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The mechanical properties of a high plasticity expansive clay

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ABSTRACT

The mechanical properties of a high plasticity expansive clay from South Africa are reported. The experimental programme considered wetting after loading tests followed by one-dimensional consolidation and unloading of undisturbed and statically compacted samples. The results of this study illustrate that laboratory preparation of expansive clays do little to alter the soil's swell characteristics. This finding is attributed to the fact that, due to numerous swelling and shrinking cycles over a geological time frame, expansive clays tend to have reworked fabric in-situ.

The effects of structure are also assessed in both undisturbed and compacted specimens by comparison of the various consolidation tests with that of a reconstituted sample. The results illustrate a gradual yielding process for both undisturbed and compacted specimens, indicating progressive disruption of structure. After consolidation, while the expansion indices were found to be similar for both compacted and undisturbed samples, measured values were lower than that of the reconstituted specimen. Such a result is indicative of some preservation of structure after testing. It is also emphasised that consolidation tests on expansive clays are likely to exist in structure permitted space if swelling strains are restricted during the inundation process. Conversely, for most results presented it is seen that the swell caused by the inundation phase is approximately as disruptive to structure as laboratory preparation and compaction.

1. Introduction

Typically characterised as being largely composed of active clay minerals, expansive clays are soils which undergo significant swell or shrinkage upon wetting and drying respectively. The presence of swelling clays has caused severe economic implications in almost every continent in the world. Jones and Holtz (1973) reported how, in the USA, the annual damage to infrastructure caused by swelling clays amounted to twice that caused by hurricanes, tornadoes and earthquakes combined. Similarly, Jones and Jefferson (2012) have referred to swelling clays as being the most damaging geohazard in Britain, costing the insurance industry over GBP 400 million per year. Other similar accounts of the soil type have been made in China (Miao et al., 2012), Africa (Williams et al., 1985; Morin, 1971; Al Haj and Standing, 2015) and Australia (Li et al., 2014).

Apart from the tendency to undergo substantial volumetric changes upon wetting and drying, an important characteristic of swelling clays is their distinct 'structure'. The effect of soil structure was identified by Leroueil and Vaughan (1990) as being as important in determining engineering behaviour as the effects of initial porosity and stress history. Whilst the terms structure, fabric, microfabric and bonding are often used interchangeably, the usefulness of distinguishing these features have been highlighted by Toll and Ali Rahman (2010, 2017). The terminology used in this study is that proposed by Yong and Warkentin (1975) who defined *structure* as being the combination of *fabric*, which refers to the geometrical arrangement of particles, and inter-particle *bonding* which results from cementation and physico-chemical interactions. Leroueil and Vaughan (1990) highlighted how yield of structure is a function of strain or strain energy, and can be brought about by either shear, compression or swelling. Considering that expansive clays undergo seasonal swell and shrinkage by varying magnitudes throughout the depth of a profile, it can be recognised that the effects of soil structure are important in characterising this problem soil.

This study considers a black expansive clay sampled from the Limpopo province in South Africa, approximately 350 km northeast of Pretoria. The testing programme consisted of 1-dimensional swell and consolidation testing which aimed to assess the differences in the

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List of notations

σ_v'	vertical effective stress
e	void ratio
m_v	coefficient of volume compressibility
c_v	coefficient of consolidation
k_{sat}	saturated hydraulic conductivity
e_{100}^*	void ratio on ICL at vertical effective stress of 100 kPa
e_{1000}^*	void ratio on ICL at vertical effective stress of 1000 kPa
C_c^*	intrinsic compression index
e_L	void ratio at the liquid limit
C_c	compression index
C_e	expansion index
$\bar{p}$	net-mean stress
σ_{vy1}'	yield stress accounting for swell induced softening (determined from 1D compression)
σ_{vy2}'	predicted yield stress of an unstructured soil having not undergone swell (determined from 1D compression)

mechanical properties of the undisturbed and laboratory compacted samples.

Swell characteristics were assessed using two conventional approaches, namely wetting after loading (swell under constant load) and loading after wetting (swell followed by consolidation) tests (ASTM, 2014a). In addition to conventional consolidation tests which were performed on undisturbed and compacted samples, the intrinsic properties of the clay were measured on a reconstituted sample in accordance with the framework outlined by Burland (1990). Testing of a reconstituted specimen provides a useful reference framework against which the effects of soil structure of undisturbed and compacted samples were compared.

2. Sampling, preparation and testing procedures

The clay tested in this study was sampled from an expansive clay profile, from a depth of between 0.5 and 1.5 m below ground level. This soil was described in situ as being a stiff, fissured and slickensided black clay containing scattered fine nodular calcrete (Day, 2020). The site was visited after wet and dry seasons to determine the range of water content and matric suction that is likely to be experienced within a given year. These investigations illustrated seasonal variations in gravimetric water content and matric suction of approximately 30–40% and 4–2.5 MPa respectively. Measurements of matric suction were conducted using the filter paper method in accordance with ASTM D5298–16 (ASTM, 2016). Furthermore, the results of wax density testing indicated an average bulk density of the order of 1805 kg/m^3.

Basic classification tests were performed to establish the soil's particle size distribution (by method of sieving (ASTM, 2017a) and hydrometer (ASTM, 2017b), Atterberg limits (ASTM, 2017c) and specific gravity (ASTM, 2014a). These results, as well as the unified soil classification (ASTM, 2017d) are presented in Fig. 1 and Table 1. X-ray diffraction testing to determine the mineralogical composition of the clay was performed on the same site by a previous researcher, the results of which are shown in Table 2.

The experimental programme carried out considered undisturbed, compacted and reconstituted specimens. Undisturbed specimens were prepared from block samples. The reconstituted sample was prepared by producing a slurry at a water content of 1.1 times the soil's liquid limit (LL) in accordance with the framework outlined by Burland (1990). Following preparation of the sample, a one-dimensional consolidation test was performed. The results from this test allowed the intrinsic properties of the clay to be quantified and compared to a range of previously documented clays. A characteristic of the intrinsic compression line (ICL) is that it illustrates the characteristics of the tested clay without the effects of structure.

A typical feature of expansive clays in situ is that they tend to possess

Table 1
Soil classification data.

Liquid limit (%)	92
Plasticity index	55
Linear shrinkage (%)	25.5
Activity	0.8
Specific gravity	2.65
Unified soil classification	CH

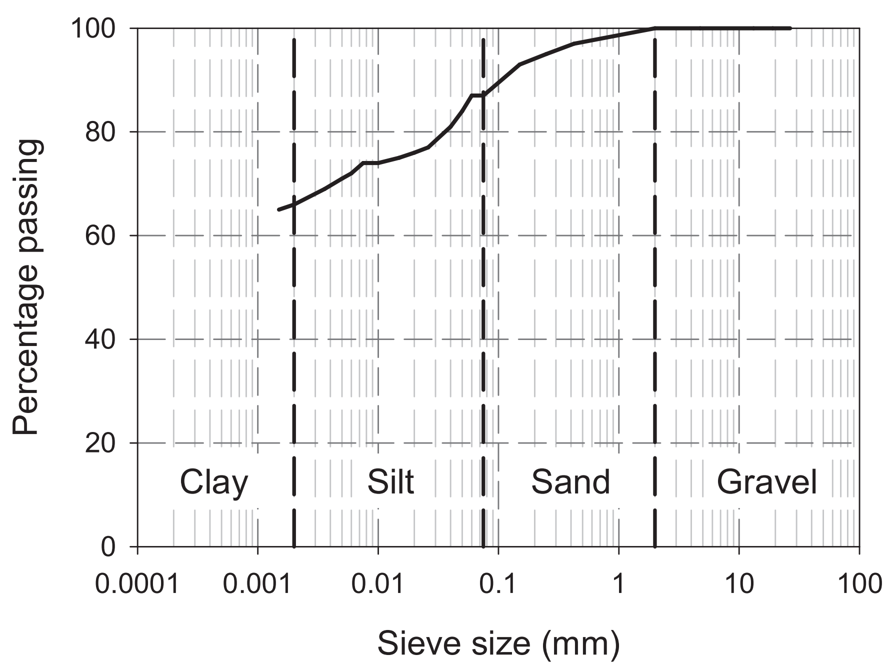

Fig. 1. Particle size distribution.

Table 2
Mineralogical composition based on X-ray diffraction (after Moses, 2008).

Mineral	Composition (%)
Smectite	58
Palygorskite	19
Calcite	5
Plagioclase	5
Quartz	4
Enstatite	4
Kaolinite	3
Diopside	2

a highly fissured macrofabric. This creates difficulties from the perspective of sampling since the clay tends to break apart along its fissures, creating a tendency towards stronger, less fissured samples (Thorne, 1984). Furthermore, this loss in macrofabric which occurs during sampling, can significantly affect measurements of soil properties related to the movement of water (Toll et al., 2018; Novak et al., 2000; Van Genuchten et al., 1999). Recognising these difficulties, preparation of the compacted samples was performed with the goal of producing samples with a fissured macrofabric. To achieve this, intact masses of clay were broken down with a cheese grater at their in-situ water content (approximately 31%) and statically compacted to a targeted dry density of 1350 kg/m^3. This created a macrofabric consisting of 'fissures' which were much more closely spaced than may be achieved from the retrieval of 'undisturbed' specimens.

The targeted initial water content and dry density were representative of the measured in-situ clay properties at the end of the dry season. The rationale for targeting in-situ properties related to this season is that it provides the most critical scenario if swell properties are to be measured (i.e. assessing the soil in its driest practical state allows for the largest estimates of swell magnitude and swell pressure to be obtained).

To allow for comparison of the swell characteristics, the initial void ratio and water content of the compacted specimens were kept as close to in-situ conditions as possible. The preparation procedure implemented in this study is in contrast to more typical approaches whereby air-dried soil is mixed with a predetermined quantity of water, allowed to equilibrate and compacted to a target density. A drawback of this more commonly used approach is that it results in a fabric with macropores which are relatively isolated. This is in contrast to the fabric type more commonly associated with expansive clays which has a series of interconnected pores (i.e. fissures) that more easily facilitate the ingress of water.

Stress paths describing the testing sequence for undisturbed and compacted samples are illustrated in Fig. 2 and can be described as follows. *Wetting after loading tests* (ASTM, 2014a, 2014b) or *swell under constant load tests* (as they are sometimes referred to in the literature) were conducted at various applied stresses. Such tests involve placing a sample in the oedometer at its in-situ water content, applying a predetermined stress (referred to as the soaking stress), and then flooding the oedometer housing with distilled water. Upon inundation, an increase in volume is typically expected at relatively low stresses (Path AB). However, if the soaking stress at the start of testing is greater than that required to keep the volume of the sample constant (i.e. swell pressure), then a reduction in volume will be observed.

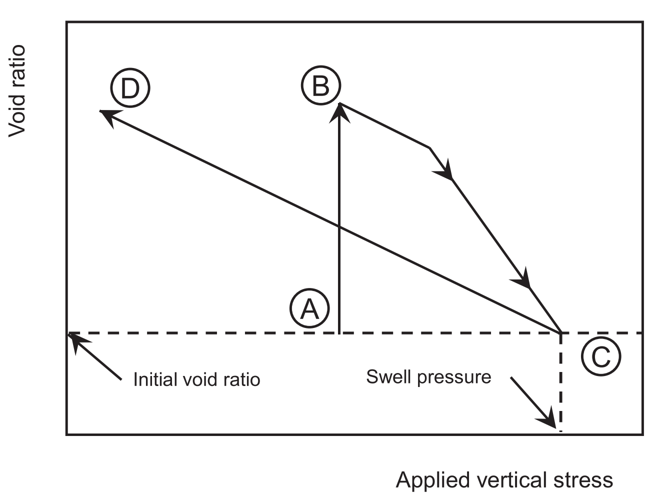

Fig. 2. Stress paths undertaken for all samples (excluding the reconstituted sample).

The purpose of conducting this test at a range of stresses is therefore to assess both the magnitude of swell expected at various depths, and to determine the stress required to completely prevent swell (i.e. the swell pressure). Predictions of swell at various depths which are based on such laboratory measurements are however, only applicable to that portion of the profile having similar properties to the samples tested. Site investigations revealed that the profile was homogeneous up to a depth of 5.5 m (corresponding to an overburden stress of approximately 100 kPa). Tests conducted at applied stresses exceeding this value served the purpose of establishing the swell pressure.

Following the swell tests, the same samples were subjected to one-dimensional consolidation (Path BC). For samples which underwent swell during the first phase of testing, consolidation was performed until volumetric strains reached or surpassed 0% (i.e. the initial sample volume prior to flooding). After consolidation, the samples were unloaded entirely, until only the weight of the top cap remained (Path CD). In addition to providing information on yield stresses and the slopes of normal consolidation and swelling lines, the consolidation tests performed on samples inundated at a soaking stress of 12.5 kPa served as a secondary check on the clay's swell properties. In accordance with ASTM D4546–14 (ASTM, 2014b), this approach to swell testing is referred to as the *loading after wetting* approach, however it is sometimes referred to as *swell followed by consolidation* (Schreiner, 1988) or the *single oedometer test* (Jennings et al., 1973). For samples which underwent compression during the flooding process, no additional stresses were added and, as for the other samples, stresses were incrementally removed. For these tests, only Path CD in Fig. 2 is applicable. Table 3 illustrates the initial properties of all samples tested.

3. Results and discussion

3.1. Consolidation test on reconstituted sample

Consolidation of the reconstituted sample was conducted up to a

Table 3
Initial sample properties.

Description	Soaking stress (kPa)	Void ratio, e_i	Gravimetric water content, w_i (%)	Degree of saturation, S_{r_i}
Compacted	12.5	0.969	33.6	91.9
Compacted	25	0.971	33.6	91.6
Compacted	50	0.908	30.3	88.5
Compacted	100	0.938	32.2	90.9
Compacted	200	0.973	34.7	94.4
Compacted	300	1.037	34.7	88.6
Compacted	400	1.027	34.7	89.4
Undisturbed	12.5	0.939	31.5	89.0
Undisturbed	25	0.888	30.3	90.5
Undisturbed	50	0.817	29.5	95.6
Undisturbed	100	0.889	30.2	90.2
Undisturbed	200	0.901	29.9	87.8
Undisturbed	300	0.992	30.3	81.0
Undisturbed	400	1.020	32.0	83.2
Undisturbed	500	1.068	30.8	76.3
Reconstituted	NA	2.481	98.5	105.2[a]

[a] This value can possibly be attributed to a slight error in the measured void ratio which can be problematic for a slurry.

maximum stress of 1 MPa, with an average of 48 h allowed between load increments. The results of this test are presented in Fig. 3. Apart from illustrating the relationship between vertical effective stress (σ_v') and void ratio (e) in Fig. 3(a), other fundamental parameters are presented in Fig. 3(b–d) such as the coefficient of volume compressibility (m_v), the coefficient of consolidation (c_v) (calculated using the root-time (Taylor's) method) and the saturated hydraulic conductivity (k_{sat}). The saturated hydraulic conductivity presented in the results was calculated using consolidation theory ($k_{sat} = c_v.\ m_v.\ \gamma_w$). While such a measurement of hydraulic conductivity is less accurate than measurements performed using the triaxial apparatus, it provides a rough indication of this clay's hydraulic conductivity to place it in context with other clays reported in the literature. The results presented in Fig. 3 are referred to as *intrinsic* clay properties since they are inherent to the soil type and independent of its natural state (Burland, 1990). Such a result is therefore useful in characterising various soil properties in the absence of structure. Since the correlations proposed by Burland (1990) have been shown to apply for a wide range of soil types, if the results of a consolidation test on a reconstituted sample can be shown to conform to the proposed framework, it eliminates the need to perform large numbers of tests (as highlighted by Al Haj and Standing, 2015).Arguably the most striking result from Fig. 3 is the extremely low saturated hydraulic conductivity (10^{-10} to 10^{-12} m/s). Considering the prominence of montmorillonite in this soil type, such a result can be expected for samples in an unfissured state.

The framework outlined by Burland (1990) is defined in terms of the intrinsic void ratios e_{100}* and e_{1000}* which correspond to the void ratios at vertical effective stresses of 100 and 1000 kPa respectively, and the intrinsic compression index, $C_c^* = e_{100}^* - e_{1000}^*$. In his original publication Burland (1990) reported that the relationships between these intrinsic parameters and the soil's void ratio at its liquid limit (e_L) were reasonably well defined. In a more recent publication, Al Haj and Standing (2015) added the results of two Sudanese clays to Burland's original dataset. This plot has been reproduced in Fig. 4 to determine whether the results of the South African clay tested in this study conforms to the original framework. The solid lines in Fig. 4 are the empirical relationships established by Burland (1990) while the broken lines present the work of Nagaraj and Srinivasa Murthy (1986) who established a relationship between the ratio e/e_L and σ_v' based on considerations of physical chemistry. It should be noted that the study conducted by Hong et al. (2010) considered intrinsic parameters for samples reconstituted at a range of water contents. However, for the purposes of this study, only those reconstituted at a water content of 1.3 times the clays' liquid limits (LL) were considered since they fall within the range specified by Burland (1990) (between LL and 1.5LL). Also included in Fig. 4 are results of Boston Blue Clay (Cerato and Lutenegger, 2004) and Kleinbelt Ton (Hvorslev, 1937).

The results presented in Fig. 4 illustrate that the intrinsic properties

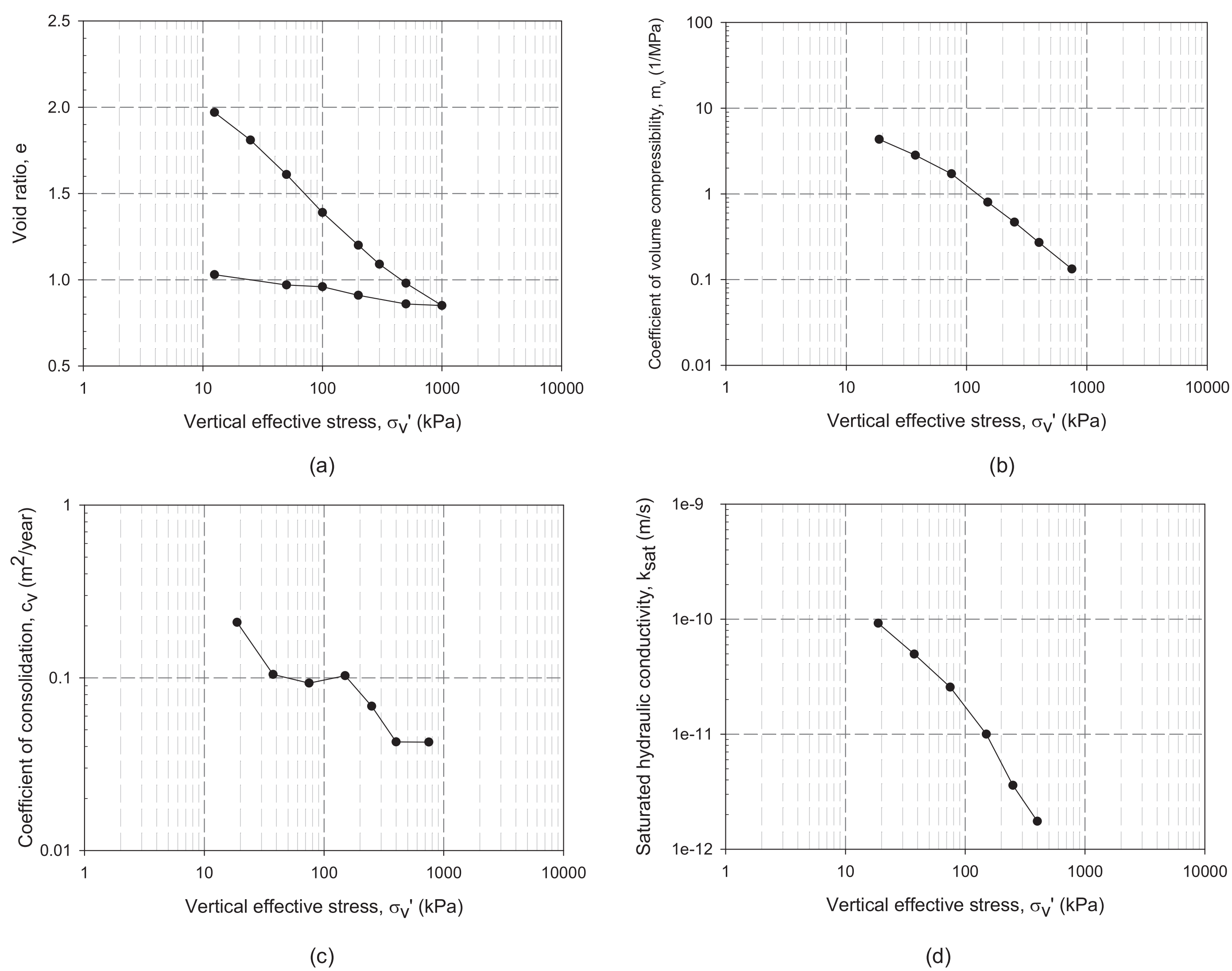

Fig. 3. One-dimensional consolidation test results of a sample prepared at $1.1w_L$ illustrating the relationship between σ_v' and a) e, b) m_v, c) c_v and d) k_{sat}.

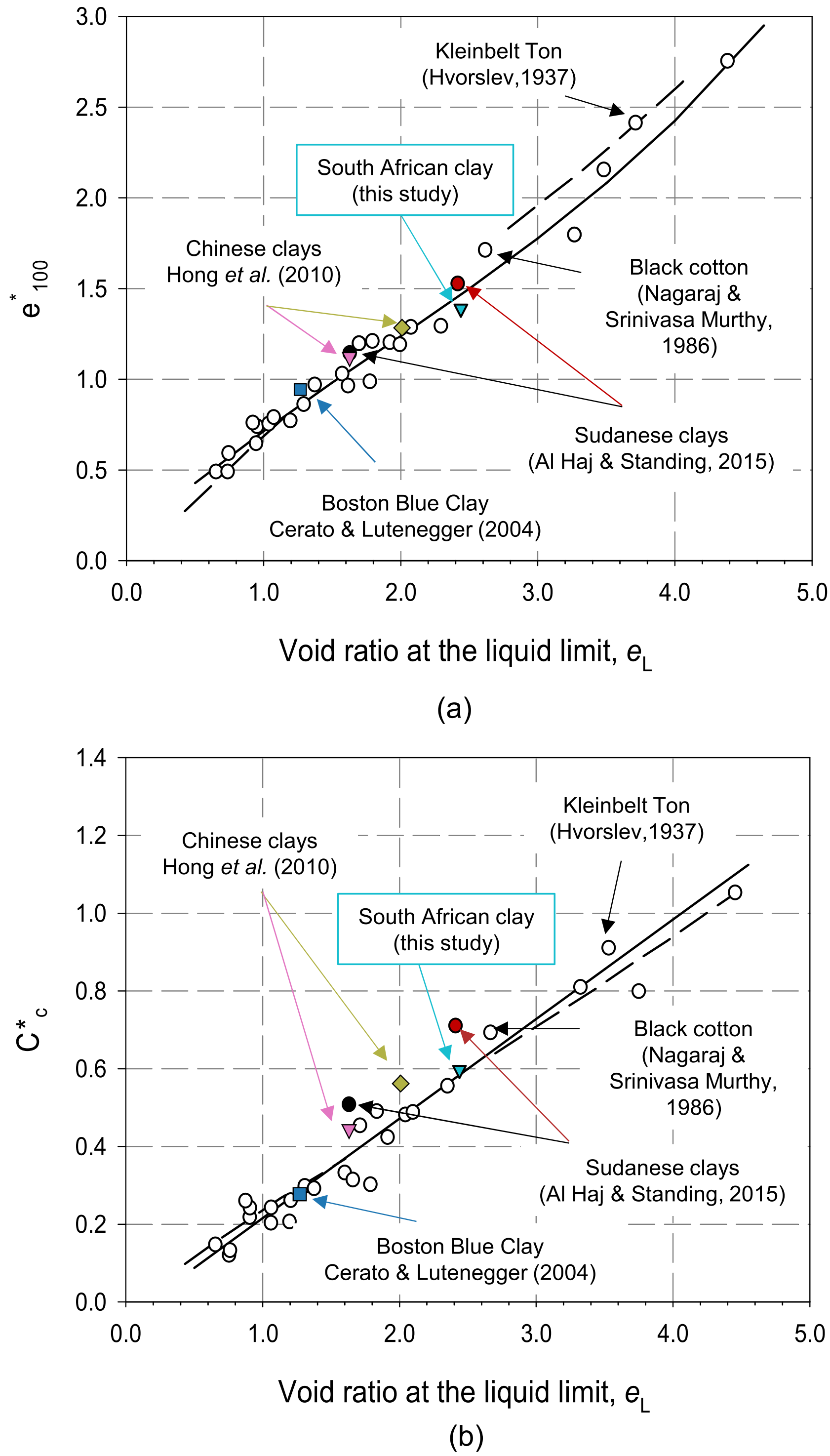

Fig. 4. Relationships between e_L and the intrinsic parameters a) e_{100}* and b) C_c* (after Al Haj and Standing, 2015).

of the clay investigated for this study conform well with Burland's framework. By illustrating close conformity to the dataset presented in Fig. 4, the intrinsic compression line (ICL) can be confidently used to assess the effects of structure and yielding of the compacted and undisturbed samples.

3.2. Determination of swell properties

Wetting after loading tests were performed to determine the amount of swell that could be expected at various depths throughout the clay profile, and to assess the pressure required to prevent swell (i.e. the swell pressure). The results of swell versus time for the compacted and undisturbed material are presented in Fig. 5. Fig. 6 summarises the final strains achieved for the various tests in the form of 'soaking under load' curves.

On the swell versus time plots presented in Fig. 5, it can be seen that a constant value of volumetric strain was achieved by the end of testing. The final amount of strain seen in these two figures is subsequently presented in Fig. 6 as discrete points. Best fit linear regression curves have been fitted through the data for compacted and undisturbed

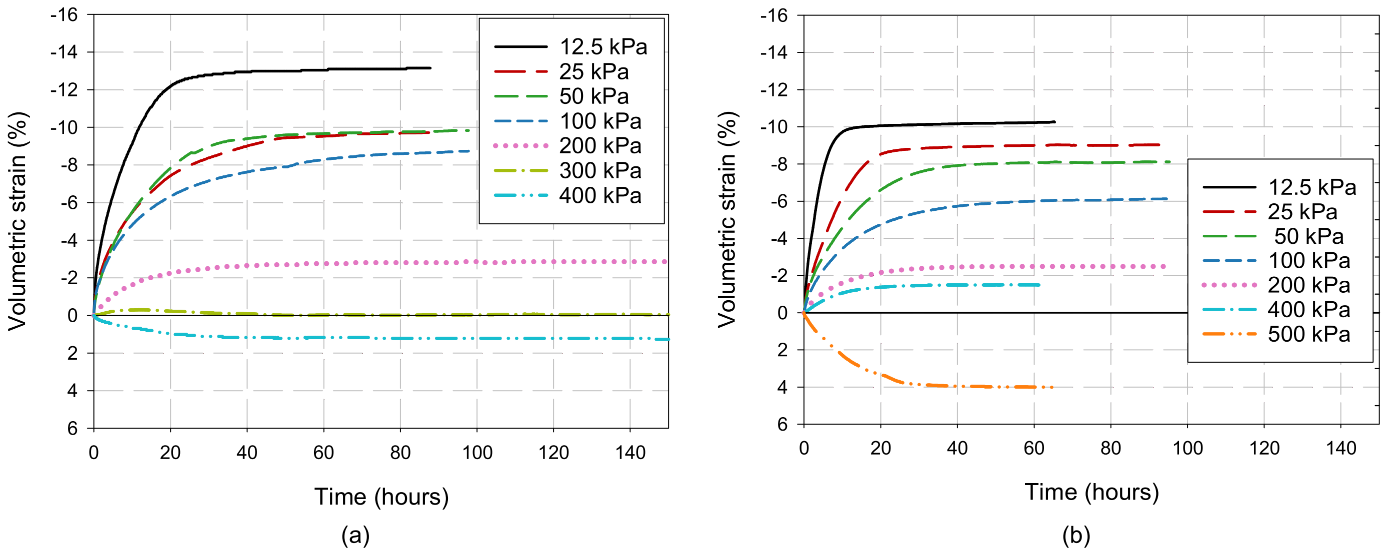

Fig. 5. Swell versus time under constant vertical stress for a) compacted and b) undisturbed samples.

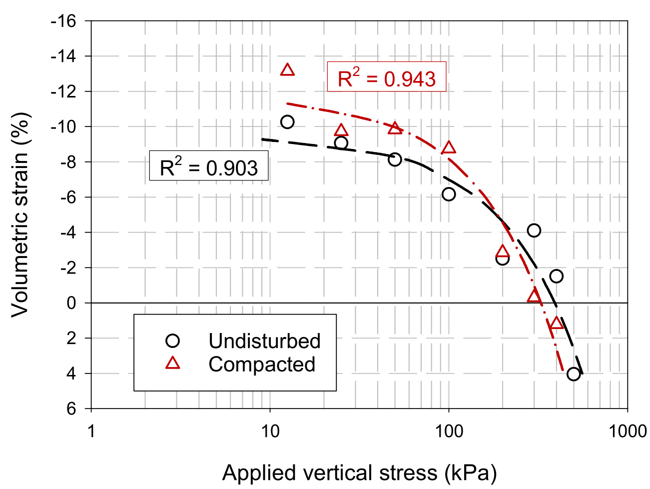

Fig. 6. Soaking under load curves for compacted and undisturbed samples.

samples respectively. Both Figs. 5 and 6 illustrate the effect of soaking stress on the magnitude of volumetric strain. At low soaking stresses, samples experienced relatively high values of swell which decreased as the soaking stress increased. This reduction in swell was observed until the magnitude of soaking stress was sufficient to induce compression of the sample upon inundation.

Fig. 6 illustrates how the general trends in swell characteristics between undisturbed and laboratory compacted samples are similar. Not only was the magnitude of swell achieved at all soaking stresses similar for the compacted and undisturbed samples, but the swell pressure also remained close. Using the regression curves plotted in Fig. 6, the stress required to achieve 0% volumetric change was 329 and 392 kPa for the compacted and undisturbed specimens respectively. While perhaps counterintuitive, this finding is one which has been observed in the literature. In studying the effects of fabric on the swelling characteristics of highly plastic clays, Armstrong and Zornberg (2017) concluded that soil fabric did not affect the magnitude of swell for laboratory prepared specimens. Furthermore, Brackley (1983) reported “no difference between swell of undisturbed and remoulded samples at similar densities and moisture contents”. Brackley (1983) attributed this finding to the fact that highly expansive soils tend to have a reworked macrostructure in situ. Such an explanation is supported by the fact that swelling clays will, over a geological time frame, undergo many cycles of swelling and shrinking. It should however be highlighted that this natural reworking process does not necessarily break down structure to the same extent as what is achieved during the preparation of a reconstituted specimen. Recognising this explanation, the findings of this study and of those reported, it would appear that changes in structure/fabric induced in the laboratory by grating and compaction, have minimal effect on the observed swell characteristics, provided that water contents and densities are similar.

After undergoing swell under various applied vertical stresses, the samples presented were subjected to one-dimensional consolidation tests. These consolidation tests were performed to investigate a range of soil parameters, the first of which was to provide a check on the swell pressure and swell potential at various applied stresses using the *loading after wetting* ASTM (2014b) approach.

3.3. Loading after wetting

In addition to the soaking under load curve presented in Fig. 6, a clay’s swell characteristics can be evaluated using the loading after wetting approach (ASTM D4546–14, 2014b). This test method typically involves inundating a sample under a small seating stress, and then, once the sample begins to exhibit negligible changes in volume with time, consolidation is performed in the conventional manner. While different seating stresses have been used by various researchers (1 kPa (Jennings and Knight, 1957) and 6 kPa (Sridharan et al., 1986)), the general approach to this testing method is to use a “small” seating stress. Research by Justo et al. (1984) revealed that this approach to quantifying swell characteristics generally produces results which are independent of the seating stress, so long as this stress is greater than 5 kPa and less than the swelling pressure.

For this reason, only the samples inundated under a soaking stress of 12.5 kPa (the smallest of those considered in this study) were considered applicable for this purpose. The loading portion of these consolidation tests for both the compacted and undisturbed samples is shown in Fig. 7 (the prefix “C” and “U” is used in the legend to denote compacted and undisturbed specimens respectively). These results have been superimposed onto the soaking under load curves. This super-imposition allows for a comparison of swell magnitude and swell pressure predicted by the two test methods.

From Fig. 7 it can be seen that the loading curves reach a volumetric strain of 0% at a pressure of 260 and 300 kPa for the undisturbed and compacted samples respectively. These values, while slightly lower than

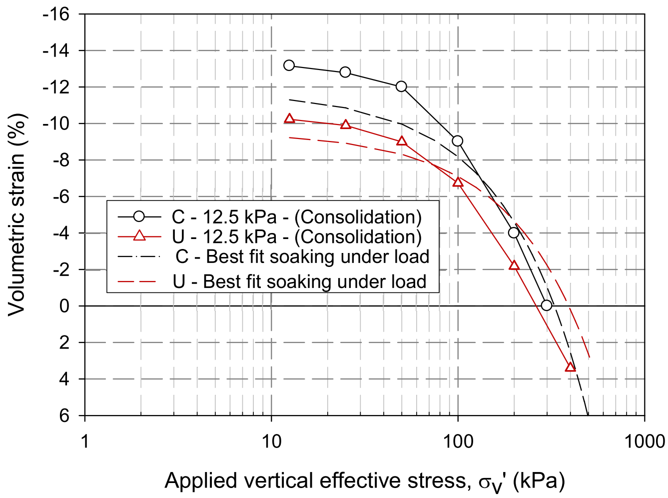

Fig. 7. The results of consolidation tests superimposed onto soaking under load curve (C – compacted; U – undisturbed).

that predicted by the soaking under load curve, do compare well with the former approach from a practical engineering perspective. It is worth noting however that this finding is contrary to that reported by Sridharan et al. (1986) who noted that the loading after wetting test and swell under load curve produced upper and lower bounds for measurements of swell pressure respectively. Detailed comparisons of the two testing approaches lie outside the scope of this study and further research on this matter is required.

Fig. 7 illustrates how the two methods predict similar values of swell potential across the range of vertical effective stresses considered. In line with previous studies, the loading after wetting tests generally plot above the soaking under load curve in the swelling zone (Al Haj and Standing, 2015; Justo et al., 1984; Justo and Saetersdal, 1981). The largest difference in volumetric change predicted within the stress range considered was for the undisturbed specimens at a vertical effective stress of 400 kPa. The discrepancy at this stress was approximately 3% volumetric strain.

The two swell tests presented in this study represent stress paths which are better suited to different construction sequences. The wetting after loading approach arguably presents the most commonly applicable construction sequence whereby the soil experiences an increase in water content after a structure has been built. Conversely, the loading after wetting represents a case whereby the soil is pre-wetter prior to construction (Schreiner and Burland, 1991). However, both of these approaches suffer from the same drawback of reducing a sample to a state of zero suction (Schreiner, 1988). For this reason, these tests should only be regarded as *indicator* tests of a worst-case scenario. In spite of this limitation, it is however useful to note that both approaches produce similar predictions. An alternative approach to these measurements is to use suction or moisture controlled oedometers (Justo et al., 1984; Schreiner, 1988). Such equipment is however, not readily available to geotechnical practitioners (Day, 2017). It is therefore useful to highlight any limitations or discrepancies among the more commercially available *conventional* swell testing.

These results illustrate that despite the limitations of the aforementioned testing methods, for the soil tested in this study, the two approaches produce consistent results. Over and above the validity of the two testing methods, the results further illustrate that the sample preparation procedure managed to retain key properties of the expansive clay.

3.4. Consolidation testing

Following the swell under load tests, all samples which underwent swell rather than compression, were subjected to conventional one-dimensional consolidation testing. The results of consolidation tests of both the compacted and undisturbed samples are shown in Fig. 8 (stresses denoted in the legend of this figure are a reference to the soaking stresses applied during wetting after loading tests) along with the intrinsic compression line (ICL) as determined from the reconstituted sample. By including the ICL on this figure, the effects of structure can be more closely evaluated. It should be highlighted that for samples which underwent compression during inundation, no loading stages were included. For this reason, the 400 kPa and 500 kPa samples in Fig. 8(a and b) respectively illustrate *only* unloading stages.

Furthermore, Fig. 9 presents a comparison of the compression (C_c) and expansion (C_e) indices of the compacted and undisturbed specimens. Since the slope of the compression line varied throughout the loading process, the values presented in Fig. 9 were calculated from the last loading increment. Since the samples were not loaded to higher stresses and their slopes had not yet converged to a single value, the reported C_c values should be interpreted as merely an indication of loading stiffness at the final stress increment. Conversely, C_e was observed to be constant during unloading, and was therefore calculated using the first and last data point of the unloading curve. Also included in Fig. 9, for the sake of comparison, are the compression and expansion indices of the reconstituted specimen. Since the value of 'soaking stress' is not applicable to this test, these values have merely been indicated as constant, horizontal dashed-dot lines, being denoted with a prefix "R" in the legend

When assessing the behaviour of structured soils, it is useful to consider the existence of two yield points, relating to a structured and destructured soil respectively. Vaughan et al. (1988), when considering the behaviour of residual soils, postulated the existence of a first yield, corresponding to the stress at which bonds start to fail, and a second (more significant) yield which occurs when the stress applied to the bonds equals the bond strength.

Following second yield, Vaughan et al. (1988) stated that the compression curve of the bonded specimen would converge with that of the destructured (intrinsic) compression line. When the second yield stress occurs to the right of the ICL, as is observed for some of the undisturbed specimens presented in Fig. 8, it is said to exist in "structure permitted space", a phenomenon which is described in more detail in the following section. However, as observed in Fig. 8, the compacted specimens exhibit progressive yielding to the left of the ICL. At increasing loads, the slope of the normal consolidation line for the compacted specimens approaches that of the ICL. This gradual yielding is attributed to the progressive disruption of structure as the samples undergo compressive strains. This phenomenon whereby compression of a structured sample exhibited yielding to the left of the ICL, followed by convergence with the normal consolidation line of the destructed material was described by Vaughan et al. (1988).

It is also worth noting that upon unloading, both the undisturbed and compacted specimens illustrate lower expansion indices (C_e) than that measured on the reconstituted specimen. This result can also be attributed to the retention of some original structure which, despite undergoing swelling and compressive strains, was not completely disrupted by the end of testing. Values of C_e also remain relatively consistent between the compacted and undisturbed samples indicating a similar degree of structure for both groups of specimens.

3.5. Structure permitted space

Drawing on the observations made by Leroueil and Vaughan (1990), Fig. 10 illustrates the behaviour that may be expected for a structured and unstructured soil. Firstly, Fig. 10 illustrates the results of a reconstituted specimen. Consolidation of such a specimen would result in a straight diagonal line separating possible stress states on the left from impossible stress states on the right. If an overconsolidated, unstructured soil is subjected to a one-dimensional consolidation test, the case illustrated in Fig. 10(a) would be expected. With an increase in load, the

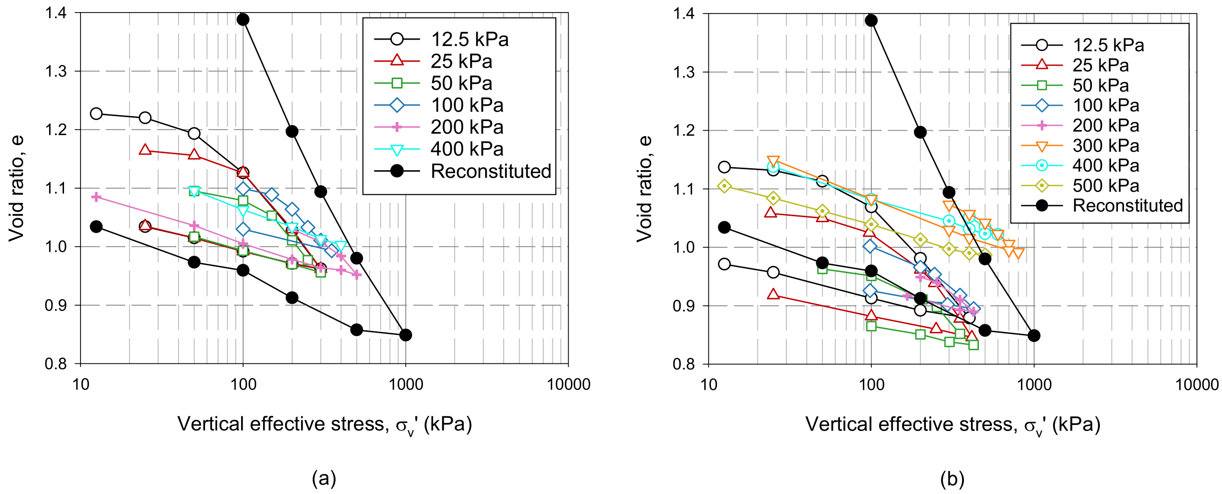

Fig. 8. Results of one-dimensional consolidation tests following swell under constant load for a) compacted samples and b) undisturbed samples.

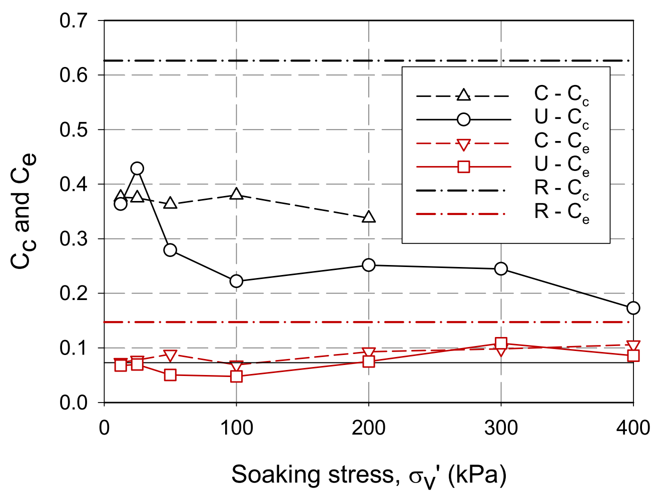

Fig. 9. Comparison of swelling and compression indices for compacted (C) and undisturbed (U) specimens. R denotes a reconstituted specimen.

void ratio would reduce at a rate dictated by C_e until the normal consolidation line (NCL) of the reconstituted specimen was reached. After this point, the stress path would follow the same trajectory as the reconstituted sample. However, the study conducted by Leroueil and Vaughan (1990) illustrated that a sample could exist at seemingly 'impossible states' if the soil was structured. In their publication, Leroueil and Vaughan (1990) defined this region as structure permitted space as illustrated in Fig. 10(b). Such a soil can exist in structure permitted space until such point that structure is destroyed, after which the stress path will converge with, and follow the NCL defined by the reconstituted specimen.

Fig. 8(a) presents the results of consolidation tests conducted on the compacted samples after being allowed to swell under various soaking stresses. From this figure it is clear that all samples lie within permissible stress states. Fig. 8(b) illustrates consolidation tests performed on undisturbed samples. From this result, it can be seen that while most samples lie within permissible stress states, those inundated at soaking stresses of 300 and 400 kPa exist in structure permitted space. Such a result is consistent with the proposition of Leroueil and Vaughan (1990) that structure should be considered a function of strain. The two samples which plotted in structure permitted space were also the two samples which experienced the least amount of volumetric strain during the swell under load tests (see Figs. 5 and 6). As a result, a greater amount of structure was preserved in these specimens.

By recognising the preservation of structure in clays where swell is restricted, it follows that an investigation of structure can possibly assist in defining the *active* zone in an expansive clay profile. The active zone, commonly defined as the portion of a clay profile undergoing seasonal swell and shrinkage, is likely to undergo a reworking process over time. This mechanical process can result in the disruption of structure. This is supported by the results presented in Figs. 5 and 6 where it is shown that samples retrieved from the upper portion of the soil horizon illustrate remarkably similar swell characteristics, regardless of whether the specimens were undisturbed or statically compacted.

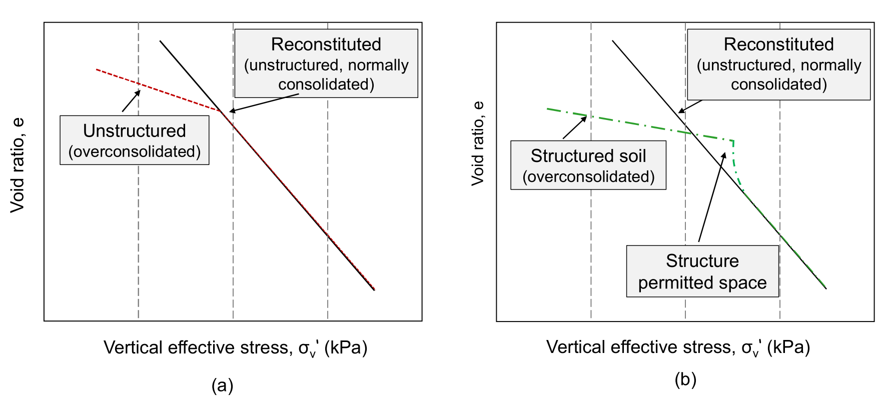

Fig. 10. One dimensional consolidation of a) an unstructured and b) a structured soil.

3.6. Yielding

The consolidation results presented in Fig. 8 illustrate different yield stresses for the various samples. Variation in the initial void ratio at the start of a consolidation test will result in different measured yield stresses. However, as described by Gens and Alonso (1992) the structural rearrangement that occurs as a result of macroscopic volumetric change (i.e. swell) can also reduce the yield stress of a clay. This can be described using the Extended Barcelona Basic Model for expansive clays (BExM) as formulated by Gens and Alonso (1992). Using the constitutive relationships outlined by the BExM framework, a conceptual description of swell induced softening is provided in Fig. 11. To illustrate this concept, consider an element of soil, initially at Point A, with its load-collapse yield curve at position LC_A.

If the suction within this soil element is reduced to Point B (under a constant net-mean stress of p_A), swelling will occur within the microfabric. While the BExM formulation states that microscopic deformations are independent of the macrofabric, the opposite is not true. Swelling strains at the microscopic level will act to increase the macroscopic void ratio which is reflected by *softening* of the clay. Graphically this can be illustrated by recognising a reduction in suction from Point A to Point B is accompanied by movement of the LC yield curve to the left (i.e. to position LC_B). This swell induced softening is positively correlated with the magnitude of swell experienced within the element of soil.

To investigate this relationship quantitatively, consider the results of two consolidation tests presented in Fig. 12. This figure illustrates both the intrinsic compression line (ICL) and the consolidation test performed on the compacted sample allowed to swell under a soaking stress of 12.5 kPa. To determine the yield stress of this sample the conventional Casagrande (1936) construction was used. The yield stress determined in this manner (σ_{vy1}') accounts for swell induced softening. In order to determine where the sample would have yielded, had it not undergone swell, a straight line was drawn from the initial void ratio of the sample (at a slope of C_e) to the ICL (Line AB). The intersection of this line and the ICL was then taken as the predicted yield stress for an unstructured specimen in the absence of swell (σ_{vy2}'). The aforementioned procedure was carried out for all samples tested and the results are presented in Fig. 13.

From Fig. 13 there are several trends which can be observed. Firstly, it is clear that swell does result in a significant reduction in yield stress. A quantitative description of this reduction is however less obvious for the dataset provided. For the compacted specimens it appears that beyond approximately 8% swell, there is little additional reduction in yield stress. Such a non-linear trend is intuitively plausible since it indicates a limiting value to which yield stress can reduce due to swell. It is however not clear from the data that the same trend is followed for undisturbed samples. While the observed scatter in predicted yield stresses in Fig. 13 (b) can be attributed to differing initial void ratios at the start of consolidation, more testing is required to definitively establish a relationship between reduction in yield stress due to swell and magnitude of swell.

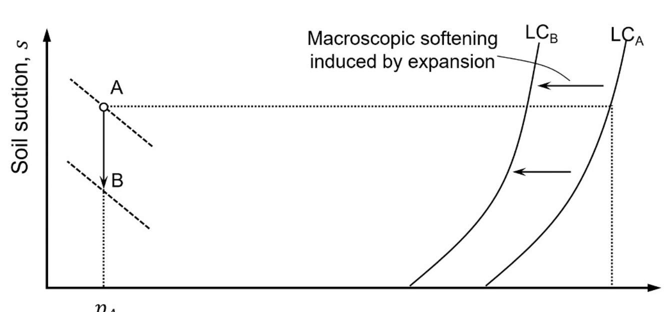

Fig. 11. Swell induced softening (after Gens and Alonso, 1992).

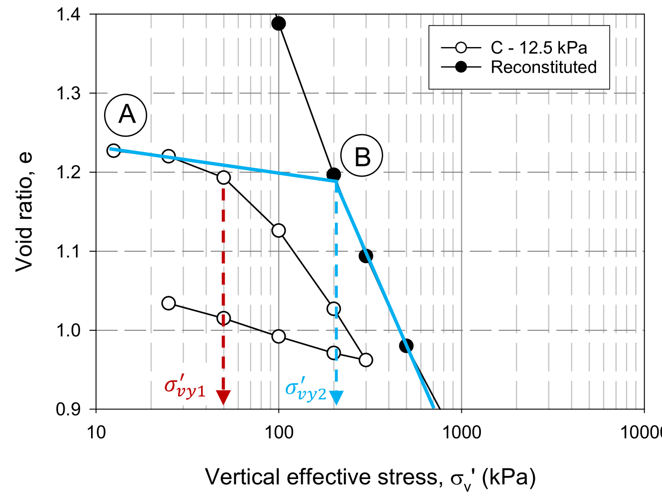

Fig. 12. Reduction in yield stress due to swell.

Another aspect of importance is the ability to assess the position of a given stress state in relation to its yield curve. This relationship is provided in Fig. 14 where macroscopic swell is plotted as a function of the ratio of current vertical stress to the yield stress of the sample (σ_v'/σ_{vy}').

In the original BExM formulation, consideration is given to both macro and microscopic strains. Whilst an adequate description of the microfabric has been shown to be useful in describing certain factors such as strain irreversibility, hydraulic conductivity and various stress path dependencies (Alonso et al., 2013; Romero and Simms, 2008), experimental measurements at the microscopic scale require sophisticated equipment (Lourenço et al., 2008; Romero and Simms, 2008). It is therefore useful to be able to deduce criteria governing hardening plasticity of a given clay with conventional testing equipment as illustrated in Fig. 14. It is also worth noting that the relationship provided in Fig. 14 is similar in shape to that conceptualised by Gens and Alonso (1992).

4. Conclusions

The mechanical properties of a black expansive clay from the Limpopo province of South Africa have been presented. By means of conventional oedometer testing, various mechanical characteristics have been investigated for undisturbed, compacted and reconstituted specimens.

The study showed that a number of key mechanical properties remain, for all practical purposes, unchanged by laboratory preparation and compaction. This finding is attributed to the fact that expansive soils undergo many drying and wetting cycles throughout their geological lifetime and as such, occur in a reworked state in situ. Key properties which were observed to remain almost unchanged between undisturbed and compacted specimens include, swell potential (from an unsaturated state), swell pressure and saturated compression and expansion indices.

Comparison of consolidation tests on undisturbed and compacted specimens with that of a reconstituted specimen allowed for the effects of structure to be investigated. All samples illustrated a gradual yielding process occurring before reaching the intrinsic compression line (ICL). Such a relationship can be attributed to the gradual disruption of structure with increasing compressive strains. After consolidation, it was found that expansion indices (C_e) for all samples were approximately the same. These values of C_e were however found to be slightly less than that measured on a reconstituted specimen illustrating some preservation of structure at the end of testing.

For the range of consolidation tests conducted (on compacted and undisturbed samples), only two specimens were found to exist in

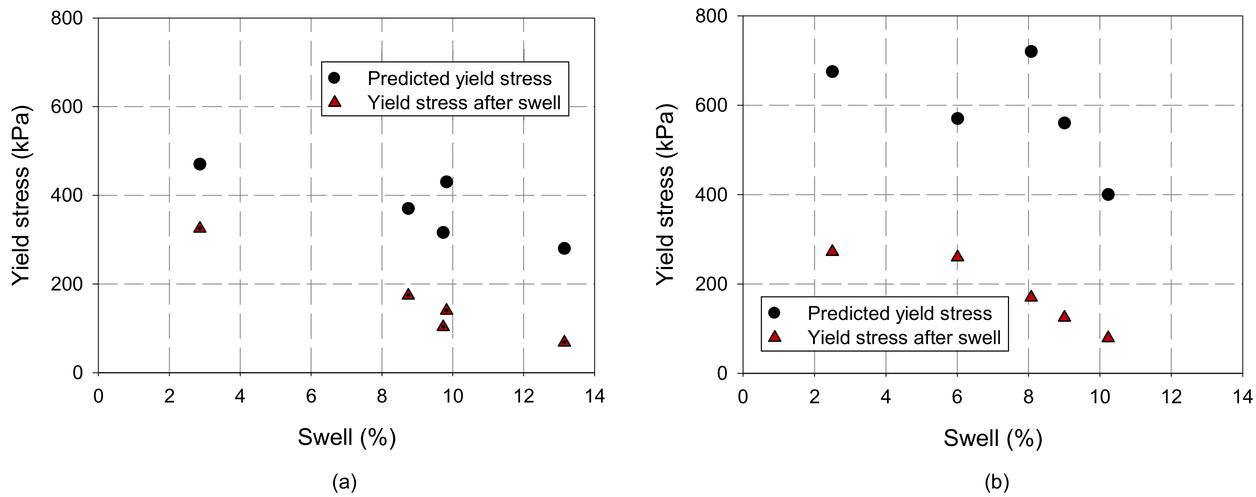

Fig. 13. Reduction in yield stress due to swell for a) compacted and b) undisturbed samples.

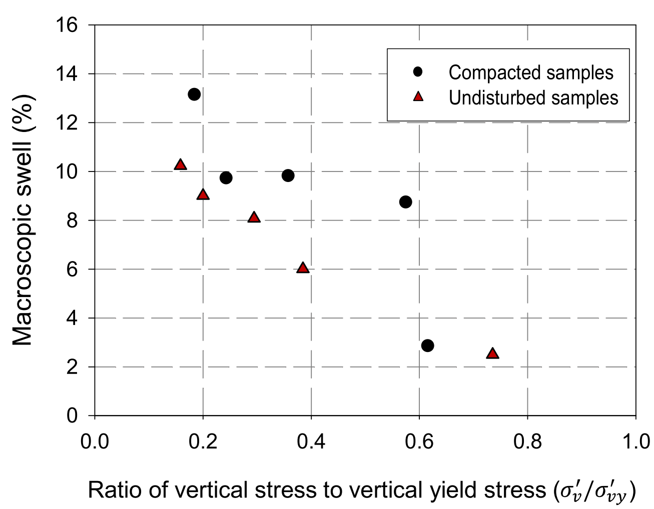

Fig. 14. Relationship between macroscopic swell and the ratio of current vertical stress to vertical yield stress.

structure permitted space. Upon further inspection it is seen that these two samples underwent the least amount of volumetric strain during the inundation phase. This finding is consistent with the suggestion that yielding of structure are more fundamentally related to strain than to applied stresses.

Finally, experimental data illustrates a reduction in yield stress with increasing macroscopic swelling. While measurements of the micro-fabric have not been presented, the results highlight that the position of a sample at a given stress state (relative to its yield curve) can be deduced from conventional oedometer testing.

CRediT authorship contribution statement

T.A.V. Gaspar: Conceptualization, Formal analysis, Investigation, Writing – original draft, Visualization. **S.W. Jacobsz:** Conceptualization, Supervision, Funding acquisition, Writing – review & editing, Project administration. **G. Heymann:** Conceptualization, Supervision, Writing – review & editing. **D.G. Toll:** Conceptualization, Writing – review & editing. **A. Gens:** Conceptualization, Writing – review & editing. **A.S. Osman:** Conceptualization, Funding acquisition, Writing – review & editing, Project administration.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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