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Effect of static pore water pressure on shear behavior of saturated kaolin clay

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Abstract: To study the mechanical properties of sedimentary marine clays in deep seabed, static pore water pressure (SPWP) is often prescribed in triaxial shear tests by applying different back pressure levels (B.P.) corresponding to in-situ position of the water table. In this paper, several undrained and drained triaxial tests have been conducted to study the effect of SPWP on the mechanical behaviors of kaolin clay, which is often used to create artificial marine clay or as a subgrade material in the centrifuge tests of offshore geotechnical engineering. The magnitude of SPWP is found to impact undrained shear strength, initial growth rate of excess pore pressure, friction angle, while has little effect on initial undrained shear stiffness and ultimate excess pore pressure. A new unique line can be determined which is parallel to total stress path by fitting all the critical failure points during undrained shearing. The influence mechanism is considered to be related to a nonuniform distribution of SPWP in different kinds of pores, which makes the clay microstructure changeable under different SPWP. There may exist a “pseudo-suction” when certain SPWP is applied, resulting in a higher friction angle of kaolin clay.

1. Introduction

Kaolin clay has been widely used in the research of geotechnical engineering as an alternative test material instead of natural clay or other kinds of artificial clay, due to its standardization and relatively high permeability. Some researchers mixed it with sands, silts or other chemicals to simulate the hard-obtaining in-situ marine soils in laboratory tests[1-3]. Some other research groups made use of kaolin clay as a subsoil material in the centrifuge tests to study the natural geohazards, seabed mobility as well as offshore foundation systems[4-6]. Because of the wide use of kaolin clay in geotechnical experiments, its shear behavior has been studied by many researchers. Among these experimental researches, a back pressure (B.P.), which is the initial water pressure applied to the pore fluid in a test specimen to obtain full saturation of test samples[7], is often applied in the range of 100kPa to 250kPa.

According to Brand[8], back pressure should be equal to in-situ static pore water pressure (SPWP), whose value is governed by the position of the water table, to obtain more realistic strength parameters for geotechnical design because different magnitudes of SPWP develop in natural sedimentary clays and back pressure is just the SPWP within clay samples in essence. As a result, the range of back pressure adopted may exceed 250kPa, which is seldom adopted in triaxial tests of kaolin clay. In contrast, Hyodo et al[9]. applied different levels of back pressure (5, 10, 15MPa) corresponding to those existing in-situ to explore the triaxial mechanical properties of methane hydrate-bearing sand in deep seabed. Then a question has to be answered: whether different magnitudes of SPWP have effect on shear behavior of soils, especially kaolin clay?

Some results of laboratory tests show a more-or-less influence of back pressure (SPWP) on the mechanical response of various types of soils. A review of the key findings about this topic shows that the influence is dependent on both soil's type and mechanical behavior concerned. Regarding kaolin clay, Allam and Sridharan[10] did some research, but they only considered whether the back pressure was applied or not. Since then, numerous experimental investigations have been taken on the shear behavior of kaolin clay, among which, however, a constant back pressure was set for each research. Some of them declared the magnitude of back pressure used[11-14], while some others only mentioned a large enough back pressure was applied to ensure saturation[15-18].

Although different magnitudes of back pressure have to be adopted during triaxial shearing tests according to the in-situ hydrostatic conditions, experimental data on evaluating the role of SPWP on shear behavior of kaolin clay remains limited so far. In the current study, the effect of SPWP on shear behavior of saturated kaolin clay has been studied by performing a series of undrained and drained strain-controlled triaxial tests with different back pressure levels but the same effective consolidated stress. Based on the experimental results, a further assumption has also been brought out on the influence mechanism of static pore water pressure. This study will provide geotechnical researchers a better knowledge of SPWP as an influencing factor of shear behavior of kaolin clay.

2. Material properties and experimental program

2.1. Materials properties

The kaolin clay used in this study is Malaysian Kaolin consisting of 47.0-53.0% SiO2 and 32.0%-38.0% Al2O3 (provided by the supplier). The particle size of the initial clay powder is 2.5-4.5μ (2.7μ on average). The specific particle size distribution shows that 35% of the particles are in the silt size (0.002 to 0.075 mm), and the remaining 65% are clay (<0.002mm). Most of the mineralogy and properties of Malaysian Kaolin has been summarized in Table 1.

As pore-fluid chemical properties have a significant influence on the associated fabric formations at the stage of deposition and thus lead to different Atterberg limits and mechanical responses of kaolin clay during triaxial testing Wang and Siu[19]; Spagnoli et al. [20], the pH value and salinity of water used to prepare clay slurry were regarded as control variables in this research. When at a pH a little larger than the isoelectric point of edge surfaces (IEPedge) with some electrolyte, kaolin clay forms a flocculated and aggregated structure[21]. Such a flocculated-aggregated structure is very similar as the fabric of some natural sedimentary clays and thus was adopted in this research. Considering IEPedge=5 for kaolinite[19] and clay powders wound not be washed in the tests so that electrolyte wound exist, the water used to prepare clay slurry was controlled as deionized and pH≈5.5. In this case, the liquid limit and plastic limit of clay samples are 65% and 38% respectively. The Atterberg limits were obtained according to the British standards 5930 (BSI 1999). As mentioned above, there was no pretreatment by any steps of washing in spite of excess salts and impurities in clay powders, as the process is the same with the one of practical use of kaolin clay, e.g. in centrifuge tests.

Table 1: Mineralogy and properties of Malaysia kaolin clay

SiO_2 (%)	47.0-53.0*
Al_2O_3 (%)	32.0-38.0*
Grain size (clay:silt:sand)	13:7:0
Specific gravity	2.73
pH (30% solid)	3.0 – 5.0
Surface area (m^2/g)	14*
Atterberg limits	
Liquid limit (%)	65
Plastic limit (%)	38

*Provided by Kaolin (Malaysia) Sdn Bhd.

2.2. Sample preparation

In this study, a one-dimensional slurry consolidometer (60 mm diameter and 400 mm height) was used to prepare solid cylindrical specimens of kaolin clay. First, the kaolin clay powder was mixed with deaired water at a water content of 130%, about twice the liquid limit to guarantee the free movement of clay particles. After being stirred in an electric mixer for five minutes, the clay slurry was poured into the one-dimensional consolidometer carefully in case of the entrapment of air bubbles. The suspension was kept still for another six hours in order to leave clay particles settling. When some supernatant liquid above the kaolin sediment was observed, siphoned it and started to compress the sediment. The clay was consolidated to 150kPa by increasing the vertical pressure step by step (5, 10, 20, 50, 100, 150kPa). Each loading step lasted for six hours. When the last step of loading had been applied for six hours and the vertical displacement for each specimen was less than 0.1 mm in this step, the sample wound be taken out from the consolidometer and then stored in a humid environment until needed for triaxial testing.

2.3. Procedure of triaxial tests

The effect of back pressure on the shear behavior of kaolin clay was studied by conducting a series of triaxial undrained and drained compression tests at a uniform preconsolidated pressure value (pc) but different initial back pressure levels (B.P.). The preconsolidated pressure is set as 200kPa which is very frequently used in triaxial tests and B.P.=50, 200, 400, 600kPa were adopted to simulate the depths of 5, 20, 40, 60m below the water table. Each sample was first trimmed to a standard size of 39.2 mm in diameter and 80 mm in height along the direction of major principal stress before being placed on the triaxial cell pedestal. Filter paper strips were used to promote the radial and axial drainage conditions. In the stage of saturation, the prescribed back pressure and a little larger radial stress were applied to the specimen to avoid negative effective stress. The same water was stored in the back pressure controller. The back pressure was kept constant throughout the consolidation and drained shearing process. During undrained shearing, the back volume was set constant. After the saturation step, the Skempton's pore pressure coefficient B was measured just before the next consolidation step.

Following some previous experimental researches[11, 14], the triaxial shearing tests were strain-controlled at a limited axial strain rate of 0.05%/min for undrained condition and 0.008%/min for drained condition. To ensure the equalization and dissipation of excess pore pressure in undrained and drained conditions in this research, the following method given in Head and Epps[7] was adopted for the first test of B.P.=600kPa to validate the prescribed axial strain rate.

Fig. 1 shows the graph of volume change against square-root time during isotropic consolidation phase of the first triaxial test. Two auxiliary lines are made as shown in Fig. 1 and their intersection is found. The value $\sqrt{t_{100}}$ can then be read off from the horizontal scale ($\sqrt{t_{100}}$ =75), the time of 100% primary consolidation is thus obtained (t_{100}=5625 seconds=93.75 minutes). The time to failure (t_f) can be determined from Eq. (1)

$$t_{\mathrm{f}} = 1.8 \cdot t_{100} \quad \text{(undrained tests)} \tag{1}$$

$$\bar{\varepsilon}_{\mathrm{f}} = \dot{\varepsilon}_{\mathrm{a}} \cdot t_{\mathrm{f}} \tag{2}$$

So t_f =168.75 minutes; the failure strain for examination ($\bar{\varepsilon}_{\mathrm{f}}$) can be calculated from Eq. (2), which is equal to 8.4% in this case. By comparing $\bar{\varepsilon}_{\mathrm{f}}$ with the actual failure strain $\bar{\varepsilon}_{\mathrm{f}}$ (the strain at which the 'peak' deviator stress is reached) in Fig. 2(a), the one for examination is found to be much lower than the actual failure strain which is above 10%, proving that the value of shearing rate was small enough in this test.

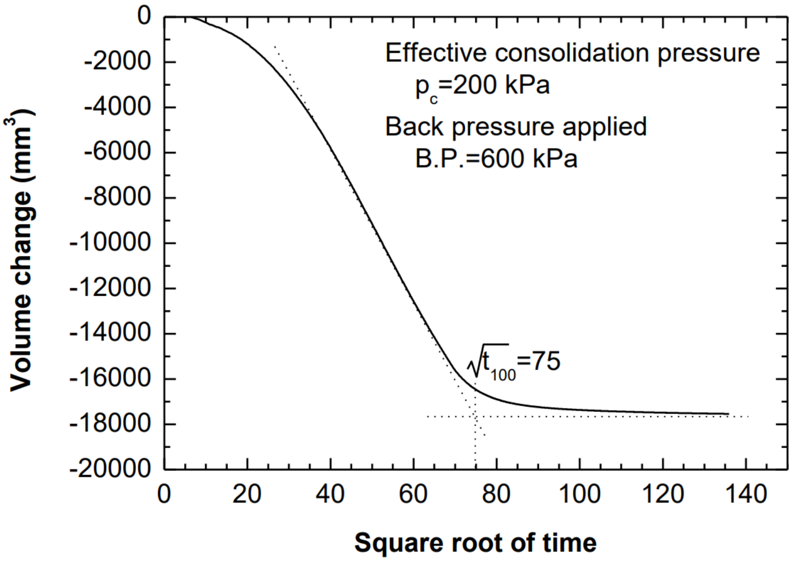

Figure 1: Volume change against square-root time during isotropic consolidation at 200kPa with B.P.=600 kPa

In addition, the test condition of *B.P.*=50kPa was not repeated in the drained test series due to the fact that the clay specimen remained unsaturated after the saturation stage if a low back pressure of 50kPa was applied and the undrained mechanical response in this case was quite different from the others.

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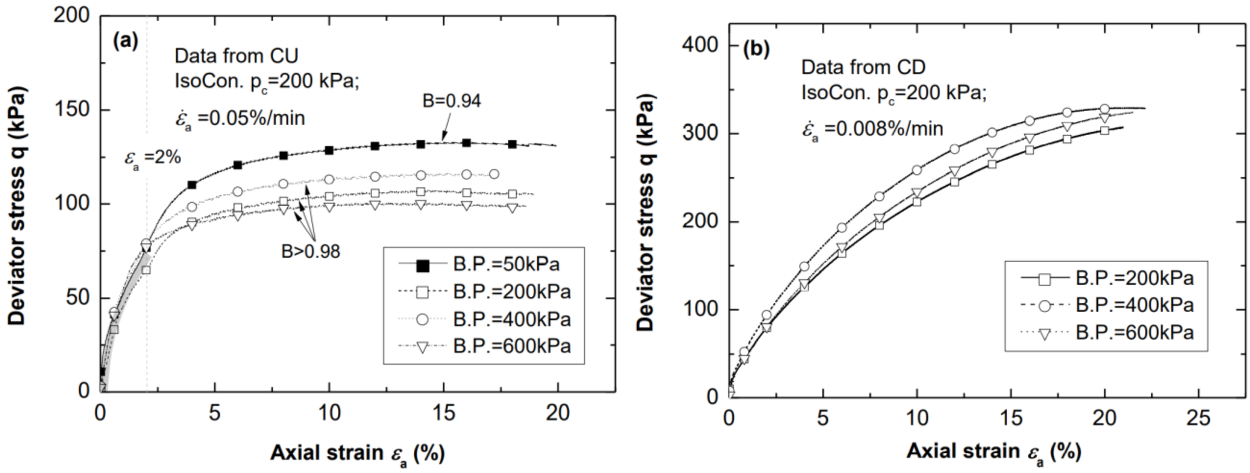
3. Test results and analysis

3.1. Stress-strain relationships

Fig. 2(a) shows the relationship between the deviator stress and the axial strain during undrained triaxial tests with different back pressure levels. All the samples were found to have a strain-hardening behavior rather than straining softening behavior, with maximum deviatoric stress (i.e. undrained shear strength q_f) occurred around the strain 14%. The deviatoric stress generally increased rapidly at low strains and slowly at high strains until failure. The undrained strength of kaolin clay with isotropic consolidated pressure of 200kPa ranges from 100kPa to 130kPa.

The *B* value, which is measured after the stage of saturation, has been marked out for each curve. Nearly all the *B* values were above 0.98, showing a near full saturation, except the one of *B.P.*=50kPa, whose *B* value was 0.94 and not large enough to be regarded as saturated (*B* should be over 0.95). Considering the remaining unsaturated state and the resulting matric suction in the test sample with *B.P.*=50kPa, it is no wonder that the undrained strength of this sample was the largest[22]. The effect of back pressure on the stress-strain behavior of saturated kaolin clay can be seen from the other three test curves. The sample with *B.P.*=400kPa showed the second largest undrained shear strength, followed by sample with *B.P.*=200kPa. The deviatoric stress of the one with *B.P.*=600kPa was the lowest. As shown in Fig. 2(a), the shear stiffness is similar for all the curves within the strain 2%.

The stress-strain responses for drained testing are presented in Fig. 2(b). Compared with the undrained test curves, the samples in drained conditions had a much larger failure strain, even beyond the strain-controlled limit of 20%. The maximum deviator stress q_f of specimens with different back pressure levels during drained shearing ranged from 303kPa to 309kPa. The variability of drained shear strength of kaolin clay with distinguished back pressures was found to be smaller under the same change of applied back pressures comparing to the undrained curves. In both drainage conditions, the two specimens with *B.P.*=400kPa had the largest maximum deviator stress. As the specimen with *B.P.*=400kPa 'always' behaved differently, authors thus start to think about the mechanism of this phenomenon. As for the other two groups, the influence of back pressure on the deviatoric stress-axial strain relationship was reversed in undrained and drained conditions.



(a) undrained (b) drained

Figure 2: Stress-strain responses for triaxial compression tests with different backpressures

3.2. Pore-pressure or volumetric responses

Fig. 3 shows the excess pore-pressure evolution normalized by initial isotropic consolidated pressure during undrained shearing. It can be seen that the excess pore pressure was building up during the whole loading process for different back pressure levels but at a different rate with increasing axial strain. For each pore pressure-strain curve, the initial slope remained large within

2% strain and then declined gradually to zero after 15% strain or so. By comparing different curves, it is found the gap between these curves starts to grow until reaching the maximum at a strain of 2%, and then it decreased and could almost be neglected at large strains, especially for the samples with *B.P.*=50, 200, 400kPa. The normalized excess pore pressure of the sample with *B.P.*=600kPa was a little higher than the other three. This difference was so small that excess pore pressure can be regarded independent of applied back pressure, which would be very useful as a rule. It is also worth noting that the larger back pressure, the higher initial growth rate of u/p_c as shown in Fig. 3, whose physical mechanism will be discussed later.

The volumetric response for kaolin clay with different applied back pressures is illustrated in Fig. 4, represented by the volumetric strain ε_v and the shear strain ε_q . In accord with the results of undrained tests, the volumetric response of samples in drained tests was contractive as well, noting that positive volumetric strain represents a decrease in the volume of the sample. Compared with Fig. 3, the gap between curves in Fig. 4 kept increasing. Considering the failure strain was beyond the measured one, it is hard to answer whether these curves would converge or diverge. As far as the recorded strain range is concerned, the volumetric response of kaolin clay during drained shearing was influenced by back pressure, especially for large shear strains.

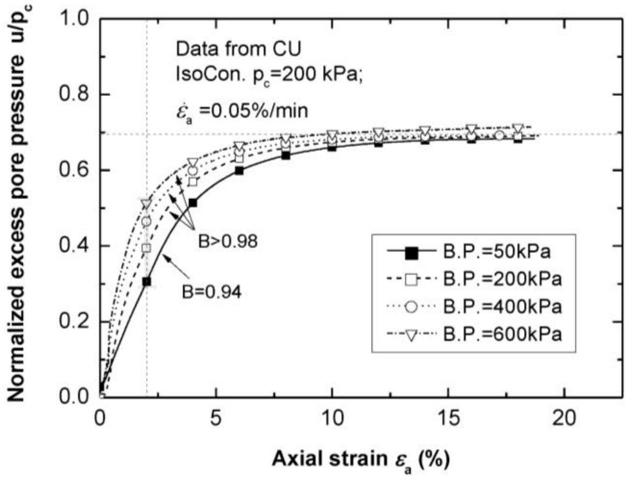


Figure 3: Pore pressure-strain responses for undrained tests

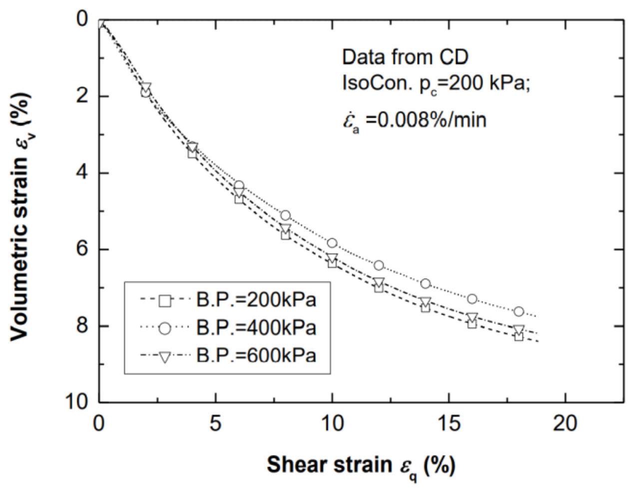


Figure 4: Volumetric responses for drained tests

3.3. Analysis of the test results

The implication of the test data presented in this paper is important for the evaluation or consideration of SPWP on the shear behavior of saturated kaolin clay. Considering the undrained and drained triaxial test results of kaolin clay using different back pressure levels in Fig. 2 to Fig. 4, some typical experimental phenomena can be emphasized as follows. First, the undrained shear strength of samples with the same effective consolidated stress differs under different magnitudes of SPWP, but their initial shear modulus remains similar. Second, the ultimate excess pore pressure built up in samples during undrained shearing is independent of SPWP, while its initial growth rate rises with increasing SPWP. Third, critical state line is unique for samples with the same SPWP and effective consolidated stress, but it differs with variation of SPWP, in particular, the slope of CSL for SPWP=400kPa is larger. These experimental results do not agree with most of the previous results with these exceptions: Brand[8], Allam and Sridharan[10] and Åhnberg[23]. The disagreement may be contributed to the distinguished soil composition used in tests.

Fig. 5 shows the most important findings in the current study. Based on the aforementioned experimental results, two dashed guiding line (GL) which are parallel to the total stress path (TSP) are plotted in Fig. 5. It seems that all of the effective stress paths of *B.P.*=200, 400, 600kPa end at the blue guiding line. The effective stress path of *B.P.*=50kPa end at the red guiding line. Considering the gap of the two dashed guiding lines is equal to the one of the two TSPs, the conclusion can be expressed as: all the effective stress paths in p-q plot of saturated kaolin clay with the same effective consolidated stress but different SPWP finally end on a unique line that is parallel to their total stress path during undrained shearing.

This finding is so simple and compendious that far more experimental tests should be taken to examine it. Some hypothesis and key points should be repeated that (1) the slope of CSL of each test group with the same SPWP is an average, otherwise the ultimate states of samples with *B.P.*=200, 600kPa in Fig. 2(a) and Fig. 2(b) will contradict with each other; (2) difference of the normalized excess pore pressure in the sample with *B.P.*=600kPa from the one in other samples is neglected to obtain the independent relationship between SPWP and excess pore pressure, that is why only the curve of *B.P.*=600kPa in Fig. 5 has some incongruity.

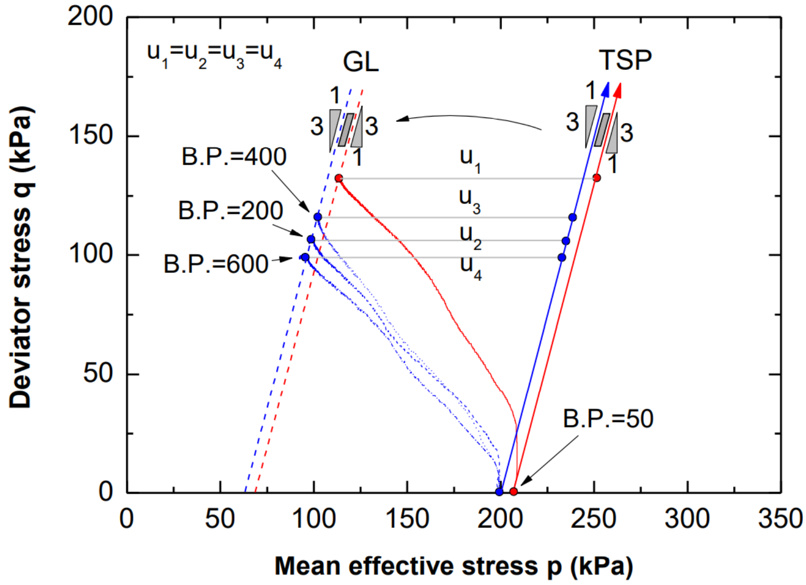


Figure 5: Relationship between critical states of undrained samples and their total stress paths

4. Conclusions

In this paper, the effect of static pore water pressure on shear behavior of saturated kaolin clay has been studied by conducting several groups of triaxial tests. The description of drained and undrained test results includes: (1) the ultimate excess pore pressure built up in samples during undrained shearing is independent of the magnitude of SPWP, but its initial slope rises with increasing SPWP; (2) the magnitude of SPWP has impact on the slope of CSL, i.e. friction angle of kaolin clay; (3) SPWP has little effect on initial undrained shear stiffness; (4) the influence of SPWP on the mechanical response during drained shearing is different from the undrained one. The most important conclusion can be expressed that all the effective stress paths in p-q plot of kaolin clay with the same effective consolidated stress but different SPWP finally end on a unique line that is parallel to their total stress path during undrained shearing.

The mechanism of the influence on shear behavior of saturated kaolin clay induced by SPWP is considered to be related to a nonuniform distribution of SPWP in different kinds of pores, which makes the clay microstructure changeable under different SPWP. When SPWP is low, clay aggregates are relatively free to move. With SPWP rising up, more aggregates are "squeezed" into a bigger and firmer assembly. Slip surface may develop in different microscopic locations at critical state. There may exist a "pseudo-suction" when certain SPWP is applied, resulting in a higher friction angle of kaolin clay. In addition, SPWP has a positive effect on the modulus of pore water and thus increases the initial growth rate of excess pore pressure. The similar ultimate excess pore pressure results from the balance of changes of pore-water modulus and degree of clay microstructure rearrangement. Verification of these interesting conclusions requires more experimental studies. As a result, the current study serves as a preliminary exploration to draw some attention to these experimental phenomena.

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