Confined Compression Test for Soils

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The ratio of lateral to vertical stress at zero lateral strain (K_o) is a soil characteristic that is important in several geotechnical applications. At present, no unanimity exists as to how K_o should be calculated or how it is influenced by stress history. The uncertainty regarding this soil characteristic may be reduced through improved measurements. At present, triaxial testing and confined compression tests are used. Triaxial testing is cumbersome and limited to certain soil types. Confined compression tests ignore the presence of wall friction and thus may introduce unknown factors in the measurement. A new confined compression tester is described that measures wall shear stresses along with vertical and horizontal stresses. Confined compression tests on Ottawa sand, alluvial sand, crushed limestone, and coal and comparisons with K_o tests conducted in triaxial apparatus on replicate specimens of the two materials are included. The new test device shows promise and should lead to improved methods for evaluating K_o in soils and provide a tool for achieving an understanding of how stress history influences this important soil characteristic.

The "at rest" lateral stress ratio for soils K_o is used in retaining wall and deep foundation analyses and estimates of the loads on buried pipes, and has been shown to influence the shearing resistance of soils. K_o also affects the bearing capacity and settlement of shallow foundations, but it is seldom used. Some uncertainty exists about how to estimate or measure this soil characteristic, and the uncertainty is greatest in very loose soils, compacted fills, and overconsolidated sediments and residual soils.

Several analytical methods use the soil shear strength as the key parameter to estimate or calculate K_o and involve a measurement followed by a calculation based on assumptions. The application of K_o estimated in this manner carries with it the indeterminacy inherent with both measurement inaccuracies and analytical assumptions. The measurement of lateral stresses for zero lateral strain is accomplished by both in situ and laboratory methods. In general, interpretation of field tests is difficult because of poorly defined boundary conditions and uncertain drainage conditions. More specifically, individual in situ K_o testers are limited to a fairly narrow range of soil types. Laboratory tests suffer from sample disturbance for any measurement, and K_o tests are limited to cohesive soils and mechanical overconsolidation (*1*).

PREVIOUS STUDIES

One early application of lateral stress ratios in particulate materials is Janssen's equation (*2*) used to calculate the vertical and horizontal stresses that ensiled bulk solids exert on container bottoms and sides. This equation includes a lateral stress ratio k. Janssen explicitly stated that k should be measured for each material to be stored. Probably because of problems associated with measuring k, Rankine's (*3*) active case, K_a, was soon employed in the Janssen equation (*4*). The use of K_a has persisted into recent structural codes for silos and was employed by Marston (*6*) when he adapted Janssen's equation to calculate the loads on buried pipes. The use of K_a is inappropriate because the Janssen/Marston equations were developed on the premise that vertical loads are reduced by the shear that occurs on the vertical walls of the silo or on the sides of the ditch and that Rankine's K_a is for the ratio of minor to major principal stresses. The Rankine expression is also for failure stresses, and there is some question as to whether this is appropriate for static loads at no lateral strain.

Jaky (*7,8*) also was interested in the ratio of principal stresses for the zero lateral strain case as they applied to ensiled bulk solids and developed a theoretical equation to predict K_o from the friction angle ϕ of the material. He later showed that this equation applied to soils and retaining walls and simplified the equation to the famous form:

$$K_o = 1 - \sin\phi$$

This equation is attractive for its simplicity and is used widely for granular soils. However, the equation is flawed by a dependency on a failure parameter. Actually K_o is dependent on deformation (*9*) and not on failure.

Hendron (*10*) used a specially instrumented oedometer to measure the lateral stresses during loading and unloading of sands. His data show that for most sands during loading Jaky's equation gives reasonable estimates of K_o. During unloading the horizontal stress increased and even exceeded the vertical stress. Subsequent work with the oedometer on clays (*11*) indicated that K_o of normally consolidated clays could be estimated from an equation similar to the Jaky equation with the 1 replaced by 0.95 and that K_o of overconsolidated clays varied with overconsolidation ratio (OCR) and plasticity index of clay. Schmidt (*12*) suggested an empirical exponential relationship between K_o and OCR. This oedometer had a small height-to-diameter ratio to minimize wall friction.

A specially designed oedometer was used to establish a relationship between K_o, liquid limit, and OCR for clays (*13*). This oedometer had a height of 3.5 in. and a diameter of 7 in. to ". . . accommodate a large enough sample with minimum sidewall friction."

Mayne and Kulhawy (*14*) conducted a statistical study of 170 soils to empirically predict K_o from ϕ and OCR and concluded that the Jaky equation is valid for normally consolidated clays and "moderately valid" for normally consolidated sands. During unloading K_o is approximately dependent

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on ϕ, and, when reloaded, horizontal stresses can be estimated from ϕ, OCR, and the maximum OCR. All of those relationships are summarized in one equation.

Feda (*9*) used zero lateral strain triaxial tests to measure K_o of sands and concluded that the Jaky equation applied only to dense sands and that the stress-dilatancy theory (Rowe) could be used to predict K_o for normally consolidated sands. For overconsolidated sands, an exponential relationship between K_o and OCR exists up to a maximum K_o, approaching Rankine's passive stress ratio K_p.

Studies of agricultural grain in zero lateral strain triaxial tests indicated that measured values of K_o were slightly lower than those predicted by the Jaky equation (*15*). This study, intended for application to calculating static lateral stresses in silos, did not include unloading and overconsolidation effects.

The preceding brief discussion illustrates that the Jaky equation may have limited application for estimating K_o. However, recent studies use the equation for calculating stresses in compacted fills (*16*) and for evaluating the effects of anisotropic consolidation on soil shear strength (*17*). No unanimity may exist concerning how K_o should be calculated, and this, in part, may result from the various methods used to measure the lateral stresses in soils and other particulate materials. K_o is an important parameter, and an improved test method may contribute useful information.

CURRENT LABORATORY TESTS

In some K_o triaxial tests, lateral strains are monitored with a circumferential strain gauge (*9*), and it is assumed that the strains throughout the height of the test specimen are uniform. A second approach is to continuously monitor volume change and axial strain and continuously adjust confining stresses to achieve zero lateral strain, which would require computer-controlled servomechanisms for effective data collection. A third approach is to use a piston entering the cell that has the same diameter as the test specimen, and, by monitoring cell volume and by maintaining the volume at a constant value, average zero lateral strain will be maintained as the test specimen shortens (*18,15*). This last approach seems the most effective, but all triaxial tests are difficult to conduct on soft soils and are awkward for obtaining reliable data during unloading.

The oedometer test allows loading and reloading cycles but has the disadvantage of unknown shear stresses acting on the sides of the test specimen. In conventional equipment (*10,11*) or special apparatus (*13,19*), those stresses are always assumed to be zero. This is highly unlikely, even in equipment where the specimen size is thin to minimize shear. It is the opinion of the authors that thin specimens may be subjected to additional measurement errors because of end effects, and some evidence exists to support this opinion (*1*).

DESCRIPTION OF THE CONFINED COMPRESSION TESTER

The confined compression apparatus used in this work is illustrated in Figure 1. The thin-walled circular cylinder is made of acrylic and rests on an acrylic platform. Acrylic was originally chosen because of its low stiffness relative to metals and to assure that the strains in the cylinder would be large enough to measure accurately. Acrylic is clear, and the sample can be viewed during test set up and during deformation. In applications where variable lateral constraint is of interest, different cylinder diameters, wall thicknesses, and cylinder material may be used. Except for frictional considerations between the platform and cylinder, the cylinder is free to expand in the radial direction. The radial expansion is quite small, as is noted later. The vertical load is applied to the bulk solid by a piston located at the top of the cylinder.

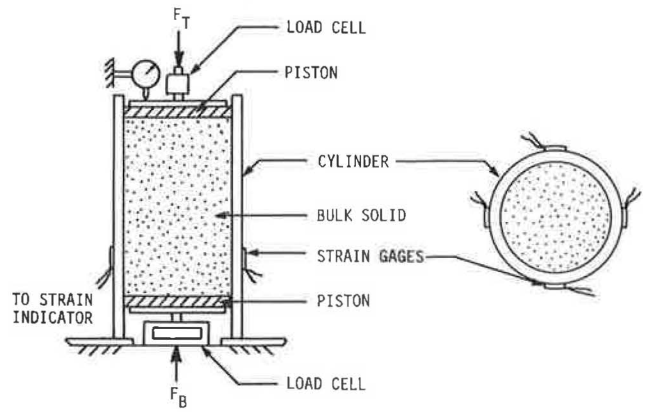

FIGURE 1 Confined compression test apparatus.

The piston is made of Delrin, which has a low coefficient of friction and also has good strength properties. The piston is backed by an aluminum stiffening plate. The piston is made slightly undersized, 2 mm on a 145-mm diameter, to keep the piston from transferring load directly to the cylinder. The load F_T at the top piston is monitored by a commercial load cell with a load capacity of 4448 N (1,000 lb). The load is supplied by a hand-operated pump and hydraulic ram. A dead load system, a lever arm and a fixed weight, could also be used in place of the hydraulic ram. The displacement of the upper piston is monitored by a dial gauge capable of measuring increments of 0.0254 mm (0.001 in.). A linear differential transformer can also be used to measure the piston movement.

At the bottom of the cylinder, another Delrin piston measures the force F_B, which is the force transferred to the soil in the axial direction. The force is monitored by a load cell of the authors' own design. The strain gauge-based load cell has a capacity of 1330 N (300 lb) and a sensitivity of 2.79 μ/N (12.42 μ/lb), where μ is in micrometers per meter of strain or microinches per inch of strain. The sensitivity is also the calibration factor of the load cell. Owing to the deflection of the load cell, the lower piston moves only very slightly when compared with the motion of the upper piston, and, it can be neglected. Again, the piston is undersized to keep the shear transmitted to the cylinder by the piston at a negligible level.

Four rectangular strain gauge rosettes are located at 90-degree increments around the cylinder at distance h_g below the top surface of the soil. Each pair of gauges measures the axial and hoop strains of the cylinder. The four axial strains are averaged to give the axial strain in the cylinder and likewise for the hoop (circumferential) strains. Because heat dissipation is a problem, 350-ohm gauges were used. Thermal drift is insignificant for the amount of time required to take the strain readings. A switch and balance unit is used along with a constant voltage strain indicator. The bridge voltage is 2 V. The 350-ohm gauges have a gauge length of 6.35 mm (0.25 in.) and a gauge area of 46.8 mm² (0.0725 in²).

The soil vertical strain ε_v is determined by dividing the vertical movement of the upper piston by the original height h of the soil specimen. A set of tests on a granular material indicated that h/D ratios did not effect the results. This assumes that the axial strain is uniformly distributed throughout the depth of the material. The horizontal or lateral stress in the soil, σ_H, is determined by realizing that σ_H acts as an internal pressure on the thin wall cylinder. By using the thin wall pressure vessel formula to relate the internal pressure to the hoop stress σ_θ in the cylinder, plus, by incorporating the plane stress/stress-strain relationship, σ_H can be determined as follows:

$$\sigma_H = \frac{2\sigma_\theta t}{D} = \frac{2tE}{D(1 - \nu^2)}(\varepsilon_\theta + \nu\varepsilon_a) \tag{1}$$

where t, D, E, and ν are the thickness, diameter, Young's modulus, and Poisson's ratio, respectively, of the cylinder. Here, ε_θ and ε_a are the average hoop and axial strains of the cylinder as was measured by the strain gauges.

E and ν of the acrylic 145-mm (5.72-in.) diameter cylinder are needed, so a 41-mm × 300-mm longitudinal slice of material was taken from the cylinder to be used as a tensile specimen. The thickness of the cylinder wall and slice was 3.33 mm (0.131 in.). Two strain gauges were used to measure the longitudinal and lateral strains of the tensile specimen made from the longitudinal slice. The stress-strain diagram was nearly linear to 400 μ. All of the tests reported here had strains less than this value. E was found to be approximately 2.2 GPa (320 ksi). Poisson's ratio over the same range was 0.38.

A check on the σ_H values obtained from Equation (1) is desirable. However, no direct check is possible. A secondary check that involves the same strains and material properties is developed next. The axial stress in the cylinder at the gauge location is

$$\sigma_a = \frac{E}{(1 - \nu^2)}(\varepsilon_a + \nu\varepsilon_\theta) \tag{2}$$

The same axial stress will be estimated by using the forces measured by the pistons. By ignoring the weight of the bulk solid, an approach similar to Janssen's (*2*) can be used to determine the axial stress. The assumption that the weight effects are negligible is good for the apparatus illustrated in Figure 1. Referring to the free body diagram, Figure 2, that illustrates the vertical loads acting on the soil, the following equation holds:

$$dF = \pi D\tau\, dx \tag{3}$$

Then, assuming that the shearing stress is related to the horizontal stress by a constant coefficient of friction,

$$\tau = \mu\sigma_H \tag{4}$$

To relate the horizontal stress to the vertical stress σ_V, it is assumed that the stress ratio constant is

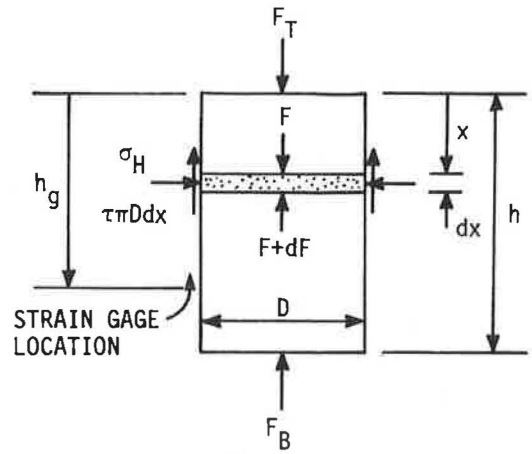

FIGURE 2 Soil free body diagram.

$$\sigma_H = k\sigma_V \tag{5}$$

Combining Equations 3, 4, and 5,

$$\frac{dF}{dx} = \mu k\pi D\sigma_V = \frac{4\mu kF(x)}{D} \tag{6}$$

where

$$\sigma_V = 4F(x)/\pi D^2 \tag{7}$$

The solution to this differential equation is of the form

$$F(x) = C_1 e^{-(\mu k/D)x} \tag{8}$$

Applying the boundary conditions

$$F(0) = F_T \tag{9}$$

$$F(h) = F_B \tag{10}$$

yields

$$F(x) = F_T(F_B/F_T)^{x/h} \tag{11}$$

and

$$\mu k = (D/4h)\ln(F_T/F_B) \tag{12}$$

The force in the wall of the cylinder at the gauge location is equal to the force at the top piston minus the force in the granular material given by Equation 11. This gives a wall stress of

$$\sigma_a = \frac{F_T}{\pi Dt}\left[1 - (F_B/F_T)^{h_g/h}\right] \tag{13}$$

Combining Equations 7 and 11 gives the average vertical stress in soil

$$\sigma_v = \frac{4}{\pi D^2}\left[F_T\left(\frac{F_B}{F_T}\right)^{x/h}\right] \tag{14}$$

PRELIMINARY TEST RESULTS

The confined compression tester was conceived to study the effects of wall roughness and stiffness on the stress distribution of ensiled particulate materials (*20,21*). Later it was recognized that the equipment has the potential to give new insights into factors that influence K_o of a wide variety of soils, especially those difficult to characterize in terms of lateral stress ratios.

Confined compression tests were conducted on Ottawa sand, an alluvial sand, crushed limestone, and coal. Hoop strains were observed to be about 0.00022 mm/mm. Because the strains were so small, the results of the confined compression tests were compared with those of K_o triaxial tests. The wall friction coefficients were compared also with direct shear tests of the Ottawa sand and coal on the same acrylic used for the confined compression tester. The wall friction coefficients agree within 0.02 and help substantiate the assumption of Equation 4. Jaky's K_o was computed from friction angles measured in triaxial tests. The confined compression tests included loading, unloading, and reloading tests. The data for those four materials are summarized in Table 1. The values for k are the ratios of horizontal-to-vertical stress as was calculated from Equations 1 and 14. The values for K_o are the ratios of minor principal stress to major principal stress calculated from the horizontal, vertical, and wall shear stresses and the equations

$$\frac{\sigma_1 + \sigma_3}{2} = \frac{\sigma_V + \sigma_H}{2}$$

$$\frac{\sigma_1 - \sigma_3}{2} = \left[\left(\frac{\sigma_V - \sigma_H}{2}\right)^2 + \tau^2\right]^{1/2} \qquad (15)$$

where σ_1 and σ_3 are major and minor principal stresses, respectively; σ_V and σ_H are the vertical and horizontal stresses, respectively; and τ is the shear stress at the wall.

The wall friction μ' was calculated from Equation 12. The confined compression test results indicate that the materials tested have a wide range of K_o values and that those values roughly correlate with the K_o as measured in triaxial tests. However, confined K_o are consistently lower than triaxial K_o and, in the case of the Ottawa sand and crushed limestone approach, Rankine's active stress ratio. The behavior of the sand is consistent with Feda's (*9*) conclusion.

The variation between the triaxial and confined compression K_o values is the basis for continued study. Although it can be argued that the small lateral strains are sufficient to mobilize active case conditions, it is important to question the reliability of both the horizontal and the vertical stresses as determined in the confined compression test.

The horizontal stress is calculated from Equation 1 by using strains measured directly on the container. An internal check on the horizontal stress calculation can be found by calculating the axial wall stress from both Equations 2 and 13. Equation 2 uses the same strains used in Equation 1, whereas Equation 13 uses the forces at the top and bottom of the container. Both equations, using independent data, give the same result and verify the strain measurements and demonstrate that the horizontal stress computation is reasonable.

The main concern, then, is the computation of vertical stress. Equation 8 assumes that the vertical stress is uniform across the diameter of the specimen. Axial symmetry and the wall shear stress require that the vertical stresses vary from a maximum along the central axis of the specimen to a minimum at the wall of the container. Shear stresses acting on the vertical and horizontal planes are maximum at the wall of the container and decrease to zero at the center of the test specimen where the vertical and horizontal stresses are major and minor principal stresses, respectively. This is a fundamental limitation of the Janssen/Marston equation, and several analytical solutions have been suggested (*22–24*). All of those solutions use assumptions on the shape of the vertical stress distribution curve to solve the problem and, as such, are inadequate to validate or modify this measuring technique. The only way to determine the vertical stress distribution across the diameter of the soil specimen confidently is to measure it, and the technique for this difficult task is under development.

Figures 3, 4, and 5, illustrating Ottawa sand data, provide evidence of the potential of the confined compression tester. Similar results were obtained from other materials. Figure 3 is a plot of vertical stress versus horizontal stress for three load/unload cycles. The virgin loading cycle has a linear slope k from which K_o can be calculated. Unloading curves indicate that horizontal stresses remain high during unloading, and, as vertical stresses approach zero, horizontal stresses are higher than vertical stresses. Residual horizontal stresses occur at zero vertical stress. For reloading, k is lower, as would be expected for a densified specimen. Figure 4 is a plot of horizontal stress versus shear stress where the slope of the curve is μ'. As was expected, the loading and reloading curves are quite linear and nearly parallel. Finally, Figure 5 presents stress-strain curves with strain hardening that is typical of confined compression tests.

TABLE 1 RESULTS OF PRELIMINARY CONFINED COMPRESSION TESTS IN COMPARISON WITH TRIAXIAL AND DIRECT SHEAR TESTS ON THE SAME OR SIMILAR MATERIALS

	CONFINED COMPRESSION			C.C.	TRIAX	THEORY	
MATERIAL	k	K_0	μ'	K_0	K_0	K_0	K_A
OTTAWA SD	0.36	0.32	0.42	0.32	0.36	0.47	0.31
ALLUVIAL SD	0.34	0.29	0.47	0.29	0.30	0.42	0.26
CRUSHED LS	0.31	0.27	0.48	0.27	0.44	0.30	0.25
COAL	0.35	0.32	0.34	0.32	0.33	0.33	0.18

CONCLUSIONS

Confined compression tests with the new test device can be conducted in 20 min when compared with triaxial tests that require as much as 2 to 3 hr. The device should accommodate a wide range of soils and allows study of the effects of overconsolidation ratios and time on K_o.

Preliminary confined compression tests on sands, crushed limestone, and coal produce stress-strain curves that exhibit strain hardening, as was expected, and the measured value of k is linear throughout virgin loading. The horizontal stresses

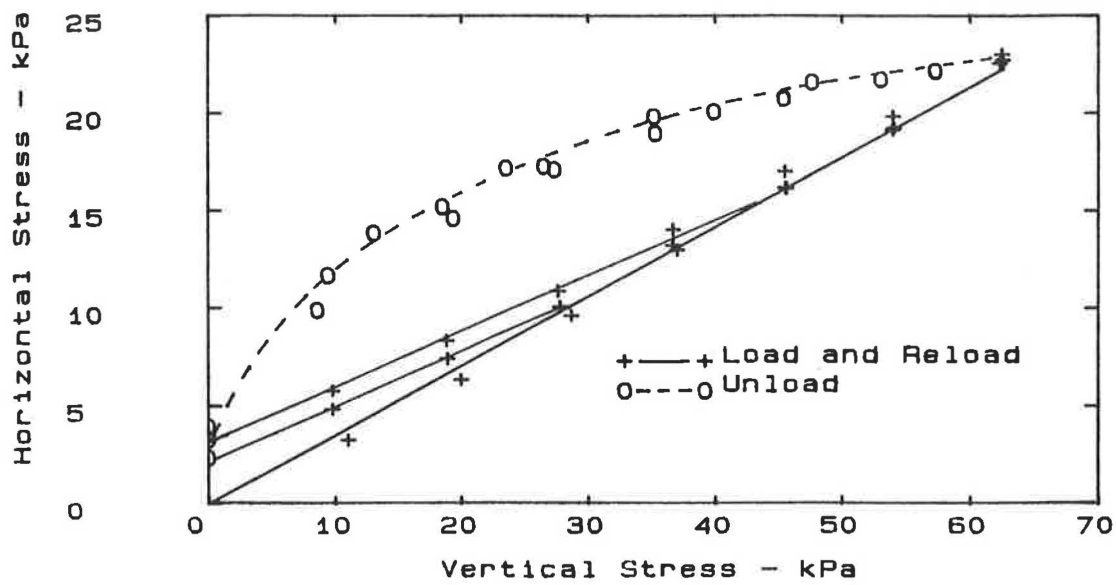

FIGURE 3 Horizontal and vertical stress curve.

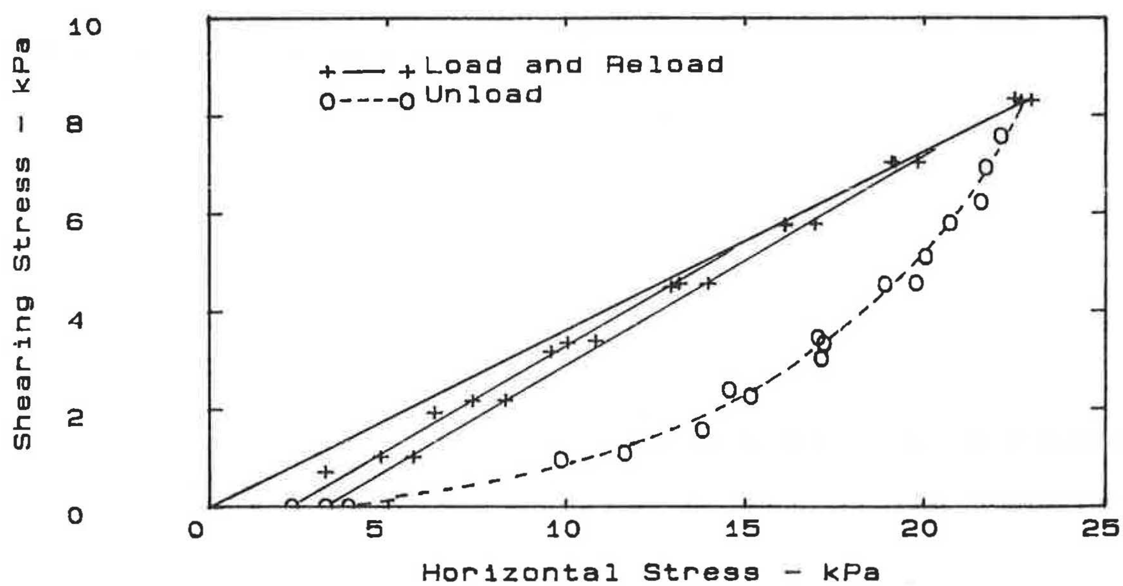

FIGURE 4 Shearing stress and horizontal stress curve.

remain higher at equivalent vertical stresses upon unloading, resulting in variable k values that approach Rankine's K_p as the vertical stresses approach zero. Residual horizontal stresses remain at zero vertical stress. The reloading curves are linear but have lower k values than the virgin loading. Wall friction coefficients determined in this confined compression test compare favorably with those measured by direct shear tests. Those data show considerable promise for the test device. However, when the confined compression results are compared with triaxial test results, the interpretation is not so straightforward.

This study illustrates the complexity of obtaining accurate K_o measurements and suggests that some previous K_o test results may have been somewhat naively interpreted and may have contributed to some of the confusion regarding accurate values of K_o. Continued development of this device appears

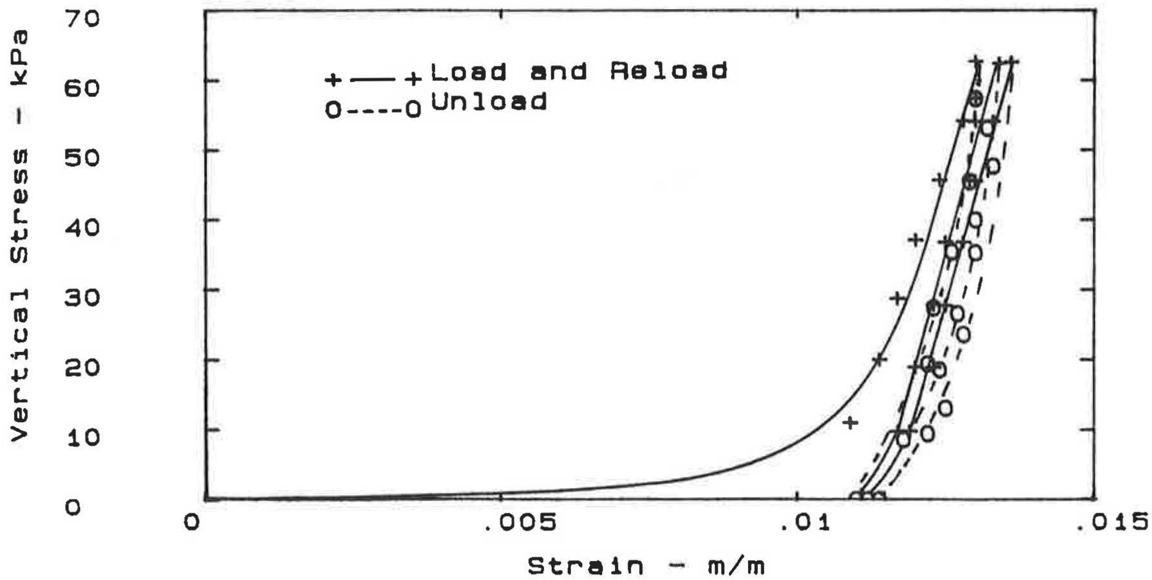

FIGURE 5 Confined stress-strain curve.

appropriate and necessary. The application of this device to saturated clays can be achieved by incorporating pore pressure transducers into the walls of the cylinder.

REFERENCES

1. M. Jamiolkowski et al. New Developments in Field and Laboratory Testing of Soils. *Proc., 11th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, San Francisco, Calif. 1985, pp. 57–153.
2. H. A. Janssen. "Pressure in Silos." *Proceedings of the Institute of Civil Engineers*, London, 1896, p. 553.
3. W. J. M. Rankine. On the Stability of Loose Earth. *Transactions Philosophical Royal Society*, London, Vol. 147, 1857.
4. M. Koenen. Berechnung des Seiten und Bodendruckes in Silozellen. *Zentrablatt Baurerwaltung*, Vol. 5, 1896, p. 446.
5. *Recommended Practice for Design and Construction of Concrete Bins, Silos, and Bunkers for Storing Granular Materials*, American Concrete Institute, 1977.
6. A. Marston. Theory of External Loads on Closed Conduits in the Light of the Latest Experiments. *Bulletin 96 Iowa Engineering Experiment Station*, Ames, Iowa, 1930.
7. J. Jaky. A Nyugalmi Nyomas Tenyezoje. *Magyar Mern. Epitesz Egylet Kozlemenyi*, Oct. 1944, p. 355.
8. J. Jaky. Pressures in Silos. *Proc., 2nd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, Rotterdam, 1948, pp. 103–107.
9. J. Feda. K_o Coefficient of Sand in Triaxial Apparatus. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 110, No. 1, 1984, pp. 519–524.
10. A. J. Hendron, Jr. *The Behavior of Sand in One-Dimensional Compression*. Ph.D. dissertation. University of Illinois, Urbana, 1963.
11. E. W. Brooker. and H. A. Ireland. Earth Pressures at Rest Related to Stress History. *Canadian Geotechnical Journal*, Vol. 2, No. 1, 1965, pp. 1–15.
12. B. Schmidt. Earth Pressures at Rest Related to Stress History. (discussion), *Canadian Geotechnical Journal*, Vol. 3, No. 4, 1966, pp. 239–242.
13. M. Sherif and D. E. Koch. Coefficient of Earth Pressure at Rest as Related to Soil Precompression Ratio and Liquid Limit. In *Highway Research Record 323*, HRB, National Research Council, Washington, D.C., 1970, pp. 39–48.
14. P. W. Mayne and F. H. Kulhawy. K_o-OCR Relationships in Soil. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 108, No. 6, 1982, pp. 851–872.
15. R. A. Lohnes, J. C. Liu, and W. H. Bokhoven. Experimental Determination of K_o in Grain. *Proc., 10th Powder and Bulk Solids Conference*, 1985, pp. 231–237.
16. J. M. Duncan and R. B. Seed. Compaction-Induced Earth Pressures Under K_o Conditions. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 112, No. 1, 1986, pp. 1–22.
17. N. Sivakugan, R. D. Holtz, and J. L. Chameau. CK_o UC Shear Strength of Normally Consolidated Soils from CIUC tests. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 114, No. 3, 1988, pp. 284–295.
18. R. G. Campanella and Y. P. Vaid. A Simple K_o Triaxial Cell. *Canadian Geotechnical Journal*, Vol. 9, No. 3, 1972, pp. 249–260.
19. A. G. Bishara, S. F. Ayonb, and A. S. Mahdy. Static Pressures in Concrete Circular Silos Storing Granular Materials. *American Concrete Institute Journal*, May/June 1983, pp. 210–216.
20. S. A. Masroor, L. W. Zachary, and R. A. Lohnes. A Test Apparatus for Determining Elastic Constants of Bulk Solids. *Proc., Spring Conference on Experimental Mechanics*, Society of Experimental Mechanics, Bethel, Conn., 1987, pp. 553–558.
21. L. W. Zachary and R. A. Lohnes. A Confined Compression Test for Bulk Solids. *Proc., 13th Annual Conference on Powder and Bulk Solids Handling*, Powder Advisory Centre, London, 1988, pp. 483–494.
22. D. M. Walker. An Approximate Theory for Pressures and Arching in Hoppers. *Chemical Engineering Science*, Vol. 21, 1967, pp. 975–997.
23. A. Livin. Analytical Evaluation of Pressures of Granular Materials on Silo Walls. *Powder Technology*, Vol. 4, 1970, pp. 280–285.
24. W. H. Bokhoven. *Stress-Strain Characteristics of Soybean Meal as Related to Flowability and Pressures of Bulk-Solids*. M.S. thesis. Iowa State University, Ames, 1988.

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