Mechanical Properties of an unsaturated buffer material in the high suction ranges

Janaka J. Kumara[1], Takeshi Kodaka[2], Ying Cui[3], Keisuke Takeuchi[4]

1 Postdoctoral Fellow, Department of Civil Engineering, Meijo University, jjkumara@meijo-u.ac.jp
2 Department of Civil Engineering, Meijo University, kodaka@meijo-u.ac.jp
3 Department of Civil Engineering, Meijo University
4 Department of Civil Engineering, Graduate School of Science and Technology, Meijo University

Abstract

Sand-bentonite composite material is widely used as a buffer material in nuclear waste repositories thanks to their excellent swelling properties and high impermeable characteristics. The unsaturated buffer material at the construction state may become saturated (at least partially) when it is opened for the operation since it is subjected to local groundwater conditions. In this study, we investigated the strength properties of sand-bentonite buffer material subjected to various degree of saturation. The volumetric behaviour was also investigated by a newly developed double-cell type triaxial testing apparatus. The experimental results suggest that, on overage both the undrained cohesion and internal friction angle reduce with the degree of saturation. The undrained cohesion however remains more or less same during the early saturation (e.g., roughly up to 50% of S_r; S_r is degree of saturation). The undrained friction angle reduces drastically when the specimens are near the quasi-saturation state. The results also indicate that the confined compressive strength increases with confining pressure. It was also found that the specimens except the ones with initial moisture content (of bentonite) exhibit continuous compression under a high confining pressure (i.e., 0.5 MPa). In contrast, the specimens under a small confining pressure (i.e., 0.1 MPa) yield expansion in post-failure state after exhibiting compression in pre-failure state.

Keywords: buffer material, nuclear waste repository, sand-bentonite, triaxial compression test, volumetric behaviour

1. Introduction

Nuclear wastes are stored at the nuclear waste repositories constructed at very deep grounds. Japanese government states that high level nuclear wastes should be stored at least 300 m below the ground surface using multi-barrier system (Kodaka and Teramoto, 2009). Leakage of nuclear waste materials could be fatal. Therefore, the nuclear waste materials should be sealed tightly by a buffer material. The buffer material acts as a sealing material between the nuclear wastes stored and the surrounding rocks as seen in Fig. 1. Bentonite-mixed composite materials have been widely used as buffer materials for nuclear waste facilities due to their excellent swelling characteristics and high nature of impermeability. The main function of the buffer material is to create an impermeable zone around the waste containers and the surrounding environment. The buffer material is also expected to swell and thereby fill up any spaces around it (Komine and Ogata, 2004). Since bentonite can absorb water and has excellent swelling characteristics, bentonite-based materials work well as buffer materials for nuclear waste repositories.

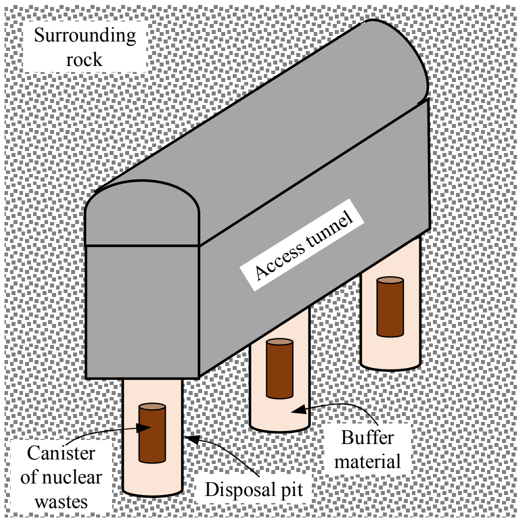


Fig. 1 A schematic diagram of a nuclear waste repository (inspired by Komine and Ogata, 2004)

The buffer material at the construction state of a nuclear waste repository is basically under unsaturated condition. However, when the nuclear waste facility is opened for its usual operation, the buffer material could be subjected to local water flow and eventually become a saturated material, at least partially. The swelling characteristics of sand-bentonite or bentonite-mixed buffer materials have widely been studied (Agus et al., 2010; Schanz et al., 2010; Wang et al., 2012), but comparatively there is very little knowledge on the strength behaviour of unsaturated bentonite-mixed buffer materials. Kodaka and Teramoto (2009) has previously reported that the shear behaviour of saturated and unsaturated sand-bentonite composite materials are different. However, their results have been obtained using direct shear test, which does not simulate the actual soil stress conditions as a triaxial compression test.

In this study, we investigated the strength properties of sand-bentonite buffer material subjected to various degree of saturation. The volumetric behaviour of the sand-bentonite composite material was also studied using a newly developed double-cell triaxial testing apparatus.

2. Methodology

2.1 Materials and sample preparation

The buffer material was produced using 70% of bentonite and 30% of silica sand referring to the Japanese specifications on the buffer materials (Mitachi, 2008). The bentonite is sodium-type bentonite, and called as Kunigel-VI. It contains around 48% of montmorillonite (Komine and Ogata, 1999; Ye et al., 2014). This bentonite is frequently used in artificial barriers against nuclear wastes in Japan (Komine and Ogata, 1999; Mitachi, 2008). The gradation curve of sand is shown in Fig. 2. The basic properties of bentonite and sand are given in Table 1. As given in Table 1, the bentonite is a water absorbing material. Therefore, under room temperature, it may contain some water in it. Therefore, a sand-bentonite composite specimen prepared without adding any water could still has some water, which is called herein as the initial water content. The sand-bentonite specimens were prepared by adding different amount of distilled water such that the specimens have various degree of saturation, which is evaluated using Eq. (1). The particle density of the composite material was evaluated using the particle density of the individual materials (see Table 1) based on their mixing proportions.

$$S_r = \frac{w\rho_s}{e\rho_w} \times 100(\%) \tag{1}$$

Where S_r is degree of saturation, w is water content, ρ_s is particle density, e is void ratio and ρ_w is density of water (i.e., 1000 kg/m^3).

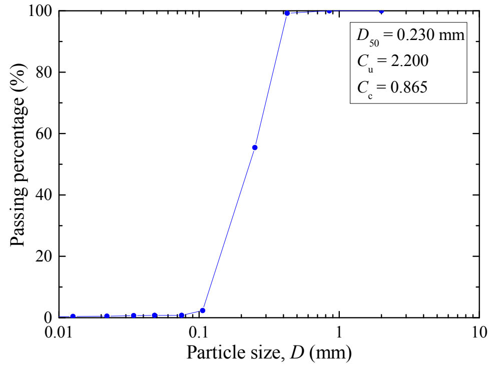


Fig. 2 The gradation curve of sand (C_u and C_c are coefficients of curvature and uniformity respectively)

Table 1 The basic properties of bentonite and sand

Property	Bentonite	Sand
Particle density, ρ_s (kg/m^3)	2767	2666
Liquid limit, w_L (%)	430.5	n/a
Plastic limit, w_P (%)	26.7	n/a
Plasticity index, I_p	403.8	n/a
Montmorillonite content (%)	48[a]	n/a

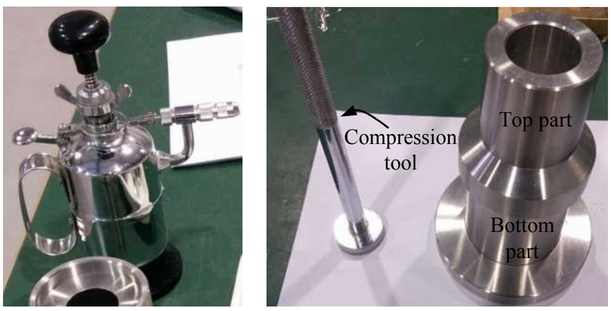
Note [a]Komine and Ogata (1999)

The specimens were prepared with a constant dry density of 1600 kg/m^3. The amount of water to be added to a sand-bentonite mixture can be evaluated using Eq. (1) when the degree of saturation is first designed since the void ratio is a function of dry density (see Eq. (2)), which was already designed at 1600 kg/m^3. The particle density of the sand-bentonite composite material (ρ_s in Eq. (2)) was calculated as 2737 kg/m^3.

$$e = \frac{\rho_s}{\rho_d} - 1 \tag{2}$$

Where ρ_d is dry density.

After bentonite and sand were taken to a container, the designed water amount was added using a high-pressure water sprayer (see Fig. 3a) while mixing the two materials properly. Since bentonite absorbs water quickly, bentonite and sand should be mixed quickly and uniformly while spreading water by small amounts. After a wet sand-bentonite material is prepared, the material was poured into the mold shown in Fig. 3b. The mold was filled in three attempts, and compressed manually during each attempts using a small bottom-flat tool of the same diameter as the mold (see Fig. 3b). The mold consists of two parts. Initially, the bottom part of it is used. The top part is connected to the bottom part for the last attempt before completely adding the material. Then, the specimen in the mold was compressed using the hydraulic jack shown in Fig. 4a to get the pre-designed dry density of 1600 kg/m^3. The hydraulic jack has a capacity of 100 kN, stroke of 120 mm and pressure of 68.9 MPa. It has a manually-controlled lever which is used to apply the hydraulic pressure. The compressed specimens have a height of roughly 80 mm. 5 mm each from the top and bottom parts of a specimen are trimmed to avoid any inhomogeneity at the edges of the specimen. A prepared specimen is shown in Fig. 4b. The trimmed soils were used to measure the water content of the specimens. The specimens of 70 mm in height and 35 mm in diameter were used for the triaxial compression tests. Two identical specimens of the same degree of saturation were prepared to obtain the strength parameters using the Mohr circle. Table 2 gives the basic information of the specimens. In the specimen notation (see Table 2), e.g., PS5-2, PS denotes “partially-saturated”, 5 denotes confining pressure of 0.5 MPa (1 for 0.1 MPa of confining pressure) and 2 denotes simply the specimen number under the same confining pressure. In Table 2, PS1-1 and PS5-1 specimens were prepared without adding water. Therefore, the specimens contain the initial water content of bentonite (i.e., around 6.7-7.0%). PS1-6 and PS5-6 specimens were prepared with a high water content (e.g., around 20%) such that they are near the quasi-saturation state (i.e., S_r > 90%; S_r is degree of saturation). The water content and the degree of saturation given in Table 2 are the measured values, which are slightly deviated from the designed values, probably due to slightly uneven distribution of water across the sample and/or loosing of water during specimen preparations (e.g., due to evaporation, etc.).



(a) (b)

Fig. 3 (a) The high-pressure water sprayer and (b) the mold used to prepare the specimens with its manual compression tool

2.2 Testing apparatus and loading condition

Unconsolidated undrained (UU) triaxial compression tests were conducted to investigate the strength properties of sand-bentonite specimens. The UU tests were conducted to simulate the field conditions appropriately. The triaxial compression tests were conducted under 0.1 and 0.5 MPa of confining pressures to obtain the strength parameters (i.e., cohesion and internal friction angle). The specimens were covered with a 3 mm thick membrane to avoid water squeeze into it in addition to the usually applied measures. The testing set up was developed such that the confining pressure applied to the outer cell also goes into the inner cell (see Fig. 5). Therefore, the confining pressure applied to the outer cell is also applied on the specimen mounted in the inner cell.

(a) (b)

Fig. 4 (a) The hydraulic jack and (b) a prepared specimen

Table 2 The basic information of the specimens

Specimen notation	Water content, w (%)	Degree of saturation, S_r (%)	Confining pressure, σ_c (MPa)
PS1-1	6.98	29.0	0.1
PS5-1	6.71	28.1	0.5
PS1-2	10.01	41.8	0.1
PS5-2	10.33	42.9	0.5
PS1-3	11.36	49.0	0.1
PS5-3	11.24	46.4	0.5
PS1-4	14.05	62.7	0.1
PS5-4	13.13	62.4	0.5
PS1-5	16.24	76.2	0.1
PS5-5	17.08	73.3	0.5
PS1-6	21.14	86.8	0.1
PS5-6	19.60	85.9	0.5

The volume change of the specimens were also measured using the newly developed double-cell type triaxial apparatus shown in Fig. 5. The volume change of the specimen is measured by the volume change of a burette attached to the triaxial testing apparatus. The volumetric strain is evaluated using Eq. (3). The volume change of a specimen is governed by the volume change in the burette and the part of the loading rod entered into the inner cell as given in Eq. (4). The volume change in the burette is measured by a differential pressure transducer, which has a capacity of 10 kPa.

$$\varepsilon_{vol} = \frac{\Delta V_s}{V_{s,i}} \times 100(\%) \tag{3}$$

Where ε_{vol} is volumetric strain, ΔV_s is the volume change of a specimen and $V_{s,i}$ the initial volume of the specimen.

$$\Delta V_s = \Delta V_b - V_{lr} \tag{4}$$

Where ΔV_b is the volume change in the burette and V_{rl} is the volume of water replaced by the part of the loading rod (i.e., a cylindrical part with 8 mm diameter) penetrated into the inner cell.

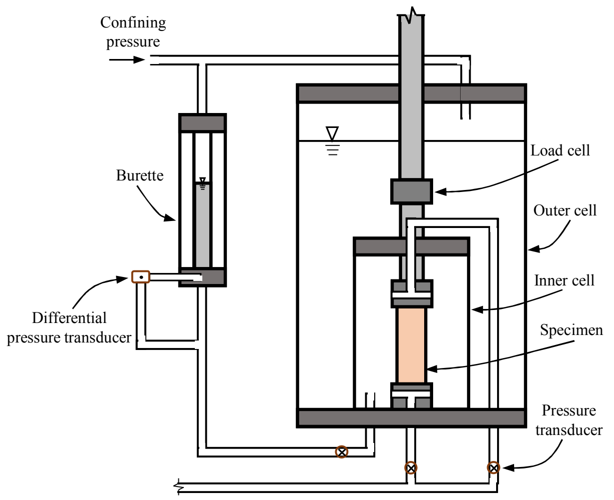


Fig. 5 Schematic diagram of the triaxial testing apparatus

The vertical load was applied with a loading rate of 0.5%/min. The shearing is conducted under undrained condition while the specimen is subjected to isotropic confining pressure. The load is applied by a Mega-torque motor. The load cell has a capacity of 10 KN. The confining pressure is applied by a pneumatic cell pressure loading system, and measured by a pressure transducer which has a capacity of 5 MPa. The loading was continued until the specimen reaches 15% of axial strain. The triaxial compression tests were performed according the Japanese standard (i.e., JGS 2009).

3. Results and Discussion

Figs. 6a and 6b show the stress-strain relationships of the specimens subjected to 0.1 and 0.5 MPa of confining pressure respectively. Fig. 6a indicates that less-saturated specimens (e.g., S_r < 63%; S_r is degree of saturation) exhibit strain-softening behaviour under a low confining pressure. That indicates a less-saturated sand-bentonite material under a small confining pressure behaves as a brittle rock, which yields a clear peak stress followed by a stress reduction. In contrast, a highly saturated sand-bentonite composite material (particularly near its quasi-saturation state) under a small confining pressure exhibits strain-hardening behaviour as a loose sand (see Fig. 6a). However, under a high confining pressure (e.g., 0.5 MPa), all the specimens except the ones under its initial moisture content of bentonite exhibit strain-hardening behaviour (see Fig. 6b). The results of stress-strain relationships also suggest a higher confining pressure yield a higher deviator stress. The results suggest that the strength properties decrease with the degree of saturation, which indicates the high strength properties of sand-bentonite composite material at the construction state weakens during the operation of a nuclear waste repository.

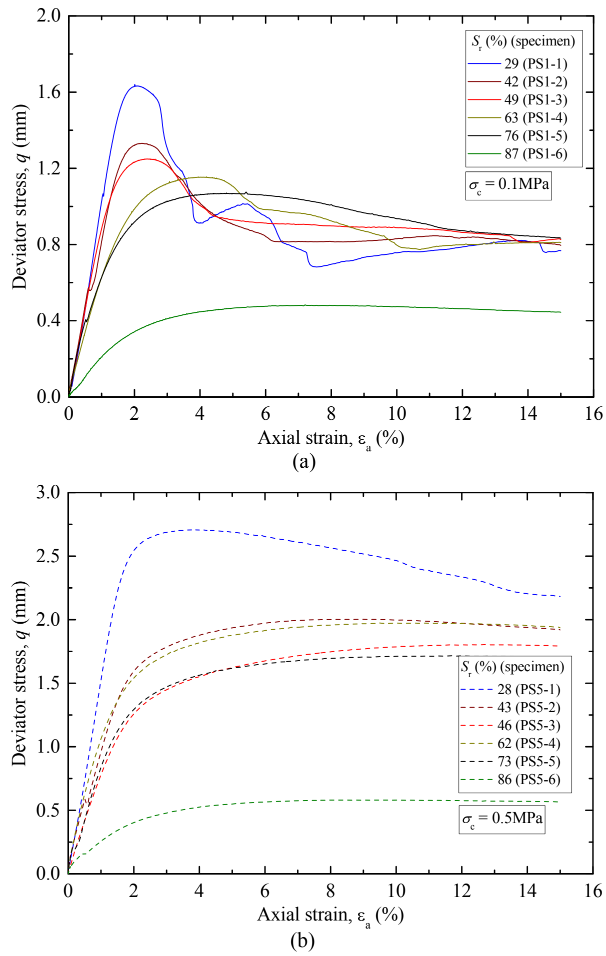


Fig. 6 Stress-strain relationships under (a) 0.1 and (b) 0.5 MPa of confining pressure respectively (S_r is degree of saturation)

Figs. 7a and 7b show the volumetric behaviour of the specimens subjected to 0.1 and 0.5 MPa of confining pressure respectively. In Figs. 7a and 7b, positive values of ε_{vol} indicate the compression behaviour while negative values of ε_{vol} indicate the expansion behaviour of the specimens. The specimens of initial moisture content of bentonite (i.e., PS1-1 and PS5-1 specimens) exhibit compression in pre-failure state followed by expansion in post-failure state independently of the confining pressure. As shown in Fig. 7a, the specimens start to yield less expansion in post-failure state with increasing degree of saturation under a small confining pressure (i.e., 0.1 MPa). As shown in Fig. 7b, under a high confining pressure (i.e., 0.5 MPa), all the specimens apart from the specimen of the initial moisture content (of bentonite) exhibit compression behaviour. It was also observed that the specimens of high water contents (i.e., except PS5-1 and PS5-2 specimens) continue to exhibit compression in post-failure state too. The compression decreases with the degree of saturation. The results hence suggest that a sand-bentonite buffer material reduces its volumetric expansion in residual strength state with time under a small confining pressure (assuming that the buffer material is subjected to continuous water flow during its operation).

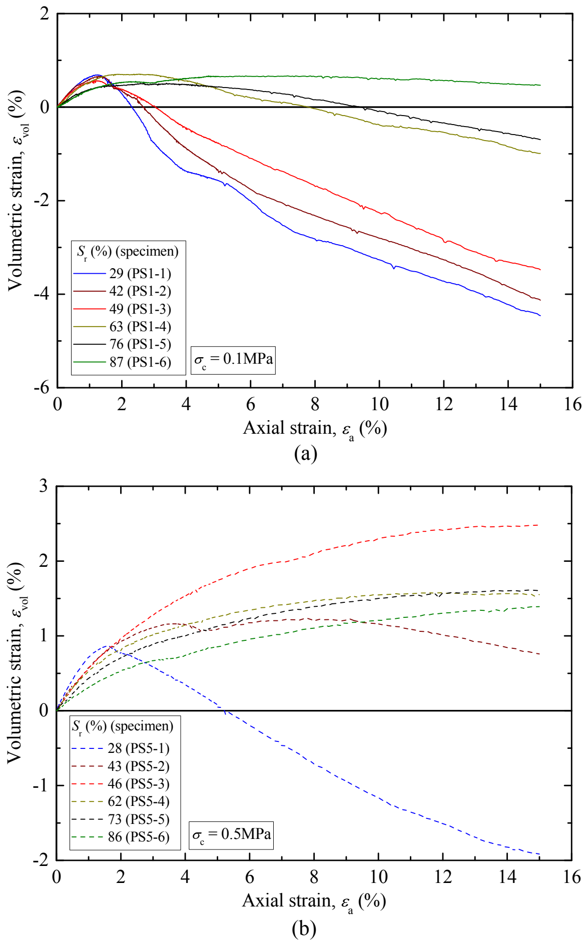


Fig. 7 Volumetric behaviour of the specimens under (a) 0.1 and (b) 0.5 MPa of confining pressure respectively (S_r is degree of saturation)

Fig. 8 shows a typical Mohr circle and its failure envelope. In case of the specimens with strain-hardening behaviour, the maximum stresses (i.e., axial and lateral stresses) either at the end of the testing (i.e., ε_a = 15%; ε_a is axial strain) or near it were considered to draw the Mohr stress circles. Fig. 9 shows the variation of strength parameters of the specimens with the degree of saturation. The degree of saturation given in Fig. 9 is the average of the two identical specimens tested under 0.1 and 0.5 MPa of confining pressure. The results indicate that, on average both cohesion and friction angle reduce with the degree of saturation. However, undrained cohesion remains unchanged during early saturation (i.e., up to 48% of S_r; S_r is degree of saturation). In contrast, undrained friction angle decreases with the early saturation, and then indicates a slight increase at 62% of S_r followed by continuous reduction. The results of friction angle indicates that a high as 35 degree friction angle of sand-bentonite buffer material at the construction state reduces to low as 6 degree at the quasi-saturation state. In comparison, the reduction of cohesion of the buffer material is not as high as friction angle since it reduces only from 0.36 MPa to 0.20 MPa from the construction state to quasi-saturation state (see Fig. 9). In a previous study, Cho et al. (2002) also reported that unconfined compressive strength of Kungel-VI bentonite decreases with water content, which agrees well with the findings of this study.

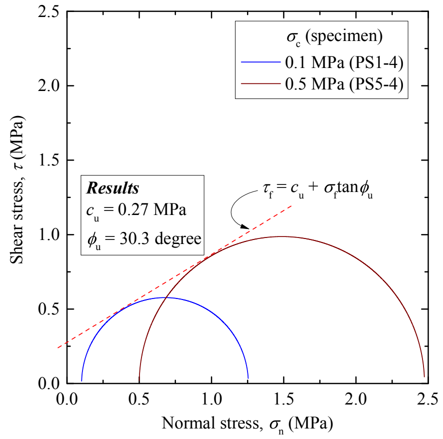


Fig. 8 A typical Mohr circle and its failure envelope (S_r = 62.6%; S_r is degree of saturation and σ_c is confining pressure)

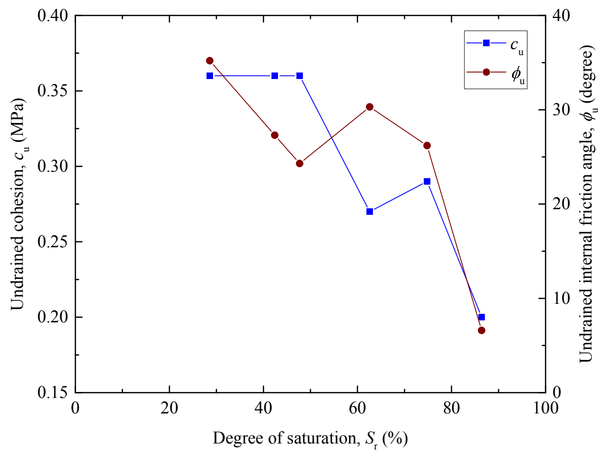


Fig. 9 The variation of undrained cohesion and internal friction angle with the degree of saturation

4. Conclusions

The strength and volume change behaviour of sand-bentonite buffer material of nuclear waste repositories were investigated by unconsolidated undrained (UU) triaxial compression tests. The specimens were prepared with various degree of saturation to understand the behaviour of buffer materials after the operation of a nuclear waste facility. The following conclusions were drawn from the study.

The specimens exhibit strain-softening behaviour under a small confining pressure (i.e., 0.1 MPa), particularly when they are still in the early saturation (e.g., roughly 50% < S_r; S_r is degree of saturation). When the degree of saturation becomes higher, the specimens start to show strain-hardening behaviour. In contrast, all the specimens apart from the one with its initial moisture content (of bentonite) under a high confining pressure (i.e., 0.5 MPa) exhibit strain-hardening behaviour. The specimens with a higher confining pressure also give higher stresses.

It was also found that the specimens except the ones with initial moisture content (of bentonite) exhibit continuous compression behaviour (both in pre- and post-failure states) under a high confining pressure (i.e., 0.5 MPa). In contrast, the specimens under a small confining pressure (i.e., 0.1 MPa) yield expansion in post-failure state after exhibiting compression in pre-failure state. The magnitude of the final expansive strain is greatly influenced by the degree of saturation, with the natural sand-bentonite specimen (i.e., which has the initial moisture content of bentonite) yielding the maximum values while the specimen near to its quasi-saturation state showing the smallest value.

The strength properties are influenced largely by the degree of saturation, particularly when the specimens near the quasi-saturation state. The undrained friction angle decreases with the degree of saturation, particularly indicating noticeable reduction near the quasi-saturation state. The undrained cohesion remains unchanged at early saturation, then start to decrease with the degree of saturation. It was also observed that the confined compressive strength increases with the confining pressure.

Reference

1. Agus, S.S., Schanz, T. and Fredlund, D.G.: Measurements of suction versus water content for bentonite-sand mixtures, Canadian Geotechnical Journal, Vol. 47, No. 5, pp. 583-594, 2010.
2. Cho, W.-J., Lee, J.-O. and Kang, C.-H.: A compilation and evaluation of thermal and mechanical properties of bentonite-based buffer materials for a high level waste repository, Journal of the Korean Nuclear Society, Vol. 34, No. 1, pp. 90-103, 2002.
3. JGS 0521: Method for unconsolidated-undrained triaxial compression test on soils, Japanese Geotechnical Society (JGS) Standard, Tokyo, Japan, 2009.
4. Kodaka, T. and Teramoto, Y.: Shear failure behavior of compacted bentonite, Proc. International Symposium on Prediction and Simulation Methods for Geohazard Mitigation, Kyoto, Japan, pp. 331-337, 2009.
5. Komine, H. and Ogata, N.: Experimental study on swelling characteristics of sand-bentonite mixture for nuclear waste disposal, Soils and Foundations, Vol. 39, No. 2, pp. 83-97, 1999.
6. Komine, H. and Ogata, N.: Predicting swelling characteristics of bentonite, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 8, pp. 818-829, 2004.
7. Mitachi, T.: Mechanical behavior of bentonite-sand mixtures as buffer materials, Soils and Foundations, Vol. 48, No. 3, pp. 363-374, 2008.
8. Schanz, T., Arifin, Y.F., Khan, M.I. and Agus, S.S.: Time effects on total suction of bentonites, Soils and Foundations, Vol. 50, No. 2, pp. 195-202, 2010.
9. Ye, W.M., Borrell. N.C., Zhu, J.Y., Chen, B. and Chen, Y.B.: Advances on the investigation of the hydraulic behavior of compacted GMZ bentonite, Engineering Geology, Vol. 169, pp. 41-49, 2014.
10. Wang, Q., Tang, A.M., Cui, Y.-J., Gelage, P. and Gatmiri, B.: Experimental study on the swelling behaviour of bentonite/claystone mixture, Engineering Geology, Vol., 124, pp. 59-66, 2012.