

Constant Volume Ring Shear Apparatus

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ABSTRACT: The paper describes a constant volume ring shear apparatus that allows the measurement of the undrained peak and residual shear strengths of cohesive soils. The undrained peak and residual strengths are applicable to seismic stability evaluations of slopes comprised of or founded on cohesive soil. The constant-volume ring shear apparatus is equipped with a mechanism to adjust the normal stress during shear and a new specimen container that allows undisturbed specimens to be trimmed directly into the container. The normal stress is adjusted during shear such that the height of the soil specimen remains constant. This results in a constant volume or undrained shear condition. The results of constant volume ring shear tests on normally consolidated Drammen clay are compared with the results of undrained direct simple shear tests. The comparison reveals that the undrained peak shear strength obtained using the constant volume ring shear and direct simple shear apparatuses are in agreement. However, the constant-volume ring shear apparatus allows the measurement of the undrained residual strength because it permits unlimited continuous shear displacement.

KEYWORDS: soft clays, clay shales, residual strength, slope stability, torsion shear tests, ring shear tests

A significant number of landslides involving cohesive soils have occurred during earthquakes. A few of the more notable landslides occurred in Anchorage during the 1964 Alaskan earthquake (Shannon and Wilson 1964) and in Sainte-Thècle and Saint-Adelphe, Canada during the 1988 Saguenay earthquake (Lefebvre et al. 1992). Following the 1964 Alaska earthquake, many of the landslides that occurred were studied intensively. The main failure mechanism proposed in the resultant literature was liquefaction of sand lenses (Shannon and Wilson 1964; Seed and Wilson 1967; Seed 1968; and Wilson 1967). However, recent studies by Updike (1984), Idriss (1985), Updike and Carpenter (1986), Updike et al. (1988), Olsen (1989), Moriwaki et al. (1989), and Stark et al. (1995) indicate that the slides were caused by a large undrained strength loss in the soft Bootlegger Cove clay involved in the slides. As a result, these landslides have led to an interest in the undrained peak and residual shear strength of cohesive soils.

The undrained peak shear strength is anisotropic, and thus the magnitude depends on the mode of shear. The values of peak undrained shear strength mobilized in different directions are measured by laboratory shear tests that simulate distinct modes of shear. The laboratory shear tests most widely used to measure the

peak undrained strength for stability analyses are triaxial compression (TC), triaxial extension (TE), and direct simple shear (DSS). However, triaxial and direct simple shear tests are not suitable for determining the undrained residual shear strength because these apparatuses impart a limited deformation along a failure surface. As a result, a torsional ring shear apparatus was developed to evaluate both the undrained peak and residual shear strengths. The main advantage of the torsional ring shear apparatus is that it continuously shears the soil in one direction. This allows orientation of clay particles parallel to the direction of shearing and the development of a residual strength condition. The ring shear apparatus results in the measurement of a complete (peak to residual) shear stress-displacement relationship.

At present there are several torsional ring shear apparatuses available, for example, the apparatus based upon the Norwegian Geotechnical Institute and Imperial College concept (Bishop et al. 1971), the Bromhead ring shear apparatus (Bromhead 1979), and the solid-ring rotation shear apparatus (La Gatta 1970). The Bromhead ring shear apparatus has facilitated the use of the torsional ring shear test in engineering practice. The Bromhead ring shear apparatus has been modified by Stark and Eid (1993) and Stark and Poeppel (1994) to measure the drained residual strength of cohesive soils and geosynthetic/geosynthetic or geosynthetic/soil interface strengths, respectively. The main objective of this research is to develop a practical apparatus and test procedure to measure the peak and residual undrained shear strengths of cohesive soils. To accomplish this objective, the Bromhead (1979) ring shear apparatus was modified to apply a constant volume condition.

This paper describes the new constant volume ring shear apparatus, results of constant volume ring shear tests on undisturbed Drammen clay, and a comparison of previous direct simple shear tests on Drammen clay to verify the accuracy of the constant volume ring shear apparatus.

Previous Constant Volume Laboratory Tests

Taylor (1952) introduced the use of a constant volume shear test to measure the peak undrained shear strength. A modified direct shear apparatus was used by Taylor (1952) to perform constant volume tests on Boston Blue Clay. The size of the direct shear specimen was 76.2 mm square by 12.7 mm thick. Figure 1 illustrates the results of a constant volume direct shear test on a normally consolidated specimen of Boston Blue Clay at a normal consolidation stress, σ'_{vc} , of 760 kPa. It can be seen from Fig. 1 that the undrained peak shear stress ratio, shear stress (τ) divided by the normal consolidation stress, is approximately 0.23. An undrained residual strength condition was not achieved because of the limited continuous shear displacement allowed in one direc-

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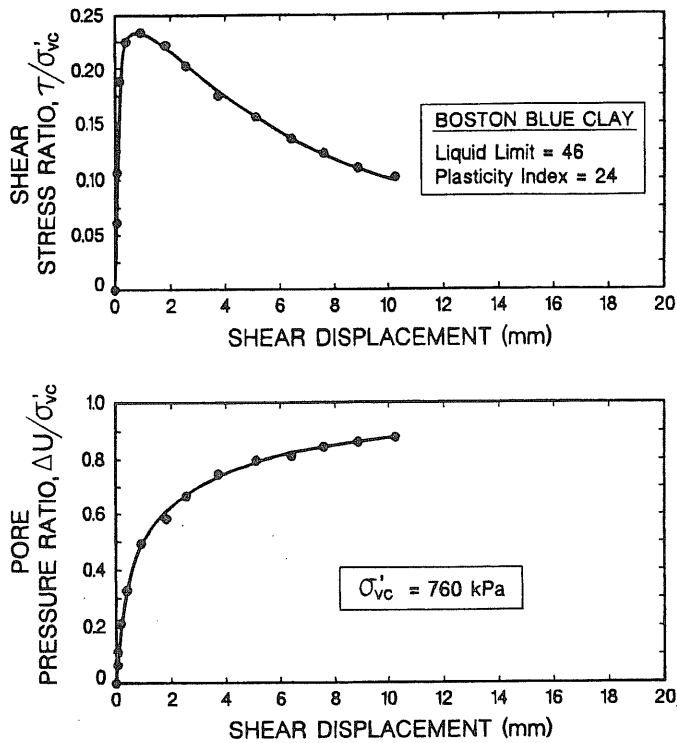


FIG. 1—Constant volume direct shear test on Boston Blue Clay (after Taylor 1952).

tion by the direct shear apparatus. It can be seen that the test was stopped after a shear displacement of approximately 10 mm.

Also presented in Fig. 1 is a relationship between pore-water pressure ratio and shear displacement. The pore-water pressure ratio, i.e., shear-induced pore-water pressure (Δu) divided by the normal consolidation stress, is estimated by equating the reduction in normal stress during shear to the shear-induced pore-water pressure. It can be seen that the pore-water pressure ratio is approaching a constant value as the shear stress ratio approaches a constant value.

Bjerrum and Landva (1966) used constant volume direct simple shear tests to measure the peak undrained shear strength of the quick clay from Manglerud. Figure 2 illustrates the shear-stress-ratio–shear-strain relationship from a direct simple shear test on a normally consolidated specimen of Manglerud clay at a normal consolidation stress of 100 kPa. The shear strain is defined as the

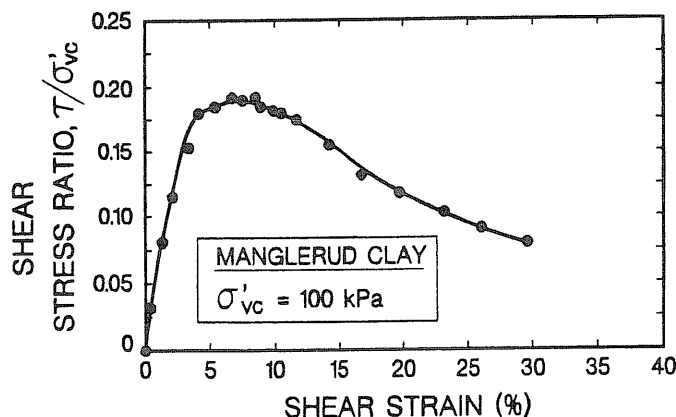


FIG. 2—Direct simple shear test on Manglerud quick clay (after Bjerrum and Landva 1966).

horizontal displacement divided by the height of the specimen. The height of the specimen was 10 mm. The liquid limit (LL), plasticity index (PI), and clay size fraction (CF) of the Manglerud clay are 27, 8, and 48%, respectively.

It can be seen from Fig. 2 that the undrained peak stress ratio is approximately 0.19 in the direct simple shear test. The undrained peak stress ratio was mobilized at a shear strain of about 6.6%. This shear strain corresponds to a shear displacement of 0.66 mm. Note that after the peak stress ratio is mobilized, the shear stress ratio starts to decrease with increasing shear strain. However, the test is terminated at a shear strain of approximately 30% (i.e., 3 mm of shear displacement) before a residual strength condition could be obtained.

In summary, triaxial, direct shear, and direct simple shear apparatuses are not suitable for estimating the undrained residual strength because a limited amount of continuous deformation can be induced along a failure surface. As a result, it was necessary to develop a torsional ring shear apparatus to evaluate the undrained residual strength, $s_{u(\text{residual})}$. The main advantage of the torsional ring shear apparatus is that it continuously shears the soil in one direction for an unlimited displacement. Other advantages of the ring shear apparatus include a constant cross-sectional area during shear, more consistent results, and the possible use of data acquisition techniques.

Original Bromhead Ring Shear Apparatus

The original ring shear apparatus is described by Bromhead (1979). The apparatus is manufactured by Wykeham-Farrance Engineering Limited in Slough, England. The ring shear specimen is annular with an inside diameter of 70 mm and an outside diameter of 100 mm. Drainage is provided by two bronze porous stones mounted on the top loading platen and the bottom of the specimen container. The specimen is confined radially by the specimen container. Stark and Eid (1993) modified the original specimen container of the Bromhead ring shear apparatus to facilitate measurement of the drained residual strength of cohesive soils. The modified specimen container allows a remolded specimen to be overconsolidated and precut prior to shear. This allows measurement of the drained residual strength available along preexisting shear surfaces present in old landslides, bedding shears in folded strata, or sheared joints or faults.

Constant Volume Ring Shear Apparatus

To investigate the undrained peak and residual shear strengths, the original Bromhead (1979) ring shear apparatus was further modified to conduct constant volume tests. The modifications include a mechanism for adjusting the normal stress during shear, such that the volume change is negligible during shear, and fabricating a new specimen container to allow undisturbed specimens to be trimmed directly into the container. Figure 3 shows a schematic side view of the constant volume ring shear apparatus. The dead weight hanger is connected to a horizontal beam. The beam is connected to a loading yoke that applies the normal load to the loading platen of the specimen container. The horizontal beam produces a loading ratio of 10:1 between the weight hanger and the loading platen. The specimen is sheared by rotating the specimen container past the stationary loading platen at a constant shear displacement rate. Two opposing proving rings form a force couple at the loading platen, which is used to determine the applied shear stress.

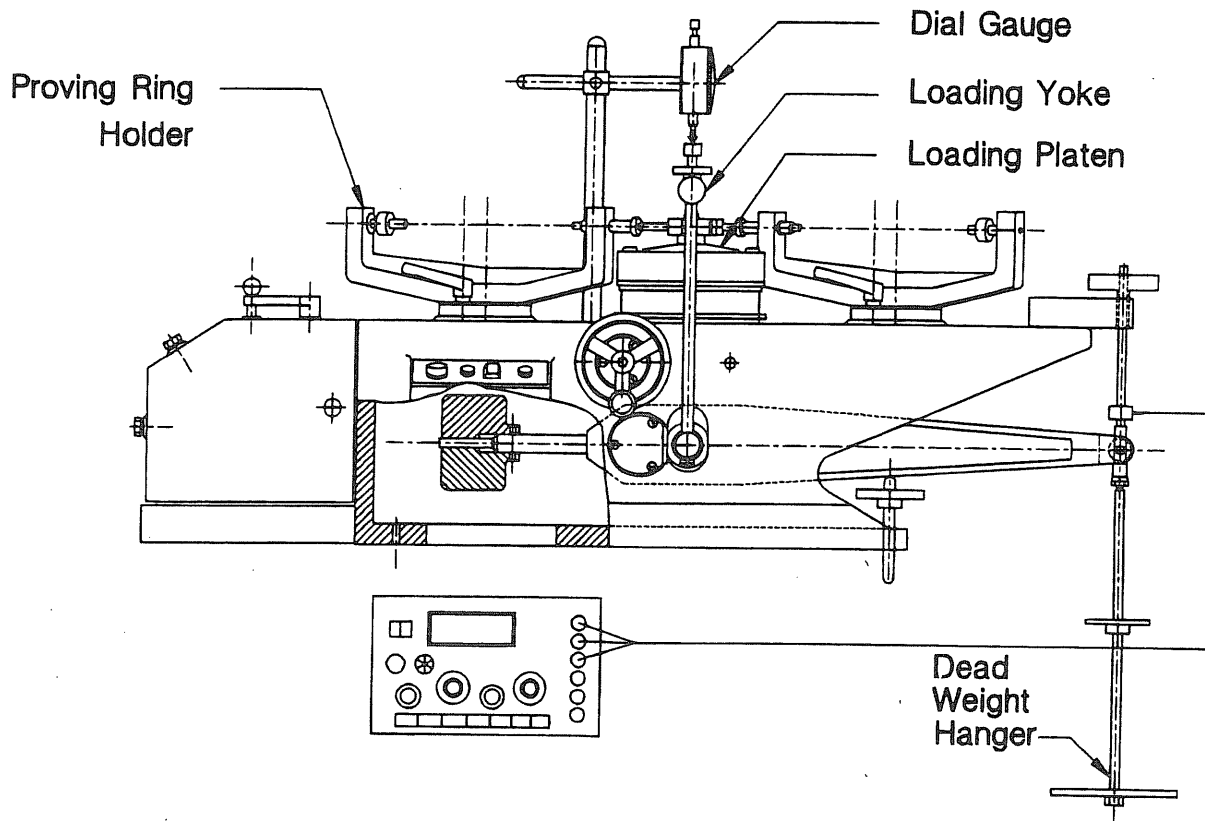


FIG. 3—Schematic side view of constant volume ring shear apparatus.

The constant volume ring shear apparatus has been developed primarily for undrained testing of soft clays, which exhibit contractive behavior during shear (i.e., a reduction of normal stress is needed to maintain constant volume). However, if an overconsolidated clay could be trimmed into the ring, the device could be also used for undrained testing of soils that exhibit dilative behavior (i.e., an increase of normal stress is needed to maintain constant volume). In such a case, the load applied to the dead weight hanger would increase during the shearing stage of the test to maintain constant volume. Therefore, the load on the hanger at the end of the test would be greater than the load required to produce the desired consolidation stress.

The ring shear specimen is annular with an inside diameter of 70 mm and an outside diameter of 100 mm. Drainage is provided from the bottom by a 6.5-mm-thick annular bronze porous stone that is secured by four screws. The specimen is confined radially by the specimen container, which is 10 mm deep. Drainage is provided from the top by an annular bronze porous stone, which is attached to the loading platen. The porous stone is secured to the loading platen using four screws. The porous stone is 5.7 mm thick and aids the transfer of shear stress to the top boundary of the soil specimen. The porous stone is serrated to develop a strong interlock with the top boundary of the soil specimen. Two intersecting groups of serration lines are grooved in the porous stone. Spacing between lines in each group is 2 mm. The indentation angle is 45° with a depth of 0.7 mm. These serrations prevent slippage at the loading platen/soil interface during shear. The annular bronze porous stone also has an inner and outer groove machined at 1 mm from the bottom edge of the stone to accommodate two O-rings. These O-rings close the gap between the top annular ring and the walls of the specimen container to minimize

the amount of extrusion that takes place during shear. The amount of soil extrusion is negligible at the peak strength, and thus the impact on the measured peak strength is insignificant. Field case histories also show that the measured undrained residual strength is in good agreement with field observations (Stark et al. 1995) and thus not significantly affected by soil extrusion.

Interpretation of Ring Shear Test Results

A uniform shear stress distribution across the failure surface is assumed to compute the applied shear stress. This assumption leads to the use of Eq 1 to compute the average shear stresses, τ_a ,

$$\tau_a = \frac{3M}{2\pi(R_2^3 - R_1^3)} \quad (1)$$

where M represents the moment, and R_1 and R_2 are the inner and outer radii of the specimen, respectively. It is recognized that a nonuniform shear displacement occurs across the failure surface because of the difference in shear displacement between the inner and outer edges of the specimen. This may lead to peak shear stresses measured in the ring shear apparatus that are slightly lower than direct shear values. However, Hvorslev (1939, 1969) showed mathematically and experimentally that the maximum shear stress computed from Eq 1 is in agreement with the peak shear stress measured in direct shear tests when an annular specimen with a ratio of R_1/R_2 greater than or equal to 0.5 is used. The ratio of R_1/R_2 is 0.7 in the constant volume ring shear apparatus. It will be shown in a subsequent section that the assumption of a uniform shear stress distribution in the constant volume ring shear apparatus is adequate for the measurement of the peak shear stress of cohesive

soils. A uniform shear stress distribution is present across the ring shear specimen at the residual condition, and thus the difference in shear displacement between the inner and outer edges of the specimen is irrelevant (Bishop et al. 1971; La Gatta 1970; and Hvorslev 1939).

A nonuniform shear displacement occurs across the failure surface because of the difference in shear displacement between the inner and outer edges of the specimen. As a result, the corrected average linear displacement (δ), calculated for the median specimen radius (i.e., 42.5 mm), is used to plot the results. To compute this corrected average linear displacement, Eq 2 is used

$$\delta = \theta r - C_{pr} \frac{r}{R} \quad (2)$$

where θ represents the rotation of the apparatus in radians, r is the median specimen radius, R is the torque arm radius, and C_{pr} is the average proving ring compression at the imposed shear stress. The first term in Eq 2 represents the average linear displacement. The second term represents the displacement of the loading platen at the median specimen radius caused by compression of the proving rings.

Unload Mechanism Description

The unload mechanism developed to maintain a constant specimen volume during shear, and thus conduct an undrained test, is illustrated in detail in Fig. 4. It consists of a steel bracket that is connected to the end of the horizontal beam. The horizontal beam applies the dead weight on the hanger to the top of the specimen. Attached to the top of the bracket is a sensitive load cell. The load cell has a capacity of 22.7 kg and is manufactured by SENSOTEC. Connected to the top of the load cell is a threaded steel rod 9 mm in diameter. This threaded steel rod goes through a 14-mm-diameter hole in an aluminum beam that is connected to the frame of the original apparatus by two screws. A nut is threaded to the top of the steel rod. The nut is threaded and rotated during shear to adjust the amount of normal load transferred to the loading platen. An electronic readout device is connected to the load cell such that the force carried by the load cell can be determined. The nut is manually adjusted such that a negligible amount of vertical displacement of the soil specimen occurs during shear.

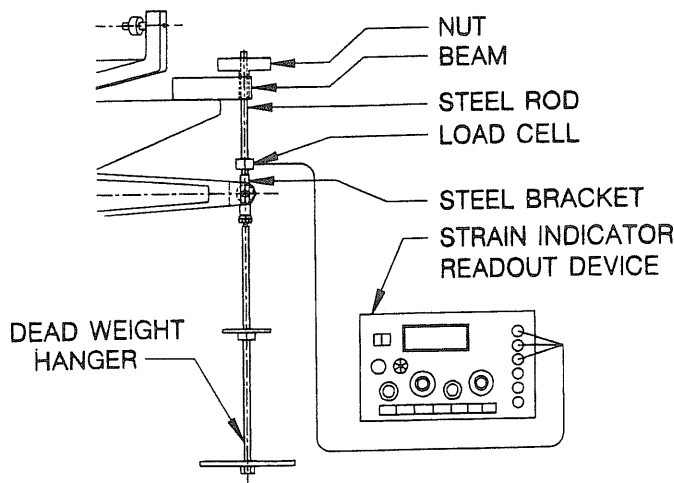


FIG. 4—Unload mechanism for ring shear apparatus.

Undisturbed Specimen Container Description

A new specimen container was also fabricated to allow undisturbed annular specimens to be trimmed directly into the container. The new container consists of the following eight parts: (1) lower outer ring, (2) inner ring, (3) 6.5-mm-thick bronze porous stone, (4) outer cutter ring, (5) upper outer ring, (6) center core, (7) inner cutter ring, and (8) holder. The assemblage and use of these eight pieces is divided between the trimming and shearing stages of the test. For the trimming phase, the inner cutter ring, inner ring, outer cutter ring, upper outer ring, and holder are assembled to create the trimming apparatus (Fig. 5). For the shearing phase, the inner and outer cutter rings and holder are removed. The porous stone, center core, and lower outer ring are assembled to create the specimen container (Fig. 6).

Assemblage of the trimming apparatus (Fig. 5) proceeds as follows: (1) the inner cutter ring is threaded into the inner ring, (2) the inner ring is threaded into the holder, (3) the outer cutter ring is secured to the top surface of the upper outer ring by four screws, and (4) the upper outer ring is threaded into the holder. The cutters possess sharp edges that allow the soil on the inside and outside of the cutter to be trimmed away. Removal of this soil permits the ring to pass over the exposed specimen and minimizes disturbance during trimming. The walls of the trimming container are coated with a film of high-vacuum silicone grease to reduce friction as the specimen passes into the empty specimen cavity.

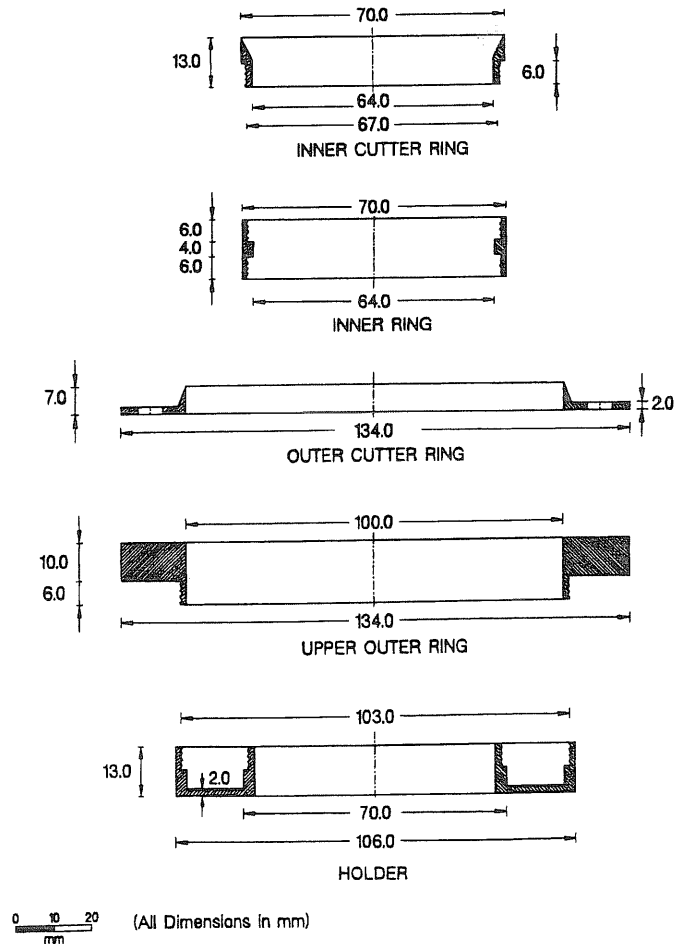


FIG. 5—Schematic of trimming apparatus.

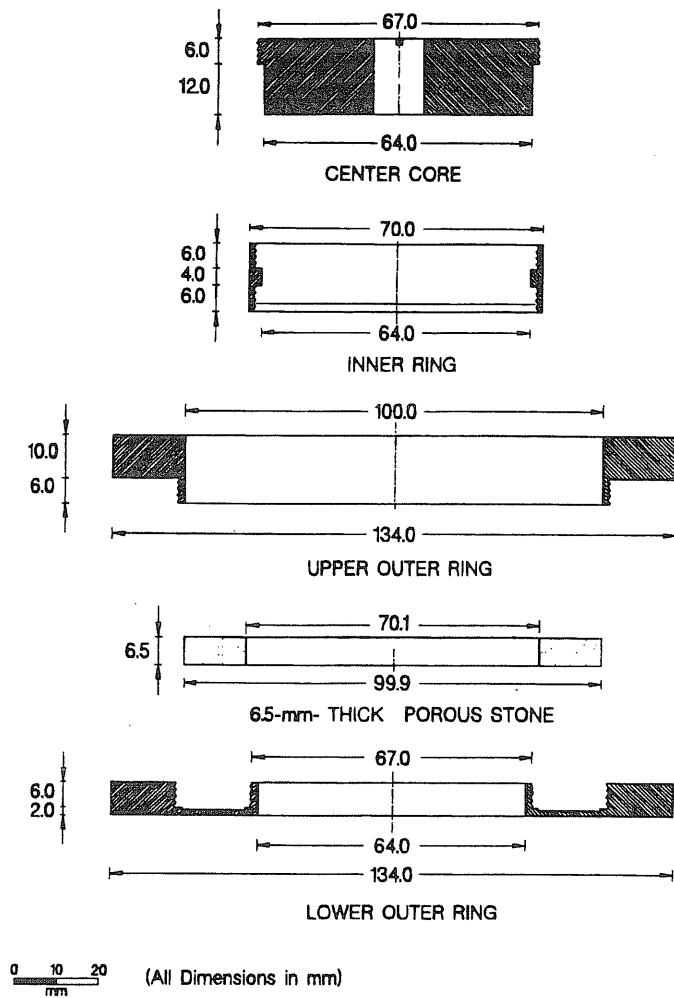


FIG. 6—Schematic of specimen container.

After the trimming apparatus has been assembled, it is placed in the trimmer holder (sharp edges down). The trimmer holder is a rigid system that consists of a flat, fixed base and a movable platform. The trimming apparatus is secured to the movable platform by two screws. The position of the platform can be fixed at any location by the hand crank. The soil sample is placed on the flat, fixed base. The trimming apparatus is moved downward until the sharp edges of the inner and outer cutter rings penetrate approximately 2 mm into the soil sample. Trimming of the soil on the outside and inside of the trimming apparatus is accomplished from around and through the middle of the trimming apparatus. A wire saw is used to trim around the outside of the apparatus, and a surgical razor blade is used to trim in the inside. Care is taken to avoid overtrimming of the soil sample, which would leave a void in the specimen.

After the excess soil has been trimmed away, the platform is moved down approximately 2 mm and the trimming process is repeated. This process continues until 17 to 18 mm of soil has passed into the specimen ring. The container is then separated from the remaining soil sample with a wire saw. To minimize the loss of moisture, the trimming process is carried out inside a temperature- and humidity-controlled room.

The trimming apparatus is removed from the trimmer holder. The outer cutter ring is unscrewed and carefully removed. The excess soil above the edge of the specimen container is removed

with a wire saw. After this trimming, the inner cutter ring is threaded out. The surface is then finished with a rigid razor blade such that the top of the specimen is flush with the top of the specimen container. The holder is then threaded out. The porous stone is secured to the lower outer ring by four screws. The lower outer ring is threaded into the upper outer ring. The center core is threaded into the inner ring to create the specimen container (Fig. 6). The specimen and specimen container are now ready for the shearing phase of the test.

The specimen is consolidated by increasing the normal load in small increments. Each load increment is maintained until the end of primary consolidation. Once the end of primary consolidation under the desired normal stress is achieved, the specimen is ready for shearing.

Constant Volume Ring Shear Test Procedure

In the constant volume ring shear apparatus, the specimen is sheared by rotating the specimen container past the stationary loading platen at a constant rate of 0.018 mm/min. The procedure described by Gibson and Henkel (1954) is used to determine the shear displacement rate. This displacement rate is selected such that the shear-induced pore-water pressures in the specimen are zero throughout the test. The normal stress is reduced during shear to maintain a constant specimen height or volume. It is assumed that the decrease in applied vertical stress during shear is equivalent to the increase in shear-induced pore-water pressure that would occur in an undrained test with constant vertical stress. The validity of the pore-water pressure assumption for constant volume tests was verified by Dyvik et al. (1987) for the direct simple shear apparatus and by Berre (1981) for the triaxial apparatus.

During a constant volume test, the height of the specimen is monitored using the dial gage that measures vertical displacement of the loading platen (see Fig. 3). The nut connected to the steel rod in the unload mechanism (see Fig. 4) is used in combination with the vertical dial gage to adjust the normal stress such that the specimen thickness remains constant during shear. Adjustments are made by manually rotating the nut an amount required to maintain negligible vertical displacement of the soil specimen. When the nut is rotated, a portion of the dead weight is transmitted to the rod, and the load cell indicates the magnitude of this load.

During shear the O-rings that close the small gap between the top annular ring and the specimen container produce friction. The walls of the specimen container and two O-rings are coated with a film of high-vacuum silicone grease to reduce this friction. To determine the remaining magnitude of O-ring friction, an O-ring friction test is conducted prior to the actual test. The setup and shearing procedure are identical to what has been described, with the exception that the soil specimen is absent. The loading platen is supported by the center post, which generates negligible friction. The amount of friction measured during the O-ring friction test is subtracted from the shear force measurements recorded during the actual test. A series of O-ring friction tests revealed that the friction shear force introduced by the O-rings is 0.006 kN. For comparison purposes, if this friction shear force is converted into an equivalent shear stress using Eq 1, it results in a "friction shear stress" of only 2.75 kPa.

The O-rings that close the small gap between the top annular ring and the specimen container may also reduce the applied normal stress. To determine the magnitude of this normal stress reduction, a compression test is conducted prior to the actual test. An annular piece of rubber is placed in the specimen container instead of the

soil specimen. The load-deflection relationship of the rubber is determined under the following two conditions: (1) O-rings attached to the top annular ring and (2) O-rings not attached to the top annular ring. The reduction in normal load caused by the O-rings is equal to the difference between the two load-deflection relationships at a given displacement. A series of compression tests reveal that the reduction in normal stress varies from 2 to 4 kPa.

To account for the deformation of the apparatus, the machine load-deflection relationship is also determined for the loading and unloading processes. This relationship allows the specimen deformation for a given normal load to be obtained by subtracting the machine deformation from the measured vertical deformations. Therefore, in a constant volume test the normal load is adjusted such that the machine deflection equals the measured vertical deformation at all normal loads.

Drammen Clay: General Properties and Description

A sample of Drammen clay was obtained using a 200-mm-diameter sampler at the Danvik-gate site in Drammen, Norway. The in situ depth of the sample is 9.33 m. This plastic marine clay sample has the following index properties: (1) natural water content of 51%; (2) liquid limit of 48%; (3) plasticity index of 24%; and (4) clay-size fraction of 60 to 70%. For this Drammen clay sample, the value of effective vertical overburden pressure is estimated to be 95 kPa and the preconsolidation pressure is estimated from an oedometer test to be 140 kPa. This corresponds to an overconsolidation ratio of approximately 1.5.

Drammen clay from Drammen, Norway, is a post-glacial marine clay deposit with a pore-water salt concentration of 25 g/L. Drammen clay has not been subjected to a load greater than the present overburden. However, it has developed an overconsolidation ratio in the range of 1.4 to 1.6 as a result of approximately 3000 years of aging (Bjerrum 1967). Drammen clay was used by Dyvik et al. (1987) and Berre (1981) to verify the validity of equating shear-induced pore-water pressure to the reduction in normal stress in a constant volume test using the direct simple shear and triaxial apparatuses, respectively. A sample of Drammen clay was obtained from the Norwegian Geotechnical Institute to compare the results of constant volume ring shear and direct simple shear tests and to investigate the accuracy of the constant volume ring shear test.

Test Results and Comparison with Previous Research

Of the three major modes of shear (triaxial compression, direct simple shear, and triaxial extension), the one most similar to the ring shear (RS) mode of shear is direct simple shear (DSS). It is recognized that the ring shear test does not produce a simple shear condition in the soil specimen. However, the results of a RS and DSS test are expected to be similar. Figure 7 shows the shear stress ratio-horizontal displacement relationship from a constant volume ring shear test on an undisturbed specimen of Drammen clay at a normal consolidation stress of 255 kPa. The RS and DSS tests were conducted at displacement rates of 0.018 mm/min and 0.0072 mm/min, respectively. In the ring shear test the thickness of the shear surface is unknown and therefore the results cannot be expressed in terms of shear strain. As a result, quantities such as modulus cannot be determined from this test.

For comparison purposes, the results of a direct simple shear test at σ'_{vc} equal to 255 kPa from Dyvik et al. (1987) have also

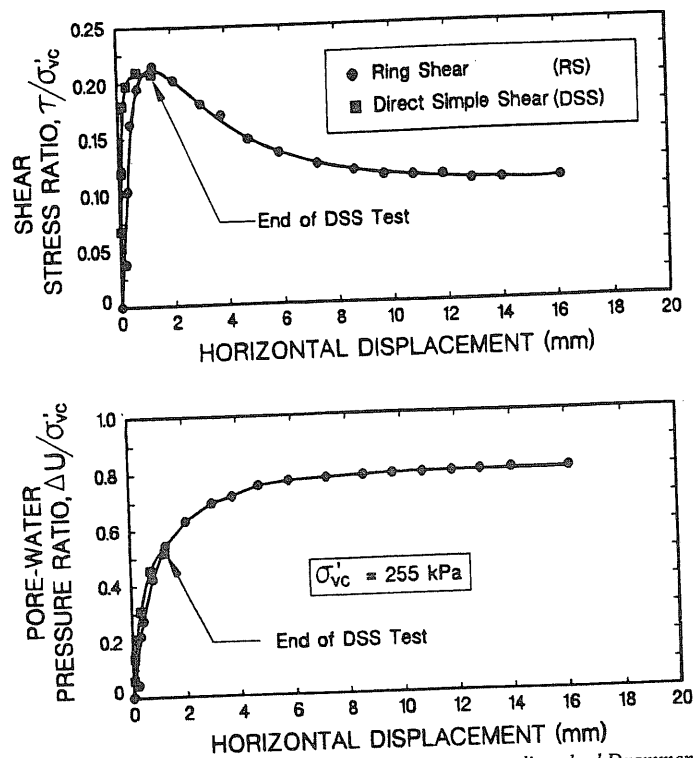


FIG. 7—Comparison of constant volume tests on undisturbed Drammen Clay at 255 kPa.

been plotted in Fig. 7. To accomplish this, the shear strain from the DSS test was converted into shear displacement for the comparison. The conversion was made by multiplying the shear strain by the height of the direct simple shear specimen at the end of the consolidation stage. It can be seen that the undrained peak stress ratio of 0.22 is reached after approximately 1.3 mm of horizontal displacement in the constant volume ring shear test. The undrained peak shear stress ratio from the direct simple shear test is also approximately 0.22 at a displacement of 0.72 mm. This agreement reinforces the validity of the constant volume ring shear test. Differences in the shear displacement at the peak stress ratio are attributed to the nonuniform shear displacement across the ring shear specimen. The horizontal displacement used to plot the ring shear data is based on the median radius of the specimen. Note that shortly after the peak undrained stress ratio is mobilized in the DSS test, shearing is stopped because of the limited shear displacement that can be applied in this apparatus.

After the undrained peak stress ratio is mobilized in the ring shear apparatus, shear displacement along the failure surface continues and the stress ratio starts to decrease with increasing displacement. Shear displacement along the failure surface causes orientation of soil particles parallel to the direction of shear. The movement and arrangement of particles along the failure surface is accompanied by a change in pore-water pressure. This continues until the undrained residual strength is mobilized. It can be seen from Fig. 7 that the undrained residual strength ratio is mobilized at a shear displacement of approximately 16 mm. The undrained residual stress ratio, $s_{u(\text{residual})}/\sigma'_{vc}$, is measured to be approximately 0.11. It can be seen that this undrained residual stress ratio is significantly lower than the peak undrained stress ratio. This large post-peak strength loss, approximately 50%, has important implications to the seismic stability of cohesive soil slopes. It is anticipated that the undrained peak strength is mobilized at a small displace-

ment and thus is related to the triggering of a landslide. If displacement continues due to earthquake shaking, the strength can be reduced to the residual value, which will probably lead to large lateral displacements.

Figure 7 also shows the pore-water-pressure ratio–horizontal-displacement relationship measured for Drammen clay in the constant volume ring shear and direct simple shear apparatuses. The pore-water pressure is assumed to be equal to the decrease in normal stress. It can be seen that the pore-water pressure ratio increases and then becomes essentially constant at the residual condition.

At the undrained residual strength condition, it is possible to compute the drained residual friction angle, ϕ'_r , of a clay from the results of a constant volume ring shear test using the following equation

$$\tan \phi'_r = \frac{\tau_r}{\sigma'_r} \quad (3)$$

where τ_r is the residual shear stress, and σ'_r is the effective stress at the residual condition. The value of τ_r/σ'_r is estimated to be 0.505 from the constant volume ring shear test. This corresponds to a value of ϕ'_r , computed using Eq 3, of approximately 27°. The measured ϕ'_r from a series of drained ring shear tests on Drammen clay, following the procedure described by Stark and Eid (1994), is 26°. This agreement reinforces the validity of the results and the constant volume test procedure.

Andersen et al. (1980) also presented the results of constant volume direct simple shear tests on normally consolidated Drammen Clay. Dyvik et al. (1987) used the results of these constant-volume direct simple shear tests on normally consolidated Drammen clay to further verify the validity of the pore-water pressure assumption. These tests were conducted at a normal consolidation pressure of 400 kPa. Figure 8 compares the normalized shear-stress–shear-displacement relationship from a constant volume ring shear test and a direct simple shear test conducted by Andersen et al. (1980). The RS and DSS tests were conducted at displacement rates of 0.018 mm/min and 0.012 mm/min, respectively. There is a small difference between the undrained peak shear stress ratio measured by the constant volume ring shear and direct simple shear apparatuses. The undrained peak stress ratio from the direct simple shear test is approximately 0.21, while the undrained peak stress ratio from the constant volume ring shear test is 0.20. This small difference is probably attributed to soil variability. There is

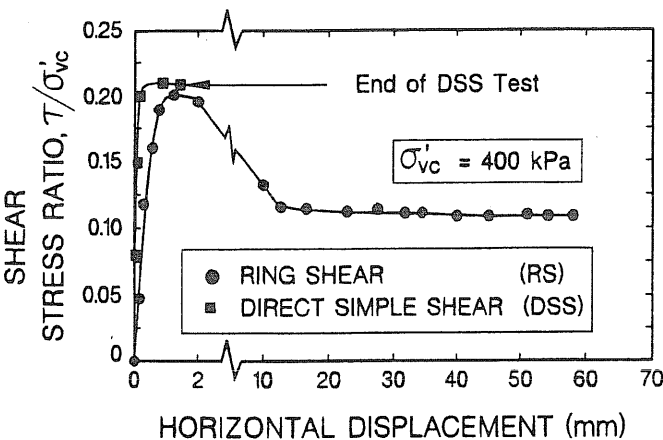


FIG. 8—Comparison of constant volume tests on undisturbed Drammen clay at 400 kPa.

also a difference in the shear displacement at which the peak shear stress ratio is mobilized, 0.87 and 1.12 mm for the DSS and RS tests, respectively. This difference in shear displacement is attributed to the nonuniform shear displacement across the ring shear specimen. It can be seen that the undrained residual strength ratio is measured to be approximately 0.11 and is significantly lower than the peak value.

The shear-stress–horizontal-displacement relationship for the constant volume ring shear test shown in Fig. 8 is plotted in Fig. 9 using the logarithm of horizontal displacement. This plotting technique was suggested by La Gatta (1970) to verify that a residual strength condition is obtained. This plotting technique accentuates the slope of the shear-stress–horizontal-displacement relationship at large displacements, allowing the horizontal portion of the curve to be clearly defined. It can be seen that a residual strength condition has been obtained.

Figure 10 illustrates the peak and residual undrained failure envelopes for Drammen clay using the constant volume ring shear and direct simple shear apparatuses. It can be seen from Fig. 10 that the peak strength at a consolidation stress of 95 kPa, which equals the estimated effective overburden stress, is slightly higher than the peak failure envelope. This is attributed to the preconsolidation pressure of 140 kPa exceeding 95 kPa. Thus, a good-quality undisturbed specimen was tested in the ring shear test.

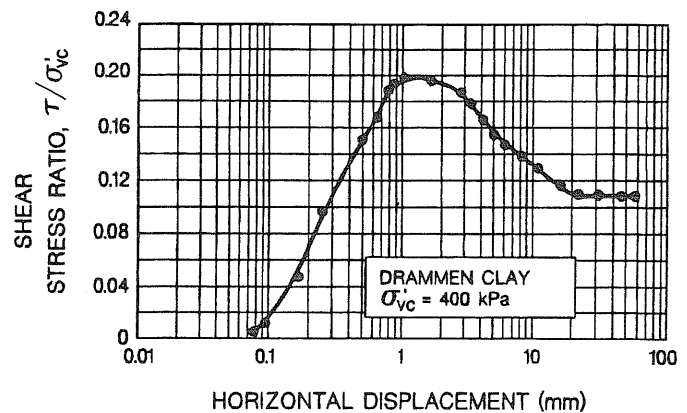


FIG. 9—Semilogarithmic presentation of constant volume ring shear test on undisturbed Drammen clay at 400 kPa.

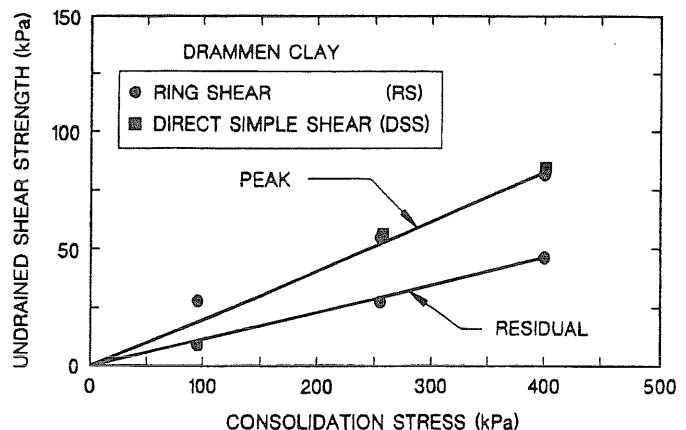


FIG. 10—Peak and residual failure envelopes for Drammen clay.

Effect of Nonuniform Displacement/Stress Condition on Ring Shear Tests Results

Hvorslev (1939, 1969) described a method to obtain the shear-stress–shear-displacement relationship for a ring shear test by graphical differentiation of the moment-twist (M - θ) relationship. In summary, the method involves the following steps: (a) the moment-twist relationship is determined from ring shear test data; (b) the relationship $F_1(\theta)$ is constructed using Eq 4 from the moment-twist relationship; (c) the term $\Delta F_1(\theta)$ is computed using Eq 5, where the term n is defined in Eq 6; (d) the values of the $F_1(\theta)$ and $\Delta F_1(\theta)$ relationships are added to obtain the $F_2(\theta)$ relationship in Eq 7; and (e) from the $F_2(\theta)$ relationship, the shear stresses, τ_1 and τ_2 , are computed using Eqs 8 and 9, respectively.

$$F_1(\theta) = M + \frac{\theta}{3} \frac{dM}{d\theta} \quad (4)$$

$$\Delta F_1(\theta) = n^3 F_1(n\theta) + n^6 F_1(n^2\theta) + n^9 F_1(n^3\theta) \quad (5)$$

$$n = \frac{R_1}{R_2} \quad (6)$$

$$F_2(\theta) = F_1(\theta) + \Delta F_1(\theta) \quad (7)$$

$$\tau_1 = \frac{3}{2\pi R_2^3} F_2(n\theta) \quad (8)$$

$$\tau_2 = \frac{3}{2\pi R_2^3} F_2(\theta) \quad (9)$$

It can be seen from Eqs 8 and 9 that the only difference in τ_1 and τ_2 is the inclusion of n . Therefore, as n increases to unity, the values of τ_1 and τ_2 converge, and thus each is adequately represented by the average shear stress, τ_a . The average shear stress is calculated using Eq 1. For the apparatus described by Hvorslev (1939), the value of n is equal to 0.5. Hvorslev (1939) found that the difference between τ_1 , τ_2 , and τ_a is not larger than 3% for the peak shear stress. The Bromhead ring shear apparatus possesses a value of n equal to 0.7. This results in values of τ_1 and τ_2 that are in excellent agreement with τ_a as shown in Fig. 11. Figure 11 illustrates the results of an analysis of the constant volume test results in Fig. 7 following the outlined methodology. It can be seen from Fig. 11 that the difference between τ_a and τ_2 at the peak shear stress is less than 0.25%. Therefore, the use of Eq 1 results in an adequate determination of the peak shear stress. At

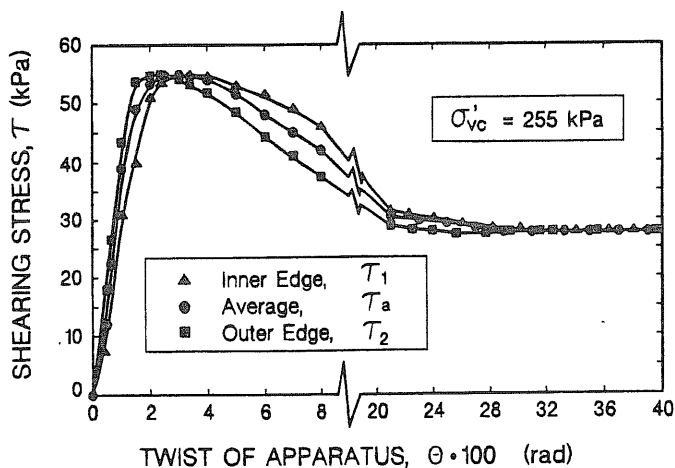


FIG. 11—Effect of nonuniform stress on undisturbed Drammen clay.

the residual condition, the values of τ_1 , τ_2 , and τ_a are identical, and thus the difference in shear displacement between the inner and outer radii is irrelevant.

Conclusions

A constant volume ring shear apparatus has been developed to measure the undrained peak and residual shear strengths of cohesive soils. The undrained peak and residual strengths are applicable to the seismic design of slopes comprised of or founded on cohesive soils. The results of constant volume ring shear tests on normally consolidated Drammen clay are presented and compared to the results of direct simple shear tests. The comparison shows that the undrained peak shear strength obtained by the two apparatuses is in agreement. Shear displacement at the peak shear stress is slightly larger in the ring shear apparatus. This difference in shear displacement is attributed to the nonuniform shear displacement across the ring shear specimen. The constant volume ring shear apparatus allows the measurement of the undrained residual strength, while the direct simple shear and triaxial apparatuses do not allow a sufficient amount of continuous displacement to achieve a residual strength condition. The undrained residual strength is significantly less than the undrained peak strength. This large post-peak strength loss has important implications to the seismic stability of slopes.

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