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Interpretation of undrained shear strength observed in confined triaxial compression tests on compacted clay

C.K. Wong¹, M.C.H. Lun² and R.C.K. Wong^{1*}

¹Department of Civil Engineering, Schulich School of Engineering,
University of Calgary, Calgary, Alberta, Canada T2N 1N4

²Thurber Engineering Ltd. Calgary, Alberta, Canada T2H 2H8

*Corresponding Author: rckwong@ucalgary.ca

Abstract

This paper presents an interpretation technique to quantify the effects of compaction state and matric suction on the undrained shear strength of compacted clay under confined undrained triaxial compression. This novel technique is based on the mathematical frameworks of SHANSEP (Stress History and Normalized Soil Engineering Property) method for saturated soil and BBM (Barcelona Basic model) for unsaturated soil. Test data of compacted Calgary till were analyzed and interpreted using the proposed technique. The interpretation technique is very useful in delineating the relative impacts of the factors on the behavioral trends in measured undrained shear strength. It was found that in addition to the initial compacted void ratio and suction, soil structure and failure mode exert significant influence on the undrained shear strength of compacted clay. This technique is attractive to engineering practitioners because the confined undrained compression tests (with no pore air and water pressure measurement) are much simpler and less time consuming compared to rigorous laboratory tests on unsaturated soil.

Keywords: compacted clay, undrained shear strength, suction, soil structure, tensile rupture, strain softening, ductile yielding

Introduction

The undrained shear strength of saturated soil s_u is one of the key input parameters in total stress analysis for short-term stability and capacity of geotechnical structures (e.g., shallow footing, pile foundation and excavated slope) under undrained loading conditions in which excess pore pressure is not allowed to dissipate. For practical applications, normalized undrained shear strengths are correlated to measured soil physical indices such as liquid index and plasticity index. For normally consolidated and lightly overconsolidated clays with low and moderate plasticity, the normalized undrained shear strength $\frac{s_u}{\sigma'_{vc}}$ falls within in a range of 0.129 – 0.270 where σ'_{vc} is the vertical consolidation stress (Bjerrum and Simons 1960; Jamiolkowski et al. 1985). Based on experimental results, Ladd and Foot (1974) proposed an equation expressing $\frac{s_u}{\sigma'_{vc}}$ as a function of overconsolidation ratio (OCR) using a procedure called SHANSEP method (Stress History and Normalized Soil Engineering Property). This so-called SHANSEP equation has been theoretically substantiated by the critical state soil mechanics (Mayne 1980; Wroth 1984).

The SHANSEP equation is a very practical tool in estimating the undrained shear strength of saturated soils. However, natural and backfill soils may not be at its saturated state in some applications. It is intuitive to question if the SHANSEP equation can be extended to characterize the undrained shear strength of compacted or unsaturated soil. A more fundamental question may be: what are the factors affecting the undrained shear strength of compacted soil and their roles? While considerable research have been focused on characterization of shear strength of unsaturated soils (e.g., Bishop et al. 1960; Fredlund et al. 1978; Escario and Saez 1986; Vanapalli et al. 1996), limited studies have focused on investigating the role of unsaturated condition on the undrained shear strength of the material (Mun and McCartney 2015). The undrained shear strength of

unsaturated soil is more complicated to interpret due to the effect of matric suction on the stress state. In addition, the hydraulic hysteresis and diffusivity may promote non-homogeneous suction within the test specimens and thus localized deformation.

For compacted soil, the effects of compaction effort (energy) and soil structure add another level of complexity to the stress-strain responses observed in an undrained test. Compaction of soil at different gravimetric water contents causes a variation in initial suction and void ratio (Olson and Langfelder 1965). Soils compacted on the dry and wet sides of the optimum moisture content retain flocculated and dispersed structures, respectively. In a flocculated structure clay particles are interlocked whereas in a dispersed structure clay particles are oriented in a preferential direction. The shear strength and hydraulic conductivity of specimens with flocculated structure are higher than those of specimens with dispersed structure (Mitchell et al. 1965). The undrained tests on compacted specimens may conform to the undrained condition in which pore air and fluid are not allowed to drain in or out of the specimen, but they may not meet the constant volume criterion.

Consolidated drained triaxial compression tests with suction control are ideal tests to investigate the strength-deformation characteristics of unsaturated soil. However, these tests can be time consuming as the shearing rate must be sufficiently low in maintaining constant pore air and water pressures during shearing. The rate effect may affect the observed behaviour of the test material. Alternatively, consolidated undrained triaxial compression tests can be used to determine the undrained shear strength of unsaturated soil, but the consolidation process takes a long time to reach pore air-water pressure equilibrium. In addition, measurements of excess pore air and water pressures at the boundary of unsaturated soil specimens in these tests may not be representative of the entire specimen, particularly at peak stress. Accordingly, the objectives of

this study are: (i) to investigate the undrained behaviour of compacted soil specimens using confined undrained triaxial compression (CUTC) tests with no pore air and water pressure measurement, (ii) to quantify factors affecting undrained behaviour and their relative roles, and (iii) to extend SHANSEP equation to model the undrained shear strength of compacted soil. The CUTC tests can be conducted in a relatively short time. They may not fully provide information on the stress-strain characteristics of the material. However, as the tests are subjected to the same undrained boundary conditions, they may allow to evaluate the effects of compaction parameters including initial saturation, void ratio and compaction state (compacted wet or dry of optimum) on the behavioural trend in undrained shear strength.

This paper endeavours to unite the effects of consolidation history, compaction state and saturation on undrained shear strength of compacted soil in a model. The model is based on a framework encompassing SHANSEP method, critical state model (CSM) and Barcelona Basic model (BBM). The first part of the paper presents background information on these models, and a detailed derivation of the proposed model. The second part provides experimental evidence substantiating the validity of the proposed model, followed by some concluding remarks.

Background

This section presents background information on SHANSEP method and Barcelona Basic model. Ladd and Foot (1974) used the SHANSEP method to normalize the undrained shear strengths of normally consolidated and overconsolidated clays with the vertical consolidation stress. They found that the experimental data follows an empirical correlation:

$$[1] \quad \frac{s_u}{\sigma'_{vc}} = S(OCR)^m$$

where S = normalized undrained strength of normally consolidated clay; m = material constant depending on soil type; OCR = overconsolidation ratio. The value of S can be shown to be a function of internal friction angle, and lies in a range of 0.25 - 0.4 while m is close to 0.8. Based on experimental evidence, Ladd and Foot (1974) validated the SHANSEP equation is applicable for both undrained shear strengths at peak and critical states and under isotropic and anisotropic conditions.

The undrained shear strength of normally consolidated clay has been predicted using critical state soil mechanics (Mayne 1980; Wroth 1984):

$$[2] \quad \frac{s_u}{p_i} = \frac{M}{2} r^{-\Lambda_o}$$

where p_i = isotropic consolidation stress; M = slope of the critical state line ($M = \frac{6 \sin \phi_{cs}}{3 - \sin \phi_{cs}}$; ϕ_{cs} = friction angle at critical state); r = the base of natural logarithm or 2 for original and modified CSM, respectively; $\Lambda_o = 1 - \frac{\kappa}{\lambda}$; λ , κ = coefficients of primary compression and recompression curves under saturated condition, respectively. For OC clay, the above equation [2] becomes

$$[3] \quad \frac{s_u}{p_i} = \frac{M}{2} r^{-\Lambda_o} (R_0)^{\Lambda_o} = S (R_0)^{\Lambda_o}$$

where R_0 = preconsolidation ratio ($R_0 = \frac{p_0^*}{p_i}$; p_i and p_0^* are current and maximum consolidation stresses, respectively). On the basis of CSM, the primary compression curve is defined by:

$$[4] \quad e = e_{p_1} - \lambda \ln \left(\frac{p}{p_1} \right)$$

in which e_{p_1} = reference void ratio at reference stress p_1 ; e = void ratio under normally consolidation stress p . For OC clay, the relationship between p_i and p_0^* is given by:

$$[5] \quad p_0^* = \frac{\exp\left(\frac{e_{p_1} - e_i}{\lambda - \kappa}\right) p_1^{\left(\frac{\lambda}{\lambda - \kappa}\right)}}{p_i^{\left(\frac{\kappa}{\lambda - \kappa}\right)}}$$

where e_i = current void ratio under consolidation stress p_i on the recompression curve.

Based on the observed effect of suction on stress-strain responses in unsaturated soils, Alonso et al. (1990) adopted the framework in CSM, and developed a so-called Barcelona Basic model (BBM) model for partially saturated soils. Since then, many constitutive models have been developed to enhance the basic features in BBM for mechanical behaviour of unsaturated soils (e.g., Wheeler and Sivakumar 1995; Chiu and Ng 2003; Sheng et al. 2008; Yao et al. 2014). Evaluation of the constitutive models is beyond the scope of this study. For simplicity, the original BBM was used herein. In BBM, several distinct features are added to CSM to quantify the role of suction on the strength-deformation characteristics of unsaturated soils. Those relevant to this study are detailed herein.

The primary compression curve in (v, p) plane may be represented by (Fig. 1):

$$[6] \quad v = N(s) - \lambda(s) \ln\left(\frac{p}{p^c}\right)$$

where v = specific volume ($v = 1 + e$); $N(s)$ = specific volume at reference stress p^c ; $\lambda(s)$ = coefficient of compressibility for a given suction s ; p = net mean consolidation stress (the difference between the mean total stress and pore air pressure). [6] reveals that the stiffness of the unsaturated soil under primary compression increases with suction. For unloading and reloading, the partially saturated soil is assumed to follow an elastic response similar to that in CSM:

$$[7] \quad dv = -\kappa \frac{dp}{p}$$

where κ = compressibility coefficient under rebound and recompression, independent of suction.

Fig. 1 illustrates the effect of suction on compression and recompression curves and preconsolidation stress under saturated and unsaturated conditions, p_0^* and p_0 in (v, p) plane, which can be shown in (p, q) stress plane of Fig. 2. The elliptical yield surface and preconsolidation stress grow with increasing suction. In addition, the suction induces an apparent cohesion resulting in an increase in the strength at critical state. The relationship between p_0^* and p_0 is depicted in Fig. 3 of (p, q, s) space. In BBM, the reference net mean stress p^c in [6] is specifically chosen to simplify the relationship between p_0^* and p_0 for each associated suction. At this stress, the partially saturated soil may reach the saturated virgin state through a wetting path which involves only elastic rebound. The relationship between p_0^* and p_0 for each associated suction is defined by a so-called loading-collapse curve (LC) (see Fig. 3):

$$[8] \quad \left(\frac{p_0}{p^c}\right) = \left(\frac{p_0^*}{p^c}\right)^{\left[\frac{\lambda(0) - \kappa}{\lambda(s) - \kappa}\right]}$$

in which p_0^* is used to define the set of yield value p_0 and the hardening parameter in the LC curve. The LC curve describes the effect of suction on p_0 . The apparent increase in preconsolidation stress (the difference between p_0^* and p_0) increases with increasing suction. The LC curve also accounts for the collapse mechanism in soil matrix observed in wetting path. The suction-dependent compressibility coefficient $\lambda(s)$ in the LC curve of [8] is defined as:

$$[9] \quad \lambda(s) = \lambda(0)[(1 - r)\exp(-\beta s) + r]$$

where β = parameter controlling the rate of increase in soil stiffness with suction; r = constant limiting the asymptotic maximum stiffness.

Now, the current consolidation stress p_i can be related to p_0 via p_0^* using LC curve. The preconsolidation ratio under unsaturated condition R_s , is defined by the ratio of p_0 to p_i .

Proposed model for undrained shear strength of compacted clay

The SHANSEP method and BBM are based on the fundamental framework of CSM. The CSM has been demonstrated to predict behaviour of normally and lightly overconsolidated clays in a consistent manner, but the model shows some deficiencies in matching the behaviour of heavily overconsolidated clays. Due to the heavily overconsolidated nature of compacted soils, novel modifications are required to implement in CSM to improve its versatility in simulating unique behaviours observed in CUTC tests. These unique behaviours include tensile rupture, brittle strain softening and ductile yielding.

To capture these behaviours, part of the elliptical yield surface used in CSM and BBM (Figs. 2 and 3) is replaced by a series of yield surfaces shown in Fig. 4. The compacted clay can sustain tension due to the suction. A tension limit line (TLL) 'a-c' is added to the stress regime at very low net mean stress p close to zero or at very high R_s . The TLL may be determined from results of unconfined tension and compression tests on compacted specimens (points 'b' and 'c') as illustrated in Fig. 4, and defined as:

$$[10] \quad q = m_t(p - p_{uc}) + q_{uc}$$

where $m_t = \frac{q_{uc} - q_{ut}}{p_{uc} - p_{ut}}$; p_{ut} , q_{ut} , p_{uc} and q_{uc} = net mean and deviatoric stresses at failure for unconfined tension and compression, respectively; $p_{ut} = n_t q_{ut}$ and $p_{uc} = n_c q_{uc}$ (n_t , n_c = slopes of stress paths for unconfined tension and compression test in (p, q) stress plane, respectively; both are equal to 3 for triaxial condition). The isotropic tensile strength σ_t of point 'a' may be

determined from extrapolation of TLL, i.e., $\sigma_t = \frac{-q_{uc}}{m_t} + p_{uc}$. The TLL of [10] denotes the occurrence of tensile rupture in compacted soils.

With increasing confining stress p_i , the compacted soil will fail in shear instead of tension. At relatively low p_i , the characteristic stress-strain curve displays a peak shear strength with shear dilation. The locus of peak shear strength is represented by Hvorslev surface which is approximated by a linear envelope 'c-d' in Fig. 4:

$$[11] \quad q = m_h(p - p_{uc}) + q_{uc}$$

in which $m_h = \tan(\alpha_e)$; α_e = Hvorslev true friction angle in (p, q) stress plane ($\tan(\alpha_e) = \sin(\phi_e)$); ϕ_e = Hvorslev true friction angle in (σ , τ) stress plane).

Shear dilation is gradually suppressed by increasing p_i resulting in a ductile yielding behaviour. This ductile behaviour may be controlled by the elliptical yield surface:

$$[12] \quad \left(\frac{p - p_h}{p_0 - p_h}\right)^2 + \left(\frac{q}{q_h}\right)^2 = 1$$

where p_h , q_h = net mean and deviatoric stresses at point 'd' demarcating the brittle strain softening response from the ductile strain hardening response. The point 'd' can be determined from the intersection of Hvorslev surface 'c-d' and critical state line 'a-d' (Fig. 4). The corresponding critical state line which is a function of suction or tensile strength is given by:

$$[13] \quad q_f = M(p_f - \sigma_t)$$

where p_f , q_f = net mean and deviatoric stresses on the CSL, respectively. Thus, the abscissa of point 'd' or stress limit p_h is given by:

$$[14] \quad p_h = \frac{q_{uc} + m_h p_{uc} - M \sigma_t}{M - m_h}$$

Fig. 4 shows that a compacted soil specimen could fail in three possible distinct regimes depending on its initial compaction state, suction, confining stress and stress path. In CSM, a saturated soil specimen below the yield surface is assumed to behave elastically, and no volume change occurs under undrained condition, i.e., no change in p . The undrained stress path will follow those illustrated by paths '1-2', '3-4' or '5-6' depending on p_i . However, the effective stress path of a compacted soil specimen under CUTC is more complicated than that of a saturated specimen. The application of a total isotropic confining stress σ to the compacted specimen under undrained condition will cause increases in pore air (u_a) and water (u_w) pressures, resulting in a net mean stress p_i . The magnitude of p_i is less than that of the applied confining stress as the pore air and water pressure parameters are less than unity. The initial suction at the initial compaction state may be reduced. Under undrained shearing, both u_a and u_w may vary and thus the suction, causing a volume change. This implies that under CUTC the test specimen may not stay on a (p, s) stress plane of constant suction or void ratio. However, if the changes in u_a and u_w are small (this aspect will be treated later), the actual effective stress path may follow those shown in Fig. 4.

Tensile rupture regime

For $p_i < p_{uc}$ (Fig. 4), the specimen will likely fail in tensile rupture mode when its stress path '1-2' intersects TLL. The undrained shear strength can be computed by substituting the initial net mean consolidation stress p_i into [10] of TLL:

$$[15] \quad \frac{s_u}{p_i} = \frac{m_t \left(1 - \frac{R_s}{R_t}\right) + n_t \frac{R_s}{R_t}}{2} = \frac{m_t}{2} + \left(\frac{n_t - m_t}{2}\right) \frac{R_s}{R_t} \quad \text{for } R_t < R_s$$

where $R_t = \frac{p_0}{p_{uc}}$ and $R_s = \frac{p_0}{p_i}$.

At such high R_s , the heavily compacted specimen tends to dilate under shearing, and its suction may increase. Thus, [15] may yield a value of $\frac{s_u}{p_i}$ lower than the actual one.

Brittle (strain softening) regime

For $p_{uc} < p_i < p_h$, the undrained brittle failure may develop according to the stress path '3-4' shown in Fig. 4. The derivation of p_i -normalized undrained shear strength at peak state for brittle regime is similar to that for tensile rupture regime, except that the locus of peak strength is represented by Hvorslev surface. Thus, $\frac{s_u}{p_i}$ is given by:

$$[16] \quad \frac{s_u}{p_i} = \frac{q_u}{2p_i} = \frac{m_t(p_i - p_{uc}) + q_{uc}}{2p_i} = \frac{m_t\left(p_i - \frac{p_0}{R_t}\right) + n_t \frac{p_0}{R_t}}{2p_i} = \frac{m_t\left(1 - \frac{R_s}{R_t}\right) + n_t \frac{R_s}{R_t}}{2} \quad ; \text{ for } R_t < R_s < R_h$$

Ductile yielding (strain hardening) regime

For $p_i > p_h$, the compacted specimen will yield when its stress path reaches the yield surface at point '6' (Fig. 4). Assuming the volume change in the compacted specimen during undrained loading is small, the stress path with respect to the critical state line, compression and recompression curves in e - $\ln(p)$ plot is shown in Fig. 5. Adopting the method used in Wroth (1984), triangles '5-7-X' and 'A-B-X' are similar. The p_f at point '7' on the critical state may be estimated:

$$[17] \quad p_f = p_i \left(\frac{\frac{p_0}{K}}{p_i} \right)^\Lambda$$

where $\Lambda = 1 - \frac{\kappa}{\lambda(s)}$. Substituting p_f from [17] into [13] of CSL yields the deviatoric stress at critical state q_f . Then, the p_i -normalized undrained shear strength at critical state for ductile yielding regime may be obtained:

$$[18] \quad \frac{s_u}{p_i} = \frac{q_f}{2p_i} = \frac{M}{2} \left(\frac{p_0}{Kp_i} \right)^\Lambda + \frac{M\sigma_t}{2p_i} = \frac{M}{2} \left(\frac{R_0}{K} \right)^\Lambda + \frac{M(1 - \frac{n_t}{m_t}) \frac{p_0}{R_t}}{2p_i} = \frac{M}{2} \left\{ \left[\frac{R_0}{K} \right]^\Lambda + \left(1 - \frac{n_t}{m_t} \right) \frac{R_0}{R_t} \right\}$$

Note that in derivation of [10], $\sigma_t = \frac{-q_{uc}}{m_t} + p_{uc} = (1 - \frac{n_t}{m_t})p_{uc} = (1 - \frac{n_t}{m_t}) \frac{p_0}{R_t}$.

From [15], [16] and [18], it can be seen that the p_t -normalized undrained shear strengths of compacted clays do not follow a single SHANSEP equation. For tensile rupture and brittle failure, $\frac{s_u}{p_i}$ increases linearly with R_s . For ductile yielding, the relationship between $\frac{s_u}{p_i}$ and R_s is nonlinear and comparable to the SHANSEP equation except that an intercept is introduced due to the apparent suction-induced cohesion.

Experimental program

Material used

The material used is a dark grey-brown till which is a glaciolacustrine deposit. The soil was collected from a slope site at Sunnyside Hill, Calgary, Alberta, Canada. This slope with inclined angles of 22 - 26° has experienced soil movement and instability in the past (Mahmoud and Burwash 1998). The soil movement was induced by a deep seated and shallow failure surfaces. The deep seated failure might be initiated by rising water table while the shallow failure was triggered by saturation under snow melting in spring or prolonged rainfall in summer. The soil was collected at several locations in the slope at depths of about 0.3 - 0.5 m, and the suction measurements were also taken at the same locations using a 2900F124 Quick Draw tensiometer (soil moisture probe) manufactured by Soilmoisture Equipment Corp. Cal., US. The soil was oven-dried, crushed and sieved through a 2-mm sieve prior to any testing. Grain size analysis shows that the soil contains 45% sand, 39.5% silt and 15.5% clay. The plastic and liquid limits are about 20 and 35%, respectively.

Uniaxial tension (UT) tests

The objective of this test series is to determine the uniaxial tensile strength of compacted specimens as a function of suction. The test apparatus consisted of a specimen reconstitution mold, specimen holders, and a metal frame. The mold was made up of two brass split halves with a dog-bone shaped configuration. The specimen dimensions are 75 mm in length, 41 mm in width at both ends, 25 mm in width at the middle and 25 mm in thickness. For preparation of compacted specimens, distilled water was blended with 120 g of crushed oven-dried soil powder to the desired water content. The soil mixture was compacted inside the mold using a metal rod in one layer to the predetermined dry density. The compacted specimen was extruded from the mold, and was kept in an air-tight plastic bag for 24 hours to allow uniform hydration within the specimen. Then, the specimen was held in an upright position using holders and clamps with two cables attached. One cable was attached to the metal frame, and the other cable was attached to a hanger with dead weights. Dead weights of 0.1 N each were added to the hanger until the specimen failed. The failed specimen was oven-dried to obtain its water content, and its suction was estimated from SWCC (Fig. 6). Five dog-bone shaped specimens were prepared for uniaxial tension tests at each target water content, and the average value of measured uniaxial tensile strengths was calculated for the analysis. Tests were repeated at varying water contents and bulk densities.

Unconfined compression (UC) test

The objective of this test series is to determine the unconfined compressive strength (UCS) of compacted specimens as a function of suction. Cylindrical specimens of 100 mm in diameter and 115 mm in height were prepared at a desired water content using a standard compaction mold and proctor according to ASTM D698. The mold with the compacted

specimens was placed inside an air-tight plastic bag and stored in an air-tight food storage container for a week. The matric suction measurements were taken at the exposed end of the compacted specimen using the Quick Draw moisture probe, and the specimen was extruded from the mold. Then, the specimen was mounted on an ELE load frame of 10 kN capacity, and a constant displacement rate of 1 mm per min was applied to shear the specimen. Lubricated platens (pre-compressed latex membrane disks with a silicone grease coating in between) were used in the test to reduce the effect of end friction on the deformation behaviour. The specimen was loosely wrapped with a piece of thin transparent plastic sheet to prevent moisture loss during shearing. The axial deformation and load were recorded using a data acquisition system. After each UC test, suction measurements were taken on the failed specimen again to compare to its initial reading, and the specimen was oven-dried to determine its water content.

Confined undrained triaxial compression (CUTC) test

The objective of this test series is to determine the undrained shear strengths of compacted specimens under confined undrained triaxial compression. The specimen preparation and extrusion were similar to those in UC tests. After the suction measurement, the specimen was mounted on two lubricated platens and encased with a latex membrane sleeve and neoprene O-rings on the platens. Then, the assembly was positioned inside a conventional triaxial cell with no drainage ports at the top and bottom platens, and a confining stress was applied to the specimen immediately. The specimen was allowed to rest without drainage for 10 min, and a constant displacement rate of 1 mm per min was applied to shear the specimen. This magnitude of confining stress and the test duration are not expected to lead to pressurized saturation of the specimens (Mun and McCartney 2015). After shearing, suction and water content measurements were conducted on the failed specimen.

The CUTC tests were repeated with five varying water contents starting from the dry side to the wet side of optimum at confining stresses of 10, 30, 50 and 100 kPa. This confining stress range falls within the stress domain at shallow depths in the slope site. The total number of CUTC tests presented in this study is twenty.

Test results

Compaction test results show that a water content of 11.6% and a dry density of 1.9 Mg/m³ correspond to the optimum condition of compacted Calgary till for standard Proctor compaction effort (Fig. 6). Data of matric suction and water content measured from the compacted specimens are included in Fig. 6, and a power function was used to curve fit the data so that matric suction could be readily estimated with measured water content.

Fig. 7 plots the measured uniaxial tensile and unconfined compressive strengths of compacted Calgary till specimens versus matric suction. The matric suction data were not measured on the UT specimens due to the small size, and they were estimated from SWCC of Fig. 6 based on the measurement of water content values of the test specimens after the tests. A power function was used to curve fit the data trend (Fig. 7). The tensile strength of compacted Calgary till increases with matric suction, but starts to level off at matric suction of about 100 kPa. The failure surfaces are rough due to the pulling effect (see pictures in Appendix). The tensile strength of compacted specimen is solely due to suction of negative pore water pressure (Wong et al. 2017). A power function was used to curve fit the UCS data (Fig. 7). During the test, vertical tensile cracks were formed on the specimen surface, and the cracks widened as the axial strain increased (see pictures in Appendix).

Figs. 8(a) - (d) show the deviatoric stress-axial strain responses of compacted Calgary till specimens under CUTC. No measurements on the volumetric strain, pore air and water pressures

were taken in these tests. The results of 20 CUTC tests are summarized in Table 1. The test results illustrate the effects of initial water content and confining stress on the overall stress-strain response. Several distinct behaviours may be broadly identified according to their responses.

At low water content and confining stress (8.1% and 10 kPa; Fig. 8(a)), the specimen compacted at the dry side of optimum displays a sharp peak strength with severe softening. Multiple vertical tensile cracks were observed in the failed specimen. It appears that the significant drop in peak strength may be related to the energy released from the crack propagation. The residual strength even drops below those observed in specimens with high water content. Increase in water content from 8.1% to 10.5 % reduces the brittleness in terms of deformation modulus, peak strength and softening. Further increase in water content (from 10.5% to 11.2%) delays the occurrence of the peak strength, and softens the soil stiffness. Single or multiple inclined shear planes were visual on the circumferential surfaces of the sheared specimen (see pictures in Appendix). The failure surfaces were relatively smooth compared to those observed in uniaxial tension tests.

The high strength mobilized at low water content may be attributed to formation of flocculated structure with interlocking at the dry side of optimum during compaction. Though no measurement in volume change was made in the CUTC test, high shear dilation rate up to 0.58 was observed in direct shear tests on compacted Calgary till specimens with low water content and at low normal confining stress (Lun 2005). The high shear dilation enhances the strength, but promotes post peak-softening after the interlocked structure has been destabilized.

As the confining stress increases from 10 kPa to 30, 50 and 100 kPa, the axial strain for peak strength mobilization increases resulting in a decrease in deformation modulus. In addition,

the shear dilation rate may be suppressed by the confining stress inducing mild strain softening. This trend was also observed in the direct shear tests on compacted Calgary till specimens (Lun 2005). It appears that the applied confining stress inhibits the development of tensile cracks and promotes the shear failure mode. As the water content exceeds the optimum water content, the peak strength behaviour vanishes, and the specimen behaves as a ductile material with strain hardening. A large strain is required for full mobilization of the ultimate strength. Bulging with no visible shear planes was detected in the failed specimen (see pictures in Appendix). At high water content on the wet side of optimum, the clay particles are dispersed in preferential orientations resulting in a low strength, but a stable structure.

Among those 20 tests, three distinct stress-strain responses are characterized with unique failure modes. There exists a behavioural transition from tensile rupture to ductile yielding with strain softening in between as the water content and confining stress increase. Once the formation of tension crack or localized shear zones occurs in the test specimen, the global average deformation behaviour becomes non-unique after the peak.

Data of Figs. 8(a) – (d) were replotted in Fig. 9 to illustrate the trends of measured maximum deviatoric stress versus net mean confining stress. The measured maximum deviatoric stress or undrained shear strength of specimens compacted on the dry side of optimum increases with increasing net mean confining stress. However, a reverse trend was detected in tests with specimens compacted on the wet side of optimum, which may be due to the positive excess pore air and water pressures induced under undrained loading (see Discussion section). For compacted specimens with high water content (wet side) with confining stress above 50 kPa (two tests at 100 kPa with 13.6 and 16.3% water contents), the total principal stress ratio even drops below values of less than 3 (Table 1).

Model evaluation

The preconsolidation stress under saturated condition p_0^* for each compacted specimen was calculated from [5], and plotted versus its initial compacted void ratio in Fig. 10. The input parameters in [5] are listed in Table 2. Fig. 10 shows that the lower the compacted void ratio, the higher the value of p_0^* is. Compacted specimens with low void ratio or high bulk density are expected to exhibit higher strength and stiffness than those with high void ratio or low bulk density illustrating that the initial compacted void ratio is one of key parameters in controlling the strength-deformation characteristics of compacted soil. The effect of suction on the behaviour of compacted soil is embedded in the preconsolidation stress under unsaturated condition p_0 , which may be estimated from [8] and [9]. In this study, no tests were conducted to measure those parameters in [8] and [9]. This study reviewed the values of the parameters reported in literature and used those having geotechnical indices comparable to those of Calgary till (Table 2). A discussion on the influences of these parameters on the estimation of p_0 is provided in next section.

Fig. 11 presents the results of p_0^* versus p_0 for 20 specimens. The p_0 values are always greater than the p_0^* value, and the differences are due to the suction effect, i.e., higher the suction, larger the difference is. The suction effect produces an apparent increase in the preconsolidation stress. For the compacted specimens on the dry side of optimum, the increase in the preconsolidation stress is about 29-43%. This percentage increase reduces to about 9% when the compacted specimen moves to the wet side of optimum. Comparing with the effect of initial compacted void ratio, the suction may play a secondary role on the strength of compacted Calgary till.

Fig. 4 illustrates that the stress limits for tensile rupture, softening (bifurcation) and ductile yielding (p_{uc} , p_h and p_0) are function of suction. The relationship of $p_{uc} = \frac{q_{uc}}{n_c}$ and [14] were used to estimate these limits, and the results are shown in Fig. 12. Three regimes may be drawn according to their failure modes. Compacted specimens with p_i and suction fall below the stress limit p_{uc} would fail in tensile rupture. Increasing confining stress will shift the tensile rupture mode to strain softening. Low suction promotes strain hardening with yielding, and high confining stress enhances the ductility of the compacted specimen.

For 20 CUTC test specimens, their measured suction and applied confining stress p_i were plotted in Fig. 12. The observed failure modes are compared with those predicted by the model. None of 20 compacted specimens fall inside the ductile yielding regime as the highest applied confining stress is 100 kPa only, i.e., the compacted specimens are overconsolidated. The model captures tensile rupture behaviour at low confining stress of 10 kPa. The regime bounded by the stress limits of p_{uc} and p_h signifies stress states at incipient instability, and bifurcation may happen. Within this bifurcation regime, the dominant mode should be characterized with peak strength and softening. However, test results reveal that both softening and ductile behaviours were observed in tests on compacted specimens with low and high water contents, respectively. For clarity, a line was drawn to divide them into two groups of strain softening and hardening accordingly (Fig. 12). These two distinct behaviours may be attributed to the fact that the effect of soil structure is not considered in the proposed model. Specimens compacted on the dry side of optimum display strain softening due to the formation of flocculated structure while specimens compacted on the wet side of optimum exhibit strain hardening due to the formation of dispersed structure. In the proposed model, the SWCC,

uniaxial tensile and unconfined compressive strengths shown in Figs. 6 and 7 are assumed to be independent of soil structure thereby affecting [15], [16] and [18].

To further investigate the effect of soil structure on the undrained shear strength, 20 CUTC test specimens were grouped into 3 ranges of water contents (8.1-8.3, 9.7-11.5 and 12.1-16.5%) corresponding to those on the dry, optimum and wet states, respectively. For each range, the p_i - normalized undrained shear strength was plotted versus p_0/p_i in Fig. 13 to determine the correlation. Linear trends observed in 3 test sets are in accordance with those predicted by [15] and [16]. Specimens compacted on the dry side of optimum display mostly tensile failure at low confining stress. Their p_i - normalized undrained shear strengths are much higher than those compacted at the wet side of the optimum, which is due to the combined effect of suction and soil structure. The p_i - normalized undrained shear strengths of specimens compacted at water contents close to optimum are slightly greater than those compacted at the wet side. This difference may be due to the effect of soil structure. Compacted specimens with optimum and wet water contents fail mostly in shear softening and ductile yielding, respectively. Results shown in Fig. 13 demonstrates that a single SHANSEP equation is not possible to quantify the undrained shear strength of compacted soil as the undrained shear strength depends on consolidation history, suction, failure mode, and soil structure.

Discussion

Pore air and water pressure responses under isotropic confining stress

Application of confining stress to a compacted specimen under undrained condition causes increases in pore air (u_a) and water (u_w) pressures simultaneously. The proposed model assumes the increases are small. Fredlund et al. (2012) presented various methods to compute the changes in pore pressure in unsaturated soil specimens. A simple procedure outlined by Hilf

(1948) was used in this study to compute the changes in u_a in compacted specimens. Hilf (1948) assumed that the matric suction is negligible, and the volume change in soil matrix induced by the net change in total stress and pore air pressure is equal to the volume change in pore. The relationship between the changes in pore air pressure (Δu_a) and total stress ($\Delta \sigma$) may be given by Hilf's solution:

$$[19] \quad \Delta u_a = \frac{1}{1 + \frac{(1 - S_0 + h S_0 n_0)}{(u_{a0} + \Delta u_a) m_v}} \Delta \sigma$$

where S_0 , n_0 = initial saturation and porosity, respectively; h = volume coefficient of air solubility in water; m_v = soil compressibility; u_{a0} = initial pore air pressure (103 kPa). [19] was used to estimate Δu_a induced by the application of total stress in the compacted Calgary till specimens on the dry (void ratio and water content of 0.41 and 8.1%, respectively) and wet side (void ratio and water content of 0.54 and 16.5%, respectively) of optimum for $h = 0.02$ and varying m_v and u_{a0} values. The results were normalized with the applied total stress to compute the pore air or water pressure parameter since the pore air and water pressures are assumed to be the same. The pore water pressure parameter B_w was plotted versus m_v in Fig. 14. For $u_{a0} = 103$ kPa, the B_w values of both specimens increase with m_v , and the B_w values of specimens compacted on the dry side are lower than those on the wet side. As the u_{a0} increases from 103 kPa to 1000 kPa, the increase in B_w value is significant and nonlinear with m_v , because the pore air compressibility is reduced with increasing u_{a0} . The m_v values of compacted Calgary till specimens lie in a range of 0.1 - 0.2/MPa. From Fig. 14, the B_w values are 0.07 and 0.14 for compacted Calgary till specimens on the dry and wet sides, respectively. For application of 100-kPa confining stress used in CUTC tests, the increase in u_a and u_w may be less than 7-14 kPa.

Pore air and water pressure responses under confined undrained triaxial compression

Fredlund et al. (2012) also adopted pore air and water pressure parameters D_a and D_w to characterize the changes in u_a and u_w under undrained shearing for unsaturated soils. Knodel and Coffey (1966) measured u_a and u_w in undrained tests on low plasticity clay specimens. The physical properties of the cohesive soil are LL of 35%, PL of 15%, void ratio of 0.42, and water content of 12.6%, which are comparable to those of compacted Calgary till. The u_a and u_w increased with increasing deviatoric stress, and the increase in u_a was lower than that in u_w . The average values of D_a and D_w from three tests were estimated and plotted versus the average values of B_a and B_w observed in undrained isotropic compression in Fig. 15. The D_a and D_w values increase with increasing B_a and B_w values or saturation. From the pore pressure responses, three test specimens may be on the wet side of optimum. Similar situation may be encountered in the CUTC tests on compacted Calgary till with high water contents (those at 100 kPa with water contents higher than 11.5% shown in Fig. 8(d)). This explains why low total principal stress ratios (< 3) were observed in these tests as positive u_a and u_w were induced under undrained shearing. As the plot of B_a versus D_a follows closely with that of B_w versus D_w , the increase in suction might be small supporting the stress paths illustrated in Figs. 4. For other specimens compacted on the dry side of optimum, small negative excess u_a and u_w would be generated. The stress path under isotropic and confined undrained triaxial compression would lie on a (p, q) stress plane of constant suction.

Sensitivity of parameters in loading-collapse curve on preconsolidation stress under unsaturated condition

The suction causes an apparent increase in preconsolidation stress according to the loading-collapse curve of [8] and [9]. The relationship between p_0^* and p_0 is controlled by

several parameters such as $\frac{p_0^*}{p^c}$, $\frac{\kappa}{\lambda(0)}$, $\frac{\lambda(s)}{\lambda(0)}$, r and β . A literature review was conducted to assess the limiting values and ranges of these parameters (Table 2). The normalized $\frac{\kappa}{\lambda(0)}$ defines the ratio of the coefficient of compression to that of recompression under saturated condition, and falls within a range of 0.1 to 0.2 in typical clayey soils. The normalized $\frac{\lambda(s)}{\lambda(0)}$ defines the relative compressibility of the soil under saturated and unsaturated conditions which decreases with increasing suction, and is always less than unity. The parameter r is the lower bound of $\frac{\lambda(s)}{\lambda(0)}$ for $\lambda(s = \infty)$. As $\lambda(s = \infty)$ is limited by the coefficient of recompression κ , thus, the values of r cannot be less than 0.1 to 0.2. For $r = 1$, $p_0 = p_0^*$ from [8] implying the suction does not produce an apparent increase in preconsolidation stress. The value of parameter r could be greater than unity for collapsible soils (Chiu and Ng 2003). The typical values of r may be 0.5 - 0.75. The β parameter controlling the rate of changes in soil stiffness with change in suction is estimated from curve fitting experimental data. Experimental evidences support that $\frac{p_0^*}{p^c}$ lies within a narrow range of 2 – 4. A sensitivity study was performed to investigate the relative roles of the above parameters affecting the apparent increase in preconsolidation stress due to suction, and the results are shown in Figs. 16(a) and (b). It can be observed that the normalized $\frac{p_0}{p_0^*}$ increases with increasing $\frac{\kappa}{\lambda(0)}$, $\frac{p_0^*}{p^c}$ and β , but decreases with increasing r . It appears that r is the dominant factor controlling p_0 though all the above parameters do not show a wide variation in measured values except β . It is important to point out that for compacted soil, tests must be carried out to determine the above parameters for different soil structures at dry and wet sides of optimum. For practical implication, clay with high plasticity, heavily compacted at the dry side of optimum

may display high preconsolidation stress, but its consistency in performance may be more difficult to be attained and maintained in the field.

Concluding remarks

This paper develops a novel interpretation methodology to determine undrained shear strength of compacted (unsaturated) soil from tests which are readily carried out in any conventional geotechnical laboratory without advanced facilities. The effects of preconsolidation induced by compaction and saturation (suction) on undrained shear strength are interpreted by uniting concepts in critical state soil mechanics, Barcelona Basic model and SHANSEP method. New features are implemented in the novel methodology in order to capture the unique behaviour of compacted soil observed in undrained shearing. The interpretation technique was used in analysis of data obtained from confined undrained triaxial compression tests with no pore air and water pressure measurement on compacted Calgary till. It was found that besides compaction void ratio and suction, failure mode and soil structure induced by compaction on dry and wet sides of optimum also exert significant influence on the undrained shear strength of compacted soil. Three unique SHANSEP equations were derived to quantify the normalized undrained shear strength according to the three different failure modes of tensile rupture, strain softening and ductile yielding. Criteria based on consolidation stress and suction were also developed for occurrence of these failure modes. Compacted soil behaves differently in mechanical and hydraulic responses in three distinct failure modes. It is of practical importance to incorporate these important aspects in design and analysis of geotechnical engineered structures built with compacted soil.

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List of symbols

B_a, B_w	pore air and water pressure parameters under isotropic compression, respectively
D_a, D_w	pore air and water pressure parameters under triaxial compression, respectively
e	void ratio under normally consolidation stress p
e_i	current void ratio under consolidation stress p_i
e_{p_1}	reference void ratio at reference stress p_1
h	volume coefficient of air solubility in water
m	material constant
m_h, m_t	slopes of Hvorslev and tension limit surfaces, respectively
m_v	soil compressibility
M	slope of critical state line
n_0	initial porosity
n_t, n_c	slopes of stress paths for unconfined tension and compression test in (p, q) stress plane, respectively
$N(s)$	specific volume at reference stress p^c
p	net mean consolidation stress (the difference between the mean total stress and pore air pressure)
p_f, q_f	net mean and deviatoric stresses on CSL, respectively
p_h, q_h	net mean and deviatoric stresses demarcating brittle response from ductile response, respectively

p_i, p_0^*	current and maximum isotropic consolidation stresses, respectively
p_{uc}, q_{uc}	net mean and deviatoric stresses at failure for unconfined compression p_{ut} , respectively
p_{ut}, q_{ut}	net mean and deviatoric stresses at failure for unconfined tension, respectively
r	constant limiting the asymptotic maximum stiffness
R_0	preconsolidation ratio under saturated condition ($R_0 = \frac{p_0^*}{p_i}$)
R_s	preconsolidation ratio under unsaturated condition ($R_s = \frac{p_0}{p_i}$)
R_t	stress ratio demarcating rupture response from brittle response ($R_t = \frac{p_0}{p_{uc}}$)
s	soil suction
s_u	undrained shear strength
S	normalized undrained strength of normally consolidated clay
S_0	initial saturation
u_a, u_w	pore air and water pressures, respectively
u_{a0}	initial pore air pressure (103 kPa)
α_e	Hvorslev true friction angle in (p, q) stress plane
β	parameter controlling the rate of increase in soil stiffness with suction
$\lambda(s)$	coefficient of compressibility for a given suction s
λ, κ respectively	coefficients of compression and recompression curves under saturated condition, respectively
Λ_o	$= 1 - \frac{\kappa}{\lambda}$
Λ	$= 1 - \frac{\kappa}{\lambda(s)}$
v	specific volume ($v = 1 + e$)
σ_1, σ_3	major and minor principal total stresses, respectively
σ_t	isotropic tensile strength
σ'_{vc}	vertical consolidation stress

ϕ_{cs}	friction angle at critical state
ϕ_e	Hvorslev true friction angle in (σ, τ) stress plane

Appendix – Pictures on test specimens showing different modes of failure

Fig. A1. Pictures showing different failure modes (left to right) (i) tensile cracking under uniaxial tension, (ii) vertical cracking under unconfined compression, (iii) localized shear deformation under confined triaxial compression, and (iv) ductile deformation under confined triaxial compression

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List of figure captions

Fig. 1. Effect of suction on primary compression and recompression curves showing the relationship between the preconsolidation stresses under saturated (p_0^* ; $s = 0$) and unsaturated conditions (p_0 ; $s > 0$) (Alonso et al. 1990)

Fig. 2. Modified yield surfaces for saturated (p_0^* ; $s = 0$) and unsaturated (p_0 ; $s > 0$) conditions in (p , q) stress plane (Alonso et al. 1990)

Fig. 3. Schematic showing yield surfaces for saturated and unsaturated conditions and loading-collapse curve in (p , q , s) stress space in BBM (Alonso et al. 1990)

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Fig. 7. Uniaxial tensile and unconfined compressive strengths as function of matric suction for compacted Calgary till

Fig. 8. Results from confined triaxial compression tests on compacted Calgary till specimens with different gravimetric water contents at confining stress (a) 10 kPa, (b) 30 kPa, (c) 50 kPa, (d) 100 kPa

Fig. 9. Results of net mean confining stress versus maximum deviatoric stress from confined undrained triaxial compression tests on compacted Calgary till with different gravimetric water contents

Fig. 10. Estimated preconsolidation stress under saturated condition p_0^* versus initial void ratio for compacted Calgary till specimens (the water contents shown in legend are the target values)

Fig. 11. Estimated preconsolidation stress under saturated condition p_0^* versus preconsolidation stress under unsaturated condition p_0 (the water contents shown in legend are the target values). The difference between p_0^* and p_0 is due to the suction effect induced by matric suction stress

Fig. 12. Relationships of suction versus limiting stresses p_{uc} and p_h showing boundaries and regimes for different behavioral modes. Data of compacted Calgary till specimens are included for evaluation. A dotted line is used to divide those within the bifurcation regime into two groups of strain softening and strain hardening accordingly. Specimens compacted on dry side of

optimum with flocculated soil structure and wet side of optimum with dispersed soil structure exhibit strain softening and strain hardening, respectively

Fig. 13. p_i - normalized undrained shear strength as function of preconsolidation ratio under unsaturated condition for compacted Calgary till according to 3 different ranges in water content (8.1-8.3% on the wet side, 9.7-11.5% near the optimum and 12.1-16.5% on the wet side)

Fig. 14. Effect of coefficient of volume change on pore water pressure parameter obtained from Hilf's analysis for different cases

Fig. 15. Relationships between measured average pore air and water pressure parameters B_a , B_w , D_a and D_w under undrained isotropic and consolidated triaxial compression on unsaturated clay (Knodel and Coffey 1966)

Fig. 16. Relationships between suction and $\frac{p_0}{p_0^*}$ for varying $\frac{p_0^*}{p^c}$ and $\frac{\kappa}{\lambda}$ (a) $r = 0.25$; $\beta = 12.5 \text{ MPa}^{-1}$ (b) $r = 0.5$; $\beta = 12.5 \text{ MPa}^{-1}$ and (c) $r = 0.75$; $\beta = 12.5 \text{ MPa}^{-1}$

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Table 1. Results of confined undrained triaxial compression tests on compacted Calgary till specimens

Confining stress (kPa)	Initial void ratio	Water content (%)	Suction (kPa)	Maximum deviatoric stress (kPa)	Stress ratio $\frac{\sigma_1}{\sigma_3}$
10	0.541	8.1	33	265	27.5
10	0.429	10.5	29	267	27.7
10	0.41	11.2	23	226	23.6
10	0.41	12.1	20	212	22.2
10	0.413	12.6	17	135	14.5
30	0.527	8.3	32	324	11.8
30	0.41	11.3	22	261	9.7
30	0.417	12.7	18	177	6.9
30	0.432	13.6	16	135	5.5
30	0.472	15.3	12	69	3.3
50	0.541	8.1	33	385	8.7
50	0.452	9.9	26	300	7
50	0.41	11.3	24	210	5.2
50	0.452	14.5	15	100	3
50	0.514	16.5	9	35	1.7
100	0.501	8.8	29	440	5.4
100	0.464	9.7	25	365	4.65
100	0.406	11.5	24	300	5
100	0.432	13.6	17	115	2.15
100	0.506	16.3	10	55	1.55

Suctions of 3 to 58 kPa were recorded at depths of 0.3 m in the slope site during May – November 2001 (Lun 2005)

σ_1 , σ_3 = major and minor principal total stresses, respectively

Table 2. Index properties of soil types and model parameters for calculations of preconsolidation stresses under saturated and unsaturated conditions

Parameters	Lower Cromer till	Compacted kaolin CK1	Compacted kaolin CK2	Jossigny silt	Scaly clay	Compacted clay	Calgary till (this study)
LL (%)	25	39	37	37	58	43	35
PL (%)	12	27	9	19	30	24	20
< 2 μm	17	4	22	34	NA	53	15.5
Compacted e	0.44-0.66	0.78-0.915	1.015-1.098	0.613-0.649	0.49-0.52	0.68-0.74	0.41-0.541
e_{p_1}	NA	NA	NA	NA	NA	0.9	0.934
p_1 (kPa)	NA	NA	NA	NA	NA	1	1
$\lambda(0)$	0.066	0.14	0.065	0.203	0.083	0.08	0.088
κ	0.0077	0.015	0.011	0.0433	0.034	0.008	0.009
β (1/MPa)	20	16.4	20	0.3	2.07	12.5	12.5
r	0.25	0.26	0.75	0.45	0.74	0.75	0.75
p_0^* (MPa)	0.02	0.055	0.04	0.240	0.40	0.20	varied
p^c (MPa)	0.012	0.043	0.01	0.155	0.18	0.10	0.012
s (kPa)	10-240	20-100	50-200	200-800	0-600	100-300	3-58

Lower Cromer till (Maswoswe 1985)

Compacted kaolin CK1 (Josa et al. 1987)

Compacted kaolin CK2 (Karube 1986)

Jossigny silt (Cui and Delage 1996)

Scaly clay – compacted kaolinitic illitic clay (Airo Farulla et al. 2007)

Compacted clay (Yao et al. 2014)

Suctions of 3 to 58 kPa were recorded at depths of 0.3 m in the slope site during May – November 2001 (Lun 2005)

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Fig. 9. Results of net mean confining stress versus maximum deviatoric stress from confined undrained triaxial compression tests on compacted Calgary till with different gravimetric water contents

Fig. 10. Estimated preconsolidation stress under saturated condition p_0^* versus initial void ratio for compacted Calgary till specimens (the water contents shown in legend are the target values)

Fig. 11. Estimated preconsolidation stress under saturated condition p_0^* versus preconsolidation stress under unsaturated condition p_0 (the water contents shown in legend are the target values). The difference between p_0^* and p_0 is due to the suction effect induced by matric suction stress

Fig. 12. Relationships of suction versus limiting stresses p_{uc} and p_h showing boundaries and regimes for different behavioral modes. Data of compacted Calgary till specimens are included for evaluation. A dotted line is drawn to divide those within the bifurcation regime into two groups of strain softening and strain hardening accordingly. Specimens compacted on dry side of optimum with flocculated soil structure and wet side of optimum with dispersed soil structure exhibit strain softening and strain hardening, respectively

Fig. 13. p_i - normalized undrained shear strength as function of preconsolidation ratio under unsaturated condition for compacted Calgary till according to 3 different ranges in water content (8.1-8.3% on the wet side, 9.7-11.5% near the optimum and 12.1-16.5% on the wet side)

Fig. 14. Effect of coefficient of volume change on pore water pressure parameter obtained from Hilf's analysis for different cases

Fig. 15. Relationships between measured average pore air and water pressure parameters B_a , B_w , D_a and D_w under undrained isotropic and consolidated triaxial compression on unsaturated clay (Knodel and Coffey 1966)

Fig. 16. Relationships between suction and $\frac{p_0^*}{p_0}$ for varying $\frac{p_0^*}{p_c}$ and $\frac{\kappa}{\lambda}$ (a) $r = 0.25$; $\beta = 12.5 \text{ MPa}^{-1}$ (b) $r = 0.5$; $\beta = 12.5 \text{ MPa}^{-1}$ and (c) $r = 0.75$; $\beta = 12.5 \text{ MPa}^{-1}$

Fig. A1. Pictures showing different failure modes (left to right) (i) specimen for uniaxial tension test, (ii) cracking under uniaxial tension, (iii) vertical cracking under unconfined compression, (iv) localized shear deformation under confined triaxial compression, and (v) ductile deformation under confined triaxial compression

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