounting for apparatus compressibility should be less than 0.05% during shear. Those strain thresholds are marked with dotted lines in Figures 1-2(c). Only the first two direct simple shear tests on pure kaolinite (3a and 3b) show total vertical strains less than 0.05% throughout shear. For reasons that are attributed to minor variations in the test apparatus and setup procedure, all subsequent tests show amounts of vertical strain which exceed that threshold. The vertical strain at failure ranged from 0.02 to 0.23% for pure kaolinite and 0.10 to 0.21% for the kaolinite-bentonite mixture.

An attempt was made to quantify the effect of this small vertical strain on shear strength near the end of the last test (DSSK8B2 5g). After the peak shear strength had been measured (37.2 kPa), the vertical strain was adjusted back to within ASTM limits (+/- 0.05%) by reducing the normal effective stress acting on the specimen at that point from 118.7 to 58.2 kPa. This was done without interrupting shear displacement. The result was a reduction of shear resistance to a relatively constant value of 34.9 kPa (a 6% reduction) for the remainder of the test. Additional insight into the effects of small vertical strain on shear strength can be made by comparing the results for 3a (vertical strains < 0.05% throughout the test) and 3d (vertical strain at failure of 0.15%). Using the results in Table 2, we see the sample with the larger vertical strain has about 8% larger strength.

Based on the above comparisons, we estimate that the vertical strains on the order of 0.1 to 0.25% measured in eight of the ten direct simple shear tests caused approximately 7% increase in shear strength. This strength increase is caused by the small amounts of contraction of the specimen during shear. To correct for this contraction, the undrained shear strength ratio measured in tests with excessive vertical strain was reduced by 7%. When this correction is applied to the measured strengths, the “corrected undrained strength ratios” shown in Table 2 are obtained.

In summary, the baseline strength ratios for subsequent use in this study are taken as average values of corrected undrained strength ratio in Table 2, which are approximately 0.23 for normally consolidated 100% kaolinite and 0.18 for normally consolidated 80-20 kaolinite-bentonite mixture. Those strength ratios were developed for strain rates that are generally compatible with the recommendation by Ladd (1991) of approximately 5%/hr strain rate for direct simple shear tests.

Shear Strength from Direct Shear Testing at Various Displacement Rates

Direct shear tests were performed to investigate the effect of shear displacement rate on the measured shear strength and to evaluate the possibility of estimating undrained strengths through rapid shearing. Various shear rates were selected for the two materials, corresponding to various multipliers on mean consolidation time t_{50}. The slowest tests were at $50 \cdot t_{50}$, which is the rate specified by ASTM for drained testing (ASTM D 3080). For pure kaolinite, shear displacement rates ranged from 0.006 to 10 mm per minute, which is the upper limit of shear displacement rate for the device used in this study. The corresponding multipliers on t_{50} ranged from 50 to 0.031. For the kaolinite-bentonite mixture, the selected shear rates ranged from 0.004 to 10 mm per minute (corresponding to $50 \cdot t_{50}$ to $0.019 \cdot t_{50}$). The pre-shear vertical consolidation stresses were σ'_c = 51.6, 104.4, and 188.7 kPa for kaolinite and σ'_c = 104.4 kPa for kaolinite-bentonite. Recall that consolidation pressures of approximately 50 kPa produce slightly overconsolidated specimens, whereas stresses near 100 and 185 kPa produce normally consolidated specimens. Bro (2007) provides a complete list of the 71 direct shear tests performed during this investigation as well as a description of the test apparatus.

Example results of direct shear tests run at different displacement rates are shown in Figure 3, which shows stress-displacement and volume change-displacement for pure kaolinite consolidated to σ'_c = 104.4 kPa. As expected, the drained test (at $50 \cdot t_{50}$) shows much higher shear strengths and contractive volume change than the rapid shear test. Moreover, note that the failure displacement (approximately 1 mm) for the rapid test is relatively small and is followed by mild softening, whereas the drained test appears to reach its maximum resistance at a much larger displacement (approximately 6 mm). We generally define shear strength as the peak shear resistance at small displacements (e.g., the 1 mm

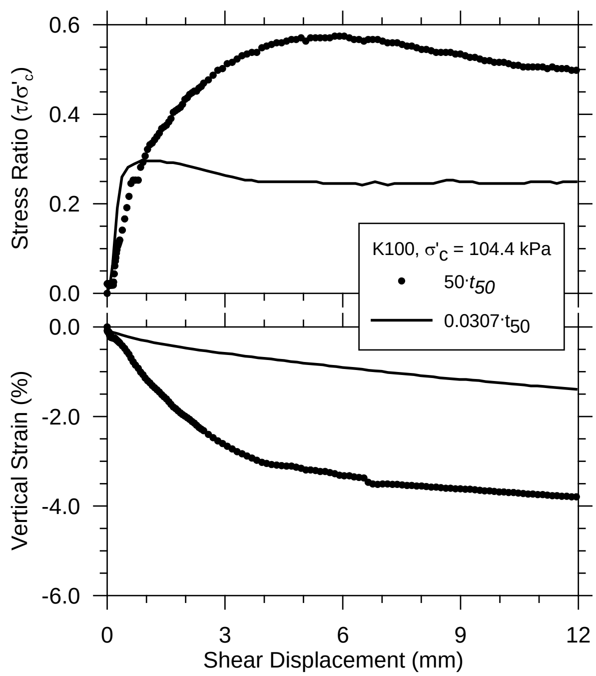

FIG. 3. Results of direct shear tests performed at fast and slow shear rates on kaolinite

resistance for the fast test in Figure 3), even though larger resistances were often measured near the end of the tests at large displacement. Stress quantities are corrected for the reduced contact area that develops during shear displacement.

Figures 4 and 5 show the variation of strength ratio with shear displacement rate for the two soil types and comparisons to the mean strength ratio from undrained simple shear tests. The data shown in these figures are for normally consolidated specimens ($\sigma'_c \approx$100 and 185 kPa). Both materials show a general trend of decreasing strength ratio with increasing shear displacement rate. For kaolinite, the measured strength ratios varied from an average of 0.53 for fully drained tests to 0.26 for the fastest tests. Similar trends of strength ratio with shear rate are observed for slightly overconsolidated kaolinite specimens with $\sigma'_c \approx$50 kPa (Bro, 2007). For the kaolinite-bentonite mixture, the measured strength ratios similarly vary from 0.40 (drained) to 0.16 (fastest).

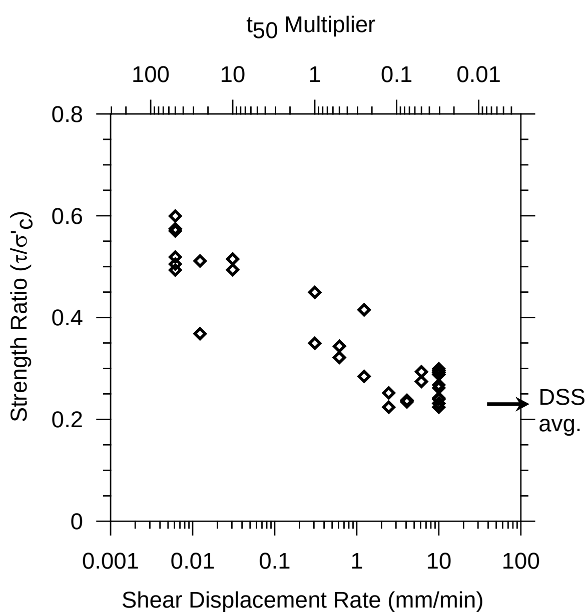

FIG. 4. Variation of strength ratio from direct shear tests on normally consolidated kaolinite with shear displacement rate.

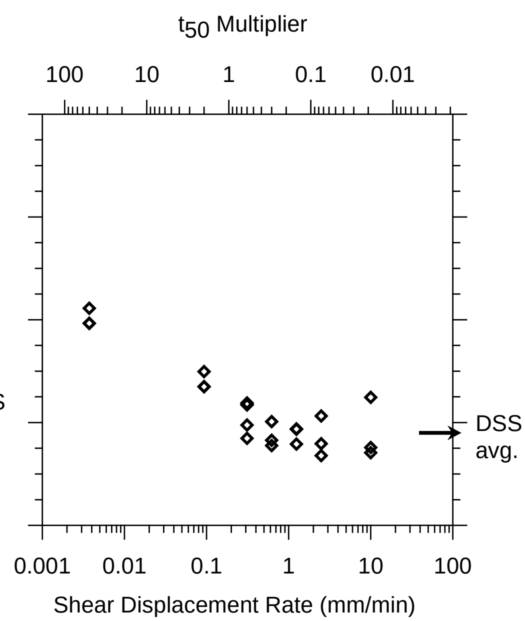

FIG. 5. Variation of strength ratio from direct shear tests on normally consolidated kaolinite-bentonite mixture with shear displacement rate.

As can be seen in Figures 4-5, the strength ratios measured in direct shear tests on both soil materials with shear displacement rates corresponding to approximately $0.125 \cdot t_{50}$ or faster reasonably approximate the undrained strength ratio measured in 'slow' direct simple shear tests. Using all direct shear test results faster than this rate (including the slightly overconsolidated results), residuals of the corresponding "estimates" of undrained strength are defined as:

$$R = \left(\tau_{ff} / \sigma'_c\right)_{DS} - \left(\tau_{ff} / \sigma'_c\right)_{DSS} \tag{1}$$

where $(\tau_{ff}/\sigma'_c)_{DS}$ is strength ratio evaluated from direct shear testing at shear rates corresponding to times of $0.125 \cdot t_{50}$ or faster and $(\tau_{ff}/\sigma'_c)_{DSS}$ is the mean undrained strength ratio from simple shear testing reported above. Positive residual indicates over-prediction of strength by direct shear test relative to constant volume simple shear testing. Figure 6 shows a histogram of the residuals, revealing a mean of 0.01 and a standard deviation of 0.04. This suggests a lack of statistically significant bias of strength ratios estimated by the rapid direct shear tests. Figure 6 spans both material types and overconsolidation ratios, but results for finer discretizations of the data are not significantly different.

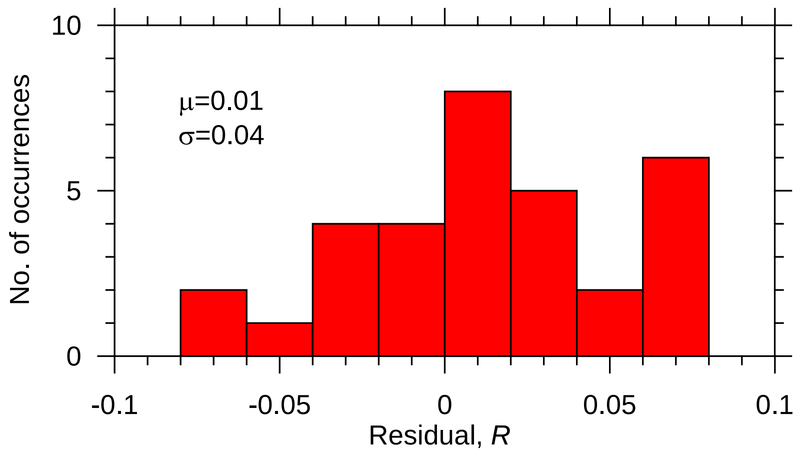

FIG. 6. Residuals of shear strength ratio estimated by direct shear tests.

Volume Change from Direct Shear Testing at Various Displacement Rates

An additional parameter of interest is the amount of vertical strain (related to change in void ratio) that occurs during shear, particularly for the direct shear tests approximating the undrained condition. In truly undrained or constant volume shear testing, the change in specimen height and void ratio during shear are zero. Figure 7 shows the observed variation of average vertical strain at failure, $\varepsilon_{vf} = \Delta H/H$ with shear displacement rate. Negative values of ε_{vf} indicate contraction of the specimen. As expected, the magnitude of vertical strain decreases with increased shear displacement rate because there is less time for drainage to occur. At the upper-limit shear displacement rate, vertical strain can be seen to approach, but not reach, zero. Typically, the average vertical strain measured at failure during tests run at 10 mm per minute was between -0.1 and -0.7% for pure kaolinite specimens, and between -0.6 and -0.9% for the kaolinite-bentonite mixture. For comparison, we note here that the average vertical strain at failure measured in the direct simple shear tests for the current study was -0.17%, whereas the limit recommended in ASTM procedures for undrained direct simple shear testing is ±0.05%.

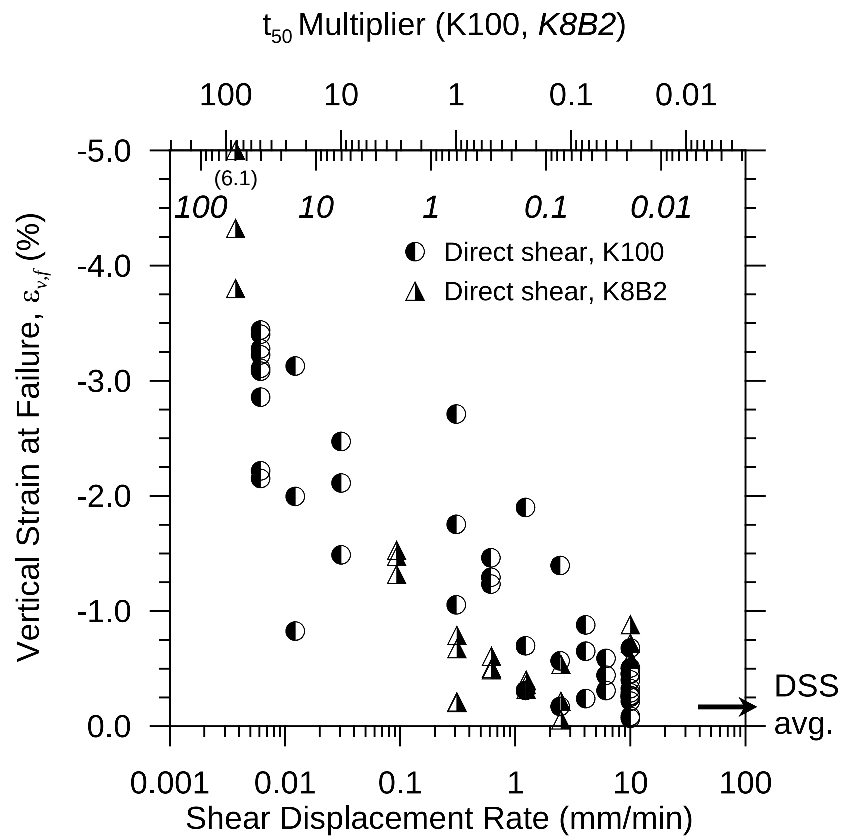

FIG. 7. Variation of vertical strain at failure from direct shear tests with shear displacement rate and comparison to mean from undrained simple shear tests.

The observed average vertical strain data illustrate a key concept that even at the fastest shear displacement rate possible with this device, some volume change of the specimen occurs during direct shear testing and true constant volume conditions are not achieved. There are several reasons for this. First, there is some soil loss due to soil squeezing out of the box during consolidation under the applied load, and to a lesser extent, during shear. Volume change due to this effect cannot be readily quantified. Second, top cap rotation can introduce vertical displacement on portions of the cap; however this effect was minimized through appropriate positioning of the vertical LVDT. Third, the soil can consolidate due to the increased major principal stresses that occur during shear testing. The latter effect is quantifiable under drained conditions by simply using the change in major principal stress combined with the virgin compression index in a standard consolidation calculation. As shown by Bro (2007), the resulting vertical strains are of the same order as the observed volume change of the drained specimens. Hence, a significant fraction of the volume change reported in Figure 7 is likely resulting from soil consolidation induced by shearing. As noted previously, fast shear rates reduce this apparent consolidation but do not eliminate it.

Limitations Related to Rate Effects

We have demonstrated general compatibility between peak shear strengths from fast direct shear tests ($t_f < 0.125 \cdot t_{50}$) and slow direct simple shear tests (strain rates of approximately 1% and 11%/hr). It is well known that undrained strength is rate dependent, increasing as the rate of shear strain increases by amounts ranging from approximately 5% to 15% per log cycle of strain rate (Ladd, 1991; Sheahan et al., 1996). The strain rates associated with the direct shear tests are unknown, because the shears strains are not measureable (due to unknown thickness of the sheared material). However, those strain rates would certainly be much more rapid than those applied in the simple shear tests. Accordingly, we wish to emphasize that our finding of approximate strength compatibility is for 'rapid' shear conditions in direct shear and 'slow' in simple shear. If the simple shear strain rates were changed to be relatively fast (e.g. for seismic applications), the rapid direct shear tests would likely underestimate undrained strengths in the absence of cyclic softening (Boulanger and Idriss, 2007).

CONCLUSIONS

We present the findings from 71 direct shear tests performed over a wide range of shear displacement rates, with corresponding times to failure ranging from $50 \cdot t_{50}$ (fully drained) to $0.02 \cdot t_{50}$. The results are compared with undrained shear strengths measured from 10 monotonic direct simple shear tests performed at relatively slow shear strain rates (approximately 1 to 10%/hr). Measured shear strengths are normalized by pre-shear vertical effective consolidation stress. The data were developed from testing of two clays composed of pure kaolinite (normally consolidated and lightly overconsolidated) and a kaolinite-bentonite mixture (normally consolidated).

We find that a reasonable approximation of the 'slow' undrained shear strength of clay specimens was produced using direct shear tests conducted at rapid shear displacement rates such that specimen failure was obtained in an amount of time equal to or less than $t_{50}/8$. The vertical strain at failure measured in such tests was observed to become small, but does not reach zero. After compiling the data from the three combinations of soil type and stress history, the bias on strength ratio determined by this approximation was found to be insignificantly small (approximately 0.01) with a standard deviation of 0.04. The bias would be expected to become negative (indicating under-prediction) if the point of comparison from undrained simple shear testing had utilized substantially faster monotonic strain rates, such as for seismic conditions (in the absence of cyclic softening).

As shown in Figures 4-5, our direct shear results confirm that drainage conditions within clay test specimens are greatly affected by shear displacement rate. Consequently, it is possible that some practicing engineers may be inadvertently measuring partially drained response by conducting direct shear tests at rates that are too fast if the desired conditions are drained. The implications of such test practices would depend on the stress history of the specimen. For normally consolidated or lightly overconsolidated materials such as those tested in the current study, drained soil strength would be under-predicted. However, for heavily overconsolidated materials exhibiting dilatant behavior during shear, drained soil strength would be over-predicted.

It should be emphasized that in the absence of active or passive control of volume change, it does not appear to be possible to achieve a constant volume condition in the specimen. A globally undrained condition is not achieved because of measurable void ratio change during the test. For this reason, direct shear tests run at fast shear displacement rates can, at best, only provide an approximation of undrained strength. This approximation may be sufficient for certain projects for which access to a simple shear apparatus is limited and a relatively crude approximation of undrained strength is sufficient to demonstrate stability against undrained failure. However, for critical projects (including any project where stability under undrained conditions is marginal), it is recommended that more accurate constant volume undrained testing be performed.

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