The prediction and performance of structures on expansive soils

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Abstract: Numerous analytical procedures have been proposed for the prediction of heave in swelling soils. These procedures have recently been examined within the context of unsaturated soil theory. This paper describes the theory related to the swelling of soils, outlines the procedure for testing these soils in a one-dimensional oedometer, and also explains how the data should be interpreted. Two case histories are also presented.

Key words: expansive soils, heave prediction, analytical procedure, unsaturated soil, one-dimensional oedometer, case history.

Introduction

Lightly loaded structures commonly suffer severe distress subsequent to their construction. Changes in the environment around the structure result in changes in the (negative) pore-water pressure, thereby producing 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.

Krohn and Slosson (1980) estimated that 7 billion dollars are 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 is spent on damage due to swelling soils as is spent on damage from floods, hurricanes, tornadoes and earthquakes. Certainly the problem is of enormous financial proportions.

The prediction of 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. Most methods have been used to a limited extent within a restricted geographical region. Only recently has there been an attempt to embrace the different methods for predicting heave within one consistent theoretical context.

It is necessary to relate soil behavior to the stress state in the soil, in order to develop a transferable science for swelling soils. The engineer 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. An attempt will be made to maintain a similar philosophical framework to that used in saturated soil mechanics.
(2) To describe a method that can be used to predict heave. The method involves the use of one-dimensional oedometer tests. Emphasis will be placed on the interpretation of the laboratory results.
(3) To briefly present two case histories involving swelling soils. The results of these studies are used to confirm the reasonableness of the proposed method.

Stress state variables controlling behavior

Three stresses must be measured, estimated or predicted in order to describe the behavior of an unsaturated soil. These are the total stress, 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 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 case. As the degree of saturation approaches 100 percent, the pore-air pressure

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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 to predict future behavior.

Stress history

Deposits in a proglacial lake are initially consolidated by the buoyant weight of the overlying sediments. The drainage of the lake and the subsequent evaporation of water over the lake sediments commences a 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).

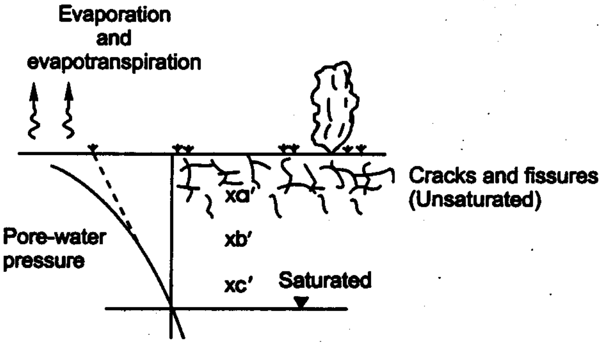
Grass, trees, and other plants also start to grow on the surface with the net effect of further drying the soil by applying a tension to the water phase. Most plants are capable of applying 10 to 20 atmospheres of tension to the water phase prior to reaching their wilting point. A high tension in the 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. 1*a*).

Year after year, the surface deposit is subjected to varying and changing environmental conditions. In response to these changes, the upper portion of the deposit swells and shrinks. Volume changes may extend to depths in excess of 10 feet (3 m). Environmental changes transmit a change in stress to the pore-water. The stress changes are isotropic. On the other hand, changes in total stress imposed by man 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.

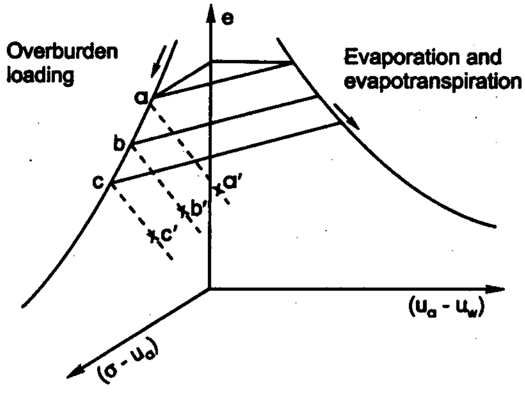
Evaporation and evapotranspiration are depleted as movements in the matric suction plane, whereas loads applied to the soil structure are shown in the net total stress plane (Fig. 1*b*). Wetting and drying due to environmental effects are visualized as changes along hysteresis loops in the matric suction plane. In arid and semi-arid regions, the natural water content gradually decreases.

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

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



a) Pore-water pressure during drying



b) Stress path during drying

Present state of stress

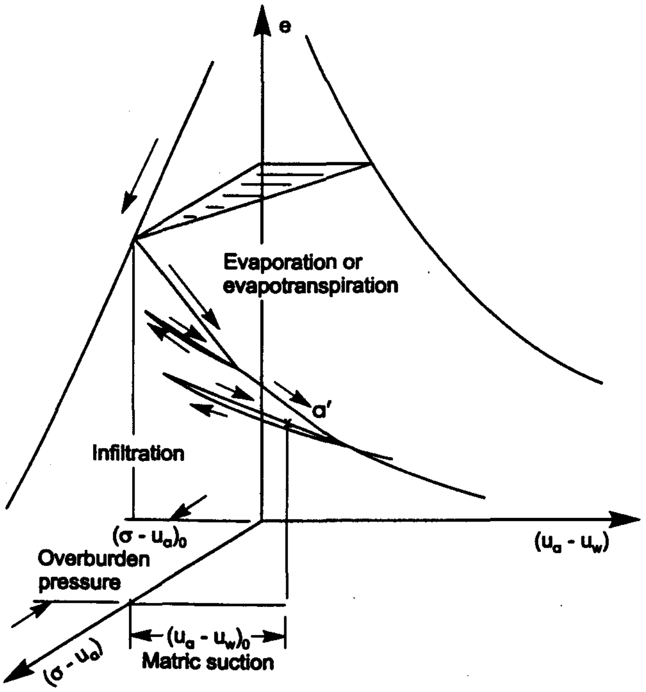
When the soil is sampled for laboratory testing purposes, the *in situ* state of stress may be anywhere along either a drying or wetting portion of the void ratio versus stress relationship. Figure 2 illustrates a typical, complex stress history. In reality, the soil has undergone thousands of cycles of drying and wetting. At the point of sampling, the soil is subjected to a specific net total stress and a specific *in situ* matric suction.

The primary laboratory information desired by the engineer for analysing a swelling problem is an assessment of: (1) the *in situ* state of stress, and (2) 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 soils information. These generally involve the use of the one-dimensional consolidation apparatus (i.e., oedometer) and are classified as the constant volume and free swell testing procedures (Noble 1966).

The oedometer can test the soil on the total stress plane. The assumption is made that it is possible to elimi-

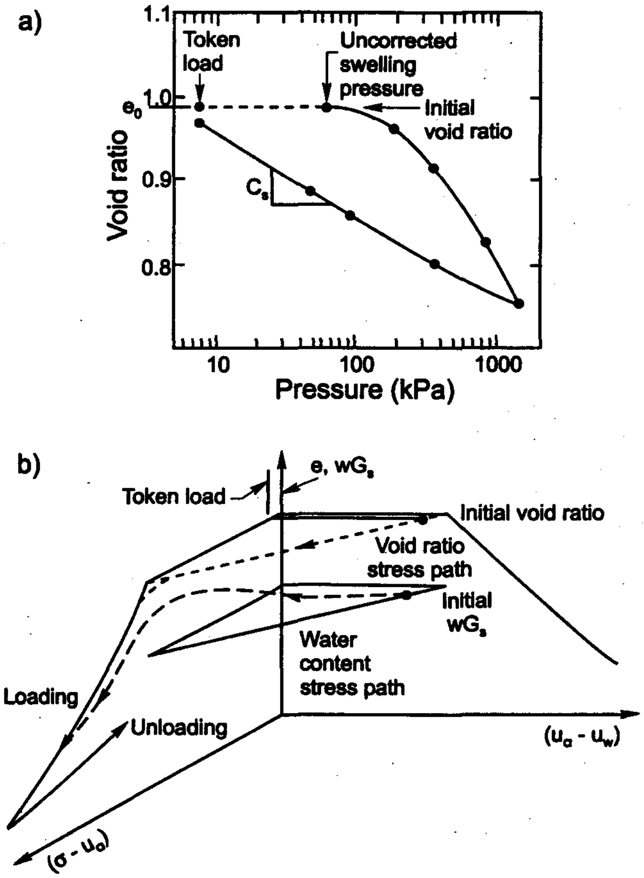
Fig. 2. Stress representation after soil has undergone a complex stress history caused by drying and wetting.



nate the matric suction in the soil by immersing of the specimen in water and obtaining the necessary soil properties and stress values from the total stress plane.

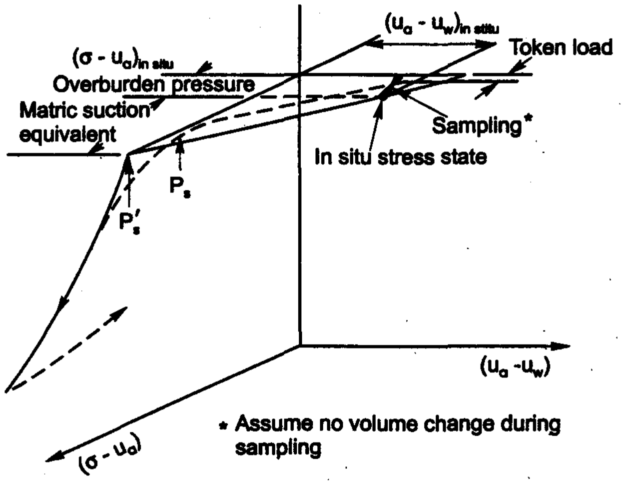
Let us first consider the constant volume oedometer test procedure. The specimen is subjected to a token load and submerged in water. As the specimen attempts to swell, the applied load is increased to maintain the specimen at a constant volume. This procedure is continued until there is no further tendency for swelling. The applied load at this point is referred to as the uncorrected swelling pressure, P_s. Then the specimen is further loaded and unloaded in the conventional manner.

Fig. 3. Interpretation of data from a constant volume oedometer test; *a*) Conventional procedure for plotting constant volume oedometer data; *b*) ideal stress path representation of constant volume oedometer data.



The test results are commonly plotted as shown in Fig. 3*a*. The actual stress paths followed during the test can be more clearly understood by use of a three-dimensional plot with the stress state variables forming abscissas (Fig. 3*b*). An understanding of the stress paths followed during the test assist in the interpretation of the data. The void ratio and water content stress paths are shown for the situation where there is a minimum of 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. Only recently; however, has the significance of sampling disturbance been recognized in evaluating the swelling pressure of a soil (Fredlund et al. 1980*a*).

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



Sampling disturbance causes the conventional swelling pressure, P_s, to fall well below the ideal or corrected swelling pressure, P_s'. The corrected swelling pressure represents the *in situ* stress state translated to the total stress plane. It is equal to the overburden pressure plus the *in situ* matric suction translated onto the total stress plane. The translated suction is called the matric suction equivalent (Yoshida et al. 1982). The magnitude of the matric suction equivalent will be lower than the *in situ* matric suction; the difference being primarily a function of the *in situ* degree of saturation. The engineer needs to obtain the corrected swelling pressure from the oedometer test in order to reconstruct the *in situ* stress conditions. The procedure for accounting for sampling disturbance is discussed later in this paper.

The free swell oedometer test can also be used to measure the swelling pressure and swelling properties of a soil. The specimen is initially allowed to swell freely with a token load applied (Fig. 5). The load required to bring the specimen back to its original void ratio is termed the swelling pressure. The stress paths adhered to can be understood from a three-dimensional plot of stress state variables versus void ratio and water content. This test has the limitation that it allows volume change and incorporates hysteresis into the estimation of the *in situ* stress state (i.e., swelling pressure). On the other hand, this procedure somewhat compensates for the effect of sampling disturbance.

Future ground movements

The prediction of future ground movements requires a knowledge of (1) the initial *in situ* state of stress, (2) the swelling moduli and (3) 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 profile are discussed under final boundary conditions.

For discussion purposes, let us assume that the final pore-water pressures go to zero. Figure 6 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_0, 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 *in situ* 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.

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.

Fig. 5. Stress path representation for the free swell oedometer test; *a*) conventional free swell data plot; *b*) three-dimensional stress path plot.

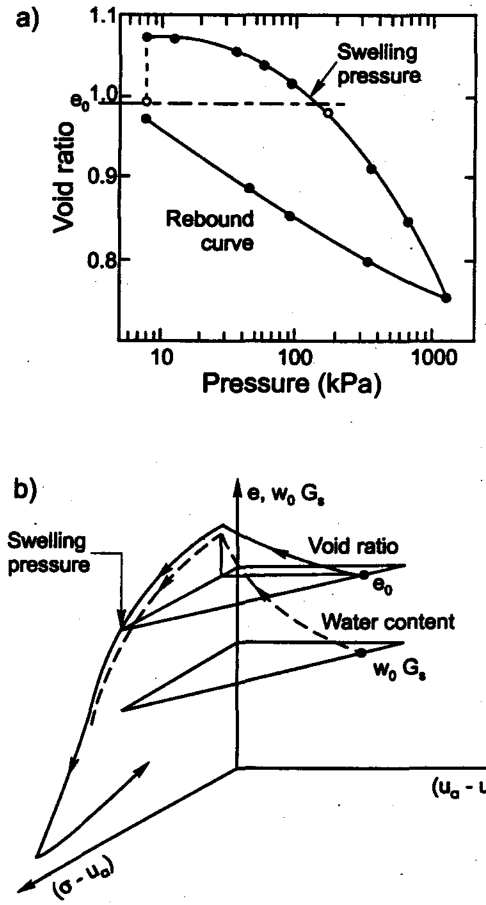
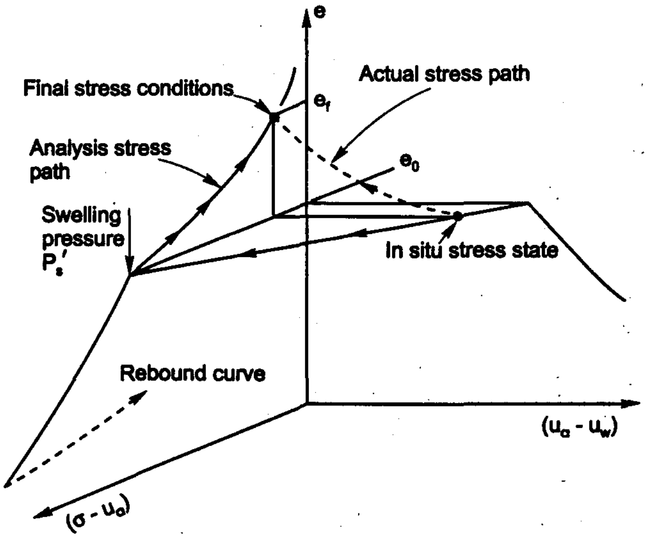


Fig. 6. Actual and analysis stress paths representing swelling of the soil.



Determination of *in situ* consolidation/swelling curve

When testing saturated clays, the laboratory oedometer test is used to reconstruct the *in situ* 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 *in situ* 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_0, 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 *in situ* 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 specimen to return to its *in situ* state of stress at its *in situ* 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 the Casagrande (1936) construction is suggested for finding the corrected swelling pressure (Fig. 7).

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 results 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 et al. 1980*b*).

Figure 8 shows a comparison of corrected and uncorrected swelling pressure data from two soil deposits. The results indicate that it is possible for the corrected swelling pressures to be more than 300 percent of the uncorrected swelling pressures.

Theoretical derivation for prediction of heave

An equation for the prediction of heave has been previously derived using the unsaturated soil theory (Fredlund et al. 1980*a*). In this paper, reference is made only to the theory necessary to use the results of an oedometer test to predict total heave. All stress paths considered are transferred to the total stress plane.

Fig. 7. Construction procedure to correct for the effect of sampling disturbance.

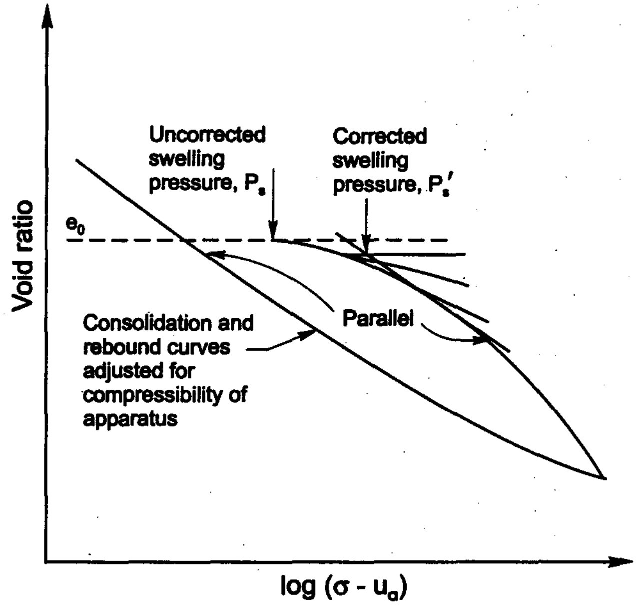
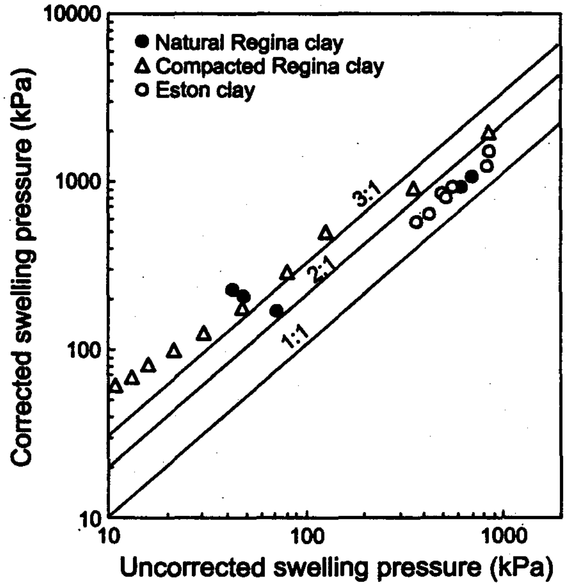


Fig. 8. Change in swelling pressure due to correction for sampling disturbance.



The rebound portion of the oedometer test data, plotted in a semi-logarithmic form, is essentially a straight line.

$$e_f = e_0 - C_s \log P_f / P_0 \quad [1]$$

where:

e_f = final void ratio,
e_0 = initial void ratio,
C_s = swelling index,
P_f = final stress state, and
P_0 = initial stress state.

The initial stress state, P_0, 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.

[2] $$P_0 = \sigma_v + (u_a - u_w)_e$$

where:

σ_v = original overburden pressure, and

$(u_a - u_w)_e$ = matric suction equivalent.

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.

[3] $$P_f = \sigma_v \pm \Delta\sigma - u_{wf}$$

where:

$\Delta\sigma$ = change in total stress due to excavation or placement of fill, and

u_{wf} = estimated final pore-water pressure.

The heave in an individual soil layer can be written in terms of changes in void ratio.

[4] $$\Delta h_i = h_i \Delta e / (1 + e_0)$$

where:

Δh_i = heave in a layer,

h_i = thickness of the layer under consideration, and

Δe = change in void ratio (i.e., $e_f - e_0$)

The heave in a layer in a strata can be written as:

[5] $$\Delta h_i = h_i \frac{C_s}{1 + e_0} \log P_f / P_0$$

[6] $$\Delta h_i = h_i \frac{C_s}{1 + e_0} \log \frac{(\sigma_v \pm \Delta\sigma - u_{wf})}{(\sigma_v + (u_a - u_w)_e)}$$

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

[7] $$\Delta h = \sum \Delta h_i$$

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_0 is most negative in this region, resulting in the largest amount of heave.

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 *in situ* 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. Any 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.

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 asphalt pavements to the Thornthwaite Moisture Index. On many smaller structures; however, it is often man-made causes such as leaky water lines and poor drainage that control the final pore-water pressure in the soil.

Example calculations

An example problem is presented to illustrate the calculations required to predict heave (Fig. 9, Table 1). Let us consider a 2-metre layer of swelling soil with an initial void ratio of 0.8, a total unit weight of 18.0 kN/m^3 and a swelling index of 0.21.

Three oedometer tests were performed which show a decrease in the corrected swelling pressure with depth (Fig. 9).

Suppose the engineering design suggests the removal of 1/3 metre of swelling clay from the surface, prior to the placement of 2/3 metre of gravel. The unit weight of the gravel is assumed as being equal to that of the clay. The 1–2/3 metres of swelling clay is subdivided into 3 strata as shown in Fig. 9.

The initial stress state, P_0, 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 [5] or [6] 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 excavation and placement of 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

Fig. 9. Calculations for example.

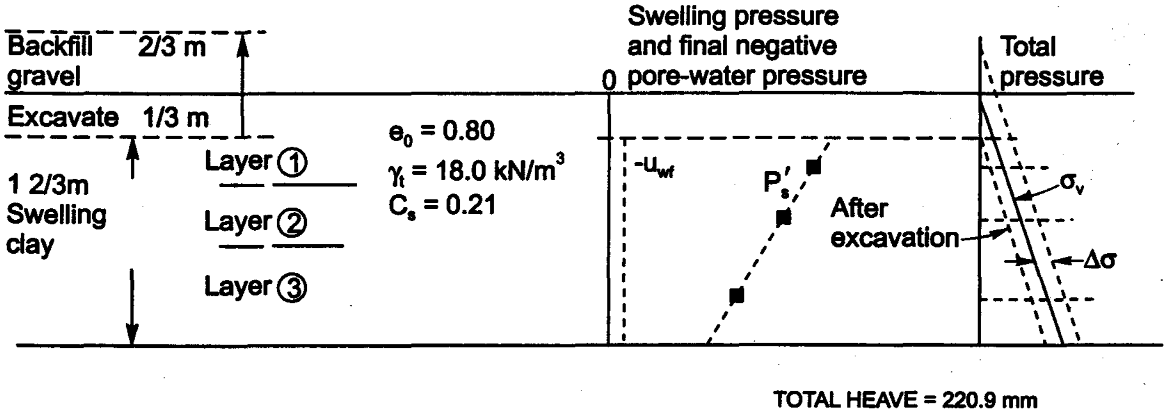


Table 1. Data for example calculation.

	Initial stress state				Final stress state		
Layer No.	Thickness (mm)	$P_0 = P_s'$ (kPa)	Initial overburden stress σ (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_i (kPa)
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

assumption is reasonable provided the final pore-water pressures are relatively small.

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.

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 et al. (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 three specimen and the swelling pressures are shown in Fig. 10. The average swelling index 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.

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 had risen to the base of the floor slab, the predicted heave was 141 mm. Assuming that the negative pore-water pressures were reduced to zero gave 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 appeared that the assumption of zero pore-water pressure was probably the most

Fig. 10. Swelling pressure versus depth for Regina clay (from Yoshida et al. 1983).

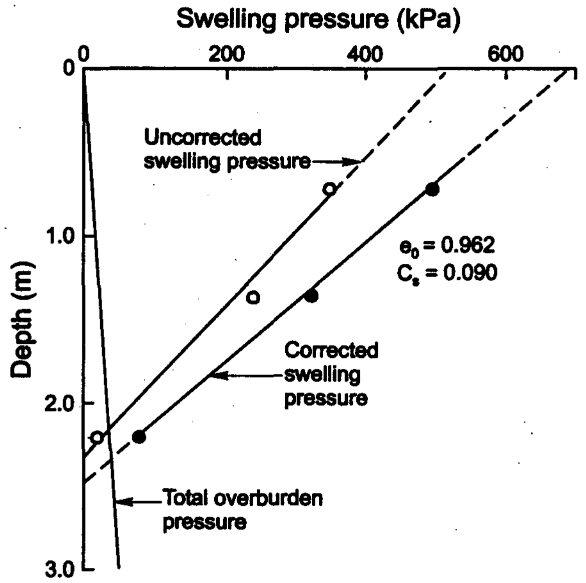
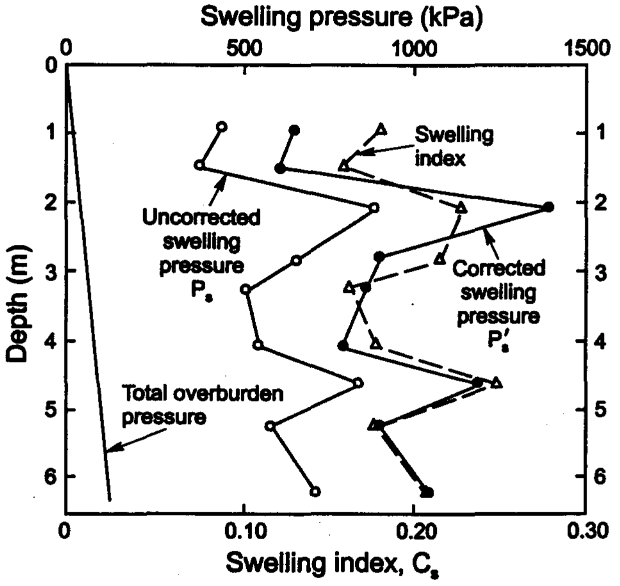


Fig. 11.



realistic for this case history. It appeared 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.

Eston School, Eston, Saskatchewan

Soils in the Eston area of Saskatchewan have long been known as extremely high swelling. The stratigraphy consists of approximately 7-1/2 metres of highly plastic, brown clay overlying the glacial till. Many light structures in the area have undergone serious distress. The building of particular interest was 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 (4 feet) below grade. The exterior concrete walls were founded approximately 1.8 m (6 feet) 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 (6 to 12 inches) of soil had been removed from below the interior footings. As much as 45 cm to 90 cm (1-1/2 to 3 feet) 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 or 6 inches) 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. 11. The average natural water content throughout the profile was 25 percent. The average elastic 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. 11 and assuming the negative pore-water pressures went to zero, the predicted heave would be in excess of 90 cm (3 feet).

Further case history studies would be useful in confirming the proposed procedure for predicting total heave.

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