

INFLUENCE OF DISCONTINUITIES ON THE BEHAVIOUR OF PARTIALLY SATURATED COMPACTED CLAY

PENGARUH KETIDAKKONTINYUAN PERILAKU LEMPUNG YANG DIPADATKAN PADA KONDISI JENUH SEBAGIAN

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ABSTRACT

The partially saturated soil behaviour is quite different from those of fully saturated soil because of the presence of matric suction pressure. Although soils are generally assumed fully saturated below the groundwater table, they may be semi saturated near the state of full saturation under certain conditions. Results obtained with the strength theory of saturated soil could not be directly applied to solve the partially saturated soil problems, in particular to the soil containing discontinuities. It has been observed that discontinuities are known to develop when such geological materials are subject to loading. In the case of strong rock, on the other hand, there has been considerable interest in the application of fracture mechanics to account for such discontinuities. Moreover, the mechanical properties of soil that usually conducted by the use of triaxial apparatus rather than biaxial device will be addressed according to the fact that field problems involving geotechnical structures are often truly or close to plane strain situation. This paper will disseminate the result of some experimental testing on the effect of discontinuities on the properties of partially saturated compacted kaolin clay specimen under triaxial and biaxial condition. A modification of the conventional triaxial apparatus was used in this study. A high air-entry disc (HAED) was used as the interface between the unsaturated soil and the pore water pressure measuring system. Two types of intact specimen and pre-crack specimen have been tested under biaxial as well as triaxial condition. It was concluded that the discontinuities on the specimen weaken the shear strength as well as compressive strength of specimen.

Keywords: discontinuities, partly saturated, matric suction, triaxial and biaxial

ABSTRAK

Perilaku tanah jenuh sebagian cukup berbeda dengan tanah sepenuhnya jenuh disebabkan karena adanya matric suction. Meskipun dibawah muka air tanah pada umumnya tanah diasumsikan sepenuhnya jenuh, tanah mungkin dalam kondisi jenuh sebagian pada kondisi tertentu. Hasil hasil yang diperoleh dari teori mekanika tanah jenuh tidak bisa secara langsung diaplikasikan untuk menyelesaikan masalah masalah tanah jenuh sebagian, khususnya pada tanah yang mempunyai retakan. Observasi menunjukkan bahwa adanya retakan bisa berkembang pada saat material tanah menerima beban. Pada kasus batuan keras, sudah mulai dipertimbangkan untuk mengaplikasikan mekanika retakan untuk memecahkan masalah ini. Terlebih lagi perilaku mekanis tanah yang biasanya diujikan dengan menggunakan alat uji triaksial pada penelitian ini akan diujikan dengan alat uji biaksial berdasarkan fakta di lapangan yang melibatkan struktur geoteknik lebih sering mendekati kondisi biaksial. Paper ini akan menampilkan hasil hasil penelitian laboratorium pada pengaruh diskontinuitas tanah terhadap perilaku tanah jenuh sebagian pada tanah lempung kaolin spesimen pada kondisi uji triaksial dan biaksial. Modifikasi alat uji triaksial digunakan dalam studi ini. Sebuah high air-entry disc (HAED) digunakan sebagai interface antara tanah tidak jenuh dan sistem pengukuran tekanan air pori. Dua tipe spesimen intak dan spesimen yang mengandung retakan awal sudah dilakukan pengujian dibawah kondisi biaksial dan triaksial. Dari hasil pengujian dapat disimpulkan bahwa adanya diskontinuitas pada spesimen tanah memperlemah tahanan geser dan juga kuat tekan spesimen tanah.

Kata-kata kunci: diskontinuitas tanah, jenuh sebagian, matric suction, triaksial and biaksial

INTRODUCTION

The behaviour of partially saturated soil is different from those of fully saturated soil because of the influence of suction. Partially saturated soils form the largest category of materials that cannot be classified by classical saturated soil mechanics concepts. Results obtained with the strength theory of saturated soil could not be directly applied to solve the partially saturated soil problems. Although soils are generally assumed fully saturated below the groundwater table, they may be semi saturated near the state of full saturation under certain conditions. The situation of partial saturation may be caused by several factors, such as variation of water table level due to natural or manmade processes.

Generally, Partially saturated soil is characterized by three phases, soil solids, water, and air. The presence of a fourth independent phase, a so called air-water interface or contractile skin was introduced by Fredlund and Morgenstern in 1977. Based on multiphase continuum mechanics, they proposed a theoretical

stress analysis of an partially saturated soil (Fredlund and Morgenstern, 1993). The analysis concluded that any two of three possible normal stress variables can be used to describe the stress state of an unsaturated soil. In contrast to saturated soil, it is possible to relate the behaviour of the soil to the effective stress only. The presence of matric suction pressure is the main difference between saturated and unsaturated soil mechanics. It has been observed that several stability problems, involving soils used as construction materials, are due to water content changes and therefore to matric suction changes that occur periodically in nature.

The present modeling of brittle soil and weak rock is based on principles of continuum mechanics in spite of the fact that discontinuities are known to develop when such geological materials are subject to loading. In the case of strong rock, on the other hand, there has been considerable interest in the application of fracture mechanics to account for such discontinuities.

Many studies have been conducted on detailed aspect of such discontinuities, but these are of limited practical application in an actual situation. The existence of cracks and fissures, which are the result of mechanical, thermal and volume-change-induced stresses, such soils are non uniform and therefore not amenable to analysis by continuum mechanics. On the other hand, fracture mechanical theory may be used to advantage to replicate their behavior (Ingraffea, 1987).

Atkinson and Bransby (1982) proposed the conventional failure criteria for soils which might be partly appropriate to yield-dominant behaviour, but not this category of brittle fracture. In practice, there is the possibility that soil behaves more like a brittle material. The soil ruptures suddenly under compressive loading like soft rock, starting from the weakest crack in it. A basic premise of fracture theory is that crack like imperfections are inherent in engineering materials. These defects have the tendency to make stresses higher, which eventually trigger off fractures when a material body is subjected to a critical load or undergoes damage under cyclic loading. This present state of fracture mechanical theory has been summarized by Anderson (2004). Lo et al (2005) modeled brittle overconsolidated clay accordingly and thereby provided a rational basis for the prediction of such soil behaviour.

Most laboratory experiments on soils for the purposes of evaluating constitutive behaviour and stability properties of soil are performed under axisymmetric or conventional triaxial conditions. However, most geotechnical field problems such as landslide problems, failure of soils beneath shallow foundations, and failure of retaining structures are truly or close to biaxial situations. Mochizuki et al (1993) reported that when soil is tested under plane strain conditions, it, in general, exhibits a higher compressive strength and lower axial strain. Behaviour of fined grained sands tested under biaxial conditions has been reported (Alshibli et al, 2004; Alshibli and Sture, 2000; Bizzarri, 1995; Han and Vardoulakis, 1991; Hans and Drescher, 1993; Marach et al, 1984; and Mochizuki et al, 1993). The plane strain testing of clay has only been initiated recently (Fauziah and Nikraz, 2008; Fauziah and Nikraz, 2007; Lo et al, 2000; Drescher et.al (1990)) and published data of such tests especially for brittle clay material is very limited.

This paper will present the experimental study of the behaviour of pre-crack partially saturated clay specimens compare to the intact specimen by the use of biaxial compression apparatus as well as triaxial test set-up, although the behaviour of overconsolidated clay (Fauziah and Nikraz, 2007) and fracture characteristics of brittle clay may also be determined by this test apparatus. Description of the apparatus, specimen preparation, testing method, procedure and data analysis will be presented in the following discussion. Some results of the testing will be compared with the known soil behaviour and previous working.

DISCRIPTION OF THE DEVICE

A conventional triaxial apparatus was modified for the purpose of biaxial testing in this study. The biaxial arrangement is placed in a cell, with the height of 300 mm, 200 mm internal diameter and 30 mm wall thickness. Figure 1 shows the schematic diagram of the biaxial compression device.

A prismatic specimen with the size of 36 mm wide, 72 mm high, and 72 mm thick, so that the aspect ratio is 2, is placed on the base pedestal where it was restrained laterally by two rigid perspex plates to restrain its out-of plane movement which make $\epsilon_2=0$. Therefore, only major (σ_1) and minor (σ_3) principal stresses acting on the soil specimen (Figure 2).

To prevent any air from passing through the disc into the measuring system, a high air-entry disc (HAED) was used as the interface between the partly saturated soil and the pore water pressure measuring system. All surfaces which are in contact with

the specimen were lubricated to avoid the likelihood of scratching and reduced friction. An LVDT (Linear Vertical Displacement Transducer) was used to measure the axial displacements of the test specimen. The global volume change of the water saturated soil specimen was monitored by the use of an automatic volume change unit which is connected to the back-pressure line. A data acquisition system consisting of an MPX 300 data logger and a set of microcomputer were used to record the displacement, loads, pressure and volume change reading of the specimen. WINHOST 2.0 package software was used to convert digital bit data from the ADU (Analogue digital Unit) to engineering units based on the calibration of the relevant measuring unit, which was done before running the plane strain test. A more detail description of this biaxial compression device can be found in previous working reported by Fauziah and Nikraz (2008) and Fauziah and Nikraz (2007).

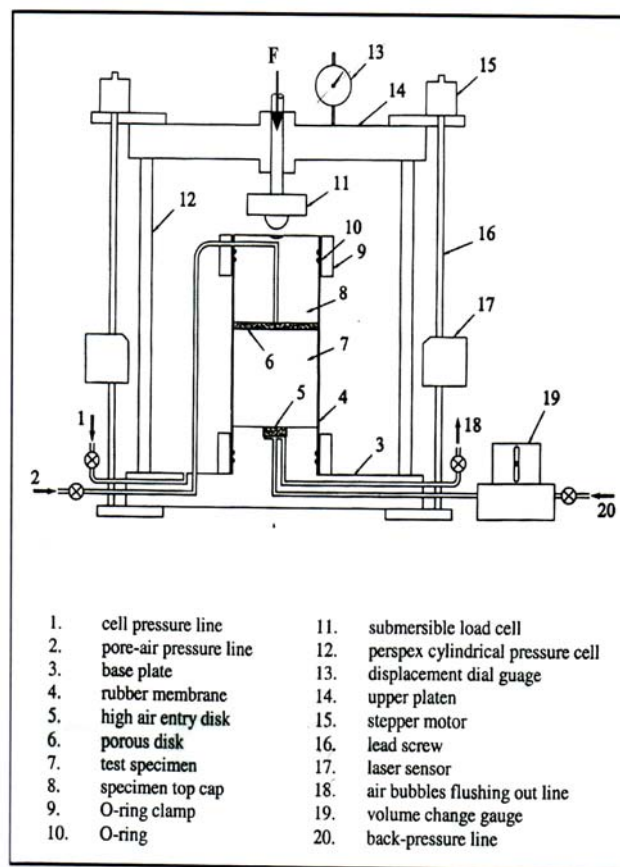


Figure 1. A schematic diagram of the plane strain device

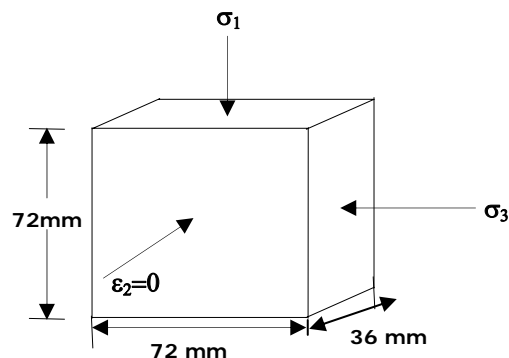


Figure 2. A schematic diagram of plane strain condition

MATERIAL PREPARATION AND METHOD

The material used in this study was commercial Kaolin clay, a product of UNIMIN PTY LTD, Australia. The basic characteristics of the material are shown in Table 1.

Table 1. Basic characteristics of kaolin clay

Type of test	Value
Liquid limit (ASTM D-4318)	53.5 %
Plastic limit (ASTM D-427)	30.76 %
Plasticity Index	22.74 %
Specific Gravity (ASTM D-854)	2.60

It took few weeks to complete the preparation of soil specimen. Initially, a kaolin clay sample was slurried to a uniform consistency of 1 ½ times its liquid limit using an electrical soil mixer. This slurry was obtained by mixing 8 kg of kaolin powder with 6 kg of water using the electric mixer for about 2 hours. Before pouring the kaolin slurry to the consolidation unit, the inner wall of the cylinder mould was greased to ease the extrusion of the sample from the mould at the end of consolidation. A steel cylindrical mould with the height of 600 mm and 150 mm in diameter was used to consolidate the slurry using a hydraulic tester in over a period of one to two weeks which the maximum of 300 kPa was applied in three stages.

To apply the pressure evenly to the slurry, two circular perspexes were placed at both ends of the slurry in the mould. The slurry was allowed to consolidate by its own weight and small pressure was applied to prevent the slurry being squeezed out between the circular perspexes and the mould. A higher vertical pressure was applied when there was no further settlement change. The soil was then extruded from the mould into lubricated formers when the consolidation process had completed. Following this, the specimen and the former were wrapped in plastic film sheeting after sealing both their faces with liquid wax. To obtain even pore water pressure, the specimens were placed in the dehumidifier for at least 2 days until it is tested.

In the plane strain test set-up, the rubber membrane was first placed over the test specimen with the aid of a sleeve stretcher. The rectangular porous plate was then placed on top of the specimen followed by the top assembly and the HAED. The rubber membrane was next slipped over the porous plate, the top assembly and the HAED and secured by the use of a set of O-rings and rectangular Perspex clamp. The specimen, together with the porous plate and the top assembly were then placed over the pedestal of the plane strain compression apparatus. The rubber membrane was next slipped over the pedestal and secured by the use of a set of O-rings and clamp set. Two rigid perspex plates were then placed and secured by the use of clamp set for biaxial test set up (Figure 3). For triaxial test set up the rigid plates were uninstalled to specimen. The pressure cell, top assembly and laser sensor set-up were then installed.

TESTING METHOD AND PROCEDURE

Two types of intact specimen and pre-crack specimen have been tested under biaxial and triaxial condition. The pre-crack specimen were made by adding the 3 cm diagonal precrack in the center of the intact specimen to make the discontinuities in the specimen. Three specimen of IBM (intact specimen, biaxial), PCBM (Pre-crack specimen, biaxial) and ITM (intact specimen, triaxial) were tested under net normal stress of 0 and maximum matric suction of 500 kPa and three specimen of IBN (intact specimen, biaxial), PCBN (Pre-crack specimen, biaxial) and ITN

(intact specimen, triaxial) were tested under matric suction of 0 and maximum net normal stress of 800 kPa. The specimen was first saturated until the value of pore pressure coefficient (B-value) of the specimen reached the value of 0.95-0.98, followed by matric suction applied and loading compression processed.

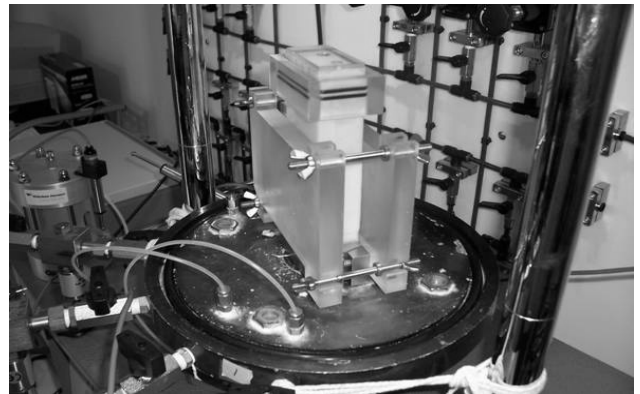


Figure 3. Side view of biaxial test set up

A cell pressure of 600 kPa, back pressure of 590 kPa and pore pressure of 600 kPa were applied to provide an initial net normal stress (NNS) of 0 and matric suction (MS) of 10 to the to the IBM, PCBM, and ITM specimens. The volume change of the soil skeleton was monitored continuously by the laser sensors. The change in volume of water in the specimen was also monitored continuously by the volume change gauge which was connected to the back-pressure line. Once the changes in the soil and water volumes had ceased, the test specimen was presumed to have fully consolidated under a matric suction of 10 kPa. The matric suction was next increased to 20 kPa by reducing the back pressure to 580 kPa. The corresponding changes in the soil skeleton and water volumes were monitored continuously until they had ceased, at which stage the corresponding void ratio and the water content of the test specimen were computed from the cumulative changes in soil skeleton and water volumes. The entire procedure, that is from increasing the matric suction to the desired value up to flushing out the air bubbles, was repeated for matric suctions of 50, 100, 200, 300, and 500 kPa respectively, which were applied by reducing the back pressure accordingly.

Another test apparatus were set up for IBN, PCBN, and ITN specimens similarly as before, except that the head was replaced with a porous disc. A cell pressure of 110 kPa, back-pressure of 100 kPa and pore-air pressure of 100 kPa were then applied to the specimen to provide an initial net normal stress of 10 kPa and matric suction of 0. The changes in soil skeleton and water volumes were then monitored continuously and when these changes had ceased, the total changes in soil and water volumes were noted. The net normal stress was first increased to 20 kPa by increasing the cell pressure to 120 kPa. Thereafter, the entire above procedure, starting from applying the net normal stress up to when the changes in soil and water volumes ceased, was repeated for net normal stresses of 50, 100, 200, 300, 500 and 800 kPa.

The specimen was then compressed by elevating the base of the confining pressure cell at a constant velocity of 0.08 mm/m with the drainage line closed at at net normal stress of 0 and matric suction of 500 kPa for IBM, PCBM, and ITM specimen and at matric suction of 0 and net normal stress of 800 kPa for IBN, PCBN, and ITN specimen. This loading rate was deduced based on the permeability of adopted kaolin clay suggested by Bishop and Henkel (1962).

The data were recorded at 3 minute interval test and it was terminated at the axial strain of about 20 % or sooner. The specimen was then taken out immediately for the purpose of moisture content test. In the analysis of the behaviour of the brittle un-

saturated clay, the pore pressure parameter would be required in order to the determined the pore pressure increments and the matric suction. The pore pressure parameters were deduced from the volumetric deformation coefficient, which was obtained by laboratory testing. This procedure was proposed by Fredlund and Rahardjo (1993), although adapted to biaxial conditions.

TEST RESULT AND DISCUSSION

The typical result given by specimens tested under biaxial condition of IBM, IBN, PCBM, and PCBN as well as specimen observed under triaxial test set up of ITM and ITN will be discussed in the following discourse. The strain softening response of the intact and pre-crack specimen tested under biaxial test set up are depicted in Figure 4 and stress-strain curves of the intact specimen tested under biaxial and triaxial condition presented in Figure 5. The peak stress and the corresponding strain of the specimen is summarised in Table 2.

Table 2. The peak stress vs the corresponding axial strain

Specimen name	Peak stress (kPa)	Vertical strain (%)
IBN	159.178	3.64
PCBN	125.336	2.73
ITN	139.464	3.03
IBM	228.395	3.87
PCBM	124.519	3.33
ITM	175.926	3.33

In general, the shear stress or deviatoric stress curves increase monotonically with the increasing vertical strain until they reach peak stresses followed by strain softening behaviour. According to Lo et al (2005) this is the typical phenomenon of specimen of brittle, hard partly saturated soil specimen and exhibit elastic only.

As can be seen in Figure 4, the shear stress of the intact specimen of IBM and IBN were higher along the vertical strain than that the specimen containing discontinuities or pre-crack of PCBM and PCBN specimen. The highest failure stress of 228.395 kPa was reached by the intact specimen of IBM, and the lowest failure stress of 124.519 kPa was derived by the pre-crack specimen of PCBM. The presence of a fissure or discontinuity makes the soil weaker as the effective area offering resistance to shear is reduced. The shear strength along a surface of discontinuity is thereby less than that of the intact material. It is also shown from the graph that the pronounced peak shear strengths were occurred to the pre-crack specimen than the intact specimen. Similar observation of pre-crack overconsolidated clay had been reported elsewhere (Lo et al, 2000).

The shear stress versus axial strain curves of intact specimen under biaxial and triaxial testing were presented in Figure 5. The graph shows that under the same matric suction and net normal stress the biaxial specimen of IBM had higher shear stress along the strain than that the specimen of ITM which was tested at triaxial test set up, as well as the IBN specimen compared to the ITN specimen. The triaxial specimen also reached earlier peak strength at 3.03 % and 3.33 % axial strain for ITM and ITN respectively than that the biaxial specimen of IBM and ITM at 3.87 % and 3.64 % respectively. This result seem to be consistent with the founding of Mochizuki et al (1993).

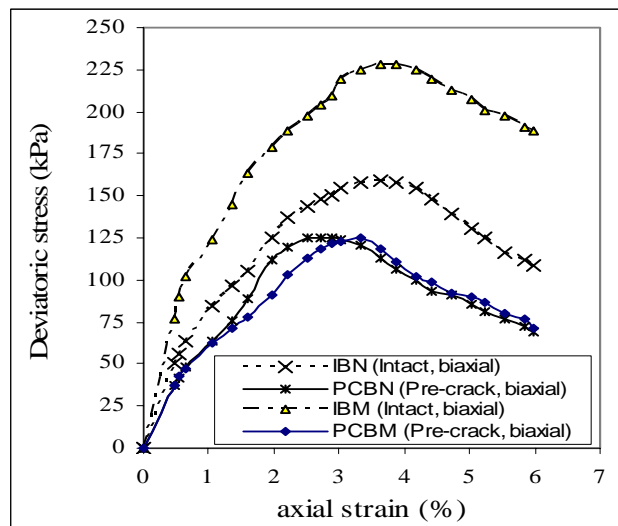


Figure 4. Stress strain behaviour of intact and pre-crack specimen under biaxial testing

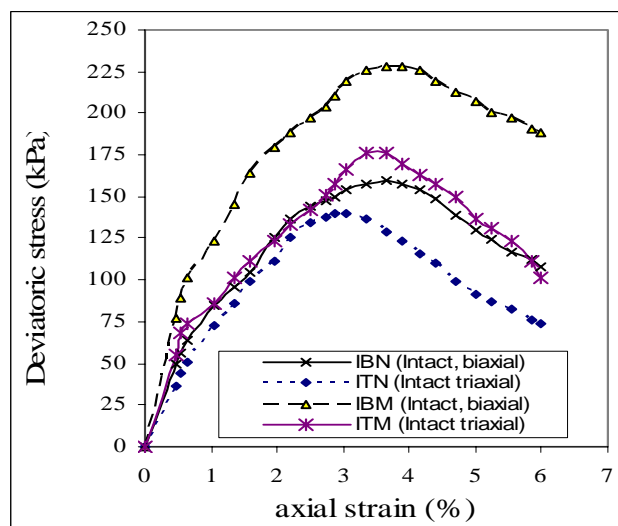


Figure 5. Stress strain of intact specimen under biaxial and triaxial testing

Figure 6 and Figure 7 present the constitutive surface of void ratio versus log net normal stress and log matric suction of the specimen. The slope of the intersection curves are the volume change index C_m and water content index D_m respectively for the case that net normal stress set to zero, whereas the slope of the intersection curves are the volume change index C_i and water content index D_i when the matric suction set to zero. C_i is the slope of the consolidation curve and is equal to the compressive index of a saturated soil, while C_m is the slope of the shrinkage curve. The change in void ratio of the IBM, PCBM and ITM specimen in relation with matric suction depicted in Figure 6, while water content changes of the IBN, PCBN and ITN specimen in connection with net normal stress changes presented in Figure 7. It can be shown from the graphs that the curves went down with the increasing of either matric suction or net normal stress for all of the specimens.

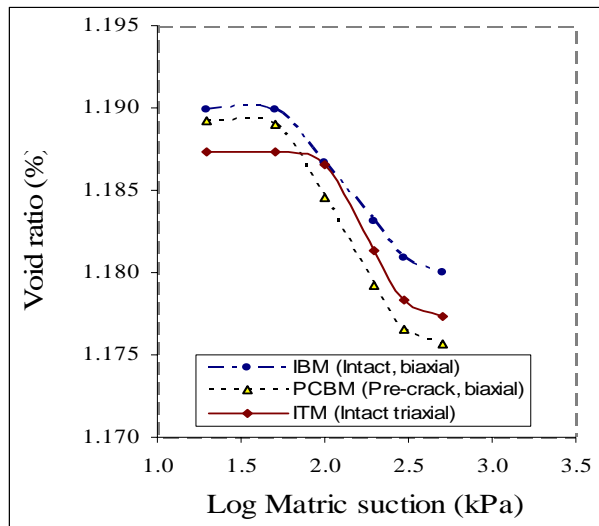


Figure 6. Void ratio versus matric suction of IBM, PCBM and ITM specimens

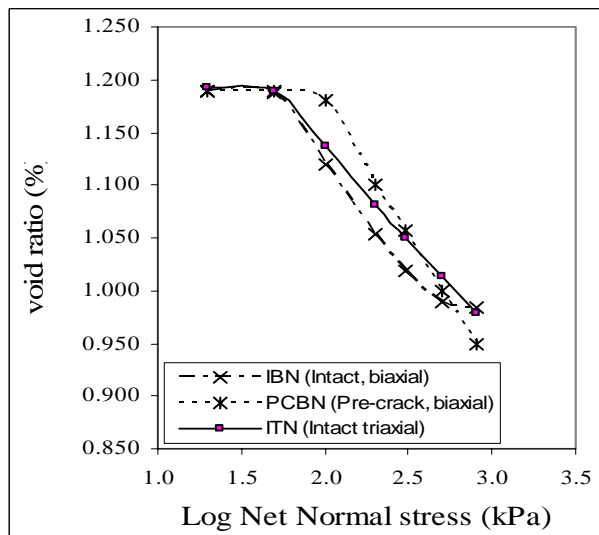


Figure 7. Void ratio versus matric suction of IBN, PCBN and ITN specimens

The value of the volume change index of the IBM, PCBM and ITM specimens were calculated as 0.011, 0.016, and 0.017 respectively. The value of the C_r of the IBN, PCBN and ITN specimens are 0.219, 0.257 and 0.232 respectively. The higher slope of ITN and ITM curves than that of IBN and IBM curve indicated that the specimen tested at biaxial condition had higher compressive strength than that of specimen tested under triaxial test set up. This is in well agreement with the observation reported by Mochizuki et al (1993). It is also clearly shown that the specimens containing precrack of PCBM and PCBN had higher compressive index than that the intact specimens of IBM and ITN.

The change in water content of the specimen in relation with matric suction (MS) and net normal stress (NNS) depicted in Figure 8 and Figure 9. The constitutive surface of water content is defined by the compressive index D_m and D_t corresponding to the matric suction and net normal stress respectively. The value of compressive index obtained by determining the gradient of the linear portion of the curve of the water content against the log of matric suction for D_m and the log of net normal stress for D_t .

As can be seen from the graph that the curve went down with the increasing of either matric suction or net normal stress for all the specimens. The curve also indicated that the higher gradient value of 0.071 and 0.068 were reached by pre-crack specimens of PCBM and PCBN respectively than that the gradient value of intact specimens of IBM and IBN which were 0.55 and 0.056 respectively (Figure 8). Similar to the curve of the volume change result on Figure 6 and Figure 7, and consistent with the stress-strain behaviour of the specimens shown on Figure 4 and Figure 5, the existence of the crack or discontinuities on the specimen not only weakened its shear strength as well as its compressive strength but they also quicken the failure of the specimen.

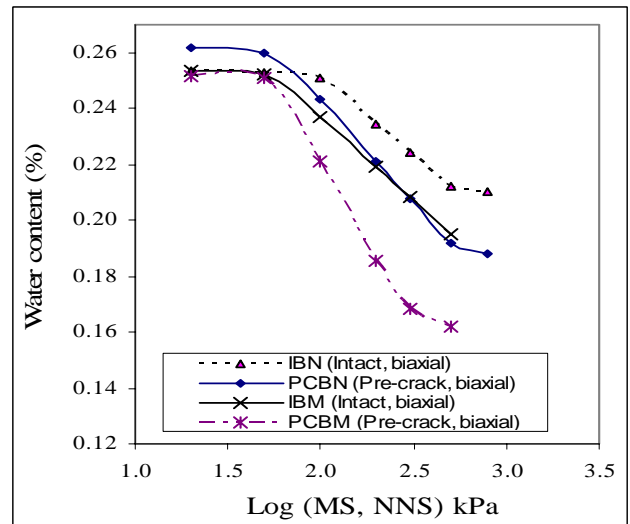


Figure 8. Water content change of intact and pre-crack specimen under biaxial testing

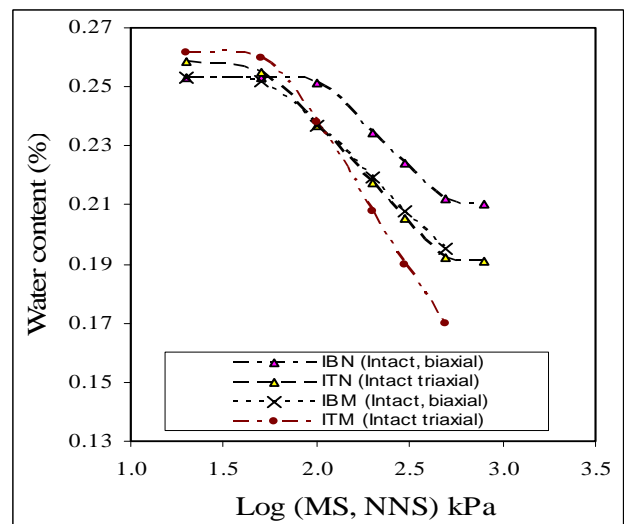


Figure 9. Water content change of intact specimen under biaxial and triaxial testing

The water content change of intact specimen under biaxial and triaxial testing were plotted in Figure 9. The graph shows that under the same matric suction and net normal stress the biaxial specimen of IBM had lower slope than that the specimen of ITM which was tested at triaxial test set up, as well as the IBN specimen to the ITN specimen. This result indicated that in general specimen tested under plane strain condition exhibits a higher compressive strength than that of the specimen tested under triaxial test set up.

CONCLUSION

The following conclusions might be drawn from this experimental study.

1. The typical of brittle hard clay and exhibit elastic only behaviour were shown by all the specimen tested.
2. Shear strength of the intact specimen were higher along axial strain than that the intact specimen. It is also shown that the pronounced and lower value of peak shear strength were occurred to the precrack specimen than the intact specimen.
3. The weaken compressive strength of the specimen containing discontinuities were indicated by their higher volume change index than the intact specimen's.
4. Under the same matric suction and net normal stress the specimen tested under plane strain condition exhibits a higher compressive strength than that the specimen tested under triaxial test setup.

ACKNOWLEDGMENT

The authors generously give acknowledgment to Professor Kwang Wei Lo from National University of Singapore and Dr Min Min Zhao for their valuable advice and support.

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