Experience With Roads and Buildings on Expansive Clays

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ABSTRACT

The procedures used for designing safe and economic structures to resist load-induced deformation are inadequate for the design of roads and buildings on expansive clays. For these clays, increase in moisture causes not only a decrease in strength but also volume expansion that results in cracked buildings and rough pavements. A method of quantifying the amount of heave expected (depending on surcharge pressures), the depth of replacement with nonswelling materials, and the final equilibrium suction values are presented. The procedure is based on one-dimensional laboratory swelling curves. Since time, site, and budget limitations frequently do not allow for complete laboratory testing, commonly used empirical correlations are discussed. A method of calibrating these correlations, which are usually based on simple index tests is presented. The importance of post construction observations and comments on increased interest in the legal aspects of the problem are presented. A review of some early publications is included along with flow charts and a program written in BASIC for computing heave.

Engineers have considerable experience in designing safe and economic structures to resist load-induced deformation. For example, engineers use a pavement design procedure that accounts for possible softening of subgrade clays due to moisture increase within the design life of the pavement. However, for swelling clays, increase in moisture causes not only a decrease in strength, but also volume expansion and hence increased pavement roughness that is not load induced. Similarly, for building foundations on expansive clays, the use of an adequate factor of safety against bearing capacity failure, with due consideration for seasonal variations in strength, is an inadequate design procedure for assuring that deformation of the structure stays within safe limits.

A brief review of some of the literature published more than 25 years ago is presented next. Despite the progress that has been made in recent years, those associated with design and construction on expansive clays would do well to remember the complexity of the problems and the limitations on the ability to accurately predict performance. The legal aspects of the problem are, therefore, becoming increasingly important. In 1983 a symposium on the Influence of Vegetation on the Swelling and Shrinking of Clays was sponsored by the British journal Geotechnique. One of the many interesting papers presented at the symposium dealt with the legal aspects of the problem (1). The Fourth International Conference on Expansive Soils held in Denver, Colorado, also included a paper on the legal aspects of design and construction on expansive clays as observed by a consulting engineer (2). The proceedings of both meetings contain many interesting technical papers; however, the two papers on the legal aspects should be considered compulsory reading because premature distress is a common feature of roads and buildings built on expansive clay subgrades.

This paper includes guidelines for identifying a swelling clay problem, a discussion of laboratory swelling curves, and a method of calibrating commonly used empirical equations for predicting the parameters necessary to describe the laboratory swelling curve. A computational routine for computing surface heave is also presented.

REVIEW

Two interesting papers (by Porter and Wooltorton) draw attention to swelling clay problems (3, 4). These papers were published in the Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering held at Harvard University in 1936. Porter reported movement observations on a 1,500-ft long concrete pavement laid in Texas in 1931 (3). Cracks 15 cm wide and 3.5 m deep were observed in the pavement shoulders, and seasonal fluctuations in elevation of 12 cm at the pavement edge and 6 cm at the pavement centerline were measured. The subgrade soils at the site were reported to have a liquid limit of about 80 to 100, a plasticity index (PI) of 52 to 74, and a shrinkage limit of from 8 to 10. It is worth noting that on the basis of Atterberg limits alone, the subgrade soils at the site would be identified today as a highly plastic clay (CH) probably exhibiting large volume change if climatic conditions were such that large variations in subgrade moisture content were to be expected.

Wooltorton describes in detail damage to numerous buildings in the Mandalay District of Burma caused by vertical and horizontal soil movements. The climate of the Mandalay District is characterized by high rainfall, but it also has an extended dry season. Seasonal moisture variations were measured at depths of 3.5 m, and cracks in the soil were measured at even greater depths. About 100 damaged buildings were examined in detail. The conclusion drawn was that the damage to the structures was due not to inferior workmanship or low bearing value of the soil as was presumed at the time, but to seasonal soil movement. Computations are presented showing that swelling pressures on the order of 1.0 ton/ft²...
developed against foundation elements in which movements were restrained. Wooltorton recommended extending foundations to depths at which moisture fluctuations are minimal by using drainage and impervious aprons to control the moisture regime in the vicinity of buildings, designing the portion of percent volume swell at a load of 1 psi—the clay layer thickness and surcharge loadings were based on the universal family of swelling curves presented in the 1956 paper.

The international soil mechanics community continued to show an interest in the problem of expansive soils. The Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering held in Switzerland in 1953 contains eight papers by authors from seven countries dealing with the measurement of building movement, laboratory measurement of swelling pressure and soil suction, field measurement of the distribution of soil moisture, and the influence of vegetation.

In 1953 extensive research was initiated by Zeitlen in Israel where serious problems with the performance of building foundations, retaining walls, pipelines, and pavements had been identified as associated with expansive clays. The group in Israel presented a classification of expansive clays based on the percent swell of air-dried undisturbed samples to determine quantitative uplift values for anticipated load conditions. Holtz and Gibbs drew attention to the importance of the sequence of loading and saturation and suggested duplicating as closely as possible in the laboratory anticipated field conditions. They also presented interesting results showing that, although index tests can be useful in identifying potentially troublesome soils, the actual degree of swelling or swelling pressure developed depends on the initial conditions of moisture content and dry density before wetting.

McDowell (6) presented in considerable detail his method of computing "potential vertical rise" on the basis of an assumed fixed relationship of one-third between linear swell and volumetric swell and on the basis of a family of universal curves for the relation between volumetric swell and load upon saturation (i.e., free access to water). The curves are identified by their percent volume change at zero load and cover the complete range of vertical load from zero load to the load required for no volume change (i.e., swelling pressure). The curves are the result of measuring percent swell on the emergence of individual specimens tested at various loads in a rigid ring consolidometer.

For this type of test, the volume change is equal to the change in height. McDowell, however, assumed that in the field, restraint conditions are such that the vertical rise would be only one-third the volume change measured for full restraint in the laboratory. This overly optimistic assumption is no doubt caused by the implied assumption of zero suction over the entire depth of the swelling clay profile. McDowell presented data that enabled the estimation of expansion characteristics and initial moisture conditions from the plasticity index. However, the importance of testing undisturbed samples of foundation soils at some convenient surcharge in moisture conditions from the plasticity index. McDowell presented data that enabled the estimation of expansion characteristics and initial moisture conditions from the plasticity index. McDowell presented data that enabled the estimation of expansion characteristics and initial moisture conditions from the plasticity index. McDowell presented data that enabled the estimation of expansion characteristics and initial moisture conditions from the plasticity index. McDowell presented data that enabled the estimation of expansion characteristics and initial moisture conditions from the plasticity index.

IDENTIFICATION OF THE PROBLEM

The existence of a combination of all or some of the following factors usually gives rise to a swelling clay problem:

1. A soil type that exhibits considerable volume change on changes in moisture content;
2. Climatic conditions, extended wet and dry seasons;
3. Possible changes in moisture content, due to changes in micro-climatic conditions, man-made, or due to vegetation; and
4. Structures (buildings, roads, etc.) sensitive to differential movement.

The authors have found that simple index tests usually have been adequate to identify soil types that are potentially troublesome as indicated in the following table:

<table>
<thead>
<tr>
<th>Plasticity index</th>
<th>Shrinkage limit</th>
<th>Free swell</th>
<th>Usually No Problems</th>
<th>Almost Always Problems</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;20</td>
<td>&gt;10</td>
<td>&gt;50</td>
<td>No</td>
<td>32</td>
</tr>
<tr>
<td>&lt;10</td>
<td>&lt;10</td>
<td>&lt;50</td>
<td>Usually No problems</td>
<td>10</td>
</tr>
</tbody>
</table>

The remaining factors are location and job dependent and require on-site inspection by experienced geotechnical engineers. Much useful information can be gained from examining test pits and from observing the depth of cracking and slickensides. Moisture content profiles as related to the plastic limit, preferable for both wet and dry seasons, can be an indication of the depth of seasonal moisture variations.

The single most important factor with respect to buildings is an adequate depth of foundation (remember the importance of the plastic limit and stress distribution). Though this is closely related to the depth of moisture variation (seasonal or man-made) it is more
fundamentally related to the swelling pressure profile with depth (7,8). All pertinent information with respect to successful foundation experience should be examined in the published literature and at the site.

LABORATORY SWELLING CURVES AND EMPIRICAL CORRELATIONS

The most commonly used laboratory test method for quantifying the swelling behavior of an expansive clay is to study the volume change upon wetting in a one-dimensional consolidation apparatus. Test results are usually presented on arithmetic scales as percent volume change versus vertical applied stress. The test method has been standardized (ASTM D 3877-80) allowing for different sequences of loading, wetting, and expansion control of volume change. The test results depend on the stress path followed and on the details of the test procedure. The soil water suction is presumed to be zero, at equilibrium. Considering how difficult it is to saturate clay samples, even using a back-pressure, it is more probable that even at equilibrium there is a small pore-water tension associated with consolidometer type testing.

The complete curve of swelling, ΔV/V versus applied pressure, is characterized by its end points and the shape of the curve between these two end points. The end points are:

1. $S_0$, the ratio in percent of ΔV/V for zero applied pressure. $S_0$ is sometimes defined as ΔV/V for an applied pressure of 1.0 psi (i.e., atmospheric pressure divided by 14.2); and

2. The applied pressure required to prevent volume expansion known as the swelling pressure ($P_0$).

Even for well-funded projects there are objective difficulties associated with sampling, laboratory testing, and estimating final design equilibrium conditions. Preliminary designs are frequently prepared based on a knowledge of the soil profile and the results of index tests such as Atterberg limits and in situ moisture and density only. Laboratory swelling studies on undisturbed samples are sometimes performed only in the final design stages, if at all. It is therefore not surprising that numerous efforts have been made to attempt to find useful correlations between index properties of swelling soils and their swelling characteristics (9,10).

Various correlations have been suggested for predicting the swelling pressure for zero movement ($P_0$) or percent swell for zero load ($S_0$), as measured in a rigid ring consolidometer, from a knowledge of the liquid limit of the soil, its initial moisture content, and dry density (11,12). A generalized form of the equations may be written as follows:

$$\log\left(\frac{P_0}{P_a}\right) = a_0 + a_L \cdot (\text{LL}) + a_a \cdot \left(\frac{\gamma_d}{\gamma_w}\right) + b_0 \cdot \left(\frac{\gamma_a}{\gamma_w}\right)$$

$$\log(S_0) = b_0 + b_L \cdot (\text{LL}) + b_a \cdot \left(\frac{\gamma_d}{\gamma_w}\right) + b_0 \cdot \left(\frac{\gamma_a}{\gamma_w}\right)$$

where

- $P_0$ = swelling pressure,
- $P_a$ = atmospheric pressure,
- $S_0$ = percent swell for zero load (%),
- LL = liquid limit (%),
- $\gamma_d$ = dry density of soil,
- $\gamma_a$ = unit weight of water, and
- $\gamma_w$ = in situ moisture content (%).

The coefficients "a..." and "b..." are determined by using techniques of multiple regression.

This was once a tedious job but with increased access to computers, statistical analysis is now easily performed. The technical literature is being flooded with new expressions having varying degrees of reliability as described by their correlation coefficients. These expressions are usually adequate for a particular set of data but frequently do rather poorly for predicting swelling behavior for another data set.

If heave predictions must be developed for a particular site and no test data exist, then there is no alternative but to use one of the published correlations considered most pertinent. However, if even a limited amount of test data are available then it is a great improvement if the coefficients are adjusted to match the available data. Now examine the preceding expression for estimating the swelling pressure and observe how the coefficients might be developed from a limited amount of data.

By illustration the data taken from the paper by Komornik and David will be used. The statistical analysis of all their data gave the following results (11):

$$a_0 = -1.868$$

$$a_L = 0.0208$$

$$a_a = 0.665$$

$$a_w = -0.0269$$

First examine the influence of initial density and moisture content on the swelling pressure for a soil of a given liquid limit. Rewriting Equation 1:

$$\log\left(\frac{P_0}{P_a}\right) = [a_0 + a_L \cdot (\text{LL}) + a_a \cdot \left(\frac{\gamma_d}{\gamma_w}\right)] - [b_0 + b_a \cdot \left(\frac{\gamma_a}{\gamma_w}\right) \cdot \left(\frac{\gamma_d}{\gamma_w}\right)^{-1}]$$

It can be observed from Equation 3 that contours of equal swelling pressure drawn with moisture content-dry density coordinates should be parallel straight lines with a slope of $-a_a/a_0$. In the absence of sufficient data, the value of $-a_a/a_0 = 0.040$ (obtained from the Komornik data) will be used (see Figure 1).

Rewriting Equation 1 again:

$$\log\left(\frac{P_0}{P_a}\right) = [a_0 + a_L \cdot (\text{LL}) + a_a \cdot \left(\frac{\gamma_d}{\gamma_w}\right)] + \left[a_0/a_\text{LL} \cdot (\text{LL}) \right]$$

If test results are plotted for the log of the swelling pressure versus $\left(\frac{\gamma_d}{\gamma_w}\right) + \left(\frac{\gamma_a}{\gamma_w}\right)$ using the value of $a_\text{LL}$ from the above, parallel straight lines may be fitted through the data points for each liquid limit to give the value for $a_\text{LL}$. The intercept of each line on the zero axis will give the value of $a_0$ for soils at all liquid limits (see Figure 2).

The use of Equation 1 implies that the values for $a_\text{LL}$ and $a_0$ will be the same for soils having different liquid limits. If the data that are available for a particular site tend to support this assumption then the final step can begin by examining other data sets with different liquid limits and [$a_\text{LL}$ + $a_0$] versus $\text{LL}$ can be plotted. The slope of the line will give the value of $a_0$ and the zero intercept will give the value of $a_\text{LL}$ (see Figure 3).

The percent swell at zero load ($S_0$) was used by McDowell (6) to characterize a family of swelling curves. Examination of these curves shows the following relationships:

$$S_0 (%) = 6.8 \left(\frac{P_0}{P_a}\right)$$

The test results are for individual specimens tested in rigid ring consolidometers.
An examination of the data presented by others (11,12) for the measured rebound swell, also measured in rigid ring consolidometers at 1.0 psi load (after performing a swelling pressure test), shows the following approximate relationship:

$$S_0(\%) = 3.0(P_{SP}/P_0)$$  (6)

This result is reasonable because the 1 psi seating load and the measurement of the percent swell after first performing a swelling pressure test would reduce the expected value of $S_0$ for a particular value of $P_0$.

Analysis of data presented by Holtz (5) shows that the ratio of $S_0$ for 1.0 psi to $P_{SP}/P_0$ for a particular soil depended on the degree of saturation and was equal to 3.5 for a degree of saturation equal to 85 percent and increased to 7.0 for a degree of saturation of 70 percent.

Equation 5 can be written in more general terms as follows:

$$S_0(\%) = S_R \cdot (P_0/P_a)$$  (7)

In the preceding paragraphs methods of predicting values of the percent swell at zero load ($S_0$) and the pressure required for zero swell, that is, swelling pressure ($P_0$) have been examined. Now it is time to examine the shape of the curve (of percent swell versus applied pressure) joining these two endpoints. On the basis of an examination of numerous laboratory swelling tests, McKeen (13) has suggested that there is a unique shape to all swelling curves if normalized values of swelling ($S_p/S_0$) are plotted versus normalized values of applied pressure ($P_p/P_0$), where $S_p$ is the percent swell for an applied stress of $P_p$. Following one of the more commonly used formulations relating volume change to applied vertical stress in one-dimensional consolidation:

$$S_p/S_0 = 6l/l + \epsilon_0 100/S_0 = -CR \cdot \log[(P_p/P_0)^n]$$  (8)

where $\epsilon_l$ and $\epsilon_0$ are the change in void ratio and initial void ratio, respectively. The test data published by McKeen are consistent with a constant value of $CR = 0.45$, where $S_0$ is the percent swell for a vertical stress of 1.0 psi and all swell values ($S_0$ and $S_p$) are for rebound on a single specimen after performing a swelling pressure test. (Note that Equation 8 is valid only for values of $P_p/P_0 > 0.01$ and $P_p > 1.0$ psi.)

The master swelling curves published by McDowell (15) are consistent with a higher value of $CR = 0.54$ where $S_0$ is the percent swell under zero load and all percent swell values ($S_0$ and $S_p$) are for individual test specimens at a constant vertical stress. (Note that Equation 8 is a reasonable description of McDowell's master curves.)

A complete family of swelling curves can then be described as follows:

$$S_p/S_0 = -CR \cdot \log[(P_p/P_0)^n]$$  (9)

$$S_0 = S_p \cdot P_0/P_a$$  (10)

$$S_p = -(C_R \cdot S_R) \cdot (P_{SP}/P_0) \cdot \log[(P_p/P_0)^n]$$  (11)

Equation 11 with $C_R = 0.54$ and $S_R = 6.82$ is a reasonably accurate description of McDowell's family of swelling curves and can be used for identifying a particular swelling curve for any known point on the swelling curve $S_p$, $P_p$ using the computational routine shown in Figure 4.

**COMPUTING HEAVE**

The prediction of heave for a pavement, or a building foundation on an expansive clay subgrade, re-
requires a knowledge of the percent expansion expected for each depth when allowed access to water (while suitably surcharged) under a controlled suction equal to that expected at equilibrium conditions. The heave is then calculated by integrating the percent expansion over the affected depth. The equilibrium suction varies with climatic conditions, the depth of the water table, the imperviousness of the surface, and the efficiency of the surface and subsurface drainage. The authors have found that these suction values can range from as low as zero, for conditions of very poor drainage, to as high as 1.0 kb/ cm² or more for impervious surfacing and very good drainage and a very deep water table (7,14). Equilibrium suction values are sometimes assumed to be equal to the distance above the water table.

The reader interested in a treatment of the problem in all its complexity is referred to the work of Fredlund (15,16). Laboratory testing is usually done in consolidometers and should be performed on good-quality undisturbed samples, taken at a time such that their in situ moisture and density represent conditions expected to exist at the time of construction. To simplify the laboratory testing procedure, the effect of expected equilibrium suction can be accounted for by increasing the applied vertical pressure above the overburden or foundation pressure. The laboratory test can then be conducted by inundating the sample.

The basic requirement is, therefore, laboratory swelling tests on undisturbed samples under various vertical pressures. These test results plus past experience used to estimate final heave and also to quantify the beneficial effects of undercutting and replacement with nonswelling material. Presented next is a computational routine the authors have used to compute heave (7,14). Since time, site, and budget limitations frequently do not allow for complete laboratory testing, the procedure described can be used if only the index properties (liquid limit, in situ dry density, and moisture) are known.

The program presented is for a single homogeneous layer with or without foundation loadings. This program can be easily modified, however, so that it can be used for a layered soil profile.

Figure 5 shows the routine for computing $P_0$ and $S_0$ from index tests if these values are not known. Shown in Figure 6 are (a) the definition of stress conditions that can be specified for the top and bottom of the layer being computed and (b) a description of the method for computing the thickness ($z_0$) of the critical layer undergoing swelling.

![Figure 4](image4.png)

**FIGURE 4** Compute $S_0$, $P_0$; for known $S_p$, $P_p$.

![Figure 5](image5.png)

**FIGURE 5** Compute $S_0$, $P_0$; from index tests.
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\[ S_{\text{avg}} = \frac{-0.4343 \cdot C_R \cdot S_0 \cdot (P_B - P_T)}{P_T} \cdot \frac{P_B \cdot \ln(P/P_0)\cdot d(P/P_0)}{P_T} \]

\[ = \frac{-0.4343 \cdot C_R \cdot S_0 \cdot (P_B - P_T)}{P_T} \cdot \frac{P_B \cdot \ln(P_B/P_0)}{P_T} \]

\[ = \frac{-0.4343 \cdot C_R \cdot S_0 \cdot (P_B - P_T)}{P_T} \cdot \frac{P_B \cdot \ln(P_B/P_0)}{P_T} \cdot \frac{P_B - P_T}{P_T} \]

HEAVE of LAYER = \( f \cdot \frac{S_{\text{avg}}}{100} + z_0 \)

Note:

1) C. McDowell assumed Restraint Factor \( f = 1/3 \)

The authors assume \( f = 1.0 \) when the % swell data is for rigid ring consolidometers.

2) \( P_T \) min = 1.0 psi

Maximum values for \( P_T \) and \( P_B = P_0 \).

FIGURE 6 Layer stresses \((P_T, P_B)\) and thickness \((z_0)\).
blame has to be assigned, the first step that has to be taken is to establish reference points on the structure and to monitor their elevation with time with respect to a reliable bench mark. It is preferable for such fixed points and an associated initial survey to be established at the time of construction. It is the experience of the authors that the discussion of the foregoing items for the owners is one of the best ways of drawing attention to what reasonably might be expected to happen within the life-span of the structure.

In recent years, there has been increasing awareness of the role of differential heave of expansive clay subgrades in the development of pavement roughness (18,19). The authors’ experience has been that pavement roughness on expansive clay subgrades sometimes starts to become troublesome as early as 4 to 5 years after pavement construction. When deciding on a pavement overlay it is important to know if pavement roughness has developed as a result of factors associated with traffic loadings or as a result of differential subgrade heaving. A slightly thicker pavement overlay may be very effective in increasing the time between pavement overlays in the case of traffic-induced roughness. Extra overlay thickness is only of limited usefulness in reducing the length of time before pavement roughness due to differential subgrade heaving becomes a problem.

Proper identification of the subgrade soils is of the utmost importance. It is also important to recognize that despite all the measures the engineer may take to design and construct the subgrade, pavement, shoulders, and drainage in order to minimize differential horizontal and vertical subgrade movements, pavement distress may occur due to moisture changes before it occurs due to traffic loading. The monitoring of pavement elevations should therefore be included in the follow-up procedures of every pavement constructed on a swelling clay subgrade.

CONCLUDING REMARKS

The methods the authors have used have a fair measure of success for identifying and quantifying the problems associated with roads and buildings on expansive clays have been described. These methods are based on the results of laboratory swelling tests or on swelling parameters as predicted from simple index tests after calibration, if possible. The computed results for heave are strongly influenced by the assumed effect of in situ cracking and lateral restraint on the relation between volume change and vertical heave and by the assumed final equivalent suction values. At the present time guidance can be found only by a study of case histories. To further complicate matters, satisfactory performance of a highway or building in terms of acceptable distortions or cracking is not judged only by the engineer; the final judgment is in the hands of the public and the courts.

REFERENCES


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