tation Series 160, Final Rept., June 1976.
15. Soils Manual. Illinois Department of Transportation, Springfield, 1976.

Publication of this paper sponsored by Committee on Environmental Factors Except Frost.

Volume Changes in Compacted Clays and Shales on Saturation

R.A. ABEYESEKERA AND C.W. LOVELL

The midwestern United States has only a limited quantity of so-called swelling soils. However, substantial volume changes may occur in the compacted clay and shale embankments of the area as they become saturated in the service environment. Since these deformations are likely to be the greatest these embankments will ever experience, their prediction and control are of considerable practical interest. To aid in the accomplishment of this important engineering objective, the results of an extensive study of these volume changes for laboratory-compacted clays and shales are reported (the study is being extended to field-compacted clays). Saturation was accomplished under high back pressures, for either triaxial or consolidation samples, and confinement simulated various embankment positions. The volume changes that resulted could be either increases (swell) or decreases (settlement), depending on the level of confinement and the compaction variables. From such testing, statistically valid prediction equations were derived in terms of (a) the compacted density or void ratio, (b) the water content or degree of saturation, and (c) the confining pressure. These equations show how the volume changes on saturation can be controlled by appropriately altering the details of the compaction specification.

Compacted clays and shales are used in large quantities in the construction of earth embankments and other fills. Ensuring the stability of these structures against a slope failure is always of major concern, and in most cases it is the sole criterion taken into consideration in actual design, which is unfortunate.

The compressibility and swelling characteristics of compacted soils and shales, particularly over the long term, must also be taken into consideration in the design phase. These volume changes are dependent on the compaction variables--e.g., water content, dry density, method of compaction, and soil and/or shale type--and on the confining pressure under which the fill absorbs water. A soil or shale compacted to a high density and confined under a relatively low pressure will tend to swell on saturation. On the other hand, a low-density soil or shale under a relatively high confining pressure will tend to compress on saturation. Under certain conditions, a soil or shale may not change in volume at all on saturation.

Accordingly, it is clear that it is extremely difficult, if not impossible, to design an earth fill to avoid volume changes in the completed structure. The most one can do is to place the fill in such a condition that the cumulative effects of the differential volume changes are within "acceptable" limits. To achieve no volume change on saturation, one would have to vary the as-compacted condition of the soil or shale over the depth and width of the fill. Although this unconventional approach is theoretically possible, it is extremely difficult to achieve in practice.

This paper discusses the volume-change characteristics of two troublesome fill materials: a highly plastic clay from St. Croix, Indiana, and a medium-hard, nondurable shale from the New Providence formation in Indiana. Statistically derived prediction equations are given in terms of (a) compacted density or void ratio, (b) water content or degree of saturation, and (c) confining pressure for fully saturated samples. These equations show how volume changes on saturation can be estimated and used in the design phase to control the cumulative effects of soil or shale compression and swelling in an earth fill.

LITERATURE REVIEW

Swelling Potential

Various systems have been advanced to define the swelling potential of soils. These systems are based on intrinsic soil properties as well as the properties of soil in a compacted state.

Ladd and Lambe (1) suggested a rating system called potential volume change, which varied linearly with the percentage of heave under a surcharge of 9.5 kPa (200 lbf/ft^2), the plasticity index of the soil, the water content of the soil at 100 percent relative humidity, and the calculated percentage of volume change resulting from drying a saturated sample from the field moisture equivalent (ASTM D426-39) to the shrinkage limit.

Seed and others (2) defined swelling potential as "percentage swell of a laterally confined sample on soaking under [7 kPa] 1 psi surcharge after being compacted to maximum density at optimum moisture content in the standard AASHO compaction test." They found that the swelling potential is a function of the activity of the soil and the percentage of clay size, and they defined the activity of the soil as the rate of change of the plasticity index with clay content rather than the ratio of the plasticity index to the clay content (3).

Kassif and Baker (4) stated that, for a quantitative evaluation of the amount of heave under field conditions, the whole range of volume change under various surcharges is required. They therefore defined swelling potential as the integral of the swell-pressure curve for the range of surcharges representing field conditions.

Kassif and others (5, p. 218) and Krazynski (6) have drawn attention to the numerous and widely different methods that have been proposed and used for testing and classifying expensive soils.

Swelling Behavior

The swelling behavior of partially saturated soils has been studied extensively (1,4-23). The factors that influence swell magnitude include type and amount of clay minerals; as-compacted condition of the soil represented by its dry density and moisture content; the structure of the soil, which varies with the compaction method; chemical properties of the pore fluid; osmotic pressure; temperature; permeability of the soil; stress history; confining pressure; alternate cycles of wetting and drying; time interval between compaction and exposure to free water; time allowed for swelling; and soil suction.

The kaolin group of clay minerals, which have fixed lattice structures, exhibit only a small degree of hydration and swell. On the other hand, the montmorillonite group, which has expanding lattice structures, exhibits a high degree of hydration and swell.

Soils compacted dry of optimum tend to swell more than soils compacted wet of optimum. The soil in the former condition is considered to have a flocculated structure, whereas that in the latter condition is considered to have a dispersed (more oriented) structure. It has also been observed that swell increases with increasing as-compacted dry density and decreasing as-compacted water content.

The method of compaction has been found to have an influence on the structure of the soil and its swelling characteristics. Static compaction tends to produce a flocculated structure, whereas kneading compaction tends to produce a dispersed or more oriented structure. Seed and others (25) compared the amounts of swell for samples of a sandy clay compacted dry and wet of optimum by static and kneading compaction. They observed that samples compacted dry of optimum by the two methods of compaction have similar swelling characteristics but that, for samples compacted wet of optimum, statically compacted samples exhibited the greater swell associated with more flocculated structures.

It has been observed that the maximum past pressure to which the soil has been subjected has an effect on its swell under a reduced load. Seed and others (24) have indicated that the effect of the prestress is to reduce the amount of swell under the smaller load.

The time rate of swelling is influenced by the method of compaction. It has been observed that a sandy clay compacted by static action tends to swell at a faster rate than soils compacted by a kneading action (2).

Baker and Kassif (7) derived mathematical relations for the increase of swell pressure with time for partially saturated clays. They showed that the dissipation function is similar to that of the consolidation process. Their theory is in agreement with the typical S shape of the curve of swell pressure versus logarithm of time, which has been experimentally shown time and again.

Kassif and Shalom (15) found that the swell pressure and the change in suction on wetting of a montmorillonitic clay were approximately equal for samples compacted wet of optimum that were not allowed to change in volume. However, for samples compacted dry of optimum, the swell pressure was equal to only a fraction of the change in suction. Kassif and Shalom also found that the swelling pressure reached about 95 percent of its maximum value after a moisture intake of only one-third of that required to saturate the sample, for a wide range of densities.

Kassif and others (14) found that the initial suction at a constant dry density decreased at a decreasing rate with increasing compacted moisture content. The swell-pressure relation was hyperbolic at low suction changes and tended to become linear at high suction changes. The percentage swell increased with increasing suction change and, beyond a threshold suction change, the rate of change of swell with change in suction was independent of the initial water content of the clay.

DESCRIPTION OF MATERIALS

Two materials highly susceptible to volume change in compacted embankments were selected for study: St. Croix clay and New Providence shale.

St. Croix Clay

The St. Croix clay used in the studies was obtained from the IN-37 relocation project in Perry County, Indiana, and is described below:

Category	Value
Soil classification	
Unified	CH
AASHTO	A-7-6
Index properties (%)	
Liquid limit	53
Plastic limit	21
Plasticity index	32
Shrinkage limit	12
Specific gravity	2.80
Natural moisture content (%)	20
Clay-size fraction (<2 μm) (%)	44

St. Croix clay is medium gray-brown at its natural moisture content and is classified as a fat clay (CH) according to the Unified Classification System. The predominant clay mineral present was found to be kaolinite. Small traces of montmorillonite were also observed in the X-ray diffraction analyses.

New Providence Shale

The New Providence shale used in the studies was sampled from a road excavation on I-265 in Floyd County in south-central Indiana. The geological description of the shale reported by the Indiana State Highway Commission (ISHC) was as follows: era, Paleozoic; period, Carboniferous; epoch, Mississippian; series, Valmeyeran (Osage); group, Borden; formation, New Providence; age, 241 million to 261 million years.

The shale is of a light gray color, is medium hard, and has a "massive" category of fissility. It is classified as "soillike" according to the classification system of Deo (25). In terms of soil classification, it is a silty clay (CL). Its soil classification and properties are summarized below:

Category	Value
Soil classification	
Unified	CL
AASHTO	A-4-10
Index properties (%)	
Liquid limit	31
Plastic limit	21
Plasticity index	10
Shrinkage limit	2
Specific gravity	2.78
Natural moisture content (%)	6
Clay size fraction (<2 μm) (%)	21

The New Providence shale is composed of quartz, kaolinite, and illite. Small traces of chlorite and vermiculite may also be present, but montmorillonite is not present.

EXPERIMENTAL PROCEDURES

The California-type kneading compactor was used to compact the St. Croix clay and the New Providence shale because it simulates field compaction more closely than either impact or static compaction. The diameter of the compacted sample was 101.6 mm (4 in). Except for the triaxial specimens of New Providence shale, kneading foot pressures were chosen to simulate dry-of-optimum, at-optimum, and wet-of-optimum conditions from impact compaction tests at three energy levels. For the triaxial specimens of New Providence shale, three kneading compaction pressures were used. The samples of very low density were merely lightly hand tamped by using a standard Proctor hammer.

Thin-walled stainless steel sampling tubes were used to obtain oedometer and triaxial specimens of St. Croix clay (low-density triaxial specimens were trimmed from the compacted samples). For New Providence shale, the as-compacted diameter was used. The approximate dimensions of the test specimens are given below (1 mm = 0.039 in):

	Oedometer Specimens		Triaxial Specimens	
Material	Diameter (mm)	Height (mm)	Diameter (mm)	Height (mm)
St. Croix clay	63.5	25.4	35.6	76.2
New Providence shale	101.6	38.1	101.6	218.4

In the oedometer test, the specimens were first loaded and, after equilibrium deformations were reached, they were back pressure saturated. In the triaxial tests, the specimens were back pressure saturated under a low effective stress and then consolidated to the required effective stress. More detailed descriptions of the test procedures used in the studies are available elsewhere (8,13,26,27).

VOLUME-CHANGE CHARACTERISTICS

St. Croix Clay

The volume-change characteristics on saturation of compacted specimens of St. Croix clay are taken from DiBernardo (8) for samples loaded one-dimensionally in a consolidation ring and from Johnson (13) for samples loaded in a triaxial cell.

One-Dimensional Volume Changes

DiBernardo (8) found that the percentage of volume change on saturation could be expressed by the following equation (R^2 = 0.86):

$$\Delta V/V_o = 25.47 - 0.872\,w - 0.0048\,P_c \quad (1)$$

where

ΔV = volume change from the as-compacted to the fully saturated condition under a surcharge load,
V_o = as-compacted volume,
w = as-compacted water content (%), and
P_c = nominal compaction pressure (kPa).

Negative values of $\Delta V/V_o$ denote swelling, and positive values denote compression. The as-compacted void ratio or density corresponding to a given moisture content and compaction pressure can be obtained from the relevant moisture-density compaction curve.

The level of surcharge also affects the multitude of volume change that will take place on saturation, although this parameter does not appear explicitly in the regression equation for volume change. The effect of sustained load on saturation is shown in Figures 1-3 (8).

Consider, for example, samples that have an initial void ratio of approximately 0.8 (Figure 1). Under sustained loads of 160, 320, and 480 kPa (23, 46, and 70 lbf/in²), volume change on saturation was positive (collapse) and amounted to 0.16, 11.6, and 12.4 percent, respectively. If we consider a set of samples all under the same sustained load, we find that the volume change on saturation generally varies from negative (swell) to positive (collapse) values as the initial void ratio increases. This trend of volume change with initial void ratio is monotonic only for a sustained load of 320 kPa. The observed variations from simple trends are probably attributable to differences in as-compacted water content or degree of saturation and perhaps the compaction prestress induced in samples that have the same initial void ratio (8).

When the void ratio just before soaking is plotted against the percentage of volume change on soaking, the spread in the data points is very much reduced (Figure 3). Here, $\Delta V/V_o$ is defined as $\Delta e/(1 + e_1)$, where Δe = change in void ratio caused exclusively by soaking and e_1 = void ratio just prior to soaking under a sustained surcharge.

Triaxial Volume Changes

Johnson (13) saturated triaxial specimens under effective confining pressures that ranged from 68 to 278 kPa (10-40 lbf/in²). The volume change from the as-compacted to the saturated-consolidated condition is shown in Figure 4 (13). For a given consolidation pressure the volume change varied from a net swell to a net compression, more or less monotonically, as the initial void ratio increased. The final void ratio was found to decrease almost linearly with the logarithm of the consolidation pressure for saturated specimens. Figure 5 (13) shows the results for specimens compacted dry of optimum (D), at optimum (O), and wet of optimum (W) for three energy levels : modified Proctor (M), standard Proctor (S), and low-energy Proctor (L).

Johnson (13) developed the following model for percentage of volume change on saturation and consolidation (R^2 = 0.95):

$$\Delta V/V_o = 28.48 - 0.000\,013\,6\,\rho_d^2 + 0.0077\,S_i\sqrt{\sigma_c'} \quad (2)$$

where

$\Delta V/V_o$ = estimated volume change (%),
ρ_d = as-compacted dry density (kg/m³),
S_i = initial degree of saturation (%), and
σ_c' = isotropic consolidation pressure (kPa).

Positive values of $\Delta V/V_o$ denote compression, and negative values denote swell. Figure 6 (13) shows the variation in percentage of volume change with dry density for three values of consolidation pressure but for samples that have the same degree of saturation. In Figure 7 (13), the consolidation pressure is held constant, and the variation of volume change with dry density is shown for three degrees of saturation.

New Providence Shale

The volume-change characteristics on saturation of compacted specimens of New Providence shale are taken from Witsman (27) for specimens loaded one-dimensionally in a consolidation ring and from Abeyesekera (26) for cylindrical specimens loaded in a triaxial cell.

Figure 1. Volume-change characteristics of oedometer specimens of St. Croix clay.

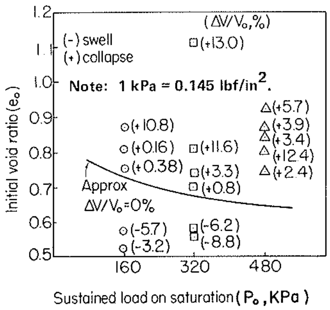


Figure 2. Volume change of oedometer specimens of St. Croix clay as a function of initial void ratio.

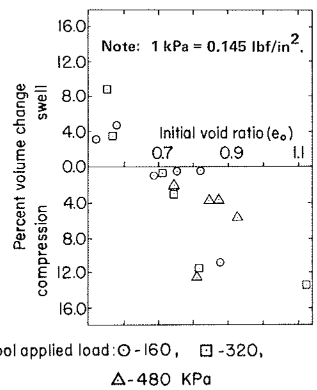


Figure 3. Volume change of oedometer specimens of St. Croix clay as a function of void ratio before saturation.

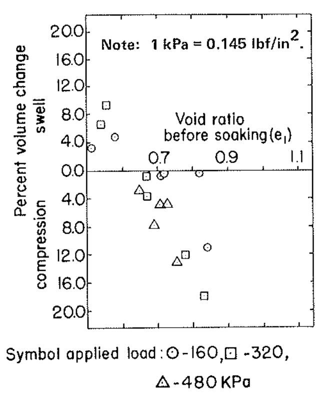
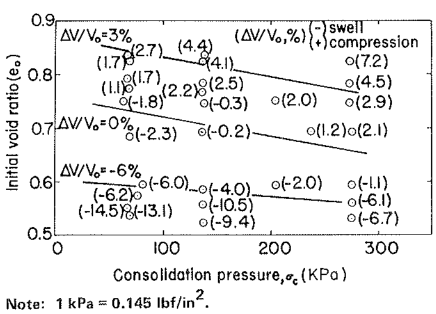


Figure 4. Contours of percentage of volume change caused by saturation and consolidation of triaxial specimens of St. Croix clay.



Note: 1 kPa = 0.145 lbf/in².

Figure 5. Final void ratio versus logarithm of consolidation pressure for triaxial specimens of St. Croix clay.

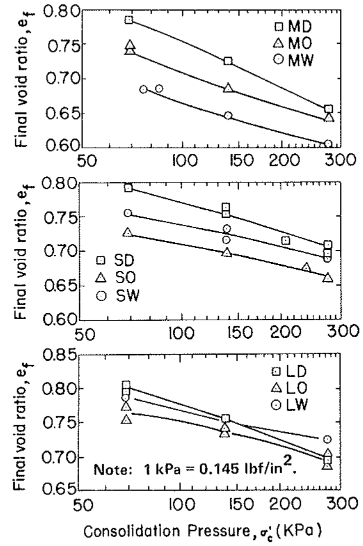
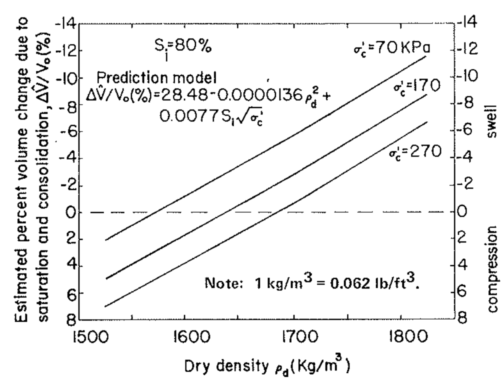


Figure 6. Prediction of percentage of volume change caused by saturation and consolidation for an initial saturation of 80 percent for triaxial specimens of St. Croix clay.



One-Dimensional Volume Changes

Witsman (27) found that the percentage of volume change on saturation could be expressed by the following equation ($R^2 = 0.80$):

$$\Delta V/V_o = 3.06 - 0.985\, e_o P_o^{1/2} + 0.000\,85\, wP_o \qquad (3)$$

where

e_o = as-compacted void ratio,
w = as-compacted water content (%), and
P_o = applied vertical pressure prior to and during saturation (kPa).

Positive values of $\Delta V/V_o$ denote swelling, and negative values denote compression.

The magnitudes of the coefficients of the variable terms in the model are such that the positive term never exceeds the negative term for the range of values of e_o, w, and P_o investigated. The swell decreases or the volume reduction increases with increasing void ratio or decreasing water content. Figure 8 (13) shows variation of volume change on saturation as a function of the vertical pressure for a constant water content of 10 percent and three void ratios. In Figure 9 (13), the void

Figure 7. Prediction of percentage of volume change caused by saturation and consolidation at a consolidation pressure of 170 kPa for triaxial specimens of St. Croix clay.

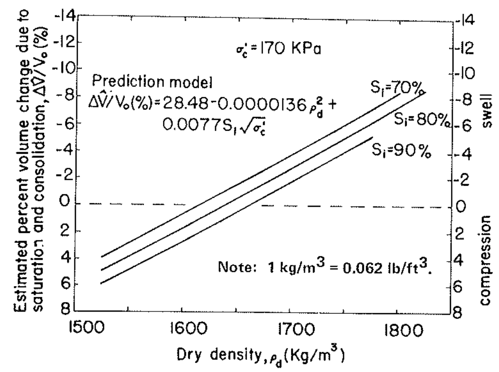


Figure 8. Effect of void ratio on volume-change model at a constant moisture content of 10 percent for triaxial specimens of St. Croix clay.

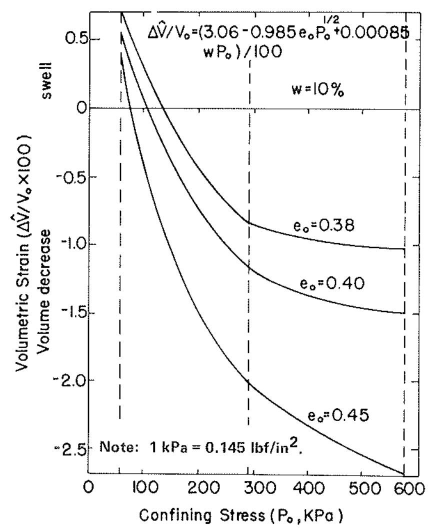


Figure 9. Effect of moisture content on volume-change model as constant void ratio of 0.40 for triaxial specimens of St. Croix clay.

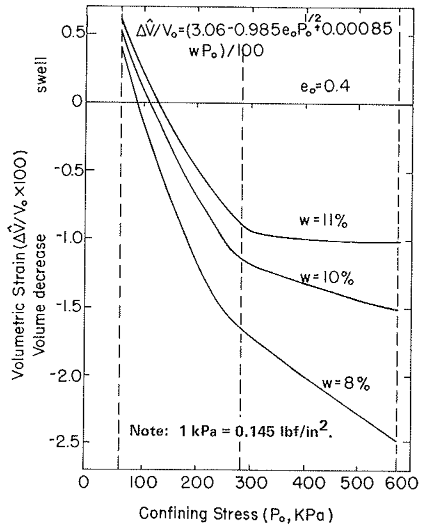
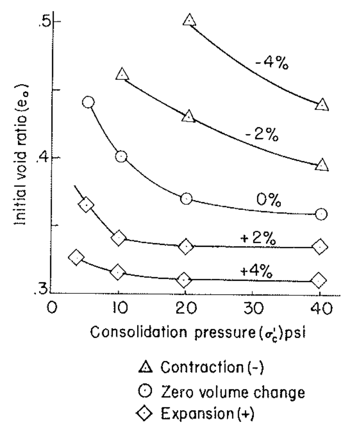


Figure 10. Contours of percentage of volume change caused by saturation and consolidation of triaxial specimens of New Providence shale.



Note: 1 lbf/in^2 = 6.89 kPa.

ratio is held constant and the curves represent three water contents. The table below gives the range in values of the variables for which the volume-change model is valid:

w (%)	e_o	P_o(kPa)
10.4-16.7	0.53-0.42	58-580
8.8-15.5	0.48-0.40	58-580
5.1-10.5	0.34-0.29	58-580

Extrapolating beyond these combinations of variables is not recommended.

Triaxial Volume Changes

The volume-change characteristics of triaxial specimens of compacted shale on saturation was studied by Abeyesekera (26). For each consolidation pressure the percentage of volume change was plotted against the as-compacted void ratio and curves estimated to fit the data as closely as possible. From these curves, sets of values for as-compacted void ratio and consolidation pressure corresponding to a given volume change were obtained. Figure 10 (26) shows contours of percentage of volume change in terms of these variables.

The model used by Johnson (13) for triaxial specimens of compacted St. Croix clay (Equation 2) was developed for the triaxial specimens of compacted shale with the following result:

$$(\Delta V/V_o)100=-93.2975+91.4634\times10^{-6}\,\gamma_d^2-18.3896\times10^{-3}\,S_i\sqrt{\sigma_c'} \qquad (4)$$

The R^2 for this equation is 0.77, and positive values denote swell whereas negative values denote compression.

When the model developed by Witsman (27) for one-dimensional volume change (Equation 3) is applied to triaxial specimens of the same shale, the equation is

$$(\Delta V/V_o)100=-89.1132-10.3783\times10^{-3}\,e_o\sqrt{P_o'}+56.7273\times10^{-5}\,wP_o' \qquad (5)$$

For this equation, positive values denote swell and negative values denote compression; however, the R^2 value is low at 0.56.

In order to arrive at an improved model for triaxial specimens of shale, Abeyesekera performed a stepwise regression and obtained the following equation, for which R^2 = 0.80:

$$(\Delta V/V_o)100 = -90.177 + 15.7213 \times 10^{-5}\gamma_d^2 - 81.3027 \times 10^{-2}\log_{10}\sigma_c' + 60.4735 \times 10^{-5} w\sigma_c' - 12.9882 \times 10^{-2} S_i\sqrt{\sigma_c'} + 64.4015 \times 10^{-2} S_i \log\sigma_c' - 26.0065 \times 10^{-3}\gamma_d + 48.6056 \times 10^{-4} e_o\sqrt{\sigma_c'} - 23.3252 \times 10^{-2} S_i \quad (6)$$

The variables e_o and $e_o \log \sigma_c'$ were found to be insignificant at the 95 percent confidence level. Since

$$\gamma_d = G_s\gamma_w/(1 + e_o) \quad (6a)$$

and

$$S_i = G_s w/e_o \quad (6b)$$

where G_s = specific gravity of solids and γ_w = unit weight of water, it will be seen that

$$\Delta V/V_o = f(e_o, w, \sigma_c') \quad (6c)$$

The first two terms of Equation 6, γ_d^2 and $\log \sigma_c'$, account for the major effects. These terms give $R^2 = 0.78$ and, when the remaining terms are also included, the value of R^2 increases only slightly to 0.80. Thus $\Delta V/V_o$ is essentially controlled by $(1 + e_o)^{-2}$ and $\log_{10} \sigma_c'$.

SUMMARY

The volume-change characteristics on saturation of compacted St. Croix clay and New Providence shale were found to depend on the as-compacted condition of the material as well as the confining pressure. Since the effective stress acting on a partially saturated specimen is difficult to define, it was not used to explain volume changes on saturation. Rather, these changes were explained in terms of the dry density or void ratio, the water content or degree of saturation, and the surcharge pressure for oedometer specimens (or the isotropic confining pressure for triaxial specimens).

All observed volume changes were correlated with the previously listed independent variables, and statistically valid correlation equations were developed. Because the state of stress in an embankment is anisotropic, the volume changes in the field will not be the same as those observed in this study for isotropically consolidated triaxial specimens. In addition, because the state of strain in an embankment is anisotropic and lateral movements do take place, the vertical strains in the field will not be the same as those observed in this study for oedometer specimens. Even so, the equations can be quite helpful in predicting the relative amounts of volume change for embankments in service as well as in showing how these volume changes can be controlled through changes in compaction specification.

ACKNOWLEDGMENT

The research reported in this paper was carried out under the sponsorship of ISHC and the Federal Highway Administration through the Joint Highway Research Project at Purdue University.

REFERENCES

1. C.C. Ladd and T.W. Lambe. The Identification and Behavior of Compacted Expansive Clays. Proc., 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, France, Vol. 1, 1961, pp. 201-205.
2. H.B. Seed, R.J. Woodward, and R. Lundgren. Predictions of Swelling Potential for Compacted Clays. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 88, No. SM3, 1962, pp. 53-88.
3. A.W. Skempton. The Colloidal Activity of Clays. Proc., 3rd International Conference on Soil Mechanics and Foundation Engineering, Zurich, Switzerland, Vol. 1, 1953, pp. 57-61.
4. G. Kassif and R. Baker. Aging Effects on Swell Potential of Compacted Clay. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM3, March 1971, pp. 529-540.
5. G. Kassif, M. Livneh, and G. Wiseman. Pavements on Expansive Clays. Academic Press, Jerusalem, Israel, 1969.
6. L.M. Krazynski. The Need for Uniformity in Testing of Expansive Soils. Proc., Workshop on Expansive Clays and Shales in Highway Design and Construction, Washington, DC, Federal Highway Administration, Vol. 1, 1973, pp. 98-136.
7. R. Baker and G. Kassif. Mathematical Analysis of Swell Pressure with Time for Partly Saturated Clays. Canadian Geotechnical Journal, Vol. 5, No. 4, 1968, pp. 217-224.
8. A. DiBernardo. The Effect of Laboratory Compaction on the Compressibility of a Compacted Highly Plastic Clay. Purdue Univ., West Lafayette, IN, M.Sc.E. thesis and Joint Highway Research Project Rept. 79-3, May 1979.
9. S.F. Gizienski and L.J. Lee. Comparison of Laboratory Swell Tests to Small-Scale Field Tests. Proc., International Research and Engineering Conference on Expansive Clay Soils, Texas A&M Univ., College Station, 1965.
10. R.J. Hodek. Mechanism for the Compaction and Response of Kaolinite. Purdue Univ., West Lafayette, IN, Ph.D. thesis and Joint Highway Research Project Rept. 36, Dec. 1972, p. 269.
11. W.G. Holtz and H.J. Gibbs. Engineering Properties of Expansive Clays. Trans., ASCE, Vol. 121, 1956, pp. 641-677.
12. W.G. Holtz. Expansive Clays: Properties and Problems. Colorado School of Mines Quarterly, Vol. 54, No. 4, 1959, pp. 89-125.
13. J.M. Johnson. The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation. Purdue Univ., West Lafayette, IN, M.Sc.E. thesis and Joint Highway Research Project Rept. 79-7, Aug. 1979.
14. G. Kassif, R. Baker, and Y. Ovadia. Swell-Pressure Relationships at Constant Suction Changes. Proc., 3rd International Conference on Expansive Soils, Haifa, Israel, 1973, pp. 201-208.
15. G. Kassif and A.B. Shalom. Experimental Relationship Between Swell Pressure and Suction. Geotechnique, Vol. 21, No. 3, 1971, pp. 245-255.
16. C.C. Ladd. Mechanisms of Swelling by Compacted Clay. HRB, Bull. 245, 1960.
17. D.R. Lambe and S.J. Hanna. Proceedings of the Workshop on Expansive Clays and Shales in Highway Design and Construction, Washington, DC. Federal Highway Administration, Vol. 1, 1973, p. 349, and Vol. 2, 1973, p. 304.
18. T.W. Lambe. The Engineering Behavior of Compacted Clay. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 84, No. SM2, May 1958, pp. 1655-1 to 1655-35.
19. G.A. Leonards and A.G. Altschaeffl. Discussion of Review of Collapsing Soils by J.H. Dudley. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM1, pp. 269-271.
20. L.P. Mishu. Collapse in One-Dimensional Compression of Compacted Clay upon Wetting. Purdue Univ., West Lafayette, IN, M.Sc.E.

thesis, Aug. 1963, p. 105.
21. J.K. Mitchell. Recent Advances in the Understanding of the Influence of Mineralogy and Pore Solution Chemistry of the Swelling and Stability of Clays. Proc., 3rd International Conference on Expansive Soils, Haifa, Israel, Vol. 2, 1973, pp. 11-25.
22. J.R. Sallberg and P.C. Smith. Pavement Design over Expansive Clays: Current Practices and Research in the United States. Proc., International Research and Engineering Conference on Expansive Clay Soils, Texas A&M Univ., College Station, 1965.
23. H.B. Seed, J.K. Mitchell, and C.K. Chan. The Strength of Compacted Cohesive Soils. ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, CO, 1960, pp. 877-964.
24. H.B. Seed, J.K. Mitchell, and C.K. Chan. Studies of Swell Pressure Characteristics of Compacted Clays. HRB, Bull. 313, 1962, pp. 12-40.
25. P. Deo. Shales as Embankment Materials. Purdue Univ., West Lafayette, IN, Ph.D. thesis and Joint Highway Research Project Rept. 45, Dec. 1972.
26. R.A. Abeyesekera. Stress Deformation and Strength Characteristics of a Compacted Shale. Purdue Univ., West Lafayette, IN, Ph.D. thesis and Joint Highway Research Project Rept. 77-24, May 1978.
27. G.R. Witsman. The Effect of Compaction Prestress on Compacted Shale Compressibility. Purdue Univ., West Lafayette, IN, M.Sc.E. thesis and Joint Highway Research Project Rept. 79-16, Aug. 1979.

Publication of this paper sponsored by Committee on Environmental Factors Except Frost.

Characterization of Expansive Soils

R. GORDON McKEEN AND DEBORA J. HAMBERG

Most expansive soils encountered in engineering problems are at a degree of saturation below 100 percent. Knowledge of the moisture condition of such soils is best obtained by measuring soil suction. Soil suction can be determined routinely by using either the thermocouple psychrometer or filter-paper methods. The volume response can be characterized by obtaining a volume-change measurement along with a determination of suction change. This measure of soil response is called the suction compression index and is a fundamental property of unsaturated fine-grained soils. An empirical method of estimating the suction compression index from index tests is provided. The soil stiffness, or the reduction in swell behavior caused by loads, must also be accounted for in making heave predictions. A nondimensional equation is presented that was developed by regression techniques from a large number of data found in the technical literature. The equation provides a tool for reducing the volumetric response of expansive soils as applied loads are increased. The use of this information in predicting heave is illustrated.

Expansive soils undergo volume changes when their moisture condition varies. Designing transportation facilities for expansive-soil areas requires consideration of the volume changes that are likely to occur. Several decades of research on this problem have produced the tools of "expansive soil mechanics". Mitchell (1) recently presented three fundamental soil characteristics that must be considered in design: soil response to load, soil response to moisture changes, and the diffusivity of water moving in the soil. Techniques for obtaining these properties are available. This paper describes techniques for characterizing the moisture and load response of natural expansive soils.

SOIL SUCTION

Soil suction is a macroscopic property of soil that indicates the intensity with which a soil will attract water. Suction results from (a) the interplay of attraction and repulsion forces of charged clay particles and polar water molecules, (b) surface tension forces of water, (c) solution potentials caused by dissolved ions, and (d) density. A distinction must be drawn between pore-water tension and suction: Tension applies to the actual pressure state of the pore water; suction is total head, which includes pore-water pressure, osmotic pressure, and adsorptive pressure.

In engineering problems, suction is considered to be composed of matrix and osmotic suction components, and their sum is termed the total suction. Matrix suction is the negative gage pressure that, through a porous membrane, will hold soil water in equilibrium with the same soil water within a soil sample. Matrix suction is the result of surface adsorption and capillary forces. Osmotic or solute suction is a negative gage pressure that will hold pure water in equilibrium with soil water through a membrane that allows only water molecules to pass. Osmotic suction results from variation in ion concentration in the pore fluid.

Two independent stress variables have been used to describe the state of stress in unsaturated soils. The preferable stress-state variables are $(\sigma - u_a)$ and $(u_a - u_w)$, where σ = total stress, u_a = pore-air pressure, and u_w = pore-water pressure. The term $(\sigma - u_a)$ is called the total stress term and $(u_a - u_w)$ is called the matrix suction term. This combination of stress-state variables is most satisfactory because the effects of environmental variables can readily be separated in terms of stress changes. This approach assumes that u_a is approximately atmospheric and the osmotic component of suction remains constant. These assumptions are adequate for many engineering problems.

SUCTION MEASUREMENT

Thermocouple Psychrometers

A psychrometer is defined as essentially two similar thermometers, one of which has a bulb that is kept wet so that the resulting evaporative cooling makes it register a lower temperature than the dry one; the difference between the readings represents a measure of the dryness of the atmosphere. The