Hollow cylinder tests on natural structured soft clay: effects of sampling and sample preparation

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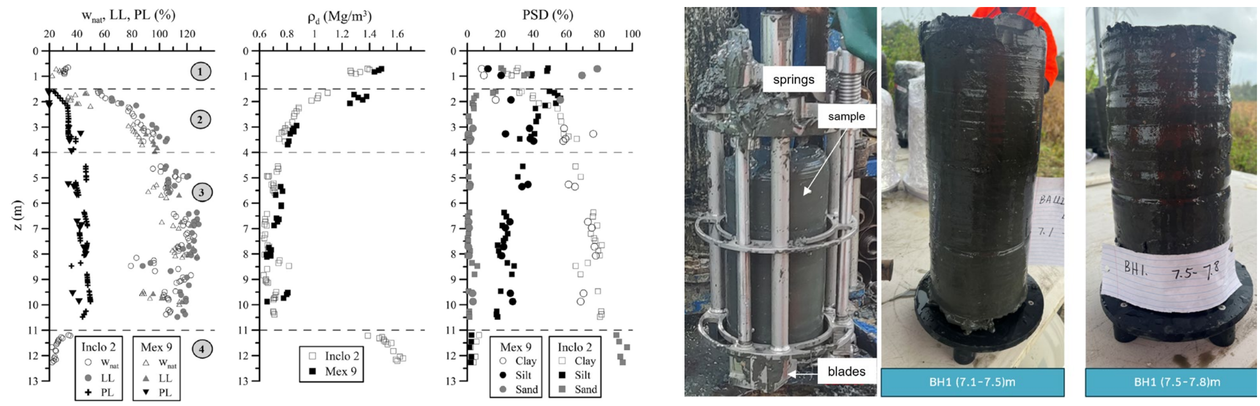
1 Introduction

One of the challenges in the characterization of natural structured soft soils lies in the selection of 'representative' soil specimens for laboratory testing. Sample disturbance as well as the natural soil variability may affect the results of laboratory tests and lead to important discrepancies between in situ and laboratory data. When it comes to Hollow Cylinder (HC) testing, those aspects become even more relevant. The large dimensions of HC specimens (typically 100 mm in outer diameter, 60 mm in inner diameter and 200 mm in height) prevent the use of conventional sampling techniques (tube samples). A convenient alternative is the use of block specimens like those obtained with the original Sherbrooke sampler [1] or the mini-Sherbrooke sampler [2]. The proved good quality of block specimens retrieved with those samplers [2, 3] gives confidence for their testing in the HC apparatus [4].

This extended abstract presents results of an experimental study aimed at evaluating the effects of sampling and sample preparation on the behaviour of block specimens of Ballina clay tested in the HC apparatus. A brief description of the sampling campaign and the assessment of sample quality, carried out via CT scanning, are presented. An important aspect evaluated in this study is the change in pore water pressure caused while boring of the inner cavity, and its potential impact on the stress-strain behaviour of tested samples. Results from HC anisotropic consolidation tests are presented to demonstrate the minimum disturbance of tested specimens when prepared according to the procedure presented below.

2 Soil tested and block sampling campaign

The soil tested in this study is Ballina clay, a marine soft clay encountered at the National soft soil Field Testing Facility NFTF [5, 6] in northern New South Wales. Ballina clay represents the soft soil deposits found along the eastern and southern coastlines in Australia. The profile at the NFTF is composed by four soil layers (Figure 1a). Layer 1 is a silty-sand alluvial crust. Layers 2 and 3 comprises the upper and lower estuarine silty clay (hereafter Ballina clay). Layer 4 is the silty-sand transition layer which overlies a sand layer and a deeper stiff clay layer. The natural water content increases with depth from 20% ($z \approx 1$ m) up to 120% ($z > 7$ m). The liquid limit is typically 10-15% higher than the natural water content whereas the plastic limit ranges between 20-53%. Natural dry density reduces from 1.50 Mg/m^3 at ground surface to 0.70 Mg/m^3 at 4.5 m depth. Overconsolidation ratio for the soft clay layers is lower than 2. Between 2 – 11 m depth the clay fraction is predominant with maximum values around 82%. A detailed characterization is presented in [7].



(a) (b)

Figure 1. (a) Soil profile at the NFTF [7]. (b) Block sampling campaign at the NFTF.

HC tests described in following sections were carried out on block samples retrieved in 2024 using the mini Sherbrooke block sampler between 5.5 – 8.5 m depth (Figure 1b) as part of a comprehensive experimental study on the anisotropic stress-strain-strength behaviour of natural soft soils [8].

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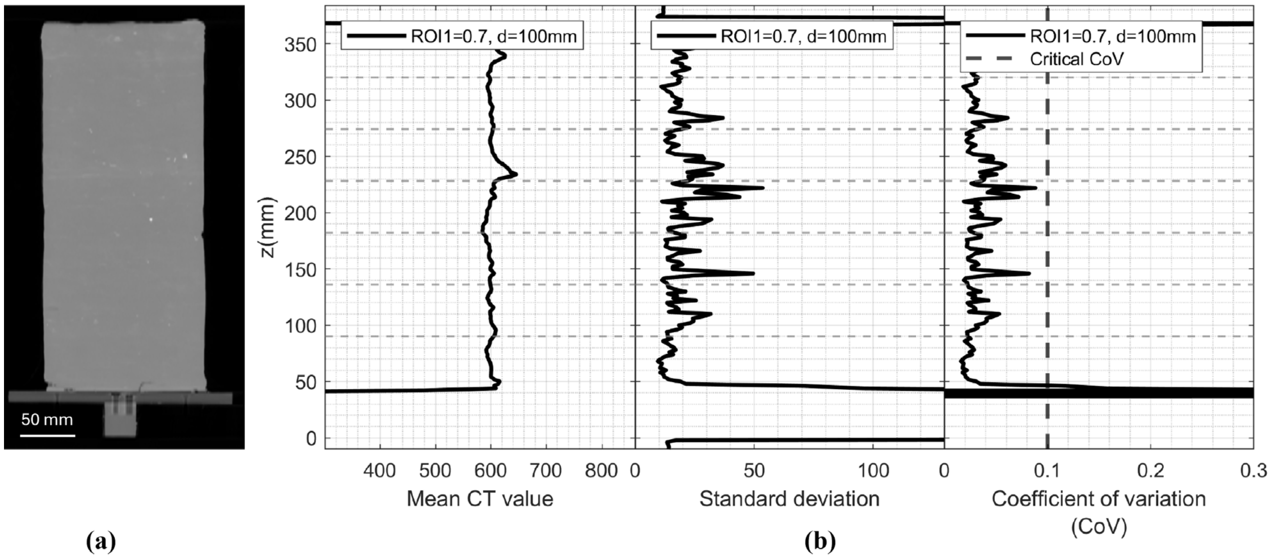
3 Non-destructive sample quality assessment

Before HC testing, the quality of each block specimen was evaluated via CT scanning using the methodology described in Pineda et al. [9]. Figure 2 shows profiles of mean CT value μ, standard deviation σ, and coefficient of variation CoV for a representative block specimen of Ballina clay retrieved from 7.8 < z < 8.1 m. Profiles in Figure 2 were estimated for a region of interest of RoI=100 mm, which represents an inner cylinder of 100 mm in diameter (equal to the outer diameter of HC specimens). The mean CT value ranges around 600 (Hounsfield units) and remains relatively constant along the block specimen. The standard deviation σ varies around 20 whereas the mean CoV varies around 0.03 (CoV<0.1) which according to [9] represents a soil specimen with low disturbance.



(a) **(b)**

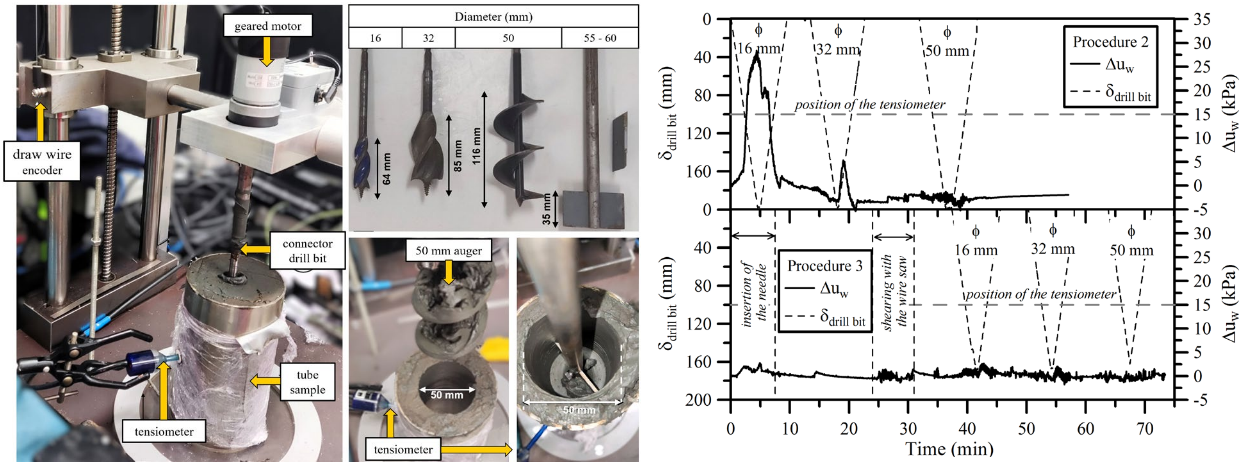
Figure 2. (a) CT scan of a block specimen. (b) Profiles of CT value, standard deviation and coefficient of variation.

4 Sample preparation

Most experimental set ups use drill bits to bore the inner cavity in HC specimens. The drill bit is rotated at the same time as advanced in the direction of the sample's axis, shearing the soil around. This process, assumed to occur under undrained conditions, induces changes in pore water pressure that are not measured, despite their potential impact on the test results. Figure 3a shows the motorized drilling device developed in this study for boring the inner cavity in Ballina clay specimens. This set up allows automatic control of the rotation speed and the vertical displacement of the drill bits. Figure 3a also shows four drill bits used in this study. This includes two 3-flute drill bits (16 mm and 32 mm in diameter), a 50 mm auger and a spade that may be adjusted to bore cavities between 50 – 60 mm.

In this study, boring tests with measurements of the pore water pressure were performed using good quality tube specimens retrieved with a 89 mm fixed-piston sampler. Those specimens, retrieved in 2014 as part of the laboratory characterization reported in [7], were carefully stored for eleven years under controlled relative humidity and temperature conditions following the recommendations described in [10]. As pointed out by Ouyang and Pineda [11], the mechanical properties of tubes specimens of very high plasticity Ballina clay retrieved with the fixed-piston sampler are not affected by long term storage. Due to the small diameter of the tube samples with respect to the outer diameter of a HC specimen (89 mm instead of 100 mm), the inner cavity was reduced to ~50 mm to maintain the 20 mm wall thickness of a standard HC specimen. Tube slices of 89mm in diameter and 200 mm in height were used in the boring tests. Three (3) different procedures were adopted for boring the inner cavity. Procedure 1 employed the 32 mm drill bit followed by the 50mm auger. Procedure 2 used the 16 mm and the 32 mm drill bits, the 50 mm auger and the 50mm spade. Procedure 3 included the following steps before using the drill bits: (i) the attachment of solid PVC plates with an inner hole of 20 mm in diameter to the top and bottom ends of the tube slice, (ii) the insertion of a 0.5mm steel wire connected to a needle through the centreline of the specimen, (iii) the connection of the steel wire to the arm of the wire saw, (iv) the rotation of the steel wire against the PVC plates aimed at shearing the soil within the 20 mm cavity. The two PVC plates were then removed, and the boring process was completed following the same steps described in Procedure 2. Changes in pore water pressure were monitored using a low capacity tensiometer ($-80 < u_w < 80$ kPa) with the tip located in the middle of the wall thickness (i.e. 10 mm away from outer and inner surfaces). The tensiometer was placed at the middle of the sample height, i.e. at h = 100 mm (see Figure 3a). Additional tests following Procedure 2 and Procedure 3 were performed using three tensiometers located at h= 50 mm, 100 mm and 150 mm.

Figure 3b shows the excess pore water pressure measured while boring of the inner cavity using Procedure 2 and Procedure 3. In Procedure 2, the use of the 16 mm drill bit causes an increase in pore water pressure of around $\Delta u_{w\text{-peak}} \approx$ 30 kPa. This value is measured when the drill bit reaches the bottom of the specimen. The pore water pressure then decreases upon retrieval. During the advance of the 32 mm drill bit, the pore water pressure shows a small reduction to $\Delta u_w \approx$ -2.5 kPa. The pore water pressure increases during the first part of the retrieval process, reaching a peak around 5 kPa and then reducing to a quasi-constant value around $\Delta u_w \approx$ -2.0 kPa. Interestingly, the use of the 50mm auger causes minor changes in pore water pressure. Results using Procedure 2 indicates that the response of the pore water pressure is strongly controlled by the volume of soil displaced ahead and behind the drill bits during their advance. After the retrieval of the 16mm drill bit, a large volume of remoulded soil was removed. That explains the small change in pore pressure during the advancement of the 32 mm drill bit. Upon retrieval, the soil remoulded accumulated behind the drill bit was then compressed upwards inducing the second peak in Figure 3b. The amount of soil remoulded by the 50mm auger, after boring with the two drill bits, was not large enough to induce important changes in pore water pressure. The inspection of Figure 3b indicates that Procedure 3, where the soil is remoulded by pure shear through the rotation of the steel wire, induces the lesser change in pore water pressure while boring of the inner cavity. Based on the results from the boring tests, Procedure 3 was adopted for the preparation of block specimens tested in the HC apparatus.



(a) (b)

Figure 3. (a) set up for boring the inner cavity in HC specimens. (b) Pore pressure response during boring the inner cavity using Procedure 2 and Procedure 3.

5 Hollow cylinder anisotropic consolidation tests

HC tests were carried out in a GDS® hollow cylinder apparatus suitable for testing specimens with 100 mm in outer diameter, 60 mm in internal diameter and a standard height of 200 mm. The cell has a maximum working pressure of 2 MPa. It is equipped with a submersible load/torque sensor (capacity of 3kN/30Nm) attached to the rigid top cap. Outer and inner pressures (p_o and p_i, respectively) and volumes (v_o and v_i, respectively) as well as back water pressure/volume are controlled/measured using pressure/volume controllers. The HC apparatus at the University of Newcastle Australia has been upgraded to test specimens with half of the standard height (100 x 60 x 100 mm) relevant in cases where only the consolidation behaviour is evaluated. The HC apparatus is also equipped with 3 local LVDTs for measuring axial displacements as well as 2 external lasers mounted on motorized linear guide rails to scan the outer diameter of the specimen during the test.

A total of eight (8) HC anisotropic consolidation tests were carried out on block specimens of Ballina clay. Specimens were recompressed to the in situ mean effective stress under a constant stress ratio η=q/p' aimed at reestablishing the stress state of the natural soil without changing its fabric. To do so, triaxial conditions were imposed by keeping outer and inner pressures equal during the tests (p_o=p_i). The intermediate stress parameter $b = (\bar{\sigma}_2 - \bar{\sigma}_3)/(\bar{\sigma}_1 - \bar{\sigma}_3)$ was kept equal to zero ($\bar{\sigma}_2 = \bar{\sigma}_3$) and no rotation of the major principal stress $\bar{\sigma}_1$ was applied ($\alpha = 0°$). Each HC specimen was subjected to: (i) saturation, and (ii) anisotropic consolidation. In the first case, p_o, p_i and back pressure u_{back} were ramped at 30 kPa/h until reaching values of p_o=p_i=300 kPa and u_{back} = 290 kPa. Values of Skempton's B parameter were always higher than 0.96 at the end of the saturation stage. The anisotropic consolidation consisted of two steps. First, an initial drained stage where the deviatoric stress was increased until reaching a value of η=q/p'=0.80 which corresponds to K_0=0.48 measured from seismic dilatometer tests for the natural clay with OCR $\approx$ 1.5 [6]. This was followed by a drained loading stage under constant q/p'=0.80 until reaching the in situ vertical stress of the tested sample. The sample quality index proposed by Lunne et al. [12] was used to evaluate the disturbance of HC specimens. This approach uses the change in void ratio Δe required to recompress the sample to the in situ mean effective stress normalized by the initial void ratio e_0. Lunne et al. proposed the following four levels of sample quality for soils with OCR<2: (1) *very good to excellent* ($\Delta e/e_0$ < 0.04), (2) *good to fair* (0.04 < $\Delta e/e_0$ < 0.07), (3) *poor* (0.07< $\Delta e/e_0$ <0.14) and, (4) *very poor* ($\Delta e/e_0$ >0.14).

Figure 4 presents representative results from two HC anisotropic consolidation tests. The following points deserve to be remarked. The in situ mean effective stress p'_0 of the blocks tested in the HC apparatus is very similar to the peak excess pore water pressure $\Delta u_{w\text{-peak}}$ reported in Figure 3b ($\Delta u_{w\text{-peak}} \approx p'_0$). This aspect highlights the importance of adopting a procedure for sample preparation that minimizes possible changes in the stress state of block specimens before testing. The volumetric strain developed at the end of the anisotropic consolidation stage remains lower than 3.0%. This corresponds to values of $\Delta e/e_0 < 0.04$ which in turn represents specimens of *very good to excellent* quality [12]. The volumetric behaviour of Ballina clay upon anisotropic consolidation is mainly controlled by axial strain ε_z with a minor contribution of the radial component ε_r and negligible influence of the circumferential strain ε_θ.

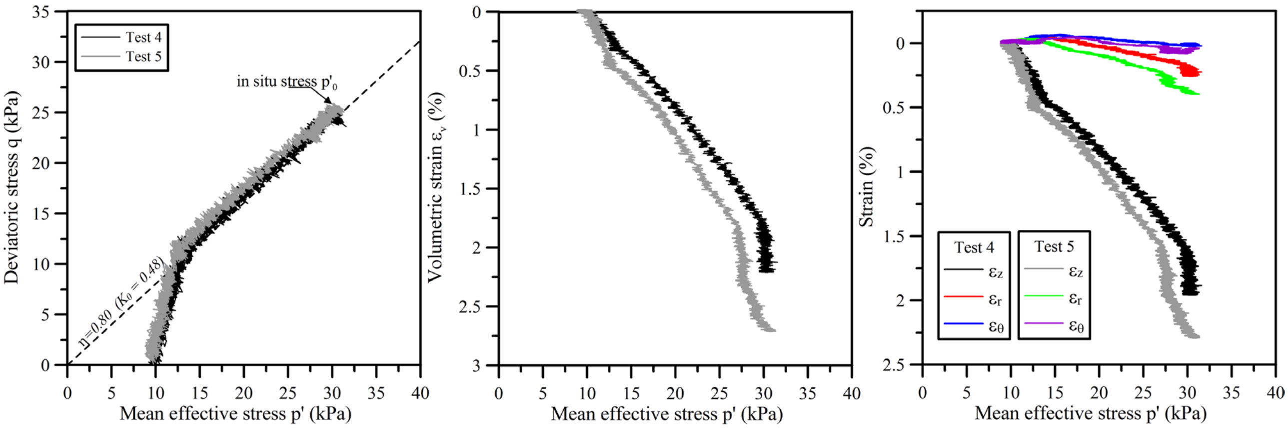


Figure 4. HC anisotropic consolidation tests on block specimens of Ballina clay.

6 Final remarks

This extended abstract discussed the influence of the sample preparation method on the quality of natural block specimens tested in the HC apparatus. Results from boring tests with measurements of pore water pressure demonstrated the importance of the excess pore water pressure generated while boring of the inner cavity which, in turn, may affect the initial state of HC specimens. A procedure for minimizing this effect was implemented which in turn produced excellent and good quality specimens according to a well-established sample quality index.

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