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New Method for Determining the Coefficient of Earth Pressure at Rest, K*₀*

Keisuke Adachi, Caren Ackley, Bruce Polan,
Michael Braverman
GHD Limited, Waterloo, Ontario, Canada

ABSTRACT
Vertical and lateral earth pressures must be considered in the design of underground structures, including tunnels, retaining walls, and deep excavations. Vertical pressure can be determined by in situ measurement, or calculated if the depth and material characteristics of the overburden are known. Lateral earth pressure is more challenging to measure accurately in situ. Underestimating lateral earth pressures may result in unanticipated soil movements and wall collapse, therefore knowledge of the lateral earth pressure, is vital for sound infrastructure design and long-term integrity. Lateral pressures are typically estimated using the coefficient K_0, defined by Terzaghi as the coefficient of earth pressure at rest. The accepted theoretical approaches to determine K_0 are based on the Rankine and Coulomb theories; however, both make assumptions and may not provide accurate K_0 values. There are several laboratory tests for K_0 determination, most of them requiring complicated and sometime cumbersome equipment. A new method for determining K_0 was developed in the GHD Geotechnical Laboratory. A series of drained K_0 tests were performed on saturated soil using modified triaxial equipment. This paper presents the experimental equipment and procedures, and discusses the laboratory K_0 measurements obtained.

RÉSUMÉ
Les pressions verticales et horizontales dans les sols doivent être considérées lors de la conception de structures souterraines telles les tunnels, les murs de soutènement et les excavations profondes. La pression verticale peut être définie soit par mesure directe, ou alors calculée à partir de la profondeur et des caractéristiques des sols, lorsque celles-ci ceux-ci sont connues. La pression horizontale est par contre difficile à mesurer avec précision. La sous-estimation des pressions latérales lors du dimensionnement de structures peut engendrer des déformations indésirables et éventuellement la rupture d'ouvrages de soutènement. La connaissance des pressions latérales est par conséquent primordiale pour un dimensionnement adéquat des structures et pour assurer leur pérennité. La pression latérale est généralement estimée à partir de la pression verticale en utilisant le coefficient des terres au repos K_0. Les approches théoriques acceptées pour estimer K_0 sont basées sur les théories de Rankine et Coulomb; cependant, les deux approches sont basées sur des hypothèses et ne donnent pas des valeurs précises de K_0. Il existe plusieurs essais de laboratoire connus pour la détermination de K_0 mais la plupart d'entre eux nécessitent des équipements compliqués et parfois fastidieux à opérer. Une nouvelle méthode de détermination de K_0 a été développée au Laboratoire géotechnique de GHD. Une série de tests K_0 drainés ont été effectués sur un sol saturé en utilisant un équipement triaxial modifié. Cet article décrit l'équipement expérimental et la procédure utilisés puis présente et discute les résultats obtenus.

1 INRODUCTION

Knowledge of vertical (σ'_V) and lateral (σ'_L) effective pressure is critical for the successful execution of deep excavations, and the design and construction of subsurface infrastructure such as tunnels, basements, and temporary or permanent retaining walls. The maximum horizontal pressure a soil can withstand while maintaining zero lateral strain, or no soil movement, is termed earth pressure at rest. Active earth pressure is the minimum lateral pressure required to increase lateral stress to the point of soil mobilization. Excavation of soil causes soil to release horizontal stress. When vertical (compressive) earth pressure overcomes the soil shear strength, the active state is mobilized. In order to determine active earth pressure, at which point the soil succumbs to shearing, knowledge of both vertical and lateral earth pressure at rest are required (Clayton et al. 2013).

Vertical effective stress (σ'_V) is dependent on the depth and mass of overburden, and in situ σ'_V can be easily measured or calculated. Lateral effective stress

(σ'_L) is also dependent on the depth and overburden mass, however additional factors influence σ'_L such as soil cohesion, friction and stress history, which are functions of various soil properties and conditions including soil type, structure, and the degree of disturbance and compaction. Common in situ field methods used for measuring lateral earth pressure require the installation of equipment, such as the dilatometer test, borehole pressuremeter test, or a lateral stress measuring earth pressure cell test (Watabe et al., 2003, Coyle and Bartoskewitz, 1977). However, each of these methods have limitations and shortcomings. During the process of dilatometer installation, some degree of horizontal compaction occurs that artificially increases lateral stresses. Similarly, the excavation of a volume of soil for equipment installation such as the borehole pressuremeter inevitably relieves some amount of pressure that increases lateral stress and artificially decreases σ'_L. Because accurate in situ measurements of σ'_L are albeit impossible, theoretical methods are commonly employed.

Conveniently, σ'_L can be obtained using σ'_V and K_0, where K_0 is defined by Terzaghi and Peck (1967) as the coefficient of earth pressure at rest.

$$K_0 = \sigma'_L/\sigma'_V \quad (1)$$

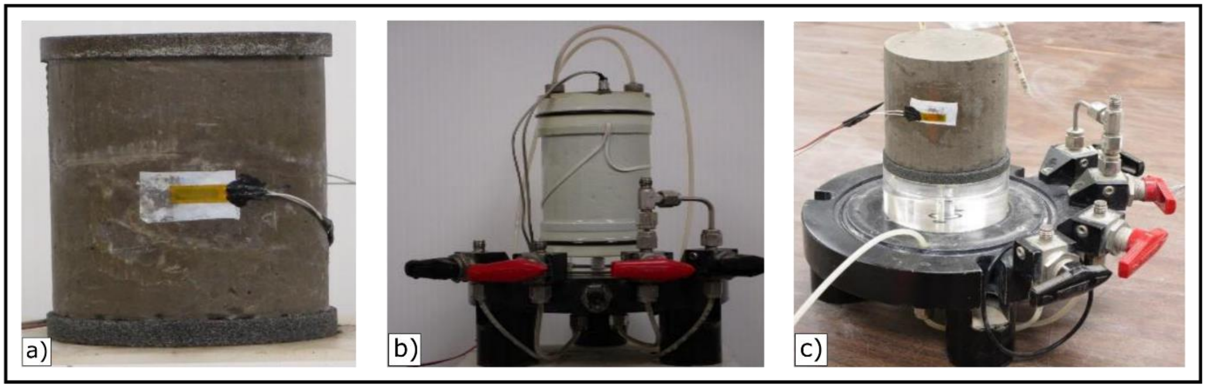
A theoretical relationship to determine K_0 can be calculated using Jaky's (1944) formula:

$$K_0 = 1 - \sin \emptyset' \quad (2)$$

which is based on Rankine theory (1857), and is dependent on the soil friction angle, $\emptyset'$. However, Jaky's formula is designed to provide a simplified calculation for an approximate value of K_0 and is generally used in absence of direct K_0 measurements, and like many theoretical methods, tends to generalize the K_0 value. Although this formula has been shown to be suitable for normally consolidated soils, it breaks down when applied to disturbed or over consolidated soils. To overcome this problem, derivatives of Jaky's formula have been used that also account for soil deformation, or degree of compaction (Alpan and Ice, 1967, Lee et al., 2013, Mayne and Kulhawy, 1982, Mesri and Hayat, 1993, Watabe et al., 2003). However, these formulas have been shown to be suitable for soils with an over consolidation ratio (OCR) of up to three (Alpan and Ice, 1967), after which the error increases.

Various laboratory methods have also been developed and employed for determining K_0 using triaxial cells and oedometers (Eliadoranu and Vaid, 2006, Goto et al., 1991, Hornig and Buchmaier, 2005, Isah et al., 2018, Laloui et al., 2006). The most common laboratory method is a special triaxial compression test that is based on direct measurements of σ'_1 – vertical, and σ'_3 – lateral effective stresses. During this test, a displacement transducer controls sample height. As the sample is compressed, a constant lateral strain condition of the soil sample is maintained via the consistent increase of the triaxial cell pressure. The change in sample volume is monitored either through the volume change of the fluid (usually de-aired water) within the triaxial cell, or through the volume of water expressed from the sample during sample compression (Laloui et al., 2006, Lo and Chu, 1991, Poulos and Davis, 1972, Tsuchida and Kikuchi, 1991, Wanatowski and Chu, 2007, Watabe et al., 2003, Guo, 2016). In both cases, the volume change of the sample is re-calculated into the change of the cross sectional area, or lateral strain, which should remain constant. Therefore, volume change is monitored throughout the test and used to calculate lateral strain.

However, these methods for determining lateral strain have notable drawbacks. Calibration of the water volume change within the triaxial cell versus cell pressure can be problematic. Inaccurate volume change measurements may result from various sources including errors in calibration, volume of the triaxial cell (larger cell imposes greater error), cell wall flex, water outlet tube deformation, membrane penetration and temperature variation (Laloui et al.' 2006, Eliadorani and Vaid, 1967, Isah et al., 2018). An example of a volume change calibration of a triaxial cell performed in the GHD Geotechnical Laboratory using a thick walled steel cell cylinder is presented in Figure 1. Comparison of the repeated volume change calibration using the measured volume change of water within the triaxial cell shows inconsistency between both the pressure increase and decrease stages for each run, as well as differences between runs. This suggests that the volume change of water within the triaxial cell cylinder is in part due to the increase of cell pressure. However, deviation of the calibration curves may be due to inconsistent compression/re-compression of the rubber O-rings, plastic tubes, and points of connection to the cell, as well as the dissolution of microscopic air bubbles that cannot be removed through standard water de-airing processes. Thus, the error in volume change measurements is unpredictable and can easily exceed the magnitude of the actual volume change of the sample during the test. In addition, the measurements of sample volume change based on the amount of water expressed from a soil sample can be misleading due to the compression of air present within voids if the sample is not completely saturated. Moreover, this method cannot be used in undrained K_0 tests where water is not expressed from the sample during consolidation, and there is not a direct relationship between volume change of the cell water and volume change of the soil sample. Regardless, the repeatability of this volume change calibration curve is quite low, rendering it useless for small strain measurements of soil samples.



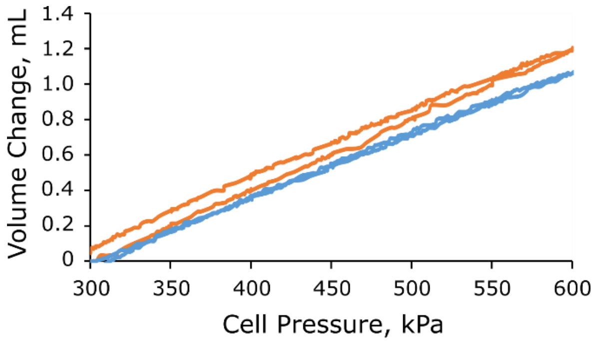


Figure 1. Measurements of volume change within a steel walled triaxial cell vs cell pressure during plunger insertion in two calibration cycles (red and blue lines) at cell pressure of in the range of 300-600 kPa.

These issues led GHD to research a more accurate method for laboratory determination of K_0. Alternate methods can be divided in two major categories; contact and non-contact (Laloui et al., 2006). Contact methods include the use of submersible mini linear variable displacement transformer (LVDT), local deformation transducer (LDT), and spring deformation gauge (SDG) placed in direct contact with the sample (Isah *et al*., 2018, Ismail and Ibrahim, 2019, Goto et al., 1991).

Figure 2. Test setup configuration showing a) sample with attached Teflon tape and strain gauge before test, b) test sample with confining rubber membrane just removed triaxial cell while sample is under negative pressure and c) soil sample after test with Teflon tape and strain gauge still intact.

Examples of non-contact methods include proximity transducers and Hall Effect gauges (Hornig and Buchmaier, 2005). However, these methods have limitations due to the complexity in mounting test equipment and instrumentation to the soil sample with proper alignment, especially on soft and weak samples. In both methods, the signal receivers are not in direct contact with the sample. Nonetheless, these methods present the same problems as the contact methods because the control points for both methods must be mounted on the sample. Moreover, the cost of this equipment can make these methods prohibitive for use in standard commercial laboratories.

In order to overcome these difficulties and complications, an experimental procedure was developed in the GHD Geotechnical Laboratory to measure and control very small strains developed within the sample during triaxial consolidation. A series of tests were performed, in which K_0 was derived using this new testing technique utilizing common and accessible equipment found in most geotechnical laboratories.

2 METHODS AND MATERIALS

2.1 Volume Change

The proposed method requires a standard 120 ohm strain gauge that is glued to a segment of thin Teflon tape and mounted on a soil sample. The strain gauge, on average, can measure 1/10,000 micro strain. Compared to the previously discussed methods, this strain gauge method has several important advantages including:

- high sensitivity
- strong output signal
- high pressure range
- cost-effective

While the use of strain gauges for deformation measurements is common practice in geotechnical rock testing, this technology is not common in soil testing. Typically, strain deformations of soil are greater than the strain gauge measuring range. For this reason, common LVDTs are a much better and more convenient choice for displacement control. Even in tests where monitoring soil material behavior within very small strains is a primary objective, strain gauges are not commonly used due to the complications with application and contact bonding between the soil sample and the strain gauge. After numerous trial and error attempts to overcome this problem, the use of thin Teflon tape as a transitional layer between the soil sample and strain gauge (Figure 2a) was found to be a simple and effective solution.

Teflon tape was used as the transitional layer due to several important physical properties. Its high electrical resistivity prevents any electrical current leakage, and its low thermal resistivity allows heat to dissipate during the test. Teflon tape also exhibits low water absorption, and has a very high percent of tensile elongation at breakage. Moreover, curing of the instant glue used for attaching Teflon tape to the soil sample is facilitated by the presence of moisture, making it suitable for this use. It was found that a 25x10 mm strip of tape is the most responsive for 50 mm diameter soil sample. Very small changes in sample diameter, which are impossible to detect with standard LVDT, are detected using this configuration.

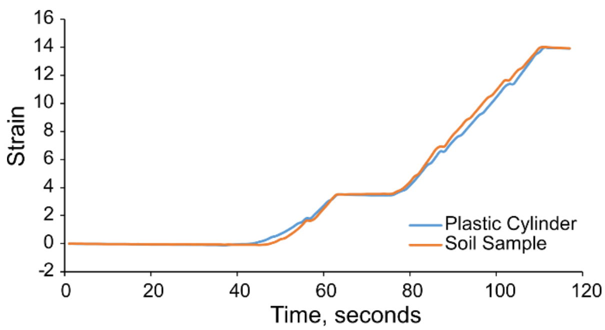


Figure 3. Timing response of the strain gauges mounted on the soil sample and the thick-walled plastic cylinder to the applied axial stress (amplification factor of 1.5 applied to the clay sample strain gauge readings). The presence of the latex membrane (Figure 2b) around the sample during a typical triaxial test plays a significant role in this K_0 test. During set-up, the Teflon tape is prone to detachment from the sample as the outer layer of the sample loses moisture. In addition, due to the low initial effective stress of 5 to 10 kPa at the start of the test, the tape may become separated from the sample. Therefore, the latex membrane confines the sample and the attached strain gauge, and prevents strain gauge detachment during all stages of this test (Figure 2c).

The response of the strain gauge to axial strain, when mounted to the sample using our technique, is shown in Figure 3. To determine the response sensitivity of the strain gauge to loading using this technique, a thick-walled plastic cylinder of 75 mm inner diameter was placed over a cylindrical soil sample of 50 mm diameter with the same height. The outer cylinder and inner sample were loaded simultaneously, and the strain of each was measured using a strain gauge attached as described above.

The strain of the soil sample was somewhat smaller due to the end effect caused by insufficient drainage during the experiment, therefore an amplification factor of 1.5 was applied. As expected, the strain of the soft soil sample developed at the same rate as that of the plastic cylinder (Figure 3), and demonstrates an almost identical timing response for both sensors. The immediate response of the strain gauge to very small strains provides the advantage of calculating actual effective stress developed within the sample at any time during the test.

2.2 Sample Materials and Test Preparation

In the course of this study, a drained triaxial K_0 consolidation test with pore pressure measurements was performed for several different materials following Head (1986). The results for two types of soil are presented and discussed. The geotechnical index properties of these materials are shown in Table 1.

Soil samples were prepared (Figure 2a) from a soil core with a height to diameter ratio of approximately 1:1. In order to avoid uneven stress application, all samples were prepared with a slightly smaller diameter than the 50 mm triaxial pedestal. Two standard 120 ohm strain gauges were attached to the middle of the soil sample, on opposite sides, in the lateral direction using instant glue. Because of the small surface area of the strain gauge, a 25x10 mm strip of Teflon tape was used as a transitional layer between the sample and strain gauge to facilitate a more secure contact. Immediately after the instant glue cured, a latex membrane was mounted over the soil sample on the triaxial pedestal. The prepared sample with the strain gauge attachment and triaxial

Table 1. Physical properties of soil samples used for K_0 testing.

Sample Material	Dry Density (kg/m^3)	Moisture Content (%)	Void Ratio	Initial Degree of Saturation (%)
sandy Silt till	1943	14.9	0.41	96
Clay	1836	18.7	0.49	100

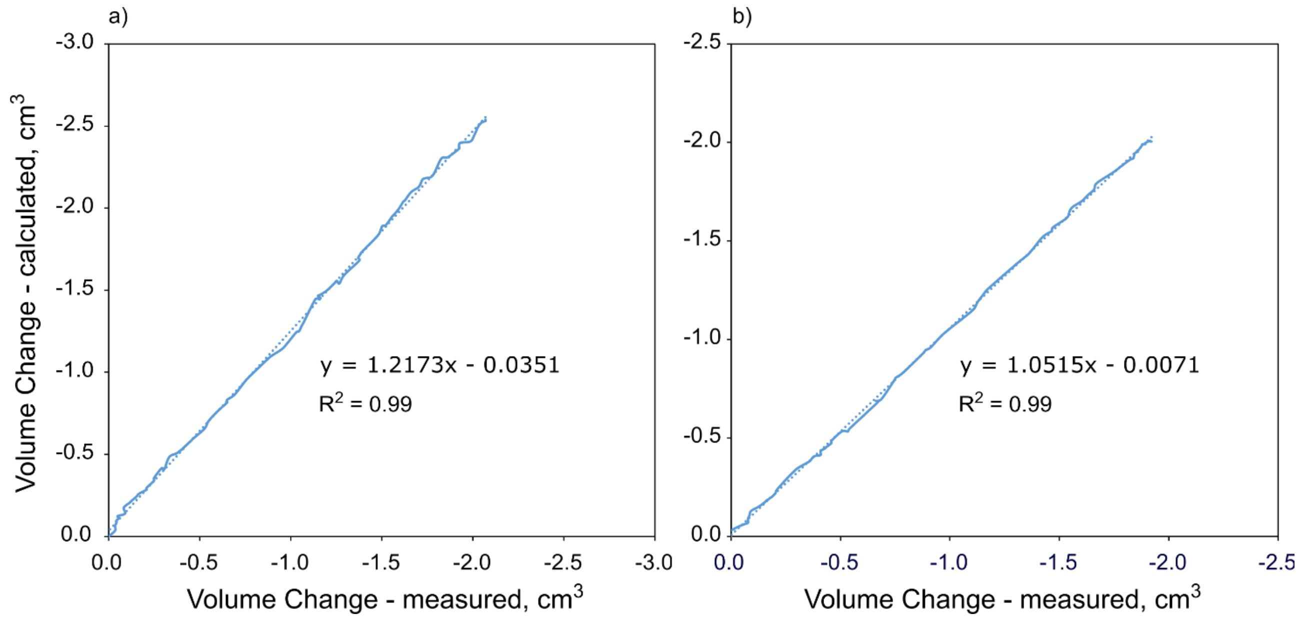


Figure 4. Comparison of volume change measured as the volume of water expressed from the sample, and calculated based on the sample height change in test controlled by strain gauge for a) sandy Silt till and b) Clay.

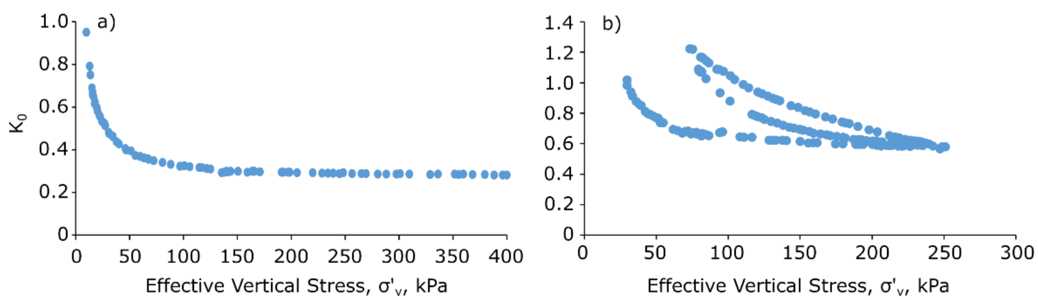


Figure 5. Relationship between K_0 and vertical effective stress for a) sandy Silt till and b) Clay. The Clay sample was subjected to one additional cycle of unloading-loading in order to evaluate K_0 values for the Over Consolidation Ratio (OCR) range.

pedestal set up is shown in Figure 3. The acrylic top plate was equipped with an extra opening for strain gauge wires with a seal around the wires to prevent water exchange between the triaxial cylinder and the sample within the latex membrane.

Samples were saturated according to ASTM D-7181; Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils, until a B-value of at least 0.95 was reached. Once saturated, the sample was subjected to a constant rate of loading. The loading rate was kept as low as possible to avoid rapid development of excess pore pressure within the sample, allowing pressure throughout the sample to equilibrate as the test progressed. Pore pressure that developed within the sample was measured at the bottom of the sample, and pore water drainage occurred through the porous stone on top of the sample. An increase in pore pressure indicates that water flowing from the sample is impeded by the soil structure, and calculations of sample diameter based on this volume change are prone to error and underestimation. Any excess of pore pressure at the bottom of the sample was recalculated according to Oda *et al*. (1992) where the empirical value that corresponds to the center of the sample is two-thirds of the pore pressure increase at the bottom of the sample. A constant rate of loading of 0.001 kN/min was applied for the Clay sample and 0.004 kN/min for the till sample.

Control of sample deformation (strain) during loading was based on the average readings of the two strain gauges that were mounted on opposite sides of the sample. When deformation was detected, confining pressure in the triaxial cell was increased manually by increments of 0.5 to 1.0 kPa to maintain zero lateral strain. As a backup control, the volumetric change of the sample was constantly recalculated according to the amount of water expressed from the pores during the test. The target maximum lateral strain limit was 0.05%, which is within the recommended range of the *Japanese Geotechnical Society (JGS) Standard 0525-2009 Method for K_0 Consolidated Undrained Triaxial Compression Test on Soils with Pore Water Pressure Measurements*. The sample load was terminated once pre-consolidation stress or overburden pressure was reached, whichever was greater, signifying that a constant value of K_0 was achieved. The Clay sample was subsequently unloaded and reloaded in order to evaluate K_0 values versus Over Consolidation Ratio (OCR).

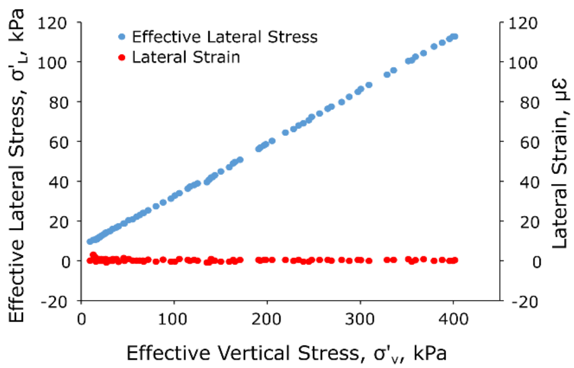


Figure 6. Typical relationship between lateral and vertical effective stress. The red line demonstrates lateral strain deviation from zero-value during K_0 consolidation.

3 RESULTS AND DISCUSSION

3.1 Volume Change and Pore Pressure Dissipation

The rate of strain for the drained triaxial test was low enough to permit pore water pressure dissipation throughout the sample. Under the low rate of strain, the pore pressure deviation in this test was within 10%. While the low rate of strain required for pore water dissipation is still preferable for this new test method, it becomes less imperative when using a strain gauge to detect strain. Figure 4 compares the volume of water expressed from the sample measured directly by the volume change transducer to the volume change that was calculated based on strain gauge-controlled displacement for each sample. The measured volume of water expressed from the sample is slightly less than that theoretically calculated based on height change for both soils. The difference can be attributed to deviations in sample saturation (Table 1).

Table 2a. Test results for the sandy Silt till sample. Rate of loading was 0.004 kN/min.

Cell Pressure (kPa)	Pore Pressure (kPa)	Effective Vertical Stress, σ'_v (kPa)	Effective Lateral Stress, σ'_L (kPa)	Specimen Diameter by Volume Change (cm)	% of Diameter Change by Volume Change Recorded	Specimen Diameter by Strain Gauge (cm)	% of Diameter Change by Strain Gauge	K_0 Value (-)
201.9	190.5	11.4	11.4	5.052	-	5.052	-	-
203.5	190.7	20.3	12.8	5.052	0.01	5.052	0.0000	0.63
207.8	190.8	35.5	17.0	5.053	0.02	5.052	0.0000	0.48
216.3	190.6	65.1	25.7	5.053	0.03	5.052	0.0000	0.40
220.8	190.7	78.5	30.1	5.054	0.04	5.052	0.0000	0.38
231.0	190.6	107.2	40.4	5.054	0.04	5.052	0.0000	0.38
238.6	190.8	130.6	47.8	5.054	0.05	5.052	0.0000	0.37
249.8	190.7	161.3	59.1	5.054	0.05	5.052	0.0001	0.37
280.3	191.0	225.9	89.3	5.055	0.07	5.052	0.0000	0.40
299.5	190.9	272.1	108.6	5.054	0.04	5.052	0.0000	0.40

Table 2b. Test results for the Clay sample. Rate of loading was 0.001 kN/min.

Cell Pressure (kPa)	Pore Pressure (kPa)	Effective Vertical Stress, σ'_v (kPa)	Effective Lateral Stress, σ'_L (kPa)	Specimen Diameter by Volume Change (cm)	% of Diameter Change by Volume Change Recorded	Specimen Diameter by Strain Gauge (cm)	% of Diameter Change by Strain Gauge	K_0 Value (-)
222.4	192.3	36.0	31.5	5.033	0.009	5.032	0.000	0.87
225.7	194.4	39.4	33.4	5.032	-0.004	5.031	-0.019	0.85
227.1	194.1	44.0	35.0	5.033	0.012	5.032	0.000	0.80
228.4	194.4	45.6	36.1	5.033	0.019	5.032	0.000	0.79
229.8	194.7	47.8	37.3	5.033	0.012	5.033	0.019	0.78
231.2	194.4	52.9	38.9	5.033	0.007	5.032	0.000	0.74
241.6	200.1	68.0	45.5	5.033	0.021	5.033	0.019	0.67
253.3	202.2	84.8	55.8	5.032	0.000	5.034	0.039	0.66
253.9	202.1	86.5	56.5	5.032	0.003	5.034	0.039	0.65
272.9	203.4	116.5	74.6	5.036	0.072	5.034	0.039	0.64

3.2 K_0 Test Results

The results of the strain gauge controlled K_0 consolidation test are presented in Figure 5, showing the relationship between K_0 and vertical effective stress. Test results for the sandy Silt till and Clay are presented in Table 2a and Table 2b, respectively. In order to evaluate this method for comparing K_0 vs OCR, the Clay sample (Figure 5b) was tested with one extra loading/unloading cycle. A maximum OCR value of 3.5 was reached in this test. In the re-loading portion of the curve, values of K_0 higher than 1.0 can be attributed to the rebound effect during unloading. Figure 6 presents typical relationship between lateral and vertical effective stress where the red line demonstrates lateral strain deviation from zero-value during K_0 consolidation.

The softer clay sample exhibited higher K_0 values compared to the denser sandy Silt till, which agrees with the logic of the Jaky's formula where $K_0 = 1 - \sin\emptyset'$. The strain gauge measurements were in good correlation with strain calculated according to sample volume change using water volume expressed from the sample. Due to the high sensitivity of this approach, the response of the sensor to lateral strain was measured with an accuracy of 0.001%.

4 SUMMARY AND CONCLUSION

This paper presents a new method for measuring small strains in soil triaxial tests to assess K_0 values for soils with a wide range of physical properties. A series of drained triaxial consolidation tests with pore pressure measurements were performed in order to determine K_0 using strain gauges attached directly to the soil sample, using Teflon tape as a transitional layer. During soil consolidation, the strain detected by the strain gauge was used as a guide throughout the test. When an increase of strain was detected, the confining pressure within the triaxial cell was slowly and incrementally increased in order to maintain zero lateral strain. Once zero strain could no longer be maintained, and the sample begins to expand uncontrollably active earth pressure was reached and an accurate K_0 value of a soil could be determined. The proposed method for obtaining very small strains in the soil sample was successfully used to obtain K_0 values for different soil types. This method has several important advantages compared to many existing techniques: it is easy to mount with minimal sample disturbance, it has high accuracy, and is relatively inexpensive. This method provides a direct measurement of strain, making this direct measurement more practical than other presently known laboratory techniques.

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