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THE PREDICTION OF HEAVE IN EXPANSIVE SOILS

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ABSTRACT:

Geotechnical problems associated with expansive soils are common throughout the world. In general, these problems are more severe in arid regions where high evaporation leads to cracking and desiccation of upper, unsaturated soil layers. The volume change theory for saturated soils has been extended to give engineers a practical tool for predicting heave in swelling, unsaturated soils. This paper describes the theory related to swelling soils, outlines the procedure for testing these soils in a one-dimensional oedometer, and explains how the data should be interpreted. Two case histories are also presented.

1. INTRODUCTION

Lightly loaded structures commonly suffer severe distress subsequent to their construction. Changes in the environment around the structure result in changes in negative pore-water pressures which in turn produce volume changes in the soil. Soils with a high swelling index, C_s, in a changing environment, are commonly found to be highly swelling soils.

In 1980, Krohn and Slosson estimated that 7 billion dollars was spent each year in the United States as a result of damage to all types of structures built on swelling soils. Jones and Holtz (1973) pointed out that more than twice as much money was spent on damage due to swelling soils as was spent on damage from floods, hurricanes, tornadoes and earthquakes. Certainly the problem is of enormous financial proportions. The problem is also global. Australia, Argentina, Burma, Canada, China, Cuba, Ethiopia, Great Britain, India, Israel, Kenya, Mexico, Spain and United States are some of the countries which must cope with expansive soils. In general, regions with more arid climates have more severe expansive soil problems (Fredlund and Rhardjo, 1993).

Predicting heave of light structures has probably received more attention than any other analysis associated with swelling soils. Numerous analytical procedures have been proposed in various countries, however, most methods have been used to a limited extent and within restricted geographical regions. Current unsaturated soil mechanics practice combines the different methods for predicting heave into one consistent theoretical context.

In developing the theory and subsequent analysis techniques, it is necessary to relate swelling soil behavior to the stress state in the soil.

Engineers must be able to visualize volume changes in terms of appropriate stress state variable changes. The success of the practice of saturated soil mechanics can be attributed largely to the ability of engineers to relate soil behavior to changes in the effective stress state variable. Swelling soils are generally unsaturated and engineers have found it much more difficult to relate soil behavior to stress state variable changes. The primary objective of this paper is to assist engineers in relating the volume change behavior of unsaturated, swelling soils to changes in the stress state. Specifically, the objectives can be summarized as follows:

1. to explain how past, present and future behavior of a swelling soil can be explained in terms of stress state variables. In doing so, a similar philosophical framework to that used in saturated soil mechanics is adopted.

2. to describe a method that can be used to predict heave. The method involves the use of one-dimensional oedometer tests. Emphasis is placed on the interpretation of the laboratory results.

3. to briefly present two case histories involving swelling soils. The results of these studies confirm the reasonableness of the presented heave prediction method.

2 STRESS STATE VARIABLES CONTROLLING SOIL BEHAVIOR

Three stresses must be measured, estimated or predicted in order to describe the behavior of an unsaturated soil. These are the total stresses, σ, the pore-water pressure, u_w, and the pore-air pressure, u_a. These variables can be combined into two independent stress state variables for unsaturated soils (Fredlund and Morgenstern, 1977). Although various combinations of independent stress state variables are possible, the $(\sigma - u_a)$ and $(u_a - u_w)$ combination has proven to be most advantageous since the effects of total stress changes and pore-water pressure changes can be separated. This is beneficial both from a conceptual and analytical standpoint since pore-air pressure can generally be assumed to be atmospheric. The $(\sigma - u_a)$ term is referred to as the "net total" stress, and the $(u_a - u_w)$ term is referred to as the matric suction. These stress state variables provide a smooth transition when going from the unsaturated to the saturated soil cases. As the degree of saturation approaches 100 percent, the pore-air pressure and the pore-water pressure become approximately equal in magnitude. When the matric suction term goes to zero, the pore-air pressure in the $(\sigma - u_a)$ term becomes the pore-water pressure.

The independent stress state variables can be used to assist in understanding the behavior of a swelling clay deposit. Let us consider a deposit of proglacial, lacustrine origin. The present physical properties and state of stress of the clay are dependent upon stress influences subsequent to deposition. When studying a potential heaving problem, the engineer must evaluate the present state of stress in the soil and determine suitable physical properties for predicting future behavior.

2.1 Stress History:

Deposits in a proglacial lake are initially consolidated by the bouyant weight of the over-lying sediments. The drainage of the lake and the subsequent evaporation of water from the lake sediments causes desiccation of the underlying sediments. The term "desiccation" is used to mean the drying of the soil by evaporation and evapotranspiration. The water table is simultaneously drawn below the ground surface. The total stress on the sediments remains essentially constant, while the stress in the water phase is reduced (i.e., it becomes negative above the water table). This gives rise to an increase in effective stress and the soil consolidates. The tension in the water phase acts in all directions and, as a result, there is a tendency for cracking and overall desaturation of the upper portion of the profile (Fig. 1).

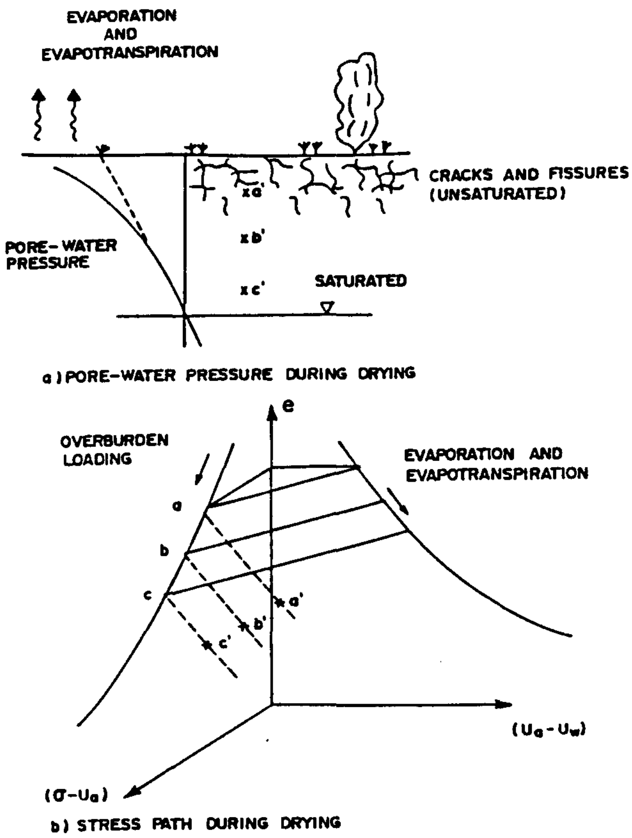


Fig. 1 Stress representation after the lake sediments are subjected to evaporation and evapotranspiration.

Grass, trees, and other plants also start to grow on the surface with the net effect of further drying the soil by applying a tension to the water phase. Most plants are capable of applying 1000 to 2000 kPa of tension to the water phase prior to reaching their wilting point. A high tension in the water phase (i.e., high matric suction) means that the soil is highly desiccated. The drying results in an affinity of the soil for water (Fig. 1a).

Year after year, the surface deposits are subjected to varying and changing environmental conditions. In response to these changes, the upper portion of the deposits swell and shrink. Volume changes may extend to depths in excess of 2.5 meters. Environmental changes transmit a change in stress to the pore-water. These stress changes are isotropic. Conversely, changes in total stress imposed by humans are generally anisotropic. It is advantageous to separate the effects of total and pore-water pressure changes in accordance with the stress state variables involved.

Evaporation and evapotranspiration are depicted as movements in the matric suction plane, whereas loads applied to the soil are shown in the net total stress plane (Fig. lb). Wetting and drying due to environmental effects are visualized as changes along hysteresis loops in the matric suction plane. In arid and semi-arid regions, the natural water content gradually decreases.

Low water contents in clay deposits indicates that the soil has the potential for swell if evaporation and evapotranspiration are not permitted from the ground surface as a result of covering the area with a building, or asphalt, etc.

2.2 Present State of Stress:

When the soil is sampled for laboratory testing purposes, the state of stress may be anywhere along either a drying or wetting portion of the void ratio versus stress relationship. Figure

2 illustrates a typical, complex stress history. In reality, the soil has undergone thousands of cycles of drying and wetting. At the point of sampling, the soil is subjected to a specific net total stress and a specific insitu matric suction.

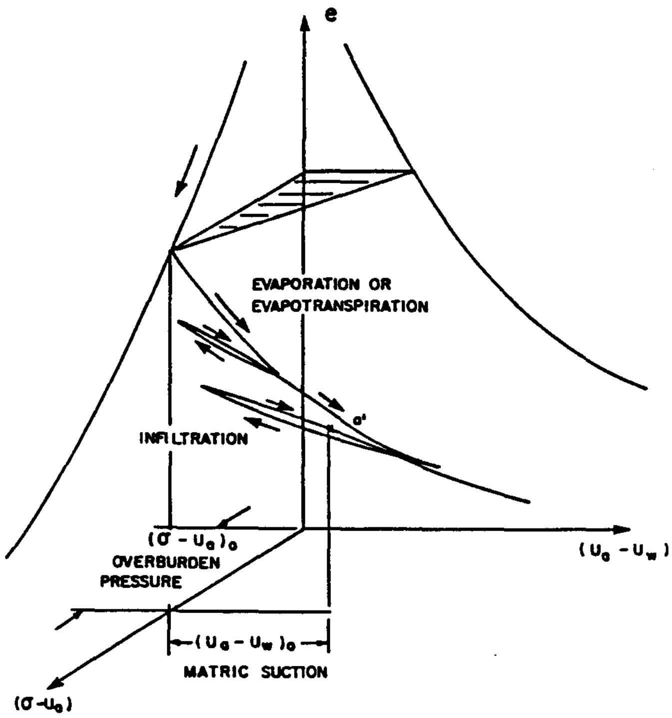


Fig. 2 Stress representation when the soil has undergone a complex stress history caused by drying and wetting.

The primary laboratory information desired by the engineer for analyzing a swelling soil situation is an assessment of: (I) the insitu state of stress, and (ii) the swelling properties with respect to changes in matric suction. It is necessary to develop a simple, rapid, and economical procedure to obtain the information required for solving practical swelling clay problems.

Several laboratory testing procedures are used in practice to obtain the required information. These generally involve the use of the one-dimensional consolidation apparatus (i.e., oedometer). In North America the common procedures are the "Free Swell," "Loaded Swell" and "Constant Volume" tests (ASTM D 4546-95, methods "A," "B" and "C", respectively).

The oedometer tests the soil on the total stress plane. The assumption is therefore made that it is possible to eliminate the matric suction in the soil by immersing the specimen in water and obtaining the necessary soil properties and stress values from the total stress plane.

Constant Volume Oedometer Test Procedure:

Let us consider the "Constant Volume" oedometer test procedure. The sample is subjected to a token load and immersed in water. As the sample attempts to swell, the applied load is increased to maintain the sample at a constant volume. This procedure is continued until there is no further tendency for swelling. The applied load at this point is referred to as the "uncorrected" swelling pressure, P_S. The sample is then further loaded and unloaded in the conventional manner. The test results are commonly plotted as shown in Fig. 3a. The actual stress paths followed during the test are more clearly understood using a three-dimensional plot with the stress state variables forming the abscissas (Fig. 3b).

An understanding and visualization of the stress paths followed during the test assists with the interpretation of the data. The void ratio and water content stress paths are shown for the situation where there is a minimal disturbance due to sampling. Even so, the loading path displays some curvature as the total stress plane is approached. In actuality, the stress path may show even more influence from sampling (Fig. 4). Engineers have long recognized the significance of sampling disturbance when determining the preconsolidation pressure for a saturated clay

(Casagrande, 1936). Fredlund, Hasan and Filson (1980) discuss the significance of sampling disturbance in evaluating the swelling pressure of a soil.

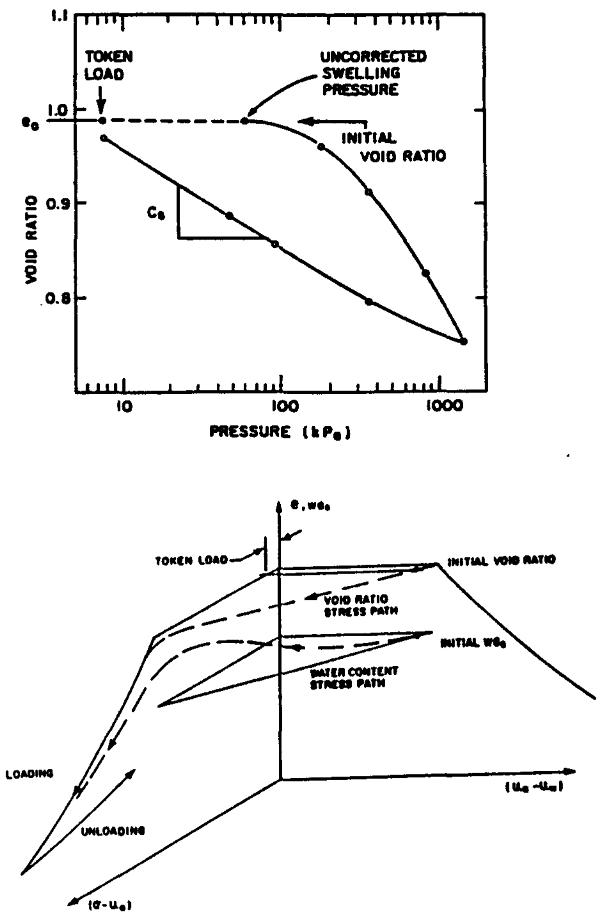


Fig. 3 Interpretation of data from a "Constant Volume" oedometer test.

Sampling disturbance causes the conventional swelling pressure, P_S, to fall below the "ideal" or "corrected" swelling pressure, P'_S. The "corrected" swelling pressure represents the insitu stress state translated to the total stress plane. It is equal to the overburden pressure plus the insitu matric suction translated onto the total stress plane. The translated suction is called the "matric suction equivalent" (Yoshida, Fredlund and Hamilton 1982). The magnitude of the matric suction equivalent is lower than the insitu matric suction, with the difference being primarily a function of the insitu degree of saturation. The engineer needs to obtain the "corrected" swelling pressure from the oedometer test in order to reconstruct the insitu stress conditions. The procedure for accounting for sampling disturbance is discussed later in this paper.

Free Swell Oedometer Test Procedure:

The "Free Swell" oedometer test can also be used to measure the swelling pressure and swelling properties of a soil. The sample is initially allowed to swell freely with a token load applied (Fig. 5). The load required to bring the sample back to its original void ratio is termed the swelling pressure. The stress paths adhered to can be visualized using a three-dimensional plot of stress state variables versus void ratio and water content. This test procedure involves both a volume increase and decrease and incorporates hysteresis into the estimation of the insitu stress state (i.e., swelling pressure). However, the advantage of this procedure is that it appears to somewhat compensate for the effects of sampling disturbance.

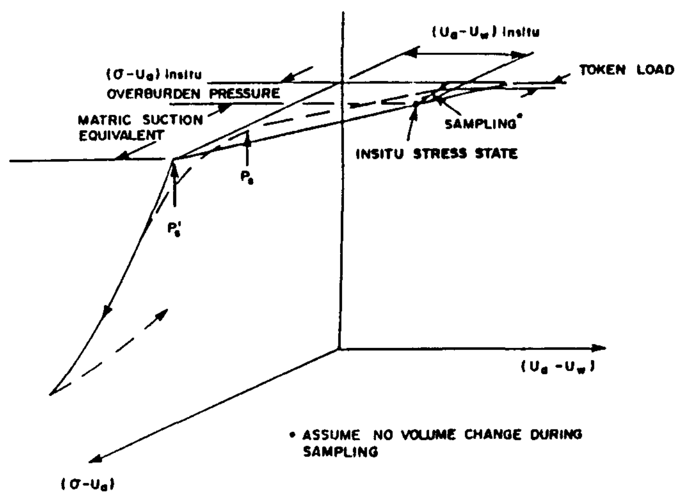


Fig. 4 Actual stress path showing the effect of sampling disturbance.

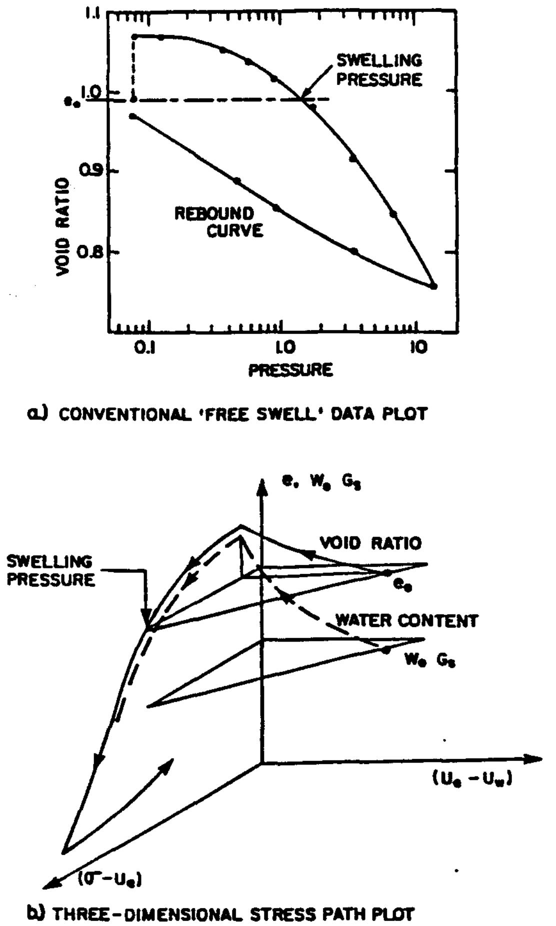


Fig. 5 Stress path representation for the "Free Swell" oedometer test.

Chinese Code Method to Measure Swelling Pressure:

There are a number of other methods which have been developed in various countries to measure the swelling pressure of a soil. Only one such method will be further referred to in this paper. The Chinese Technical Code for Construction in Expansive Soils Regions, GBJ112-87, describes a method which has been called the "Swell Ratio and Swell Pressure Test." There are two variations to the procedure, both of which use a conventional oedometer apparatus. The first procedure is carried out to obtain reference data for use in foundation deformation analyses. The first procedure is performed on a single soil specimen which is loaded to a pre-determined pressure and then saturated. The applied pressure is determined in accordance with the site construction requirements and should be slightly larger than the anticipated loading conditions. After equilibrium is reached, the soil is unloaded following standard oedometer procedures. The swell ratio for the applied load is determined using:

$$\delta_1 = \frac{H_2 - H_o}{H_o} \qquad [1]$$

where: δ_1 = swell ratio for procedure 1,
H_2 = specimen height after swelling,
H_o = initial specimen height before any load is applied.

The second Chinese procedure is carried out to obtain the swelling pressure. In this procedure, several specimens are prepared and subjected to different initial applied loads. The swell ratio for each load is then calculated using:

$$\delta_2 = \frac{H_2 - H_1}{H_o} \qquad [2]$$

where: H_1 = specimen height after initial loading,
δ_2 = the swell ratio for procedure.

The various swell ratios, δ_2 , are plotted versus the applied loads, and the intersection of the curve with the zero percent swell ratio axis is referred to as the swelling pressure. Figure 6 shows the stress paths followed during "Swell Ratio" tests.

Equations 1 and 2 have been used to compute the swell ratios for the second Chinese procedure. However, the final computed swelling pressures will differ significantly depending on the computation method chosen. Current research underway at the University of Saskatchewan is comparing the swelling pressures computed using the Chinese methods with those computed using the "Constant Volume" and "Free Swell" tests.

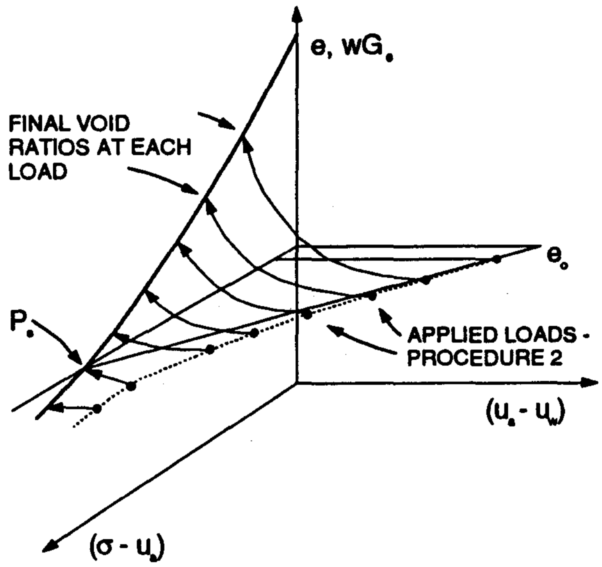


Fig. 6 Stress paths for Chinese "Swell Ratio" swelling test - procedure 2.

2.3 Future Ground Movements:

The prediction of future ground movements requires a knowledge of: (i) the initial insitu state of stress, (ii) the swelling moduli and (iii) the final state of stress. The initial state of stress can be quantified from the "corrected" swelling pressure. The swelling moduli can be obtained from the rebound data. The final state of stress corresponding to several years after construction must be estimated on the basis of local experience. Possible final pore-water pressure profiles are discussed later in this paper.

For discussion purposes, let us assume that the final pore-water pressures go to zero. Figure 7 shows the stress path that would be followed by a soil element at a specific depth. Swelling would follow a path from the initial void ratio, e_o, to the final void ratio, e_f , along the rebound surface of the matric suction plane. The rebound surface can be assumed to be unique (Matyas and Radhakrishna 1968; Fredlund and Morgenstern 1976). Therefore, it is also possible to follow a stress path from the insitu stress state to the "corrected" swelling pressure and then proceed along the rebound curve in the total stress plane to the final stress condition. The advantage of the latter stress path is that the soil properties determined in the total stress plane can be used to predict total heave.

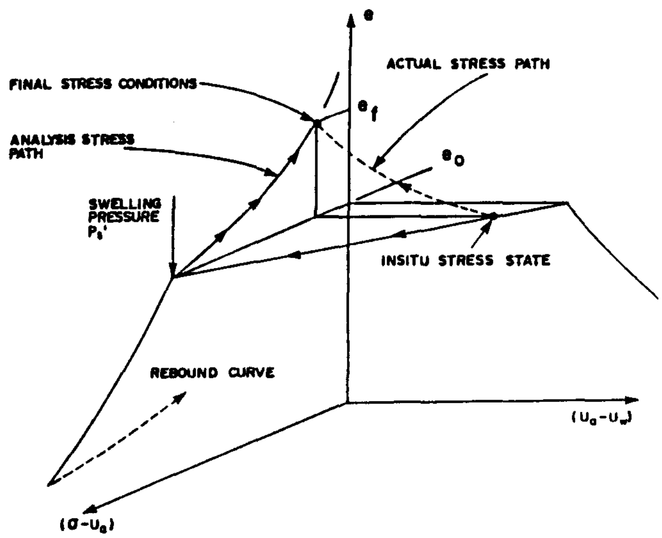


Fig. 7 "Actual" and "analysis" stress paths representing swelling of the soil.

The effects of excavation, replacement of soil with a relatively inert material (e.g., gravel) and loadings can also be taken into account by using appropriate moduli for loading and unloading. However, it is preferable to assume that there is insufficient time for the soil to respond to each loading and unloading, and that long term heave is in response to the net loading or unloading.

2.4 Determination of Insitu Consolidation / Swelling Curve:

When testing saturated clays, the laboratory oedometer test is used to reconstruct the insitu void ratio versus effective stress plot. Likewise, the laboratory oedometer test on desiccated soils can be used to construct a void ratio versus pressure plot for analysis purposes. Often the entire laboratory loading curve is on the recompression portion; not even reaching the virgin compression branch. The preconsolidation pressure of the clay may exceed the highest load applied in the laboratory. The "corrected" swelling pressure indicates the present insitu state of stress on the total stress plane. The lower, "uncorrected" swelling pressure shows the effect of sampling disturbance. Upon access to water in the field, the soil swells along the rebound curve. The laboratory rebound curve in the vicinity of the initial void ratio, e_o, must be translated upward to pass through the "corrected" swelling pressure in order to show the stress path that would be followed.

The following procedure is suggested for obtaining the "corrected" swelling pressure. First, an adjustment should be made to the laboratory data in order to account for the compressibility of the oedometer apparatus. Desiccated, swelling soils have a low compressibility and the compressibility of the apparatus can significantly affect the evaluation of insitu stresses and the slope of the rebound curve (Fredlund 1969). Second, a correction must be applied for sampling disturbance. Sampling always increases the compressibility of a soil and does not permit the laboratory sample to return to its insitu state of stress at its insitu void ratio. Casagrande (1936) proposed an empirical construction on the laboratory curve to account for the effect of sampling disturbance when assessing the preconsolidation pressure of a soil. Other construction procedures have also been proposed (Schmertmann 1955). A modification of Casagrande's construction is suggested for finding the "corrected" swelling pressure (Fig. 8).

The need for applying a correction to the laboratory measured swelling pressure is revealed in numerous ways. First, it would be anticipated that such a correction is necessary as a result of experience in determining preconsolidation pressure. Second, attempts to use the "uncorrected" swelling pressure in the prediction of total heave result in predictions which are too low. Predictions using "corrected" swelling pressure may often be twice the magnitude of those predicted when no correction is applied. Third, the analysis of oedometer results from desiccated deposits often produce results which are difficult to interpret if no correction is applied for sampling disturbance (Fredlund, Krahn and Hasan, 1980).

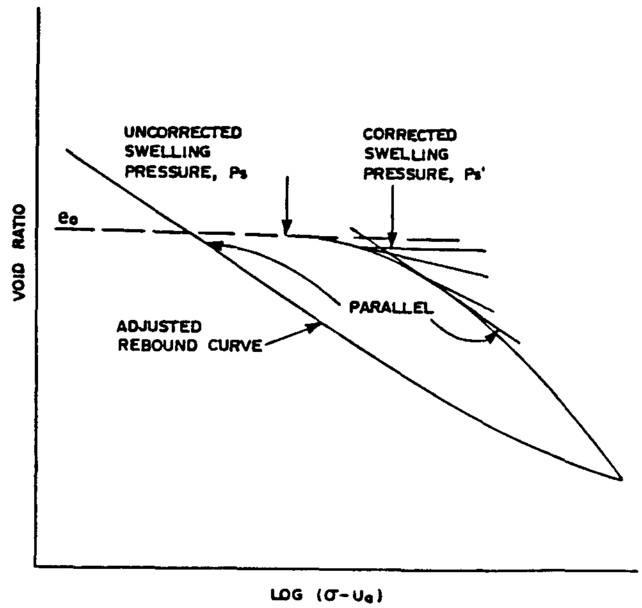


Fig. 8 Construction procedure to correct for the effect of sampling disturbance.

Figure 9 shows a comparison of "corrected" and "uncorrected" swelling pressure (i.e., "Constant Volume" oedometer tests) data from 2 soil types. The results indicate that it is possible for the "corrected" swelling pressures to be more than 300 percent of the "uncorrected" swelling pressures.

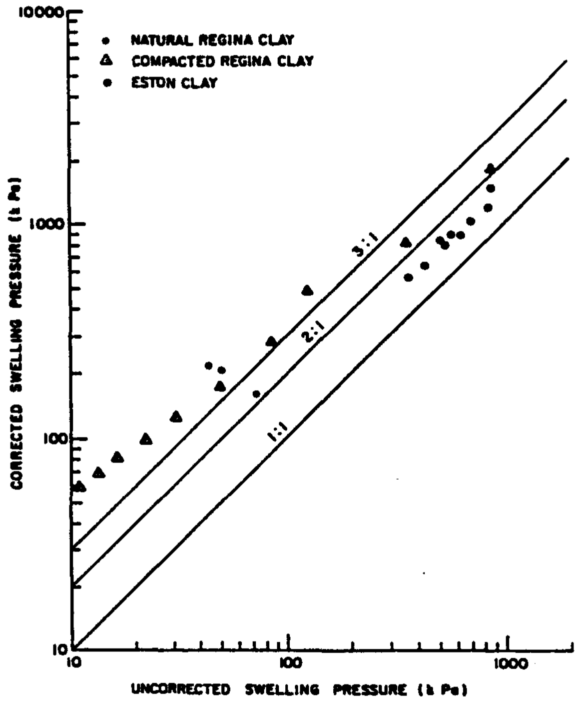


Fig. 9 Change in swelling pressure due to correction for sampling disturbance.

Figure 10 shows uncorrected and corrected swelling pressure results obtained on samples of compacted Regina Clay. Also shown are the results of a series of "Free-swell" oedometer tests performed on the same soil. The swelling pressure from the "corrected" Constant Volume tests are slightly greater than those from the Free-Swell tests.

3. THEORETICAL DERIVATION FOR PREDICTION OF HEAVE

The continuity requirement that must be satisfied for an unsaturated soil is as follows (Fredlund, Hasan and Filson, 1980):

$$\frac{\Delta V}{V} = \theta_a + \theta_w \qquad [3]$$

where: V = volume of a referential element,
θ_a = change in amount of air (by volume) in the element
θ_w = change in amount of water (by volume) in the element.

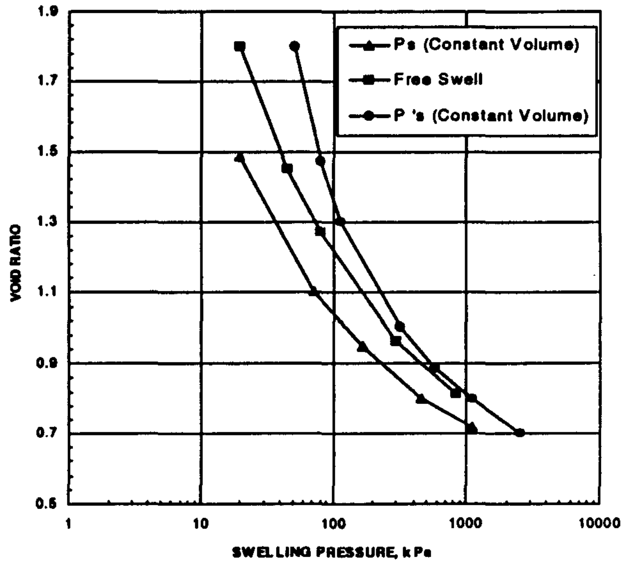


Fig. 10 Void ratio versus swelling pressure for compacted Regina Clay.

The change in volume can also be written as the sum of orthogonal linear strains (Fredlund, 1973).

$$\frac{\Delta V}{V} = \varepsilon_x + \varepsilon_y + \varepsilon_z \qquad [4]$$

where: $\varepsilon_x, \varepsilon_y, \varepsilon_z$ = the strain in the x,y,z directions.

The linear strains can be linked to the stress state variables using an extension of the elasticity formulation for saturated soils (Fredlund and Morgenstern, 1976). Strain in the y - direction is given by:

$$\varepsilon_y = \frac{(\sigma_y - u_a)}{E} - \frac{\mu}{E}(\sigma_x + \sigma_z - 2u_a) + \frac{(u_a - u_w)}{H} \qquad [5]$$

where: E = Young's modulus with respect to $(\sigma - u_a)$,
μ = Poisson's ratio,
H = elastic modulus with respect to $(u_a - u_w)$.

The above equation can also be written for the x-and z- directions. In an oedeometer, it is assumed that strains in the x- and z- directions are equal to zero. Therefore, if the above equations, written for ε_x and ε_z, are equated and substituted into Eq. 5, the resulting equation for strain in the y - direction is:

$$\varepsilon_y = m_1^s d(\sigma_y - u_a) + m_2^s d(u_a - u_w) \qquad [6]$$

where: $m_1^s = \dfrac{(1+\mu)(1-2\mu)}{E(1-\mu)}$;, the soil compressibility modulus associated with a change in $(\sigma - u_a)$,

$m_2^s = \dfrac{(1+\mu)}{H(1-\mu)}$; the soil compressibility modulus associated with a change in $(u_a - u_w)$.

Equation 6 shows the relationship between deformation and the stress state variables for unsaturated soils. From a conventional soil mechanics standpoint, it is often more convenient to use the change in void ratio, Δe, rather than strain, ε, for the formulation of a heave equation. Equation 4 then becomes:

$$\frac{\Delta e}{(1+e_o)} = m_1^s d(\sigma_y - u_a) + m_2^s d(u_a - u_w) \qquad [7]$$

where: e_o = the initial void ratio.

The amount of heave associated with an arbitrary layer of soil (i.e., i^{th} layer) is computed by multiplying the vertical strain in the layer, ε_i, by the thickness of the layer, h_i. Written in terms of void ratios, the heave in a specific layer is computed as follows:

$$\Delta h_i = h_i \frac{\Delta e}{1+e_0} \qquad [8]$$

The change in void ratio, Δe, refers to the difference between the initial and final stress states.

$$\Delta e = e_f - e_0 \qquad [9]$$

where: e_f = the final void ratio.

Use of Eq. 8 to calculate soil heave requires a knowledge of the initial and final void ratios. This information is available from oedometer test data and from estimations of the final pore-water pressures.

The rebound portion of the oedometer test data, plotted in semi-logarithm form, is essentially a straight line and can be writted as follows:

$$e_f = e_o - C_s \log \frac{p_f}{p_o} \qquad [10]$$

where: C_s = swelling index,
p_f = final stress state,
p_o = initial stress state.

The initial stress state, p_o, is the sum of the overburden pressure and the matric suction transferred to the total stress plane (i.e., matric suction equivalent). The initial stress state is always equal to the "corrected" swelling pressure.

$$p_o = (\sigma_v - u_a) + (u_a - u_w)_e \qquad [11]$$

where: σ_v = original overburden pressure,
$(u_a - u_w)_e$ = matric suction equivalent (i.e., transferred to the total stress plane - see Figure 3 b).

It is necessary to have some understanding of the "corrected" swelling pressure versus depth relationship for the deposit under consideration. The final stress state, p_f, must account for total stress changes and the final pore-water pressure conditions.

$$p_f = (\sigma_v - u_a) \pm \Delta\sigma - u_{wf} \qquad [12]$$

where: $\Delta\sigma$ = the change in total stress due to excavation or placement of fill,
u_{wf} = estimated final pore-water pressure.

When Eq. 9 is solved for the change in void ratio, Δe, and substituted into Eq. 8, the heave in any layer within a strata can be written as:

$$\Delta h_i = h_i \frac{C_s}{1+e_o} \log \frac{p_f}{p_o} \qquad [13]$$

$$\Delta h_i = h_i \frac{C_s}{1+e_o} \log \frac{(\sigma_v - u_a) \pm \Delta\sigma - u_{wf}}{(\sigma_v - u_a) + (u_a - u_w)_e} \qquad [14]$$

The total heave Δh, is the sum of the heaves computed for each layer.

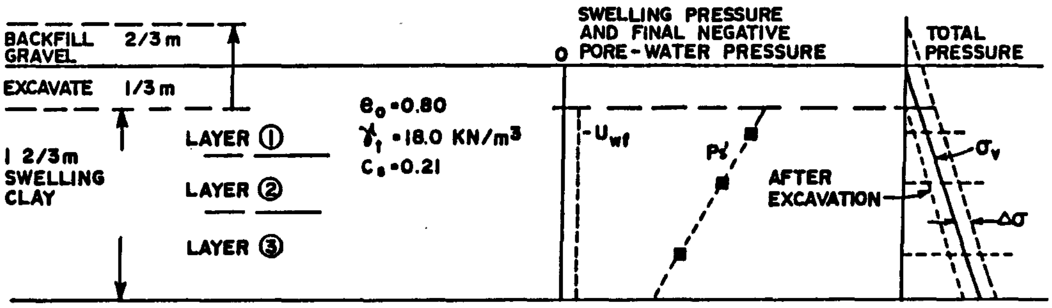
$$\Delta h = \Sigma \Delta h_i \qquad [15]$$

The matric suction is often a maximum near the ground surface of a deposit. This is also the zone of lowest overburden pressure. Therefore, the ratio of p_f and P_o is most negative in this region, resulting in the largest amount of heave.

3.1 Initial and Final Pore-water Pressure Boundary Condition:

The initial and final stress states must be known in order to perform a heave analysis. The initial and final total stresses can be computed using conventional total stress theory. The initial and final pore-air pressure is equal to atmospheric pressure. The need to know the initial insitu pore-water pressures is circumvented through the manner in which the laboratory oedometer test data is interpreted. One of three possibilities provides the most logical estimation of the final pore-water pressure conditions.

First, it can be assumed that the water table will rise to ground surface, creating a hydrostatic condition. This assumption produces the greatest heave prediction. Second, it can be assumed that the pore-water pressure approaches zero throughout its depth. This may be a realistic assumption, however, it should be noted that it is not an equilibrium condition. Third, it can be assumed that under long-term equilibrium conditions the pore-water pressure will remain slightly negative. This assumption produces the smallest prediction of heave. It is also possible to have variations of the above assumptions with depth. As well, there may be a limit placed on the depth to which wetting will occur. All of the above assumptions produce similar predictions of heave in most cases. This is due to the fact that most of the heave occurs in the uppermost soil layer where the matric suction change is largest.



		INITIAL STRESS STATE	FINAL STRESS STATE				
LAYER NO.	THICKNESS (mm)	$P_o = P_s'$ (kPa)	INITIAL OVERBURDEN σ_v (kPa)	CHANGE IN TOTAL STRESS $\Delta\sigma$ (kPa)	FINAL PORE-WATER PRESSURE u_{wf} (kPa)	$P_f = \sigma_v \pm \Delta\sigma - u_{wf}$ (kPa)	Δh_1 (mm)
1	333.	800.	9.0	+6.0	-7.0	22.0	60.6
2	500.	608.	16.4	+6.0	-7.0	29.4	76.7
3	833.	300.	28.4	+6.0	-7.0	41.4	83.6

TOTAL HEAVE = 220.9 mm

Fig. 11 Calculations for Example

The choice of a final pore-water pressure boundary condition can vary from one geographic location to another depending upon the climatic conditions. Russam and Coleman (1961) related the equilibrium suction below asphaltic pavements to the Thornthwaite Moisture Index. On many smaller structures, however, it is often human-made factors such as leaky water lines and poor drainage that control the final pore-water pressure in the soil.

3.2 Example Calculations:

An example problem is presented to illustrate the calculations required to predict heave (Fig. 11). Let us consider a 2-meter layer of swelling soil with an initial void ratio of 0.8, a total unit weight of 18.0 kN/m3 and a swelling index, C_s, of 0.21.

Three oedometer tests were performed which show a decrease in the "corrected" swelling pressure with depth (Fig. 11). Suppose the engineering design suggests the removal of 1/3 meter of swelling clay from the surface, prior to the placement of 2/3 meter of gravel. The unit weight of the gravel is assumed as being equal to that of the clay. The 1-2/3 meters of swelling clay is subdivided into 3 strata as shown in Fig. 11.

The initial stress state, p_o, can be obtained by interpolation of the laboratory data to the midpoint of each layer. The final stress state, p_f, must take into account changes in the total stress and the final pore-water pressure. The final pore-water pressure is assumed to be -7.0 kPa. Equations [13] or [14] can be used to calculate the heave in each layer. The total amount of heave is computed to be 22.1 cm.

Two assumptions are made concerning the heave analysis in the Example. First, it is assumed that the independent processes of excavating and placing the gravel fill do not allow sufficient time for equilibrium to be established. Therefore, the soil responds only to the net changes in stress. Second, the designation of a final negative pore-water pressure assumes that near saturation, the slopes of the rebound curves on the matric suction plane and the total stress plane approach the same value. This assumption is reasonable provided the final pore-water pressures are relatively small.

4. CASE HISTORIES

Two case histories are briefly presented to demonstrate that the proposed method for predicting total heave can be used with a reasonably high degree of confidence.

4.1 Slab-on-Grade Floor, Regina, Saskatchewan:

In 1961, the Division of Building Research, National Research Council, undertook to monitor the performance of a light industrial building which was being constructed in north-central Regina. Details of the study have been presented by Yoshida, Fredlund and Hamilton, 1983. Instrumentation was installed to monitor ground movements at various depths below the slab. Water content changes were monitored using a neutron moisture meter probe. Undisturbed samples were taken as part of the subsurface exploration prior to the construction of the building. Constant Volume oedometer tests were performed on 3 samples and the swelling pressures are shown on Fig. 12. The average swelling index, C_s, was 0.09.

Approximately one year after construction, the owner noticed considerable cracking of the floor slab. Precise level surveys showed the maximum total heave to be 106 mm. The owner had also noticed a significant increase in water consumption (i.e., 35,000 litres). It was discovered that a leak had occurred in the hot water line beneath the floor slab, at the location of maximum heave. The leak was immediately repaired.

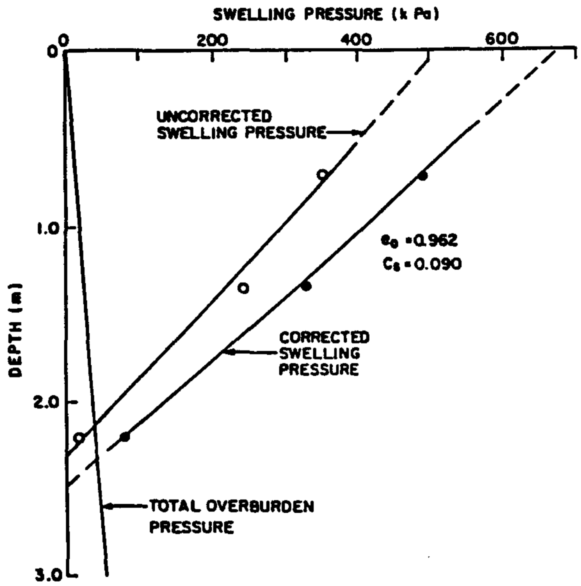


Fig. 12 Swelling pressure versus depth for Regina clay (from Yoshida, Fredlund and Hamilton, 1983).

Heave analyses were performed using the laboratory oedometer data. Various assumptions were made concerning the final pore-water pressures. When it was assumed that the soil had become saturated and the water table rose to the base of the floor slab, the predicted heave was 141 mm. Assuming that the negative pore-water pressures were reduced to zero, resulted in a total heave prediction of 118 mm. Assuming a final pore-water pressure of -50 kPa, gave a total heave prediction of 66 mm. On the basis of the heave analysis, it appears that the assumption of zero pore-water pressure was probably the most realistic for this case history. It appears that further heave would likely have taken place had the leak not been repaired. The prediction of heave at various depths also showed close agreement with the actual measurements.

4.2 Eston School, Eston, Saskatchewan:

Soils in the Eston area of Saskatchewan have long been known to be extremely high swelling. The stratigraphy consists of approximately 7-1/2 meters of highly plastic, brown clay overlying a glacial till. Many light structures in the area have undergone serious distress. The building of particular interest is the Old Eston School constructed in the late 1920's.

The school building was constructed on concrete strip footings and a wooden basement floor was supported by interior surface concrete footings. The school was a two-storey structure with classrooms in both the lower and upper levels. The lower floor was approximately 1.2 m low grade. The exterior concrete walls were founded approximately 1.8 m below grade.

A substantial amount of heave had taken place below the interior footings. Although the performance had not been precisely recorded, the heave in one portion of the basement area had been severe. On two occasions during the history of the school, 15 cm to 30 cm of soil had been removed from below the interior footings. As much as 45 cm to 90 cm of total heave had occurred during the life of the school according to maintenance records. Large amounts of differential heaving of the floor (i.e. 15 cm) were measured in 1960. The school was demolished in 1967.

In 1981, a subsurface investigation was conducted adjacent to the location of the old school. Undisturbed soil samples were taken and "Constant Volume" oedometer tests were performed. The results are presented in Fig. 13. The average natural water content throughout the profile was 25 percent. The average plastic limit was 27 percent and the average liquid limit was 100 percent. The average swelling index was 0.21. Due to a lack of detailed information on the soil and performance conditions of the school, it was not possible to do a precise total heave analysis. It was of interest, however, to perform an approximate analysis. Using the corrected swelling pressure from Fig. 13 and assuming the negative pore-water pressures went to zero, the predicted heave was computed to be in excess of 90 cm. This agrees well with the observed heave listed in the school records.

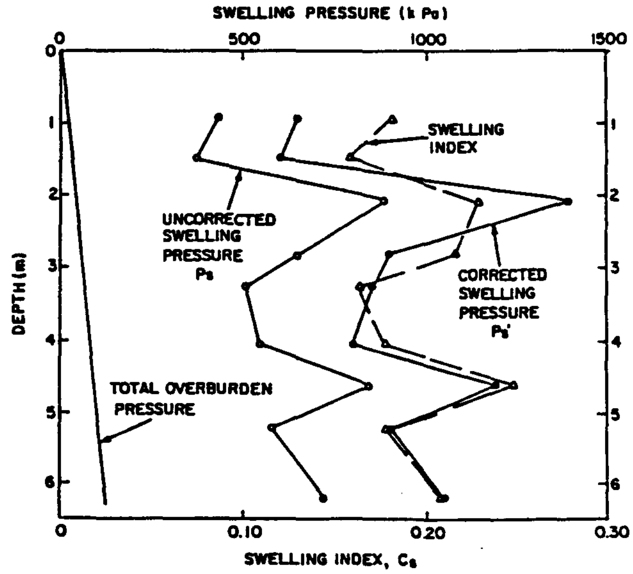


Fig. 13 Swelling index and swelling pressure versus depth for Eston Clay.

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