Research Article

Experimental Study on Swelling Behavior and Its Anisotropic Evaluation of Unsaturated Expansive Soil

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Evaluation of swelling behavior is important for designing structures in expansive soils areas, especially for highway that the swelling pressure generated upon pavement and retaining structures in both vertical and horizontal directions due to infiltration. In this study, modification was made on unsaturated consolidation oedometer to provide synchronized measurement of vertical swelling strain (VSS) and lateral pressure (LP) of expansive soil under constant net normal stress and controlled matric suction. Vertical swelling (VS) test and lateral swelling (LS) test were conducted to investigate the anisotropic swelling behavior. The influence of mean net stress and net stress ratio on VSS was investigated, and the anisotropic swelling behavior of unsaturated expansive soil was characterized using anisotropic swelling ratio. The results show that the VSS nonlinearly decreased as the mean net stress increased and increased as the net stress ratio increased. The expansive soil would rapidly enter the passive state due to lateral swelling pressure under relatively low surcharge, with major principal axis rotating from vertical direction to lateral direction, which advances the possibility of passive failure for light retaining structures. The anisotropic swelling behavior objectively exists and varies with matric suction and net normal stress, which should not be ignored for engineering application.

1. Introduction

Expansive soils are widely distributed all over the world. More and more expressways have been built with direct utilization of expansive soil as subgrade fill materials in Southern China, a humid and subtropical climate zone with hilly and mountainous terrain [1]. Ascribed to the montmorillonite and its mixed layer mineral, the expansive soils exhibit large volume change when the upper load and/or their moisture content varies [2–4]. If this volume change is restricted, the remarkable swelling pressures can be created [5–8], resulting in severe damage to surrounding structures [9–12]. With the water content increasing to equilibrium water content due to capillary effect and water migration through the cracks in pavements, the expansive soils as underlay in pavement not only swell vertically but also swell laterally. It was reported that the lateral swelling pressure would not be equal to the vertical swelling pressure and might exceed the vertical swelling pressure in some cases [13, 14]; the shoulder wall would be easily damaged when suffering from such considerable lateral pressures, and subsequently the pavement structure fails. The demand for a high-quality pavement has increased rapidly in recent years [15]; it is thus important for durable pavement to evaluate the anisotropic swelling behavior of expansive soil.

Problems with respect to the lateral swelling pressure of expansive soils have been of particular concern during the past several decades. Several measurements of lateral pressure were conducted by using kinds of techniques; Komornik and Zeitlen [16] modified an oedometer ring with a thin wall section using electrical strain wires to determine the lateral pressure, where the lateral pressure was found much larger than the vertical loads after inundation. Ofer [17] pointed out that the errors in measurement of lateral...
pressure would occur if any lateral strains were allowed in oedometer ring. A similar apparatus with lateral strain compensation was developed for testing compacted clay samples with different initial densities under constant vertical pressure; the results showed that the lateral swelling pressure increased as initial density increased. Avsar et al. [13], Windal and Shahrou [18], Monroy et al. [19], and Abbas et al. [20] also used the same technique to investigate the lateral pressure and the axial strain. Besides, the hydraulic triaxial apparatus that the stress could be controlled in both axial and radial direction [21–24] and 3D swelling shrinkage apparatus that the boundary condition could be changed [25–27] were also commonly used to investigate the volume change behavior of expansive soil after hydration. Meanwhile, other testing techniques for measuring swell pressure were also developed. Kate and Katti [28] used a container tank to investigate the variation of lateral pressure of black cotton soil with various surcharge, where the lateral pressure was measured by the dial gauge through the movement of piston. Ikizler et al. [29] designed a rigid steel cubic box (300 mm × 250 mm × 300 mm) with soil pressure transducer attached on the surface of the box, for the measurement of swelling pressures in both lateral and vertical directions under constant volume condition. Zhang et al. [30] used an adjustable oedometer ring to measure the lateral swelling pressure and vertical swelling strain under different vertical stress after immersion and lately developed a cubic testing device for the measurement of lateral swelling pressure and its variation with swelling strain subjected to different vertical stress and dry densities [31]. Results of these studies revealed that the lateral pressure increased with the increment of inundated vertical pressure under lateral confined condition [18, 32]. The variation of lateral pressure with time showed a fast increment in the early stage of wetting until approaching to a peak value, and then the lateral pressure would reduce and lately maintain stable [29, 33, 34]. Moreover, anisotropic swelling behavior, the measured lateral swelling pressure which differs from the measured vertical swelling pressures after inundation, was found by many researchers [2, 9, 35, 36], which should be attributed to particle orientation. During consolidation process, clay particles that randomly oriented are preferred to rotate to the direction perpendicular to the direction of major consolidation stress [37]. Swelling pressures in the direction perpendicular to the particle orientation is greater than that in the direction parallel to the particle orientation [4]. However, most of the previous studies were conducted from an initially unsaturated state to a fully saturated state, which failed to provide the lateral pressure of expansive soil with respect to changes in matric suction. Apparently, lateral pressure of expansive soil mobilizes because of rainfall infiltration or moisture exchange from the environment [38, 39] and the development of lateral pressure with matric suction, and the influence of net normal stress and matric suction upon swelling need further understanding.

This paper tried to evaluate the swelling anisotropy of unsaturated expansive soil under controlled matric suction. Firstly, a retractable oedometer ring integrated with a micro-Earth pressure was used in unsaturated consolidation oedometer for simultaneously synchronized measurements of vertical swelling strain (VSS) and lateral pressure (LP) generated by expansive soil. Secondly, considering the swelling anisotropy caused by particle orientation, modification was made upon the specimen preparation method. Swelling tests with decreased matric suction and constant net normal stress were performed to the specimen prepared by convention preparation method, and the one prepared by modified preparation method, respectively. Furthermore, a comparative study was conducted based on the swelling behaviors obtained from two different swelling tests, and the effect of mean net stress and net stress ratio on volume change was further discussed. At last, a heave estimation method considering the influence of mean net stress and matric suction was proposed, and an anisotropic swelling ratio was introduced to characterize the swelling anisotropy of unsaturated expansive soil.

2. Test Apparatus

2.1. Test Apparatus. The unsaturated consolidation (UC) oedometer was modified for synchronized measurement of VSS and LP of soil specimen under the formed net normal stress and matric suction, which consists of loading system, testing system, monitoring system, and air and water control system, as shown in Figure 1.

In testing system, the traditional oedometer ring was replaced by a self-made retractable ring with an Earth pressure sensor integrated for the measurement of LP during testing, as shown in Figure 2.

The retractable ring uses stainless-steel material with the wall thickness of 1.5 mm to ensure adequate stiffness. For the convenient placement of specimen, an adjustable opening joint and an adjusting screw were designed to slightly control the opening and closing of the retractable ring. On the opposite side, a groove (20 mm × 18 mm × 20 mm) was designed for placing the micro-Earth pressure sensor, curved plate, and plastic acrylic separator. The soil specimen (ϕ61.8 mm × 30 mm) was surrounded by the ring wall and the curved plate during the test. The lateral pressure generated from the specimen would be delivered to the Earth pressure sensor though the stainless-steel curved plate. The vertical surface of the groove was smeared with lubricant for reducing the friction with the curved plate. The micro-Earth pressure sensor (ϕ20 mm × 12 mm) is a resistance strain type, which can measure the pressure ranging from 0 to 1000 kPa with an accuracy of ±1.0 kPa. Errors would occur during measurement if the curved plate directly touches the Earth pressure sensor, so a plastic acrylic separator (ϕ12 mm × 1 mm) was used to attach to the pressure sensor for ensuring stable and accurate readings. In addition, due to the area of the face between specimen and curved plate differing from the area of the face between the curved plate and Earth pressure, the value of LP is 0.283 times of the reading of Earth pressure sensor after calculation.

The loading system included pressure transducer, reaction frame, and cell base. After the apparatus setup, the reaction frame was fixed, and the vertical pressure can be loaded or unloaded with the cell base moving up and down.
The pressure transducer could measure the pressure ranged from 0 to 500 kPa with an accuracy of $\pm 1.5$ kPa, which is used to precisely control the vertical pressure. The cell base would stop until the vertical pressure reaches the target value, which is monitored and controlled by software interface. The air and water controlling system, respectively, used air pump and water pump to control the pore air pressure and pore water pressure; the air pressure and water pressure transducers were also included in the pumps. In addition, a bubble flushing cell was included in this system, which is used to get rid of the cumulative bubble during testing process.

The monitoring system included all the transducers and sensors that were connected to the software interface in computer through data logger. During testing, all the readings of these sensors are saved by software interface. Compared with the conventional oedometer, this apparatus was conducive to measuring the VSS and LP subjected to specific net normal stress and matric suction.

### 2.2. Calibration

In a previous study, the retractable ring with its attachments was applied successfully for measuring the lateral pressure of expansive soil after immersion [30], where the $K_0$ condition was verified to be achieved where the maximum lateral strain recorded was less than 0.04% under the highest lateral loading, according to ASTM D2435/D2435M-11 [40]. However, the apparatus was placed into the confining cell in this study, and the air pressure applied inside the cell acts on the pressure sensor that might affect the measuring system. So, the calibration for investigating the influence of air pressure upon the Earth pressure sensor was conducted, the water pressure remained constant as 40 kPa during the calibration, and thereby the maximum air pressure would be 240 kPa to achieve the target matric suction of 200 kPa. The results show that the reading of Earth pressure sensor remained zero when the maximum value of air pressure was applied during calibration, which indicated that the air pressure has no impact on the readings of the Earth pressure sensor when the air pressure applied is less than 240 kPa.

### 3. Material and Methodology

#### 3.1. Materials

The soil sample was obtained at a depth of about 5.0–6.0 m below the ground at K173 + 300 of LongBai (Longlin-Baise) Expressway in Guangxi Zhuang Autonomous Region, located in the southwest of China (23°48’11″N; 106°43’11″E), which belongs to the part of Baise Basin.

Atterberg limits, particle size distribution, and indices of swelling were determined in accordance with the Chinese Test Methods of Soils for Highway Engineering (JTG E40-2007) [41]. The clayey mineral of the soil sample mainly consists of I/S (illite/montmorillonite) mixed layer mineral with a high content of 45%. And this soil shows a highly physicochemical activity based on the content of montmorillonite (16.6%) and exchangeable cations (mainly Ca$^{2+}$ and Mg$^{2+}$). A summary of these soil characterizations is shown in Table 1; the swelling potential of this soil was determined to be medium according to the classification standard of expansive soil in highway engineering [42].

In addition, the Proctor compaction test using dry preparation method was also conducted, according to ASTM D1557-12e1 [43]. The Proctor compaction curve was shown in Figure 3.

As illustrated in Figure 3, the maximum dry unit weight of Baise expansive soil was 17.2 kN/m$^3$, and the optimum water content was 17.9%. The expansive soil is normally used as filler material for lower embankment, where the compaction degree requires more than 93%. Therefore, 93% of maximum dry unit weight condition and the optimum water content were chosen for specimen preparation.
3.2. Methodology. The literature review suggests that the clay particles after compaction would prefer to the direction perpendicular to the direction of compaction; it provides the possibility for the measurement of the VSS in original lateral direction and the LP in original vertical direction if the compacted specimen was rotated by 90 degrees. According to this consideration, two kinds of tests were performed on specimens with different specimen preparation method, namely, VS tests and LS tests. Specimens used for VS test series were prepared by conventional method, while the ones for LS test were prepared by modified specimen preparation method.

3.2.1. Specimen Preparation. Before specimen preparation, the soil samples from the field were firstly dried at the temperature of 105°C in oven, then pulverized, and screened to obtain clay powder with grain sizes ranging below 1 mm. Secondly, the clay powder was completely mixed with distilled water to reach the desired water content. To prevent loss of water and achieve uniformity of water content, the mixed soil was stored in sealed plastic bags and put in container for 72 h. At last, the mixed soil should screen again to make sure the gain size is below 1 mm before specimen preparation, since it is easy to be clustered when the expansive soil is mixed with water. On purpose of obtaining homogeneousness specimens, the specimen was sampled from the prespecimen as shown in Figure 4.

The prespecimen was prepared in a self-made specimen preparation box by static compaction with a compaction rate of 3 mm/min. All the prespecimens were compacted layer by layer with thickness of 10 mm in each. After static compaction, the prespecimen was demolded from the box, which was a cube with the side length of 70 mm. For VS test, the specimen was directly sampled from the prespecimen by an oedometer cutting ring (ϕ61.8 mm * 30 mm) along the perpendicular direction, where most of the microstructural particles orient to horizontal along the perpendicular direction. As for specimen in LS test, the cube was firstly rotated by 90 degrees, then sampled from the rotated prespecimen by an oedometer cutting ring along the perpendicular direction; at that time, most of the microstructural particles orient to vertical along the perpendicular direction. All the specimens in oedometer cutting rings were subsequently demolded.

To justify if the modified specimen preparation method could meet the demand for investigating anisotropic swelling behavior and if the particle orientation really existed in compacted Baise expansive soil, the SEM micrographs were, respectively, taken from the vertical direction (compaction direction) and horizontal direction (perpendicular to compaction direction) of the compacted Baise expansive soil as shown in Figure 5.

It indicated that the compaction truly results in the significantly preferred horizontal orientation on particles, which are the stepped face-to-face structures in Figure 5.

3.2.2. Swelling Tests. Before the setup of modified UC oedometer, several times of draining and fulfilling of water were firstly underwent by controlling the water pump to exhaust the air within the pipes. Then, a water pressure of max 50 kPa was applied under the HAVE disk. The HAEV disk began to be deaired and saturated; it can be considered as saturation until water was found to be appeared on the top surface of the HAEV disk. Closing the stop-flow valve would be helpful for accelerating the saturation process of the HAVE disk. After the saturation of HAVE disk, the demolded specimen was transferred into the retractable oedometric ring. Then, the adjustable joint was positioned by adjusting the screw to keep an initial reading of lateral pressure about 2 kPa~3 kPa to ensuring good contact. The retractable ring was then put into the modified UC oedometer, and the apparatus was setup as shown in Figure 1.

During the testing process, the stop-flow valve should keep open, and the water pressure should keep constant, which can be controlled by the software interface. After the vertical pressure (σz), air pressure (ua), and water pressure (uw) were both applied on specimen, the soil specimen was about to inundate under the circumstance of the given net normal stress (σz − uw) and matric suction (ua − uw). The required matric suction level was applied by decreasing the air pressure stepwise, which should enter the next level until the change of lateral pressure; vertical deformation readings collected at 300-second intervals were, respectively, less than 1 kPa, and 0.01 mm in a 4-hour time frame.

In addition, at each level, the applied vertical pressure should also decrease according to the reduction of air

Table 1: Physical and index properties of Baise expansive soil.

| Soil specific gravity | Atterberg limits (%) | Grain size (%) | Free swelling ratio (%) | Montmorillonite content (%) | Cation exchange capacity (meq·100g⁻¹) | Specific surface area (m²·g⁻¹) |
|----------------------|----------------------|----------------|-------------------------|-----------------------------|--------------------------------------|---------------------------------|
| 2.75                 | 56.3                 | 21.4           | 34.9                    | 52.1                        | 47.9                                 | 82                              |
|                      | Wp                   | Wr             | Ip                      | Silt                        | Clay                                 |                                 |

Figure 3: Proctor compaction curve of Baise expansive soil.
pressure to keep constant net normal stress. The specimen should always inundate with constant net normal stress, and the VSS and LP under certain net normal stress and matric suction can be recorded by the monitoring system in real time.

4. Test Program

The VS test and LS test were performed on compacted Baise expansive soil specimens in the modified UC oedometer. These specimens were all prepared at initial water content of 17.9% and dry unit weight of 16.0 kN/m³. The initial matric suction was about 500 kPa measured by filter paper method, according to ASTM D5298-16 [44]. Tests in this study were involved in ten tests series, where the ID represents the testing method and net normal stress; a summary of those test series is provided in Table 2.

Each test was conducted under the constant net normal stress and followed the same decreased matric suction path. For example, VS-50 referred to a VS test subjected to a net normal stress of 50 kPa. 200—100 referred to the matric suction that decreased from 200 kPa to 100 kPa. The specimen with initial matric suction would be compressed once the NNS was loaded on, because no swelling pressure would produce to fight back the external load when the increment of matric suction was zero, which would completely change

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**Figure 4:** Schematic view of specimen preparation. (a) VS test. (b) LS test.

**Figure 5:** SEM micrographs of the compacted Baise soil sample in (a) vertical direction and (b) horizontal direction.
its initial volume and dry unit weight. Therefore, the matric suction set for the test series all started from 200 kPa and then decreased to zero.

5. Result Analysis

The VSS varying with matric suction of these specimens under various NNS in real time were recorded through laboratory suction controlled swelling test, similar tendency was found in all test series, and the variation curves of VSS with matric suction of all test series in VS test and LS test are plotted in Figure 6.

As shown in Figure 6, in both VS test and LS test, the VSS gradually increased as matric suction decreased, and the growth of VSS became faster when the matric suction decreased from 50 kPa to 0 kPa; this phenomenon should be attributed to the unique microstructure of expansive soil. When the expansive soil specimen started to absorb water, the macropores within the soil were firstly filled with water that resulted in a light expansion like common clay. Meanwhile, the crystal layer in montmorillonite mineral also absorbed water, and the distance within the crystal layers would become further and further during this period, which promoted the swelling process. As for common clay, the light expansion would gradually remain stable when the macropores were filled with water. However, as for expansive soil, the swelling process within the crystal layer would continue to swell so that a huge increase of VSS was recorded in swelling process. Besides, the VSS decreased with the increase of NNS at each matric suction level, and the NNS would reduce the swelling deformation of expansive soil specimen.

The LP values in all test series were also recorded at each matric suction level under different NNS, and Figure 7 illustrates the development of LP versus s (matric suction) under various NNS. The transient readings of LP values were recorded once the NNS was loaded on, which are also demonstrated in Figure 7 as the LP values under the matric suction of 500 kPa.

From Figure 7, the LP of all test series increased with the decreased matric suction and tended to be stable with the matric suction approaching zero. It was interesting to notice that the development of LP with matric suction seems quite opposite from that of VSS; the LP experienced a fast augment when the matric suction was larger than 50 kPa. This kind of phenomenon could be explained as follows. The expansive soil specimen would firstly fight back the external load (e.g., NNS or lateral confinement) before the VSS occurred. However, the VSS would not occur until the produced vertical swelling pressure was greater than the loaded NNS; at that time, the LP has already increased to a considerable value that caused a great growth in LP when the matric suction decreased from 500 kPa to 50 kPa.

In addition, it also indicated that the LP increased with the increase of NNS at each matric suction level. As mentioned above, the LP of expansive soil consists of the lateral pressure caused by vertical stress which obeys the rules of elastic mechanics, and the lateral swelling pressure because of wetting. A recommended value of Poisson’s ratio is ranging from 0.2 (dry sand) to 0.5 (saturated clay tested under undrained conditions) [45], which shows a fact that Poisson’s ratio would increase with the decrease of matric suction. Therefore, the LP caused by vertical stress would also increase with the decreased matric suction. Besides, the lateral swelling pressure of expansive soil produced during this wetting process, which also increased with the vertical pressure [20, 30], so the double effect of lateral swelling pressure and the LP caused by vertical stress all led to the growth of LP with the increased NNS.

Furthermore, the LP recorded without NNS in VS-0 and LS-0 was 44.7 kPa and 77.0 kPa, respectively, which is the pure lateral swelling pressure. It indicated that some lateral swelling pressures would still produce even without any surcharge with lateral confinement. These results agree with the monitoring results of field investigations [46, 47]; the lateral swelling pressure monitored at 15~25 cm below the ground was ranging from 20 kPa to 50 kPa, where the field condition could be considered as a zero surcharge and lateral confined condition as well. Thus, the lateral swelling pressure near the ground surface cannot be neglected; however, it was usually considered as zero in a previous study.

6. Discussion and Findings

In this section, the influence of stress conditions including mean net stress and net stress ratio on volumetric strain (equals VSS) was investigated, and the possibility of the emergence of passive failure was justified according to the

| Type | Test ID | Dry unit weight (kN/m³) | Water content w₀ (%) | Net normal stress σz-w₀ (kPa) | Matric suction u₀-uₙₐ (kPa) |
|------|---------|-------------------------|----------------------|-----------------------------|-----------------------------|
| VS   | 0       | 0                       | 0                    | 0                           | 0                           |
| VS   | 12.5    | 12.5                    |                      | 200                        | 100                         |
| VS   | 25      | 25                      |                      | 100                        | 50                          |
| VS   | 50      | 50                      |                      | 100                        | 0                           |
| VS   | 100     | 100                     |                      | 100                        | 0                           |
| LS   | 0       | 0                       | 0                    | 0                           | 0                           |
| LS   | 12.5    | 12.5                    |                      | 200                        | 50                          |
| LS   | 25      | 25                      |                      | 100                        | 0                           |
| LS   | 50      | 50                      |                      | 100                        | 0                           |
| LS   | 100     | 100                     |                      | 100                        | 0                           |
Figure 6: Variation of VSS under different NNS with matric suction in (a) VS test and (b) LS test.

Figure 7: Variation of LP under different NNS with matric suction in (a) VS test and (b) LS test.
net stress ratio and passive Earth pressure coefficient. At last, the anisotropic swelling behavior of unsaturated expansive soil was characterized.

6.1. Effect of Mean Net Stress on VSS. The volumetric strain of expansive soils resulted mainly from the mean net stress applied on specimens [48]. In this section, the effect of mean net stress on volume change was investigated. The expansive soil specimen was setup in oedometer ring; hence the volumetric strain is equal to the VSS in this study, and the mean net stress can be computed using

\[ p = \frac{\sigma_v + 2\sigma_l}{3} \]  

where \( p \) is the mean net stress, kPa; \( \sigma_v \) is the vertical net stress, kPa, which is equaling to the NNS in this study; and \( \sigma_l \) is the lateral net stress, kPa, which is equal to the LP in this study. Figure 8 indicates the trends of VSS with mean net stress under each matric suction level.

From Figure 8, the VSS nonlinearly decreased with the increase of mean net stress. Comparing Figure 8(a) with Figure 8(b), it was observed from the slope of those curves that the influence of mean net stress on VSS was more significant in VS test series. Although the maximum VSS in VS test series was greater than that of LS test series, its maximum mean net stress was less. Once the initial dry unit weight and water content were chosen, the content of the montmorillonite in the expansive soil specimen was determined after compaction, and so did the swelling potential. During wetting, the swelling potential converted to swelling pressure if the deformation was not allowed, or swelling strain if the specimen was free to swell. The different NNS and boundary condition results in the different combination of swelling strain and swelling pressure in vertical or lateral directions. Because of the determined swelling potential, the expansive soil that produces larger VSS would experience smaller LP, so the specimen under the same NNS experienced less mean net stress.

As the underlay in pavement, the prediction for the heave of expansive soil is necessary and important. Based on the experimental data, the VSS (\( \epsilon_v \)), mean net stress (\( p \)), and matric suction (\( s \)) were normalized by

\[ \frac{\epsilon_v}{\epsilon_{vsat}} = a \left( \frac{p}{p_{atm}} \right)^\beta \left[ 1 - \left( \frac{s}{s_0} \right)^\theta \right], \]

where \( \epsilon_{vsat} \) is VSS at certain net normal stress and zero matric suction, \%; \( p_{atm} \) is standard atmospheric pressure, which equals 101.3 kPa; \( s_0 \) is initial matric suction, kPa; \( a, \beta, \) and \( \theta \) are fitting parameters. The fitting results are shown in Table 3; it shows good correlation with the measured data that the fitting correlation coefficients are over 0.97.

Figure 9 displays the comparison between the predicted VSS using (2) and the measured VSS from the VS and LS tests. It can be seen that the model prediction provides a good agreement with the test measurements. This indicates that the estimation method proposed in (2) is able to reflect the moisture-sensitive and stress-dependent behavior of expansive soil and can be used for heave estimation if the height of expansive soil layer was known.

6.2. Effect of Net Stress Ratio on VSS. Net stress ratio describes the boundary condition of the specimen experienced, which is the ratio of lateral pressure to vertical pressure and can be calculated by

\[ K_\theta = \frac{\sigma_l}{\sigma_v}, \]

where \( K_\theta \) is the net stress ratio; the other symbols in equation are mentioned above. Figure 10 shows the variation of VSS with net stress ratio (\( K_\theta \)) for specimens under different NNS and matric suction, with the exception of VS-0 and LS-0 test series.

From Figure 10, it is obvious that net stress ratio also influences the VVS. Observed from the inclination of these curves, it indicated that the influence of net stress ratio on VSS became stronger as the NNS increased. Moreover, an increase in net stress ratio resulted in a growth of VVS, which should be attributed to the increase in lateral restraint imposed on specimens; the majority of the swelling potential of the specimen would be released directly in the vertical direction. Furthermore, comparing Figure 10(a) with Figure 10(b), the influence of net stress ratio in VS test series was stronger than that of LS test series. For the same net stress ratio, the magnitude of VSS in VS test series was greater than that of LS test series; however, the specimens in LS test series would experience a much stronger lateral confinement since its maximum \( K_\theta \) was approaching 7.0.

Figure 11 demonstrates the development of stress state in VS test series upon wetting. In VS test series, the NNS remained constant while the LP increasingly grew due to wetting; the LP was minor principal stress when the matric suction was relatively higher, so the Mohr circle was still under the shear strength failure envelope at that time. However, as the matric suction continuously decreased, the lateral swelling pressure reached a significant value that resulted in a remarkable growth of the LP. Simultaneously, the major principal stress axis would rotate from vertical direction to lateral direction when the NNS was relatively low, where the LP became greater than the NNS and became the major principal stress. During this period, the diameter of the Mohr circle would keep increasing until it reaches the shear strength failure envelope, as shown in Figure 11. It also indicated that the lateral swelling pressure would advance the possibility of passive failure for structures founded near the ground surface with lateral confinement.

To justify if the specimen entered passive state, laboratory saturated shear strength tests were conducted to obtain the passive Earth pressure coefficient \( K_p \). The specimens were prepared at the same initial water content as 17.9%. The initial dry unit weights were selected ranging from 14.0 kN/m\(^3\) to 16.0 kN/m\(^3\), which included the full range of final dry unit weight of VS test series that is ranging from 14.7 kN/m\(^3\) to 15.8 kN/m\(^3\). The results are shown in Table 4.
Table 4 indicated that the internal friction angle of saturated specimen increased with the surcharge because of the increased dry unit weight. In Table 4, $K_p$ varied from 1.53 to 1.90, while the net stress ratios of specimen in VS test series under zero matric suction were 4.54 for VS-12.5, 2.58 for VS-25, 1.44 for VS-50, and 1.07 for VS-100, respectively. The value of net stress ratio in VS-12.5 and VS-25 was beyond the $K_p$ value. It indicated that the passive failure would truly occur for light structures with lateral confinement founded at shallow depths of 1.25 m if the soil unit weight was assumed as 20 kN/m$^3$.

6.3. Characterizing the Anisotropic Swelling Behavior. Although the variation of VSS and LP in VS test series and LS test series showed similar tendency, the magnitude of these values was obviously different. It was found that the magnitude of VSS in VS test series was greater than that of LS test series, while the magnitude of LP in VS test series was less than that of LS test series. In a previous study, the swelling pressure ratio (SR) was used to describe the swelling anisotropy, which is the ratio of pressures measured in lateral direction to the ones in vertical direction after inundation [9, 49]. The calculated values of SR in some cases were over 1.0 when the vertical equivalents are relatively low. However, the swelling ability in the direction that is normal to the particle orientation is greater than that in the parallel direction [5]. Therefore, it is confused to characterize the anisotropic swelling behavior by SR value.

In this section, considering that both the LP in VS test series and LS test series were measured under the same NNS and matric suction, the anisotropic swelling behavior of unsaturated expansive soil was characterized using the anisotropic swelling ratio (ASR) as

$$\text{ASR} = \frac{\text{LP}_v}{\text{LP}_l}$$

where $\text{LP}_v$ and $\text{LP}_l$ are the LP in VS test series and LS test series, respectively. Figure 12 shows the variation of ASR with matric suction with respect to the different NNS.

It was observed that the ASR was fluctuant as matric suction decreased, which indicated that the stress redistribution continuously occurred during wetting. The condition of zero NNS is special where the measured LP values are exactly the values of pure lateral swelling pressure. The ASR with respect to the NNS of 0 kPa exhibited an obvious variation that the ASR increased from 0.36 to 0.64 as the matric suction decreased from 200 kPa to 0 kPa. It might be suggested that the

![Figure 8: Variation of VSS under different matric suction with mean net stress in (a) VS test and (b) LS test.](image)

![Figure 9: Comparison between the predicted VSS using (2) and the measured VSS from laboratory test.](image)
anisotropic swelling behavior would reduce but still remain as the expansive soil gradually absorbs water from the environment.

Moreover, the NNS also had influence on ASR that the variation of ASR became smaller with the increase of NNS. The ASR varies from 0.36 to 0.81 although no obvious relationship was found among the ASR, NNS, and matric suction. Therefore, it is improper to easily multiple an empirical coefficient to the vertical swelling pressure to estimate the lateral swelling pressure. The anisotropic swelling behavior should be taken into consideration when designing geo-infrastructure that was mainly suffered from the external loads from lateral direction.

![Variation of VSS under different matric suction with net stress ratio $K_0$ in (a) VS test and (b) LS test.](image)

**Figure 10:** Variation of VSS under different matric suction with net stress ratio $K_0$ in (a) VS test and (b) LS test.

![Development of stress state upon wetting.](image)

**Figure 11:** Development of stress state upon wetting.

| Initial dry unit weight (kN/m³) | Initial water content $w_0$ (%) | Internal friction angle $\varphi$ (°) | Cohesive force $c$ (kPa) | Passive Earth pressure coefficient $K_p$ |
|-------------------------------|-------------------------------|-----------------------------|------------------------|--------------------------------------|
| 14.0                          | 12.1                          | 0.1                         |                        | 1.53                                 |
| 14.5                          | 13.6                          | 0.6                         |                        | 1.62                                 |
| 15.0                          | 17.9                          | 1.4                         |                        | 1.70                                 |
| 15.5                          | 16.2                          | 2.8                         |                        | 1.77                                 |
| 16.0                          | 18.1                          | 3.4                         |                        | 1.90                                 |

**Table 4:** Results of laboratory shear strength tests.
experience a much stronger lateral confinement with
NNS = 100kPa
NNS = 50kPa
NNS = 25kPa
NNS = 12.5kPa
NNS = 0kPa
Mean value

Figure 12: Variation of anisotropic swelling ratio with matric
suction.

7. Conclusions

An experimental study was conducted for investigating the
anisotropic swelling behavior of expansive soil by using an
unsaturated consolidation oedometer, where the specimens
were, respectively, sampled from the compacted cubic
prespecimens from vertical and lateral directions, consid-
ering the preferred orientation of soil particles after
compaction.

Similar tendency was found in the variations of VSS and
LP of expansive soil specimens subjected to different NNS
and matric suction in VS and LS test series. The VSS
gradually increased with the reduction of matric suction; this
augment became faster when the matric suction decreased
from 50 kPa to 0 kPa. Meanwhile, the LP also increased with
the decreased matric suction. However, opposite from the
development of VSS, the LP experienced a fast augment
when the matric suction was larger than 50 kPa. This phe-
omenon should be attributed to the unique microstructure
of expansive soil and the boundary condition.

The VSS nonlinearly decreased with the increase of mean
net stress for the same matric suction; the influence of mean
net stress on VSS was more significant in VS test series.
Although the maximum VSS in VS test series was greater
than that of LS test series, its maximum mean net stress was
less. Because of the determined swelling potential, the ex-
pansive soil that produces larger VSS would experience
smaller LP, so the specimen under the same NNS experi-
enced less mean net stress. An estimation method consid-
ering the effect of mean net stress and matric suction for
predicting the heave of expansive soil was proposed, the
model prediction provides a good agreement with the test
measurements.

The net stress ratio ($K_0$) also had impact on the VVS, and
an increase in net stress ratio resulted in a growth of VVS,
which should be attributed to the increase in lateral restraint
imposed on specimens. For the same net stress ratio, the
magnitude of VSS in VS test series was greater than that of
LS test series; however, the specimens in LS test series would
experience a much stronger lateral confinement with
maximum net stress ratio of 7.0. During the test, the NNS
remained constant, while the LP increasingly grew because
of wetting; hence, the diameter of the Mohr circle would
keep increasing until it touches the shear strength failure
envelope. Associated with calculated passive Earth pressure
coefficient $K_p$ based on shear strength test, the lateral
swelling pressure advances the possibility of field passive
failure for light structures founded at shallow depths with
fully lateral confinement.

The magnitude of VSS and LP values was quite different
between the VS test series and LS test series. Anisotropic
swelling ratio (ASR) was defined based on the test results of
VS and LS test series. It was observed that the ratio was
fluctuant as matric suction decreased, and the NNS had
influence on the swelling anisotropy, where the ASR varied
from 0.36 to 0.81. It is evident that the ASR under zero NNS
increased from 0.36 to 0.64 as the matric suction decreased,
which suggested that the anisotropic swelling behavior
would reduce but still remain as the expansive soil gradually
absorb water from the environment. Therefore, it is im-
proper to easily multiple an empirical coefficient to the
vertical swelling pressure to estimate the lateral swelling
pressure.

Data Availability

All the data presented and analyzed in the manuscript were
obtained from laboratory tests at Changsha University of
Science and Technology in Changsha, China. All the lab-
atory testing data were presented in the figures and tables in
the manuscript. The authors will be very pleased to share all
the raw data. The data included in this manuscript are
available upon request from the corresponding author.

Conflicts of Interest

On behalf of all authors, the authors state that there are no
conflicts of interest.

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