# Experimental Study of Wellbore Instability in Clays

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Experimental Study of Wellbore Instability in Clays

Naeem O. Abdulhadi, A.M.ASCE1; John T. Germaine, M.ASCE2; and Andrew J. Whittle, M.ASCE3

Abstract: This paper presents the results of an extensive program of laboratory model wellbore tests that have been performed to study wellbore instability in saturated clays. The tests were conducted on resedimented Boston blue clay (RBBC) anisotropically consolidated to vertical effective stresses up to 10 MPa by using two custom-built thick-walled cylinder (TWC) devices with outer diameters \( D_o = 7.6 \) and 15.2 cm. The experimental program investigated the effects of specimen geometry, mode of loading, strain rate, consolidation stress level, and overconsolidation ratio (OCR) on deformations of the model wellbore measured during undrained shearing. Results indicate that for normally consolidated clays most of the change in cavity pressure occurs at volumetric strains less than 5% after which the borehole becomes unstable. Increases in outer diameter and strain rate led to a reduction in the minimum borehole pressure. Stress-strain properties were interpreted overconsolidation ratio (OCR) on deformations of the model wellbore measured during undrained shearing. Results indicate that for normally consolidated clays most of the change in cavity pressure occurs at volumetric strains less than 5% after which the borehole becomes unstable. Increases in outer diameter and strain rate led to a reduction in the minimum borehole pressure. Stress-strain properties were interpreted by using an analysis procedure originally developed for undrained plane strain expansion of hollow cylinders. The backfigured undrained strength ratios from these analyses for normally consolidated specimens range from \( s_u / \sigma'_w = 0.19–0.22 \). Overconsolidation greatly improves the stability of the borehole, and interpreted undrained strength ratios from the TWC tests are consistent with well-known power law functions previously developed for elemental shear tests. DOI: 10.1061/(ASCE)GT.1943-5606.0000495. © 2011 American Society of Civil Engineers.

CE Database subject headings: Boreholes; Clays; Laboratory tests; Model tests; Saturated soils; Shear strength.

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Introduction

Shallow oil reservoirs at depths less than 1,000 m are found within weak rock formations. Most of these formations are poorly lithified, typically consisting of hard clays and lightly cemented granular materials. The wells are drilled within reservoir and overburden rocks that are much weaker and more deformable than those typically encountered in deep rock reservoirs (depths greater than 3,000 m). Most existing deep wells pass vertically through these upper weak sediments and are usually successfully cased and cemented to mitigate effects of minor near-surface soil disturbance. In contrast, shallow field development concepts rely on a small number of surface drilling locations, with high-angle wells and complex directional trajectories. In these situations, control of the drilling operations is closely related to an understanding of wellbore instability mechanisms.

Wellbore stability methods commonly employed in the design of deep wells are established on assumptions appropriate to the strength and deformation characteristics of hard lithified rock formations (e.g., sandstones and mudstones). These materials fail in a brittle manner creating classic borehole breakout failure patterns in the rock (Santarelli and Brown 1989). The accuracy of these conventional models has been verified by numerous laboratory model experiments (e.g., Hoskins 1969; Santarelli 1987; Ewy and Cook 1990; Kutter and Rehse 1996; Haimson and Song 1998) and comparison with actual field drilling results (e.g., Edwards et al. 2004).

However, no comparably validated methods exist for predicting the stability of shallow boreholes drilled in hard clays and poorly lithified rocks (some work on soft shales has been reported by Wu 1991; Marsden et al. 1996). These materials are expected to undergo large plastic deformations (borehole squeezing), creating a more extensive zone of disturbance around the borehole. The amount of data regarding the strength and deformation properties of clays in the relevant range of consolidation pressures (2–10 MPa) is surprisingly limited. The research work undertaken by Bishop et al. (1965), Nakken et al. (1989), Taylor and Coop (1993), Petley (1994), Amorosi and Rampello (2007) among others, forms a major part of the available database. In addition, almost no model tests exist on borehole stability conducted on soils.

The present study addresses the problem of wellbore instability in soils through a comprehensive program of laboratory model borehole tests conducted on resedimented Boston blue clay (RBBC) under simulated wellbore in situ conditions. The testing program evaluates the effects of different key parameters on the borehole response during undrained shearing including specimen outer diameter, mode of loading, cavity volumetric strain rate, consolidation stress level, and stress history. The tests make use of two automated, high pressure thick-walled hollow cylinder (TWC) devices that have been developed to study the stability of vertical wellbore at reduced-scale. This paper describes the testing procedures and equipment used to carry out the experimental program, presents the data obtained from the model borehole testing, and analyzes the results by using cylindrical cavity expansion theories.
Experimental Program

Equipment

The model borehole experimental program was carried out by using two different TWC devices. The first small diameter device was employed to test annular specimens with inner diameter \( D_i = 2.5 \) cm, outer diameter \( D_o = 7.6 \) cm, and height \( H = 15.2 \) cm. The second apparatus has the same inner diameter \( D_i = 2.5 \) cm, an outer diameter \( D_o = 15.2 \) cm, and height \( H = 22.8 \) cm. Both devices allow for independent control of the vertical stress and the radial pressures acting on the inner and outer walls of the annular soil specimen and pore water pressure (i.e., 4-axis control).

The small diameter TWC apparatus (Fig. 1) has a steel chamber that allows it to operate at confining pressures up to 20 MPa. The large diameter TWC chamber is made of aluminum and can operate at pressures up to 10 MPa. The chambers in both devices enclose the annular specimens, base pedestals, top and bottom drainage lines, floating top caps, top and bottom annular platens, and annular porous stones. In each case, the soil specimen was sealed with internal and external custom-made latex rubber membranes. The top and bottom annular platens were used to seal the internal cavity membrane with the use of O-rings.

The small diameter TWC system was axially loaded through the use of screw driven mechanical load frame whereas the large diameter TWC system used a high capacity hydraulic load frame. The axial load was applied to the specimen through a hardened steel piston that enters the top of the TWC chamber through a double O-ring seal. An external load cell and displacement transducers were used to measure the axial load and displacement.

Three custom-designed actuators (pressure-volume controllers; Sheahan and Germaine 1992) were used to regulate precisely the pressure and measure the volume change in the external cell, internal cavity, and pore water in the specimen. The cell, internal cavity, and pore water pressures were measured by high performance diaphragm-type pressure transducers.

A closed-loop automated control was carried out with a PC and custom-designed software that allows for simultaneous control of external cell pressure, internal cavity pressure, back (pore water) pressure, and axial force based on signals from the transducers measuring the quantities. With this system, it is possible to perform all phases of the TWC test including initial pressure up, back pressure saturation, stress path consolidation, and undrained or drained borehole closure. Further details of the test equipment can be found in Abdulhadi (2009) and Abdulhadi et al. (2010a).

Resedimented Boston Blue Clay

The entire experimental program was conducted on RBBC, a soil resedimented from the laboratory from natural Boston blue clay (BBC), an illitic clay of low plasticity (CL) and medium sensitivity. RBBC exhibits characteristics very similar to many natural, unconsolidated clay deposits, including stress-strain-strength anisotropy, low to medium sensitivity, significant strain rate dependency, and fairly typical consolidation characteristics. Its reproducible behavior, 100% saturation level, and local availability make it an ideal research material to investigate fundamental aspects of soil behavior without having to take into account the variability of natural soils.

RBBC was prepared by mixing the dry soil powder with distilled, deaired water at 100% water content to produce a stable homogenous slurry. Sodium chloride was added to the slurry (to achieve a concentration of 16 g/l) to prevent segregation during sedimentation. The slurry was then vacuumed to remove air bubbles, placed in individual consolidometers and loaded incrementally to a prescribed maximum (preconsolidation) pressure \( \sigma'_{pc} \) before unloading to overconsolidation ratio, OCR = 4. Because the stress ratio, \( \frac{K_0}{\sigma'_{pc}}/\sigma'_{pc} \approx 1 \) for RBBC at OCR = 4 (Sheahan et al. 1996), the unloaded sample is close to a hydrostatic state of stress, and shear strains imposed on the sample as it is removed from the consolidometer are minimized. Further details on the resedimentation procedure are provided by Abdulhadi (2009).

The average index properties of RBBC used in this experimental program can be summarized as follows: plastic limit, \( w_p = 23.5 \pm 1.1\% \); liquid limit, \( w_l = 46.6 \pm 0.9\% \); plasticity index, \( I_p = 23.2 \pm 1.2\% \); specific gravity, \( G_s = 2.81 \); fine fraction (passing number 200 sieve), \( > 98\% \); and clay fraction \( < 2 \mu m \) was 56 \pm 1\%.

A comprehensive experimental program to characterize the elemental mechanical behavior of RBBC was performed. It included constant rate of strain consolidation and \( K_0 \)-consolidated undrained triaxial compression and extension shear tests performed over a wide range of consolidation stresses. Further details on the engineering properties can be found in Abdulhadi (2009) and Abdulhadi et al. (2010b).

Test Procedures

After consolidation was completed, the soil was removed from the resedimentation consolidometer and prepared for testing. The specimens were extruded from the plexiglass consolidometer with a hydraulic jack, taking advantage of the lubricant on its interior. The central core of the annular specimen was then formed by drilling with a drill press. The successive use of four different standard metal drill bit sizes (12.7, 19.05, 23.8, and 25.3 mm) created the model borehole with the specimen fixed in a mold. Small
differences in the diameter of the largest two, progressively-used drill bits ensured minimal disturbance to the final inner surface of the annular test specimen. Note that the soil specimen has the same outside diameter as the resedimentation consolidometer, and therefore the outer surface does not require trimming. Only the ends of the specimen were trimmed by using a hacksaw and blade.

A dry setup was adopted in the TWC apparatus to prevent the specimen from swelling. Paper filter strips were applied to the outer surface of the TWC specimen to improve distribution and drainage of pore pressures during initial saturation, consolidation, and loading. After completing the specimen setup, the water lines were vacuumed to remove the air and flushed with water. With the drainage valves closed, an initial isotropic stress increment was applied to obtain positive pore pressures within the clay. The specimen was then back-pressure saturated to 400 kPa by increasing the external cell, internal cavity, and back pressures while maintaining the measured initial effective stress constant with no deviatoric load. This process was essential to make sure that the specimens were fully saturated. The Skempton pore pressure coefficient, $B$, determined by incrementally increasing the internal and external pressures and measuring the back pressure, was generally greater than 95%.

The soil specimen was anisotropically consolidated into the normally consolidated (NC) stress range in two stages at an axial strain rate of 0.3%/h to a predefined stress ratio, $K_0 = 0.55$, on the basis of results from the triaxial test program (Abdulhadi et al. 2010b). The first stage of stress path consolidation in the TWC apparatus started from the hydrostatic condition and approached a target value, $K_0 = 0.55$ at $\sigma' = 0.8\sigma'_p$. The second stage involved stress path consolidation at $K_0 = 0.55$ to the specified maximum vertical effective stress ($\sigma'_{vm} = 1.5\sigma'_p$). The internal cavity and external cell pressures were interconnected during this procedure. Once the target maximum vertical effective stress was reached, the stresses were kept constant for 48 h to ensure minimal rates of volume strain before simulation of borehole closure. Overconsolidated (OC) specimens were unloaded to prescribed vertical consolidation effective stress values ($\sigma'_{oc}$) at an axial strain rate of 0.2%/h, and $K_0$ conditions were approximated by adjusting the cell pressure by using the following expression (Schmidt 1966; Ladd et al. 1977):

$$K_{0(OC)} = K_{0(NC)}(OCR)^{0.43}$$

where $K_{0(OC)}$ and $K_{0(NC)} = \text{target overconsolidated and normally consolidated earth pressure coefficients}$, respectively. The exponent 0.43 was obtained from previous data on RBBC.

After rebounding, final stresses were maintained for another 48 h. Because the reconsolidation in the TWC apparatus was carried out at stresses significantly higher than those imposed during initial preparation in the resedimentation consolidometer, effects of the setup disturbance were minimized.

Mechanisms of instability were introduced by reducing the internal cavity pressure within the model wellbore and keeping the external cell pressure and axial stress constant. Cavity contraction was performed by drawing out cavity fluid with the pressure-volume controller at a standard cavity volumetric strain rate of 10%/h. The tests were terminated at 20% cavity volumetric strain. The drainage valves were closed to maintain undrained conditions during cavity contraction (globally within the specimen) while the pore pressures were measured at the top and base of the specimen. One test in the series was performed by increasing the internal cavity pressure (pressuremeter mode) and keeping the external pressure and axial stress constant (see Table 1).

The original cavity pressure–cavity volume strain curves were corrected for system compliance (e.g., compressibility of cavity fluid and tubing), internal membrane resistance, and axial strain during borehole closure. The system compliance, which has a significant effect on the results especially for tests consolidated to high pressures, was measured by performing reference tests on dummy aluminum specimens in the small and large TWC devices. Several cycles of unloading-loading were performed in these reference tests to verify the repeatability of the results. Axial strains were taken into account in the calculation of the cavity volumetric strain assuming uniform (cylindrical) deformations of the specimen. Further piston friction and area corrections were made for the axial load measurement because the tests employed external load cells.

### Variables Investigated

The model borehole experimental program investigated the effects of the following variables on cavity deformations measured during undrained shearing: specimen outer diameter ($D_o = 7.6$ cm and 15.2 cm), mode of loading (cavity unloading/loading), cavity

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<th>OCR$^a$</th>
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$^aOCR = \frac{\sigma'_{vm}}{\sigma'_{oc}}$
volumetric strain rate ($\dot{c}_{v_{av}} = 0.5–60\%/h$), consolidation stress level ($\sigma'_{cv} = 1.5–10$ MPa), and stress history (OCR = 1–8). The effects of boundary conditions and drainage (drained and undrained tests) have also been evaluated and the results are discussed by Abdulhadi et al. (2010a).

Table 1 summarizes the anisotropically consolidated undrained (CAU) model borehole tests. For each test, the table provides the specimen initial dimensions, preshear consolidation stress conditions, and undrained borehole closure conditions. The majority of the tests were performed in the small diameter TWC apparatus. The “reference” test conditions most commonly used in the experimental program were OCR = 1, 10%/h strain rate, and internal cavity depressurizing. The model borehole tests were essentially consolidated in the TWC apparatus to one of four nominal maximum vertical effective stress levels: 1.5, 3, 6, and 10 MPa. These stress levels reflect the range of in situ stresses pertinent to shallow oil field developments. Two tests were performed in the large TWC apparatus to examine the effect of the outer boundary on test results. Additional tests were conducted on RBBC with OCR of 2, 4, and 8. Strain rate effects during borehole closure were also investigated by varying the cavity volumetric strain rate over approximately two orders of magnitude ($\dot{c}_{v_{av}} = 0.5–60\%/h$). The overall model borehole experimental program consisted of 24 TWC tests. Some of these tests were repeat tests in which the repeatability of the test results was demonstrated. The minimal variability in the repeat tests indicated that any variations observed in test results are solely linked to the variables investigated.

Overview of Model Borehole Behavior for Reference Testing Conditions

Fig. 2(a) shows the net internal cavity pressure inside the borehole, $p_i - u_0$ (where $p_i$ is the total internal cavity pressure and $u_0$ is the pore pressure at the start of shearing), versus cavity volumetric strain, $\dot{c}_{v_{av}}(= \Delta V/V_0)$, for a reference baseline model borehole test that was initially anisotropically normally consolidated ($K_{0(NC)} = 0.55$) to $\sigma'_{cv} = 10$ MPa in the small diameter TWC apparatus before undrained cavity contraction at average volumetric strain rate, $\dot{c}_{v_{av}} = 10\%/h$ (test TWC 7; Table 1). Fig. 2(a) shows the nonlinear relationship between the cavity pressure and volumetric strain. The shearing starts from net internal pressure, $[p_i - u_0] = 5.5$ MPa ($= K_{0(NC)}\sigma'_{cv}$). The bulk of the pressure drop occurs within the first 5% volume strain before the borehole becomes unstable (i.e., deforms without further reduction in cavity pressure) when the net pressure reaches $[p_i - u_0] = 1.5$ MPa at volume strains in the range $\dot{c}_{v_{av}} = 7–8\%$. The axial strain during this procedure was minimal, particularly at small volume strains, as shown in Fig. 2(b).

Fig. 2(c) shows the measured excess pore pressure, $u_c (= u - u_0)$, versus cavity volumetric strain owing to the reduction of the internal cavity pressure for the same reference test. The results show that the pore pressure increases continuously with the cavity volumetric strain. The pore pressures measured at the minimum net cavity pressure (corresponding to $\dot{c}_{v_{av}} = 7–8\%$) are $u_c = 0.8$ MPa and rise to $u_c = 1.1$ MPa at the end of shearing. Those pressures are relatively small compared with the consolidation stresses. However, the pore pressure regime in the TWC is very complex because nonuniform stresses and strains across the specimen wall (developed owing to the difference between the internal cavity and external cell pressures) generate large pore pressure gradients during undrained cavity contraction. By using the theoretical relationship presented in Bishop and Henkel (1962) between the equalization of nonuniform pore pressure $(1 - u'/u_0)$ and the time factor ($T = c_i t/h^2$, in which $c_i$ is the coefficient of consolidation, $t$ is elapsed time, and $h$ is the drainage length), the time to reach 95% pore pressure equalization is 60–80 min. Consequently, it is unlikely that the pore pressures are fully redistributed within the TWC specimen at the standard rate of shearing and were estimated to be only 80–90% equilibrated (less redistribution at beginning of shearing). Therefore, the measurements at the specimen ends [Fig. 2(c)] are considered to represent “average” pore pressure conditions across the width of the specimen.

The rate of shearing in the undrained tests is of great importance. Although very fast tests tend to better simulate the true undrained case (i.e., minimal pore water migration within the specimen), the resultant pore pressures measured are not very reliable. On the other
hand, tests performed at sufficiently slow rates allow for pore pressure equilibration within the specimen (i.e., representative pore pressure measurements) but violate the undrained assumption locally. Therefore, the standard rate (10%/h) was chosen to simulate the "true" undrained behavior as much as possible and at the same time obtain somewhat representative average readings of pore pressures. Subsequent sections investigate the effect of pore pressure redistribution on the test results and analysis.

Posttest observations and measurements of the TWC test specimen and radiography scans indicate that the cross-sectional radial deformations are uniform (i.e., little distortion of the cross section). In addition, vertical cracks were observed at the inside wall of the borehole for specimens initially consolidated to maximum vertical effective stresses, $\sigma'_{mv} \geq 6$ MPa.

The method used to interpret the model borehole tests is based on micro-meter cavity expansion theory. Silvestri (1998) proposed a method to obtain the stress-strain properties from measurements of cavity expansion in a thick-walled cylinder of clay for undrained plane strain conditions. The same interpretation method is also applicable to cavity contraction as the problem geometry is assumed to be unique and applicable to the entire soil mass, the strain distribution across the TWC wall results in shear stress variations. For example, Fig. 4 shows the shear strain distribution across the small TWC specimen wall at $\varepsilon_{cav} = 7.5\%$. At this volume strain, the shear strain at the inner wall $\gamma_a = 7.2\%$ and at the outer wall $\gamma_b = 0.8\%$. For the reference test (TWC7), the corresponding shear stress at the inner wall was $q_{ha} = 2.2$ MPa (reached failure) and at the outer wall $q_{hb} = 1.8$ MPa. The natural shear strains at the inner radius ($\gamma_a$), outer radius ($\gamma_b$), and distance $r(\gamma_a)$ are linked to the distortion parameter, $\chi$ (and hence $\varepsilon_{cav}$), through the following expressions (Silvestri 1998):

$$\gamma_a = \ln \chi$$

$$\gamma_b = \ln (1 - \beta + \beta \chi)$$

$$\gamma_r = \ln (1 - \beta_r + \beta_r \chi)$$

where $\beta = a^2/r^2$.

The interpreted shear stress-strain results for the reference test (small diameter TWC specimen; $\beta = 0.111$) are presented in Fig. 3. The $y$-axis in the plot is the shear stress, $q_{ha} = (\sigma_0 - \sigma_r)/2$, and the $x$-axis is the natural shear strain, $\gamma = |r'|/r|^2$, where $r'$ is the deformed state of the initial radial coordinate $r$. The results show that the shear-stress curve reaches a maximum shear stress, $q_{ha} = 2.2$ MPa at $\gamma \approx 3\%$ and exhibits almost perfectly plastic behavior beyond peak shear resistance (i.e., the material does not strain soften). The undrained shear strength ($s_u$) of the clay is equated to the maximum value of shear stress in the horizontal plane; $s_u = q_{hMAX} = (\sigma_0 - \sigma_r)/2$. This yields an undrained strength ratio, $s_u/\sigma_u' = 0.22$, which is similar to that obtained in a direct simple shear (DSS) mode. These results are consistent with previous knowledge of anisotropic strength properties of RBBC (Whittle et al. 1994). The initial portion of the stress-strain curve is not well defined (lack of enough data points), so it is not possible to estimate the initial shear modulus readily by this procedure.

Although the shear stress-strain curve obtained in Fig. 3 can be assumed to be unique and applicable to the entire soil mass, the strain distribution across the TWC wall results in shear stress variations. For example, Fig. 4 shows the shear strain distribution across the small TWC specimen wall at $\varepsilon_{cav} = 7.5\%$. At this volume strain, the shear strain at the inner wall $\gamma_a = 7.2\%$ and at the outer wall $\gamma_b = 0.8\%$. For the reference test (TWC7), the corresponding shear stress at the inner wall was $q_{ha} = 2.2$ MPa (reached failure) and at the outer wall $q_{hb} = 1.8$ MPa. The natural shear strains at the inner radius ($\gamma_a$), outer radius ($\gamma_b$), and distance $r(\gamma_a)$ are linked to the distortion parameter, $\chi$ (and hence $\varepsilon_{cav}$), through the following expressions (Silvestri 1998):

$$\gamma_a = \ln \chi$$

$$\gamma_b = \ln (1 - \beta + \beta \chi)$$

$$\gamma_r = \ln (1 - \beta_r + \beta_r \chi)$$

where $\beta = a^2/r^2$. 

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**Figure 3.** Interpreted shear stress-strain behavior for a typical test on NC RBBC in small diameter TWC apparatus

**Figure 4.** Interpreted shear strain distribution across small diameter TWC specimen at $\varepsilon_{cav} = 7.5\%$ ($\gamma_a = 7.2\%$)
Effect of Specimen Geometry

Abdulhadi et al. (2010a) demonstrated that the initial specimen height has minimal impact on the measured borehole response and pore pressures by testing small diameter TWC specimens \(D_0 = 7.6\) cm with \(H = 15.2, 11.5,\) and 8.2 cm (i.e., \(H/D_0 = 2, 1.5,\) and 1.1). On the other hand, increases in the outer diameter have a significant impact on the minimum borehole pressure and redistribution of pore pressures within the clay specimen, as illustrated in Fig. 5. These results were obtained from tests performed on small diameter \(D_0 = 7.6\) cm; \(H = 15.2\) cm) and large diameter \(D_0 = 15.2\) cm; \(H = 22.8\) cm) TWC specimens (both with \(D_i = 2.5\) cm). The RBBC specimens were anisotropically normally consolidated to \(\sigma'_{vc} = 6\) MPa before carrying out undrained cavity contraction.

The effect of the outer diameter on the interpreted shear stress-strain behavior is shown in Fig. 6 (with \(\beta = 0.111\) and 0.0278 for the small and large diameter specimens, respectively). The figure shows that both tests have comparable stress-strain curves and reach peak undrained strength, \(s_u = 1.25\) MPa (i.e., \(s_u / \sigma'_{vc} = 0.21\)). Test TWC23 (larger specimen) exhibits some strain softening after reaching the peak stress. These results demonstrate that the interpretation method produces an almost unique stress-strain curve for the soil that is independent of the specimen dimensions. Furthermore, the results confirm that the analysis is not very sensitive to the internal redistribution of pore pressures taking place within the specimen because more migration is expected to occur in the smaller specimen owing to the reduced drainage path length.

Effect of Loading Mode

Undrained cavity unloading (contraction) and loading (expansion) tests were performed on NC RBBC to evaluate the effect of loading mode on the model borehole response. The specimens were anisotropically normally consolidated to \(\sigma'_{vc} = 3\) MPa in the small TWC apparatus. Fig. 7(a) shows the net internal cavity pressure versus cavity volumetric strain. The volume strains for both borehole contraction and expansion are plotted with the same sign to facilitate comparisons. The contraction and expansion curves have very

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**Fig. 5.** Effect of specimen outer diameter on undrained model borehole behavior of NC RBBC: (a) net internal cavity pressure versus cavity volume strain; and (b) average excess pore pressures

**Fig. 6.** Effect of specimen outer diameter on interpreted shear stress-strain behavior of NC RBBC

**Fig. 7.** Effect of loading mode on undrained model borehole behavior of NC RBBC: (a) net internal cavity pressure versus cavity volume strain; and (b) average excess pore pressures
similar profiles (i.e., the evolution of net pressure with strain) even at small strains. The cavity loading test (TWC24) reaches a maximum $p_i - u_i = 2.7$ MPa whereas the unloading test (TWC9) has a minimum $p_i - u_i = 0.5$ MPa. Fig. 7(b) shows that the measured average excess pore pressures are virtually coincident for both tests ($u_e \approx 0.4$ MPa at end of the test).

Fig. 8 shows that the contraction and expansion tests have very similar interpreted shear stress-strain properties, reaching comparable maximum shear stress, $q_h \approx \pm 0.54$ MPa (i.e., $s_u/s'_{ec} \approx 0.185$). Because $q_h$ is defined as $(\sigma_0 - \sigma_\theta)/2$, the expansion test yields a negative shear stress ($\sigma_\theta > \sigma_0$). These results confirm that the stress-strain-strength properties are isotropic in the horizontal plane after stress path consolidation. Moreover, the behavior is in agreement with the predictions made by Aubeny et al. (2000) by using a generalized effective stress soil model, MIT-E3 (Whittle and Kavvadas 1994), for pressuremeter tests on Boston blue clay (BBC). The numerical analyses demonstrated that the undrained strength ratios obtained in a pressuremeter shear mode are approximately 0.2 (i.e., similar to those measured in a DSS mode), and the predicted stress-strain curves show almost perfectly plastic behavior beyond peak shear resistance. Ladd et al. (1979) also suggested that the pressuremeter undrained shear strength is similar to that of simple shear.

**Effect of Strain Rate**

The strain rate sensitivity of the undrained borehole behavior was evaluated by shearing NC RBBC specimens at cavity volumetric strain rates varying within approximately two orders of magnitude: slow rate (0.5%/h), moderate or standard rate (10%/h), and fast rate (60%/h). All three specimens were anisotropically consolidated to $\sigma_0' = 6$ MPa in the small diameter TWC apparatus before undrained cavity unloading. The slow test (TWC17) was aborted at cavity volumetric strain, $\varepsilon_{cav} = 10.5\%$. Fig. 9(a) shows the net cavity pressure versus cavity volumetric strain. The standard and fast rate tests have comparable borehole response whereas the curve for the slow rate test indicates higher net pressures at a given strain level. The borehole becomes unstable at $p_i - u_i = 1.2$ MPa for the slow rate test and at $p_i - u_i \approx 0.8$ MPa for the other two rates. In addition, the initial cavity stiffness increases slightly with increasing strain rate.

The average excess pore pressures measured during undrained borehole closure are shown in Fig. 9(b) for the three rates. The figure shows that the measured pore pressures increase with decreasing strain rate: $A\varepsilon_{cav} = 10\%$, the pore pressure, $u_e = 0.83$ MPa in the slow rate test, $u_e = 0.55$ MPa in the standard rate test, and $u_e = 0.35$ MPa in the fast rate test. The difference between the standard and slow rate tests is thought to be partly attributable to the pore water redistribution but mainly owing to the inherent "viscosity" of the soil skeleton (i.e., true rate dependence in soil behavior). Pore pressure measured in the fast rate test shows that very little redistribution occurs at the beginning of shearing and then increases toward the end of the test where it approaches the standard rate curve. In fact, the pore pressure decreases slightly to a negative value before increasing with volume strain because the soil at the inner wall is unloading (resulting in decreasing pore pressures), and negligible redistribution of pore pressures exists at the beginning of the test. The fast rate test thus simulates the closest condition to a "true" undrained shear case, whereas the pore pressures in the slow rate test are believed to be fully equilibrated. The pore pressures in the standard rate test are estimated to be 80–90% equilibrated.

The interpreted stress-strain curves are presented in Fig. 10. The curves are comparable for the three tests indicating that the strain rate (and hence, the degree of internal pore water migration) has minor effects on the interpreted stress-strain behavior. The difference in excess pore pressures generated in the three tests [Fig. 9(b)] is relatively small (especially when compared to the consolidation stresses) and does not appear to affect the contraction curves significantly nor the subsequent stress-strain curves. The standard and fast rate tests reach a peak shear strength, $s_u = 1.25–1.30$ MPa (i.e., $s_u/s'_{ec} = 0.215–0.221$), whereas the slow rate test attains a peak strength, $s_u = 1.15$ MPa (i.e., $s_u/s'_{ec} = 0.196$). The trends
in pore pressures and strength with strain rate are consistent with the observations made by Sheahan et al. (1996) in which it was observed that increasing strain rates are associated in the case of NC RBBC in triaxial compression with an increase of approximately 6% in strength and decrease of approximately 12–15% in pore pressures per log cycle of strain rate.

Effect of Consolidation Stress Level

A series of TWC tests have been performed on NC RBBC to investigate the effects of consolidation stress level on the undrained borehole closure behavior in the small TWC apparatus. The specimens were anisotropically consolidated to vertical consolidation effective stresses, $\sigma_{0v} = 1.5$–10 MPa. Fig. 11(a) shows the net internal cavity pressure normalized in relation to the initial radial stress, $p_i - u_o / \sigma_{i0}$ (where $\sigma_{i0} = K_{i(0NC)} \sigma'_{vc} = p_i - u_o$), versus cavity volumetric strain. The results demonstrate that the overall normalized behavior is similar for the four tests; the bulk of the pressure drop occurs within the first 3–4% cavity volumetric strain and the borehole becomes unstable when the net pressure ratio, $[p_i - u_o] / \sigma_{i0}$ = 0.25–0.35 at volumetric strains in the range $\varepsilon_{cav} = 5$–8%. The initial cavity stiffness ratio decreases slightly as stress level increases. Fig. 11(b) shows the average excess pore pressure normalized in relation to the vertical consolidation effective stress, $u_e / \sigma_{vc}$. The pore pressures increase with volumetric strain to a range $u_e / \sigma_{vc} = 0.10$–0.14, with lower values for the highly stresses specimens. These results show that the normalized pore pressure behavior is similar for the four tests.

Fig. 12 shows the normalized interpreted shear stress-strain behavior ($q_h / \sigma'_{vc}$ versus $\gamma$) for the four same tests. The curves are characterized by a peak shear strength reached at $\gamma = 2$–3% followed by almost perfectly plastic behavior beyond peak shear resistance. The undrained strength ratios range only from $s_u / \sigma'_{vc} = 0.19$–0.22, with lower values for tests at $\sigma'_{vc} = 1.5$–3 MPa.

Effect of Stress History

The effect of stress history on the undrained model borehole behavior was investigated by performing tests with nominal values of OCR: 1, 2, 4, and 8. The four RBBC specimens were initially anisotropically consolidated to a common maximum vertical effective stress, $\sigma'_{vc} = 6$ MPa. Three specimens were then unloaded to the target OCR before undrained borehole closure. The axial load was not controlled during borehole closure at OCR = 4 and 8 (Tests TWC14 and TWC16; Table 1). Keeping the axial stress constant requires the axial load to be reduced to compensate for the reduction in the internal cavity pressure acting on the underside of the top cap. This could not be performed because the preshear lateral stress ratio, $K_h = 1$ (i.e., $\sigma'_{h} / \sigma'_{i} \approx 1$) for these tests and the current TWC design cannot impose axial extension. This limitation also prevents 1-D swelling to OCR = 8. Instead, the TWC16 specimen was unloaded along a specified stress path to $K = 1$.
Fig. 13(a) shows the net internal pressure ratio \( \left( \frac{p_t - u_0}{\sigma'_v} \right) \) versus cavity volumetric strain. The results demonstrate that the borehole is able to sustain net internal pressures, \( \frac{p_t - u_0}{\sigma'_v} \leq 0 \). The internal bore pressure at the end of the test decreases from \( \frac{p_t - u_0}{\sigma'_v} = \sigma'_0 \) to \( \frac{p_t - u_0}{\sigma'_v} = -0.55 \) at OCR = 8. Moreover, OCR = 4 and 8 RBBC specimens do not reach limiting values of \( \frac{p_t - u_0}{\sigma'_v} \) even at large volumetric strains. Fig. 13(a) also shows a significant increase in borehole stiffness at higher OCR. The normalized average excess pore pressures \( \left( \frac{u_e}{\sigma'_v} \right) \) are presented in Fig. 13(b). As OCR increases, the excess pore pressures decrease and become increasingly negative. The pore pressures at the end of the test decline from \( \frac{u_e}{\sigma'_v} = 0.10 \) at OCR = 1 to \( \frac{u_e}{\sigma'_v} = -0.66 \) at OCR = 8.

Fig. 14(a) presents the normalized interpreted shear stress-strain behavior \( \left( \frac{q_h}{\sigma'_v} \right) \) versus \( \gamma \) from tests at OCR = 1, 2, 4, and 8. In general, the undrained shear strength is mobilized at shear strains, \( \gamma = 2-4\% \), with no indication of postpeak strain softening. The test at OCR = 8 (TWC16) exhibits strain hardening behavior, with increased resistance throughout the test, reaching a maximum strength ratio, \( \frac{q_h}{\sigma'_v} = 0.95 \), at the end of shearing.

The undrained shear stress ratio increases with increasing OCR and can be described by using the SHANSEP power law equation (Ladd and Foott 1974)

\[
s_u/\sigma'_v = S(OCR)^m
\]

where \( S = \) undrained strength ratio of NC clay; and \( m = \) scaling exponent.

Fig. 14(b) summarizes the measured undrained strength ratios versus OCR for the CAU model borehole tests and \( K_0 \)-consolidated undrained \( (CK_0U) \) triaxial compression and extension tests performed on RBBC at high consolidation stresses (triaxial data obtained from Abdulhadi 2009). The value of \( S \) for the TWC is \( 0.21 \pm 0.01 \), corresponding to an average between \( S \) values measured in triaxial compression and extension tests, whereas the exponent \( m = 0.72 \) is consistent with elemental shear tests. These parameters are very similar to those obtained by Ahmed (1990) for RBBC from the DSS test (\( S = 0.20 \) and \( m = 0.74 - 0.82 \)). In addition, the results are in excellent agreement with the predictions made by Aubeny et al. (2000) by using the MIT-E3 constitutive model for the pressuremeter tests on NC and OC BBC.

**Summary and Conclusions**

A program of laboratory model borehole tests have been performed to study wellbore instability in clays by using two automated, high pressure thick-walled cylinder (TWC) devices with outer diameter \( D_o = 7.6 \) cm and 15.2 cm. Both devices allow for independent
control of external cell pressure, internal cavity pressure, back pressure, and axial force.

The present paper evaluated the effects of the following variables on borehole response of resedimented Boston blue clay (RBBC) during undrained shearing: specimen outer diameter, mode of loading, cavity volumetric strain rate, consolidation stress level, and stress history. The main findings can be summarized as follows:

1. Model borehole closure data on specimens of RBBC normally consolidated to high pressures indicated that most of the reduction in cavity pressure occurs at cavity volume strains less than 5% before the borehole becomes unstable. The shear stress-strain behavior, interpreted by using the framework proposed by Silvestri (1998) demonstrates that the soil reaches a maximum shear stress in the horizontal plane ($q_{0 \text{MAX}} = s_u$) at shear strain, $\gamma = 2-4\%$, and exhibits almost perfectly plastic behavior beyond peak shear resistance. The backfigured undrained strength ratios, $s_u/s_{vc} = 0.19-0.22$ are similar to those obtained in a direct simple shear (DSS) mode.

2. Increases in the outer diameter of the TWC specimen have a significant impact on the minimum borehole pressure and redistribution of pore pressures within the specimen. On the other hand, the interpreted shear stress-strain curves for the small and large diameter TWC tests yielded comparable results confirming that this stress-strain curve is unique for the soil.

3. The contraction and expansion curves from cavity unloading and loading tests had very similar evolution of pressure with volume strain even at small strains. The pore pressures were observed to be independent of the mode of loading. In addition, the interpreted shear stress-strain-strength properties were almost identical. The undrained strength ratios, $s_u/s_{vc} \approx 0.19$, are in agreement with previous studies on pressuremeter tests (e.g., Ladd et al. 1979; Aubeny et al. 2000).

4. The effect of increasing strain rate in the undrained borehole closure tests was to decrease the minimum net cavity pressure, increase initial cavity stiffness, and decrease pore pressure development. The shear stress-strain curves were comparable for the three rates indicating that the strain rate has minimal effect on the interpreted stress-strain behavior.

5. The consolidation stress level has a small effect on normalized cavity contraction curves for NC RBBC, whereas the initial cavity stiffness decreased slightly as stress level increased. The measured normalized average pore pressures were comparable as were normalized shear stress-strain curves.

6. The measured undrained borehole response was shown to be considerably affected by the stress history of the material. As OCR increased, a significant improvement is shown in borehole stability such that OC test specimens were able to sustain net pressures less than zero. The excess average pore pressures decreased and become increasingly negative as OCR increased. The relationship between the undrained strength ratio and OCR is consistent with element tests, with SHANSEP equation parameters $S = 0.21$, and $m = 0.72$.

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**Notation**

The following symbols are used in this paper:

- $CAU = \text{anisotropically consolidated undrained shear test}$
- $CK_{0U} = K_0 \text{-consolidated undrained shear test}$
- $a, d = \text{inner radius, deformed inner radius}$
- $B = \text{Skempton’s pore pressure parameter}$
- $b, b' = \text{outer radius, deformed outer radius}$
- $c_r = \text{coefficient of consolidation}$
- $D_i = \text{specimen inner diameter}$
- $D_o = \text{specimen outer diameter}$
- $G = \text{specific gravity}$
- $H = \text{specimen height}$
- $I_p = \text{plasticity index}$
- $K = \text{lateral stress ratio}$
- $K_0 = \text{coefficient of lateral earth pressure at rest}$
- $K_{0\text{(NC)}} = \text{coefficient of lateral earth pressure at rest for NC soil}$
- $K_{0\text{(OC)}} = \text{coefficient of lateral earth pressure at rest for OC soil}$
- $m = \text{OCR exponent in SHANSEP equation for undrained strength ratio}$
- $OCR = \text{overconsolidation ratio, } \sigma_{um}/\sigma_{vc}$
- $p_i = \text{internal cavity pressure}$
- $p_e = \text{external cell pressure}$
- $q_h = \text{shear stress in the horizontal plane, } (\sigma_0 - \sigma_r)/2$
- $q_{ha} = \text{shear stress in the horizontal plane at inner radius}$
- $q_{hb} = \text{shear stress in the horizontal plane at outer radius}$
- $r, r' = \text{radial coordinate, deformed radius}$
- $S = \text{undrained strength ratio for NC soil in SHANSEP equation}$
- $s_u = \text{undrained shear strength}$
- $u_e = \text{excess pore pressure}$
- $u_0 = \text{pore (back) pressure at start of shearing}$
- $V, \Delta V = \text{current volume, change in volume}$
- $V_0 = \text{initial volume}$
- $w_L = \text{liquid limit}$
- $w_p = \text{plastic limit}$
- $\beta = \text{size parameter}$
- $\gamma = \text{shear strain}$
- $\gamma_a = \text{shear strain at inner radius}$
- $\gamma_b = \text{shear strain at outer radius}$
- $\gamma_r = \text{shear strain at radius } r$ $\epsilon_{zz} = \text{axial strain}$
- $\epsilon_{cav} = \text{cavity volumetric strain}$
- $\dot{\epsilon}_{cav} = \text{cavity volumetric strain rate}$
- $\sigma_r = \text{radial stress}$
- $\sigma_{r0} = \text{initial radial effective stress (at end of consolidation)}$
- $\sigma_p = \text{circumferential (hoop) stress}$
- $\sigma_{p0} = \text{batch preconsolidation pressure}$
- $\sigma_e = \text{vertical effective stress}$
- $\sigma_{vc} = \text{vertical consolidation effective stress (pre shear)}$
- $\sigma_{vm} = \text{maximum vertical consolidation effective stress}$
- $\chi = \text{distortion parameter}$

**References**


