Hydro-Mechanical Characterization of an Organic Black soil

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ABSTRACT: This paper presents the results of an experimental study aimed at characterizing the hydraulic and mechanical behavior of the natural 'black' soils from the Liverpool Plains (NSW, Australia). Black soils correspond to the shallow organic and expansive materials that serve as foundation soil for a vast section of the inland freight rail in New South Wales. Black soils are commonly subjected to seasonal moisture variations which generate large changes in suction for soils located at shallow depths. This effect has important consequences not only on the water retention capacity but also on the saturated mechanical properties of the natural soil. The study presented in this paper includes the determination of the Water Retention Curve, the evaluation of the swelling and compressibility behaviour, and the estimation of the drained shear strength parameters. Microstructural analysis is used to evaluate changes in soil fabric upon saturation.

Keywords: Expansive soil; swelling behavior; compressibility; strength; microstructures

1. Introduction

Expansive soils are commonly subjected to seasonal moisture variations which generate large changes in suction for materials located at shallow depths. This effect has important consequences not only on the water retention capacity but also on the saturated mechanical properties of the natural soil which are needed for settlement and stability analyses.

This paper describes the results of a laboratory investigation aimed at characterizing the hydraulic and mechanical properties of an Australian organic black soil. The characterization study presented in the paper included the determination of the Water Retention Curve (WRC), the assessment of the swelling pressure and the swelling strain under 1D condition, the estimation of the compressibility parameters at saturated conditions, and the estimation of the drained shear strength parameters. Macroscopic results are complemented with a microstructural study aimed at assessing changes in soil fabric due to hydraulic and mechanical effects.

2. Materials

The material tested in this study is the natural black soil encountered at Liverpool (NSW, Australia). This is a natural expansive and organic stiff clay of very high plasticity which serves as foundation soil for the inland freight rail in New South Wales.

Laboratory tests described below were carried out on a hand-carved block specimen retrieved from around 1.0 m below the ground surface. The natural soil is mainly composed of dioctahedral smectite - either montomorillonite or beidellite (48.9 %), amorphous (26.6 %), quartz (11.2 %), plagioclase (6.5 %), k-feldspar (3.4 %), calcite (2.1 %) and kaolinite (1.3 %). Figure 1 shows the particle size distribution of the natural soil, estimated using a particle size analyzer [1] (Sedigraph III, Micrometrics). Clay, silt and sand contents are equal to 80 %, 17 % and 3 %, respectively. The density of solid particles is 2.63 Mg/m^3. An important feature of the Australian black soils is the presence of organics. According to the Loss of Ignition (LOI) method [2], the organic content is around 8%.

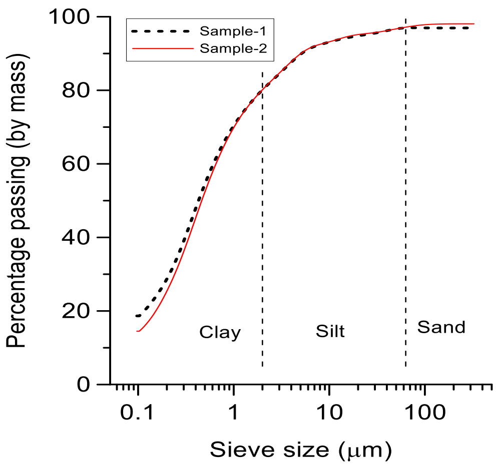

Figure 1. Particle size distribution chart

The electrical conductivity (EC) of the bulk material (solids + water) and the pore fluid were found to be 3.7 mS/cm and 13.5 mS/cm, respectively. The EC of the pore fluid is equivalent to a salinity of 12.8 gr of NaCl/L. The liquid limit, estimated using the fall cone test [3], varies around 95 % ±2 %. Negligible influence of previous soil drying and pore fluid salinity on liquid limit was observed for this soil. Plasticity index ranges around 58 % - 67 %.

The water content of the natural block specimens ranged around 27 – 32 %. It corresponds to a natural void ratio between 0.86 – 1.01 and degree of saturation about 82 % - 84 %.

Table 1. Index properties of tested material

Parameters	Value	Unit
ρ_{solids}	2.63	Mg/m^3
$EC_{pore\ fluid}$	13.5	mS/cm
EC_{bulk}	3.7	mS/cm
Liquid limit	95 ± 2	%
Plastic limit	37.65	%
Organic content (OC)	8	%
USCS Symbol	CE	
Particle size distribution		
Clay	80	%
Silt	17	%
Sand	3	%

3. Testing program

A series of mechanical tests were carried out to evaluate the water retention capacity, one-dimensional swelling and compressibility behavior as well as the shear strength parameters of the natural black soil. The microscopic tests were complemented with Mercury Intrusion Porosimetry (MIP) tests aimed at assessing changes in soil fabric during one-dimensional swelling and compression tests. A detail description of the main results is given in the following sections.

4. Water retention capacity

The water retention capacity of the black soil was evaluated via the Water Retention Curve (WRC). The WRC, defined in terms of the gravimetric water content w vs total suction ψ, was obtained by exposing the natural soil to a controlled drying-wetting path using the vapor transfer technique [4]. The estimation of the WRC followed a multi-stage approach. Different suctions were achieved upon drying by exposing samples to air drying at controlled laboratory room conditions for different lengths of time. When target values of water content change are reached, the samples were sealed and stored at controlled conditions for at least 48 h to allow them to reach suction equilibrium prior measuring the total suction using the WP4C dew-point psycrometer [5]. Soil wetting was induced by allowing the specimens to absorb water vapor within sealed containers filled with pure water. Different moisture contents were achieved by controlling the length of water vapor absorption. Moisture equilibrium (48 h) was also allowed prior suction measurement. Cylindrical specimens, 10 mm in height and 40 mm in diameter, were used in the WP4C apparatus. Drying and wetting stages were repeated until measured suction was close to the upper (300 MPa) and the lower (0.05 MPa) limits of the psycrometer, respectively.

Figure 2 shows the WRC for natural black soil. Total suction measured at natural water content is about 2.2 MPa. A linear relationship between water content and total suction is observed upon drying and wetting for suctions higher than 1 MPa. The WRC shows important hysteresis mainly at low values of suction (ψ < 10 MPa), where the macro porosity controls the water retention capacity of the soil [6]. The experimental data shown in Figure 2 have been fitted using the modified van-Genutchen model by Romero & Vaunat (2000)[6] defined as:

$$w = w_{sat} c(\Psi - \Psi_0)\left[\frac{1}{1+[\alpha(\Psi - \Psi_0)]^n}\right]^m \quad (1)$$

with:

$$C(\Psi - \Psi_0) = 1 - \frac{\ln\left[1+\frac{(\Psi-\Psi_0)}{a}\right]}{\ln(2)} \quad (2)$$

Where; w_{sat} and Ψ_0 refer the water content and total suction at saturated state, a represents the maximum suction at dry conditions, n is the slope of the inflection point, m is related to the residual water content where as α is mainly associated with the air entry value of the material (AEV) ($\alpha \approx$ 1/AEV). The correction function $C(\Psi-\Psi_0)$ takes into account the linear relationship between total suction and water content at high suction, a typical observed in stiff clays and rocks. Experimental measurements presented in Figure 2 were fitted using the same values for a = 700 MPa, m = 0.35, n = 0.75 and w_{sat} = 40%. Parameter, α took a value of 0.85 and 2.15 in drying and wetting paths, respectively. It means an air entry value, AEV equal to 1.18 MPa for the natural soil.

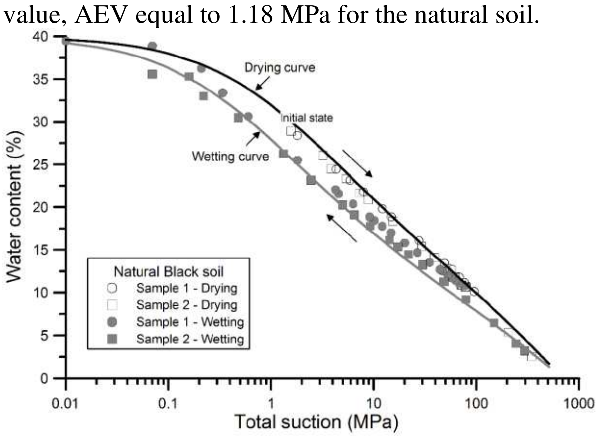

Figure 2. Water retention curve of natural soil (SWCC)

5. Swelling behavior

One-dimensional free swelling (FS) tests, swelling pressure (SP) tests and controlled deformation swelling pressure (CDS) tests were carried to study the swelling behavior of natural black soil. One-dimensional tests were carried out in a stainless steel oedometer cell (48 mm in diameter and 20 mm in height) that uses a 1 mm thick oedometer ring to trim specimens directly from either tube/block samples or thus minimizing sample disturbance.

In CDS tests, specimens were allowed to swell axially upon soaking until achieve pre-established values axial strains. At this point, the vertical movement of the piston was restricted and the axial load registered. All the specimens used in one-dimensional tests were saturated under constant total vertical stress of 30 kPa. A total of nine tests were carried out in this study. Table 2 summarizes the initial conditions of each specimen.

Table 2. List of tests

Test	Water content	Void ratio
	%	
FS-1	28.5	0.98
FS-2	28.4	0.92

SP-1	32.6	1.01
SP-2	28.5	0.91
CDS-1	28.7	0.91
CDS-2	27.5	0.92
CDS-3	26.8	0.84

Figure 3 shows the strain vs time curve obtained from free swelling (FS) tests. The two specimens reported in this figure have very similar initial water content ($w \approx$ 28.5%) which corresponds to a soil suction around 2.0 MPa (see Figure 2). The maximum swelling strain in the denser specimen is 6.52 %, more than three times larger than the value trimmed from the outer part of the block (2.0 %). The swelling mechanism upon wetting is controlled by suction (water content) and degree of saturation (which is affected by water content and void ratio). The similar water content of the two specimens reported in Figure 3 indicates that differences in initial void ratio are responsible for the important variation in the maximum swelling strain. Specimen with e_0 = 0.98 was trimmed from the periphery of the natural block specimen whereas the second sample (e_0 = 0.92) represents the materials form the central part of the block. In the former case, it is plausible to assume that disturbance caused by sampling may be responsible for the lower swelling potential of this specimen.

The results shown in Figure 4, obtained from swelling pressure (SP) tests, are consistent with Figure 3. Denser specimen shows a swelling pressure around 330 kPa. This value is around 3 times larger than the loose sample. Differences in void ratio and moisture content (or degree of saturation) have an important effect on the development of the swelling pressure. The higher water content and void ratio (and degree of saturation) of the specimen trimmed from the periphery of the block explains the faster achievement of the maximum swelling pressure, which took place in less than 1 day of soaking. More than 3 days were required to achieve the maximum swelling pressure in the dense specimen. It may be noted that the ratio of maximum swelling pressure between dense and loose specimens (~ 3) is very similar to the value obtained from the free swelling tests.

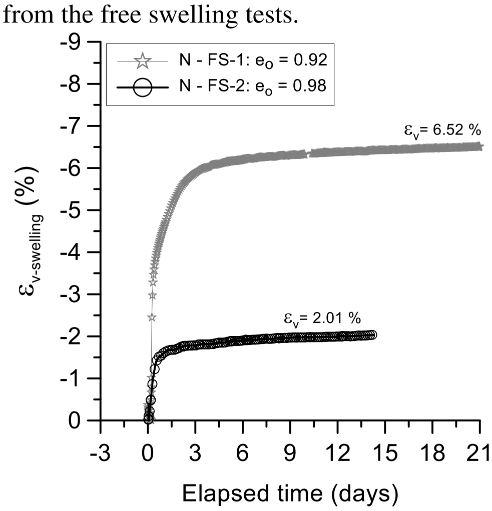

Figure 3. Time domain plot of free swelling(FS) tests

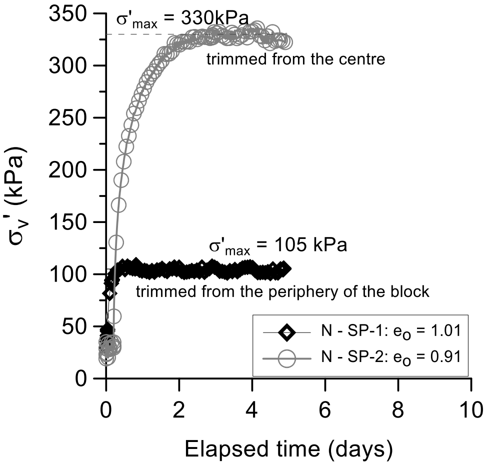

Figure 4. Time domain plot of swelling pressure(SP) tests

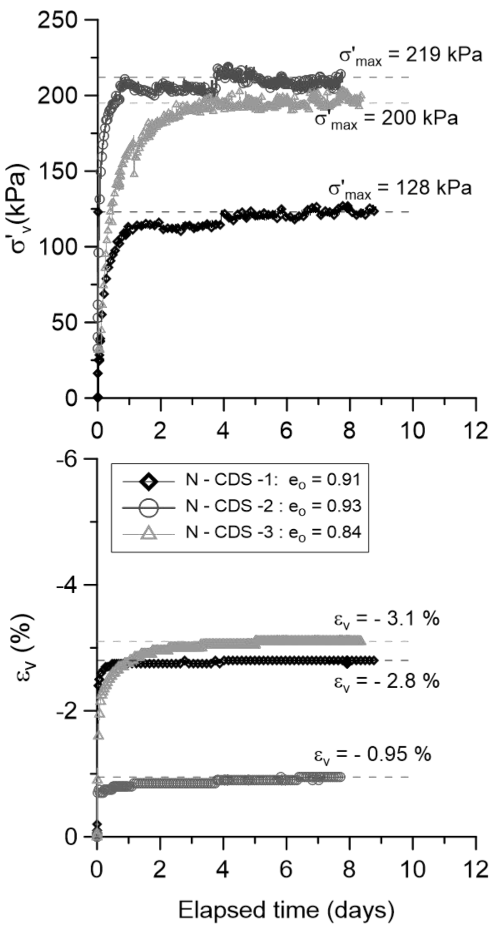

Figure 5. Time domain plot of controlled deformation swelling pressure (CDS) tests

The results of three controlled deformation swelling pressure (CDS) tests are presented in Figure 5. There, stress vs time and strain vs time curves are shown. In these tests, specimens were allowed to swell axially upon soaking until achieving a target value. Then, vertical deformation was restricted and axial load was recorded until achieving a constant value. Specimens with similar initial void ratio (see Table 2) were allowed to swell until achieving two different axial strains (-0.95 % and -2.8 %). A third sample, with a lower initial void ratio, reached an axial swelling strain of -3.1 %. For the two specimens with similar initial void ratio, higher swelling pressure (212 kPa compared to 123 kPa) was achieved by the sample subjected to the lowest axial expansion. The specimen with the lowest initial void ratio achieved the swelling pressure of 195 kPa despite it was allowed to swell more than the other two specimens.

The swelling behavior of natural black soil is summarized in Figure 6. A non-linear stress-strain relationship is observed for specimens with similar initial void ratio. Despite of the limited tests results presented in this figure, there is clear that the position of the stress-strain curve is affected by the initial void ratio of tested specimens. Figure 6 shows that small variations in the initial void ($\Delta e \approx \pm 0.1$) have a strong impact on the maximum swelling strain and swelling pressure.

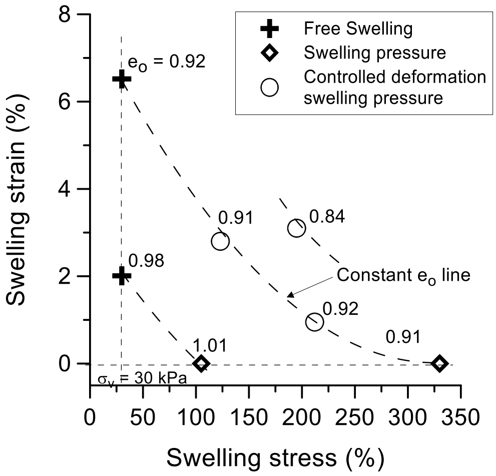

Figure 6. Swelling stress-strain curve

6. Compressibility behavior

Two Incremental Loading (IL) tests were carried out in this study to evaluate the compressibility characteristics of the natural black soil. IL tests reported here were performed in the oedometer cell described above. Initial conditions for the two tested specimens are summarized in Table 3.

Table 3. List of tests

Test	Water content	Void ratio
	%	
IL-1	28.0	1.05
IL-2	27.0	0.86

Despite the important differences in initial void ratio, which produces distinct swelling responses upon saturation, both specimens show a similar response once the preconsolidation stress, σ'_{prec}, is passed. The compressibility curve obtained from these tests are depicted in Figure 7. The normally consolidated range is characterized by a compression index, Cc, around 0.33. The preconsolidation stress is equal to 95 kPa for the specimen trimmed from the periphery of the block (IL-1, $e_0 \approx 1.1$). In the specimen obtained from the center of the block this value increase up to 300 kPa (IL-2, $e_0 \approx 0.86$). The unloading behavior is characterized by a swelling index, Cs, around 0.05.

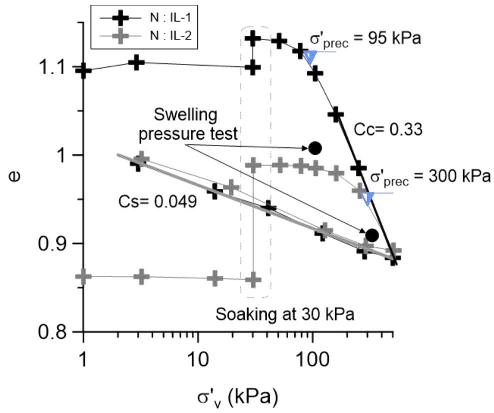

Figure 7. Compressibility curve

Figure 8 shows the variation of the vertical consolidation coefficient, Cv, with the stress level obtained for the two specimens used in the IL tests. Cv reduces with increasing the vertical effective stress from a maximum value of 0.8 m^2/year to 0.15 m^2/year. However, the reduction in cv with σ'_v is less marked in specimen IL-2 (lower initial void ratio). This suggests some dependency of the initial void ratio (and fabric) on Cv for natural black soil. Figure 9 shows the variation of the secondary consolidation coefficient, $C\alpha$, with the vertical effective stress. A clear dependency of the initial void ratio is observed in this figure. In the normally consolidated zone ($\sigma'_v < \sigma'_{prec}$) $C\alpha$ shows values within 0.001 – 0.02. Lower values are observed for the denser specimen (IL-2) which also suggest the influence of the soil fabric on the compression characteristics of black soil.

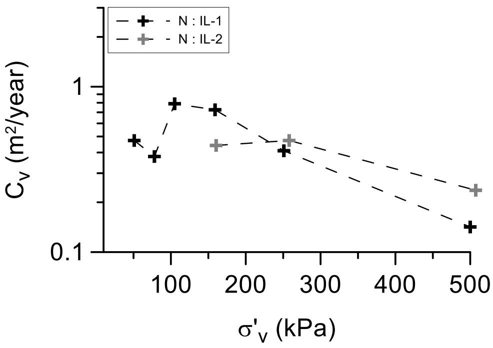

Figure 8. Variation of coefficient of consolidation

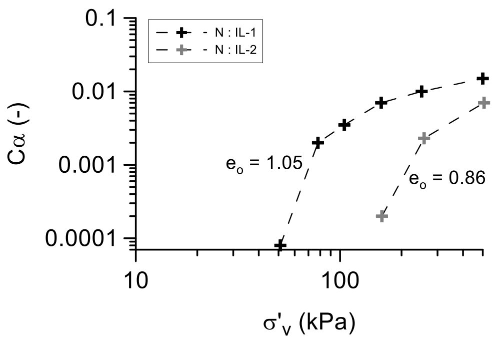

Figure 9. Variation of $C\alpha$

7. Shear strength behavior

Drained direct shear (DS) and drained ring shear (RS) tests were performed on natural and remoulded specimens, respectively, to evaluate the drained shear strenght parameters (c' and ϕ') of black soil.

Natural specimens were tested in a Geocomp® (Geocomp®, USA) direct shear apparatus. Cylindrical specimens (80 mm in diameter and around 30 mm height) were subjected to consolidation prior to shearing. Three vertical effective stresses were used in the consolidation stage (50 kPa, 100 kPa and 200 kPa). A consolidation time of 24 h was applied in all cases before shearing. Finally, each specimen was subjected to shearing under a low shearing rate ($5x10^{-3}$ mm/min) to allow the dissipation of any excess of pore pressure. The shearing stage finished when a horizontal displacement around 10 mm was achieved.

Drained residual shear strength of remoulded specimens was evaluated in a Bromhead ring shear apparatus [7]. Ring shear tests were perfomed according to the ASTM D6467-13[8]. Specimens were consolidated to vertical effective stresses of 27 kPa, 51 kPa and 100 kPa. A shearing rate of 0.24 °/min was adopted in order to ensure drained conditions. A summary of the shear tests is given in Table 4.

Figure 10 shows the shear stress vs horizontal displacement curves obtained from the direct shear tests on natural specimens. A britle behaviour is observed irrespective of the stress level applied. The ratio between vertical and horizontal displacements, δv/δh, indicates that dilation takes place at the beginning of the shearing phase in specimens consolidated to vertical stresses lower than 200 kPa. The specimen specimen loaded to 200 kPa shows a initial compression stage followed by dilation as the shear stress approaches the peak value

The shear stress vs horizontal displacement plots obtained from ring shear tests on remoulded soil are presented in Figure 11. Figure 11 shows a strong reduction in shear stress after reaching the peak value, which is achieved at horizontal displacements lower than 1 mm. Then, shear stress reduces around 50 %, irrespective of the confining stress applied. A constant value of shear stress (i.e. residual condition) is achieved for horizontal displacement higher than 25 mm. The strong brittle response observed in the ring shear tests may be attributed to the creation of a weak soil fabric, which is easily destroyed upon shearing, during the preparation of the remoulded specimens.

Table 4. List of shear tests

S.N.	Water content	Dry density	Void ratio
	%	Mg/m^3	
Direct shear test			
1	28.6	1.32	0.98
2	28.1	1.36	0.93
3	28.8	1.39	0.89
Ring shear test			
1	52	1.28	1.06
2	51	1.29	1.04
3	52	1.29	1.03

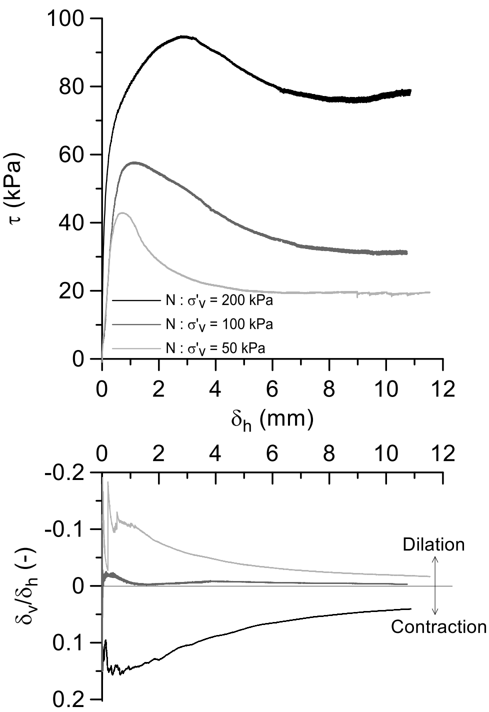

Figure 10. Stress-displacement curve during shearing stage

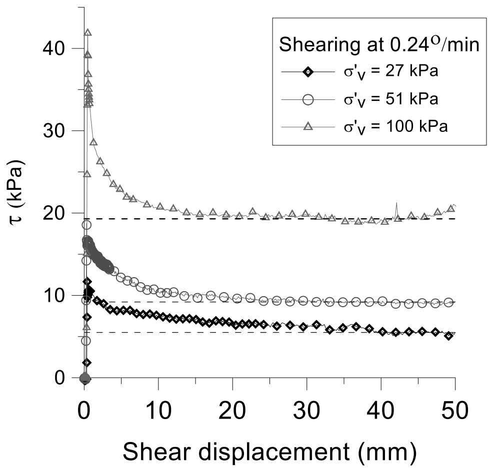

Figure 11. Stress-displacement curve during shearing stage for remolded soil

The drained shear strength envelopes obtained from direct shear and ring shear tests are shown in Figure 12. A linear Morh-Coulomb failure envelope has been used in this figure to obtain the shear strength parameters c′ and ϕ′. Peak shear strength envelope for natural black soil is characterized by ϕ′=20° and c′ = 22 kPa. Post-peak conditions from direct shear tests are also plotted in Figure 12. This condition is represented by ϕ′=20° and c′ = 0 kPa. On the other hand, residual conditions obtained from ring shear tests are characterized by ϕ′=11° and c′ = 0 kPa. These results indicate that the amount of shear displacement applied in the direct shear apparatus causes the erasement of the effective cohesion without affecting the frictional part of the shear strenght. On the other hand, the very low friction angle of the remoulded material is caused by the acuumulation of large horizontal displacement towards residual sliding shear-particle realignment. The residual friction angle obtained from ring shear tests is consistent with mineralogical composition of the black soil, in particular with the presence of montmorillonite.

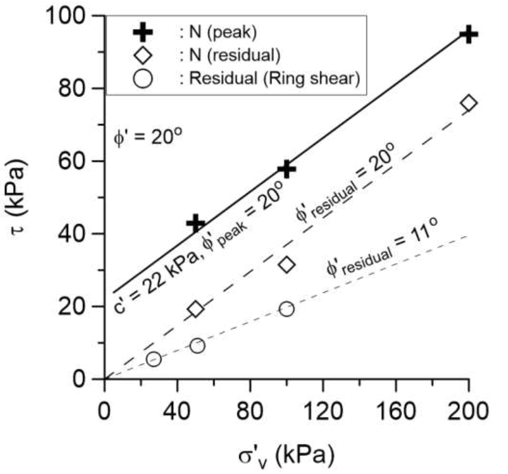

Figure 12. Shear strength envelope

8. Soil micro-structure

A microstructural analysis was performed in this study to evaluate changes in soil fabric during one-dimensional compression tests. Mercury Intrusion Porosimetry (MIP) tests were carried out on intact specimens as well as those subjected to FS, SP, CDS and IL tests. The principle of MIP is based on the Washburn equation [9], which relates the applied mercury injection pressure p to an equivalent entrance pore size d:

$$p = -\frac{4\sigma_{Hg}cos\theta_{nw}}{d} \tag{3}$$

where σ_{Hg} is the surface tension of mercury (0.484 N/m at 25° as adopted by Diamond (1970)[10]; Delage & Lefebvre (1984)[11] and θ_{nw} is the mercury-soil contact angle (assumed equal to 140° as adopted by Romero & Simms (2008)[12]. Values of the void ratio associated with intruded mercury are computed from the test results obtained during the intrusion stage as $e_{MIP} = V_{Hg}/V_{solids}$, where V_{solids} is the volume of the dry solids used for the MIP test and V_{Hg} is the cumulative volume of intruded mercury at the current pressure. The pore size density (PSD) function is estimated from the derivative of the cumulative intrusion curve according to:

$$f(\log x_m) = -\frac{\delta(e_{MIP})}{\delta(\log d)} \tag{4}$$

where log(xm) is the midpoint of the pore diameter class. MIP tests were carried out using an AutoPore IV 9500 porosimeter (Micromeritics®). The mercury intrusion pressure incrementally increased to the maximum value of 228 MPa (minimum entrance pore diameter of approximately 6.5 nm) using an intrusion rate of 0.001 mL/g.s. MIP tests were performed on small cubical sub-samples (≈ 5 mm side) trimmed from specimens previously subjected to air permeability tests. Freeze drying was first applied to the MIP samples as recommended by (e.g. Delage & Pellerin, 1984[13]). After treatment, the specimens were sealed in airtight plastic bottles and stored in a desiccator prior to microstructural analysis.

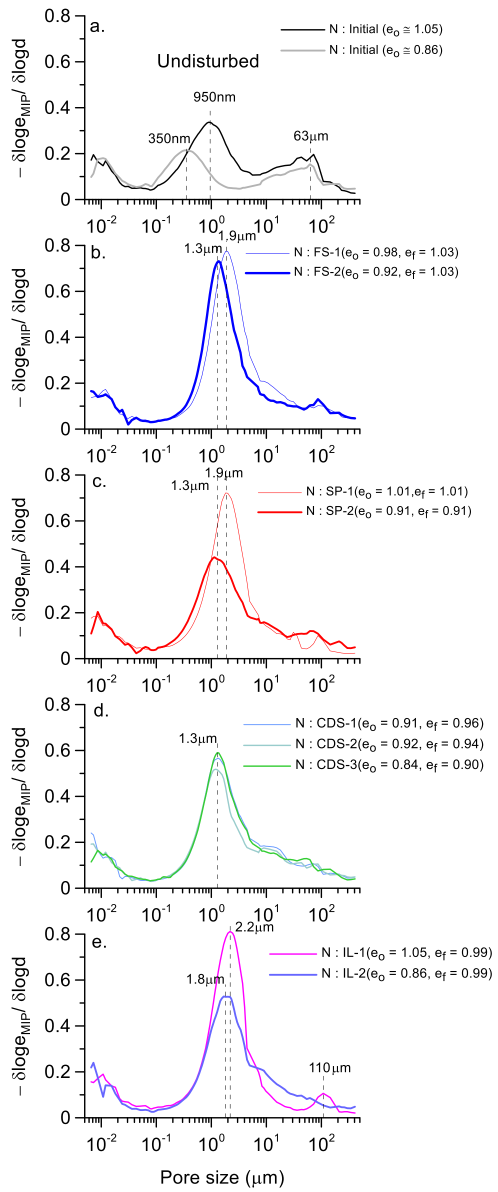

Figure 13. Pore size distribution (PSD) charts

Figure 13 shows the PSD curves obtained from MIP tests. The PSD of two intact specimens ar shown in Figuree 13(a), one with e_0 = 0.86 (trimmed from the center of the block specimen) and another sample trimmed from the periphery of the block (e_0 = 1.05). It can be seen that the natural soil is characterized by a bi-modal PSD. A common dominant pore size around 63 µm (macro pores) is observed for both specimens irrespective of their distict initial void ratio. The micro porisity is represented by dominant pore sizes equal to 0.35 µm and 0.95 µm for the dense and loos specimens, respectively. Figure 13(b) shows the PSD for two specimens subjected to free swelling upon soaking. Soaking induces a modification of the PSD towards a mono-modal distribution which is represented by a dominant pore size thar ranges between 1 – 2 µm, depending on the initial void ratio of the specimen. The saturation of the soil under free swelling conditions leads to the erasement of the macro pores and the increase in the size of the micro pores. It is interesting to note that the dominant pore size for specimens saturated under constant volume conditions, SP tests, (Figure 13(c)) as well as those subjected to controlled deformation swelling , CDS tests, (Figure 13(d)) is similar to the value for saturated soil under free swelling conditions. No important variations in PSD, with the exception of the change in the peak density, is observed for specimens used in incremental loading (IL) tests. In summary, saturation of black soil promotes the creation of a mono-modal PSD with dominant pore sizes around 1 - 2 μm. This value seems not affected neither by the boundary conditions during saturation and loading (i.e. FS, SP, CDS or IL).

9. Concluding remarks

An experimental characterization of a natural black soil from NSW (Australia) has been described in this paper. The mineralogical composition of this soil makes it prone to expand upon saturation. If expansion is restricted, swelling pressures above 300 kPa may be developed. A non-linear stress-strain behaviour represents the swelling behaviour of this soil with maximum swelling strains above -6 % and swelling pressure around 330 kPa. Minor increase in natural void ratio (for instance due to sampling disturbance or moisture content variation, leads to important reduction in both swelling strain and swelling pressure. Swelling upon saturation causes an irreversible change in soil fabric from a bi-modal towards a mono-modal pore size distribution, as demonstrated by MIP results.

The natural black soil tested in this study is represented by a friction angle of 20° and an effective cohesion around 22 kPa. Effective cohesion can be easily erased by the accumulation of small amount of shear displacements. The effective friction angle may reduce drastically towards a residual value of 11° with the accumulation of large shear displacements. Such a dramatic reduction in friction angle is consistent with the high proportion of montmorillonite in the black soil. This feature should be considered in the stability analysis of geotechnical infrastructure founded on black soil.

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