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Determination of the permeability of overconsolidated clay from pressuremeter pressure hold tests

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Abstract

A method was developed to interpret the horizontal permeability (k_h) from pressuremeter pressure hold tests (PHT) of approximately 3 min duration. The method relies on a regression analysis of the numerical analysis simulating the consolidation of clay under a constant pressure boundary during undrained expansion. The method was applied to a series of PHTs performed in deep clay formations in the Seattle area. The interpreted permeabilities are thought to be more representative of *in situ* conditions than those determined by laboratory testing by virtue of reduced disturbance during sample recovery and preparation. Results could be improved with a further exclusion of the creep effect on PHT.

Key words: Pressuremeter, consolidation, overconsolidated clay, permeability, *in situ* testing, finite element analysis

Introduction

One of the major challenges with overconsolidated clay is determining representative *in situ* soil parameters. Due to the high degree of state dependence, sample disturbance, and sample size, consistent and reliable laboratory data are often difficult to obtain (Schmertmann 1955; Lo 1970; Morgenstern and Thomson 1971; Olson and Daniel 1981; Hight et al. 2007). *In situ* test methods are often used to minimize disturbance and

maintain *in situ* stresses as close to the native state as possible (Wroth 1984). Historically, pressuremeters have provided detailed information regarding the shear modulus, horizontal earth pressure coefficient, and drained and undrained shear strength of soil.

Consolidation tests using a pressuremeter are typically conducted by holding the maximum strain (~10%) constant also known as the Strain Hold Test (SHT) (Clarke et al. 1979). Radial consolidation after expansion of a borehole has been commonly portrayed by the excess pore-water pressure decay measured at the cavity surface. The ability to measure excess pore-water pressure change with a diaphragm type transducer allows determination of the consolidation and/or permeability coefficients by interpreting the excess pore-water pressure dissipation curve (Fahey and Carter 1986; Fioravante et al. 1994; Huang et al. 1990; Jang et al. 2003).

During the SHT test, the excess pore-water pressure decay at a constant borehole strain is monitored to determine the excess pore-water pressure response with time. This requires the operator to continually adjust the pressure to maintain a near constant strain in the pressuremeter probe. This has the same effect as an excess pore-water pressure dissipation test for a piezocone (CPTu) pushed into a cohesive deposit. At high pressures, however, this decay can be rapid and the rate of pressure correction is often difficult to maintain. This test also requires the presence of a pore-water pressure transducer to be located on the surface of the dilatometer and in intimate contact with the surrounding soil at all times. This is not always feasible because not all pressuremeters are equipped with pore-water pressure transducers and the presence of a steel lantern may also prevent pore-water pressure measurements.

This paper discusses the development of a modified analytical method designed to interpret the horizontal permeability and the corresponding coefficient of horizontal volume change from a conventional pressure hold test. This study is not intended to provide a comprehensive solution but rather, is designed to further the discussion and science associated with pore-water pressure decay at the surface of a borehole during a pressure hold test. The steps associated with interpreting strain data when measured in heavily overconsolidated cohesive soils are provided followed by an example of the application of the solution using data from the Greater Seattle area.

Interpretation of Soil Permeabilities from Pressuremeter Pressure Hold Tests (PHT) in Clay

Consolidation during the PHT

As with most pressuremeter interpretation methods, polar coordinates and plane strain conditions are assumed for the analysis. This assumes that all strains are in the radial and circumferential directions and that no axial strain is permitted. It is also assumed that the cavity annulus is fully saturated and that excess pore-water pressure only develops within the plastic region, as described by Carter et al. (1979). Because of the low *in situ* permeability of clay, tests in clay are typically treated as an undrained expansion. However, by maintaining the expansion pressure for a certain amount of time, the excess pore-water pressure would dissipate to the outer field, elastic-plastic boundary. As a result, radial consolidation takes place with the further expansion of the cavity until steady state conditions are reached, as illustrated in Figure 1. It is important to note, that as dissipation of the excess pore-water pressures occur, the effective stress of the ground is increasing and the outer field elastic-plastic boundary is adjusting accordingly. This

response is similar to that of a constant strain rate consolidation test (Crawford 1965; Vaid et al. 1979; Leroueil 1988). The constant area ratio defined as $\Delta A/A$ and is given as the change in area / the current area per unit length or in terms of circumferential strain, ε_c . Mathematically, this is expressed in Eq. (1).

$$\Delta A/A = 1 - 1/(1 + \varepsilon_c)^2 \quad (1)$$

The governing equation of the radial consolidation is given as a function of excess pore-water pressure (Randolph and Wroth 1979),

$$\frac{\partial u}{\partial t} = c_h \frac{\partial^2 u}{\partial r^2}, \quad (2)$$

where c_h is the coefficient of consolidation in the testing plane (horizontal).

As a closed-form solution for pressure-hold test consolidation does not exist, the finite element code Abaqus (Abaqus 2014) is used to simulate the transient response of soils with a fluid-stress coupled model under fixed boundary loadings. The soil is modelled as a Modified Cam Clay (MCC) material but with perfectly plastic behaviour for undrained expansion following the onset of yielding. Undrained expansion is a necessary and sufficient condition in order to accumulate excess pore-water pressures in the near-field soils surrounding a pressuremeter probe. The volume change calculated by the MCC only occurs upon cessation of undrained expansion, corresponding with the start of the pressure hold, consolidation test. Finally, an overconsolidation ratio (OCR) equal to 2 was also assumed. A lower OCR has been assumed in order to maintain the likelihood of saturation of the soils while remaining an overconsolidated soil. In the event that higher OCRs are required, then the method of determining the point of saturation for unsaturated soils during a pressuremeter test (Elwood et al. 2015) must be employed. The model is

meshed with 100 axisymmetric elements and 503 nodes (Figure 2), graded by a factor of 1/25 with the finest discretization occurring nearest to the borehole boundary. The soil mass was generated assuming a distance to the outer boundary that is 40 times the borehole radius to ensure no boundary effects were observed and the ground can be reasonably considered as an infinite elastic-perfectly plastic medium.

Figure 3 illustrates the excess pore-water pressures and deformations calculated by the numerical model at i) the initial stage prior to undrained expansion; ii) immediately prior to the cessation of loading and the start of the PHT; and iii) completion of the PHT and excess pore-water pressure dissipation.

The typical profile of excess pore-water pressure at different stages of consolidation reported by Randolph and Wroth, 1979 as calculated from the numerical analysis is shown in Figure 4. Unlike one-dimensional consolidation, where the dissipation region is fixed, the dissipation boundary in the case of radial consolidation migrates towards the far field boundary. While the migration of pore water leads to a transient increase of excess pore-water pressure within a given annular region, excess pore-water pressure will eventually return to initial values when consolidation is complete. This also leads to a variation in volumetric change $\varepsilon_v (\varepsilon_v = \varepsilon_\theta + \varepsilon_r)$ in the radial direction, as shown in Figure 5. In this example, the void volumes in the initial elastic zone remain the same at the end of consolidation. For these simulations, the modified Cam Clay model assumes that the shear modulus G is constant and Poisson's ratio is defined as a function of the mean effective stress and the void ratio.

These simulations were conducted to verify the numerical model by illustrating the assumptions and findings reported by Randolph and Wroth, 1979. Figures 4 and 5 were

determined based on the output of the numerical simulations and the parameters used in the models are as shown.

A typical loading curve obtained from the Alaskan Way Viaduct replacement tunnel in Seattle, Washington is shown below in Figure 6. The PHT is highlighted and magnified 200% in Figure 6 to illustrate the raw data obtained from the test. This brief PHT will be analysed to illustrate the coefficient of volume change of the soil as well as the horizontal permeability for the initial state of stress. Note that the test data has been fit using ideal models described by Gibson and Anderson, 1961 and Jeffries, 1988.

Rather than excess pore-water pressure decay at the cavity boundary, the overall compression in the soil medium is more representative of consolidation progress. As the volumetric strain in the region outside the plastic region eventually becomes zero, consolidation is assumed to only occur within the soil annulus where the materials initially yield. Thus, the calculation of the overall consolidation can be expressed as

$$\bar{U}_t = \frac{\int_{r_0}^{r_p} \Delta \varepsilon_v_t}{\int_{r_0}^{r_p} \Delta \varepsilon_v_{t=\infty}} = \frac{V_{p,t=0} - V_{p,t}}{V_{p,t=0} - V_{p,t=\infty}} = \frac{(r_{p,t=0}^2 - r_{0,t=0}^2) - (r_{p,t}^2 - r_{0,t}^2)}{(r_{p,t=0}^2 - r_{0,t=0}^2) - (r_{p,t=\infty}^2 - r_{0,t=\infty}^2)}, \quad (3)$$

where the subscript p denotes the variables at the elastic-plastic boundary. The radius r at time t is the initial radius $r_{t=0}$ and is then subjected to an additional radial displacement Δr .

Then, the overall consolidation can be written as

$$\bar{U}_t = \frac{2r_{0,t=0}\Delta r_{0,t} - 2r_{p,t=0}\Delta r_{p,t} + \Delta r_{0,t}^2 - \Delta r_{p,t}^2}{2r_{0,t=0}\Delta r_{0,t=\infty} - 2r_{p,t=0}\Delta r_{p,t=\infty} + \Delta r_{p,t=\infty}^2 - \Delta r_{p,t=\infty}^2} \quad (4)$$

Dividing both numerator and denominator by the initial cavity radius $r_{0,t=0}^2$ and removing the infinitesimal terms, Eqn. (4) becomes

$$\bar{U}_t = \frac{\varepsilon_{c,t} - R^2 \varepsilon_{p,t}}{\varepsilon_{c,t=\infty} - R^2 \varepsilon_{p,t=\infty}} \quad (5)$$

where R is the ratio of the boundary of the plastic region to the initial cavity radius, and ε_c and ε_p are the circumferential strains induced by consolidation at the cavity surface and plastic boundaries, respectively.

Eqn. (5) indicates that the average consolidation percentage of a pressure hold test $\bar{U}_t$ can be approximated by the variation of radial displacement measured at the cavity surface if the term $R^2 \varepsilon_p$ is considerably smaller than the cavity strain. Figure 7 shows that the difference between the average consolidation $\bar{U}_t$ and the radial consolidation U_r

represented by the change of cavity strain, i.e., $U_r = \frac{\varepsilon_{c,t}}{\varepsilon_{c,t=\infty}}$ is minimal and therefore the

assumption is considered as reasonable. Also included in Figure 7, is the comparison of the results with the degree of consolidation $U_{\Delta u}$ at the cavity surface indicated by the excess pore-water pressure decay. While excess pore-water pressure dissipation at the membrane-soil interface is significantly faster than average dissipation across the annulus, expansion of the cavity occurs at a reasonably slow rate during consolidation and presents a more gradual and recognizable initial phase of consolidation.

When the computed effective stress path is plotted as is shown in Figure 8, it can be seen that there is no deviation from the total stress path until the onset of yielding at which time, excess pore-water pressures can develop. This response is as would be expected for the MCC model. Note though, in Figure 8, because of the assumption of perfect plasticity during undrained expansion there is no clear deviation of the effective and total stress paths on the initial yield surface. Once the yield surface has been contacted, excess

pore-water pressures accumulate until the undrained expansion is ceased. At this time, the excess pore-water pressures are permitted to dissipate and the yield surface begins to grow as the effective stress path converges with the total stress path.

Radial Displacement of the Cavity Surface in the Consolidation

As shown in Figures 4 and 5, the excess pore-water pressure and volumetric strains within the soil at different radii are non-uniform; consolidation takes place with the soil being compressed in the near-cavity region while the soil is unloaded further away from the borehole. In this case, the consolidation coefficients adopted for these two stress paths should be distinguished accordingly. This is because, a single consolidation coefficient used to predict the coupled effect of excess pore-water pressure dissipation and radial displacement would lead to erroneous results (Fioravante et al. 1994; Goh and Fahey 1996).

The consolidation coefficient for excess pore-water pressure dissipation in the horizontal plane is given by Equation (6)

$$c_h = \frac{k_h}{\gamma_w m_h}, \quad (6)$$

where γ_w is the unit weight of the pore fluid, k_h is the *in situ* horizontal permeability (for a small-strain, pressuremeter test in a low permeability medium, it is assumed that k_h is a constant throughout the expansion); and m_h is the volumetric compressibility of the soil, which varies spatially and temporally with the current state of the soil skeleton (Randolph et al. 1979). In terms of Cam-Clay parameters, the elastic and plastic definitions can be calculated using Equation (7).

$$\begin{aligned} m_h^p &= \frac{\lambda}{(1+e)\sigma'_r} \\ m_h^e &= \frac{\kappa}{(1+e)\sigma'_r} \end{aligned} \quad , \quad (7)$$

where λ and κ are the volumetric compression indices for the plastic and elastic deformations of soil, respectively. These two parameters are related by the plastic volumetric strain ratio, λ , defined as $1 - \kappa/\lambda$. Alternatively, λ can also be estimated as $1 - C_r/C_c$ where C_r and C_c are the slopes of the compression under vertical loadings in oedometer tests. Mayne (1980) published a range of λ for clay based on 105 tests. Mayne indicated that regardless of the clay composition, there was an upper bound around 0.9 and a lower bound around 0.2 for λ . Accordingly, the ratio of λ to κ ranges from 1.25 to 10 for most soils.

Despite the excess pore-water pressure dissipation further hardening the soil in the initial plastic region, the distance to the elastic-plastic boundary undergoes negligible change after the consolidation. The soil is therefore assumed to continue to yield in the initial yield zone ($r/r_0 < R$) while it unloads and reloads elastically in the initial elastic region ($r/r_0 > R$). Thus, for a cavity having the same plastic boundary ratio, R after expansion, the subsequent consolidation can be decomposed into two domains governed by the first and second expressions in Eqn. (7), respectively. R can be expressed as a function of excess pore-water pressure at the cavity surface Δu_{max} and undrained shear strength s_u for an elastic perfectly plastic soil (Gibson and Anderson 1961),

$$R = \exp(\Delta u_{max}/s_u) \quad (8)$$

Randolph and Wroth (1979) suggested that the dimensionless form of time T should be used to present the consolidation for a particular ratio of G/s_u , or $\Delta u_{max}/s_u$ in the case of finite cavity expansion. The time factor is only given when the elastic volumetric compressibility m_h^e is taken into account,

$$T^e = \frac{kt}{m_h^e \gamma_w r_0^2} = \frac{k(1+e)\sigma'_r t}{\kappa \gamma_w r_0^2} \quad (9)$$

To evaluate the effect of plastic deformation within the yielded region on the radial displacement of the cavity surface during consolidation, PHT on three types of soil with different geo-mechanical properties (Table 1) were simulated with a varying critical state pore pressure parameter, λ (Mayne and Swanson 1981).

Replacing m_h with the elastic term m_h^e and assuming that e is equal to the initial void ratio e_0 and σ'_r is equal to the effective radial stress at yielding (i.e., $\sigma'_r = \sigma'_0 + s_u$) in Eqn. 8, the effect of λ can be isolated as shown in Figures 9 and 10, in which the natural logarithm and square root of time factor T are used respectively. Figure 10 is an extension of a consolidation curve to the case of purely elastic deformation ($\lambda = 0$) from the work of Randolph and Wroth (1979).

Regardless of the absolute magnitude of the consolidation coefficient, the deformation of soil in each domain (plastic or elastic) is proportional to its corresponding compressibility. The integrated radial displacement at the cavity surface has the same fashion of variation for a constant ratio of λ to κ . Nevertheless, a slight mismatch of the curves is observed for high values of λ , primarily because the void ratio is significantly reduced as a result of consolidation in highly porous soil so the time factor T determined with the initial void ratio e_0 is not as accurate as that for soils with low compressibility (e.g. $\lambda = 0.2$).

Interpretation of the consolidation coefficient by Clarke et al. (1979), Huang et al. (1990), and Jang et al. (2003) all use the time at 50% consolidation, i.e., t_{50} , be read from a full excess pore-water pressure dissipation curve. Thus, the test needs to be performed over hours, or even days, for soils with extremely low permeability. This creates challenges for in-situ pressuremeter testing when site access and/or work force is limited.

The consolidation curves in Figure 9 exhibits a sigmoidal function regardless of the soil type and permeability. The maximum derivative of each of the curves also occurs at nearly identical points regardless of the soil properties and corresponds with the point of U_{50} or t_{50} . This makes determination of the consolidation percentage possible simply by reading the maximum derivative from an unfinished consolidation curve.

When the same data is plotted with respect to the square root of the dimensionless time factor, T^e as shown in Figure 10, it would appear that the maximum derivative occurs between 15 to 20% of U_{max} regardless of the material characteristics. This observation makes the interpretation more practical since, as will be discussed later, most clays will consolidate to 20% after several minutes in a hold test.

Interpretation of Permeability Based on the Root Time Consolidation Curve

As demonstrated by Carter et al. (1979), radial consolidation is also affected by *in situ* soil parameters, including the permeability k , rigidity index I_r , and maximum excess pore-water pressure Δu_{max} generated at the end of an expansion.

A further study was performed to evaluate the sensitivity of the maximum rate of the degree of consolidation, $(dU)_{max}$ to these factors with the numerical model. The value of $(dU)_{max}$ was found to vary over a range between 0.12 and 0.24 with a mean value and standard deviation of 0.17 and 0.022, respectively (Figure 11).

The parameters used in the sensitivity analysis are shown below in Table 2. These values were obtained from data provided for the detailed site investigations for the Alaskan Way Viaduct replacement tunnel in Seattle, Washington. This project and the subsequent interpretation of parameters is described later sections.

Note that s_u is determined using the formula $s_u = \frac{M\sigma'_0}{\sqrt{3}} \left(\frac{OCR}{2} \right)^\Lambda$ (Cao 2001) with the assumption of an $OCR=2$. Further analysis has indicated that the variance of the undrained shear strength has no effect on the results.

With an acceptable degree of deviation, the average of 17% consolidation is considered the stage at which the maximum derivative of cavity radial displacement would be found in the root time curve. In the same manner as Clarke and Allan's (1990) 50% excess pore-water pressure decay curve, the time factor T^e for 17% percent consolidation can be plotted against the maximum excess pore-water pressure normalized by undrained shear strength (Figure 12). Instead of the single curve for elastic consolidation, however, multiple curves are constructed for various plastic volumetric strain ratios by fitting the numerical results for three types of soils with the exponential or power function

$$\begin{aligned} \Delta u_{\max}/S_u &= a \cdot \exp(b \cdot \ln T^e) \\ \text{or } \Delta u_{\max}/S_u &= a \cdot (T^e)^b \end{aligned} \quad (10)$$

where a and b are constants for a particular volumetric strain ratio Λ . The solution, which considers only elastic deformation (i.e., $\Lambda=0$, not shown in Figure 10) would result in a low time factor and therefore underestimate the permeability as well as the coefficient of consolidation.

Note that the curve fitting parameters have no physical meaning and are based solely on power law curve fitting to numerically simulated data as shown in Figure 12. Figure 12 provides a series of curves fit to the numerical data for a series of soil stiffnesses. These a and b parameters were then plotted with respect to the critical state pore pressure parameter, A in order to provide a means of estimating the dimensionless time factor T^e for any given A using Equations (11) and (12) below. These data are shown below in Figure 13.

The ratio of $\Delta u_{\max}/s_u$ in Figure 11 can be calculated under the assumption of undrained expansion (Gibson and Anderson 1961),

$$\Delta u_{\max}/s_u = \ln\left(\frac{G}{s_u} \cdot \frac{\Delta A}{A}\right), \quad (11)$$

where G and s_u can be independently interpreted from the pressuremeter testing curves and the probe volumetric expansion ratio per unit length, $\Delta A/A$ is given above as Equation 1.

Thus, rewriting Eqn. (11) with the time factor given in Eqn. (9), permeability k can be expressed as

$$k = \frac{m_h^e \gamma_w r_0^2}{t_{17}} \left(\frac{\Delta u_{\max}}{a s_u} \right)^{1/b}, \quad (12)$$

where, coefficients a and b need to be determined for a particular kind of soil by oedometer testing and the elastic volumetric compressibility m_h^e can alternatively be estimated from the constrained modulus M ,

$$m_h^e = 1/M = \frac{1-2\nu}{2G(1-\nu)}. \quad (13)$$

The other parameters in Eqn. (12) can be interpreted from a full cycle of a pressuremeter test. Figure 14 shows the hold time required for 17% consolidation in soils of different stiffness. The peak of derivatives in the root time consolidation curve can be reached in under three minutes in a hold test in most clayey materials with permeabilities varying from $\sim 10^{-10}$ to 10^{-7} cm/s, while in a high permeable medium ($>10^{-6}$ cm/s) the initial consolidation can barely be captured in the log of radial displacement because pore water quickly dissipates at the start of the hold test.

Pressuremeter Hold Tests in Seattle Clay Deposits

In order to demonstrate the application of the proposed method of analysis, the analytical method described above was used on data obtained from the ground investigation associated with the design of the Alaskan Way Viaduct replacement tunnel. This tunnel extends beneath downtown Seattle. During this ground investigation, pre-bored pressuremeter tests were carried out in 17 boreholes to measure soil properties including the shear modulus, drained and undrained strength, as well as the horizontal earth pressure at rest. The tests were performed in a variety of deposits including a dense glacial till; stiff glacio-lacustrine and lacustrine clays and silts; as well as a glacial outwash sand and gravel. Pre-bored pressuremeter tests were employed in the most of boreholes. While PHT were performed in both granular and clayey materials, only the stiff clay deposits are interpreted in this study because the range of the cohesive soil permeabilities is within the interpretable range shown in Figure 13.

Other than the well-studied Lawton Clay deposits, the lacustrine and glacio-lacustrine clays in the Pre-Vashon Glacier unit have been well investigated in the Seattle area (Troost and Booth 2008). Having been overridden by at least two glaciations, the clays in

this deposit are expected to have a highly variable stress history and a wider variation of *in-situ* conditions (Troost and Booth 2008).

One Dimensional Consolidation Test on Clay Samples

The consolidation characteristics of the two clay deposits within the proposed tunnel alignment were evaluated by oedometric testing on relatively undisturbed soil samples recovered using a standard Pitcher sampler. Soil properties including the coefficient of consolidation, c_v and volumetric compressibility, m_v , were calculated from the test results (Table 3 and Figure 15). The specimens were found to exhibit a significant variation in compressibility during virgin compression and swelling/recompression, possibly as a result of their different overconsolidation ratios (Mesri et al. 1978). However, the plastic volumetric compression ratio, λ is less dependent on the stress history and was found to be fairly consistent for all of the clay samples examined as part of this study. With a variety of laboratory findings, Mayne and Swanson (1981) also demonstrated that λ is insensitive to *in-situ* stress anisotropy.

Table 3 suggests, that as expected, the λ parameter is consistent throughout the various boreholes and depths of recovery with an average of 0.812 and a standard deviation of 0.013. A constant value of λ equal to 0.815 has been selected for the Seattle clay in this study. This then results in time factor, T^e constants, $a = 2.60$ and $b = 0.39$ for Eqn. (10).

Interpretation of Pressuremeter Test Curves

A slightly undersized pocket was drilled for each of the pressuremeter test holes using a 74.6 mm diameter tricone bit. The instrument was then advanced into the test pockets as far as possible without causing damage to the instrument. After the pressuremeter was secured in place, the test was performed by injecting compressed nitrogen until a

maximum cavity strain of around 10% was achieved. The first PHT was conducted after a cavity strain of approximately 1 to 2%. The cavity strain was calculated with the average value of radial displacements monitored for three axes using $\varepsilon_c = \Delta r / r_0$. Immediately following the PHT, the borehole pressure was reduced and an unload-reload loop was completed to evaluate the shear modulus of the soils. The borehole expansion was conducted at a normal rate around 5 kPa/s so that undrained conditions could be assumed for the loading phase.

In order to obtain the variables needed to evaluate Eqn. (12), the pressuremeter test curves were interpreted by focusing on the identification of the undrained shear strength, s_u and the shear modulus, G . Another important aspect to recognize was the initial disturbance (overcutting or undercutting), because it is important to understand the true cavity strain for the excess pore-water pressure calculation. The presence of disturbance usually results in a shifting of the stress-strain curve.

Noting the concerns above, an interpretation strategy to determine the degree of disturbance for any one test was developed based on the method proposed by Jefferies (1988). An independent analysis of the contraction curve yields a reliable value of undrained shear strength, which is minimally affected by the initial disturbance. The shear modulus was derived from the unload-reload cycles. The contraction curve was reproduced after s_u and G were determined and, by assuming that s_u is not significantly mobilized in the loading phase, the expansion curve was also plotted using the Gibson and Anderson (1961) undrained cavity expansion model. Finally, the initial disturbance was identified by reconstructing the entire test curve and the field data shifted accordingly.

As the pocket is smaller than the instrument body, the pressuremeters were pushed in with a certain amount of axial force and the resultant lateral deformation of soil led to a higher lift-off pressure than expected, as shown in Figure 16. Two pressure hold tests were attempted after material yields, as indicated from the pressuremeter expansion curve (e.g., Figure 16). However, in most cases, drilling introduced a certain degree of disturbance to the surrounding soils. Most clays were weak and the borehole surface was severely smeared by the drilling fluid (e.g., Figure 17), resulting in boreholes that were actually oversized for the pressuremeter.

Figures 16 and 17 indicate the field data stress-strain curves as well as the ideal interpretation for both the cavity expansion and contraction. A non-linear regression was conducted to fit the field and idealized expansion and contraction curves and a coefficient of determination was determined. The ideal curves appear to fit the field data well with a minimum $R^2=0.83$. It is important to note, that most errors occur during the initial expansion and immediately before and after an unload-reload loop. Difficulties matching the initial elastic portion with model curves were also encountered by Jefferies (1988). Silvestri and Abou-Samra (2008) showed that soil remoulding and initial drilling unloading significantly changes the soil structure in the near-borehole region and linear sections of the initial expansion are not representative of the true stiffness of the virgin soil.

Interpretation of Pressure Hold Tests

During each pressuremeter test within the clay deposits, two PHTs were carried out after the onset of yielding. The hold tests were maintained for approximately 3 minutes with the radial displacement monitored at a logging rate of 5 seconds per measurement. The

proposed method to determine the permeability requires the input of time at 17% consolidation, which corresponds to the point where the maximum cavity strain rate is observed in the root time curve. An example of two sequential pressure hold tests in Seattle clay is shown in Figures 18 and 19.

By differentiating the curve constructed from the discrete data points, the maximum slope of the strain plot was found. The consolidation rate, i.e., the derivative of the curve, is very sensitive to the inevitable scatter in the measurements if consecutive points are used in the calculation. A technique to reduce the noise and smoothen the variation is to fit the plot with a highly representative function, in this case a cubic polynomial function. The peak shown in the best fitting derivative curve is linked to 17% consolidation.

The permeability for the clay in borehole TB233 is calculated based on Eqn. (12) for both pressure hold tests (Table 4). To determine the ratio of $\Delta u_{max}/S_u$, the pore water pressure dissipated during past hold tests must be subtracted from the value calculated by Eqn. (12). The reduction of Δu_{max} was estimated with the final percent consolidation at the end of the hold test, but care should be taken because the consolidation presented in terms of the cavity strain significantly underestimates the excess pore-water pressure dissipation at the cavity surface. Thus, a higher permeability was usually interpreted as a result of an overestimated $\Delta u_{max}/S_u$ for the pressure hold test conducted after the first cycle. For this reason, only the permeabilities interpreted from the first test are considered for all of the tests in this study.

Interpretation of Results of Pressure Hold Tests in Seattle Clay

Values of horizontal permeability k_h within Seattle clay were determined at six different locations with the t_{17} obtained from the root time consolidation curves (Figure 20), as

presented in Table 5. While all of the tests were conducted over a small range of depths between 28 to 44 m below the ground surface), the horizontal permeability, even at the same depth (e.g., TB233 and TB321), is remarkably inconsistent. Hoopes and Hughes (2014) explained that multiple geological processes, including glacial ice loading and shearing, subglacial pore-water drainage, and deposition and erosion of pro-glacial sediments, combined with ancient earth surface activities such as landslides, might be responsible for the spatial variability and discontinuity of deformation zones even in the same deposits. For example, the test in TB233 was performed in a clay tightly locked in by the surrounding glacial tills, which reasonably exhibits a higher strength than other clays resting in a large deposit.

However, the consistency of the interpretation could still be justified by tests that were conducted within relatively close proximity. At depths of approximately 40 m below the ground surface, the pressure hold tests conducted in neighbouring boreholes (TB221 and TB318) yield nearly the same horizontal permeabilities. The interpreted permeabilities from pressure hold tests in the same borehole (TB318) at depths only 10 m apart are also comparable.

Figure 21 shows horizontal permeabilities interpreted from PHT compared with the vertical permeabilities of clay samples measured by laboratory tests. The permeameter testing was performed on tube samples using a flexible wall permeameter by Shannon & Wilson Inc. The sample was consolidated to σ'_{v_0} before conducting a constant head test. Included in Figure 21 are the permeabilities estimated from the oedometer testing also performed by Shannon and Wilson.

After the overburden was subjected to multiple cycles of compression and unloading during the cycles of glaciation and interglacial periods, anisotropy of the permeability is likely present within the clay deposits. However, Adams et al. (2013) demonstrated experimentally that the reorientation of fine particles ($0.2-0.7\mu\text{m}$) did not typically occur when subjected to uniaxial compression and subsequent unloading. In these experiments, they demonstrated an anisotropic permeability ratio, r_k where $r_k = k_h/k_v$, varies from 1.2 to 1.9 for a vertical effective stress change between 0.4 to 10 MPa. Tavenas et al. (1984) also reported anisotropic permeability measurements on various clay samples that r_k is generally less than three in natural clays. Thus, the difference between the permeabilities measured by *in situ* and laboratory methods might not be attributed to the intrinsic permeability anisotropy. Instead, the quality of the test and the limited accuracy of the testing technique itself could at least be partially responsible for the discrepancy in the results.

As shown in Figure 21, the permeabilities determined by permeameter tests are approximately two or three orders of magnitude higher than those determined by PHT and oedometer tests. As only a very limited flow would be observed in the low permeability clay, the compliance of the testing system may contribute to a significant error in the measurement. Leakage in the external fittings, saturation of membrane and porous stone, and preferred flow through the interface between the membrane and sample would all lead to extra flow and thus overestimation of permeability from the calculation (Tavenas et al. 1983). Provided that the sampling and permeameter testing were performed with adequate control, error could also result from the excessive hydraulic gradient and soil volumetric changes due to perturbation from the injection pressure

(Olson and Daniel 1981). Consolidating the sample to the initial vertical effective stress would also be insufficient to restore the original soil structure. It has been well documented (Schmertmann 1955; Lo 1970; Morgenstern and Thomson 1971; Olson and Daniel 1981) that sampling and preparation of most overconsolidated soils is sufficient to impact the results of most laboratory tests. Additionally, the high horizontal stresses, measured by Hoopes and Hughes (2014), would have more impact on the hydraulic conductivity in the undisturbed state.

In situ permeability may be better represented by mean effective stress, p' calculated for the clays where the PHTs were performed (Figure 22). A trend was observed where the permeability decreases exponentially with an increasing mean effective stress, in accordance with the permeability-stress relation suggested by Vaughan (1994).

The evaluation of permeability through the application of Terzaghi's solution to oedometer testing was also considered unreliable because the assumption that c_v is a constant through the consolidation does not reflect the actual consolidation characteristics at the point where c_v is interpreted. Compared with the true permeabilities measured by a falling head test during each step of consolidation, Tavenas et al. (1983) reported that permeabilities calculated based on c_v likely underestimated the actual permeability by as much as a factor of 10.

While the consolidation parameters would also experience a considerable variation in PHT, up to the 17% of consolidation where the analysis is performed, the soil might not undergo significant structural alteration when compared to tests conducted at the end of expansion. Thus, in addition to considerably shortening the time required to carry out a

test, the interpretation would be less affected by assuming a constant m_h^e and k for the initial period of consolidation.

In addition to excess pore-water pressure dissipation, the creep behavior of clays has also been noted as a reason for the time-dependent expansion of the pressuremeter membrane during hold tests (Fahey 1988; Goh 1994; Hoopes and Hughes 2014). Anderson et al. (1988) validated the creep effect in pressuremeter testing of cohesive soils but did not quantify the creep effect under the same stress conditions for each stage of loading; thus, there is still no direct evidence that creep would dominate the effective stress change on cavity deformation. A calibration study with only creep allowed (i.e., under totally drained conditions) during a pressuremeter hold test would be valuable to decouple the stress-induced deformation from measurement.

Summary and Conclusions

The consolidation characteristics under constant radial loading on the cavity surface were numerically assessed for varying constitutive parameters for clays. A statistical analysis of model results suggests that the lapse time at a low consolidation percentage (~17%) can be extracted from the root time curve of the radial displacement during the pressuremeter pressure hold test. The deformation of the borehole during PHT is governed by the ratio, R of the plastic boundary, r to the initial borehole radius, r_0 and the plastic volumetric strain ratio of soil λ . Their effects on the time obtained for 17% consolidation were distinguished in the curves characterized by a non-dimensional time factor T^e . A method was developed to interpret the permeability of clay deposits with a 3-min pressure hold test after the near-field soil yields during the initial expansion.

The proposed technique was applied to data from PHT in a Seattle clay with reasonable results. The oedometeric test results were used to determine A and the other required inputs were determined from the conventional interpretation of pressuremeter testing data. Care must be taken when estimating excess pore-water pressure $\Delta u_{\max}$ at the cavity surface at the start of consolidation due to its power-law relation with the interpreted permeability. Collins and Yu (1996) predicted the variation of the generated excess pore-water pressure as a function of the overconsolidation ratio under the undrained cavity expansion. $\Delta u_{\max}$ will be underestimated by Eqn. (12) in normally consolidated clay and overestimated in heavily consolidated clay. In either case, a correction may need to be adopted for excess pore-water pressure estimation and an independent excess pore-water pressure measurement also used to calibrate the findings herein. However, a more unpredictable volumetric strain path for soils under different consolidation conditions, as discussed by Fioravante et al. (1994), would increase the difficulties of back-analyzing the horizontal permeability from a PHT.

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Figure Captions

Figure 1 Soil states in the cavity annulus before and after consolidation with fixed boundary loadings

Figure 2 Geometry of the numerical model for PHT analysis

Figure 3. Calculated excess pore-water pressures and deformations

Figure 4 Variation of excess pore-water pressure in the soil annulus radius during PHT ($Ir=G/S_u=20$, $k=10^{-7}$ cm/s, and $\Delta u_{max}/S_u=1.6$) (modified from Randolph and Wroth 1979)

Figure 5 Typical variation of volumetric strain in the soil annulus radius during PHT ($Ir=20$, $k=10^{-7}$ cm/s, and $\Delta u_{max}/S_u=1.6$) (modified from Randolph and Wroth, 1979)

Figure 6 Fitting the pressuremeter testing data with the model curve (TB233, 44.3 m)

Figure 7 Representation of consolidation by excess pore-water pressure decay and radial displacement compared to the true average consolidation curve ($Ir=20$, $k=10^{-7}$ cm/s, and $\Delta u_{max}/S_u=1.6$)

Figure 8. Effective and total stress paths during undrained expansion and consolidation

Figure 9 Variation of the normalized radial displacement during consolidation with the logarithm of time ($\Delta u_{max}/s_u = 1.6$ or $R = 4.95$)

Figure 10 Variation of the normalized radial displacement during consolidation with the square root of time ($\Delta u_{max}/s_u = 1.6$ or $R = 4.95$)

Figure 11 The varying values of $(dU)_{max}$ in root time consolidation curves for different *in situ* parameters and induced excess pore-water pressure conditions from the numerical computation

Figure 12 The non-dimensional time factor curve for 17% consolidation during a PHT

Figure 13 *a* and *b* parameters based on numerical results for varying Λ

Figure 14 Time to reach 17% consolidation for PHT in soils of different stiffness ($\Lambda = 0.75$)

Figure 15 Settlement of clay samples at increasing consolidation pressure

Figure 16 Fitting the pressuremeter testing data with the model curve (TB233, 44.3 m)

Figure 17 Fitting the pressuremeter testing data with the model curve (TB318, 28 m)

Figure 18 First hold test at 44.3 m for borehole TB233

Figure 19 Second hold test at 44.3 m for borehole TB233

Figure 20 Root time curve of cavity strain variation during pressure hold tests in Seattle clay deposits; the point of 17% consolidation in each test is marked by a star symbol.

Figure 21 Permeability measured by PHT and laboratory testing versus effective vertical stresses

Figure 22 Variation of permeability responding to *in situ* stresses

Table Headers

Table 1 Three types of soils for assessment of plastic consolidation

Table 2 Input parameters for sensitivity analysis

Table 3 Oedometer testing results on undisturbed clay samples

Table 4 Interpretation results of PHT in borehole TB233

Table 5 Interpretation of results of six PHTs in Seattle clay

Draft

Table 1. Three types of soils for assessment of plastic consolidation

Type	Initial Effective Stress, σ_0' (MPa)	Initial Void Ratio, e_0	Undrained Shear Strength, s_u (MPa)	Shear Modulus, G (MPa)	Elastic Volumetric Compression Index, κ
#1	1	1.5	0.577	23.1	0.06
#2	2	1.0	1.154	46.2	0.03
#3	3	0.6	1.732	69.3	0.015

Draft

Table 2. Input parameters for sensitivity analysis

Fixed Parameters						Variables		
σ'_0 (MPa)	s_u^* (MPa)	M	e_0	κ	λ	I_r	k (cm/s)	R
2	0.25, 0.5, 1.154, 2	1	1	0.0205	0.178	20, 40, 60, 80	10^{-8} , 10^{-7} , 10^{-6} , 10^{-5} , 10^{-4}	1.44, 2.22, 3.43

Table 3. Oedometer testing results on undisturbed clay samples

Borehole ID	Geological Unit	Recovered Depth (m)	PI (%)	Coefficient of Volume Change c_v (cm ² /s)	Recompression Index C_r	Virgin Compression Index C_c	Volumetric Compression Ratio A
221	Glacio-lacustrine Clay	38	25	$9.0 \times 10^{-3} \sim 4.5 \times 10^{-3}$	0.087	0.457	0.809
238	Lacustrine Clay	38	17	$8.0 \times 10^{-3} \sim 1.6 \times 10^{-2}$	0.052	0.272	0.810
316	Glacio-lacustrine Clay	38	20	$6.4 \times 10^{-3} \sim 7.9 \times 10^{-3}$	0.051	0.262	0.806
221	Glacio-lacustrine Clay	42	20	$8.0 \times 10^{-3} \sim 1.6 \times 10^{-2}$	0.068	0.353	0.808
218A	Lacustrine Clay	44	51	$5.2 \times 10^{-3} \sim 1.6 \times 10^{-2}$	0.165	0.836	0.802
318	Glacio-lacustrine Clay	48	20	$17.1 \times 10^{-3} \sim 3.2 \times 10^{-2}$	0.043	0.261	0.837

Table 4. Interpretation results of PHT in borehole TB233

Test #	ε_c	$\Delta u_{max}/s_u$	m_h^e (kPa ⁻¹)	t_{17} (s)	k (cm/s)
1	0.78%	1.88	4.25×10^{-7}	61	4.3×10^{-9}
2	4.19%	2.81	4.25×10^{-7}	59	1.25×10^{-8}

Draft

Table 5. Interpretation of results of six PHTs in Seattle clay

Borehole No.	Depth (m)	PI (%)	w_c (%)	s_u (kPa)	G (kPa)	σ'_{v_0} (kPa)	σ'_{h_0} (kPa)	m_h^e (kPa ⁻¹)	k_h (cm/s)
TB216	41	27	21	692	6.6×10^5	405	1325	3.83×10^{-7}	8.5×10^{-9}
TB221	40	24	28	759	3.6×10^5	401	1149	7.09×10^{-7}	2.1×10^{-9}
TB233	44	16	18	1410	6.0×10^5	443	1917	4.25×10^{-7}	4.8×10^{-10}
TB321	44	31	25	180	1.6×10^4	436	557	1.63×10^{-5}	6.2×10^{-8}
TB318	28	36	29	417	8.2×10^4	282	615	3.09×10^{-6}	5.0×10^{-9}
TB318	38	26	28	673	9.8×10^4	378	1062	2.58×10^{-6}	2.5×10^{-9}

Draft

Figure 1. Soil states in the cavity annulus before and after consolidation with fixed boundary loadings

65x40mm (300 x 300 DPI)

Figure 2. Geometry of the numerical model for PHT analysis
57x13mm (300 x 300 DPI)

Draft

Figure 3. Calculated excess pore-water pressures and deformations

201x84mm (300 x 300 DPI)

Figure 4. Variation of excess pore-water pressure in the soil annulus radius during PHT ($I_r=G/S_u=20$, $k=10^{-7}$ cm/s, and $\Delta u_{max}/S_u=1.6$) (modified from Randolph and Wroth 1979)

190x106mm (300 x 300 DPI)

Figure 5. Typical variation of volumetric strain in the soil annulus radius during PHT ($I_r=20$, $k=10^{-7} \text{ cm/s}$, and $\Delta u_{\max}/S_u=1.6$). (modified from Randolph and Wroth 1979)

188x104mm (300 x 300 DPI)

Figure 6. Fitting the pressuremeter testing data with the model curve (TB233, 44.3 m)

81x42mm (300 x 300 DPI)

Figure 7. Representation of consolidation by excess pore-water pressure decay and radial displacement compared to the true average consolidation curve ($I_r=20$, $k=10^{-7}$ cm/s, and $\Delta u_{\max}/S_u=1.6$)

192x108mm (300 x 300 DPI)

Figure 8. Effective and total stress paths during undrained expansion and consolidation

64x46mm (300 x 300 DPI)

Figure 9. Variation of the normalized radial displacement during consolidation with the logarithm of time ($\Delta u_{max}/S_u=1.6$ or $R=4.95$)

186x104mm (300 x 300 DPI)

Figure 10. Variation of the normalized radial displacement during consolidation with the square root of time ($\Delta u_{max}/S_u=1.6$ or $R=4.95$)

189x107mm (300 x 300 DPI)

Figure 11. The varying values of $(dU)_{\max}$ in root time consolidation curves for different *in situ* parameters and induced excess pore-water pressure conditions from the numerical computation

63x45mm (300 x 300 DPI)

Figure 12. The non-dimensional time factor curve for 17% consolidation during a PHT
68x52mm (300 x 300 DPI)

Figure 13. a and b parameters based on numerical results for varying Λ

68x51mm (300 x 300 DPI)

Figure 14. Time to reach 17% consolidation for PHT in soils of different stiffness ($\Lambda=0.75$)

58x37mm (300 x 300 DPI)

Figure 15. Settlement of clay samples at increasing consolidation pressure

62x46mm (300 x 300 DPI)

Figure 16. Fitting the pressuremeter testing data with the model curve (TB233, 44.3 m)

81x42mm (300 x 300 DPI)

Figure 17. Fitting the pressuremeter testing data with the model curve (TB318, 28 m)

83x44mm (300 x 300 DPI)

Figure 18. First hold test at 44.3 m for borehole TB233

64x40mm (300 x 300 DPI)

Figure 19. Second hold test at 44.3 m for borehole TB233

64x40mm (300 x 300 DPI)

Figure 20. Root time curve of cavity strain variation during pressure hold tests in Seattle clay deposits; the point of 17% consolidation in each test is marked by a star symbol.

114x126mm (300 x 300 DPI)

Figure 21. Permeability measured by PHT and laboratory testing versus effective vertical stresses

61x41mm (300 x 300 DPI)

Figure 22. Variation of permeability responding to in situ stresses

62x47mm (300 x 300 DPI)