Coupled Water Content Method for Shrink and Swell Predictions

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ABSTRACT

Expansive soils pose great challenges for the design of foundations. One of the most difficult issues for designing pavements and foundations on expansive soils is to predict the volume change of the soils. In this study, current methods for movement predictions are summarized and their relationships and shortcomings are discussed. It is found that all these methods have the same theoretical bases. Based on the theory of unsaturated soil mechanics, it is found that the water content and the mechanical stress can be used to determine the soil status. Hence, a void ratio versus mechanical stress and water content surface is constructed which coupled both mechanical stress and suction’s influences on volume change of expansive soils. The new surface is developed into a coupled water content method which can be used to predict the potential vertical swell and the potential vertical shrink simultaneously, while all the existing methods can only predict the potential vertical rise (PVR). The method is used to analyze data collected from a construction site at Arlington, TX. The predicted movements match the measured data reasonably well. The method is simple and overcomes the shortcomings existing in the current movement prediction methods for expansive soils.

Key words: shrink-swell soils, movements, unsaturated soils, constitutive surface, water content, suction, mechanical stress, volume change, potential vertical rise.
INTRODUCTION

Expansive soils often cause difficult performance problems for buildings constructed on these soils. Statistical data indicate that each year huge loss has been caused by expansive soils (1-4). Prediction of the vertical movements of the ground surface is required for the sake of foundation design. Currently there are many methods to predict the movements of expansive soils (5-16). Research indicates these methods have common theoretic basis. However, suction-based methods and one dimensional consolidation based methods are not easy to implement in practice. In addition, most methods focus on the calculation of potential vertical rise, which is corresponding to a short term condition. The potential vertical shrink, which is corresponding to a long term condition, is greatly neglected. In this paper, the relationships between different methods and possible shortcomings for these methods are presented. A water content based method is used to predict the movements of four footings at Arlington, Texas and the results match the measured filed measurements reasonably well (5). In this paper, the method is modified to predict both potential vertical rise and potential vertical shrink simultaneously and the shortcomings in the existing methods are eliminated.

CURRENT METHODS FOR PREDICTING THE MOVEMENTS OF EXPANSIVE SOILS

Any method to predict the movements of expansive soils must include two components (5): 1. the continuity equation for the moisture flow to predict the moisture variation, 2. the constitutive law to predict the volume change of the soils due to the moisture variation. Currently there are a number of methods for predicting the volume change of expansive soils. According to the state variables used in their constitutive laws, they can be classified into three categories: 1. Suction based methods which use the matric suction as a stress state variable, 2. Water content based methods which use water content as a state variable, and 3. Consolidation test based methods which use the equivalent effective stress as a stress state variables. A brief description of these three methods is given as follows:

Suction Based Method

Many researchers use suction as a stress state variable to predict the volume change of expansive soils (6-11). The constitutive law of the suction based method is that the volume change of the unsaturated soil due to moisture variation is linearly proportional to the suction variation in log scale, i.e.

\[ \frac{\gamma_h}{1+e_0} \Delta \log(u_u - u_w) \]

Where \( \gamma_h \) is the matric suction compression index, equal to the slope of the volumetric strain versus the matric suction in log scale, \( u_u - u_w \) is the matric suction, e is the void ratio, \( e_0 \) is the initial void ratio, and \( \Delta \) stands for the variation of variables.

McKeen (9) proposes that the range over which the matric suction compression index is applicable is between 1.5 and 4.5 in log kPa scale for all the soils. Fig. 1a illustrates the void ratio versus suction curve for a sample from a depth of 1.2m to 1.8m at a site in Arlington, Texas.

Water Content Based Method

The constitutive law of the water content based methods is proposed by Briaud et al.(5), indicating that there is a linear relationship between the volume change of unsaturated soil and the water content variation:

\[ \frac{\Delta V}{V} = \frac{\Delta w}{E_w} \]

Where \( E_w \) is the slope of water content versus volumetric strain curve, and \( w \) is the water content.

Fig.1b shows the relationship between the water content \( w \) and the void ratio curve for the same soil in Fig.1a. The solid line shows the free shrink test curve, where there is an approximate linear relationship over the range between the swell limit and the shrinkage limit and the dotted line is the 100% degree of saturation curve. It can be seen the most of the volume change of the soil occurs when the degree of saturation is still high, and that shrinkage limit is 13%.

The PVR method (12), Vijayvergiya-Ghazzaly method (13) and shrink test method (5) are typical water content based methods even though the constitutive laws are not expressed as Eq.2 explicitly. The water content methods are
considered as empirical methods because none of the current water content methods gives out a continuity equation to predict the water content variation.

One Dimensional Consolidation Test Based Method

Some researchers (14-16) use the one dimensional consolidation test to predict the volume change due to suction. The core concept is to find the “equivalent effective stress” variation due to suction variation, and use the one-dimensional consolidation compression index to calculate the volume change due to suction. The constitutive law for one dimensional based method is as follows:

\[ \frac{\Delta V}{V} = \frac{C_s}{1+e_0} \Delta \log_{10} \sigma' \]  

Where \( C_s \) is the compression index and \( \sigma' \) is very similar to the Bishop equation, that is,

\[ \sigma' = \gamma \rho_s u_w \]  

Where \( \gamma \) is a coefficient related to the degree of saturation and is a variable.

The typical consolidation test based methods are double consolidation method (14), Salas and Serratosa method (15), and Fredlund et al method (16). In Fredlund et al’s method the equivalent effective stress is obtained by projecting the current soil status on the void ratio constitutive surface to the zero suction plane and using the rebound branch as constitutive law for the volume change calculation. A complicated correction procedure for the swell pressure is needed to get the initial equivalent effective stress.

Relationships between Different Methods

Fredlund and Morganstern (17) propose the constitutive relations for volume change in unsaturated soils by using the two stress state variables as follows:

\[ d \varepsilon_i = m_1' d (\sigma_m - u_a) + m_2' d (u_a - u_w) \]  

\[ d \theta = m_1'' d (\sigma_m - u_a) + m_2'' d (u_a - u_w) \]  

Where \( m_1' = \) coefficient of total volume change with respect to mechanical stress; \( m_1'' = \) coefficient of pore-water volume change with respect to changes in mechanical stress; \( m_2' = \) coefficient of total volume change with respect to changes in matric suction; \( m_2'' = \) coefficient of pore-water volume change with respect to changes in matric suction; and \( \sigma_m = \) the mean mechanical stress.

Zhang(18) re-explains the physical meanings of these parameters as follows: \( m_1' \) is the inverse of bulk modulus of the soil, which represents the volume change of soil for a given mechanical stress variation; \( m_1'' \) is a triple of the coefficient of expansion due to matric suction variation; \( m_2' \) represents the ability of mechanical stress to squeeze water out of the soil; \( m_2'' \) is related to the slope of soil-water characteristic curve which indicates the matric suction variation for a given amount of water adding to the soil. From Equations 5a and 5b, it is found that the suction based methods and the water content methods are not independent. They are related by the soil water characteristic curve. The soil water characteristic curve gives out the relationship between water content and matric suction. If one neglects the influence of hydraulic hysteresis, there is a unique relationship between the water content and the matric suction. Fig.1c shows the soil water characteristic curve for the same sample as discussed above. Usually it is assumed that there is a linear relationship between the water content and the log of the matric suction in the range of interest:

\[ (u_a - u_w) = g(w) \]  

or

\[ w = C_w \log(u_a - u_w) + d \]  

Where under this assumption \( m_2'' = \rho_s g \frac{C_w}{(u_a - u_w)} \)

Comparing Equations 1, 2 and 5a, the following relationship is obtained:

\[ \frac{\Delta V}{V} = \frac{\Delta \varepsilon_i}{1+e_0} = \gamma_n \Delta \log(u_a - u_w) = \gamma_n \frac{\Delta w}{C_w} = \Delta w \frac{\Delta w}{E_w} = \frac{\Delta w}{(1+e_0)(u_a - u_w)m_2'} \]  

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Comparing Equations 3 and 5a, the following relationship is obtained:

\[
\chi = \frac{m_s^*}{m_i^*}
\]  

(8)

**Prediction of Suction or Water Content Variations**

As mentioned before, prediction of moisture or matric suction variations is needed for the implementation of all the methods. The continuity equation for the suction based method is Richard’s Equation:

\[
\frac{\partial}{\partial x} \left[ k \left( \frac{\partial (u_s - u_w)}{\partial x} \right) \right] + \frac{\partial}{\partial y} \left[ k \left( \frac{\partial (u_s - u_w)}{\partial y} + 1 \right) \right] = m_s^* \frac{\partial (u_s - u_w)}{\partial t}
\]

(9a)

Where \( k \) is the coefficient of hydraulic conductivity for unsaturated soils, which is a function of matric suction.

Usually water content based methods are considered as empirical methods. No differential equation has been proposed for the calculation of water content variations in geotechnical engineering. By substituting equation (6a) into (9a), we can derive the differential equation of moisture movements in terms of water content,

\[
\frac{\partial}{\partial x} \left[ k (w) g'(w) \frac{dw}{dx} \right] + \frac{\partial}{\partial y} \left[ k (w) g'(w) \frac{dw}{dy} + 1 \right] = \rho_d \frac{dw}{dt}
\]

(9b)

Where \( g'(w) = \frac{du}{dw} \). Both Equations (9a) and (9b) can be considered as forms of Richard’s equation (19). Equation (9b) can be used to predict the water content variations by combining the site-specific boundary conditions. It has been used in soil physics (21) but rarely used in geotechnical engineering.

For one dimensional consolidation test based method, prediction of matric suction is also needed. Equation 9a can be used for this purpose. However, most current methods predict the matric suction variations by assuming the soil will change from current conditions to complete saturation (20). In this way the potential vertical rise (PVR) or total heave can be calculated.

**Problems Associated with Different Methods in Practice**

Equations 7a and 7b present the relationship between suction based methods and water content based methods when the mechanical stress is zero. It indicates that both methods have the same theoretical basis. They can be interchanged with each other through the soil-water characteristic curve when numerical simulation is performed. One dimensional consolidation test based method includes the influence of mechanical stress as shown in Equation 8. All these methods can be represented by Equation 5a. As a result, if used properly, the same result can be obtained by all the three methods. However, when used in practice, the complexity and accuracy for different methods are different. When there is mechanical stress variation, the relationship is similar but with greater complexity.

There are several potential problems associated with the suction based methods in current practice. First, the relationship between the volumetric strain \( \Delta V/V \) and the matric suction in log scale \( \Delta \log(u_s - u_w) \) is linear only over a certain suction range and this range is not known in most cases. A common range (from 32 kPa (2.5 pF) to 31620 kPa (5.5 pF)) is proposed by Mckeen (10) for all soils. A fixed range over which to use the suction compression index \( \gamma_h \) is welcome by many geotechnical engineers due to the time-consuming process of obtaining the void ratio versus suction curve. However, it is a questionable assumption. Observations indicate that the range of the linear relationship is varying for different soils. For example, in Fig. 1a there is a linear relationship between the void ratio and the suction variation in the range from 1000 kPa (4pF) to 20000 kPa (5.3 pF). If the range from 32 kPa (2.5 pF) to 31620 kPa (5.5 pF) is used to determine the matrix suction compression index \( \gamma_h \), the matrix suction compression index \( \gamma_h \) will be underestimated 400%. To use the suction method reasonably, the range for the linear relationship between the void ratio and the matric suction in log scale should be identified even though more efforts are needed for the lab testing. Second, the suction compression index \( \gamma_h \) is also influenced by the mechanical stress as illustrated in Fig.2a, which shows that void ratio constitutive surface for the same soil. Fig.2a indicates that the
suction compression index is also a function of mechanical stress. Third, current matric suction has to be known when one uses the suction based method. In practice, it is not easy to accurately measure field matric suction. Measuring the matric suction for a soil sample usually requires sampling soil from the field. During this process, the matric suction in the soil will change dramatically due to stress release. To obtain accurate suction measurements, field suction tests are required, which makes the method difficult to apply.

To use one-dimensional consolidation based methods, both the current matric suction and the corresponding $\chi$ value must be known. Therefore, how to accurately measure the field suction is also a problem. In addition, it is extremely difficult to determine the $\chi$ value in Bishop’s equation. Research indicates that $\chi$ is a function of both mechanical stress and suction and varies greatly (24). The difficulties make the one-dimensional consolidation based method very difficult to implement. As a result, the one-dimensional consolidation based methods are not used very frequently in practice.

For the water content method, there are also limits for the applicable range of the constitutive law. The upper limit is the swell limit and the lower limit is the shrinkage limit. Some water content methods include the influence of mechanical stress by empirically adjusting the calculation result with soil dry density; they can be considered as coupled mechanical stress and water content method. The PVR method (12) gives the upper limit $0.4LL+9$ (swell limit) and the lower limit $0.2LL+7$ (shrinkage limit) and adjusts the potential vertical rise with the soil density and the applied mechanical stress. However, the PVR method only gives out the potential vertical swell of the soil from the current soil moisture without considering the potential vertical shrink, or the potential vertical amplitude of movements, which will be discussed later. Different from suction compression index, shrink-swell index $E_s$ for water content method is not influenced by mechanical stress (5). Some researchers also find that the gravimetric water content at the shrinkage limit is the same for different mechanical stress levels (18). The findings indicate that water content based methods have some advantages over suction-based method. However, the water content method proposed by Briaud et al (5) is actually an uncoupled analysis of unsaturated soils where only the influence of moisture variation on the volume change of expansive soils is considered. The objective of this paper is to develop a simple empirical coupled water content based method which can overcome the shortcomings in current methods as discussed above.

**A COUPLED WATER CONTENT METHOD**

All the above methods are based on the same assumption that the soil is elastic and the same assumption is used in this paper. For shallow foundations on expansive soils, the assumption that expansive soils are elastic is considered as a reasonable assumption in that a drying process can cause an expansive soil to yield (25). Although under nearly all conditions, the stress history for an expansive soil is unknown, it is reasonable to assume that the soil has experienced the maximum dryness in the past. Consequently, if an expansive soil has some plasticity, this kind of plasticity must have been eliminated by a long history of wetting-drying cycles. This may be the reason why most expansive soils are usually heavily over-consolidated. It is well known that the volume of the expansive soil is influenced by the mechanical stress and one may argue that the soil will yield under a combination of mechanical stress and matric suction variations. However, for pavements and light residential or commercial buildings where the methods are used, the mechanical stress due to the superstructure is so small that its influence on soil yielding can be neglected. As a result, for engineering purpose, expansive soils can be considered as elastic.

The constitutive surfaces express the soil state variables such as void ratio, water content and degree of saturation of the soils as a function of the two independent stress state variables, that is, the matric suction and the mechanical stress. If the soil is elastic, the constitutive surfaces will be unique and all the parameters in Equations 6a and 6b can be obtained. Figs 2a, 2b and 2c show the void ratio, degree of saturation, and water content constitutive surfaces for a sample from a depth of 1.2m to 1.8m at a site in Arlington, Texas.

With these three constitutive surfaces, all the soil state variables can be determined if any two of the five state variables, i.e., void ratio, water content, degree of saturation, net normal stress and matric suction are known. An interesting combination is made when the two known state variables are water content and the mechanical stress. In this case, the water content constitutive surface in Fig.2c can be used to get the corresponding matric suction value of the soil, and then the void ratio constitutive surface and degree of saturation surface (Figs.2a and 2b) can be used to get the void ratio and the degree of saturation of the soil, respectively. Also, if the future soil status (the water content and the mechanical stress) of the soil is known, the volume change of the soil can be calculated. A routine boring log usually gives out the soil layer distribution with depth and the water content profile. The net normal stress can be calculated for soils at different depths $z$ by using the soil unit weight $\gamma$, i.e. $\sigma = \gamma z$. Combining this net normal mechanical stress profile with the water content profile can give the status including the void ratio profile at
any depth provided that the three constitutive surfaces are known. If the future soil water content profile and mechanical stress are known, the volume change of the soil can be predicted, which will be discussed later.

A void ratio versus total mechanical stress and water content surface can be obtained by using the above procedure as shown in Fig.3. It is noted that the mechanical stress axis is in logarithmic scale. To avoid the mathematical problem associated with the origin of a log scale for mechanical stress, the mechanical stress for a free shrink test is assumed to be 1 kPa. Curve ABD is the void ratio versus water content curve for the free shrink test and is the same as what is shown is Fig. 1b. Segment AB represents the volume change when the soil is drying from the swell limit (which is determined by a free swell test) to the shrinkage limit. Segment BD represents the constant void ratio after the soil reaches the shrinkage limit. Form B to D, the soil water content will continue to decrease but no volume change occurs.

Curve AFC is the curve corresponding to the one dimensional consolidation test on a fully saturated sample in the void ratio, mechanical stress and water content space; the degree of saturation is S=100% and the matric suction is \( u_a - u_w = 0 \) kPa on this line. Curve NFE represents the void ratio versus mechanical stress in log scale from the one dimensional consolidation test, which is the projection of Curve AFC on the void ratio versus mechanical stress plane. For the consolidation test, soil is considered to be completely saturated, that is, S=100%, so the corresponding water content for different void ratio can be calculated by \( w = S \varepsilon / G_r = e / G_r \). The projection of Curve AFC on the mechanical stress and water content plane is Curve AFC.

Surface ABCA represents the range where most of the volume change occurs and surface BCEDB represents the range where there is no volume change, i.e. the zone with water content below the shrinkage limits for different total mechanical stress level.

In surface ABCA, when the total mechanical stress increases the swell limits of the soil decreases. As shown in Fig.3, Curve FGH represents the shrink test curve when the total mechanical stress is 10 kPa. The water content variation range (FG) for mean mechanical stress (\( \sigma_m - u_a \))=10kPa is smaller than when the mean mechanical stress is 1 kPa (AB in Fig.3); so is the void ratio variation range. Fig.3 also shows that the slopes of the void ratio versus water content curve remain approximately constant. This is in agreement with Briau et al. (5) who performed shrink tests with different loading stresses and found that the swell-shrink modulus \( E_w \) is not influenced by applied external load. Therefore, the mechanical stress only changes the range of water content variations, i.e. the difference between the swell limits and the shrinkage limits, and the range of void ratio variation. The swell limits at different mechanical stress levels are determined by the one-dimensional consolidation test and the shrinkage limit is determined by free shrink test. With these two tests, Fig. 3 can be obtained. With Fig.3, the void ratio for any soil sample can be determined by its water content and mechanical stress. The mechanical stress is related to the soil profile and can be determined by its depth below the ground surface. If the future water content is known, the corresponding volume change of the soil can be calculated. The application can be described as follows:

1. From the boring log plot, obtain the water content profile versus depth, 
2. Calculate the corresponding total mean mechanical stress for each depth, 
3. Plot the profile on the void ratio versus water content and mechanical stress surface to get the corresponding void ratio distribution, and 
4. Calculate the maximum potential vertical swell and the potential vertical shrink; and the sum is the maximum range of the vertical movements.

Fig.3 shows that Point O can be determined once the water content and the mechanical stress are known. The potential void ratio increase (potential vertical swell) is Curve OF and the potential void ratio decrease (potential vertical shrink) is Curve OG. The total void ratio variation range is Curve GOF. This method considers the influence of both mechanical stress and matric suction and can be considered as a coupled water content method.
An Example of the Proposed Method

A site in Arlington, Texas was selected to show the application of the proposed method. The predominant soil type at the site is classified as borderline between CL and CH according to the Unified Soil Classification System. The soil stratigraphy, the average soil properties and the parameters for each soil layers are shown on Fig.4. A total of 63 dry borings were performed at the site over a period of two years; 61 borings were done to a depth of 3m and 2 to a depth of 7m. Each boring consisted of pushing Shelby tubes continuously without any drilling and without adding water or drilling mud. The water level in the 7m deep standpipes varied between 4m and 4.8m below the ground surface over a period of 2 years. Fig.5 shows all the soil water content profiles obtained over a two year period between June 24, 1999 and November 2001 for four footings RF1, RF2, W1 and W2. Briaud et al.\(^{(5)}\) used the measured water content profiles to predict the movements of the four footings. The results reasonably matched the measured movements over a period of two years. The same results can be obtained by the method proposed in this paper. This paper focuses on the ability of the new method to predict the potential vertical rise and potential vertical shrinkage.

Fig.6 shows the void ratio profile W1-1 and W1-7of w1 footing at boring 1 (06/24/1999) and boring 7 (11/17/2000) and the maximum water content range for the site as shown in Fig.5. The two thick solid lines are the maximum and the minimum envelopes of the void ratio variations, which are corresponding to curve AC and BC in Fig.3, respectively. The change in the maximum and the minimum envelopes of the void ratio variations is due to the change in the soil categories. The vertical volume change can be calculated by the following equation:

\[
\Delta H = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} \frac{\Delta e_i}{1+e_i} H_i
\]

Where \(H_i\) is the thickness of layer \(i\), and \(\Delta H\) is the potential vertical movement.

The maximum possible vertical movement can be obtained by computing the area between two solid lines in Fig.6, for this example the value is 0.9623m. The potential vertical shrink for footing W1 can be obtained by computing the area between current W1-1 void ratio profile and the left-sided solid line corresponding to the shrinkage limits, the value is 0.4058m and the potential vertical swell is 0.5564m, which is the area between current void ratio profile and the right-sided solid line corresponding to the swell limits. In the same way, the potential vertical shrink and the potential vertical swell for W1-7 can be obtained, which are 0.3512m and 0.6111m respectively. The total vertical movement for W1-7 is the same as W1-1 for the same boring at different time.

The relative movement for Boring W1 between the W1-1 (06/24/1999) and the W1-7 (11/17/2000) period can be calculated by comparing their void ratio profiles and the movement is -0.0453m (shrink), which is much smaller than the potential vertical movements. The reason will be explained in the next section.

Discussion

Potential vertical swell, potential vertical shrink and total vertical movements of expansive soils

Slab will distort into either an enter lift (Fig. 7a) or an edge lift mode (Fig.7b) when the beneath expansive soil experiences a change in its moisture content \((23)\). Center lift is a long term condition which occurs when the moisture content of the soil around the perimeter of the slab decreases and the soil shrinks relative to the soil beneath the interior of the slab. Conversely, edge lift is a seasonal or short term condition and occurs when the soil beneath the perimeter becomes wetter than the soil beneath the interior of the slab. The differential movement is needed for design. To design a structure on expansive soils, both damage modes in Fig. 7 should be considered. For the center lift case, the potential vertical shrink of the soil needs to be estimated, and for the edge lift case the potential vertical swell should be considered.

The PVR method can only predict the potential vertical rise at the current soil moisture condition. The PVR values are calculated by comparing the current soil moisture with the possible maximum water content in the future. Consequently, the calculated results greatly depend on timing of the sampling. Fig. 8 shows the calculated PVR values for the Arlington site over the two years’ period (06/24/1999-10/13/2001). As can be seen, the PVR value is dependent of the current soil moisture status and therefore the timing when the soil is sampled. When the samples are obtained at different time of the year, the soil water content is different, and so is the calculated PVR value. It is obvious that the PVR is not a good value for design purposes or at least misleading. The method proposed by Fredlund et al. \((16)\) has the same problem because it also compares the current moisture status with the condition when the soil is saturated. From Fig.3, both the potential swell and the potential shrink at the present time can be
obtained. The potential vertical swell plus potential vertical shrink should be used as a guide for design, which is more reasonable than the PVR value only.

Climate controlled movements vs. potential vertical movements

As we discussed before, the volume change for footing W1 between the W1-1 and W1-7 is much smaller than the potential vertical movement of the soil. The is due to the fact that calculations for the potential vertical movements are performed for the extreme conditions, that is, the soil water content either increases from current status to swell limit, or decreases to the shrinkage limit throughout the whole soil profile. Under real conditions, the water content variations are much smaller than the extreme conditions, which can be seen in Fig.6. At the same time, because the climate varies seasonally and the permeability coefficients of expansive soils are usually very low, the water content in the soil will not increase or decrease unanimously. Instead, soil water content will increase at some depths while decrease at some other depths. Comparing the void ratio profile W1-1 with W1-7, one can find that soil swells (void ratio increases) near ground surface and shrinks below the depth of 0.8m. These two inverse volume changes cause the actual induced vertical movements far less than the predicted potential vertical movements. Fig.9 shows the observations of the movements for the four footings caused by the climate variations over a period of two years. The actual maximum vertical movement is 50+30=80mm, much less than the maximum potential vertical movement. In this way, a design based on climate-controlled vertical movements and that based on extreme conditions will be greatly different. Further research is needed in this direction. Numerical simulation will be able to give a better understanding for the problem. Comparing the calculated PVR values in Fig.8 with the observed vertical movements range in Fig.9, it is found that the PVR method underestimates approximately 2/3 of the range of the vertical movements, which may be attributed to the fact that the PVR method assumes the volume change of the soil is the same in all directions while the actual vertical movements mainly occur in the vertical direction due to the lateral restraint.

CONCLUSION

Current methods for predicting the movements of expansive soils have been reviewed. The differences and their relationships are discussed. The potential problems associated with their applications in current practice are investigated. The volume change behaviors of expansive soils are explained by using the constructed constitutive surfaces. Research indicates the soil status of a soil sample can be determined uniquely by its depth and water content, which leads to a coupled water content method. By using the new method, both the potential vertical rise and the potential vertical shrink can be obtained.

The criteria for design are rechecked and research indicates that current practice by using the PVR method as design criteria is misleading. It is the total vertical movement, which is the sum of potential vertical rise and potential vertical shrink, rather than the PVR that determines the possible damage to house. Research also indicates that the predicted vertical movements by using extreme conditions as limit conditions are much bigger than that measured under the climate-controlled conditions. More accurate and simple method to predict the suction or water content profile is needed to determine the differential movements. Numerical methods may be the way to achieve it.

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REFERENCES


LIST OF FIGURES

FIGURE 1 Experimental results for a sample from a depth of 1.2m to 1.8m at Arlington, Texas.
FIGURE 2 Constitutive surfaces for a soil for a sample from a depth of 1.2m to 1.8m at Arlington, Texas.
FIGURE 3 Constructed void ratio versus mechanical stress and water content Surface.
FIGURE 4 Soil stratigraphy at the site.
FIGURE 5 Water content profiles of the site over a period of two years.
FIGURE 6 Void ratio profiles of the site at different conditions.
FIGURE 7 Two typical damage modes caused by expansive soil.
FIGURE 8 Calculated PVR values for the site over a period of two years.
FIGURE 9 Measured footing movements over two years.
\[ y = -0.0427 \ln(x) + 0.8195 \]
\[ 1 \leq \log(u_a - u_w) < 3.5 \]

\[ e = -0.2662 \log(u_a - u_w) + 1.5969 \]
\[ 3 \leq \log(u_a - u_w) < 4.3 \]

\[ e = -0.0531 \log(u_a - u_w) + 0.8559 \]
\[ 1.5 \leq \log(u_a - u_w) < 3.5 \]

\[ e = -0.0851 \log(u_a - u_w) + 0.8622 \]
\[ 1.5 \leq \log(u_a - u_w) < 4.0 \]

**void ratio vs. matric suction curve**

**void ratio vs. water content curve**
FIGURE 1 Experimental results for a sample from a depth of 1.2m to 1.8m at Arlington, Texas.
FIGURE 2 Constitutive surfaces for a soil for a sample from a depth of 1.2m to 1.8m at Arlington, Texas.
FIGURE 3 Constructed void ratio versus mechanical stress and water content Surface.
FIGURE 4 Soil Stratigraphy at the Site

- Dark Gray Silty Clay: Trace Fine Sand
  - $\sigma_u = 151.5$ kPa
  - $\gamma = 20.3$ kN/m$^3$
  - $w_{mean} = 20.73\%$
  - $E_w = 0.752$, $f = 0.39$
  - $u = 3.42$ pF
  - $%SW = 5.17$
  - LL = 51.3, PL = 22.3
  - $%<0.002 = 47.7$

- Brown Silty Clay, trace fine Sand: Calcareous
  - $\sigma_u = 179.8$ kPa
  - $\gamma = 20.4$ kN/m$^3$
  - $w_{mean} = 19.74\%$
  - $E_w = 0.869$, $f = 0.39$
  - $u = 3.41$ pF
  - $%SW = 4.31$
  - LL = 40.4, PL = 17.1
  - $%<0.002 = 45.5$

- GWL
  - 4.27 m (Jun. 25/99)
  - 4.8 m (Feb. 1/01)
  - 4.0 m (Jul. 15/01)
FIGURE 5  Water content profiles of the site over a period of two years.
FIGURE 6. Void ratio profiles of the site at different conditions.
FIGURE 7. Two typical damage modes caused by expansive soil (after PTI manual, 1992)
FIGURE 8. Calculated PVR values for the site over a period of two years.
FIGURE 9. Measured footing movements over two years