Influence of anisotropy on the compressibility and earth pressure characteristics of sensitive Leda clay

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ABSTRACT
A better understanding of the lateral earth pressure in Leda clay, which is vulnerable to catastrophic landslides, would enhance the designs of retaining walls, basements, and slopes. Insights into the effects of various state parameters on the magnitude of lateral forces would be of interest. One-dimensional consolidation (loading/unloading/reloading) tests were conducted on high-quality undisturbed soil samples obtained at different depths from a site in the Ottawa region to measure the changes in lateral earth pressure with stress history. Test results indicate that soil anisotropy plays a strong role in influencing the compressibility characteristics of the soil. In addition to effective friction angle and OCR, the at-rest lateral earth pressure coefficient also depends on the level of anisotropy for sensitive Leda clays. This paper also analyzes the validity of the widely used K_0 equations in sensitive clays and evaluates its dependence on the stress history.

RÉSUMÉ
Une meilleure compréhension de la pression latérale des terres dans l'argile Leda, qui est vulnérable aux glissements de terrain catastrophiques, améliorerait la conception des murs de soutènement, des sous-sols et des pentes. Des informations sur les effets de divers paramètres d'état sur l'ampleur des forces latérales seraient pertinent. Pour mesurer les changements de la pression latérale des terres avec l'historique des contraintes, des essais de consolidation unidimensionnels de chargement/déchargement/rechargement ont été effectués sur des échantillons de sol non remaniés de haute qualité obtenus à différentes profondeurs d'un site de la région d'Ottawa. Les résultats des tests indiquent que l'anisotropie du sol joue un rôle important en influençant les caractéristiques de compressibilité du sol. En plus de l'angle de frottement effectif et de ratio de surconsolidation, le coefficient de pression latérale des terres au repos dépend également du niveau d'anisotropie des argiles Leda sensibles. Cet article analyse également la validité des équations K_o largement utilisées dans les argiles sensibles et évalue sa dépendance à l'historique des contraintes.

1 INTRODUCTION

Most of the major cities in Eastern Canada, like Ottawa, Montreal and Quebec City, are underlain by relatively thick deposits of sensitive marine clay called Champlain Sea clay or Leda clay. Several catastrophic failures, such as the Lemieux landslide (near Ottawa in 1993), the St Jean-Vianney slide (that killed 31 people near Quebec City in 1971), and the relatively recent St Jude landslide (that cost the lives of four in 2010) have occurred in Leda Clay (LC) deposits which underlie vast regions of Eastern North America (Eden and Mitchell , 1970; Quinn et al., 2011; Locat et al., 2017). In addition to these dramatic events, during the last few decades, several hundred smaller landslides have occurred in LC deposits in the Ottawa region alone. These failures highlight the risks and uncertainties associated with the design, construction and maintenance of geotechnical structures on this problematic soil.

In designing the geotechnical structures such as dams, embankments and retaining walls, it is quite important to have an accurate estimation of in-situ stress state of the soil under both static and dynamic loads. The coefficient of earth pressure at rest (K_0), which is expressed as the ratio of effective horizontal stress (σ'_h) to effective vertical stress (σ'_v), is often used to define the in-situ stress state of soil under no lateral deformation condition. As per the revised National Building Code of Canada (NBCC), the peak ground acceleration (PGA) levels that are routinely used in geotechnical designs have increased from 0.21g in the pre-2005 NBCC codes to 0.42g in 2005 and now to 0.61g in NBCC 2020 for Ottawa (which is typical of the changes in most of the cities in this region). Significant uncertainties exist in the determination of K_0 while designing the above-mentioned earth structures for these high levels of dynamic loading intensity. These uncertainties in K_0 determination also aggravated the costs associated with the design and construction in recent years to achieve the required safety factor against the revised earthquake loads. Hence, it is evident that the accurate determination of K_0 is very crucial for the safe and economical design of earth structures.

In the past, several attempts have been made to determine the value of K_0 through direct measurements in field and laboratory. In field, investigations using self-boring pressure meter (SBP), total stress cell (TSC), hydraulic fracturing test (HFT) and geophysical methods provide a direct estimation of K_0. In laboratory, K_0 is usually evaluated through instrumented oedometer or a suitably designed triaxial apparatus, e.g., Hamouche et al. (1995); Butcher and Powell (1995); Fioravante et al. (1998). In addition to these direct methods, K_0 can also be evaluated through empirical correlations derived from field tests such

as cone penetration test (CPT) and flat dilatometer test (DMT) e.g., Mayne and Kulhawy (1990).

These previous investigations have demonstrated that K_0 is significantly affected by several factors such as soil type, density (Chu and Gan 2004), particle shape (Hendron 1963), plasticity index, effective friction angle (ϕ'), stress history and loading mode (Brooker and Ireland 1965; Hamouche et al. 1995).

In the current design practice, geotechnical engineers commonly use Jaky's (1944) equation to estimate K_0 (Eq.1), which was originally proposed for normally consolidated soils.

$$K_0 = 1 - \sin \phi' \quad [1]$$

Several studies have highlighted that this equation underestimates K_0 for overconsolidated soils (Lirer et al. 2011 and Lee et al. 2014) and proposed modifications by incorporating the effects of stress history through over consolidation ratio (OCR) (Wroth 1973; Meyerhof 1976; Mayne and Kulhawy 1982). Some of the proposed equations by considering the effects of stress history are as follows (μ is the Poisson's ratio):

$$K_0 = [(1 - sin\ \phi') \times OCR] - \left[\frac{\mu}{(1-\mu)}\right](OCR - 1)$$

(Wroth 1973) [2]

$$K_0 = (1 - \sin \phi')(OCR)^{0.5}$$

(Meyerhof, 1976) [3]

$$K_0 = (1 - \sin \phi')(OCR)^{\sin \phi'}$$

(Mayne and Kulhawy 1982) [4]

These equations were formulated based on the assumption of isotropic behaviour of soil and ignore the effect of soil fabric, thus the anisotropy.

It is a well-established fact that the natural soil deposits are inherently anisotropic, and their strength and deformation characteristics depend on both the magnitude and direction of the applied loads, e.g.,Oda (1972); Sivathayalan and Vaid (2002).

Only very few attempts were made to investigate the effect of soil fabric on compressibility characteristics and K_0 of soil (Okochi and Tatsuoka 1984; Doran et al. 2000; Sivakumar et al. 2002, Sivakumar et al. 2009; Northcutt and Wijewickreme 2013). Sivakumar et al. (2002) and Sivakumar et al. (2009) developed a theoretical relationship between K_0 and OCR based on anisotropic elasticity and evaluated its validity through laboratory tests on undisturbed and reconstituted stiff London clay and Belfast upper boulder clay. Northcutt and Wijewickreme (2013) evaluated the effect of soil fabric ensued from different specimen reconstitution techniques on K_0 through one-dimensional (1-D) consolidation testing on Fraser River sand specimens. Invariably all the previous studies on the effect of soil anisotropy on K_0 were carried out either on stiff clay or reconstituted sand samples. Considering the significance of sensitive Leda clay deposits in Eastern North America, it is imperative to study the effects of soil anisotropy on K_0 for these sensitive clays. Previously, several studies have been carried out to assess the K_0 for sensitive clays through in-situ and laboratory tests, e.g., Silvestri and Morgavi (1982); Haamouche et al. (1995). Silvestri and Morgavi (1982) performed 1-D consolidation tests on undisturbed sensitive clays of eastern Canada and reported K_0 varies between 0.29 to 0.96 through different phases of consolidation. Hamouche et al. (1995) determined K_0 for sensitive Leda clays at three different locations in Quebec through in-situ soil tests and found that the commonly used K_0 relation for clayey soils (Eq. 3) overestimates K_0 for sensitive clays. However, all these studies ignore the effect of anisotropy on the lateral stress development for sensitive Leda clays. So far, to the best of our knowledge, the effect of anisotropic soil fabric on lateral stress development on LC deposits is not fully explored.

Therefore, the major objective of this research paper is to evaluate the fabric dependency of compressibility characteristics and lateral earth pressure at rest for sensitive Leda clays through 1-D consolidation tests. The tests were carried out using customized 1-D fixed ring consolidometer apparatus developed at Carleton University (CU). The different fabric orientations of undisturbed Leda clay samples were obtained by trimming the samples vertically and horizontally. Evaluating the anisotropic characteristics of soil through soil specimens trimmed at different orientations or samples pluviated with bedding planes inclined at different angles to the vertical is commonly employed by several researchers (Oda 1972; Tatsuoka et al. 1986). The consolidation test results obtained from horizontally and vertically trimmed samples were analyzed and compared to deduce the effect of anisotropic soil fabric on K_0 for sensitive LC deposits.

2 CU 1-D FIXED RING CONSOLIDOMETER

The one-dimensional consolidation tests on undisturbed Leda clay samples were carried out using the custom-built 1-D fixed ring oedometer, which can measure lateral stress and pore water pressure in addition to vertical stress and vertical displacement. The oedometer used in this study is shown in Figure 1. The soil specimen is confined in a 76 mm diameter by 38 mm high stainless-steel ring, which has a cutting edge at the top (to facilitate sample preparation). The ring is sealed to a base that has a porous stone connected to a drainage port and a pore pressure transducer.

A stainless-steel type Bellofram air cylinder with 140 mm bore and 130 mm stroke lies between the Nullmatic 40-100 pressure regulator, and a base connected to a load cell is used to apply the required vertical stress to the soil sample. This setup is capable of supplying a maximum vertical stress of about 2800 kPa. The applied vertical load to the soil sample is measured using a 3000-lb capacity Honeywell load cell. Using the in-house data acquisition system associated with the test setup, this load cell enables a measurement resolution of 0.10 N, which translates to 0.02 kPa of vertical stress.

The horizontal stress developed in the soil specimen is measured by four strain gauges attached to the outer wall of the consolidation ring. The schematic illustration of the oedometer ring and the position of strain gauges are shown in Figure 2. A Honeywell gauge pressure transducer with a range of 100 psi is connected to the base of the sample provides continuous measurement of pore water pressure throughout the test duration. An externally mounted linear variable differential transformer (LVDT) (Model 7DCDT-1000) facilitates the continuous measurement of vertical displacement.

Figure 1. CU 1-D fixed ring consolidometer

While the samples obtained along the vertical and horizontal orientations are used as a proxy to infer anisotropic characteristics, technically, the testing adopted herein is not a proper representation of the effects of fabric anisotropy. The sample obtained along the vertical orientation possesses an axisymmetric fabric about the vertical loading direction (both the principal stress and fabric axes are aligned in this case). The sample obtained along the horizontal orientation is subjected to an axisymmetric vertical load (as in the other case); however, it lacks fabric symmetry about the vertical axis. Thus, the measured horizontal response is an average representation of the fabric in the vertical and horizontal planes.

A metal loading cap (pressure pad) transfers the applied load vertically to the specimen through a ball seating. The porous stone glued to the top-loading cap facilitates free drainage at the top during loading. The test setup is then connected to a water reservoir to create a saturated system that ensures that the pressure measurements do not suffer due to system compliance. The data acquisition program collected one data point for every 0.1 seconds immediately after applying load increment. However, the time interval between data points was gradually increased, and the program saved about 2400 data points over 24 hours.

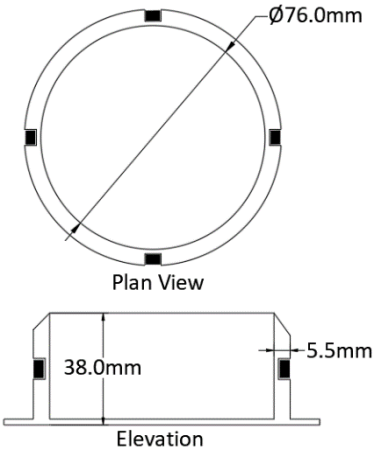

Figure 2. Position of strain gauges in Oedometer ring

3 TEST MATERIAL

The consolidation tests were performed on high-quality ‘undisturbed’ samples of sensitive Champlain Sea clay obtained from depths ranging from 5.82 to 21.55 m from a site located close to the Canadian Museum of Nature in downtown Ottawa. The clay fraction (< 2 μm) of samples obtained at 5.82 m and 21.55 m depth are 79% and 38%, respectively.

The natural moisture content (WC), liquid limit (LL), plastic limit (PL), plasticity index (PI), and liquidity index (LI) of the tested samples are summarized in Table 1. It can be noted that WC, LL and PL decrease with an increase in depth for this site which is consistent with the observations made by Mitchell and Klugman (1979) from other sites located in the same region. The liquid limits of the test samples were less than their corresponding moisture content, which leads to a liquidity index greater than unity. This characteristic is quite common in sensitive Leda clays and has been reported by other researchers (Eden and Crawford 1957, Gillot 1979).

Table 1. Properties of tested clay specimens

Depth(m)	WC (%)	LL (%)	PL (%)	PI (%)	LI
5.82	76.8	54.2	37.0	17.2	2.32
8.26	69.3	48.6	32.3	16.4	2.26
11.0	61.5	44.9	33.2	11.7	2.41
17.41	34.5	29.7	20.7	9.0	1.54
21.55	35.9	31.6	21.8	9.8	1. 44

The average activity (Activity = PI/clay percentage) of these soil samples is about 0.24, which classifies the soil as inactive (Skempton 1953, Holtz et al. 2011). These "inactive" characteristics of sensitive clay are consistent with the observation made by Eden and Crawford (1957). Further, the major mineral composition of Champlain Sea

clay is mica (Brydon and Patry 1961) and the observed activity is consistent with the activity reported by Mitchell and Soga (2005) and Holtz et al. (2011) for clay minerals with predominant mica content.

4 SPECIMEN PREPARATION AND TEST SETUP

The originally obtained undisturbed block samples were sized to about 127 mm in diameter and 152 to 204 mm in height and wrapped up in a wax coating. Two test specimens, one trimmed along horizontal orientation and the other trimmed along vertical orientation, were obtained from each block sample. The specimen preparation procedures were similar for vertical and horizontally trimmed soil samples. Firstly, the block samples were carefully placed on a clean flat surface and trimmed to a size larger (by about 1.5 cm) than the required test specimen size using a sharp blade. Then the wax surrounding the soil sample was removed, and the end faces were carefully trimmed using a wire saw to have smooth, parallel end surfaces. After this, the trimmed specimen is mounted between the two platens of the soil lathe. It is trimmed further by slowly rotating the specimen about the vertical axis until the sample diameter is slightly higher than the required test specimen diameter. The specimen was then transferred to a flat glass surface. The cutting edge of the oedometer ring was pushed slowly into the specimen with a minimum force to avoid any disturbance. The ring was pushed in until 0.5 cm of the specimen protruded on either side of the consolidation ring, and this excess soil was trimmed later using the wire saw. Lastly, the consolidation ring with the test specimen is transferred to the oedometer setup for consolidation.

Table 2. Load increments at different loading stages

Depth (m)	5.82 & 21.55	8.26	11	17.41
Step	Vertical Stress (kPa)			
1	12.5	12.5	12.5	12.5
2	25	25	25	25
3	50	50	50	50
4	100	100	100	100
5	200	200	200	200
6	400	400	400	400
7	800	200	800	200
8	1600	400	1600	100
9	800	800	800	200
10	400	1600	400	400
11	200	-	200	800
12	400	-	-	1600
13	800	-	-	2200
14	1600	-	-	-

Before starting consolidation, the specimen was inundated with water to keep it under complete saturation. An initial seating load of about 12.5 kPa was applied for a few hours to prevent the swelling of the soil sample. The specimen was then consolidated in stages with a load increment ratio of two and subjected to a maximum consolidation pressure of about 1600 to 2200 kPa. Each stress increment was maintained in the soil specimen for about 24 hours prior to the next increment. The variation of horizontal stress, axial strain, and pore water pressure were monitored continuously during the consolidation process.

A total of 10 consolidation tests were carried out for this research program. Eight tests (four on vertical and four on horizontal orientations) consist of loading, unloading, and reloading stages. The remaining two tests (one on vertical and one on horizontal orientations) only consist of loading and unloading stages. Table 2 presents the schedule of loading followed in all the tests on samples trimmed along both vertical and horizontal orientations taken at different depths.

5 RESULTS OF 1-D CONSOLIDATION TESTS

5.1 Effect of anisotropy on compressibility characteristics

The 1-D consolidation curves of samples obtained at a depth of 5.82 m trimmed along horizontal and vertical orientations are shown in Figure 3. This curve illustrates the relationship between void ratio (e) and the effective vertical stress (σ'_v). The initial void ratio (at 25 kPa) of the two samples was somewhat different, but the trend of the variation of $e\ vs\ \log(\sigma'_v)$ curve is similar irrespective of the orientation of the sample. Both curves show the three distinct phases of deformation: limited compression initially in the over-consolidated region, steep virgin compression upon exceeding the maximum past, and essentially linear unloading/reloading phase.

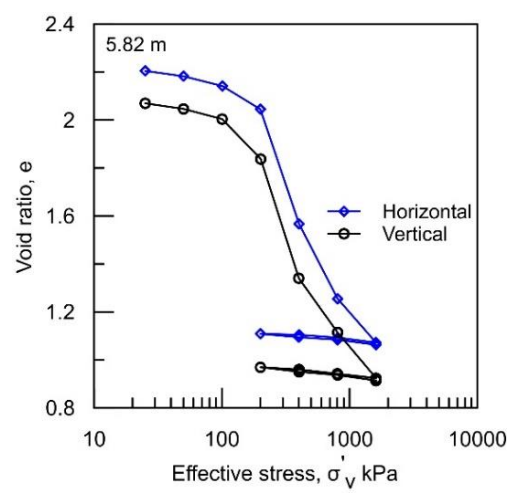

Figure 3. 1-D consolidation curves of samples from 5.82 m depth

The preconsolidation pressure estimated using Casagrande's (1936) procedure for horizontally and vertically trimmed samples are 190 kPa and 200 kPa, respectively. Further, it can be observed that in both samples, a significant compression is triggered during the virgin compression loading, once the applied stresses exceed the preconsolidation pressure, and that the compression index is dependent on the stress level. The average compression indices of vertical (C_{cv}) and horizontally (C_{ch}) oriented samples are found to be 0.627 and 0.635, respectively. In contrast to the virgin compression curves, the unloading and reloading curves are almost flat in both samples.

Figure 4 shows the variation of preconsolidation pressure of both horizontally and vertically cut samples from different depths. Irrespective of the depth, the preconsolidation pressure of vertically cut samples is higher than that of horizontally cut samples. This is generally expected (except in the case of heavily overconsolidated soils) and is consistent with the results reported by Chai et al. (2012) from the consolidation tests on undisturbed Ariake clay and Silvestri et al. (1989) from direct simple shear tests on two Canadian sensitive clays. It also observed that the differences between σ'_v and σ'_h increase with depth. This indicates that the K_0 value gradually decreases to its NC value with increasing depth.

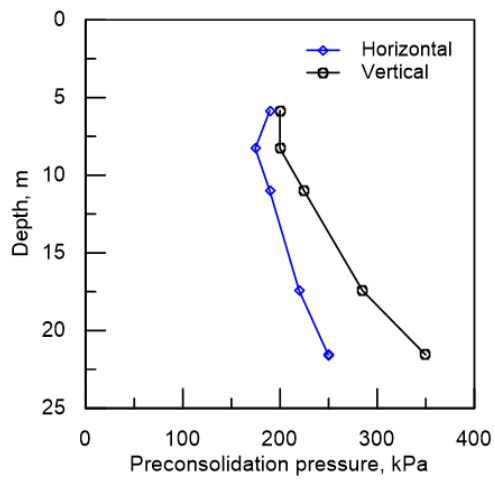

Figure 4. Variation of preconsolidation pressures of samples from different depths.

The compression indices of horizontally cut samples are higher (about 1% to 23%) than the vertically cut samples except at a depth of 21.55 m (Figure 5). This is because the horizontally cut samples are less rigid than the vertically cut samples, which might be due to the directional differences in the soil fabric and the stress history during soil sedimentation (Khan, 1993). The deviation noted at a depth of 21.55 m could be attributed to the specimen disturbance.

Figure 6 presents the variation of the computed coefficients of consolidation with effective vertical stress during the loading phase for both horizontally (C_h) and vertically (C_v) oriented samples were obtained at a depth of 5.8 m. The coefficient of consolidation is usually determined based on the time logarithm method (Casagrande's method) or the time square root method (Taylor's method).

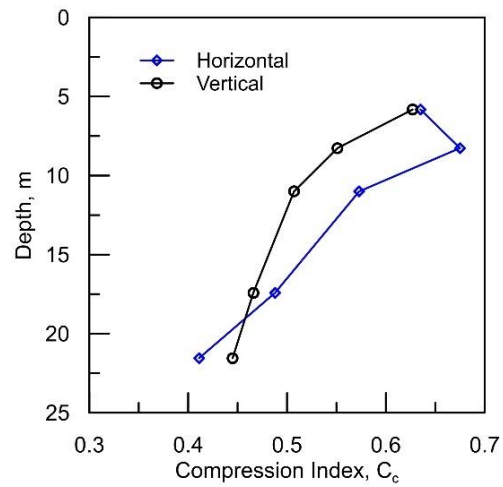

Figure 5. Variation of compression indices of samples from different depths.

The results obtained from Casagrande's method are presented here. It can be observed that during the loading phase, the coefficients of consolidation (both C_h and C_v) increases with the increase in σ'_v until the preconsolidation pressure. This is because, at stresses below the preconsolidation pressure, the orientation of clay particles does not change significantly (Quigley and Thompson, 1966).

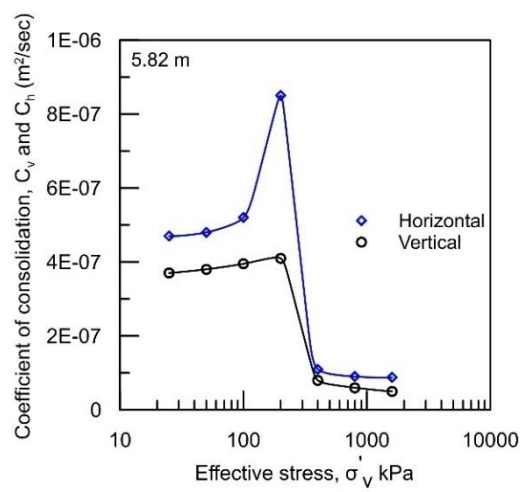

Figure 6. Variation of the coefficients of consolidation during the loading phase for the sample from 5.82 m depth.

Beyond the preconsolidation pressure both C_h and C_v decreases gradually with the continuous increment of σ'_v . One of the potential reasons for this behaviour could be an abrupt development in the parallelism of the soil particles

as a result of the breakdown of the soil structure (Quigley and Thompson, 1966). Figure 7 presents the variation of the maximum C_h and C_v for the samples obtained at different depths. It can be observed from Figures 6 and 7 that C_h is higher than C_v and this finding is consistent with the data reported in Seah and Koslanant (2003) for sensitive Bangkok clay and Khan (1993) for weathered clay crust. This higher C_h could be because the in-situ clay particles are oriented along the horizontal direction providing more surface area in the vertical direction (Khan 1993). The similar trend of variation of C_h and C_v noted in Figure 6, is observed for all the samples obtained from different depths.

5.2 Effect of anisotropy on at-rest lateral earth pressure coefficient

This section presents the effect of anisotropy on the at-rest earth pressure coefficient of both horizontally and vertically cut samples obtained at a depth of 5.82 m. Similar data for the samples obtained at different depths are presented in Alshawmar (2014). For brevity, those data are not presented in this paper. Figure 8 presents the typical variation of effective horizontal stress (σ'_h) with effective vertical stress (σ'_v) for both horizontally and vertically trimmed samples during loading-unloading and reloading stages.

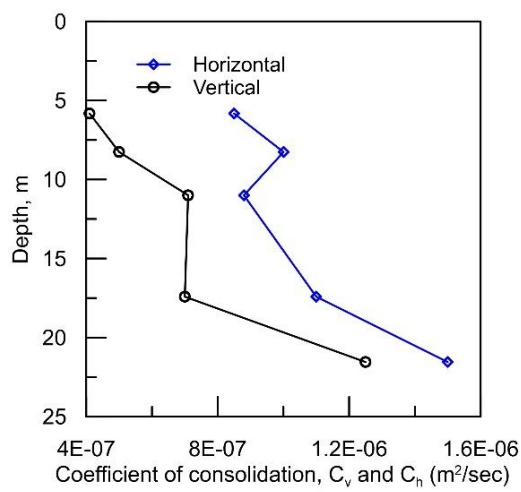

Figure 7. Variation of the maximum coefficients of consolidation for samples obtained from different depths.

It can be noted that during the loading phase, almost a linear relationship exists between σ'_h and σ'_v which indicates that the K_0 is essentially constant during this phase. A non-linear relationship between σ'_h and σ'_v during both unloading and reloading phases is observed. These observations are consistent with the data reported by Brooker and Ireland (1965) for remoulded cohesive soils and Campanella and Vaid (1972) for undisturbed sensitive Haney clay. A similar trend of variation in σ'_h vs σ'_v is observed for all the samples obtained at different depths.

During loading, the K_0 for both vertically (K_{0v}) and horizontally (K_{0h}) trimmed specimens decrease slightly with an increase in σ'_v until the preconsolidation pressure σ'_p, and beyond σ'_p, K_0 remains almost constant. It can be implied that during virgin compression loading, where overconsolidaton ratio (OCR) is unity, K_0 is constant. At this depth of 5.82 m, and during the loading phase, an average value of K_{0h} of 0.62 and K_{0v} of 0.54 is observed. Similar values of K_0 is also observed for samples obtained at different depths.

During unloading, both K_{0v} and K_{0h} increases as σ'_v decreases which indicate that K_0 increases with an increase in OCR. This is consistent with the formulation proposed by Jaky (1944) for K_0 in overconsolidated soils. It can be noticed that during reloading, K_0 decreases with the decrease in OCR. These observations hold true for all the soil samples obtained from different depths. It is also observed that the at-rest lateral earth pressure coefficient in the horizontal orientation is higher than those in the vertical orientation during the loading-unloading-reloading stages. This observation of $K_{0h} > K_{0v}$ is noticed for all the samples from different depths even though these samples have been subjected to different stress histories. This difference between K_{0h} and K_{0v} could be attributed mainly to the variation in stress history during the sedimentation stage.

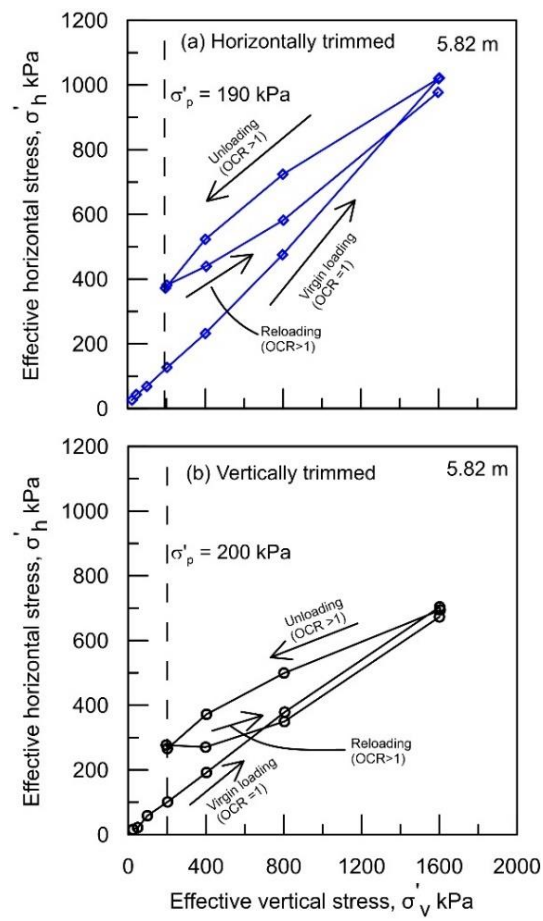

Figure 8. Variation of σ'_h with σ'_v for samples obtained at 5.82m

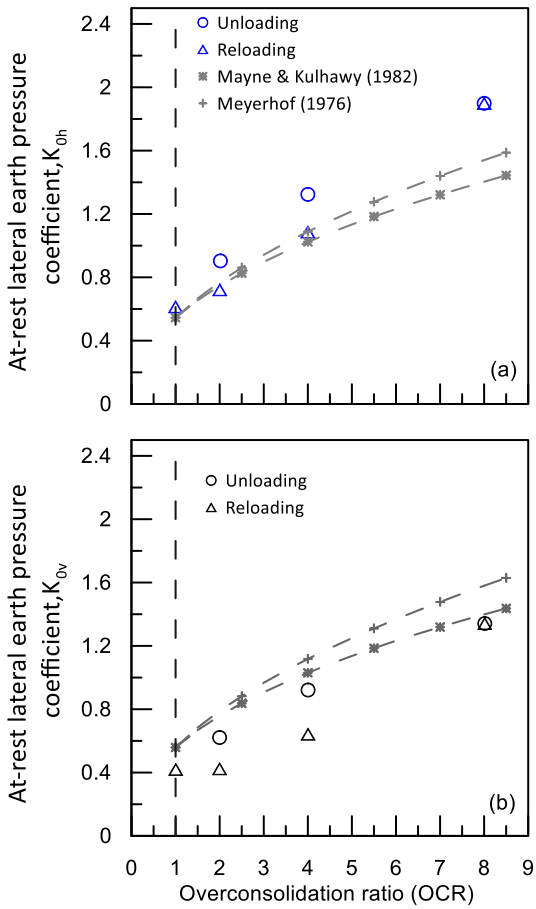

Figure 9. Comparison of K_0 from routinely used equations in practice.

Figure 9 compares the K_{0h} and K_{0v} obtained from the samples at a depth of 5.82 m during the unloading and reloading phases with the K_0 estimated from Eq. 3 and 4. These equations proposed by Meyerhoff (1976) (Eq.3) and Mayne and Kulhawy (1982) (Eq. 4) are routinely used in geotechnical practice to estimate K_0 for overconsolidated clays. An effective friction angle (ϕ') of 27.1º and 26.2º is used to estimate K_0 from Eq. 3 and 4 for horizontally and vertically trimmed samples, respectively (Alshawmar 2014). Similar range of ϕ' is also reported by Theenathayarl (2015) through simple shear tests on undisturbed Leda clay.

It can be noticed that both Eq. 3 and 4 overestimates K_0 for the vertically trimmed sample and almost underestimates K_0 for horizontally trimmed samples. The substantial inaccuracy noted in these routinely used equations for the estimation of K_0 indicates that in addition to ϕ' and OCR, K_0 also depends on the degree of anisotropy of the soil. As noted previously, the deformation mechanics of the horizontally cut samples is rather complex due to the lack of axisymmetry. The results presented represent an average response of the original horizontal and the transposed vertical fabric.

6 SUMMARY AND CONCLUSION

An experimental investigation was carried out to assess the anisotropic characteristics of sensitive Champlain Sea clay and its influence on the at-rest lateral earth pressure coefficient. For this purpose, one-dimensional consolidation tests were carried out on undisturbed Leda clay samples obtained from different depths and trimmed under horizontal and vertical orientations. Based on the presented experimental results, the following conclusions were made:

- The soil fabric plays a critical role in affecting the compressibility characteristics, which is evident from the differences noted in the compression index and coefficient of consolidation.
- The at-rest lateral earth pressure coefficients, termed K_{0v}, and K_{0h} herein depend on the overconsolidation ratio, and they increases with increasing OCR.
- The routinely used empirical equations (Eq. 3 and 4) to estimate K_0 do not consider effects of anisotropy. They seem to work satisfactorily for certain soils but not necessarily for Sensitive Leda clay.
- A reliable value of the at-rest earth pressure coefficient for Leda clay can be predicted by proposing a more accurate relationship for K_0 by considering the degree of anisotropy of the soil fabric.

7 ACKNOWLEDGEMENTS

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