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/ft^2

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 structure to resist unbalanced forces due to subgrade soil swelling pressures, and not using contact pressures (from dead load and normal live load) of less than 1.0 ton/ft^2. Wooltorton drew attention to the fact that similar soils and associated foundation problems existed in India, South Africa, and Sudan (4).

Twenty years later, two papers were published that dealt with the identification of swelling clays, laboratory testing, and the prediction of surface heave as a function of restraining load. These two papers, one by Holtz and Gibbs (5) published in the Transactions of the American Society of Civil Engineering (ASCE) and the other by McDowell (6) published in the Proceedings of the 35th Annual Meeting of the Highway Research Board, in addition to providing practical guidance on design and construction, strongly influenced subsequent research on the subject of swelling clays.

Holtz and Gibbs presented a classification of expansive clays based on the percent swell of air-dried undisturbed samples when allowed to swell upon saturation (under a vertical pressure of 1.0 psi) in a rigid ring consolidometer. In addition to plasticity index and shrinkage limit, they suggested the use of the "free-swell test" to identify possible troublesome soils. The free-swell value is the percent increase in volume when 10 cc of air-dry soil passing the Number 40 sieve is slowly poured into a 100 cc graduate filled with distilled water. They suggested the use of laboratory tests in the rigid ring consolidometer on compacted or 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 relationship 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 submergence 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 offset by his 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, he recommended testing of undisturbed samples of foundation soils at some convenient surcharge in order to identify the swell versus load curve pertinent to the problem at hand. The potential vertical rise is then computed considering the thickness of the clay layer, gravity loading, and surcharge loadings due to any structure. McDowell developed a slide rule for computing potential vertical rise as a function 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 hosted a First International Colloquium on expansive clays in Haifa, in 1961. It was attended by a small group of researchers from all over the world (India, the United States, Spain, Canada, Australia, South Africa, and the United Kingdom). Five full-fledged international conferences have since been held at Texas A&M University in 1965 and 1969; Haifa, Israel, in 1973; Denver, Colorado, in 1980; and Adelaide, Australia, in 1985.

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 have generally been adequate to identify soil types that are potentially troublesome as indicated in the following table:

	Usually No Problems	Almost Always Problematic
Plasticity index	<20	>32
Shrinkage limit	>13	<10
Free swell (5)	<50	>100

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 (remembering the importance of isolation of grade beams). 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, $\Delta V/V$ 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_o, the ratio in percent of $\Delta V/V$ for zero applied pressure. S_o is sometimes defined as $\Delta V/V$ 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_o).

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_o) or percent swell for zero load (S_o), as measured in a rigid 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:

$$\text{Log}(P_o/P_a) = a_o + a_L \cdot (LL) + a_d \cdot (\gamma_d/\gamma_\omega) + a_\omega \cdot (\omega_o) \quad (1)$$

$$\text{Log}(S_o) = b_o + b_L \cdot (LL) + b_d \cdot (\gamma_d/\gamma_\omega) + b_\omega \cdot (\omega_o) \quad (2)$$

where

P_o = swelling pressure,
P_a = atmospheric pressure,
S_o = percent swell for zero load (%),
LL = liquid limit (%),
γ_d = dry density of soil,
γ_ω = unit weight of water, and
ω_o = 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):

$a_o = -1.868$

$a_L = 0.0208$

$a_d = 0.665$

$a_\omega = -0.0269$

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

$$(\gamma_d/\gamma_\omega) = [\text{Log}(P_o/P_a) - a_L \cdot (LL) - a_o/a_d] - [a_\omega/a_d] \cdot (\omega_o) \quad (3)$$

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 $(-a_\omega/a_d)$. In the absence of sufficient data, the value of $-a_\omega/a_d = 0.040$ (obtained from the Komornik data) will be used (see Figure 1).

Rewriting Equation 1 again

$$\text{Log}(P_o/P_a) = [a_o + a_L \cdot LL] + a_d \cdot \{\gamma_d/\gamma_\omega + [a_\omega/a_d] \cdot (\omega_o)\} \quad (4)$$

If test results are plotted for the log of the swelling pressure versus $\{\gamma_d/\gamma_\omega + [a_\omega/a_d] \cdot (\omega_o)\}$ using the value of a_ω/a_d determined above, parallel straight lines may be fitted through the data points for each liquid limit to give the value for a_d. The intercept of each line on the zero axis will give the value for $[a_o + a_L \cdot LL]$ (see Figure 2).

The use of Equation 1 implies that the values for a_ω/a_d and for a_d 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 $[a_o + a_L \cdot (LL)]$ versus LL can be plotted. The slope of the line will give the value of a_L and the zero intercept will give the value of a_o (see Figure 3).

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

$$S_o(\%) = 6.8(P_o/P_a) \quad (5)$$

The test results are for individual specimens tested in rigid ring consolidometers.

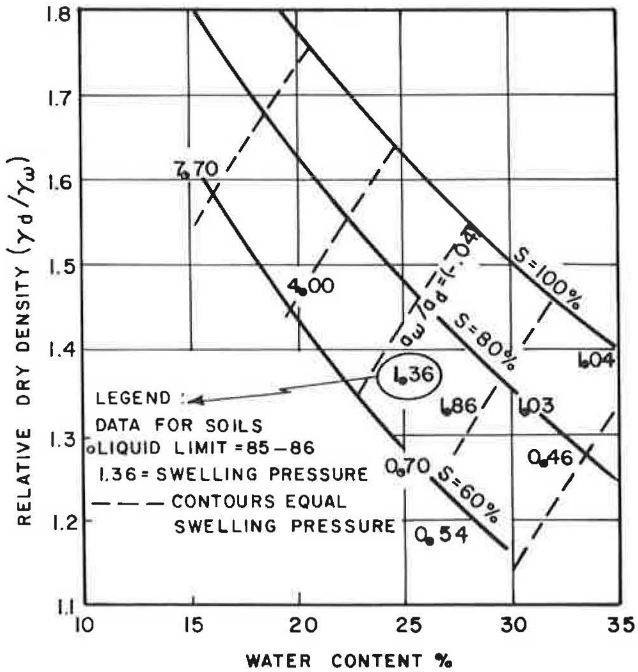


FIGURE 1 Determination of (a_ω/a_d) for swelling pressure correlations.

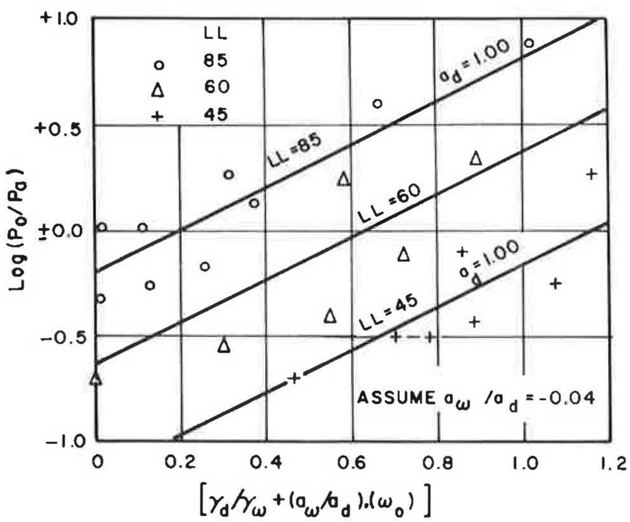


FIGURE 2 Determination of (a_d) for swelling pressure correlations.

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:

$$S_o(\%) = 3.0(P_o/P_a) \tag{6}$$

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_o for a particular value of P_o.

Analysis of data presented by Holtz (5) shows that the ratio of S_o for 1.0 psi to P_o/P_a 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:

$$S_o(\%) = S_R \cdot (P_o/P_a) \tag{7}$$

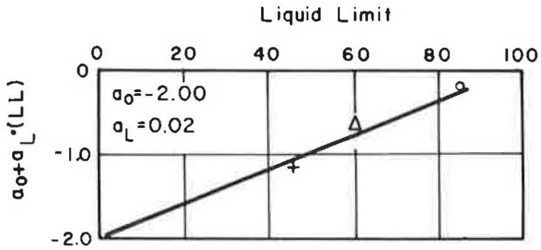


FIGURE 3 Determination of (a_o and a_{LL}) for swelling pressure correlations.

In the preceding paragraphs methods of predicting values of the percent swell at zero load (S_o) and the pressure required for zero swell, that is, swelling pressure (P_o) have been examined. Now it is time to examine the shape of the curve (of percent swell versus applied pressure) joining these two end points. 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_o) are plotted versus normalized values of applied pressure (P_p/P_o), 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:

$$S_p/S_o = \Delta\ell/1 + \ell_o \cdot 100/S_o = -CR \cdot \text{Log}[(P_p/P_o)] \tag{8}$$

where $\Delta\ell$ and ℓ_o are the change in void ratio and initial void ratio, respectively. The test data published by McKeen are consistent with a constant value of $C_R = 0.45$, where S_o is the percent swell for a vertical stress of 1.0 psi and all swell values (S_o 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 $P_p/P_o > 0.01$ and $P_p > 1.0$ psi.)

The master swelling curves published by McDowell (6) are consistent with a higher value of $C_R = 0.54$ where S_o is the percent swell under zero load and all percent swell values (S_o 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:

$$S_p/S_o = -C_R \cdot \text{Log}[P_p/P_o] \tag{9}$$

$$S_o = S_R \cdot P_o/P_A \tag{10}$$

$$S_p = -(C_R \cdot S_R) \cdot (P_o/P_A) \cdot \text{Log}(P_p/P_o) \tag{11}$$

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-

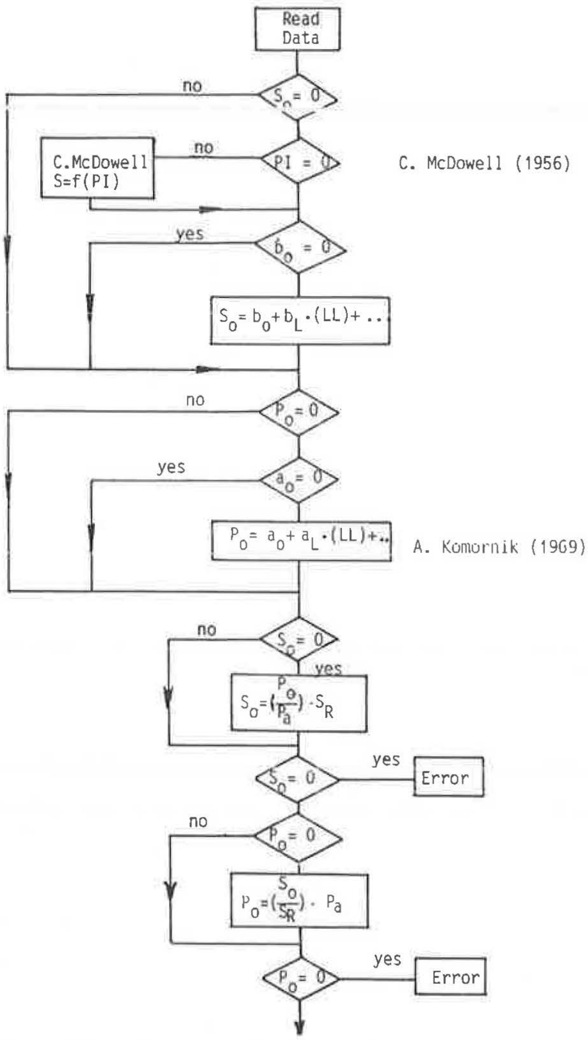


FIGURE 4 Compute S_o, P_o; for known S_p, P_p.

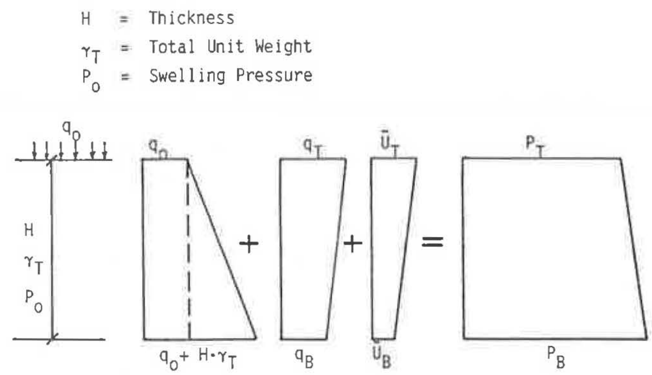
quires a knowledge of the percent expansion expected for each depth when allowed access to water (while suitably surcharged) under a controlled suction equal to that expected at equilibrium conditions. The heave is then calculated by integrating the percent expansion over the affected depth. The equilibrium 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/cm^2 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 can then be 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_o and S_o 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_o) of the critical layer undergoing swelling.



q_o = Uniform surcharge stress
q_T, q_B = Applied foundation stresses (top & bottom)
$\bar{U}_T, \bar{U}_B$ = Equivalent suction stresses (top & bottom)
[opposite in sign & = or < actual estimated suction]

$P_T = q_o + q_T + \bar{U}_T$: $P_B = q_o + H \cdot \gamma_T + q_B + \bar{U}_B$

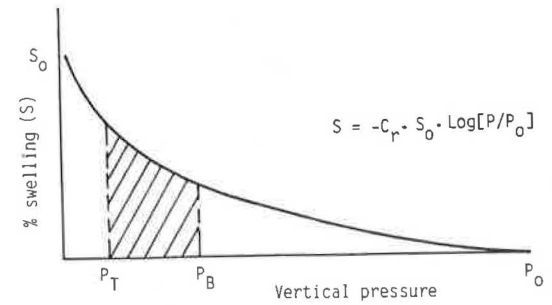
COMPUTE THICKNESS OF CRITICAL LAYER (z_o)

Case I $P_T = P_B$
- if $P_T > P_o$ — z_o = zero
- if $P_T < P_o$ — $z_o = H$

Case II $P_B > P_T$
- if $P_T \geq P_o$ — z_o = zero
- if $P_B \leq P_o$ — $z_o = H$
- if $P_T < P_o < P_B$ — $z_o = \frac{P_o - P_T}{P_B - P_T} \cdot H$, $P_B = P_o$

Case III $P_T > P_B$
- if $P_B \geq P_o$ — z_o = zero
- if $P_T \leq P_o$ — $z_o = H$
- if $P_B < P_o < P_T$ — $z_o = \frac{P_o - P_B}{P_T - P_B} \cdot H$, $P_T = P_o$

FIGURE 5 Compute S_o, P_o; from index tests.



$$S_{avg} = \frac{-.4343 \cdot C_r \cdot S_o}{(P_B - P_T)/P_o} \int_{P_T}^{P_B} \text{Ln}(P/P_o)\, d(P/P_o)$$

$$= \frac{-.4343 \cdot C_R \cdot S_o}{(P_B - P_T)/P_o} \cdot [P/P_o \cdot (\text{Ln}(P/P_o) - 1)]_{P_T/P_o}^{P_B/P_o}$$

$$= \frac{-.4343 \cdot C_R \cdot S_o}{(P_B - P_T)/P_o} \cdot \{\frac{P_B}{P_o}[\text{Ln}(\frac{P_B}{P_o}) - 1] - \frac{P_T}{P_o}[\text{Ln}(\frac{P_T}{P_o}) - 1]\}$$

$$\text{HEAVE of LAYER} = f \cdot \frac{S_{avg}}{100} \cdot z_o$$

Note:

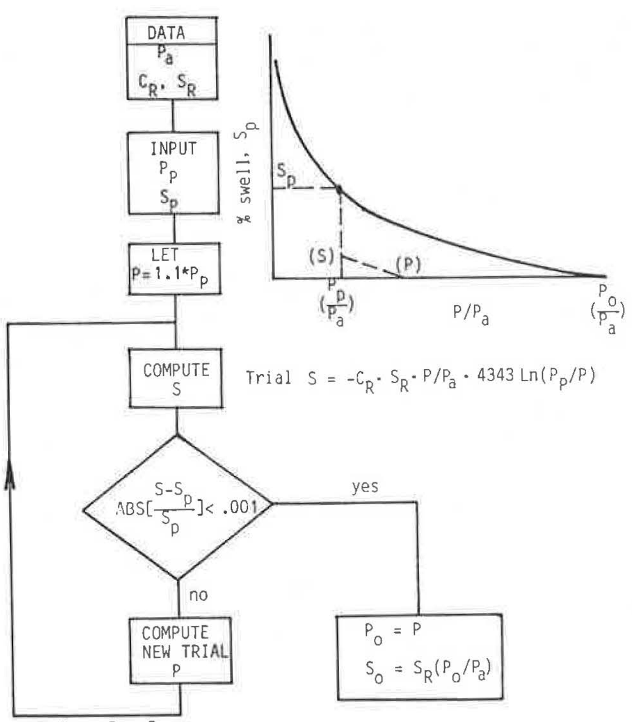
1) C. McDowell assumed Restraint Factor f = 1/3

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 $P_B = P_o$.

FIGURE 6 Layer stresses (P_T, P_B) and thickness (z_o).



$$\frac{S}{S_p} = \frac{P - P_p}{P_o - P_p}$$

New Trial $P = (S \cdot P_p + S_p \cdot P - S_p \cdot P_p)/S$

FIGURE 7 Compute heave of layer.

Figure 7 shows the equations used for computing the average percent swell and, hence, for the total heave of the layer. Figure 8 shows a data sheet for the program shown in Figure 9.

POST-CONSTRUCTION OBSERVATIONS

The performance of structures on expansive clay subgrades can be adversely affected by an unusually dry season or by a sequence of unusually dry seasons (or wet seasons) such as might occur once in 25 yr and therefore possibly outside the time span of reported experience for a particular area. Man-made environment, trees, sewer lines, leaky faucets, and flower beds in the vicinity of structures have all been experienced as sources of changes in subgrade moisture content and associated movements or pressures (17).

It is a reasonable assumption that sometime within the lifespan of every structure founded on expansive clays movements will occur that will be detrimental to the structure. It is surprising how difficult it sometimes is to know whether such movements are heave or settlement. If repairs have to be initiated or, as is becoming increasingly common,

DATA

UNITS			
L$		METERS	LENGTH
F$		TONS	FORCE
GW	γ_w	1.0	WATER
PA	Pa	10.0	ATM. PRESS

		LAYER 1	LAYER 2	LAYER 3	
QΦ	q_o	1.0			SURCHARGE TOP OF LAYER
H	H	100			LAYER THICKNESS
F	f	1.0			RESTRAINT FACTOR 1/3 TO 1.0
DT	γ_T/γ_w	2.00			RELATIVE TOTAL UNIT WEIGHT
QT	q_T	0			FOUNDATION STRESS
QB	q_B	0			TOP AND BOTTOM
UT	$\bar{U}_T$	10.0			EQUIVALENT SUCTION
UB	$\bar{U}_B$	10.0			STRESS, TOP AND BOTTOM
PI	PI	0			C. McDOWELL (1956) PLASTICITY INDEX
CM$		"A"			CODE - INITIAL MOISTURE "O" OPT : "A" AVG : "W" WORST
LL	LL	85			LIQUID LIMIT (%)
DD	γ_d/γ_w	1.50			RELATIVE DRY DENSITY
WΦ	w_o	22			INITIAL MOISTURE CONTENT (%)
SΦ	S_o	0			% SWELL - ZERO LOAD
PΦ	P_o	0			SWELLING PRESSURE
SR	S_R	6.8	$(\frac{S_o}{P_o}) \times P_a$		S_R = 6.8 C. McDOWELL (1956); S_R = 3.0 REBOUND, KOMORNIK (1969)
CR	C_R	0.54	$\frac{S}{S_o} = -C_R \cdot \text{LOG}(P/P_o)$		C_R = 0.54 C. McDOWELL-DATA; C_R = 0.45 McKEEN - DATA

AΦ	a_o	-2.00	(LOG) $P_o = a_o$	BΦ	b_o	0	(LOG) $S_o = b_o$
AL	a_L	0.02	$+ a_L \cdot (LL)$	BL	b_L	0	$+ b_L \cdot (LL)$
AD	a_d	1.00	$+ a_d \cdot (\ )$	BD	b_d	0	$+ b_d \cdot (\gamma_d/\gamma_w)$
AW	a_w	-0.04	$+ a_w \cdot (\ )$	BW	b_w	0	$+ b_w \cdot (w_o)$

RESULTS

ZΦ	z_o	4.95	(m)	
S1	AVG. % SWELL	0.96	(%)	
PΦ	P_o	20.9	(TON/m²)	
DH	HEAVE	.047	(m)	

FIGURE 8 Data sheet.

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]LIST

100  REM  HEAVE CALCULATIONS
105  REM  SINGLE LAYER
110  REM   G.WISEMAN  14/AUG/1984

120  REM ANY CONSISTENT UNITS
130  REM  UNIT WEIGHT RELATIVE TO
      WATER
500  GOSUB 5000: REM  DATA
1000  REM   COMPUTE % SWELL (S0)

1001  REM  COMP SWELL PRESS (P0)
1005  IF S0 <  > 0 THEN  GOTO 108
     0
1010  IF PI = 0 THEN  GOTO 1060
1020  IF CM$ = "O" THEN  LET S0 =
     .227 * (PI - 15)
1030  IF CM$ = "A" THEN  LET S0 =
     .289 * (PI - 11)
1040  IF CM$ = "W" THEN  LET S0 =
     .335 * (PI - 4)
1045 S0 = 1.25 * S0
1060  IF B0 = 0 THEN  GOTO 1090
1070 S0 = B0 + BL * LL + BD * DD +
     BW * W0
1075 S0 = 10 ^ S0
1080  IF P0 <  > 0 THEN  GOTO 111
     0
1090  IF A0 = 0 THEN  GOTO 1110
1100 P0 = (A0 + AL * LL + AD * DD
      + AW * W0)
1105 P0 = PA * 10 ^ P0
1110  IF S0 = 0 THEN  LET S0 = P0
      / PA * SR
1120  IF S0 = 0 THEN  PRINT "DATA
      ERROR": STOP
1200  IF P0 = 0 THEN  LET P0 = S0
      / SR * PA
1210  IF P0 = 0 THEN  PRINT "DATA
      ERROR": STOP
1220  IF S0 < 0 THEN S0 = 0
1230  IF P0 < 0 THEN P0 = 0
5100  REM "O"OPT,"A"AVG,"W"WORST
5110  DATA   0,"A"
5120  READ LL,DD,W0
5130  DATA 85,1.50,22
5140  READ S0,P0
5150  DATA   0 ,0
5160  READ SR,CR
5170  DATA   6.8,0.54
5180  READ A0,AL,AD,AW
5190  DATA -2.00,0.02,1.00,-0.04
5200  READ B0,BL,BD,BW
5210  DATA  0,0,0,0
5300  RETURN
]RUN
UNITS =METERS,TONS
LAYER THICK.  =100 METERS
ACTIVE THICK. =4.946
SWELLING PRESS=20.893
% SWELL (S0)  =14.207
AVG % SWELL   =.955
HEAVE=  .047 METERS
1500  REM  COMPUTE CRITICAL DEPTH
      (Z0)
1510 PT = Q0 + QT + UT
1520 PB = Q0 + QB + UB + H * DT *
     GW
1530  IF PT <  > PB THEN  GOTO 15
     60
1540  IF PT >  = P0 THEN Z0 = 0
1550  IF PT < P0 THEN Z0 = H
1555  GOTO 2000
1560  IF PT > PB THEN  GOTO 1610
1570  IF PT >  = P0 THEN Z0 = 0
1580  IF PB <  = P0 THEN Z0 = H
1590  IF PT < P0 AND P0 < PB THEN
     Z0 = (P0 - PT) / (PB - PT) *
     H:PB = P0
1600  GOTO 2000
1610  IF PB >  = P0 THEN Z0 = 0
1620  IF PT <  = P0 THEN Z0 = H
1630  IF PB < P0 AND P0 < PT THEN
     Z0 = (P0 - PB) / (PT - PB) *
     H:PT = P0
2000  REM  COMPUTE HEAVE
2003  IF  ABS ((PB - PT) / PB) <
     0.001 THEN S1 =  - 0.4343 *
     CR * S0 *  LOG (PT / P0): GOTO
     2030
2005  IF PT < (PA / 14.2) THEN  LET
     PT = PA / 14.2
2010 S1 = PB / P0 * ( LOG (PB / P
     0) - 1) - PT / P0 * ( LOG (P
     T / P0) - 1)
2020 S1 =  - .4343 * CR * S0 / (P
     B - PT) * P0 * S1
2030 DH = F * S1 / 100 * Z0
2035  IF Z0 < H THEN  LET DH = F *
     S1 / 100 * Z0
3000  REM  PRINT RESULTS
3010  DEF  FN A(X) =  INT (1000 *
     X + .5) / 1000
3100  PRINT "UNITS =";L$;",";F$
3110  PRINT "LAYER THICK.  =";H;"
      ";L$
3120  PRINT "ACTIVE THICK. ="; FN
     A(Z0)
3130  PRINT "SWELLING PRESS="; FN
     A(P0)
3140  PRINT "% SWELL (S0)  ="; FN
     A(S0)
3150  PRINT "AVG % SWELL   ="; FN
     A(S1)
3160  PRINT "HEAVE=  "; FN A(DH);
     " ";L$
4900  END

5000  REM  DATA
5010  READ L$,F$
5020  DATA  "METERS","TONS"
5030  READ GW,PA,Q0
5040  DATA  1.0,10.0,1.0
5050  READ H,F,DT
5060  DATA   100,1.0,2.00
5070  READ QT,QB,UT,UB
5080  DATA  0,0,10.0,10.0
5090  READ PI,CM$: REM  MCDOWELL
```

FIGURE 9 Program listing.

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 item with the owners is one of the best ways of drawing attention to the limitations of available knowledge and 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 can 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 with 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.

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