HIGHWAY RESEARCH BOARD

Bulletin 313

Moisture, Density, Swelling and Swell Pressure Relationships

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Contents

Lateral Swelling Pressure on Conduits from Expansive Clay Backfill

G. KASSIFF, Senior Lecturer, Kumasi College of Technology, Ghana;[1] and J. G. ZEITLEN, Professor of Soil Engineering, Israel Institute of Technology, Haifa.

The phenomenon of lateral swelling pressures exerted by expansive clays is well-known. Some structures (piles, retaining walls, conduits, etc.) crack or fail either by action of swelling forces in the horizontal direction or by combinations of lateral with other forces.

This paper deals with a study of the lateral pressures developed along conduits buried in a swelling clay. These pressures originated from the horizontal swelling of the clay backfill during moisture variations.

The study included laboratory investigations of the clay backfill swelling characteristics, as well as field experiments on two pipelines buried at different moisture-density conditions. The pipelines were equipped with strain gages installed in the longitudinal direction of the conduits, thus making possible the determination of the horizontal moment distribution along the pipelines caused by the lateral swelling of the backfill. From the moments, the load distribution diagrams, as well as the elastic lines, were established. The elastic lines were compared with direct field measurements of the horizontal movements of the conduits.

The maximum moments were also computed by assuming various load distributions and statical conditions of the pipe ends. A reasonable correlation was found between the assumed theoretical conditions and the results obtained from the measurements, which may help future prediction of swelling pressures from clay backfills.

● THE PHENOMENON of lateral swelling pressures exerted by expansive clays on structures is well known. Some structures such as piles, retaining walls, and conduits crack or fail either under the action of horizontal swelling forces or by combinations of the lateral with other forces. For example, cases of horizontal shear of piles have been attributed mainly to the lateral swelling of the clay (1), and rupture of conduits has been

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[1] On loan from Israel Institute of Technology, Haifa.

found to result from non-uniformity of the swelling forces exerted by the clay (2,3).

Vertical swelling behavior has been the subject of appreciable research and publication (4,5,6), but little has been reported on horizontal action. This paper reports that portion of a research on buried conduits in which lateral swelling pressures were developed from clay backfill placed at two different initial moisture-density conditions. The purpose is to show typical lateral load distributions and the corresponding horizontal movements under the different conditions. A method was developed for the evaluation of the swelling pressures and the resulting maximum bending moments. The limitations of future assessment of pressures by this method are pointed out.

THEORETICAL CONSIDERATIONS

The lateral pressures acting on a conduit laid in expansive clay are dependent on the backfill properties. Upon the ingress of moisture into the clay backfill, opposing lateral pressures develop. If the backfill soil were absolutely uniform and the increase of moisture on both sides of the conduit equal, the pressures would be nullified and only influence ring stresses. However, this is not the actual case, since it is not practicable under field conditions to ensure that the backfill soil will not have some variation in moisture or density along both sides of the pipe. Moreover, even if the backfill is completely homogeneous, non-uniform moisture conditions on the sides might result from accidental flooding, non-uniform irrigation, etc., thus producing inequalities in the opposing pressures.

The pressure on one side of the conduit is partly cancelled by pressure on the opposite side, but such forces are non-uniform along the length of the pipe. As a result the pipe deforms longitudinally under the net differential load. Should measurements of longitudinal deformations be utilized to calculate lateral loads, only the final (not differential) load, under which the structure is statically in equilibrium, would be found. It is not feasible to deduce the original swelling pressure on each side of the conduit from these final loads, because any pair of opposing lateral pressures may yield the same differential pressure. Hence, the following approach was used for the evaluation of the lateral swelling pressure.

A laboratory investigation was conducted on the swelling characteristics of the backfill soil. The results assisted in assuming the magnitude of the swelling pressures that may develop in the field. With these data in hand, the moments under various load distributions and assumptions as to the end conditions of the pipe were determined.

The various load distributions that were tried were as follows:

Case (a). - Distributed load, q, twice as large as the opposing load, acting on the middle portion of an individual conduit (l = 4.0 m), hinged at the ends (Fig. 1a). The moments:

$$M_A = 0 \qquad (1)$$

$$M_C = M_{max} = \frac{ql^2}{24} \qquad (2)$$

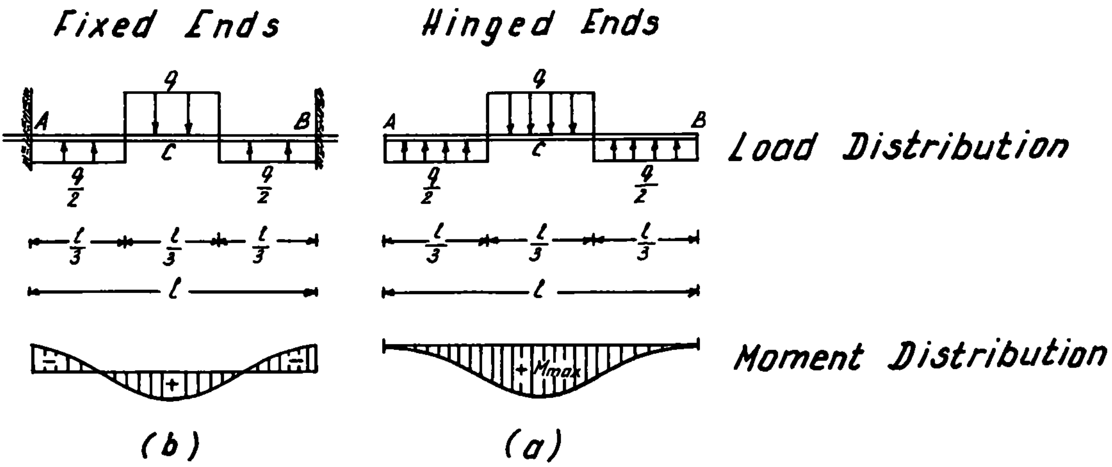

Figure 1. Load and moment distributions of a conduit: (a) with hinged ends, (b) with fixed ends.

Assuming the ends are fixed, (Fig. 1b) the moments:

$$M_A = M_C = \frac{ql^2}{48} \qquad (3)$$

Case (b). - Distributed load, q, acting on an individual conduit (l = 4.0 m) along one side. The conduit is assumed to rest on an elastic subgrade along the other side (Fig. 2) and fixed at the ends, which simulates either rigid connections or a long pipe subjected to moisture changes different from the section analyzed.
The moments according to the formulae of Hétényi (7):

$$M_A = M_B = - \frac{q}{2\lambda^2} \left(\frac{\text{Sin h}\lambda l - \text{Sin }\lambda l}{\text{Sin h}\lambda l + \text{Sin }\lambda l}\right) \qquad (4)$$

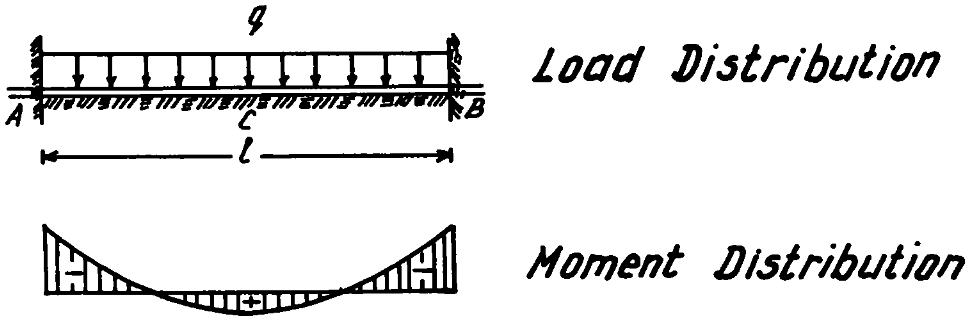

Figure 2. Load and moment distributions of a fixed conduit subjected to subgrade reaction.

$$M_C = \frac{q}{2} \left(\frac{\mathrm{Sin}\,\frac{\lambda l}{2} \;\; \mathrm{Cos\,h}\,\frac{\lambda l}{2} \; - \; \mathrm{Cos}\,\frac{\lambda l}{2} \;\; \mathrm{Sin\,h}\,\frac{\lambda l}{2}}{\mathrm{Sin\,h}\lambda l + \mathrm{Sin}\,\lambda l} \right) \qquad (5)$$

where

$$y = \sqrt[4]{\frac{K}{4EI}} \quad \text{(The characteristic of the beam)}$$

$K = K_o B$

K_o is the coefficient of subgrade reaction,

B is the outside diameter of the pipe,

E is Young's modulus of the pipe material, and

I is the moment of inertia of the pipe section.

The maximum deflection takes place at point C and may be computed from

$$y_c = \frac{q}{K} \left[1 - \frac{2(\mathrm{Sin\,h}\,\frac{\lambda l}{2} \;\; \mathrm{Cos}\,\frac{\lambda l}{2} + \mathrm{Cos\,h}\,\frac{\lambda l}{2}\,\mathrm{Sin}\,\frac{\lambda l}{2})}{\mathrm{Sin\,h} + \mathrm{Sin}} \right] \qquad (6)$$

The calculated moments and displacements were then compared with the maximum values obtained in field measurements and the validity of the swelling pressures assumed was thus examined.

LABORATORY INVESTIGATIONS OF THE CLAY BACKFILL

Classification. - The results of the basic classification tests on the clay, sampled from the experimental site, are summarized in Table 1.

TABLE 1

Clay sizes (% smaller than 5 microns)	65 - 70
Liquid limit (%)	78 - 85
Plastic limit (%)	20 - 22
Plasticity index (%)	58 - 63
Shrinkage limit (%)	9.5 - 11
Activity	1.1 - 1.2
Free swell (%) (8)	130

It is obvious that the clay under study is of a highly expansive nature.

Swelling Behavior. - The swelling characteristics of the remolded clay were investigated for a wide range of moisture and density obtained by varying the energy applied to the Harvard Miniature Compactor. Three compaction curves produced in the laboratory and one obtained in the experimental site in the field served as the basis for the swelling behavior study. It should be noted that the density range tested was well below the maximum density of about 1.4 t per cu m as obtained by A.S.T.M. Standard Compaction test.

Two types of swelling tests were performed on samples prepared at placement conditions corresponding to selected points on the compaction curves: (1) percent swelling under the backfill pressure and (2) swelling pressure with zero movement.

Swell under Backfill Pressure. — The test procedure followed the one suggested by Holtz and Gibbs (8), in which the amount of swelling of a compacted specimen under a certain load and subjected to soaking is determined. The specimens were tested under a pressure of 0.13 kg per sq cm, which corresponds to the backfill load (assuming a depth of 0.9 m and unit weight of 1.4 t per cu m).

The test results were produced in terms of "equal swelling lines" projected on the compaction curves (Fig. 3). The amount of swelling varied under the conditions tested from 1 percent at high initial moisture content to 7.5 percent at the driest conditions. Actually the amount of swelling increased under combined conditions of low moisture and high density.

Swelling Pressure with Zero Movement. — According to the test procedure (8), the clay specimen is restrained from swelling and the maximum pressure required to keep the specimen from moving is measured. The results are presented in Figure 4 in terms of "equal pressure lines" projected on the compaction curves. The swelling pressures were found to vary from 0.05 kg per sq cm to 0.45 kg per sq cm, increasing with combined conditions of low moisture and high density.

THE FIELD STUDY

Program. — The program of the field study included measurements of longitudinal deformations and movements in the horizontal direction of two

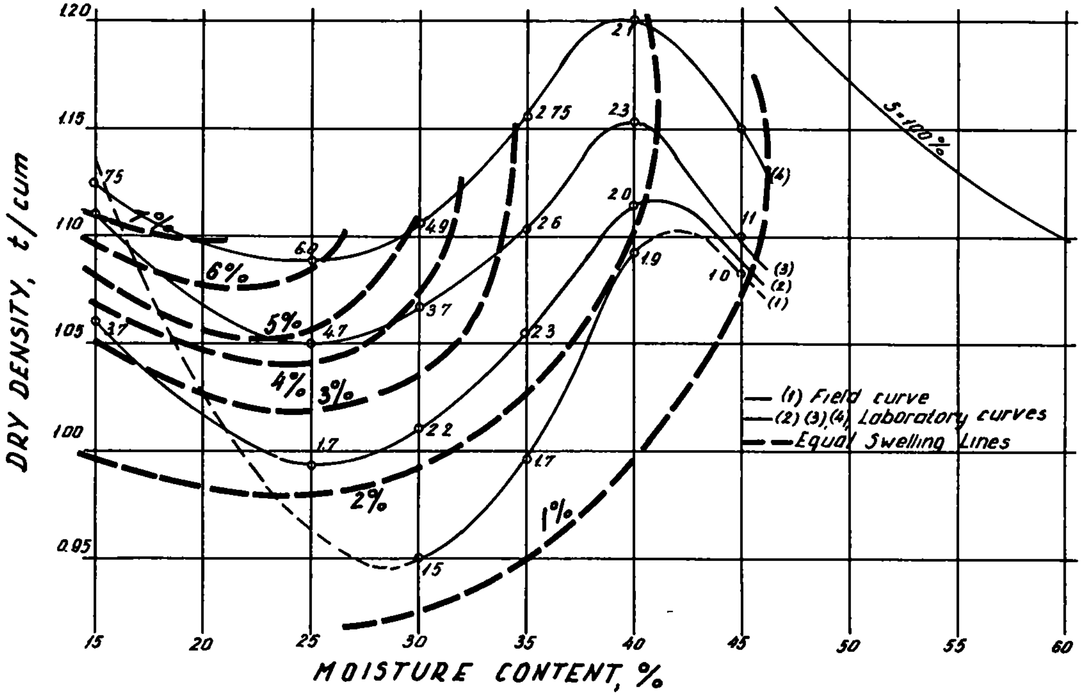

Figure 3. Equal swelling lines for compacted clay.

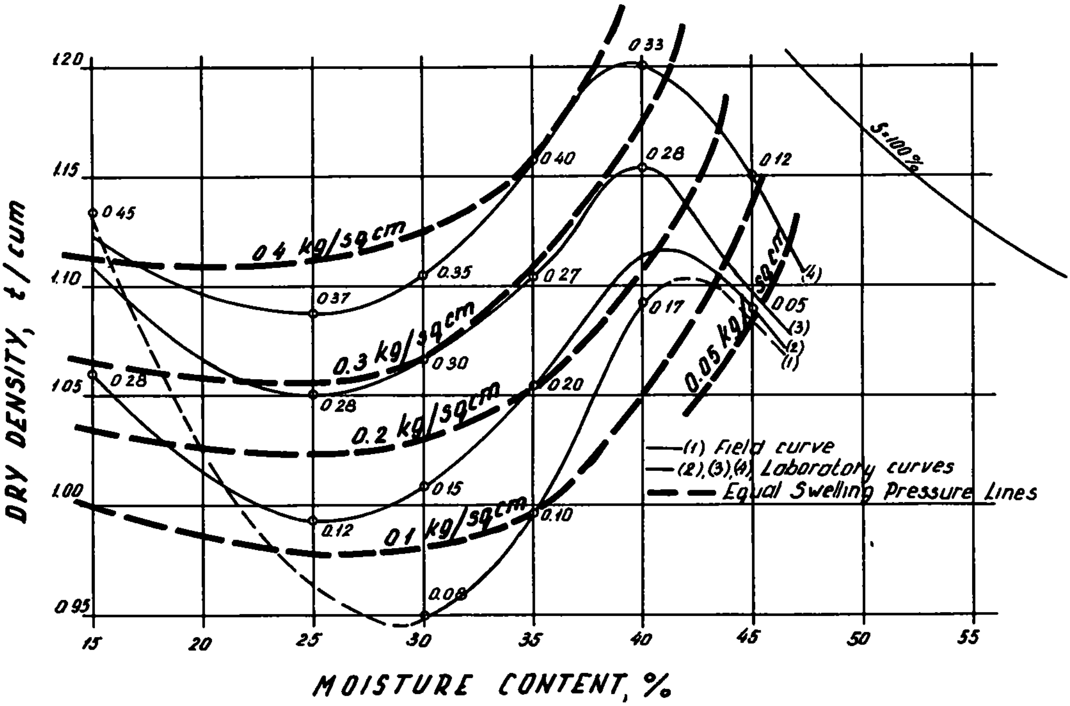

Figure 4. Equal pressure lines for compacted clay.

experimental asbestos-cement pipelines subjected to moisture variations in the clay backfill. The pipe diameter was 4 in., the length of each line 20 m and they were buried at a depth of 0.9 m. In the center of each pipeline a standpipe of the type used for irrigation was installed. The individual pipes (4.0 m long each) on both sides of the standpipe were mounted with vibrating wire strain gages (9) in the longitudinal direction of the pipelines and horizontal with both sides of the pipe section. Metal plates were attached to the top of these pipes for measurement of horizontal movements. The plates were made accessible and at the same time protected by a 6-in. aluminium sleeve resting on the pipeline.

The backfill placement conditions varied as follows: Pipeline I was buried under compacted wet soil, simulating an installation of pipeline under wet conditions. Compaction was accomplished by hand tamping. Pipeline II, on the other hand, was placed under dry backfill, simulating a practice often encountered in Israel, in which the trench and backfill are left exposed to drying before backfilling. Backfilling was accomplished by hand tamping of dry clay in lumps.

The behavior of the pipelines was observed under gradual moisture changes caused by the winter rains of 1958/59 and later under concentrated irrigation.

Distribution of Horizontal Moments.— The zero readings on the vibrating wire strain gages were taken before backfilling at the end of the summer of 1958. Later readings were taken periodically in conjunction with determinations of moisture changes in the backfill. Upon the swelling of the clay backfill, considerable horizontal transverse loads were

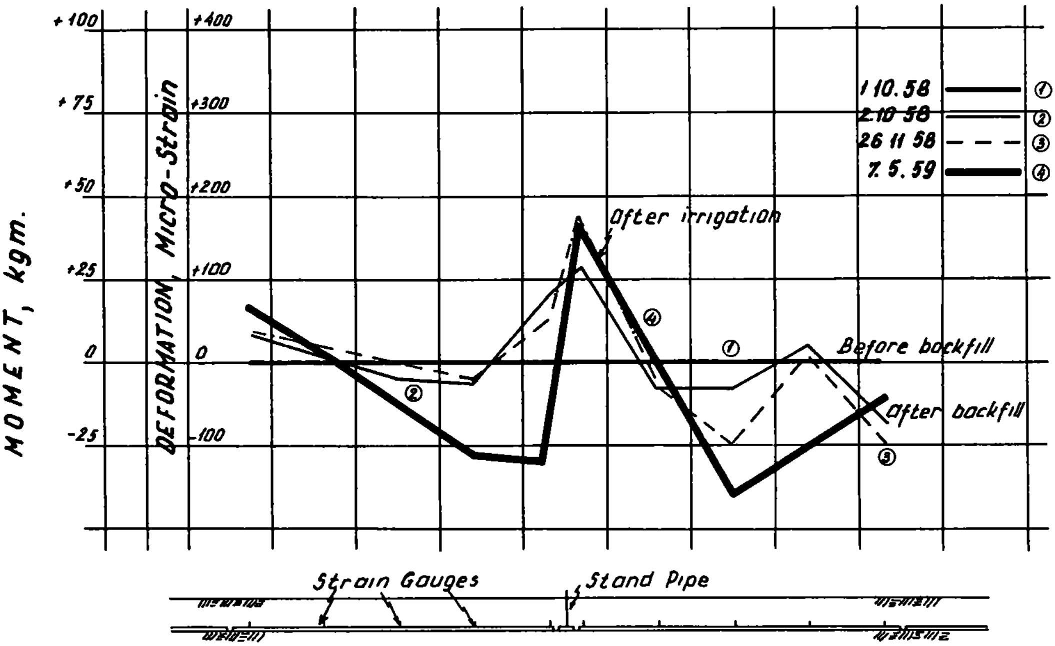

Figure 5. Distribution of longitudinal moments in the horizontal direction, pipeline I.

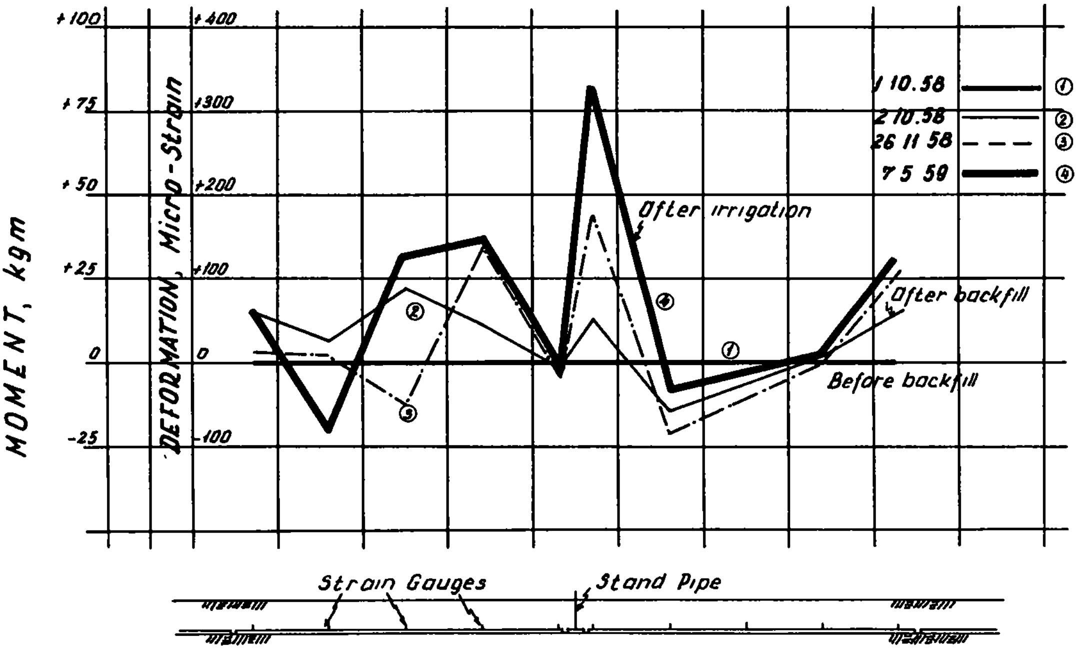

Figure 6. Distribution of longitudinal moments in the horizontal direction, pipeline II.

developed along both pipelines. The horizontal moment distributions due to these loads and their variation with time are presented in Figure 5 for pipeline I and in Figure 6 for pipeline II.

Distribution of Horizontal Loads. — From the moment diagrams the lateral load distributions along the pipelines were established. These are presented in Figures 7(a) and 8(a) for pipeline I and II respectively.

From the load-distribution variation with time it may be seen that the loads developed by the backfill lateral pressure on both pipelines (namely, after completion of the backfilling operation) are low in comparison to those caused by the nonuniform swelling pressures exerted by moisture variations in the clay. The difference between the load distributions (due to backfilling and swelling) increased with the increase of moisture content in the backfill and reached a maximum value after irrigation. In pipeline II this difference was much more critical than in the other pipeline, probably due to the initial dry soil conditions in pipeline II.

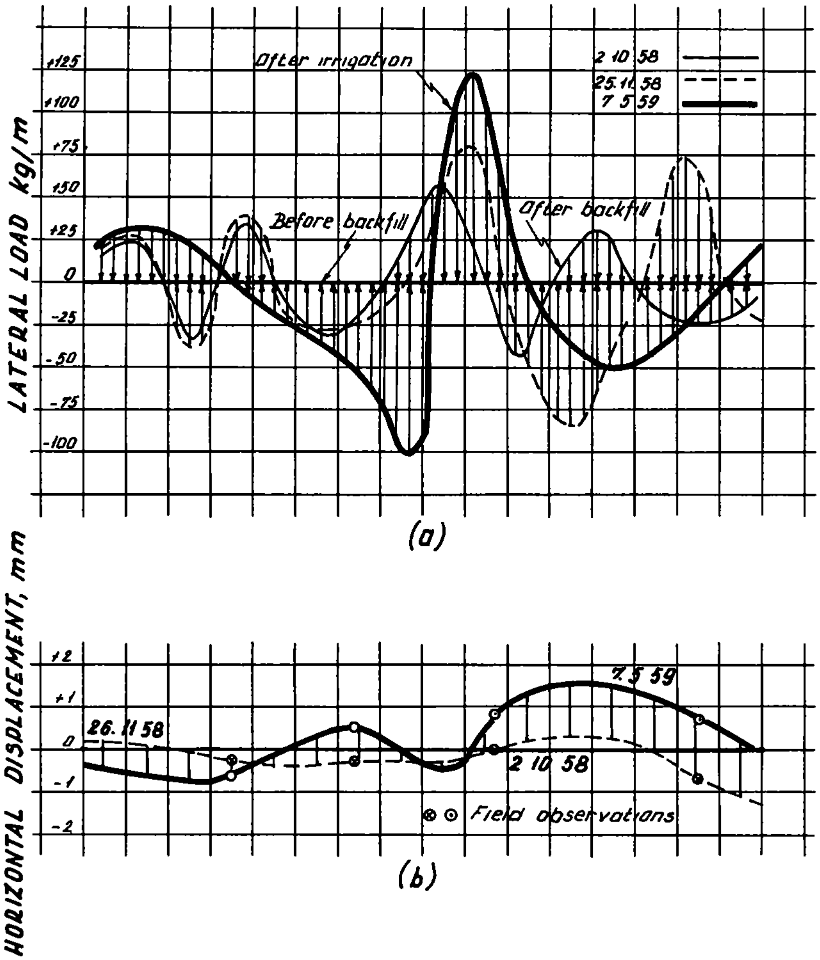

Figure 7. Lateral load diagram and horizontal displacement of pipeline I.

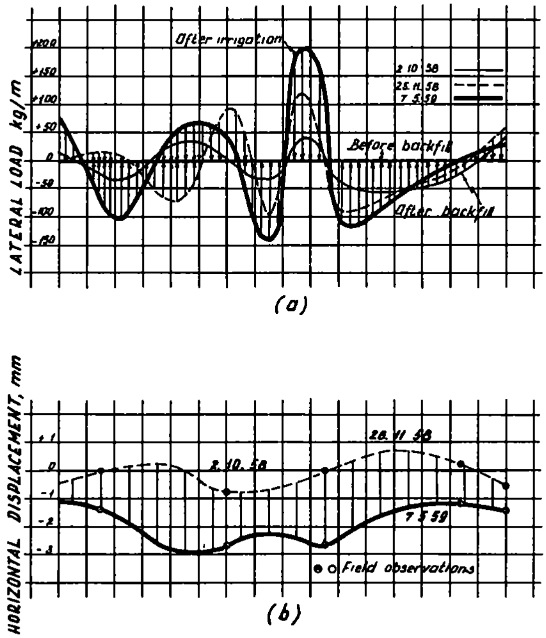

Figure 8. Lateral load diagram and horizontal displacement of pipeline II.

Horizontal Movements. — The techniques developed for the measurement of the horizontal movement of the pipelines consisted essentially of precise measurements at various time intervals of angles between points along the pipelines and a fixed base line. The measurements were made possible by placing optical targets on top of the metal plates attached to each pipeline. The results obtained by measurements were correlated with the elastic lines established by calculations from the moment distributions. A reasonable agreement was found between the two methods, as may be seen from Figures 7(b) and 8(b). It may be noted that the greatest loads evolved at zones where the lateral movement was restrained.

CORRELATION BETWEEN CALCULATIONS AND EXPERIMENTAL RESULTS

Pipeline I. — From the laboratory study (Fig. 4) it may be seen that the maximum lateral swelling pressure that may be exerted under the soil placement conditions of pipeline I is about 0.15 kg per sq cm. Allowing for some release of pressure from the horizontal movements, it is justified to assume, for comparison, that the swelling pressure was reduced to 0.1 kg per sq cm. The resulting lateral load, then, is 120 kg per m, assuming that the pressure acts normal to the whole outside diameter of the pipe; i.e., 12 cm.

When this load was applied to the statical conditions described in Case (a) the resulting moments at the hinged ends and the middle of the conduit were found to be 0 kgm and 80 kgm, respectively. However, when it was assumed that the ends were fixed, the moments at the middle and the ends were computed to be 40 kgm. The application of the same load coupled with the conditions described in Case (b) yields moments of

TABLE 2

COMPARISON OF MEASURED AND COMPUTED VALUES, PIPELINE I

Values	Maximum Load on Pipe (kg/m)	End Moment (kgm)	Middle Moments (kgm)	Maximum Horizontal Movement (mm)
Measured values	125	38	38	1.5
Computed values				
Case (a) hinged	120	0	80	2.5
Case (a) fixed	120	40	40	0.5
Case (b)	120	41.5	10	1.0

41.5 kgm at the ends and 10 kgm at the middle, assuming K_o = 1.0 kg per cu cm for the tamped clay (10). The maximum computed horizontal movement resulting from the assumptions of Case (b) was found to occur in the center and equal to 1 mm. the values obtained by the calculations may be compared to the measured values in Table 2. In this case a reasonable correlation was obtained between the measured values and values arrived at by assuming the statical conditions of Case (a) with fixed ends.

Pipeline II. — The swelling pressure obtained in the laboratory study (Fig. 4) under the soil conditions of pipeline II was 0.4 kg per sq cm. However, allowing for the relatively large horizontal movement of this pipeline, a lateral swelling pressure of 0.2 kg per sq cm is assumed for the theoretical calculations. The resulting load, hence, is 240 kg per m.

The results of measurements and theoretical computations for this pipeline are compared in Table 3. In this case it may be noted that the approach based on subgrade reaction produced the best correlation with field values.

TABLE 3

COMPARISON OF MEASURED AND COMPUTED VALUES, PIPELINE II

Values	Maximum Load on Pipe (kg/m)	End Moment (kgm)	Middle Moments (kgm)	Maximum Horizontal Movement (mm)
Measured values	200	80	40	2.7
Computed values				
Case (a) hinged	240	0	160	5
Case (a) fixed	240	80	80	1
Case (b)	240	83	20	2.5

CONCLUSIONS

The theoretical approach suggested in this paper for determining the lateral longitudinal moments and displacements of pipes buried in expansive soils has shown that it is possible to estimate the swelling pressures that may occur in the field. It is obvious, however, that there are limitations to the use of the approach for further prediction of lateral pressures on conduits or other structures until much more research is conducted on the subject.

ACKNOWLEDGMENT

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Studies of Swell and Swell Pressure Characteristics of Compacted Clays

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Factors affecting the values of swell and swell pressure developed by compacted samples of an expansive soil on exposure to water under controlled condition are examined.

Data are presented and discussed to illustrate such factors as:

1. The effect of time on swell pressure. It is shown that considerable periods of time are required before a sample develops its full swell pressure after exposure to water.
2. The effect of sample shape on measured swell pressures. It is shown that sample faces must be perfectly plain if reliable measurements are to be made.
3. The effect of volume change on measured swell pressures. It is demonstrated that volume expansions of as little as 0.1 percent during swell pressure measurement may cause an error of large magnitude in the values of swell pressure observed.
4. The relation between soil structure, as determined by compaction method, and the amount of swell and the swell pressure.
5. The effect of stress history on swell and swell pressure.
6. The relationship between swell and swell pressure. It is shown that the magnitude of the swell pressure cannot be reliably predicted from a knowledge of the amount of swell.

Explanation of the results is offered, where possible, in terms of the fundamental aspects of the system composition and structure. Practical implications of the results are pointed out with particular reference to pavement design.

● ENGINEERING PROBLEMS and failures resulting from the swelling of clays have been encountered in many areas of the world in connection with highway fills, highway subgrades, building foundations, canal linings, and other structures. When a swelling soil is encountered, the engineer may have available several courses of action for the solution of the associated problems. Among the solutions to be considered are the following:

1. By-pass the site.
2. Remove the expansive soil and replace it with a nonexpansive soil.
3. Prevent access of water to the swelling soil.
4. Make the soil nonexpansive by appropriate chemical treatment.
5. Allow expansion to occur under selected loading conditions and design the structures in such a manner that they are able to withstand displacement without distress.
6. Prevent swelling by appropriate surcharge loading and design structures to withstand the swell pressures that may develop.

The relative merits of each solution must be judged, of course, with respect to each particular project. In cases where some swelling is allowed to occur, satisfactory engineering will require reliable estimates of the probable magnitude of the swell and swell pressures developed.

During the past decade, numerous studies have been conducted to investigate the swelling characteristics of clay soils; yet, because of the complexity of the problem, considerable difficulty has been encountered in the development of satisfactory procedures. One of the primary reasons for this has been the lack of information concerning the many factors that might influence swell and swell pressure measurements in soils. In the absence of an understanding of all factors involved, accurate evaluations of swelling characteristics cannot be reasonably expected.

Much progress has in fact been made, however, in reporting on swelling studies, a number of investigators have pointed out, from time to time, procedural changes influencing the test data and, in conducting studies on soils for pavement design purposes, the authors have noted a number of other factors affecting the results of swelling tests. Recent developments in understanding the structure of compacted clays have provided a basis for explaining the influence of many of these factors, and thus it would seem that an appropriate stage has been reached when a presentation and analysis of available data might usefully contribute to an improved understanding of the subject.

CAUSES OF SWELL AND SWELL PRESSURES IN COMPACTED CLAYS

The fundamental factors causing swelling have been studied by many investigators. The influence of mechanical factors in swelling was illustrated a number of years ago by Terzaghi (1), while Bolt (2) has obtained data on the role of physicochemical effects in clay expansion. More recently Lambe (3), Ladd (4), and Lambe and Whitman (5) have outlined the causes of clay expansion in some detail.

It is generally agreed that expansion of a soil on exposure to water and/or external load reduction is attributable to the combined effect of several factors. The fact that a soil takes in water on exposure to free water or on load release is, of course, a direct consequence of a water pressure difference inside and outside the sample. Free water can only enter under the influence of a hydraulic gradient. The factors responsible for water pressures less than those in free water are the fundamental factors controlling swelling and the development of swelling pressures. Conditions within a fine-grained soil that give rise to a swelling tendency may be classified into two general categories — physicochemical and mechanical.

Physicochemical Components of Swelling

The surface structure of the clay minerals is such that water molecules are strongly attracted to them. This adsorptive effect is responsible for a portion of the water required to satisfy the swelling tendency of a soil. In addition to the unique surface structure of clay minerals, the particles usually possess a net negative electrical charge and, in order to achieve electrical neutrality in the soil, cations are attracted to the surfaces. These cations and their water of hydration take up space and thus tend to keep particles apart. More important from a swelling standpoint, however, is the fact that in the vicinity of the clay particles the cationic concentration is much higher than in the free water with which the soil communicates. The difference in ionic concentrations gives rise to an osmotic pressure difference that causes water to flow from the point of low concentration to the point of high concentration (i.e., into the soil), unless this osmotic pressure is balanced by applying a tension stress to the free water or applying a pressure to the water of high ionic concentration.

In partially saturated soils the affinity of the soil for water, as a result of the adsorptive and osmotic properties, acting in conjunction with the surface tension of water, gives rise to curved air-water interfaces. The pressure differential across these interfaces is given by $2t/r$ where t is the surface tension of water and r is the radius of the meniscus. Water deficiencies arising from mechanical effects in partially saturated systems will also be reflected through curved air-water interfaces. The relationships between radius of curvature, osmotic, adsorptive, pore air, and hydrostatic pressures in partially saturated soils have been recently analyzed by Mitchell (6).

Mechanical Components of Swelling

Though the effects to be considered below may appear purely physical in nature, it is important to keep in mind that they often depend on the affinity of water for soil surfaces and on water surface tension, both of which are physicochemical in origin. Particles within a soil mass may be deformed under load in two ways. The first, an actual elastic compression of solid particles, is probably insignificant within the practical range of loadings. In the second, volume changes resulting from the bending of platy clay particles may be appreciable. Particles could be held in bent positions by either external loading or through water surface tension effects. Release of the stresses by unloading or exposure to free water could result in expansion of the soil mass. Terzaghi (1) has demonstrated that the compression-expansion curve of almost any clay soil may be duplicated by a properly chosen sand-mica flake mixture. Physicochemical effects in such mixtures may be assumed negligible. By making reasonable assumptions concerning moduli of elasticity, it may be shown that a typical clay plate, acting as a simple beam between other particles, may be held by menisci in a deformed state where the deflection is about 10 percent of the span.

A mechanical component of swelling that may be of importance in compacted clays (partially saturated clays) arises from the compression of the air in the voids as water enters the soil during swelling. If there are many interconnected tubular air voids and the air is initially at atmospheric pressure, water entering the soil from all directions will cause the air to be compressed and exert a disruptive pressure along the walls of the tubular voids. If the soil structure is too weak to withstand these pressures, then expansion may occur. Such a phenomenon has been suggested as the cause of slaking when a dry sample is immersed in water without confinement.

In compacted clays both physicochemical and mechanical factors may be significant contributors to the swell or swell pressure that develops on exposure to water. A typical example of the influence of both types of factors is provided by the results of tests on an expansive sandy clay soil having a liquid limit of 35 percent, a plastic limit of 19 percent, 25 percent by weight finer than 2 microns and montmorillonite as the dominant clay mineral. Several samples of the soil were prepared by kneading compaction at a water content of 17.3 percent to a density of 111.3 pcf. The samples were then permitted to swell in solutions of calcium acetate under a surcharge pressure of 0.1 kg per sq cm. The results, plotted as percent swell vs calcium acetate concentration in Figure 1, show that the amount of swell decreases as the electrolyte concentration increases, but that it appears to reach a constant value of about 1.1 percent above a concentration of about 1.5 Normal. The higher swells at low electrolyte concentrations are attributable to osmotic pressure differences between the swelling solution and the fluid between clay particles. Evidently the concentration at the midplane between particles was of the order of 1.5 Normal.1/ Since some 1.1 percent swell appears to be independent of electrolyte content, it seems reasonable to attribute it to factors other than osmotic pressure. It is unlikely that water adsorptive forces would be significant at a water content of 17.3 percent (this soil has been observed to absorb water freely from the atmosphere to a water content of only about 5 percent). The basic swell of 1.1 percent is more likely due to a combination of the mechanical factors already outlined.

FACTORS AFFECTING THE SWELL AND SWELL PRESSURE OF COMPACTED CLAY AND THEIR MEASUREMENT

The brief summary of swelling mechanisms presented in the previous section would tend to indicate that a great variety of factors might influence the swell and swell pressure characteristics of a given soil. In the case of compacted clays, the molding water content and density would obviously be expected to be of paramount importance. The data in Figure 1 show that the nature of the imbibed fluid plays an important role. Somewhat less obvious but perhaps of equal importance is the method of compaction used for sample preparation. As illustrated by Seed and Chan (7), this is due to the fact that the method of compaction determines the structure of a compacted clay, and the structure influences the behavior of exposure to water. All of these factors may be considered under the general category of the influence of compositional factors on swell and swell pressure.

In order to evaluate properly the influence of the compositional factors, however, reliable and interpretable methods of measurement must be used. Studies have indicated that accurate determination of the expansion characteristics of a soil, in particular the swell pressure, is not always a simple task and that carefully controlled test conditions must be main-

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1/ The actual concentration is a function of the clay exchange capacity, particle spacing, pore fluid electrolyte content, and other factors. It cannot be measured directly.

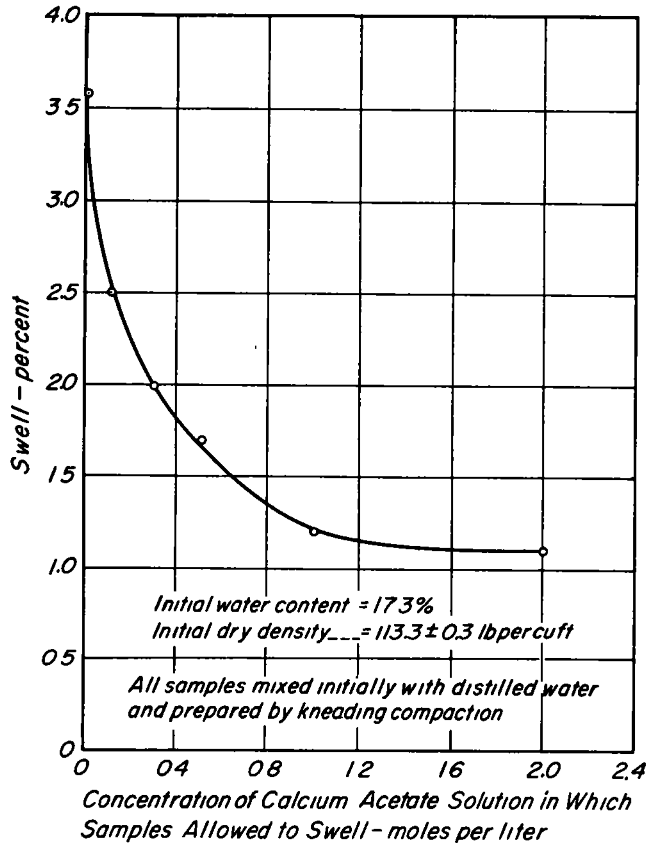

Figure 1. Effect of concentration of absorbed solution on swell of sandy clay

tained. Although, at the present stage, the effects of all of the variables in test condition that might reasonably be expected to influence the test results have not been evaluated, the influence of many factors has been reliably established. Included in the latter category are the effect of time on swell pressure development, the effect of sample shape on swell pressure, the effect of volume change on swell pressure, the effect of temperature on swell readings, and the influence of stress history on swell. All of these factors may be considered under the general category of the influence of measurement methods on swelling characteristics, and their effects are considered in the next section. Test results illustrating the influence of compositional factors are discussed in the following section.

THE INFLUENCE OF TEST CONDITIONS ON SWELL AND SWELL PRESSURE MEASUREMENTS

The Effect of Time on the Development of Swell and Swell Pressure

When a compacted soil is exposed to water, time is required for the movement of water into the sample under the hydraulic gradient set up by the negative water pressures within the soil relative to free water. The process is in many ways analogous to the process of consolidation wherein the movement of water in a loaded clay is retarded by the low permeability. A typical swell vs log time curve for a sample of compacted sandy clay is shown in Figure 2. It may be noted that for this relatively thin sample of 0.63-in. initial thickness, a considerable period is required for full expansion to occur. It may also be seen that the shape of the curve is essentially the reverse of a typical curve for consolidation.

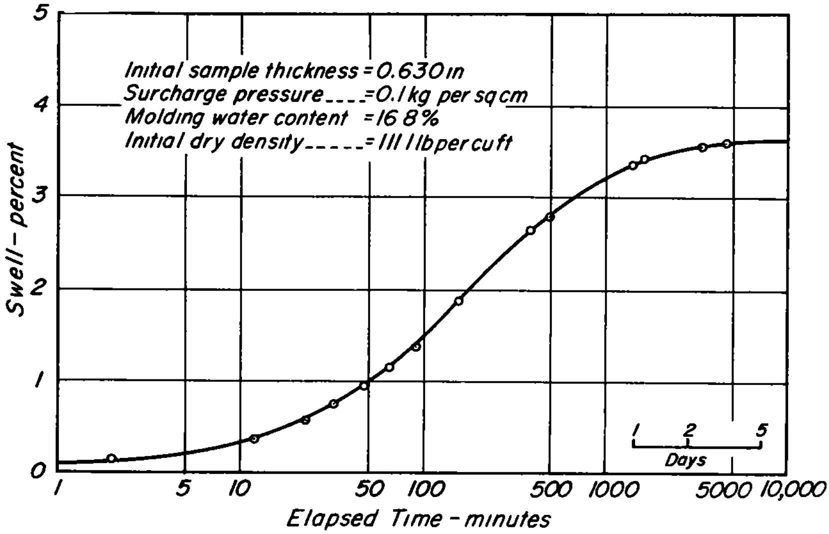

Figure 2. Increase in swell with time for thin sample of compacted sandy clay

The development of swell pressure on exposure of an expansive soil to water proceeds in much the same manner. A typical test result showing the rate of development of swell pressure for a sandy clay soil is shown in Figure 3. The procedure used for the measurement of the pressures shown in Figure 3 and for the other swell pressures obtained in this investigation was, unless otherwise indicated, as follows: Samples were mixed to the desired water content and compacted in 4-in. diameter molds, using a kneading compactor to form specimens approximately 2.5-in. high. The samples were then subjected to static pressure until moisture was exuded. The pressure was then released and the sample allowed to stand for half an hour. A perforated plate with a vertical stem was placed on top of the sample, and the mold containing the sample was placed in an expansion pressure device (see Fig. 4) so that the stem of the plate firmly contacted the center of a horizontal proving bar fixed at each end.[2] A dial gage was mounted to record subsequent deflections of the proving bar. Water was poured on the upper face of the specimen and the pressure that developed as the sample tended to expand was observed by noting the deflection of the proving bar. Calibration of the proving bar permitted computation of the expansion pressure. It may be noted that this is the same procedure used by a number of State Highway Departments in connection with the design of pavements.

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[2] A seating load of 0.4 psi was used.

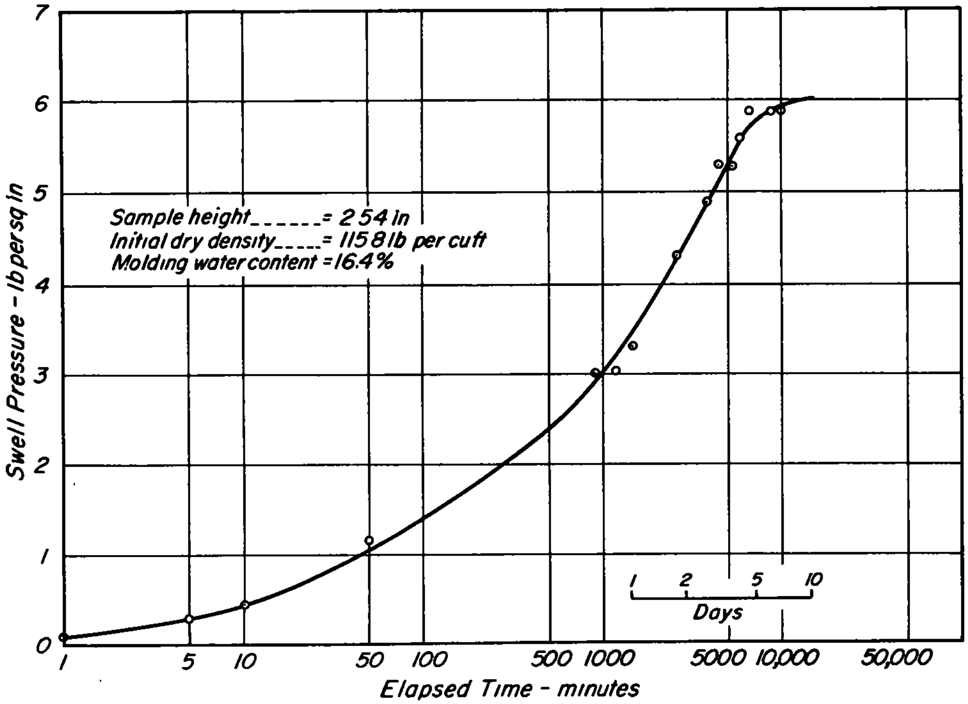


Figure 3. Increase in swell pressure with time after exposure to water for compacted sample of sandy clay

It should also be noted, however, that in this procedure the sample is not maintained at constant volume, inasmuch as the proving bar must deflect upwards to measure the expansion pressure, and the sample is thus allowed to expand by an amount equal to the deflection of the proving bar. Thus, while the actual sample expansion is quite small (0.003 in. per psi or less depending on the thickness of the proving bar used), the true swell pressure at zero volume change is not measured, but rather an expansion pressure corresponding to some small amount of swell is determined. The marked effect of very small volume increases on the observed pressures will be illustrated in a subsequent section.

From Figure 3 it may be seen that the expansion pressure increased relatively rapidly for the first day but that the rate of increase was very low after this time. At the end of one week, however, the pressure had reached a value significantly higher than after one day. Figure 5 shows the results of a series of tests on the same sandy clay conducted over a range of water contents likely to be of greatest practical interest for pavement design considerations (expansion pressures less than 1.5 psi). The pressure developed after 7 days was at least 100 percent greater than the pressures developed after 1 day. This would seem to indicate that even when samples are compacted to a condition approaching full saturation, considerable periods of time are still required to determine ultimate values of swell pressure if samples of 2.5-in. thickness are employed.

Similar time effects have been reported by DuBose (8) for the development of swell in samples of highly plastic clay. For samples 5 in. in thickness, full swell was not developed even after 50 days.

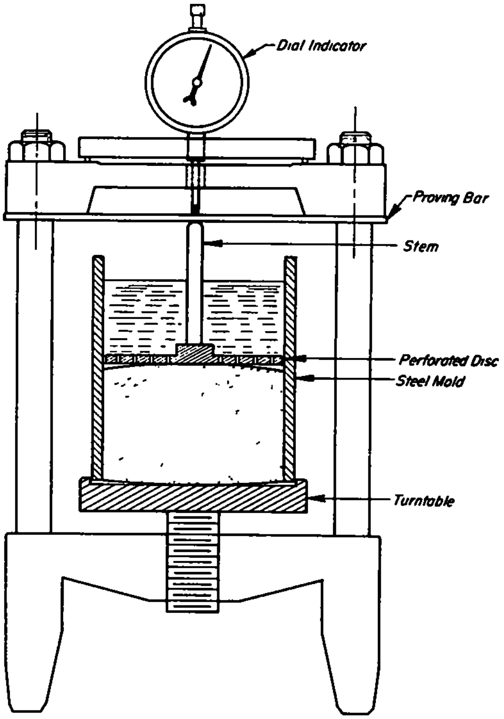


Figure 4. Sample curvature in expansion pressure test

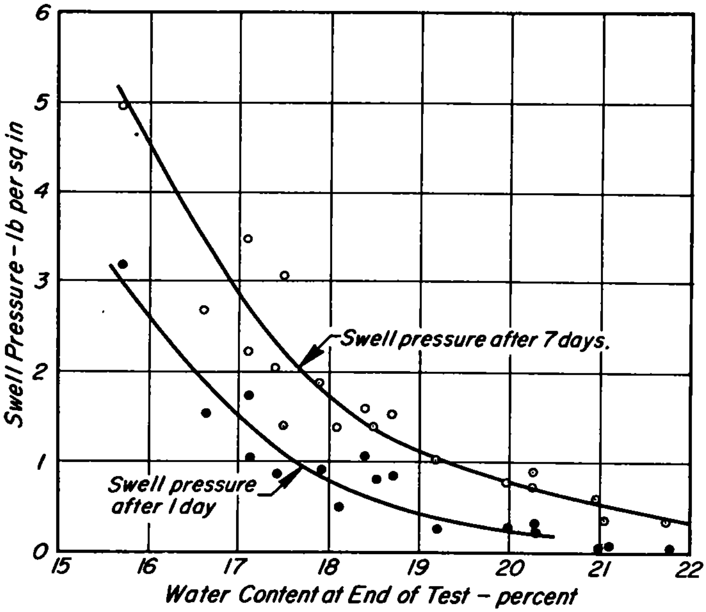


Figure 5. Effect of time on expansion pressures developed by samples of sandy clay

The long time required for the development of full swell or swell pressure is a result of the time required to effect the stress transfer in the water from a large negative value existing before exposure of the sample to water up to the zero value that must exist throughout the system at equilibrium. The pore-water-pressure change requires the flow of water into the partially saturated soil in order to flatten menisci and satisfy water adsorptive forces. The process is slow due to the low permeability of compacted clays; and the rate decreases with increasing time because of the continually decreasing gradient tending to draw water in. Unfortunately, after about 24 hours the rate of change of swell or swell pressure may become so slow that observers are tempted to conclude that the process is completed; yet considerable swelling may still develop after the initial 24-hour period. Considerable care is required to determine the maximum values for any given test.

The Effect of Sample Shape on Swell Pressures

It was observed, using the procedure previously described, that in some cases the deflection of the proving bar decreased rather than increased following the addition of water to the sample — a confusing result in view of the extremely light seating load to which the samples were subjected. These results remained unexplained until it was noticed that these samples developed considerable curvature of their upper and lower surfaces in the half-hour waiting period following the release of the static compactive load. This curvature is likely to occur in all samples but more particularly with those exhibiting considerable rebound after static compaction, because friction between the sample and walls of the mold will tend to restrict the rebound at the edges of the sample. At the center, greater rebound is possible because of the absence of edge effects.

It was concluded that for these samples the perforated plate was not in contact with the full surface of the sample, as shown in Figure 4, and that the decrease in proving-bar deflection was due to a progressive decrease in curvature caused by the seating load (the expansion of the sample due to the addition of water being insufficient to offset this effect). Thus it would seem reasonable to conclude that even in cases where expansion pressures were recorded, the sample actually expanded appreciably to fill the gaps between the original curved surfaces of the specimen and the horizontal base plate and the perforated plate before any expansion pressure could be developed near the edges of the specimen. As a result, the observed pressures would be appreciably less than those that would be developed by a sample in contact with its restraining boundaries over its entire surface area.

To check the validity of these conclusions, samples of sandy clay were prepared using the procedure previously described, and curvature was allowed to develop during the period following static compaction. Just before placing the samples in the expansion pressure device, the soil at the curved ends was trimmed off to form plane surfaces. The expansion pressures developed by these trimmed samples are compared with those of identical untrimmed samples in Figure 6. The similar compositions of both trimmed and untrimmed samples are indicated by the plot of dry density vs water content at the bottom of Figure 6.

The results indicate that the samples whose ends were trimmed prior to testing developed considerably higher pressures than did the untrimmed

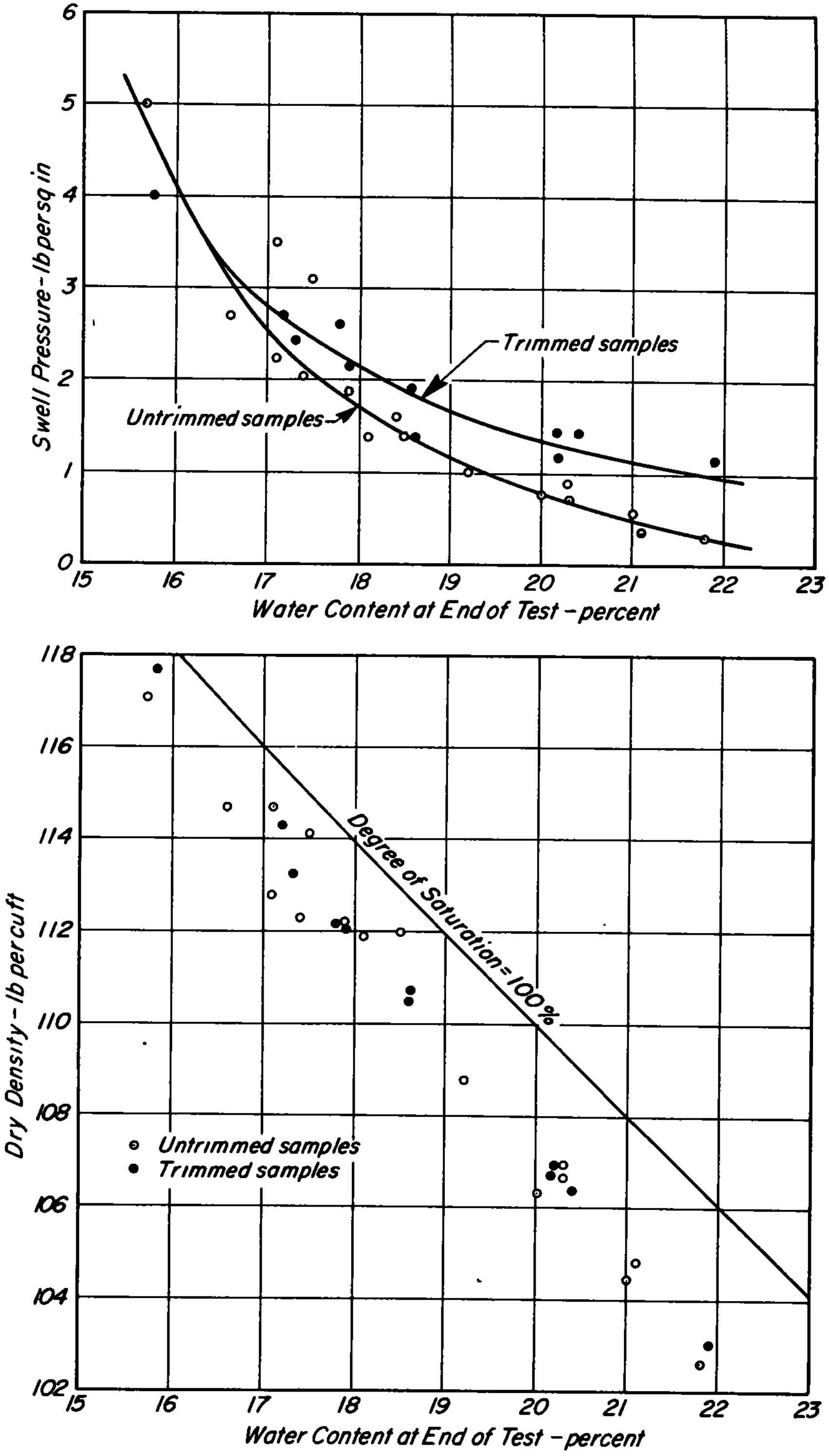


Figure 6. Effect of trimming on swell pressures developed by compacted samples of sandy clay

specimens. The difference between the expansion pressures developed by trimmed and untrimmed specimens increases with water content within the range investigated, and in some cases was as great as 100 percent. The effect is greatest at the higher water contents. This is apparently due to the fact that at the lower water contents there is little rebound of the sample after removal of the static pressure and, thus, little tendency for curvature of the ends to develop. This lack of curvature in the drier samples was evident during the tests. Similar results have been obtained for samples of silty clay prepared in the same manner.

It is thus apparent that the determination of reliable swell pressures is dependent on insuring perfectly plane sample surfaces and is particularly important for statically compacted samples or any samples subjected to static pressures. Unfortunately, however, trimming the ends of a thick sample prepared in this way may not be the proper procedure for obtaining satisfactory results, because the removal of material from the center part of the sample will result in a specimen of nonuniform density, and the swell pressure exerted by the edges may then exceed that exerted by the center portion; furthermore, some pressure may result from the expansion of the soil due to the gradual release of friction forces between the sample and the walls of the mold.

The problem of curvature at sample ends is not the only difficulty encountered in making reliable determinations of swell pressures for compacted samples. Barber (9) has presented data illustrating the marked differences in results that may be obtained due to variations in the interval of time between compaction and the start of the test. Typical results are presented in Figure 7. It is apparent that this factor needs careful consideration in the correct interpretation of swell pressure data.

The Effect of Volume Change on Swell Pressure

The significant effects of volume change on the swelling pressure developed by compacted soils has been recognized for some time. Barber (9), Dawson (10), and DuBose (8), among others, have presented data illustrating the marked reduction in swell pressures caused by volume changes of even a fraction of 1 percent. Typical results reported by Dawson (10) for samples of silty clay compacted by the standard Proctor procedure are presented in Figure 8.

It is apparent that a careful assessment of the amount of expansion permitted in any test is required in the evaluation of the test data, and that comparative test results can only be obtained where volume changes are the same for all soils. Probably the most convenient and often the most significant standard of comparison is that in which samples are maintained at constant volume. However, many test procedures do not satisfy this requirement, and it is of interest to compare swell pressure data for samples maintained at constant volume with those obtained using proving bar procedures. The proving-bar method of swell pressure measurement must necessarily permit some sample expansion, because the deflection of the bar forms the basis for evaluation of the pressure. Though only a very slight expansion of samples occurs by this or other usual methods of swell pressure measurements, the measured pressures are likely to be somewhat lower than those that would be developed if constant volume were maintained.

To investigate the influence of such effects on the expansion pressures developed by samples of sandy clay, three series of tests were run,

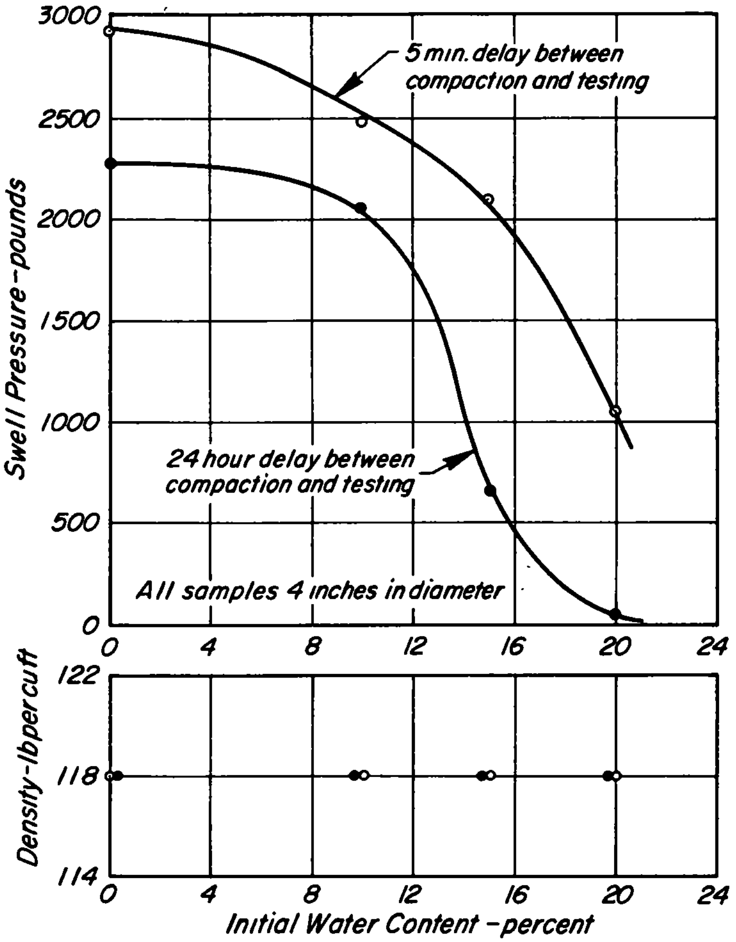


Figure 7. Effect of interval between compaction and testing on swell pressure recorded at constant volume. (Values from E. S. Barber, 1956.)

using the swell pressure device shown in Figure 4 with proving bars of 1/8-in. thickness and 1/4-in. thickness and using a loading yoke instead of a proving bar over the stem of the perforated plate to maintain constant volume. For the series maintained at constant volume, the sample faces were trimmed prior to the addition of water; data were obtained for both trimmed and untrimmed specimens using the 1/8-in. proving bar; and data on untrimmed specimens only were obtained with the 1/4-in. proving bar. With the 1/8-in. bar an expansion pressure of 1 psi caused a deflection of 0.003 in.; with the 1/4-in. bar the deflection was 0.0004 in. per psi; and using the loading yoke, constant volume was essentially maintained by application of appropriate loads. A slight volume increase is possible with the loading yoke due to the elastic compression of the vertical stem above the perforated plate. Compression of the stem was computed to be 0.000376 in. per 10 psi expansion pressure exerted by the 4-in. diameter samples.

Figure 9 shows the relationship between swell pressure at the end of 7 days and water content for the four conditions of test. It may be seen

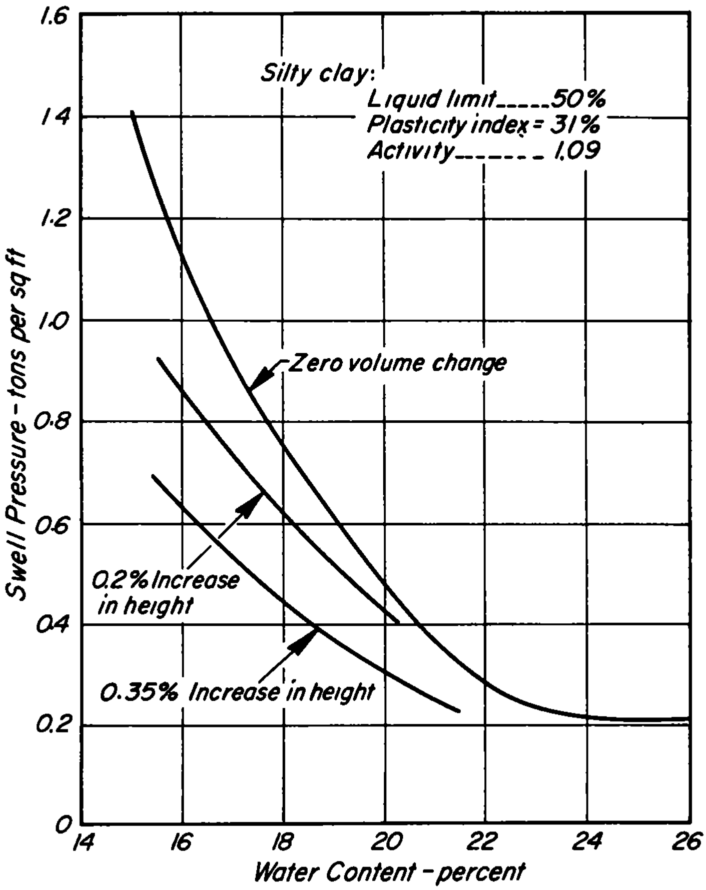


Figure 8. Effect of volume change on measured swell pressures. (After R. F. Dawson, 1956.)

that the measured swell pressure is very sensitive to the amount of volume change actually allowed to take place during the measurement. It is evident that the measured pressures, even in a relatively stiff system (e.g., with a 1/8-in. proving bar) are much lower than the pressures measured under constant volume conditions.

In cases where the relationship between amount of swell and swell pressure is desired, suitable results can be obtained using the proving-bar type of equipment with bars of different thicknesses in conjunction with the loading yoke apparatus. Typical data obtained in this way for samples of sandy clay, prepared by kneading compaction at various water contents using a constant compactive effort, are presented in Figure 10. The density vs water-content curve for the samples studied is shown in the lower part of the figure.

The soil was compacted in 4-in. diameter molds to form samples 4.5 in. in height; after compaction, the samples were trimmed off to leave a specimen with a height of 2.5 in. in the mold. Three sets of samples were tested. In the first, the samples were given access to water with a 3/16-in. thick proving bar used to measure expansion pressures; in the second, a 3/8-in. thick proving bar was used; and in the third, the samples were confined between the base plate and a loading yoke with a lucite piston. In the third set, sufficient pressure was applied to the yoke to prevent

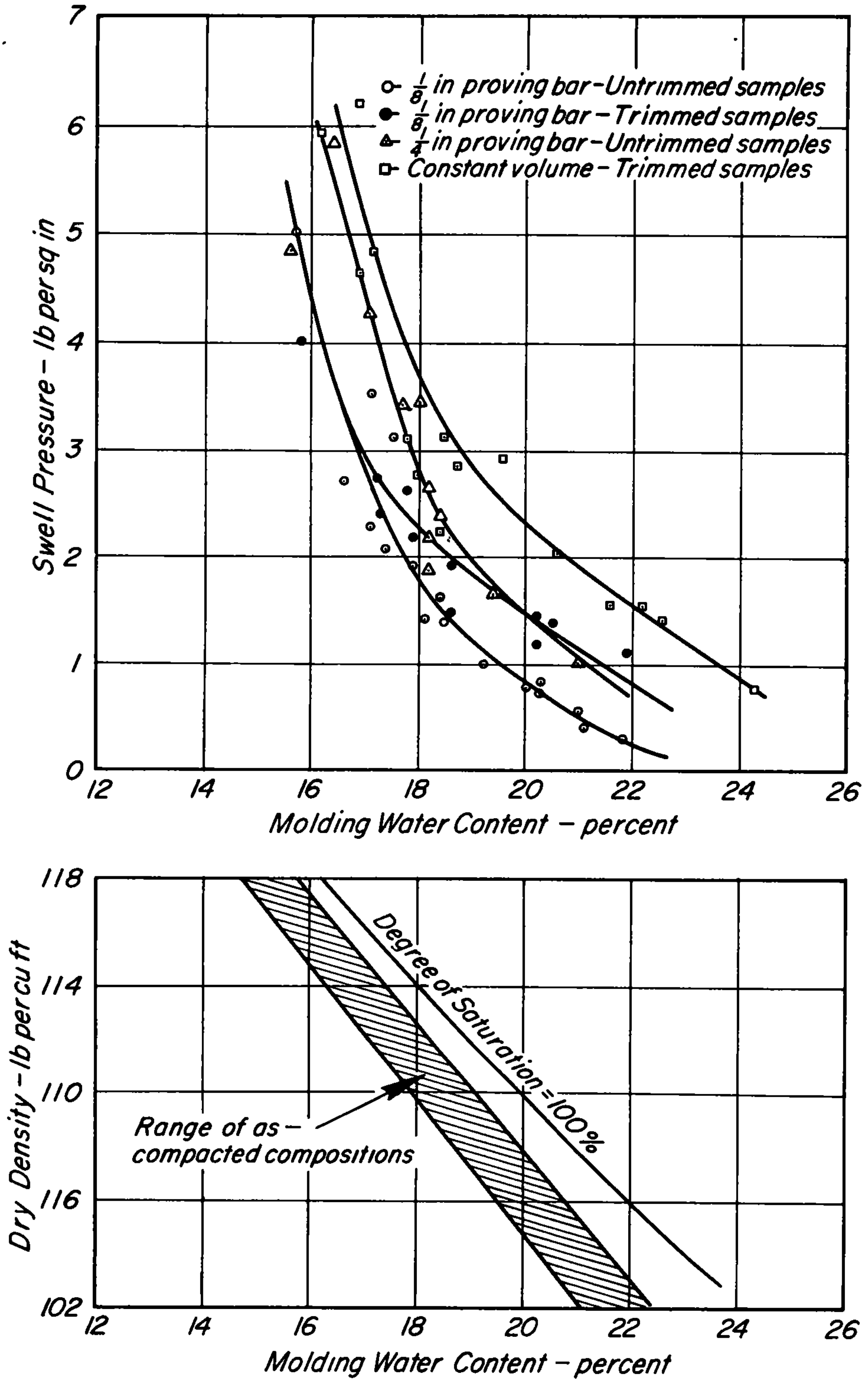


Figure 9. Effect of sample restraint and trimming on the swell pressures developed by compacted samples of sandy clay.

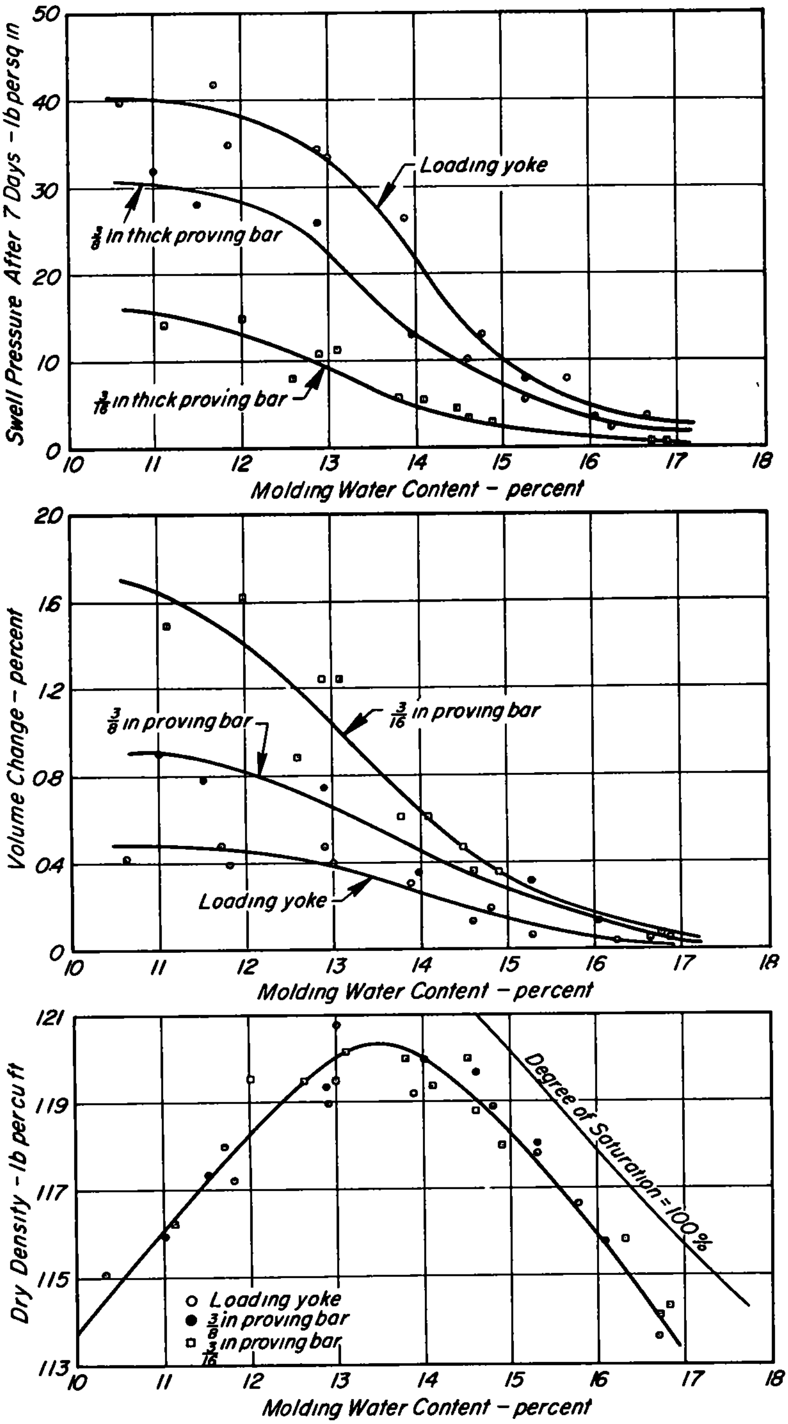


Figure 10. Effect of volume change on swell pressure of compacted sandy clay.

apparent expansion of the specimen as indicated by a dial gage mounted on the yoke. Actually, however, a slight volume change could occur due to compression of the lucite piston. The magnitude of this volume change could be evaluated and is included in consideration of the test results.

The expansion pressure after 7 days and the volume change in percent for each of the samples are shown in Figure 10 as functions of molding water content. It may be seen that the samples having the lowest water content at compaction had the highest expansion pressures, and the greater the expansion that is permitted during the test, the lower is the measured pressure.

The results in Figure 10 have been used to prepare Figure 11, which shows the expansion pressure as a function of volume change at different compaction water contents and densities. Presentation of the data in this form provides a convenient means for assessing the magnitude of swell pressure for any degree of expansion and would be useful in practical situations where, very often, the swelling pressure against a structural member and the deflection of the members are mutually dependent.

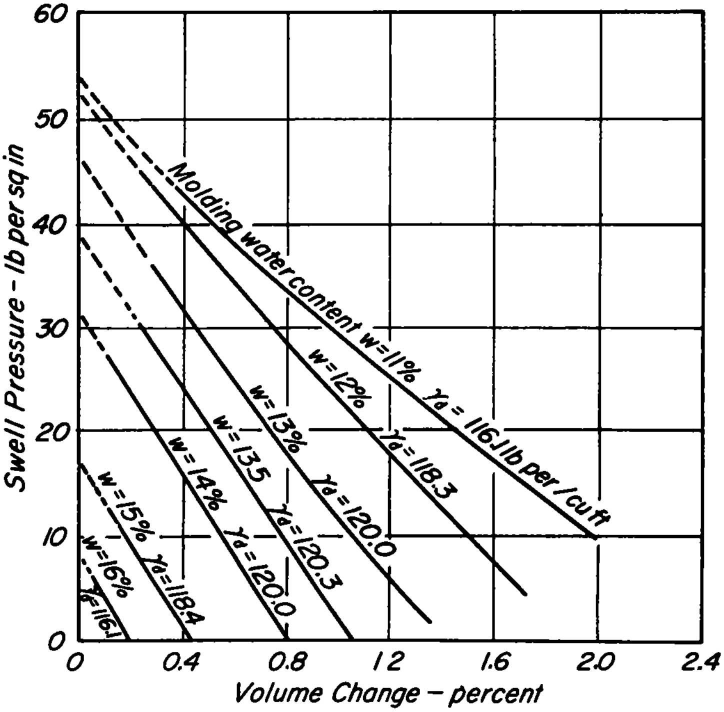


Figure 11. Effect of volume change on swell pressure developed by samples of sandy clay prepared using constant compactive effort.

Effect of Temperature on Swell and Swell Pressure Measurements

An additional factor affecting the results of swell pressure measurements is temperature variation during the test. The magnitude of swell and swell pressure is a function of temperature for purely physicochemical reasons, as the interparticle forces and pore water pressures are known to be temperature sensitive. Exclusive of the influence of physicochemical factors, however, temperature effects are also important through their influence on the volume of the components of the measuring system.

It has been found, for example, that temperature variations of 10 to 15° F can cause a change in observed swell up to 0.5 percent in sample of 1/2-in. thickness. Differences of this magnitude may be intolerable if the specimens being investigated only swell 1 or 2 percent. While reliable data have not yet been obtained concerning the effect of temperature variations on swell pressure, it might be expected to be rather large, because as shown in Figure 11, very small volume changes lead to large changes in swell pressure.

Effect of Stress History on Swelling of Compacted Soils

The relationships between the amount of swell that will develop when a compacted sample is exposed to water and the surcharge pressure resisting expansion is often of interest in practical problems. The labor involved in determining such a relationship could be appreciably reduced if the same sample could be used for determination of several points on the percent swell vs surcharge pressure curve by means of successively reducing the surcharge load after equilibrium has been reached under the previous load. However, test results show that the amount of swell is significantly influenced by the stress history of a sample, and no such simple procedure can be used.

The effects of stress history in this type of test are demonstrated by the results of the following study: Three compacted specimens of 4-in. diameter all having the same initial composition of 13.1 percent water content and dry densities of 122.5 to 122.8 pcf were trimmed to a thickness of 3/4 in. and placed between porous discs. Surcharge pressures were applied and measurements made of the amount of swelling when the samples were immersed in water. Specimen No. 1 was tested under an initial surcharge of 10 psi. After equilibrium had been achieved, the surcharge pressure was reduced to 5 psi; after a second equilibrium was attained, the pressure was further reduced to 1 psi and swell measurements continued for an additional 28 days to insure ultimate equilibrium. Specimen No. 2 was subjected to an initial surcharge pressure of 5 psi, and, when no further swell appeared likely, the surcharge was reduced to 1 psi. Specimen No. 3 was maintained under 1 psi throughout the test.

Figure 12 summarizes the changes in thickness occurring in each of the samples throughout the test period. It may be seen that the amount of swell of Specimen No. 1, on which the load was reduced in stages from 10 to 1 psi, was less than that of Specimen No. 2, on which the stress was reduced from 5 to 1 psi, and that both of these specimens swelled less than Specimen No. 3 which was subjected to only 1 psi throughout.

The results of a similar series of tests reported by Barber (9) for a clay of low plasticity are presented in Figure 13. For these studies, compacted samples were placed under initial pressures of 8,000, 1,000, 500, 200, and 20 psf and allowed to swell to equilibrium conditions; the pressure on all samples was then reduced to 20 psf and the final equilibrium conditions determined. The marked differences in final expansion resulting from the use of different initial pressures are readily apparent.

It is clear from these results that stress history does have a significant effect on the amount of swell of a compacted soil and that erroneous results may be obtained if the equilibrium conditions of compacted specimens after swelling are predicted from measurements made by progressively reducing the surcharge pressure on a single specimen. The results also indicate that if a sample is used to measure swell pressure it cannot then

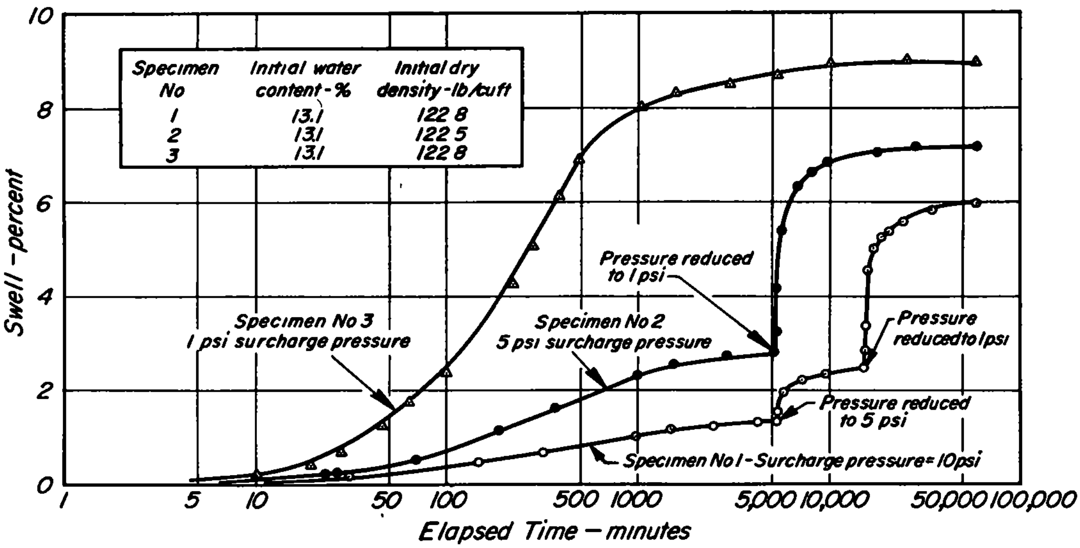


Figure 12. Effect of stress history on swell of compacted samples of sandy clay.

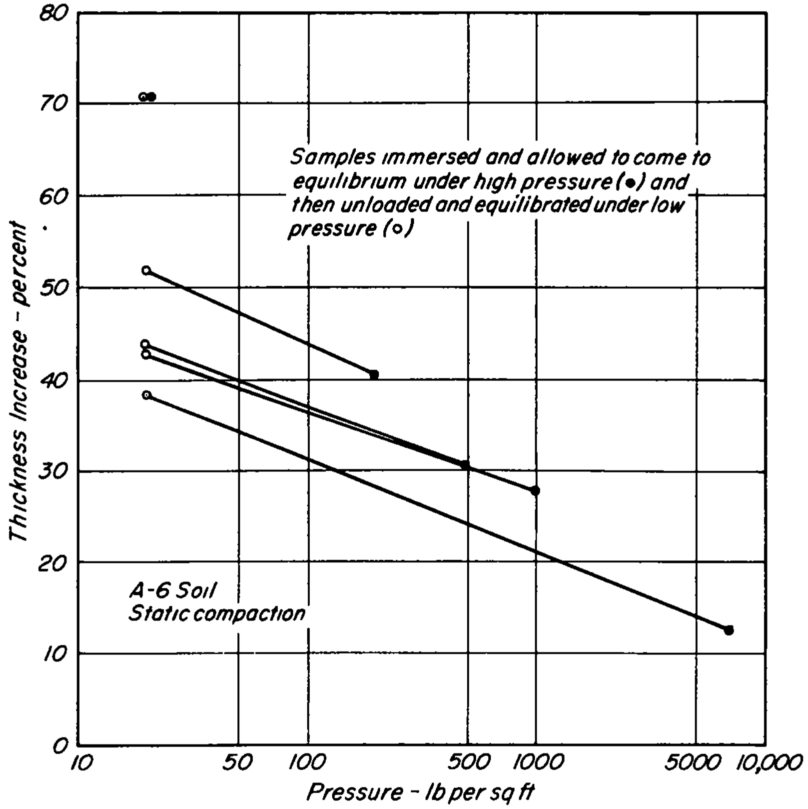


Figure 13. Swell vs pressure relationships for different initial loads. (Data from E. S. Barber, 1956.)

be unloaded and allowed to swell in the hope of determining the swelling characteristics of the soil. Separate samples are required for swell pressure and swell measurements.

Recent developments in the study of the structure of compacted clays provide a possible explanation for this type of behavior. It has been shown, Lambe (3), Seed and Chan (7), that samples compacted dry of optimum, as were the samples used to obtain the data in Figure 12, exhibit flocculent structures; i.e., the clay particles are more or less randomly oriented. The pore-water pressures are negative and of rather large magnitude in such soils, and it is likely that many of the platy particles are held in a deformed state by menisci. On exposure to water, the relief of these high water tensions enables deformed particles to straighten, water adsorptive forces to be satisfied, and double layer osmotic pressures to be balanced. It is reasonable to assume also that some rearrangement of the internal structure of the soil will take place as water is imbibed and the tensions in the water are released. As water moves in, deformed particles tend to straighten and effective particle spacings tend to increase. The form of the structure adjustment will be a function of the restraints applied to the sample and could reasonably be expected to take the path of least resistance.

When sample expansion can take place with ease, as in the case of low surcharge pressures, then large over-all volume increase of the specimen may occur and a relatively high swell may result. When vertical expansion is restrained by the application of high surcharge pressures it is likely that internal particle reorientations occur such that the initial structure, which could be considered to consist of clusters or aggregates of randomly oriented particles, shifts in the direction of a more dispersed or parallel structure as the dissipation of stresses and the movement of particles proceed. The tendency for particle orientation to become more parallel in a direction normal to the direction of stress application when saturated clays are subjected to high pressure has already been discussed by Lambe (3), while Seed and Chan (7) have shown that parallel particle orientations exhibit lower swelling characteristics than random orientations. Thus, it seems likely that the subsequent swelling characteristics of samples previously subjected to high pressures are less than those of identical samples subjected directly to low pressures because of the increased degree of particle orientation induced by the high pressure application.

If the above mechanism were correct, it would be expected that the initial structure of a compacted clay would influence the magnitude of the stress history effect. It would be further anticipated that the stress history effect would be more pronounced in samples having flocculent structures than in samples having dispersed structures, because internal particle adjustments in dispersed systems might be expected to be fairly small.

THE ROLE OF COMPOSITIONAL FACTORS IN DETERMINING THE SWELL AND SWELL PRESSURE CHARACTERISTICS OF COMPACTED CLAYS

As pointed out at the outset of this paper, the swelling characteristics of a compacted soil are dependent on several fundamental factors. The relative influence of these factors for a given soil is, in most cases, determined by more or less controllable conditions that are imposed on the composition of the compacted soil. By composition of a compacted soil is meant the water content, density, and structure of the material in the

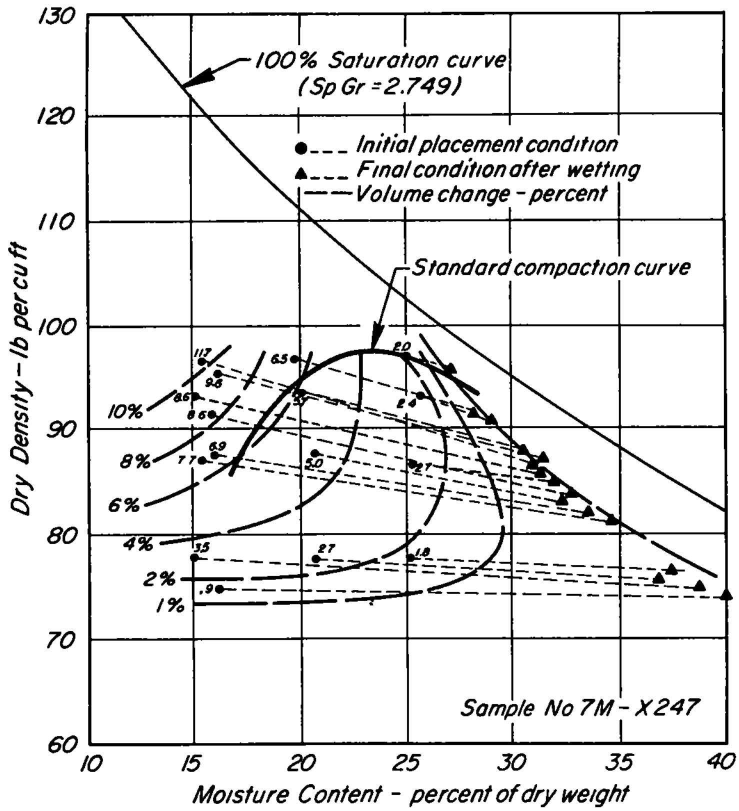


Figure 14. Percentage of expansion for various placement conditions when under 1-psi load. (After W. G. Holtz and H. J. Gibbs.)

as-compacted state. The composition of the fluid to which the compacted soil is exposed during swelling is a further important variable as shown by Ladd (4) and Mitchell (6). The structure is, in many cases, determined by the method of compaction and the molding water content, as illustrated by Seed and Chan (7) and Lambe (3). The influence of these various compositional factors is illustrated in the following paragraphs.

Influence of Density and Water Content on Swelling Characteristics

The influence of compacted density and molding water content on the swell and swell pressure of compacted clays has been studied by numerous investigators in recent years. Probably the most convenient way of presenting the results of such studies is that of plotting contours of equal expansion effects on a standard dry-density vs water-content plot. Typical examples of such plots, obtained by Holtz and Gibbs (11), are presented in Figures 14 and 15. These data were obtained from studies of the relationships between compacted density, water content, and swelling characteristics of samples of a highly plastic clay prepared by impact compaction. It is readily apparent that for this method of compaction an increase in molding water content at any given density causes a decrease in swell and swell pressure; however, an increase in density at any given water content may increase or decrease the swell depending on the range of densities involved. This latter effect is due to the fact that changes

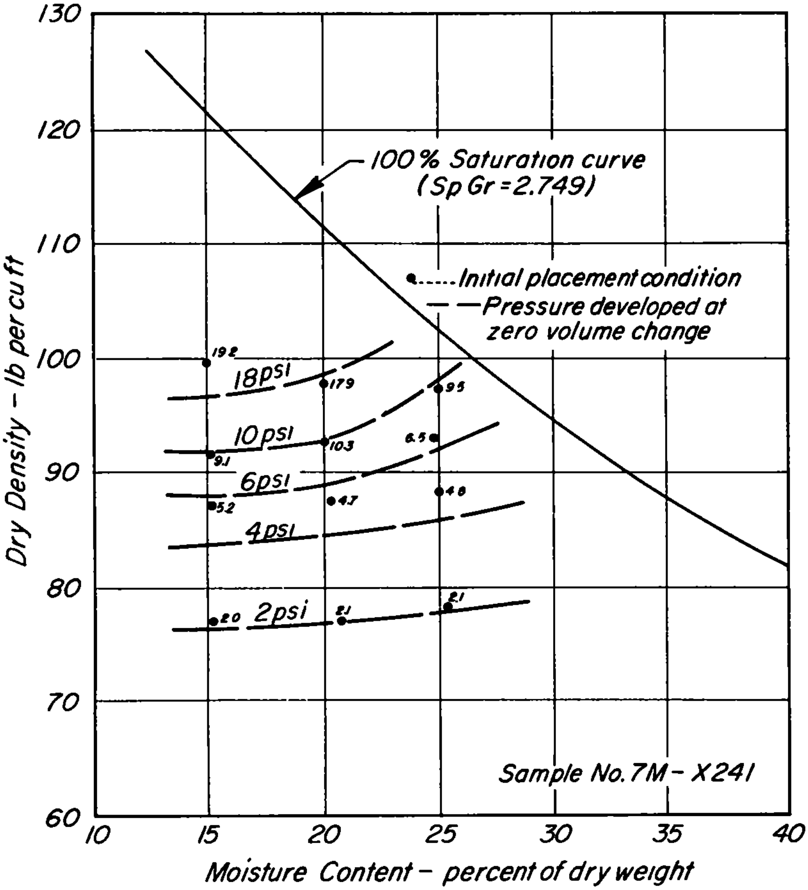
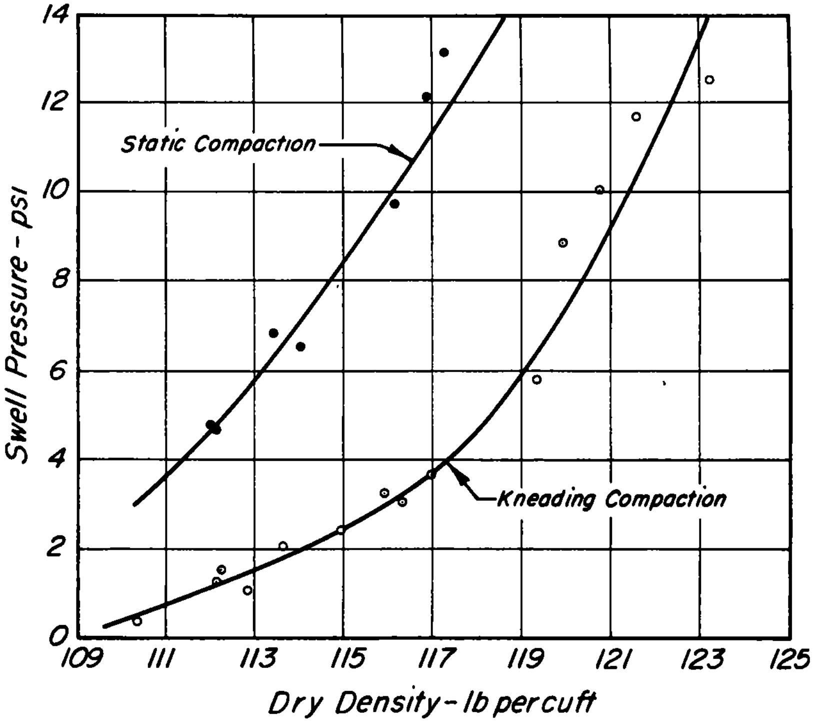


Figure 15. Total uplift pressure caused by wetting — for various placement conditions. (After W. G. Holtz and H. J. Gibbs.)

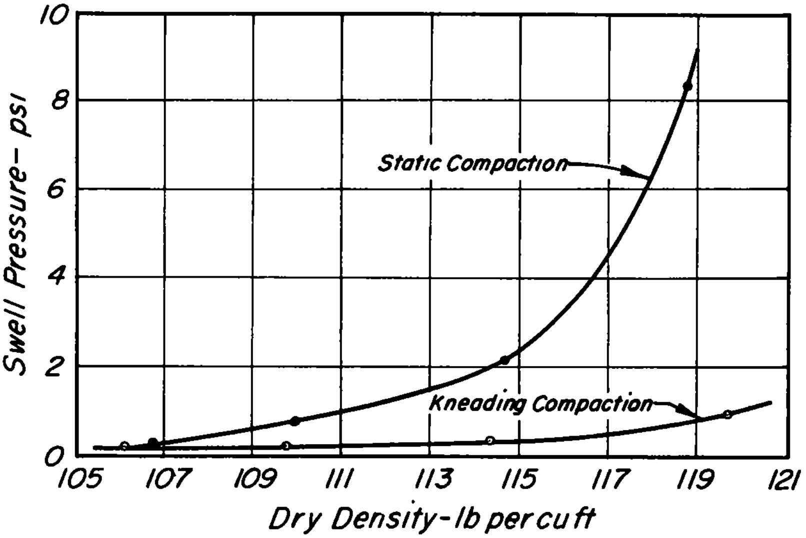
in density, at high degrees of saturation, are accompanied by changes in soil structure, and the swelling characteristics reflect the effects of both density and structural changes.

The Effect of Soil Structure on Swelling Characteristics

In addition to the initial density and water content, the swell and swell pressure of compacted samples are greatly influenced by their initial structures. As shown by Seed and Chan (7), for many soils the method of compaction provides a simple method for the inducement of different structures in compacted samples at identical water contents and densities. However, such structural variations generally can only be obtained at water contents greater than optimum for the compactive effort and procedure used. Under those conditions it has been shown that kneading compaction tends to create a dispersed structure and static compaction a flocculent structure in many clays. The influence of such structural differences on the swell of samples of sandy clay is shown by the data in Figure 16. It may be seen that for samples compacted dry of optimum, the swell is relatively insensitive to method of compaction, because both methods yield flocculent structures. Wet of optimum, however, the flocculent sample prepared by



(a) Pittsburg Sandy Clay



(b) Vicksburg Silty Clay.

Figure 16. Effect of method of compaction on swell pressure for samples compacted to high degree of saturation.

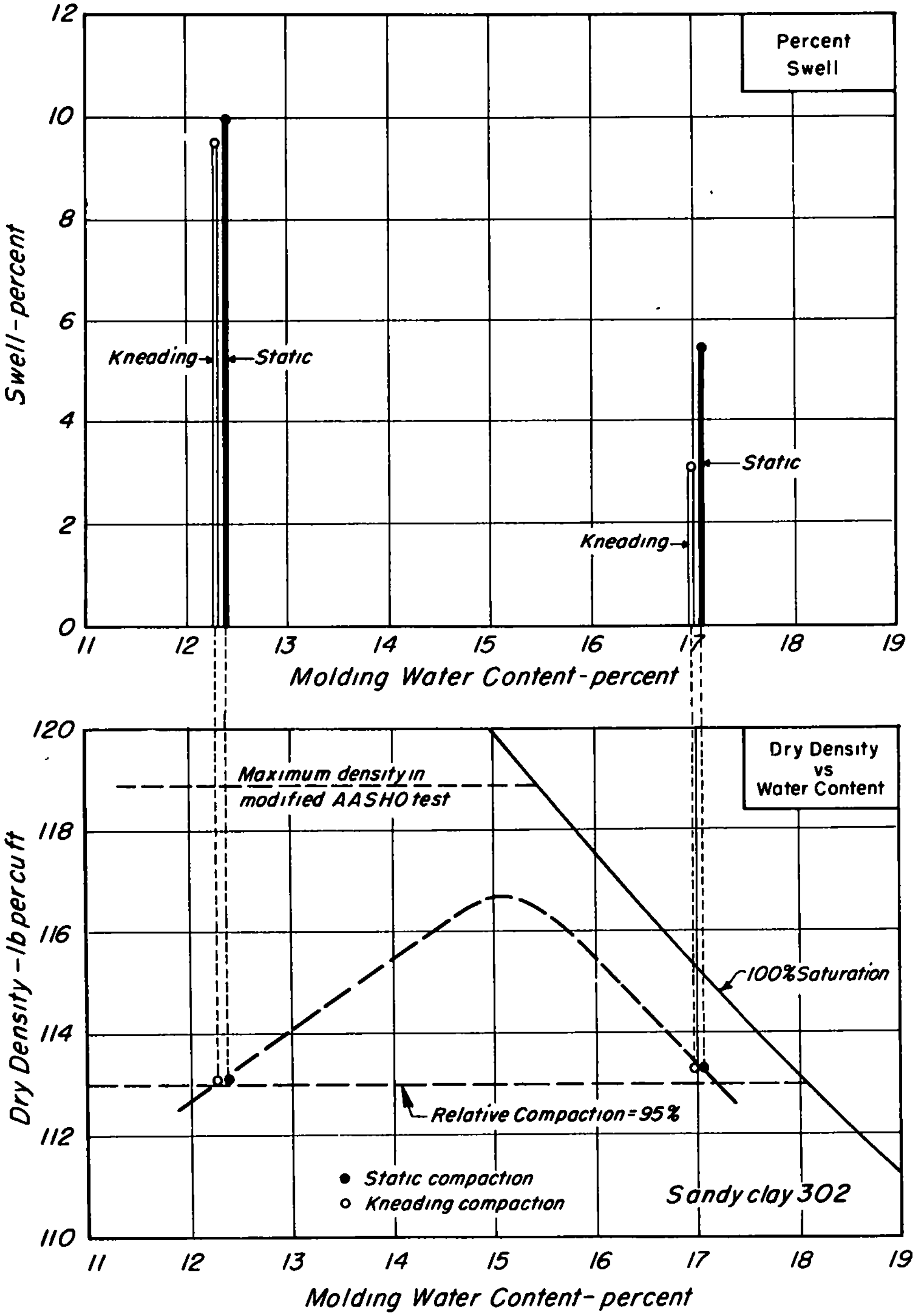


Figure 17. Swell characteristics of samples of sandy clay prepared of kneading and static compaction.

static compaction swells considerably more than the sample with a dispersed structure prepared by kneading compaction. That the swell pressure is similarly affected by structure is illustrated by Figure 17, which presents data for samples of two clays prepared at various densities wet of optimum by kneading and static compaction. The swell pressures of the statically compacted samples are greater than those for the kneading compacted samples at the same molding water content over the entire range of densities investigated.

The Effect of Swelling Solution Composition on Swell

The effect of the composition of the solution to which a compacted soil is exposed on the swelling characteristics has been discussed in connection with swelling mechanisms and Figure 1. Similar measurements have been reported by Ladd (4). Such behavior (i.e., decreased swell with increased electrolyte content) might be expected with any soil in which physicochemical components, particularly osmotic pressures, are significant contributors to the swell. Figure 1 presents data for samples prepared by kneading compaction. These data are compared in Figure 18 with the results of similar tests on samples prepared by static compaction to the same

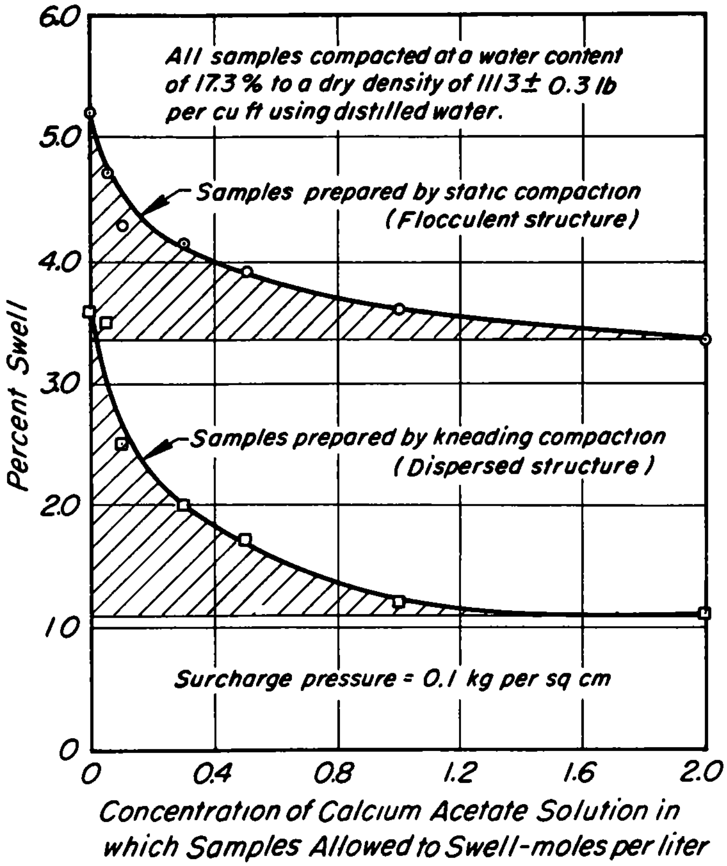


Figure 18. Effect of structure and electrolyte concentration of absorbed solution on swell of compacted sandy clay.

density and water content. It may be noted that the effect of solution composition, shown by the shaded areas, is about the same for each method of compaction, suggesting that the electrolyte-sensitive factors influencing swell are relatively insensitive to structure. The greater overall swell of the statically compacted samples is readily apparent.

Discussion of the Relationships Between Composition and Swelling Characteristics

The behavior summarized in Figures 14 through 18 appears to form a consistent picture relative to the known characteristics of compacted clays, physicochemical principles, and soil structure. Structure appears to be one of the major variables governing swelling behavior. The data in Figures 16, 17, and 18 show that flocculent structures consistently swell more and develop higher swell pressures than dispersed structures. Study of Figure 18 and of data presented by Ladd (4) indicates that the proportion of the swell that is insensitive to the electrolyte content of the solution to which the sample is exposed is much greater for flocculent than for dispersed samples. If it is assumed that the effect of the electrolyte content is a measure of the physicochemical components of the swell, then it would seem that mechanical factors are primarily responsible for the difference between the swelling characteristics of the two structures. Compaction to a flocculent structure might be expected to give a greater number of deformed particles and higher internal stresses because of the random particle arrangements and the inability of particles to slide into unstressed parallel positions, as is the case by kneading compaction.

The increase in swell and swell pressure with increase in density at a given water content is a logical consequence of several factors. In low density samples, the particle deformations would be expected to be less than in high density samples in the as-compacted state. The average interparticle spacing of low density samples is greater than in high density samples; thus, interparticle repulsive forces due to interacting double layers would be less, and, therefore, the samples would require less water to reach an osmotic equilibrium. Furthermore, structural adjustment within the samples could take place more easily in low density samples.

Both swell and swell pressure decrease with increasing molding water content because structures are generally more dispersed at higher water contents for any method of compaction, and the natural desire for the soil to imbibe water to satisfy adsorptive and double layer pressures decreases with increasing water content.

The form of the relationship between swell pressure or swell and molding water content for samples prepared at constant compactive effort is the result of variations in the combined effects of density, water content, and structure. At water contents a few percent below optimum, the swell is reasonably insensitive to changes in molding water content. This is because the density increases with increasing water content, and the effects of the two variables tend to cancel each other out. At water contents in the vicinity of optimum, the density is not changing rapidly and the effects of water content and structure predominate. Structure undergoes a marked shift towards greater dispersion as optimum water content is reached. Thus the swell and swell pressure decrease rapidly as the water content changes from just below to just above optimum for the compactive effort being used. When water contents considerably wet of optimum are reached, further structural changes have more or less ceased and physical interaction between particles becomes low. Physicochemical effects and the effects of decreased density are probably the important factors influencing the swelling characteristics in this range, and their influence is shown by a gradual decrease in swell and swell pressure as the water content increases.

SUMMARY AND CONCLUSIONS

The factors responsible for the swelling characteristics of compacted clays have been briefly reviewed. These factors may be conveniently considered to be of two types: physicochemical factors, which are functions of interparticle electrical forces, particle surface structure, pore fluid composition, the surface tension properties of water, and the composition of the water to which the soil is exposed; and mechanical factors, which include the effects of elastically deformed particles or particle groups and the compression of air in the voids during imbibition of water.

Some of the factors affecting the results of swell and swell pressure measurements have been examined. It was shown that considerable attention to detail is necessary if meaningful values of swell pressure characteristics are to be determined. Periods up to a week or longer are often necessary for the determination of equilibrium swell and swell pressure in compacted samples of appreciable thickness (2.5 in.), even if the initial degree of saturation is over 90 percent. Data have been presented that show that, for samples prepared by static compaction, curved surfaces may develop during the period between the end of compaction and the beginning of a swell pressure determination as a result of the rebound of the central portion of the sample. The effect of the curved surface is to give a swell pressure less than the true pressure. Furthermore, changes in the interval between compaction and testing may also influence the results.

The extreme sensitivity of measured swell pressure to volume change was illustrated. For the samples studied, for example, the measured pressure may be as much as 100 percent too low if as little as 0.1 percent expansion occurs, depending on the initial conditions. The combined effects of sample surface curvature, insufficient time of test, and expansion can lead to observed values of swell pressure that are several hundred percent too low, as shown by the test results in Figure 19.

The effect of stress history on the swell of a compacted sandy clay has also been shown to be large. If a sample is first exposed to water under a high surcharge and allowed to swell, and the sample is later unloaded to a low surcharge, the ultimate swell will be significantly less than if the sample had been subjected to a low surcharge on initial exposure to water. Such behavior precludes the use of the same sample for the determination of more than one point on a swell vs surcharge pressure curve or the prediction of swell from swell pressure data.

Finally, the roles of density, water content, soil structure, and the nature of the solution to which the soil is exposed in influencing the swelling characteristics of compacted clay were summarized. It was shown that swell and swell pressure increase with increasing density, decreasing water content, increasing randomness of structure (flocculent structure), and decreasing electrolyte concentration in the absorbed water. The data

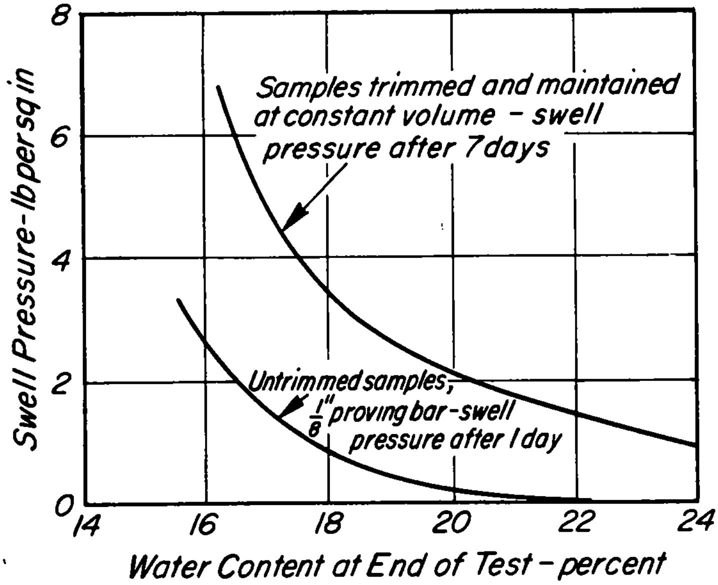


Figure 19. Range of observed swell pressures for similar samples of sandy clay tested under different conditions.

all appear consistent with known characteristics of compacted clays, physicochemical principles, and soil structure.

It is believed that recognition of the importance of the many factors controlling swell and swell pressure may aid in the selection of appropriate compaction conditions and in improved methods of predicting the effects of swelling in engineering practice.

ACKNOWLEDGMENTS

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Experiences With Expansive Clay in Jackson (Miss.) Area

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The very expansive, slickensided Yazoo clay in the Jackson, Miss., area has presented many problems to engineers. Upheavals of roads and streets of as much as one foot are common. Structures on hillsides in which conventional foundations were used have moved downhill several feet. Many slides have occurred in excavation slopes where the Yazoo clay was exposed.

Laboratory investigations and observations have indicated certain characteristics that can be taken into account in design and construction to eliminate, or at least minimize, undesirable performance of the expansive clay.

● JACKSON, MISS., lies in a physiographic belt known as the Jackson Plateau, which is a subdivision of the Gulf Coastal Plains Province. This belt runs in a generally east-west direction across the State of Mississippi with its western end at the eastern edge of the Yazoo River basin and becomes indistinguishable near the Alabama line on the east. The predominant soil of this belt is a fat, very expansive, stiff marine clay called the Yazoo clay because of its outcropping along the Yazoo River basin.

GEOLOGIC HISTORY OF AREA

The Yazoo clay was deposited in glacial times, probably in the rather shallow waters of the Gulf of Mexico that covered most of Mississippi during several periods. The deposit is generally thick—about 400 ft around Jackson. In later periods the Yazoo clay was covered with a loessial-type material that ranges in depth today from 0 to 10 or 12 ft. The loessial-type material is generally classified as a lean clay.

CHARACTERISTICS OF YAZOO CLAY

The top 10 to 15 ft of the Yazoo clay became highly weathered while exposed during its earlier history and this changed the color from the normal dark blue to a yellowish green. The many shrinkage cracks that developed in the weathered part closed during subsequent periods when the overburden was placed, but they remain in evidence today as joints or slickensides. The unweathered part is not jointed. Atterberg limits and volume change tests on a hundred or so samples of Yazoo clay from the Jackson area showed liquid limit values ranging from 50 to 120, plasticity index values ranging from 30 to 80, and volume changes from the liquid limit ranging from 70 to 190 percent.

In connection with the paving design for the new Jackson Municipal Airport, laboratory-soaked CBR tests were performed on several specimens of typical Yazoo clay. Figure 1 shows the results of such a test on a typical sample with liquid limit of about 70 percent and volume change from liquid limit of about 100 percent. The graph at the bottom shows the moisture-density relationship, and the middle graph shows soaked CBR values versus molding moisture contents. It should be noted that the soaked CBR values are low (about 5) at low molding moisture contents but increase to a peak value (about 15) at a moisture content just above optimum. The upper plot of percent swell versus molding moisture content shows that the amount of swell is highest at low molding moisture contents and decreases rapidly as optimum is approached. At a molding moisture content of 8 1/2 percent, the height of the specimen increased 18 percent; while at optimum (15 percent) the swell was only 2 percent. Even less swell was experienced where the molding moisture content exceeded optimum. These characteristics and relationships have been found by a number of investigators (1) of clays and are considered typical of such soils.

The effects of drying on the amount of swell during subsequent wetting when specimens of the previously described sample were molded at 95 percent modified and optimum are illustrated by Figure 2. Specimens were allowed to dry in an air-conditioned room for 10 days, after which they were submerged for 8 days. The shrinkage in 10 days amounted to about 0.5 percent; and subsequent 8 days of soaking produced a swell of about 12 percent. A laboratory CBR test similar to the one shown in Figure 1 was performed on this material except that the compactive effort used was approximately 95 percent of modified AASHO effort. The specimen molded at about optimum moisture content showed a swell of about 2 percent when soaked for 4 days as compared to about 9 percent for the sample of (Figure 2) that had been allowed to dry for 10 days.

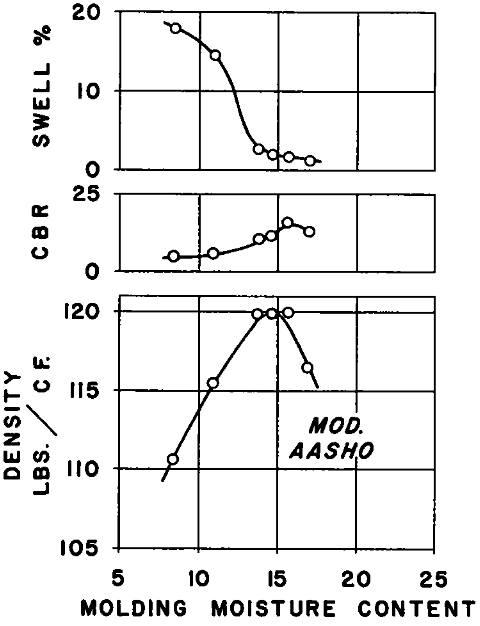

Figure 1. CBR test — Yazoo clay.

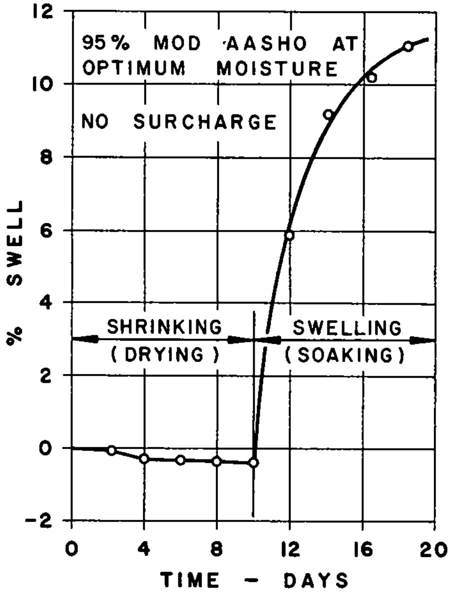

Figure 2. Effects of drying on amount of swell.

When air-dried Yazoo clay is subjected to wetting, it slakes quickly, whereas material at natural moisture content is affected very little by water. An air-dried lump about the size of a tennis ball was placed in a dish containing about 1/2 in. of water and at the same time a similar-sized specimen at natural moisture content was submerged. At the end of an hour the air-dried specimen was almost completely slaked while the other specimen showed only a barely noticeable amount of slaking.

Because of the structure of the soil it is very difficult to arrive at a shear strength for the highly fissured, weathered Yazoo clay; the shear strength of the unweathered material can be determined more satisfactorily. A study of values available from a number of projects around Jackson showed that shear strength values of the weathered portion range from about 0.4 to over 1 ton per sq ft, and those for the unweathered portion ranged upward from 1.75 tons per sq ft. Attempts to use various types of triaxial and direct shear tests on the weathered material have not proven satisfactory, and most engineers have used the unconfined compression test for design purposes. General practice in the area has been to use about the lower quartile of the range of unconfined compression tests, or even lower, for design shear strengths. This has proven so satisfactory that it raises a question as to whether these values are ultraconservative. However, until further investigation indicates conclusively that higher values can be used reasonably, it is believed that the more conservative approach should be used.

From the physical characteristics of the previously discussed clay it may be stated that detrimental swelling can be expected when Yazoo clay is allowed to dry below optimum and is then wetted. Slaking will occur when air-dried material is wetted.

DIFFICULTIES

Many cases of structure distress due to heaving or sliding of the Yazoo clay are to be found. These "distresses" vary from very slight movements to displacements of as much as 1 ft and are to be found in every type of structure: private homes, public buildings, utility lines, streets, highways. A few examples of the most evident to the casual passer-by will be discussed.

Before citing these examples it would be well to call attention to the change of moisture content within soil. When soil is exposed to the atmosphere, it goes through drying and wetting cycles according to the weather. When soil is covered with a highway or airport pavement there is a tendency for soils with moisture contents below optimum to become wetter while those with higher moisture contents may tend to dry slightly (2). These tendencies, however, may be changed if the soil is covered with a floor slab for a building to be artificially heated or cooled, thereby producing a different subgrade thermal regime.

Roads and streets show some of the most striking examples of troubles that stem from the presence of Yazoo clay. Examination of the profiles of pavement surfaces shows that most of the "wavy" condition is caused by upheavals above the as-built grade. Cracks in the pavement and the curbs are wider at the top than at the bottom as a result of this upward movement. Figure 3 shows the high point of such a movement with a differential displacement in the curbing of about 3 in. The lower side of the curbing was estimated to be about 1 in. above as-built grade. There is no readily

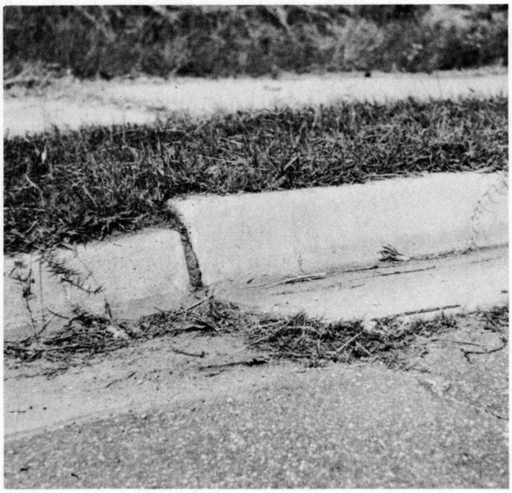

Figure 3.

Figure 4.

apparent explanation for the difference in amount of heave of the two sides of the joint. Probably it is due to difference in resistance to heave. Many of these areas of heave can be identified as places where the pavement was cut, the subgrade trenched for utility connections, and the trench walls allowed to dry; or where dry backfill was placed around structures.

No "drunken forests" have been observed but a "tipsy sidewalk" was seen in front of one home in Jackson (Figure 4). Indications are that the utility lines to the house are located in a trench along the right-hand side of the walk and that heave occurred over the trench. Figure 5 shows one case at a bridge abutment where the upheaval amounted to 5 in.

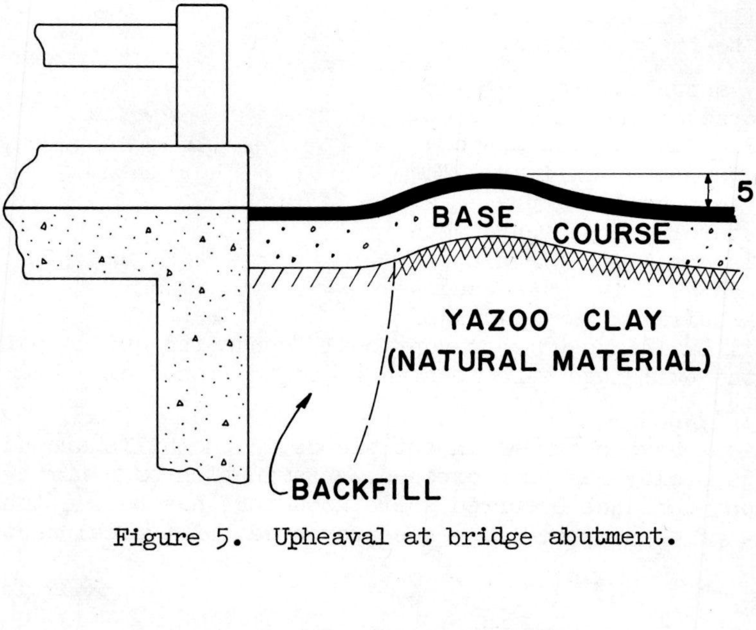

Figure 5. Upheaval at bridge abutment.

The natural ground, Yazoo clay, was excavated to give working room and the space backfilled with non-heaving material. It is noted that the heave has occurred over the interface between the natural ground and the backfill. The face of the natural ground would have had ample opportunity to dry considerably while the bridge was being constructed and before the backfill was placed. Little effort is made during the course of normal construction to prevent cut faces or fill material from losing moisture, and therefore, little imagination is required to envision a detrimental amount of drying taking place in cut faces, trench walls, road foundations, and stockpiles of fill or backfill material.

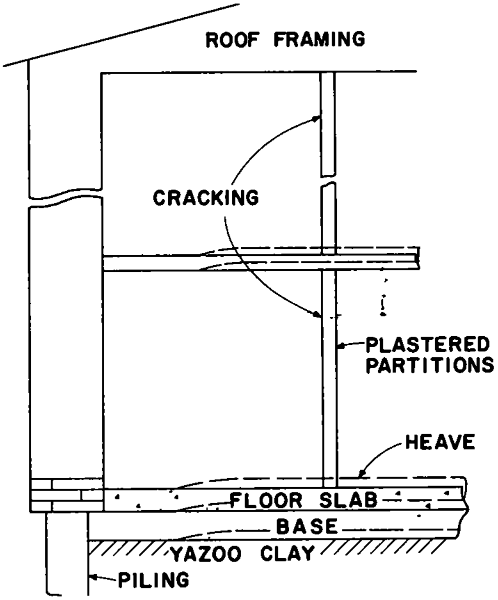

Figure 6. Upheaval in 4-storied building.

With respect to distresses in buildings, cases of differential vertical movement of as much as 1 ft in ordinary dwellings have been reported. One interesting case of a 4-story building is illustrated in Figure 6. The bearing walls were placed on piling and showed no movement. Partition walls on the lower floor were placed on a slab with a few inches of base course laid directly on Yazoo clay. Upheaval in the interior of the floor slabs forced partition walls upward causing much damage to interior walls, floors, and ceilings all the way to the top floor.

Figure 7 shows a case where upheaval has cracked the exterior wall severely. The window sill has been displaced upward over an inch. The interior of this building presents a rather disturbing appearance with cracked walls and floor slabs uplifted as much as 2 in. The condition of this building is typical of several small buildings with conventional foundation and no piling. In every case the buildings were located where the Yazoo clay had little or no overburden.

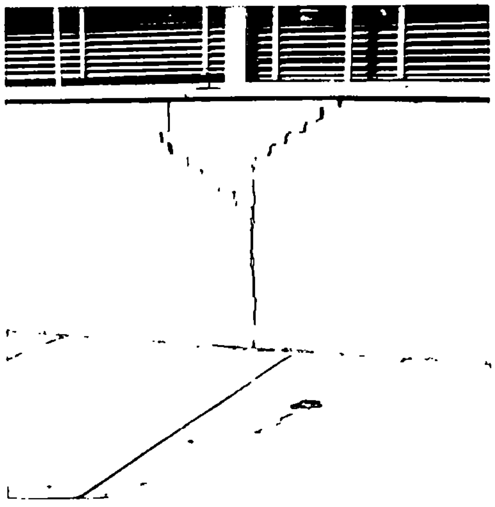

Figure 7.

Many slides have occurred in cut slopes in the hills around Jackson where the Yazoo clay has been exposed. Examination of a number of slides indicated that none has occurred where good turf has been established or where drying of the exposed face was prevented. A typical section of a

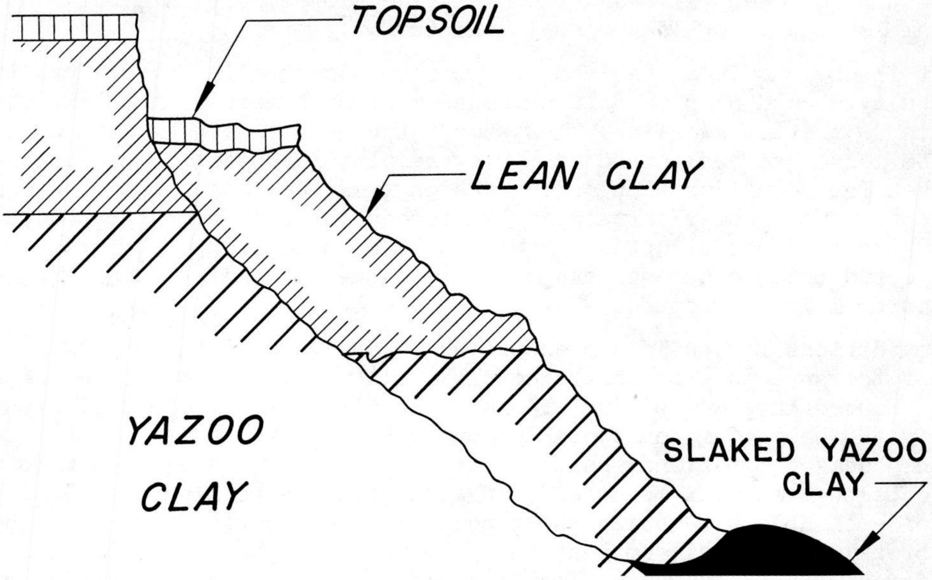

Figure 8. Section of a slide.

slide investigated in considerable detail is shown in Figure 8. This area was bare of vegetation, had been exposed for several weeks, and was observed to have developed large shrinkage cracks. Several rains occurred during the time the cut face was exposed. When the slide was repaired, the slaked material was found at the toe of the slope. The other material showed no evidences of slaking and became quite stable under a little traffic of earth-moving equipment. A typical slide along a roadway is shown in Figure 9. It has been necessary for maintenance forces to remove slaked material that has flowed out over the roadway here several times.

Figure 9.

DESIGN AND CONSTRUCTION

It is considered theoretically possible to construct an embankment of Yazoo clay for a roadway or street that would be entirely satisfactory for many years. However, it is considered improbable that such a structure could be maintained so that detrimental moisture change of the Yazoo clay could be entirely prevented. It is believed that any circumstances that removed vegetative cover from the slopes or changed the thermal regime produced by the pavement would cause volume change in the clay. Therefore, it is believed necessary to restrict the use of Yazoo clay as a fill material to the bottoms of deep fills and to cover side slopes of such embankments with ample amounts of leaner soils and turf.

It is obvious from the laboratory data presented in Figure 1 that Yazoo clay should be placed in an embankment at a moisture content above optimum to prevent swelling. The shear strength of such an embankment will be less than one compacted at lower moisture contents, and this must be taken into consideration during design from the standpoint of stability and settlement. Under certain circumstances the soil might dry out after being compacted wet of optimum, but little or no detrimental shrinkage may be expected because the shrinkage limit is only slightly less than the compaction moisture content.

Foundations for buildings should be so designed that vertical movement of the structure of the Yazoo clay will not be reflected in the structure. Poured-in-place piling are recommended with sufficient clearance under grade beams for considerable movement to take place without affecting the beams. Experience has shown that there is little or no moisture change below 8 ft. It is considered good practice to line the top 6 or 8 ft of each hole with builders' paper, plastic sheeting, or some other bond-breaking material, so that the pile would not be subjected to an upward force in case of heave along the piling. Sufficient length of piling is provided below the 8-ft depth to absorb the imposed load in skin friction and end bearing. Slab foundations should not be used even when as much as a foot of base course material is placed between the slab and the Yazoo clay.

Cut slopes should be completed, blanketed with topsoil, and grassed in as nearly a continuous operation as possible to prevent drying of the clay. Although no slides were observed on slopes definitely identified as flatter than 3 to 1, flattening slopes above should not be considered sufficient protection against sliding. The use of 4 to 1 or 5 to 1 slopes for cut sections, however, is recommended wherever possible as added precaution. Establishment of turf by the quickest method possible (preferably solid sodding) is considered highly essential.

The foregoing discussion indicates that a prime effort in both design and construction where Yazoo clay is involved should be expended in keeping the material from drying during construction operations. It is believed that adherence to these recommended design principles and construction procedures will do much to eliminate the troublesome effects of Yazoo clay. It is further believed that application can be made to expansive clays in other sections of the country.

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Osmotic Interpretation of the Swelling of Expansive Soils

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● VOLUME CHANGES and swelling pressures represent the most important properties of soils if the latter are employed for engineering purposes, especially as supporting bases and other structural components of roads and highways. The problem of the consolidation of water-saturated soils under applied loads has been studied quite thoroughly in soil mechanics. In contrast, the external work performed by swelling soils with increase in moisture content that can seriously affect the superimposed structures has attracted only minor attention. In the particular case of subgrades, Wooltorton (1) has clearly indicated that swelling must be considered as a factor in pavement design and in Hveem's method swelling is considered of equal importance with the generally accepted criterion of minimum deformation. In the classical method of the California Bearing Ratio, its originator, Porter, indicates that no swelling should occur under a surcharge that is equal to the weight of the contemplated superstructure.

From a practical point of view, the subject of greatest interest is the external work that potentially expansive soils can perform against the superimposed structure during swelling. This work is measured by the product of volume increase and swelling pressure and is performed against the weight of the superstructure. Hence, it is not sufficient, as is generally done, to judge a soil solely on the basis of volumetric change; one must also take into account the swelling pressure which supplies the force involved in volume changes and which together with the volume changes determines the total detrimental work.

Experience has shown that both factors are functions of the initial moisture content and this value must be established for each case. This moisture content is of particular importance with respect to the swelling pressure. For air-dry soils of low moisture contents, swelling pressures of the order of 11 kg per sq cm (11.3 tons per sq ft) have been observed (2). Relatively common expansive soils at moisture contents that correspond to working conditions, lying in the vicinity of the plastic limit, give swelling pressures from about 0.5 to 3 kg per sq cm (0.5 to 3 tons per sq ft).

The range of initial moisture contents of interest in soil engineering is quite large. One may accept as a minimum moisture content the hygroscopic moisture of air-dry soils, i.e., the moisture content that is in equilibrium with the vapor pressure of the atmosphere. The hygroscopic moisture is not a constant for a given soil but depends on the temperature and the degree of humidity of the air. The latter is usually given as relative humidity and expressed as

$$H = \frac{p}{p^o} 100$$

where p^o and p are the respective water vapor pressures of the saturated and the actual atmosphere. The condition of air-dryness at ambient temperature (25 C) generally refers to either of two cases:

(a) in dry air with H = 50 and

(b) in practically saturated air with H = 96, which corresponds to the water vapor pressure of a 10 percent aqueous solution of sulfuric acid.

The highest water content for manipulated soils corresponds to the liquid limit at which the mechanical resistance of a soil is practically zero. Within this large range, the moisture contents of greatest interest are those that correspond to actual working conditions, such as the optimum moisture for compaction and the plastic limit. Increase in moisture content above the latter results in rapid decrease of the bearing capacity of cohesive soils.

From the point of view of swelling, one is not interested in fluctuations of moisture content that do not pass above the shrinkage limit because, by definition, no volume changes take place in this moisture range and the external work is zero. With respect to the shrinkage limit, it must be stated that the value obtained by the method of the U. S. Bureau of Public Roads is a minimum value. In many expansive soils at moisture contents about and below the plastic limit, the reduction in volume no longer equals the volume of water lost. Residual contraction starts when the volume change is smaller than the volume of water lost and proceeds gradually until the shrinkage limit is reached. For this reason, some institutions such as the Road Research Laboratory of England consider as shrinkage limit the intercept of the theoretical extrapolation of the indicated relationship below which appreciable contractions are not expected to occur. It is exactly in this moisture range in which high swelling pressures are found accompanied by small volume changes. These combinations are, of course, less detrimental than smaller pressures associated with relatively large volume changes.

The swelling of soils is a consequence of their ability to take in free water and in doing so to overcome the resistance of the soil-water system to expansion, the viscous resistance to the movement of water throughout the soil system, and the confining forces that are due to existing surcharges. The capacity for water sorption is in turn a result of the tendency of the system to pass into a more stable condition under loss of free energy (second principle of theromodynamics). Hence, the forces that determine the retention of water by a soil are also responsible for its swelling characteristics. Therefore, every theory or explanation of swelling must necessarily start from a consideration of the nature and magnitude of the forces that determine the suction of water into a soil at a certain initial moisture content.

The primary purpose of this paper is to point out the various factors involved in this problem and how the organization of these factors may then serve as a dependable guide for experimentation.

MECHANISM OF WATER SUCTION BY SOILS

It is necessary to examine first the various mechanisms that determine the sorption of water by soils, particularly, starting at an initial moisture content of the order of the shrinkage limit.

The following statements recapitulate the available knowledge on the fixation of water by soils (3,4,5):

1. Dry soil fixes water vapor from the atmosphere or liquid water by polar absorption. This is particularly important for the clay fraction because of its high degree of dispersion or large specific surface (surface per unit volume or mass) and its large energy per unit of surface. This is a consequence of the small size produced by physical weathering and the alterations due to chemical weathering which produce electric surface charges not balanced within the mineral structure. These ionic charges are predominantly of negative sign. They give rise to a field of potential energy that acts on the hydrated ions in the aqueous medium surrounding the particles and on the water molecules themselves because of their polar nature. The latter are oriented toward the solid surface and are immobilized to a greater or lesser degree. Opposed to this action is the kinetic energy of the ions and molecules.

Water held by this electric field is quite strongly bound and does not possess the same physical properties as free water, and in its mechanical properties, resembles more or less a rigid solid. The binding energy is greatest at the solid interface and falls off with increasing distance as an exponential function; this means that the binding energy decreases with increasing thickness of the films around the particles, which thickness is determined by the water content of the system and its total internal surface. The kinetic energy is a maximum at an infinite distance from the particle surfaces (free water) and decreases with decreasing distance from the solid surfaces.

The strongly retained (immobilized) water corresponds to a film thickness of up to 10 molecules, or approximately 2 to 3 millimicrons (mμ); practically, it represents the hygroscopic moisture in an atmosphere of $H = 50$ ($pF = 6.0$) and the loss in free energy manifests itself as heat of wetting. For higher water contents as for hygroscopic moisture at $H = 96$ ($pF = 4.7$), film thicknesses of the order of hundreds of molecules (20 - 40 mμ) are reached involving only partial immobilization of the water molecules. The differential heat of wetting (i.e., the heat developed by adding an additional gram of water to a moist soil system) is very small or zero at a hygroscopic moisture content corresponding to $H = 50$. These films act as semirigid or plastic bodies and flow or deform under relatively small stresses. This explains why polar adsorption reaches a minimum at hygroscopicities corresponding to $H = 100$ in the vicinity of which capillary condensation is added to polar adsorption. Water held in excess of that corresponding to hygroscopicity at $H = 100$ cannot be explained by polar adsorption.

2. Clay soils at moisture contents at or above their hygroscopicities at $H = 100$ are able to sorb more liquid water with less energy but in larger quantity. This gives rise to film thicknesses in the order of hundreds of millimicrons called lyosorptive layers by Ostwald and it is these that represent the swelling water. Commonly, this water suction is attributed to the capillary action of concave menisci in the soil system or to the resulting capillary potential which gives an adequate explanation for water sorption in coarse, granular, and silty soils. Undoubtedly, capillary suction also plays a role in water sorption by clay soils, but it does not explain water sorption into saturated clay systems in which the absence of an airphase prevents the formation of menisci. In expan-

sive, and hence, compressible soils, saturation occurs at moisture contents of the order of magnitude of the optimum for compaction; suction, however, continues up to moisture contents as high as the liquid limit and even if dispersed in an excess of free water the particles retain envelopes of more or less strongly bound water.

During the period 1931-33, Mattson (6) proposed the osmotic forces as causative agents of water retention by clays. According to him, this osmotic imbibition occurred after saturation of the polar adsorption capacity. The ideas of Mattson represent an application to soils of the swelling mechanism proposed by Wilson in 1916 generally for proteins and particularly for collagen, which is of such importance in the tanning process. Except for the work of Winterkorn (5,7,8), this interpretation has not found wide application in the field of soil engineering in which the capillary interpretation prevails despite its definite shortcomings.

From the physicochemical point of view, osmotic suction is based on two general phenomena, namely osmosis without membrane and Donnan equilibrium. These fundamental phenomena will be discussed and applied to the swelling of soils and related problems. This shall be done in a quantitative or at least semiquantitative manner.

Mattson's concept of osmotic imbibition is fundamentally based on the concept of the Donnan equilibrium while the theory treated here considers as the fundamental phenomenon osmosis without membrane combined with the concept of osmotic activity. The Donnan equilibrium modifies but does not govern osmotic suction and its influence is important only in saline soils. The osmotic swelling itself is a consequence of the cation exchange characteristic of clays. Table 1 gives data by Mattson showing that swelling increases with an increasing value of T, the base exchange capacity expressed in milliequivalents per 100 gm of soil.

TABLE 1

Clay Soil	T value	Volume (cu cm)		
		Dry	Swollen	
			H^+	Na^+
Nipe	4	0.90	0.92	0.90
Norfolk	21	1.04	1.42	2.30
Sharkey	80	1.12	1.75	7.20

Osmotic suction is possible only in polar solvents that are able to disperse the exchange cations. It does not take place in nonpolar solvents such as hydrocarbons and carbon tetrachloride in which, of course, capillary suction can occur.

OSMOSIS WITHOUT MEMBRANE

The molecules or ions of a substance in solution possess kinetic energy that permits them to diffuse spontaneously when the solution is in contact with the pure solvent or with a more dilute solution, until uniform distribution is reached.

When a solution A containing a certain number of particles per unit volume (concentration) is placed in contact with a pure solvent or more dilute solution B by means of a membrane or semipervious wall a b that permits only the passage of the pure solvent, then two cases may be differentiated (Fig. 1).

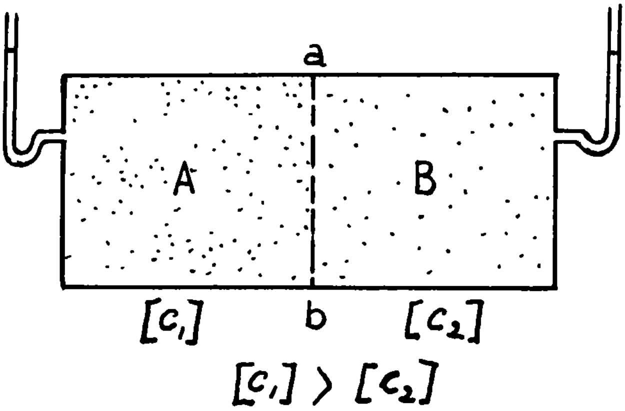


Figure 1.

Case A. — If the semipermeable wall a b is freely moveable, then it will displace itself in the direction of B while solvent from compartment B moves into compartment A until the concentrations in both compartments are equal.

Case B. — If the semipermeable wall a b is fixed, then the tendency to equalize the concentrations in both compartments will result in the movement of solvent from B to A. This causes a hydrostatic or osmotic pressure to develop in compartment A and a suction in compartment B. This process continues until the hydrostatic pressure in compartment A pushes the solvent into the compartment B with the same force as the osmotic phenomenon sucks the solvent from compartment B into compartment A. At this point, a permanent equilibrium is reached. One sees that it is the osmotic phenomenon that determines the hydrostatic or osmotic pressure and not the reverse. The osmotic pressure can be measured by the magnitude of the external pressure that must be applied at A in order to prevent osmotic liquid movement through the semipermeable wall.

In 1885 Van t'Hoff applied the gas laws to the case of dilute solutions and conceived the osmotic pressure as equivalent to the pressure that the dissolved substance would exert if it were in gas form in the same volume as occupied by the solution. In accordance with this concept, the osmotic pressure can be calculated in the following way from the molecular concentration (c)[1]

$P = R\ T\ (c)$ where

---

[1] Number of particles in one liter of water expressed as multiples of the Avogadro number (6.02×10^{23}).

P = osmotic pressure
R = universal gas constant = 0.082 liter atmospheres
T = absolute temperature
(c) = molar concentration.

The pressure P_o developed when one mol of solvent (18 g of H_2O is osmotically incorporated into a liter of molar concentration (c) has been derived thermodynamically by Van t'Hoff on the basis of the osmotic work performed which itself is related to the vapor pressures at the respective concentrations. If M denotes one additional mol of solvent, p and p_o the vapor pressures of the solution and the pure solvent, respectively, then:

$$P = \frac{RT}{M} \ln \frac{p_o}{p}$$

The law of Van t'Hoff states that the osmotic pressure depends only on the number of dispersed particles irrespective of their nature, hence they may be atoms, ions, molecules, or colloidal or even larger particles. The respective systems are called ideal solutions if they obey this law closely, and distinction is made between ideal and real solutions in the same manner as between ideal and real gases. In the case of dilute solutions, the dissolved and dispersed particles can move independently of each other and consequently follow the law quite closely. In other cases such as in concentrated electrolyte solutions, ionic attraction and repulsive forces act between the particles and result in a deviation from the ideal osmotic law. Each ion tends to surround itself with ions of opposite charge with resultant change in ionic activity. The interionic forces increase in strength with increasing concentration and increasing ionic charge. As a consequence, the real osmotic activity is smaller than that calculated for ideal solutions. For this reason, Lewis developed the concept of activity (1901-07) and introduced the coefficient of activity f, which is defined as follows:

$$f = \frac{\text{real activity}}{\text{concentration}} = \frac{a}{(c)}$$

For very dilute solutions, a approaches (c) and $f \rightarrow 1$. With increase in concentration and valency f changes as illustrated below:

Activity coefficients f

(c)	0.01	0.001	0.0001
K^+ in KCl	0.89	0.96	0.99
Cu^{++} in $CuSO_4$	0.40	0.75	0.91
Al^{+++} in $AlCl_3$	0.08	0.45	0.78

From a thermodynamic point of view, the presence of a semipervious wall is not absolutely necessary for restraint of diffusion and development of osmotic pressure. It is sufficient if, due to the influence of other forces such as adsorption and electrostatic attraction, one component is free to diffuse while the movement of the other is restrained. From this arises the concept of osmosis without membrane, which is of special importance for the swelling of clays.

One must keep in mind that osmosis is a consequence of the kinetic energy of the dispersed particles and that the forces that play the role of a semipermeable membrane must necessarily decrease the osmotic activity of the particles by restraining their freedom of movement and hence their kinetic energy. This is the essential basis for an osmotic interpretation of water sorption by clay soils above a certain initial moisture content.

Consequently, application of the law of Van t'Hoff to real systems involving osmosis without membranes must make use of the activity f(c) rather than of the concentration alone. This is important because several authors have used concentrations rather than activities in the application of Van t'Hoff's law to the osmotic swelling of clay-water systems (7,9,10). Although this has been done more for the purpose of illustration and as a guide to experimentation, it could be misleading to the uninitiated who may believe in the reality of the very high osmotic pressures that are obtained by such calculations.

OSMOTIC PRESSURE OF THE SOIL-WATER SYSTEM

As a starting point for application of the concept of osmosis without membrane to soil-water systems a peptized, salt-free, clay soil is taken at a moisture content between the liquid and the plastic limit.

The water occupying the free spaces between the solid particles acts as a dispersion medium for the exchange cations. The latter are subjected to two counteracting influences: (c) the electrostatic attraction by the electric charges on the surfaces of the solid particles and (b) the dispersive tendency which is due to the kinetic energy of the particles and because of which the latter attempt to distribute themselves uniformly throughout the liquid phase. The cations are in a hydrated condition, which means that the radius of the kinetic particulate unit is that of the particle plus its bound water molecules.

The distribution of exchange ions around particles dispersed in an aqueous medium has been studied by Duclaux (11) under the assumption that the particles are spherical and that the electrostatic and kinetic forces are in equilibrium. He arrived at the conclusion that the concentration of the cations decreases exponentially with increasing distance from the particle surfaces. Later, Winterkorn (7) applied the same calculation to plane particle surfaces that are more in accordance with the shape of real swelling clays and arrived at the conclusion that, practically, there is no change in concentration of the exchange cations with increasing distance up to a film thickness of the order of hundreds of molecules. Schmid (12) arrived at a similar result for microporous systems. Consequently, it should be permitted to assume a uniform distribution of cations in the aqueous medium for the moisture range in which we are interested. For higher moisture contents, the cations will tend to become more concentrated toward the particle surfaces in accordance with the classical theory of the Helmholtz double layer, which is applicable for dilute clay suspensions.

Accepting a uniform concentration of the exchange ions in the aqueous dispersion medium, one can calculate the molar concentration from the exchange capacity of the clay (Hissink "T"-Value); the valency Va of the cations in the case of homoionic clays or the median valency in the case of heteroionic clays and the moisture content. The exchange capacity "T" is usually expressed in milliequivalents per 100 g of soil. Hence, the number of equivalents per 100 g of soil is "T" per 1,000 and of moles "T" per 1,000 x Va when Va represents the valency. If this number of moles exists in 100 g of soil that contains i g of water, then the molar concentration [c] or the number of moles of exchange ions per liter of aqueous dispersion medium becomes

$$[c] = \frac{\frac{"T"}{1000\ Va}\ 1000}{i} = \frac{"T"}{i\ Va} \qquad (1)$$

An "ideal" dispersion of cations of a certain concentration, placed in contact with pure water by means of a membrane that allows free passage of the water molecules but not of the cations, develops an osmotic suction that can be translated into a hydrostatic pressure whose magnitude according to Van t'Hoff is

$$(P_o)_{ideal} = R\ T\ \frac{"T"}{i\ Va} \qquad (2)$$

where $(P_o)_{ideal}$ is the hydrostatic pressure in atmospheres that equilibrates the developed osmotic pressure, R is the gas constant (0.082 l atm), and T the absolute temperature.

The theoretical osmotic pressures, calculated under assumption of "ideal" behavior, are inversely proportional to the initial moisture content and are of the order of magnitude of tens of atmospheres. Thus, for "T" = 30, i = 28.6, T = 273 + 25 = 298 C and Va = 1, one obtains

$$(P_o)_{ideal} = 0.082 \times 298\ \frac{30}{28.6 \times 1} = 25.6 \text{ atm } (pF = 4.41)$$

The osmotic pressures actually developed in real soil-water systems are necessarily smaller than those just calculated for ideal systems. In a real soil-water system, the role of the assumed semipermeable membrane is filled by the electrostatic attraction between the cations and the negatively charged particle surfaces. This attraction decreases the effectiveness of the kinetic energy of the cations, which is the cause of their diffusion tendency and of the resultant osmotic pressure. Hence, one may write

$$(P_o)_{real} = (P_o)_{ideal}\ f$$

where f is a coefficient smaller than 1 that corresponds to the activity coefficient of Lewis and expresses the real osmotic activity of the dispersed ions under the prevailing environmental conditions.

The coefficient of activity f must decrease with decreasing moisture content, since the restraining electric attraction of the cations by the charged particle surfaces increases with increasing proximity of the cations, i.e. with decreasing moisture content. Therefore, f should be expressed as a function of the moisture content f(i), the maximum value of this function being unity. The general expression is

$$(P_o)_{real} = (P_o)_{ideal} \times f(i) = RT\ \frac{"T"}{i\ Va}\ f(i) \qquad (3)$$

In equation (3), R, T, "T", and Va are constants for a particular soil at constant temperature and $(P_o)_{real}$ is a function of the moisture content. The decrease of $(P_o)_{real}$ with increasing initial moisture content is due to two factors:

(a) dilution: $(P_o)_{real}$ changes inversely with change in moisture content;

(b) activity: higher initial moisture content implies less electrostatic restraining influence, i.e. greater diffusibility and less osmotic pressure. This is expressed by the factor f(i).

The factor f(i) can be calculated if one knows $(P_o)_{real}$. Equation (3) can then be transformed as follows:

$$f(i) = \frac{(P_o)_{real} \times i \times Va}{R\ T\ "T"}; \tag{4}$$

its solution implies the experimental determination of $(P_o)_{real}$.

The logical method for the determination of $(P_o)_{real}$ is by means of the consolidation test as used in soil mechanics. If one applies to a soil-water system that is in communication with free water a constant pressure of sufficient magnitude, then free water is squeezed out of the soil with corresponding decrease in volume until the system has reached a state of equilibrium. Let P be the externally applied pressure that causes equilibrium to be reached at a water content of i. If we consider the cell of the consolidation apparatus as an osmometer, then the hydrostatic or osmotic pressure developed by a puddled soil-water system at a water content of i must be compensated by the externally applied pressure in order that a permanent equilibrium may be reached. This signifies that the osmotic pressure fills a role similar to that of the pore pressure for saturated systems in soil mechanics and that the applied P measures directly the $(P_o)_{real}$. Therefore,

$$(P_o)_{real} = P$$

$$P = \frac{R\ T\ "T"}{Va} \times \frac{1}{i} \times f(i) \tag{5}$$

where all terms except f(i) can be measured experimentally and f(i) can be calculated from

$$f(i) = \frac{P\ i\ Va}{R\ T\ "T"} \tag{6}$$

for each moisture content as long as the condition $(P_o)_{real} = \underline{p}$ holds.

From theory, it may be expected that f(i) is a decreasing exponential function of i. This is confirmed by plotting the logarithms of experimentally obtained f(i) values against the respective i values. This has been done in Figure 2 which shows the straight line relationships obtained for several ionic modifications of Putnam soil. This relationship is also shown in Figure 3.

With soil A, one obtains straight lines that can be extrapolated to f(i) = 1 or log f(i) = 0; they intercept the abscissa at a moisture content i_o which can be expressed by the equation

$$\log f(i) = K(i - i_o). \tag{7}$$

Accordingly, f(i) falls exponentially with increase in moisture content above a certain initial moisture content i_o for which the maximum theoretical value of the osmotic pressure holds in accordance with Eq. 3. As could be expected, the value i_o is of the same order of magnitude as the shrinkage limit of the soil. Consequently, in this type of soil, the osmotic water suction gives rise to pore pressures and associated expansion,

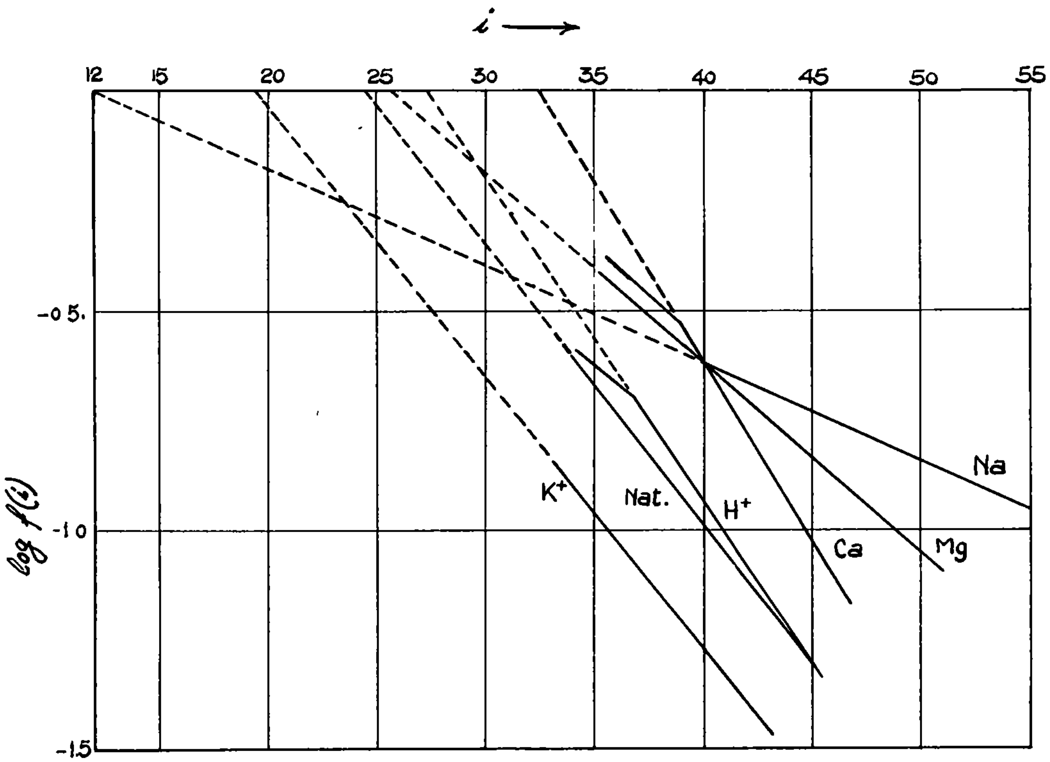


Figure 2. Relation between i and log f(i)

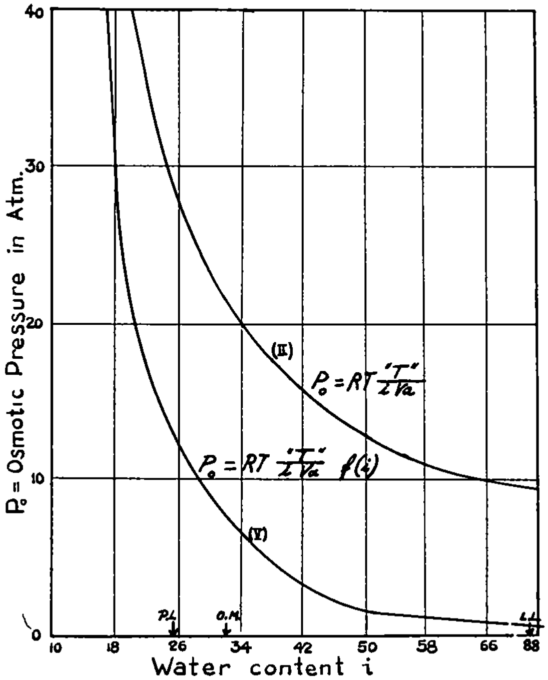


Figure 3. Osmotic pressure of Na-clay.

starting at moisture contents that are of the same order of magnitude as the shrinkage limit. The negative angular coefficient K can be calculated by the equation

$$K = \frac{\log f(i)}{i - i_o} \tag{8}$$

Its value determines the osmotic behavior of a soil. For the same soil, the K value depends on the nature of its exchangeable cations. Highly hydrated cations such as Na^+ give negative values of K that are smaller than those for cations that have a higher position in the Hofmeister lyotropic series such as K^+ or the bivalent cations. This results from the force that attracts the cations to the particle surfaces being a function of their charge (valency), radius, and degree of hydration.

In the case B, one obtains a straight line that is broken at point O. The straight line B follows Eq. 7, but the value i_o is larger than the shrinkage limit. Evidently, at the moisture content that corresponds to point O and is around 36 to 40 percent, the solid volume is of the same order of magnitude as the pore volume and contact is established between the more energetically bound semirigid water envelopes around the solid particles. In the case of flocculating cations, such as Ca^{++} and H^+, that cause greater attraction forces between the particles, a certain structure is developed and the consolidation pressure is partially consumed by the deformation of this structure. This results in a change of the angular coefficient. With the new angular coefficient, the straight line must tend toward i_o = shrinkage limit because the structure formed by these soils cannot resist the shrinkage forces. By deformation of this structure or by a new structural arrangement, the shrinkage limit is finally reached.

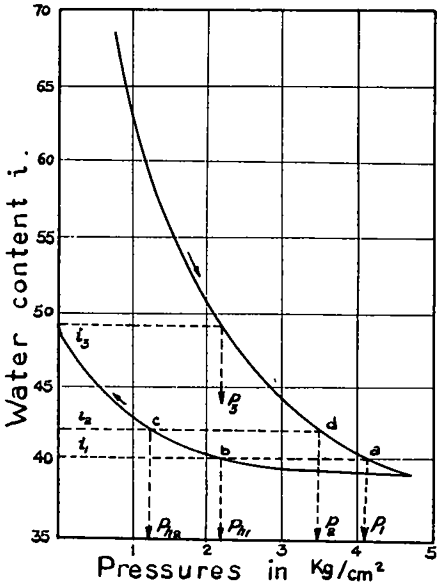


Figure 4. Consolidation-swelling curves of Na-Putnam soil.

EXPERIMENTAL DATA

The work of Winterkorn and Moorman (13) on Putnam clay soil of podsolic origin contains the data necessary for the application of the theoretical concepts that have been developed. It contains the engineering soil characteristics, the "T" value, the nature of the exchange cations from which the median valency can be calculated, and the consolidation curves for the natural soil and for the homoionic Na^+, H^+, K^+, Ca^{++}, and Mg^{++} soils. In Figure 4 the moisture contents have been calculated from the voids ratios given. The temperature has been assumed to be 25 C or 298 A.

In Table 2 are given the data taken from the work of Winterkorn and Moorman (13) and in Table 3 are found the values for f(i), K, and i_0 that have been calculated by means of the Eqs. 7 and 8 from the consolidation pressures P and the equilibrium water contents i taken from the consolidation graphs of Winterkorn and Moorman (13). Figure 2 shows plots for log f(i) versus i with the straight lines extended to obtain the i_0 values.

TABLE 2

CHARACTERISTICS OF THE PUTNAM CLAY SOIL[1]
AND ITS HOMOIONIC MODIFICATIONS

Characteristics	Natural	H^+	Na^+	K^+	Mg^{++}	Ca^{++}
Liquid limit	64.5	56.4	88	52.8	56.3	61.9
Plastic limit	23.5	24.8	25.4	27.7	25.4	27.0
Plasticity index	41.0	31.6	62.6	25.1	30.9	34.9
Shrinkage limit	17.9	16.3	11.8	19.4	12.4	12.4
Hygroscopicity	5.8	5.3	4.2	4.6	4.4	5.7
Vacuum moist equiv.	57.1	54.3	water-logged	53.7	58.7	58.9
Optimum moisture (Proctor)	28.6	30.9	31.3	28.4	31.2	32.2
% passing No. 200 sieve	98.1	98.7	97.5	98.5	98.6	98.5
% clay 0.005 mm	54.0	48.0	61.0	44.0	49.0	45.0
% colloids 0.001 mm	33.0	26.0	48.0	21.0	25.0	22.0

[1] Base exchange capacity, "T" = 30 me per 100 g; cations in natural soil in me per 100 g—H^+ = 12.0, Ca^{++} = 11.6, Mg^{++} = 4.5, Na^+ = 1.4, K^+ = 0;

$$\text{Average valency (calc.)} = \frac{30}{\frac{12}{1} + \frac{11.6}{2} + \frac{4.5}{2} + \frac{1.4}{1}} = 1.4.$$

TABLE 3

OSMOTIC ACTIVITY f(i) OF PUTNAM CLAY AND ITS HOMOIONIC MODIFICATIONS FROM THE CONSOLIDATION GRAPHS OF REFERENCE

Soil Modification	i (%)	P (atm)	f(i) Eq. 6	K Eq. 8	i_o	i_o/S.L.
Natural	33.3	4	0.255	-0.064	24	1.34
	34.0	3.5	0.228	-0.064		
	34.6	3	0.198	-0.066		
	37.7	2	0.145	-0.061		
	41.2	1	0.079	-0.064		
	45.2	0.5	0.434	-0.064		
Sodium	40.6	4	0.222	-0.0228	12	1.02
	41.7	3.5	0.200	-0.0236		
	44.0	3	0.180	-0.0233		
	49.3	2	0.135	-0.0233		
	59.1	1	0.081	-0.0232		
	68.0	0.5	0.0465	-0.0238		
Potassium	31.8	4	0.174	-0.063	19.8	1.02
	32.5	3.5	0.156	-0.0635		
	33.3	3	0.137	-0.064		
	35.5	2	0.097	-0.0645		
	39.8	1	0.0545	-0.063		
	43.0	0.5	0.0295	-0.066		
Calcium	36	4	0.398	-0.105	-	-
	37	3.5	0.354	-0.094	-	-
	38.2	3	0.315	-0.0835	32.2	2.6
	39.8	2	0.219	-0.0795		
	43.6	1	0.1195	-0.081		
	47.2	0.5	0.0645	-0.0795		
Magnesium	34.6	4	0.378	-0.046	25.4	2.05
	35.5	3.5	0.344	-0.046		
	37.0	3	0.304	-0.0445		
	39.8	2	0.218	-0.046		
	45.7	1	0.125	-0.0445		
	50.0	0.5	0.0685	-0.0435		
Hydrogen	35.5	4	0.194	-0.080	-	-
	37.0	3.5	0.177	-0.072	26.6	1.63
	38.0	3	0.156	-0.071		
	39.8	2	0.109	-0.073		
	43.6	1	0.060	-0.071		
	46.8	0.5	0.032	-0.073		

The results shown in Table 3 and Figure 2, which have been obtained by considering water suction in clay soils as an osmotic phenomenon without membrane due to the exchange ions, lead to the following conclusions:

1. The direct application of Van t'Hoff's law implies that the hydrostatic or osmotic pressure developed (Eq. 2) is directly proportional to the base exchange capacity and inversely to the valence of the exchange ions. This is in qualitative agreement with the known facts on the influence of the quantity and nature of the clay minerals and organic matter on water sorption and with the lesser activity of polyvalent cations. On the other hand, this does not explain the enormous influence of the nature of the cation, for example, Na^+ and K^+. The same law would make the osmotic pressure, under otherwise equal conditions, increase with increasing temperature, a phenomenon that according to our knowledge has not as yet been investigated experimentally. The simple application of Van t'Hoff's law considers the osmotic pressure inversely proportional to the initial water content of the soil system, which we have called the dilution factor, but it does not explain the quantitative relationship between both factors and leads to extremely high values for the osmotic pressure developed.

2. The basic hypothesis is the introduction of the concept of the osmotic activity of the exchange ions, which is a direct and necessary consequence of the fact that osmotic water suction in the case of clay soils is "osmosis without membrane." The activity is a function that decreases exponentially with increase in moisture content and the course of the decrease depends on a specific soil constant K, which is the basic factor that determines the osmotic pressure actually developed. Figure 3 shows the influence of the "dilution" factor and of the activity factor on the magnitude of the osmotic pressure developed by Na^+-Putnam clay.

The value for K depends for each soil on the nature of the exchange ions and decreases in the following order:

	Na^+	Mg^{++}	K^+	H^+	Ca^{++}
K =	-0.023;	-0.045;	-0.063;	-0.072;	-0.081

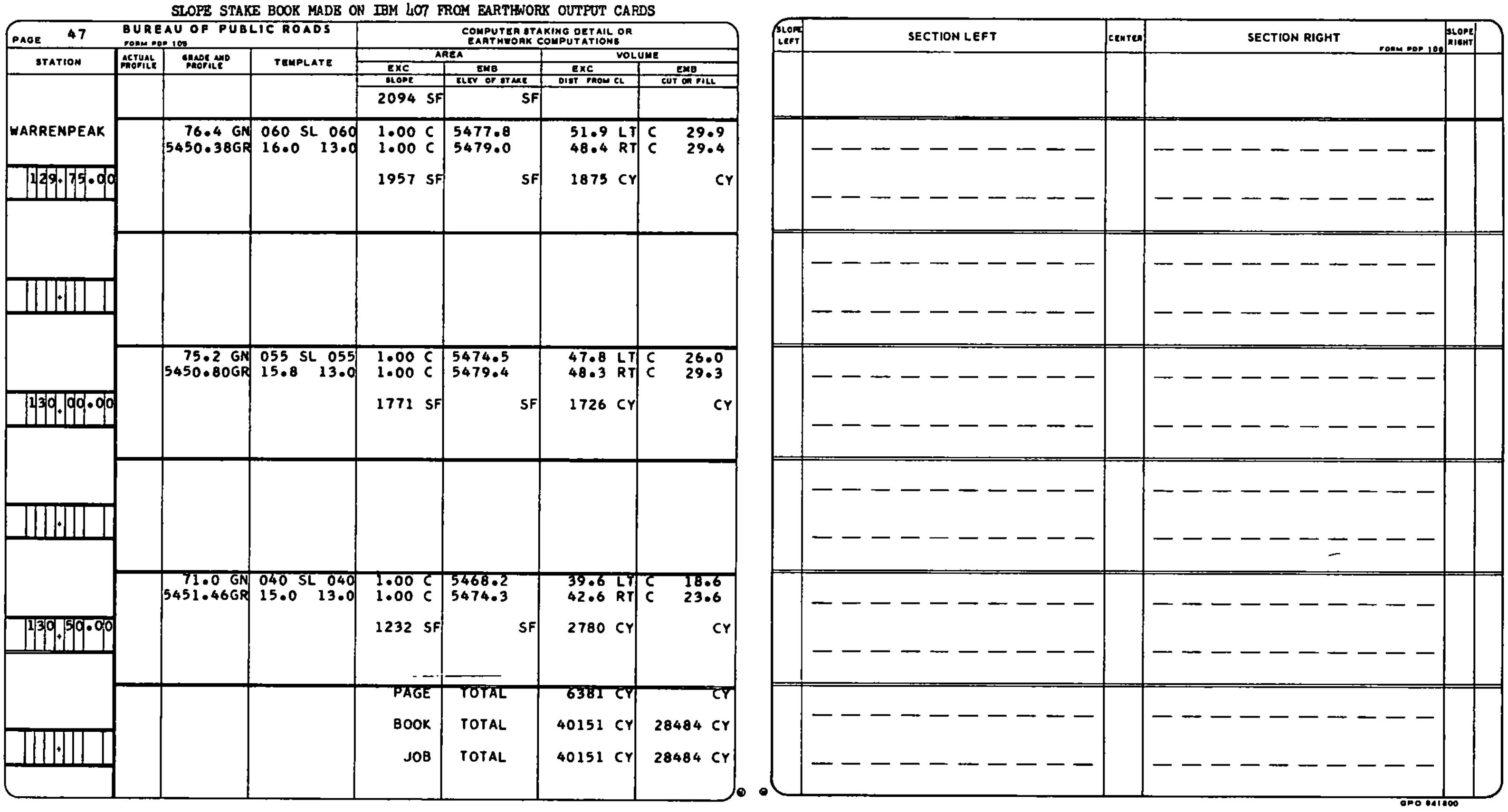
which corresponds to their degree of hydration or solvatation. The osmotic pressure developed is, of course, a decreasing function of the moisture content (Eq. 3).

3. It is an important experimental fact that the maximum value for the activity (i.e. $f(i) = 1$) corresponds to a moisture content i_o that is practically identical with the shrinkage limit for the Na^+- and K^+-Putnam clay soils. The ratio $i_o/S.L.$ increases for the other ions in the order

	$Na^+ = K^+$	nat	H^+	Mg^{++}	Ca^{++}
$\frac{i_o}{S.L.}$ =	1.02	1.34	1.63	2.05	2.6

This signifies that the bivalent cations, and likewise H^+ which also has a flocculating effect, exhibit aggregation tendencies and form a structure as the moisture content decreases and the particles come closer together. One must keep in mind that at a water content of i_o, the pore volume and the solid volume of the system are approximately equal, giving a chance for contact between the particles, and that i_o is that water content at which the relationship, "decrease of volume of the system =

SLOPE STAKE BOOK MADE ON IBM 407 FROM EARTHWORK OUTPUT CARDS



PAGE 47 BUREAU OF PUBLIC ROADS

FORM PDP 105

COMPUTER STAKING DETAIL OR EARTHWORK COMPUTATIONS

STATION	ACTUAL PROFILE	GRADE AND PROFILE	TEMPLATE	AREA EXC SLOPE	AREA EMB ELEV OF STAKE	VOLUME EXC DIST FROM CL	VOLUME EMB CUT OR FILL
				2094 SF	SF		
WARRENPEAK		76.4 GN 5450.38GR	060 SL 060 16.0 13.0	1.00 C 1.00 C	5477.8 5479.0	51.9 LT 48.4 RT	C 29.9 C 29.4
129+75.00				1957 SF	SF	1875 CY	CY
+							
		75.2 GN 5450.80GR	055 SL 055 15.8 13.0	1.00 C 1.00 C	5474.5 5479.4	47.8 LT 48.3 RT	C 26.0 C 29.3
130+00.00				1771 SF	SF	1726 CY	CY
+							
		71.0 GN 5451.46GR	040 SL 040 15.0 13.0	1.00 C 1.00 C	5468.2 5474.3	39.6 LT 42.6 RT	C 18.6 C 23.6
130+50.00				1232 SF	SF	2780 CY	CY
				PAGE	TOTAL	6381 CY	CY
				BOOK	TOTAL	40151 CY	28484 CY
+				JOB	TOTAL	40151 CY	28484 CY

SLOPE LEFT	SECTION LEFT	CENTER	SECTION RIGHT FORM PDP 106	SLOPE RIGHT	

GPO 941800

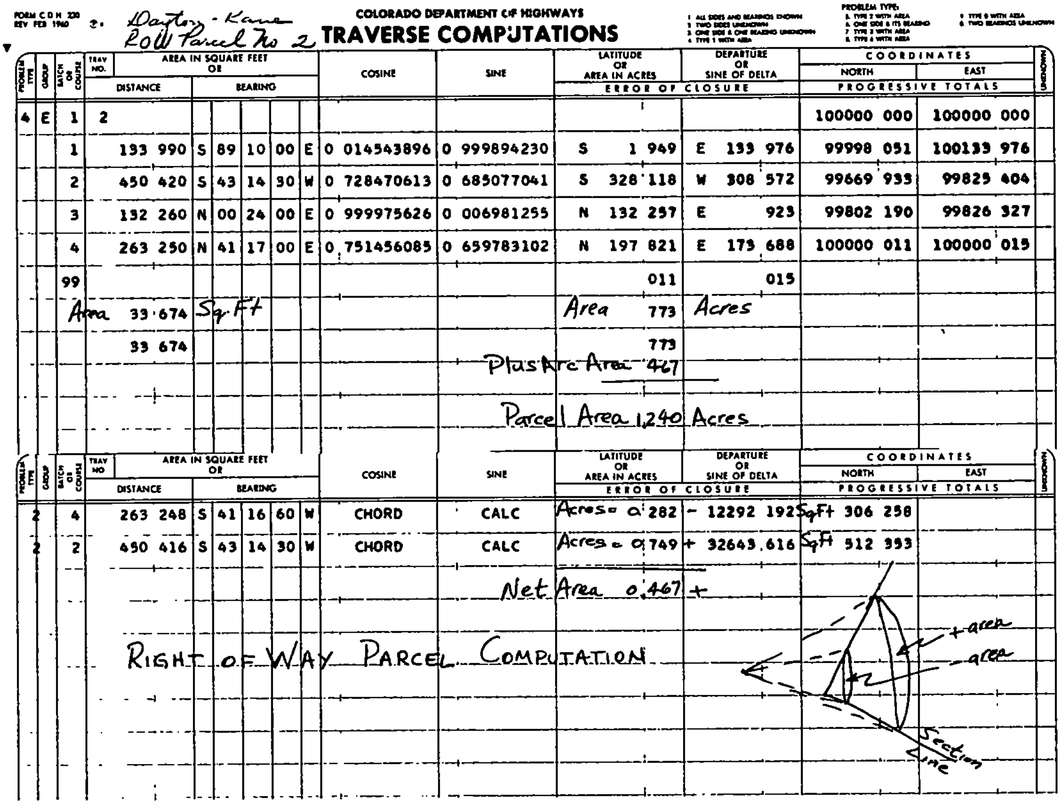
COORDINATES OF RIGHT-OF-WAY OWNERSHIPS FROM ORIGINAL PLAT DESCRIPTIONS

OWNERSHIP CO ORDINATES

JOB	PARCEL	CRNER	NS CO ORD	EW CO ORD
HOLMANTAOS	HES 317	1	53,890.72	75,302.70
HOLMANTAOS	HES 317	2	53,721.40	75,251.53
HOLMANTAOS	HES 317	3	53,642.24	75,218.44
HOLMANTAOS	HES 317	4	53,541.72	75,175.36
HOLMANTAOS	HES 317	5	53,641.44	74,935.20
HOLMANTAOS	HES 317	6	53,765.13	74,423.93
HOLMANTAOS	HES 317	7	54,068.73	73,842.37
HOLMANTAOS	HES 317	8	54,177.70	73,936.32
HOLMANTAOS	HES 317	9	53,955.91	75,053.01

ERRORS

HOLMANTAOS	HES 317	1.36	0.31	53,640.88	75,218.75



FORM C D H 230
REV FEB 1960

Dayton - Kane
ROW Parcel No 2

COLORADO DEPARTMENT OF HIGHWAYS

TRAVERSE COMPUTATIONS

PROBLEM TYPE:
1 ALL SIDES AND BEARINGS KNOWN
2 TWO SIDES UNKNOWN
3 ONE SIDE & ONE BEARING UNKNOWN
4 TYPE 1 WITH AREA
5 TYPE 2 WITH AREA
6 ONE SIDE & ITS BEARING
7 TYPE 3 WITH AREA
8 TYPE 6 WITH AREA
9 TYPE 8 WITH AREA
0 TWO BEARINGS UNKNOWN

PROBLEM TYPE	GROUP	BATCH OR COURSE	TRAV NO. / AREA IN SQUARE FEET OR DISTANCE	BEARING					COSINE	SINE	LATITUDE OR AREA IN ACRES (ERROR OF CLOSURE)	DEPARTURE OR SINE OF DELTA (ERROR OF CLOSURE)	COORDINATES NORTH (PROGRESSIVE TOTALS)	COORDINATES EAST (PROGRESSIVE TOTALS)	UNKNOWN
4	E	1	2										100000 000	100000 000	
		1	133 990	S	89	10	00	E	0 014543896	0 999894230	S 1 949	E 133 976	99998 051	100133 976	
		2	450 420	S	43	14	30	W	0 728470613	0 685077041	S 328 118	W 308 572	99669 933	99825 404	
		3	132 260	N	00	24	00	E	0 999975626	0 006981255	N 132 257	E 923	99802 190	99826 327	
		4	263 250	N	41	17	00	E	0 751456085	0 659783102	N 197 821	E 173 688	100000 011	100000 015	
		99									011	015			
		Area	33·674	Sq.Ft							Area 773	Acres			
			33 674								773				
										Plus Arc	Area 467				
										Parcel	Area 1,240	Acres			
PROBLEM TYPE	GROUP	BATCH OR COURSE	TRAV NO / AREA IN SQUARE FEET OR DISTANCE	BEARING					COSINE	SINE	LATITUDE OR AREA IN ACRES (ERROR OF CLOSURE)	DEPARTURE OR SINE OF DELTA (ERROR OF CLOSURE)	COORDINATES NORTH (PROGRESSIVE TOTALS)	COORDINATES EAST (PROGRESSIVE TOTALS)	UNKNOWN
2		4	263 248	S	41	16	60	W	CHORD	CALC	Acres= 0.282	- 12292 192	SqFt 306 258		
2		2	450 416	S	43	14	30	W	CHORD	CALC	Acres = 0.749	+ 32643.616	SqFt 512 333		
										Net	Area 0.467	+			

RIGHT OF WAY PARCEL COMPUTATION

TABLE 5

RELATIONSHIP BETWEEN OSMOTIC PRESSURE AND SOIL-WATER CONSTANTS

Cation	L L	Osmotic Pressure for i = L L		P L	Osmotic Pressure for i = P L		Vacuum Moist Equiv	Osmotic Pressure for i = V M E		Opt Moist	Osmotic Pressure for i = Opt M	
		atm	pF		atm	pF		atm	pF		atm	pF
Natural	64.5	0.021	1.32	-	-	-	57.1	0.070	1.84	-	-	-
Na$^+$	88.0	0.142	2.15	25.4	14.1	4.1	water-logged	-	-	31.3	8.2	3.9
K$^+$	52.8	0 107	2.03	27.7	8.4	3.9	53.7	0.090	1.95	28.4	7.3	3.8
H$^+$	56.4	0.093	1.97	-	-	-	54.3	0.138	2.14	-	-	-
Ca^{++}	61.9	0.025	1.40	-	-	-	58.9	0.426	2.63	-	-	-
Mg	56.3	0.269	2.42	-	-	-	58.7	0.199	2.29	-	-	-

the more closely, the more permeable the soil is. Table 5 contains vacuum moisture equivalents and the corresponding osmotic suction values calculated by means of Eq. 5. The osmotic suction values approach the applied suction of 0.55 atmospheres in the following order:

$$Na^+ < nat < K^+ < H^+ < Mg^{++} < Ca^{++}$$

This order coincides with that of the permeabilities given by Winterkorn and Moorman (13). The calculated osmotic pressure for the most pervious soil (Ca^{++}) is 0.426 atm, which is closest to that for the applied suction. Waterlogging of the Na$^+$ soil, the least permeable, prevented the determination of its vacuum moisture equivalent.

It has been shown that the swelling of soils is due to the action of the osmotic pressure that is developed in the aqueous phase of a compacted soil-water system that is in contact with free water. For constant compactive effort (Standard Proctor) at optimum moisture content, it is possible to calculate by means of Eq. 5 the osmotic pressure of the respective moisture contents by using data given in the work by Winterkorn and Moorman (13). Results of such calculations are found in Table 5 for the Na$^+$- and K$^+$-Putnam clays for which the extrapolation to i_o, from which f(i) is determined, can be considered as permissible in the range of the i value at the optimum moisture content because of the coincidence of i_o and the shrinkage limit. For the other soils, it is necessary to have consolidation test data at lower moisture contents, because the optimum moisture content falls in the range where K changes or is likely to change because the shrinkage limit is smaller than i_o.

The same problem for the calculation of f(i) appears in the case of the plastic limit and for this reason only the osmotic pressures that pertain to the Na$^+$ and K$^+$ modifications are given in Table 5. The osmotic pressures calculated for these two cases give an idea of the magnitude of the suction force that clay soils are capable of developing at low initial humidities. They also reveal the P.I. as an index of osmotic activity since the larger P.I. of the Na$^+$ modification corresponds to a larger K value. This is a consequence of the relatively small influence of the nature of the exchange cations on the plastic limit and their marked influence on the liquid limit. It is the latter constant that is of major importance from the point of view considered here (see Figure 4).

The properties characteristic for clays (such as plasticity and compressibility) are intimately connected with their cation exchange capacity which in turn governs the osmotic suction. It is well known that rocks that have not suffered chemical weathering (e.g. quartz flower ground to the size of clay particles) show very low base exchange capacity and also no plasticity or water-retaining power exceeding capillary saturation. All this leads to the conclusion that osmotic water suction plays a fundamental role with respect to these properties.

The ability of clays to retain water, even when under external pressure, above that required for capillary saturation goes parallel with their plasticity properties and several indices for their measurement have been proposed. Skempton (7) proposed the index of compressibility, which is the numerical value of the tangent of the approximately straight line obtained by plotting the logarithms of the pressures applied in the consolidation test against the corresponding water contents.

$$I_c = \frac{i_1 - i_2}{\log P_1/P_2}$$

The limiting value of this function is the inverse of $\frac{d \log P}{di}$ which implies that P varies exponentially with i. The importance of the index of compressibility is that it is inversely correlated with the liquid and plastic limits but has a general proportionality with the content in <0.002 mm clay. This is to be expected, since not only the quantity of the clay fraction but also the character of the component minerals and the nature of the exchange cations are determinant factors. The relationship between Terzaghi's coefficient of compressibility and Skempton's index of compressibility is apparent.

Operating at high moisture contents where the logarithmic relationship of Skempton represents a closer approximation, Eq. 5 should hold true if the osmotic interpretation is correct. This equation implies that P is not a simple exponential function of i, but a function of a function of i. It seemed indicated to calculate I_c in accordance with the osmotic concept and to compare it with the findings of Skempton. Using the logarithms in Eq. 5, one obtains:

$$\log P = \log (RT''T'') + \log \frac{f(i)}{i}$$

where RT"T" is a constant for each individual soil at a fixed temperature and the index of compressibility the inverse of $\frac{d \log P}{di}$:

Setting $\frac{f(i)}{i} = s$, one obtains

$$\frac{ds}{di} = \frac{i \frac{df(i)}{di} - f(i)}{i^2}$$

$$\frac{d \log P}{ds} = \frac{0.434\ i}{f(i)}$$

$$\frac{d \log P}{di} = \frac{ds}{di} \times \frac{d \log P}{d S} = \frac{0.434\ i \frac{d\ f(i)}{di} - 0.434\ f(i)}{i\ f(i)}$$

Since f(i) is an exponential function of i :

$$\frac{d\,f(i)}{di} = K\,f(i) = \frac{Kf(i)}{0.434}$$

$$\frac{d\log P}{di} = \frac{\frac{0.434\,iK}{0.434\,f(i)} - 0.434\,f(i)}{i\,f(i)} = K - \frac{0.434}{i}$$

Consequently: $$I_c = \frac{i}{K - \frac{0.434}{i}} \qquad (9)$$

TABLE 6

INDEX OF COMPRESSIBILITY, NATURAL PUTNAM CLAY SOIL

i (%)	P (atm)	$\frac{d\log P}{di}$	$\frac{1}{\frac{d\log P}{di}}$	I_c, after Skempton
33.3	4	-0.077	12.9	-
34.0	3.5	-0.077	12.9	12.5
34.6	3	-0.078	12.8	8.8
37.7	2	-0.0725	13.8	17.6
41.2	1	-0.0745	13.2	11.6
45.2	0.5	-0.074	13.5	13.3
Mean	-	-	13.2	12.9

Table 6 gives values of I_c according to Skempton (Col. 5) and values calculated by means of Eq. 9 for the natural Putnam soil using the experimental data of Winterkorn and Moorman (13). The values calculated by means of Eq. 9 are in better agreement with each other than the I_c values after Skempton in which the term 0.434/i is neglected. On the other hand, these calculations show why the straight-line relationship between log P and i is only an approximation and that the true index of compressibility is K - 0.434/i. The approximative character of the linear relationship between log P and i has also been indicated by Macey (18) and by Terzaghi with respect to the relationship between the voids ratio and log P.

There have been other empirical formulae proposed to express the relationship between P and i. Freundlich and Posniak (15) found with other materials:

$$P = P_o \times C^k$$

where P_o and k are constants and C is the concentration expressed in grams of colloids per 1000 gm of colloid plus water; this relationship was applied by Terzaghi to soils. Mattson (6) proposed:

$$P = \frac{B}{i^3}$$ where B is a constant and i the cubic centimeters of water retained per gm of bentonite clay.

These relationships represent only gross approximations and consequently do not fit the experimental data very well. This, of course, is to be expected, because in accordance with osmotic considerations P is a function of a function of i.

It would be desirable to apply Eq. 5 to other soils and engineering characteristics that involve a sufficiently high water content for osmotic suction to play a determinant role. For this purpose, data on consolidation behavior, base exchange capacity, and nature of the exchange ions are required. Unfortunately, these latter properties have not been given sufficient attention by most experimenters. It is hoped that this state of affairs will be rectified and that the osmotic theory in its present form may be tested and verified on the basis of a larger body of experimental evidence. If additional experimental evidence confirms the conclusions drawn from the data on Putnam soil, then a unified physical interpretation can be given to apparently unrelated empirical soil characteristics and be connected by these characteristics with the fundamental properties of the soil at hand ("T" value and character of exchange ions) at moisture contents at which osmotic suction is a determinant factor.

On the other hand, recent investigations have demonstrated the relationship existing between the physical properties of soils and clays and their mineral components. Each of the major known groups of clay minerals has a more or less well defined range of exchange capacity and consequently of osmotic capacity:

Clay Mineral Group	"T" value in m e per 100 g
Montmorillonite	60-100
Illite	20-40
Kaolinite	3-15

Their osmotic activity, however, will depend on the nature of the exchange cations among which the Na^+ ion plays an outstanding part.

OSMOTIC INTERPRETATION OF SWELLING

In the osmotic interpretation of consolidation data obtained on remolded saturated clay soils, equilibrium was reached when the applied pressure P equals the hydrostatic pressure of osmotic origin developed by the dissolved exchange ions.

$P = (P_o)_{real}$ in equilibrium with a water content i. Before the equilibrium condition is reached, the applied constant pressure P performs consolidation work that involves plastic deformation and expelling of water through the pores of the system. For the latter, viscous flow resistance must be overcome. This resistance follows the law of Poiseuille which relates the flow rate to the pressure gradient and the pore radius. Therefore, a certain period of time is necessary for equilibrium to be attained at each constant applied pressure and this period will be the greater the smaller is the effective pore radius. The elimination of water implies an increase in the concentration of the exchange ions and of their osmotic activity "T"/i and f(i) in Eq. 3, i.e. in the osmotic pressure $(P_o)_{real}$ which at a certain water content i reaches the value of P. The osmotic interpretation differs from the mechanical model of Terzaghi in the explanation of the equilibrium condition, but agrees with it on the influence of the time factor.

If the consolidation pressure is removed at equilibrium and contact with free water is maintained, osmotic suction starts because of the dilution tendency of the exchange ions. The system begins to swell, but the swelling is opposed by the deformation resistance of the system and the viscous resistance against the inflow of water. If the previously applied consolidation pressure has been sufficiently high to produce a high $(P_o)_{real}$ at equilibrium condition, this pressure will overcome the resistance of the system and result in swelling which may even perform external work by lifting a surcharge; in other words, a swelling pressure P_s is developed that may be measured by a surcharge that just prevents swelling. Swelling involves an increase in moisture content i; therefore, the initial $(P_o)_{real}$ existing at the removal of the surcharge decreases while the system swells until it becomes equilibrated by the combined effect of the internal resistance and the remaining surcharge. This decrease of the swelling pressure P_o with increasing i makes the swelling process essentially different from the consolidation process which proceeds at a constant pressure.

The possibility of realizing external work from the swelling process and of having a swelling pressure P_s that can be determined experimentaly is a consequence of a low initial moisture content and of an advanced state of consolidation. Such a state can be obtained by consolidation of a system from a high initial moisture content to a point where the moisture just fills the pore space.

By this reasoning, it is possible to explain the shape of both the consolidation and swelling curves as corresponding to existing equilibrium conditions. In consolidation, the applied load P balances the osmotic or hydrostatic pressure existing in the aqueous phase of the system at the equilibrium water content, whereas in swelling, the osmotic pressure is balanced by the surcharge and the internal resistance of the system to expansion and water intake.

Figure 4 presents swelling and consolidation curves derived for Na^+-Putnam clay from the data by Winterkorn and Moorman (13). We consider an initial state a with a water content i that is in equilibrium with pressure P_1 equal to $(P_o)_{real_1}$. If the pressure P_1 is reduced to P_{s2} while the system is in contact with free water, then $(P_o)_{real_1} - P_{s1}$ determines the swelling and the associated decrease in osmotic pressure and increase in i. At the moisture content i_2, the value of the osmotic pressure reaches that of the supercharge P_{s2} and swelling stops. From P_{s1} to P_{s2}, the swelling produces external work that is measured by the area i_1 i_2 c b (pressure x volume, if $i_2 - i_1$ is used to express the volume increase). If one wants to complete the cycle, one must increase the supercharge to P_2 (point d); then, following the consolidation curve, water in the amount of $i_2 - i_1$ is expelled and the initial state is reached by performing an amount of work given by the area $i_2 - i_1$ d a. The area a b c d or P_1 P_2

P_{s1} P_{s2} measures the difference between the work supplied to the system during the consolidation process by the action of the supercharge P_1 and the external work realized from the osmotic swelling. This difference is a measure of the internal resistance of the system to swelling and is called swelling hysteresis.

For small amounts of swelling ($i_2 - i_1 \longrightarrow 0$), the area P_1 P_2 P_{s1} P_{s2} is approximately equal to $P - P_s$ $(i_2 - i_1)$. Hence, for the same increment $(i_2 - i_1)$, $P - P_s$ is a measure of the internal resistance of the system to swelling. This can be read directly from consolidation-expansion equilibrium graphs made from plotted experimental data.

For the case in which the moisture content is i_3 and $P_s = 0$, the consolidation pressure equals P_3. This is the minimum value for an effective consolidation and corresponds to a $(P_o)_{real}$ that is equal to the internal resistance of the system. Higher pressures lead to further consolidation and to $(P - P_s) > 0$; i.e., to swelling pressures that after removal of the external load permit the system to perform external work.

A relationship between the applied consolidation pressure P and the maximum swelling pressure P_s, which the system can produce and which corresponds to a small increase in moisture content (i.e. at practically constant volume), can now be established. A coefficient α that permits the dividing of P into P_s and the internal resistance is introduced in the following manner:

$$\alpha P = P_s; \quad (1 - \alpha) P = P - P_s$$

$$\alpha P + (1 - \alpha) P = P \tag{10}$$

When $\alpha \longrightarrow 1$, $P \longrightarrow P_s$ and $P - P_s = 0$, then one is dealing with strongly consolidated soils having a low moisture content, a high $(P_o)_{real}$, and consequently high swelling pressures if the internal resistance does not compensate for the $(P_o)_{real}$ that has been developed.

When $\alpha \longrightarrow 0$, $P_s \longrightarrow 0$ and $P - P_s \longrightarrow P$, the system is one of low consolidation, high moisture content, low $(P_o)_{real}$, and small swelling pressure. When $(P_o)_{real}$ is equal to or smaller than the internal resistance, no expansion or production of external work is possible. For each soil there exists a maximum value for P at which $\alpha = 0$ and $P_s = 0$.

From the consolidation-swelling graphs given by Winterkorn and Moorman (13) for natural and homoionic Putnam soils, the P and P_s values for moisture contents have been taken that were practically equal for the extreme cases, i.e. for $P_s = 0$ and, in the case of maximum values, for equal P values. These are shown in Table 7.

It can be seen that the internal resistance compensating $(P_o)_{real}$ for $P_s = 0$ corresponds to a moisture content and degree of consolidation at which contact between the solid particles or the strongly bound water films around these particles is already possible. This moisture content is distinctly higher for the Na^+ soils.

It can be expected that α is a function of the degree of consolidation which determines the free pore space, and of the moisture content, which determines $(P_o)_{real}$. The swelling curves of Winterkorn and Moorman (13) have a flattened shape and it was possible to extract only a few points that indicate in a preliminary manner that α decreases linearly with i and increases exponentially with P_1 starting at a P_o value that corresponds

TABLE 7

Soil Modifications	i (%)	P (atm)	P_s (atm)	$P-P_s$ (atm)	$a = \frac{P_s}{P}$
Natural	36.2	2.8	0	2.8	0
	34.0	4	0.9	3.1	0.25
Potassium	34.7	2.4	0	2.4	0
	31.8	4	0.5	3.4	0.125
Magnesium	37.7	2.8	0	2.8	0
	34.8	4	1.2	2.8	0.30
Sodium	49.0	2.2	0	2.2	0
	41.3	4	1.6	2.4	0.40
Hydrogen	38.4	2.8	0	2.8	0
	36.2	4	0.7	3.3	0.175
Calcium	37.0	2.6	0	2.6	0
	33.4	4	2.2	1.8	0.55

with $a = 0$. It is desirable to ratify these relationships on the basis of a larger body of experimental data.

The osmotic interpretation of the consolidation and swelling phenomena occurring in saturated clay soil systems permits the recognition and organization of the causative factors of these phenomena in the following manner:

1. The active swelling force is the osmotic pressure of the aqueous phase of the soil-water system. This force decreases with increasing water content. The soil characteristics that govern this force have been treated earlier.

2. Opposed to this active force is an internal resistance of the soil-water system expressed by the coefficient a. The internal resistance is the sum of (a) the mechanical resistance of the system to expansive deformation and (b) the viscous resistance to the incorporation of swelling water into the pores of the system. This sum is a function of the viscosity of the water and the permeability of the system.

3. The capacity for swelling and for developing a swelling pressure that is able to perform external work depends on the relative magnitude of the swelling pressure and the internal resistance.

4. Remolded, noncemented, expansive soils possess a low resistance to deformation and a high one to viscous flow. The coefficient approaches unity when a high pressure of consolidation results in a low initial moisture content and a high $(P_o)_{real}$; $a \longrightarrow 0$ under opposite conditions.

5. Undisturbed or cemented, structure-endowed, expansive soils possess a high resistance to deformation and may absorb a large part of $(P_o)_{real}$; consequently $a \longrightarrow 0$.

6. When the clay fraction of a soil occupies the spaces between the coarse granular components of a soil, then the swelling of the clay results in expansion of the system if the combined volume of clay particles and osmotic water exceeds that of the intergranular pore space. If the combined volume of clay and osmotic water is smaller than the intergranular porosity, then $\alpha \longrightarrow 0$; if larger, then $\alpha \longrightarrow 1$, as shown by Winterkorn and Choudhury (20) and Winterkorn (21).

7. When the initial soil-water system is not watersaturated, then water intake can take place by dislocating air without increase in total volume. It is impossible, however, to expell all air. Occluded air will be compressed by the swelling pressure in accordance with Boyle's law and dissolved in part in accordance with Henry's law. The result of this is a decrease in active swelling pressure. Altogether, the swelling press that a given soil in a given condition can develop depends fundamentally on the osmotic activity of the cations in the aqueous phase and on the value of α.

THE SWELLING OF SUBGRADES

The swelling pressure and the volume changes that may take place in a subgrade underneath a highway pavement are now examined. A subgrade soil compacted at Proctor optimum moisture content to maximum dry density, which expands against a supercharge that equals the weight of the contemplated superimposed structure is assumed. Of primary interest is the swelling pressure that may be developed with volume changes small enough to leave the structure unaffected. This means that the soil volume should remain practically constant or $\frac{\Delta V}{V} < 1$ percent.

For the determination of the maximum swelling pressure in the laboratory under conditions that duplicate as closely as possible those in the field, one can use the apparatus of Hveem (15) or its adaptation for determining the C.B.R. as used by Duarte Marchetti, and Ruiz (22) in the laboratories of the Highway Department of the Province of Buenos Aires. Test specimens are made with compaction requirements similar to those specified for the road project — at optimum moisture content as well as on the dry and wet side of it. The mold with the specimen is placed within a rigid frame. The surface of the specimen is covered with a porous plate and a compressometer is fastened between this plate and the upper arm of the frame. The mold with specimen is submerged in water and the compressometer dial registers simultaneously the swelling pressure and the expansion. One uses compressometers that, at maximum swelling force reached as a function of time, register a volume increment less than 0.5 percent of the original volume. This means that the volume remains practically constant and that a condition of permanent equilibrium exists between swelling pressure and supercharge.

Table 8 gives swelling pressures measured by this method for typical cases. These cases show as a generally observed characteristic the rapid fall of the swelling pressure with increase in initial moisture content.

The basic purpose of this work is to develop a theory that permits the interpretation of the preliminary results that have been obtained so far with different soils of the Province of Buenos Aires and that can serve as a guide to the correction of swelling, a problem of great technical and economic interest.

TABLE 8

MAXIMUM SWELLING PRESSURES AT PRACTICALLY CONSTANT VOLUME OF TYPICAL SOILS[1] OF THE PROVINCE OF BUENOS AIRES

Soil	Characteristics	Compaction Moisture (%)	Max. Swelling Pressure (kg/cm^2)
A	SL = 18 LL = 34 PL = 23 Passing 200 = 98% HRB Class. A-6(9)	20 23[2] 25	0.172 0.080 0.011
B	SL = 10 LL = 44 PL = 25 Passing 200 = 84% HRB Class. A-7-6(12)	18 23[2] 28.5	1.490 0.529 0.230

[1] Compacted by the Standard Proctor Method at, below, and above optimum moisture content.

[2] Optimum.

The first point to be kept in mind is the initial moisture content. From the osmotic point of view this governs the active force of swelling, which is the osmotic pressure developed. Considering soils compacted by the standard Proctor method at optimum moisture content, the compaction work employed amounts to 5.5 lb x 25 blows x 18 in. = 12.4 ft-lb per cu ft = 60.4 kg per cu dm. These amounts are used up by the frictional and cohesive forces that oppose themselves to the displacement of the particles. The actual work performed in this compaction process is necessarily smaller than that required for consolidation to the same density starting with an excess of water. This is because of the work involved in overcoming the viscous resistance of the latter. Consequently the amounts of work involved in compaction and consolidation are not strictly comparable. Also, a soil-water system compacted at optimum moisture content with the Proctor standard method is not saturated, but possesses an air-pore volume of the order of 5 percent. Thus, in order for such systems to be saturated, an increase in moisture content of the order of 3 to 5 percent of the dry weight of the soil is necessary.

The fact of unsaturation implies that osmotic suction can superimpose itself on capillary suction. The capillary suction later stops at saturation while swelling continues as long as the osmotic pressure exceeds the resistance to volume increase.

The problem of establishing a moisture content for subgrade soils that is representative of that in equilibrium under field conditions is well known and need not be treated here. It need only be stated that there exist different criteria for judging the danger of water saturation and swelling under service conditions. These criteria are reflected in the different test methods proposed for obtaining data to be used in design.

As examples, the following are mentioned:

1. The criterion of Porter: saturation in the presence of free water, the specimen being confined by a supercharge equal to the pressure exerted by the pavement.

2. The criterion of Hveem: no water exudation when the specimen is submitted to a pressure of 28 kg cm^{-2}.

3. The criterion of McDowell: saturation under conditions of normal and lateral confinement that duplicate service condition.

All this shows the great need for a theory to explain the suction of water and the swelling of a soil system after the existing capillary porosity has been saturated. Such a theory is particularly important for soils of marked swelling characteristics.

The osmotic theory explains, at least qualitatively, the experimental and observational facts. For a certain degree of densification, low initial moisture contents give rise to large osmotic pressures particularly in the case of Na^+ exchange ions possessing high osmotic activity. Osmotic suction does not stop at pore saturation of a soil and may still be present to a marked degree at the moisture content of the liquid limit. The osmotic suction produces a hydrostatic pressure (analogous to the pore pressure) that acts against the solid phase and tends to expand the system against the resistance of the solid phase. The osmotic suction forces must also overcome the resistance of the porous system to the entrance and distribution of water. Simultaneously with water sorption, the osmotic pressure decreases tending to a new equilibrium value. Low initial moisture contents and associated high osmotic pressures may also exist in granular-cohesive soil systems that are possessed of a granular skeleton, the intergranular spaces of which contain the swelling clay material. Under such conditions, internal swelling may occur without external increase in the volume of the soil system (20,21).

There are two basic reasons for the importance of the character of the exchange ions in the swelling process. Ca^{++} and H^+ ions possess less osmotic activity than the Na^+, K^+, and Mg^{++} ions as expressed by the value of the constant K and a greater tendency to aggregation, which tendancy favors formation of a flocculated structure and results in increased internal resistance to swelling. This explains the reduction of volumetric swelling by the incorporation into a soil of portland cement or hydrated lime often mentioned in foreign and domestic literature (23,24,25,26). No study seems as yet to have been undertaken on the effect of these admixtures on the swelling pressure though this research with soils of the Province of Buenos Aires is being planned. Theoretically, small admixtures of hydrated lime should be sufficient for this purpose as long as they are uniformly distributed and given enough time for effective cation exchange. It is important to keep in mind that the influence of Na^+ exchange ions on the physical properties of soils are already pronounced when these ions represent only about 15 percent of the total exchange capacity (27). Hence, marked changes in volumetric expansion and swelling pressure which codetermine the swelling work may be expected already with additive concentrations well below those calculated on the basis of the "T" value.

The action of soluble salts, such as $CaCl_2$, employed in the control of soil swelling is of a more complex nature. Its theoretical treatment employs the Donnan equilibrium, to be shown later.

As previously indicated, experiments show a rapid fall of the maximum swelling pressure, at practically constant volume of the system, with a small increase in moisture content when the moisture content is in the range of the optimum for compaction. The osmotic theory permits the derivation of the function that connects P_s and i.

When Eqs. 5 and 10 are combined,

$$P_s i = \frac{R\ T\ "T"}{Va} \times \alpha \times f(i) \qquad (11)$$

which expresses the condition of equilibrium between the excess osmotic pressure developed with respect to the internal resistance of the system and the supercharge P_s. Eq. 11 shows the rapid fall of P_s with increase in i; the term R T "T"/Va is a constant for each specific soil and temperature and is directly proportional to the base exchange capacity and inversely to the valency of the exchange ions. The term α f(i) is a combined expression of the internal resistance α and the osmotic activity f(i); both functions decrease with increasing moisture content i and are influenced by the nature of the exchange cations.

The term α f(i) can be evaluated provided that the remaining terms of Eq. 11 can be determined experimentally. Preliminary experiments show that α f(i) falls exponentially with increasing i, because log $[\alpha\ f(i)]$ is a decreasing straight line function of i. With some soils, this function deviates from linearity at low moisture contents. This may be interpreted as a consequence of internal expansion in systems possessing a granular skeleton with fixed intergranular porosity (20,21).

The importance of these preliminary results and of the application of the osmotic theory lies in the fact that they explain why one cannot expect simple relationships to exist between P_s and i inasmuch as the first is of necessity a function of a function of the moisture content. The swelling of soils and particularly of hydrophilic colloids in general has been considered for many years as a complex phenomenon in which purely mechanical factors are of importance though the fundamental explanation must be sought in the electric charges of the micelles and in osmotic phenomena (Ostwald and Mündler, 1919). The osmotic theory gives a first rational explanation of the influence of these factors when the moisture content of the soil is such that osmosis without membrane is the predominant phenomenon. This occurs in soils when the moisture content exceeds that of the shrinkage limit and becomes the exclusive factor in the saturated state where the capillary forces cease to exist. For the future development of this theory, it is necessary that methods be found that permit separate determination of α and f(i) and the establishment of correlations that connect these functions with moisture content, degree of densification, and soil characteristics.

THE INFLUENCE OF FREE ELECTROLYTES—DONNAN EQUILIBRIUM

So far, only the influence of water on soil-water systems in the absence of free electrolytes or salts has been considered. Soils containing more than 0.2 percent of soluble salts are called saline and one differentiates between weakly salinized (0.2 to 0.5 percent) and strongly salinized (> 0.5 percent) soils (Rosov). At a soil moisture content of 20 percent the salts present in the liquid phase cause by themselves alone osmotic pressures of the order of tens of atmospheres and even larger if they are separated from free water by means of a semipermeable membrane. In the absence of such a membrane, free diffusion takes place and no osmotic pressure is developed.

The free electrolytes interfere, however, with the osmotic pressure developed by the exchange ions which are held in the electrostatic adsorption field of the solid soil particles. This interference gives rise to a counterpressure called "Donnan effect." Consider the simplest case can be taken from Winterkorn (8), namely a homoionic Na^+ clay in a NaCl solution of molar concentration a. The contact surface AB in Figure 5 is pervious to water and free ions but not to the electrostatically restrained exchange ions. The molar concentration of the Na^+ exchange ions shall be called z.

If there were no Na^+ exchange ions, then Na^+ and Cl^- ions of the external solution would diffuse into the internal water phase until the concentration in both solutions were the same. In the presence of the Na^+ exchange ions, the external Na^+ and Cl^- ions diffuse only until thermodynamical equilibrium is established. The latter requires that the concentration product in both solutions be equal.

If q is the loss in concentration of the external solution, then the equilibrium condition is

$$q\ (z + q) = (a - q)^2$$

FIGURE 5

DONNAN EQUILIBRIUM

Internal Solution				External Solution		
$Clay^-$	Na^+ z		A B	Na^+ a	Cl^- a	Initial State
$Clay^-$	Na^+ $z + q$	Cl^- q	A B	Na^+ $a - q$	Cl^- $a - q$	State of Donan Equilibrium

which permits the calculation of q as a function of the known z and a values:

$$q = \frac{a^2}{z + 2a}$$

The number of external ions that cross the boundary AB depends, therefore, on the relationship between a and z.

First case: $a = 0$. This applies to nonsaline soils, i.e., soils not containing free electrolytes. The molar concentration of the internal solution corresponds to that calculated from the base exchange capacity as employed in the preceding paragraphs.

Second case: $a < z$. This applies to clays of high base exchange capacity and low salt content. The equation for the Donnan equilibrium shows that the osmotic pressure developed under equilibrium conditions is smaller than that calculated from Van t'Hoff's law because it depends now on the difference in concentration of the internal and external ions. This is the basic principle of Mattson's theory of "osmotic imbibition" (6). In this case, the difference or excess e will be equal to a, and $e = (2q + z) - 2(a - q) = 4q - 2a + z$. As the value for q is unknown, one substitutes its value as a function of z and a and obtains

$$e = \frac{4a^2}{z + 2a} - 2a + z = \frac{z^2}{z + 2a}$$

Then, in accordance with Mattson, the osmotic pressure becomes

$$P = RT \frac{z^2}{z + 2a}$$

and water will be sucked from the external to the internal solution. According to the author's concept, the right hand part of this equation should be multiplied by f(i) to account for the actual osmotic activity of the exchange ions.

Third case: $a > z$. This applies to clays with low base exchange capacity and concentrated electrolyte solutions. In this case, $q \longrightarrow 1/2a$, which means that at the limiting case the external Na^+ and Cl^- ions tend to distribute themselves uniformly throughout the entire liquid medium, thus annulling the osmotic effect of the exchange cations. One sees clearly that the increase in free electrolyte concentration counteracts the osmotic suction and hence swelling. This is a well-known experimental fact shown, e.g., in the classical experiments by Winterkorn and Eckert and Schappler (8) who determined the free swelling of a bentonite Na^+ clay in solution of NaCl of increasing concentration and of Ca^{++}-Putnam soil in solutions of $CaCl_2$. However, experiments concerned with the influence of free electrolytes on the consolidation phenomenon and the swelling pressure are not known to the author.

Besides the use of Donnan equilibrium in a case where the clay was homoionic and had a cation in common with the electrolyte, the Donnan equilibrium can also be applied to more complicated systems where there are no common ions or where the ions are polyvalent. The resulting equations are more complicated, but the direction of the electrolyte influence is the same.

In recapitulation, the basic concept is that the osmotic suction of water is due to the retention of the exchange ions by the electrostatic charges on the surfaces of the clay particles which counteracts their kinetic energy and restrains their ability to diffuse. Osmotic suction occurs in the absence of free electrolytes in accordance with the phenomenon of osmosis without membrane to which the equation of Van t'Hoff can be applied using, however, the "osmotic activity" instead of the molar concentration. The Donnan equilibrium that is produced in the presence of free electrolytes appears to act as a reducing and not as a determinant factor with respect to osmotic suction. Mattson's ideas are useful in the explanation of the depressing action of salts on osmotic suction in the case of saline soils.

In order to clarify the different concepts, the osmotic pressure that Na^+-Putnam clay can develop at optimum moisture content from the different points of view will be calculated. It is assumed that the dissociation of NaCl in Na^+ and Cl^- ions is complete, that their activity in dilute solutions is unity, and that the "osmotic activity" f(i) of the exchange Na^+ is the same as in the absence of the electrolyte. These simplifications are acceptable in first approximation when the free electrolyte concentration is small.

Taking: Optimum moisture content = 31.3; "T" = 30; Va = 1; T = 273 + 25

$\log (f(i))_{i = 31.3} = 0.0235 (31.3 - 12) = - 0.454 = 1.546;$

$f(i) = 0.352$

Applying the equation of Van t'Hoff directly in the absence of free electrolyte, i.e. for a = 0, one obtains

$$z = [c] = \frac{"T"}{I\ Va} = \frac{30}{31.3 \times 1} = 0.96$$

$P_o = 0.082 \times 298 \times 0.96 = 23.4$ atm. This P_o holds for the case that the aqueous phase behaves as an ideal solution. In the absence of electrolytes, there exists no Donnan equilibrium and consequently the cation excess is $z = [c]$. Introduction of the osmotic activity f(i) reduces this value to $P_o = 0.082 \times 298 \times 0.96 \times 0.352 = 8.2$ atm.

We consider now the presence of 0.19 g of NaCl per 100 g of dry soil, which when dissolved in the aqueous phase of the soil-water system gives a = 0.1. This free electrolyte may diffuse freely and does not modify P_o according to Van t'Hoff's law. Application of the Donnan equilibrium reduces the active concentration to the excess of internal over external concentration and one obtains

$$e = \frac{(0.96)^2}{0.92 + (2 \times 0.1)} = 0.80$$

which gives an osmotic pressure of

$$P_o = 0.082 \times 298 \times 0.80 = 19.6 \text{ atm.}$$

This is 16 percent less than that calculated according to Van t'Hoff. Considering now the real osmotic activity of the exchange ions as affected by the reduction produced by the Donnan equilibrium, one obtains

$$P_o = 0.082 \times 298 \times 0.80 \times 0.352 = 6.9 \text{ atm.}$$

Thus, the presence of the NaCl reduces the $(P_o)_{real}$ from 8.2 to 6.9 atm.

These calculations show that the effect of an assumed Donnan equilibrium in the case of nonsaline soils is much smaller than that of the osmotic activity f(i). This is a logical consequence of the concept of osmotic suction as due to osmosis without membrane.

In turn, the Donnan equilibrium gives a first if semiquantitative explanation of the influence of free electrolytes in the case of saline soils. The calculation made before involving high concentrations of NaCl do not correspond with actual behavior, inasmuch as the simplified assumptions, by means of which calculation was made possible, are incompatible with reality.

With respect to practical applications, such as adding soluble salts for the correction of soil swelling, one must keep in mind that such salts produce ionic interchanges and modify the soil properties. Also, one can use only such salts as do not expand by crystallization in their hydrated forms. For economical and technical reasons, the most promising salt for the indicated use is calcium chloride, which has been employed in practice for a long time mainly because of its hygroscopic properties.

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Moisture, Density and Volume Change Relationships Of Clay Soils Expressed as Constants of Proportionality

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The project's purpose was to devise a method for establishing emperical relationships between volume change and density, and/or volume change and moisture content.

The soil samples employed were homoionic soils prepared from a Warsaw soil by leaching portions of the soil with suitable electrolytes containing one of the cations chosen, namely hydrogen, aluminum, ferric, and calcium ions.

The prepared soils were tested in various combinations of density and moisture content conditions for swell characteristics.

Additions of organic compounds were investigated to determine the influence of natural soil organic matter on swell properties of the prepared homoionic soils. Lignin was used to represent natural soil organic matter in equilibrium with soil microbial activity. Hypotheses derived from the investigation are given.

● SUBGRADE RESEARCH was initiated in the United States about 1925. Failures of pavements designed to ultimate standards by engineers were investigated to determine contributing factors other than pavement deficiencies. Improved construction methods, thicker pavements, and perfected materials did not prove to be the solution to the pavement failures.

Studies conducted in Michigan (1) indicated that more than half of the pavement failures were contributed to by subgrade failures. These subgrade failures resulted from differential expansion or settlements of the subgrade materials. Studies (2) of flexible road surfaces also proved that the subgrade was a contributing factor to road surface failures. Later investigations (3,4) established definite correlations between concrete pavement failures in the form of crack patterns and the soil texture of the subgrade soil materials.

Pavement pumping, blow-ups, faulting, and longitudinal cracking has also been closely correlated to subgrade soils and soil characteristics (5,6). Investigations and studies of pavement failure have indicated that the failed sections are the consequence of conditions of excessive moisture and resulting loss of cohesion; high percentages of fines, silts, or fine sands and resulting ice lense formations with heaving; excessive volume change, shrinkage, or swelling, resulting from moisture variations or the physical or chemical alteration of the soil constituents. Swelling may also result from the removal of earth surcharge; however, this condition is not pertinent to this study.

The presence of excessive moisture in the subgrade is one of the most important factors in eliminating the susceptibility of the soil body to volume change.

The presence of organic material in the soil body further complicates the stabilization of the subgrade material. The organic material is accepted to be compressible and easily decomposed by soil microorganisms. The organic matter is also believed to possess a capacity of absorbing considerable moisture. There are very few known facts concerning the effects that varying proportions of organic materials in the soil have on the strength and stability of the subgrade. In general, organic materials have been considered detrimental and their elimination from subgrade soils has been practiced, except in very minor amounts. The following are the essential facts used as the basis of this study:

1. The clay and colloidal fraction of a soil are capable of base exchange and adsorption activities, both of which are surface phenomena (7).

2. Cohesive soils contain sufficient proportions of clay and colloidal materials to provide for surface phenomena to influence the properties of the total soil body.

3. Water molecules are dipoles attracted by ions through polarization and orientation in the clay mineral fraction, either on the broken bonds or between oxygen planes (8).

4. The character of the adsorbed ions saturating the exchange positions of the clay mineral and colloids determines the amount of water that will be taken up under optimum conditions during swelling (9).

5. Principles of base exchange and adsorption allow the surface character of soil colloids to be altered by chemical means so that the water-attractive character of the colloid's surface may be altered to water-repellent properties (9).

6. Soil organic compounds may have a base exchange capacity (10,11).

7. The organic matter in a soil is protected by the clay minerals, and the clay minerals have been found to inhibit peptization of various proteins (12).

8. The exchange positions and adsorption points of a clay mineral may be filled with organic cations instead of easily hydrated cations or water molecules, which would normally occupy them (13).

9. Of the organic materials found in soils, lignin has been determined to be the most resistant to microbial decomposition (14).

10. An excess of organic material in a soil beyond that adsorbed by the clay and colloids can in itself become an agent for absorbing water molecules and result in swelling of the soil mass.

This study was carried out under these stated premises, to begin to evaluate the effects produced by varying exchangeable bases and organic matter additions on volume change, density, and moisture content and to establish emperical relationships between the variables.

TABLE 1

MECHANICAL ANALYSIS OF UNTREATED SOIL

Analysis	Particle Size (mm)	Cumulative % Passing
Sieve (US Standard Sieve):		
1- to 1 1/2- in.	-	100.0
1- in.	-	100.0
3/4- in.	-	98.7
3/8- in.	-	96.7
No. 4	4.8	94.2
No. 10	1.98	89.8
No. 20	0.833	86.3
No. 40	0.417	73.8
No. 60	0.246	64.0
No. 140	0.104	56.6
No. 200	0.074	55.2
Hydrometer		
	0.032	54.6
	0.020	52.4
	0.012	43.9
	0.009	40.2
	0.0061	36.5
	0.0032	29.4
	0.0013	26.9

MATERIALS

The soils chosen for study were typical of some of the road building soils available in various locations of southeastern Michigan. A description of the soil is given in Table I, with the grain-size distribution shown in Figure 1. The samples obtained were taken from the B and C horizon and were a yellowish-brown to reddish-brown coherent gritty loam.

Cationic modifications of the raw soil were made to produce a homoionic soil. The procedure adopted was that previously used by Winterkorn (15). The process consisted of leaching the soil with electrolyte containing the desired cation in enough concentration to saturate the base exchange capacity of the soil. It must be assumed that the process results in base saturation.

Homoionic soils of "hydrogen," "aluminum," "ferric," and "calcium" were produced by leaching the raw soil with the corresponding chlorides and subsequently washing the treated soils with distilled water until the washings were tested as free of the chloride ions.

Lignin was chosen to represent the organic matter in equilibrium with the soil microbiological activity. It was assumed that, by the use of lignin, no further organic decomposition would take place in the soil.

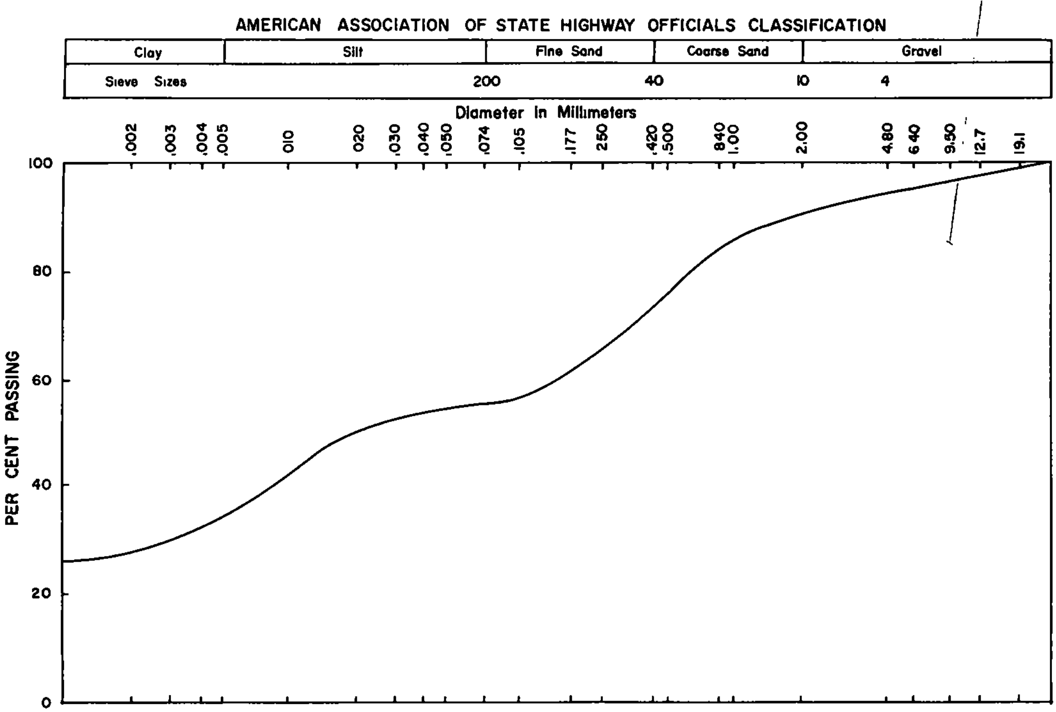

Figure 1. Particle size distribution of Warsaw soil sample (Horizons B and C).

METHODS OF TESTING

The homoionic soils used were air dried to a uniform moisture content and fractioned to insure that all the material passed a No. 4 sieve.

The method of testing adopted for the swell studies was the standard method for the determination of volume change of soils, first proposed by W. K. Taylor (16) of the Bureau of Public Roads and designated in the AASHO Standard Specifications as T 116 - 54.

The apparatus used in these studies was devised from a Soiltest consolidometer body and frame to which a perforated piston was added. A micrometer guage was used to measure the upward movement of the perforated piston as it was forced upward by the swelling action of the soil. The soil sample was maintained at the moisture level of saturation throughout the entire tests. The apparatus was modified from the standard AASHO designation, which accommodates a 4-in. diameter sample 1.5625 in. thick, to a much smaller sample, which after compaction to near-Proctor standards was 2 1/2 in. in diameter and 1 in. thick. The diameter thickness ratio is thus maintained.

A calculated weight of the desired soils were compacted to approximately 120 pcf, dry density, at varying moisture contents from 9 to 14 percent. This compaction was done with static loading to insure uniformity.

Readings were recorded at 15-min intervals for the first hour, then hour intervals for the next 5 hours, and twice daily for the remaining

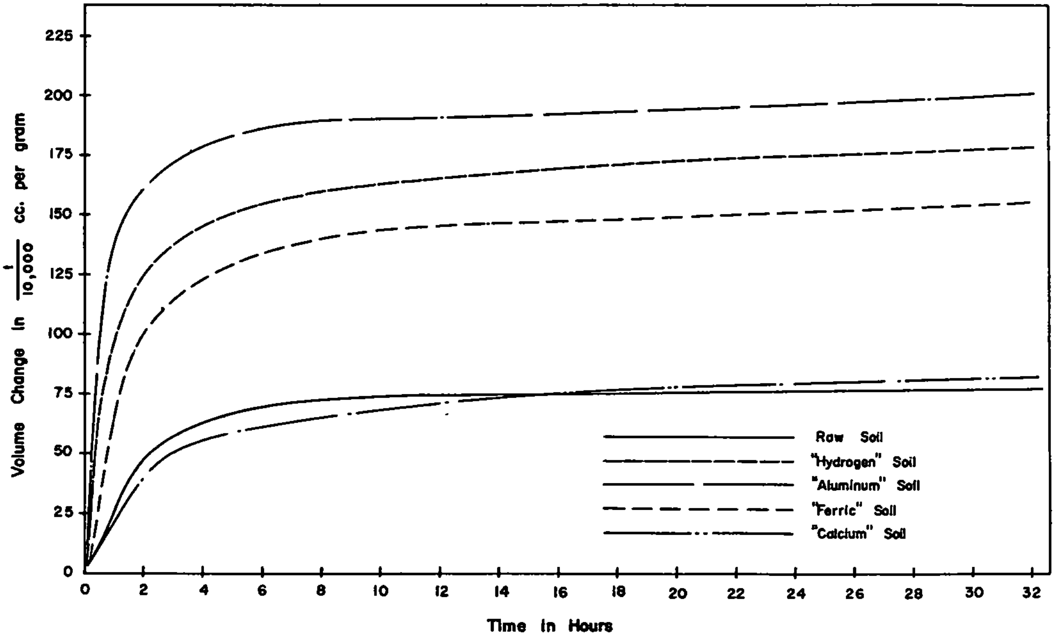

Figure 2. Swelling of homoionic soils compacted at their respective optimum moistures and densities.

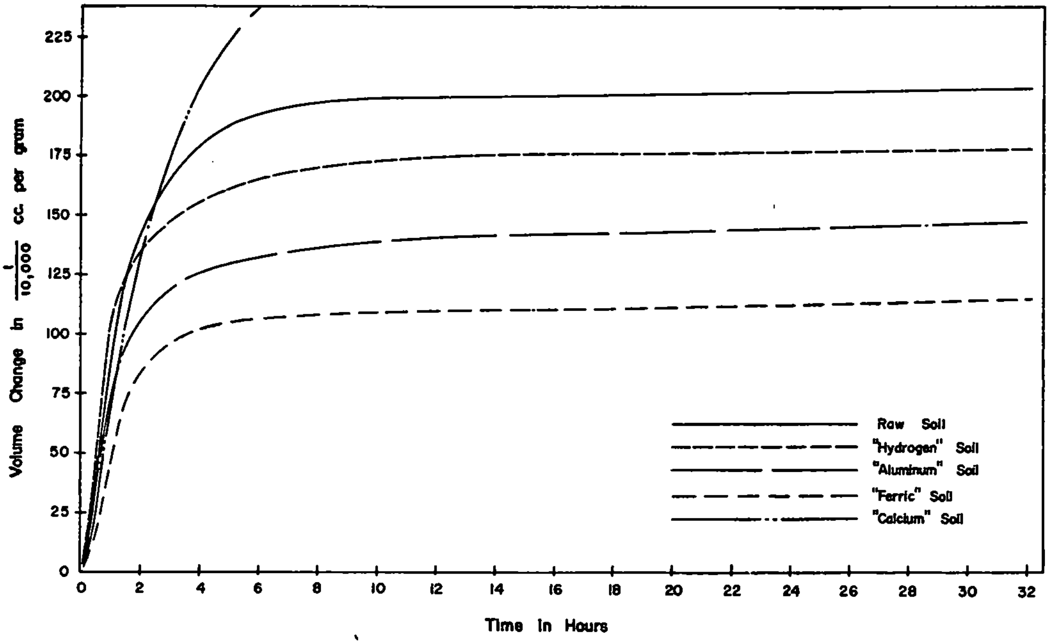

Figure 3. Swelling of homoionic soils compacted at a density of 120 pcf and a moisture content of 13 percent.

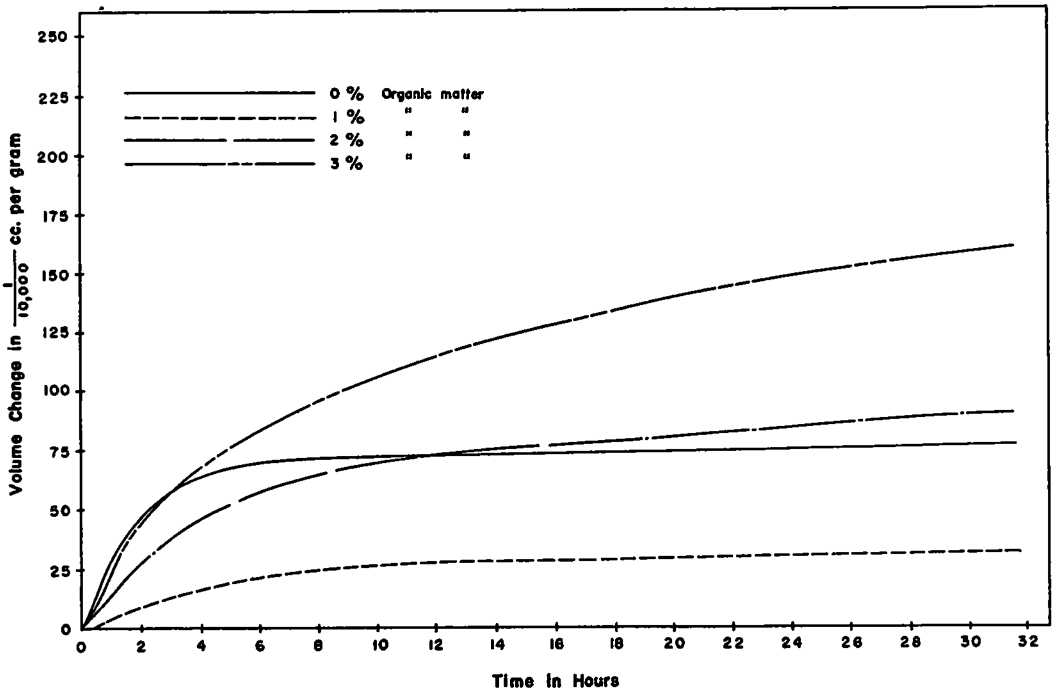

Figure 4. Swelling of "raw" soil at various percentages of lignin.

period that appreciable movement occurred. Readings were terminated on a given sample when the vertical movement of the piston did not exceed 0.001 in. in an 18-hour period.

EVALUATION OF TEST RESULTS

The general effects of the various ions of the homoionic soils on the swell characteristics may be compared to the swell characteristics of the raw soil. Figure 2 shows that the soil samples were compacted at their respective optimum moistures and densities. When the soil samples were compacted at a uniform dry density and moisture content, variations in swell were observed. An example of this is shown in Figure 3, where a uniform dry density of approximately 120 pcf and a moisture content of 13 percent were maintained. When employing optimum moisture and density the Al, Fe, and H soils exceeded the raw soil in swell, and the Ca was slightly lower in swell. When compacting the samples at uniform density and moisture, the Ca soil exceeded the raw soil in swell and the H, Al, and Fe soils were respectively lower than the swell of the raw soil.

The additions of organic matter in the form of lignin varied the effects upon swelling, dependending on the ion of the homoionic soil with which it was incorporated and on the amount of lignin added. In some instances small amounts of lignin decreased the swelling, while greater amounts added to the same soil increased swelling. Figure 4 shows this effect of varying percents of lignin added to the raw soil. Figures 5, 6, 7, and 8 show respectively the effects of varying percents of lignin on homoionic soils of H, Al, Fe, and Ca.

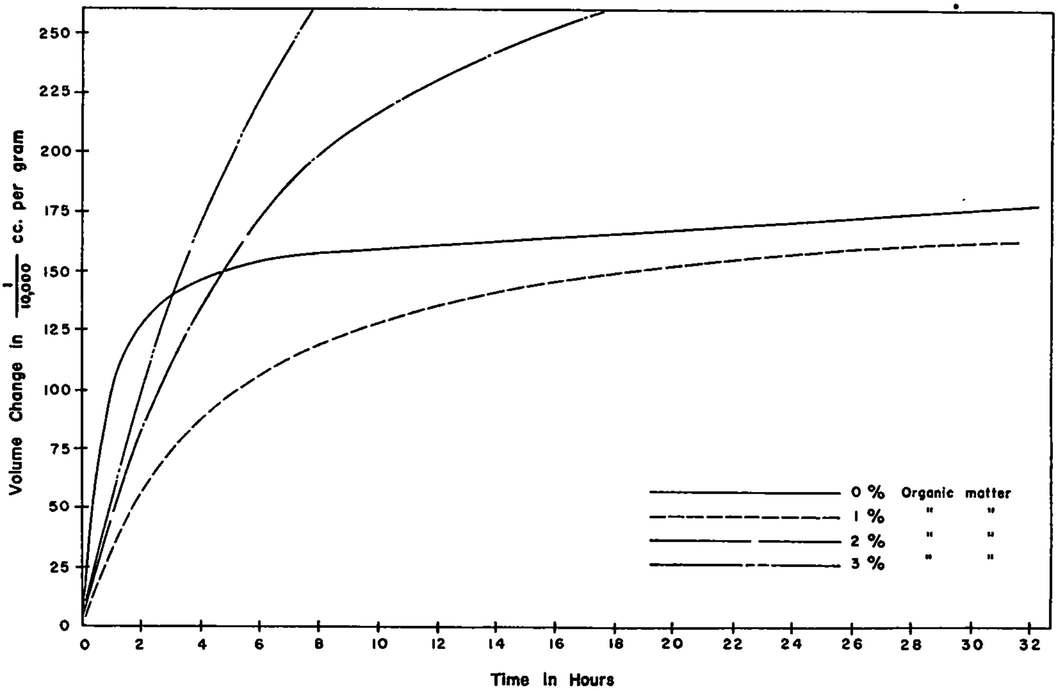

Figure 5. Swelling of "hydrogen" soil at various percentages of lignin.

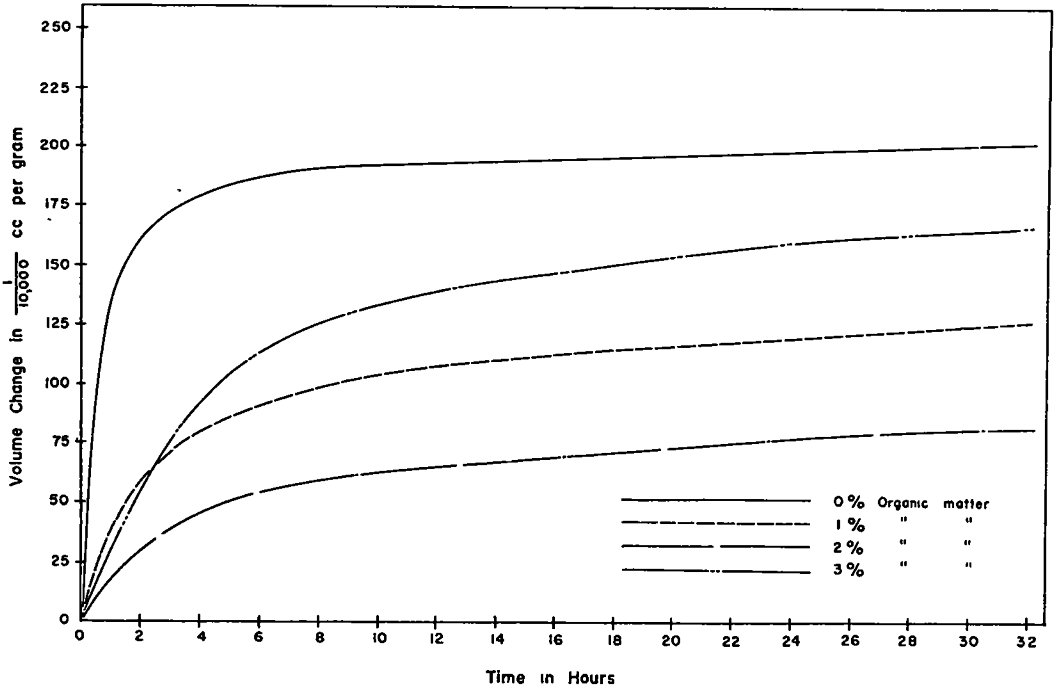

Figure 6. Swelling of "aluminum" soil at various percentages of lignin.

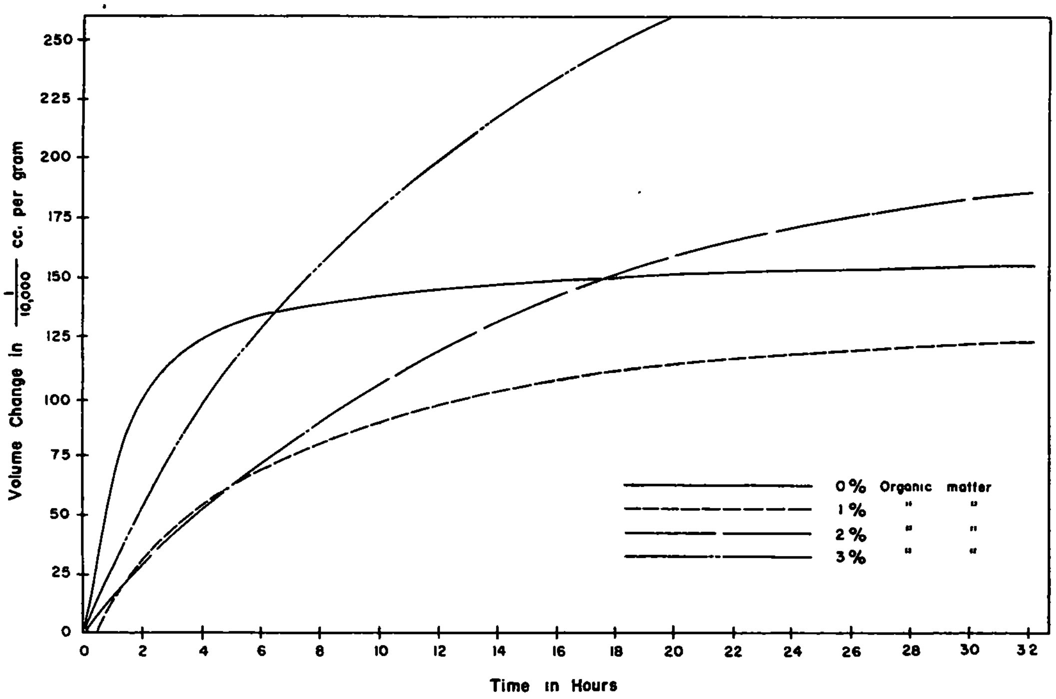

Figure 7. Swelling of "ferric" soil at various percentages of lignin.

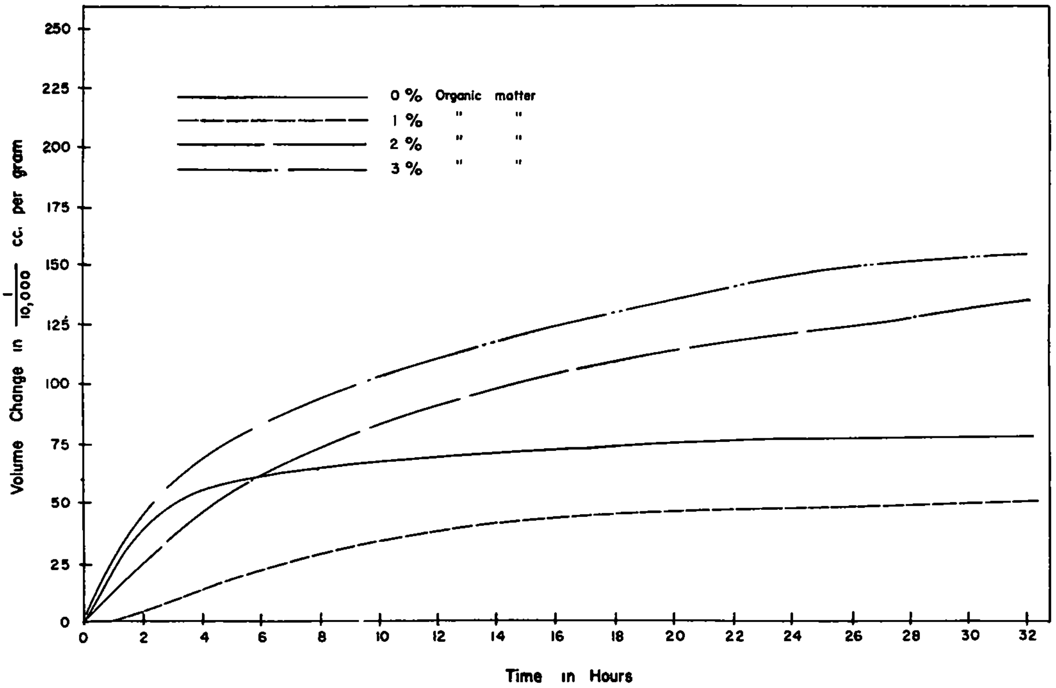

Figure 8. Swelling of "calcium" soil at various percentages of lignin.

Additions of any lignin to a hydrogen soil increased swelling over the raw soil. Additions of 1 percent to the raw, ferric, and calcium soils reduced swell, whereas over 1 percent additions increased swelling. The aluminum soil would tolerate additions of lignin up to 3 percent without any increase in swell, and 2 percent lignin had only one half the swell of the nonadditive homoionic aluminum soil.

It can be generally stated that, for two soil samples compacted at the same dry density, the volume change for that compacted at the lower moisture content is greater. In correlation to moisture content, in any two samples of a given soil compacted at equal moisture contents but with varied densities, the one compacted at the greater dry density will exhibit the larger volume change.

To express the relationships of the moisture-density effects on swelling of soils, constants of proportionality in the form of exponential curves were developed.

The development of the constants of proportionality were based on the following considerations:

1. For two samples compacted at the same density, the one with the lower moisture content indicates greater swelling.

2. For two samples compacted at the same moisture content and occupying the same volume, the sample with the higher dry density has the greater swelling.

3. In the computations, D_2 always refers to the density of the sample at the higher moisture content, and w_2 refers to its moisture content.

The relationships and defination of the constants are as follows:

$$K_1 = f\ (D_2/D_1)$$

$$K_2 = f\ (w_1/w_2)$$

$$K_3 = f\ (t)$$

$$K_4 = (K_1)\ (K_2)$$

$$K = (K_4)\ (K_3)$$

Therefore $K = (K_1)\ (K_2)\ (K_3)$

S_1 and S_2 are corresponding swellings, and are expressed as:

$$S_2/S_1 = f\ (D_2/D_1,\ w_1/w_2,\ t)$$

If the densities of the two samples are equal:

$$S_2/S_1 = f\ (w_1/w_2,\ t)$$

If the moisture contents of the two samples are equal:

$$S_2/S_1 = f\ (D_2/D_1,\ t)$$

If the time of swell is equal for the two samples:

$$S_2/S_1 = f(D_2/D_1, w_1/w_2)$$

If the densities, moisture contents and time of swell are equal:

$$S_1 = S_2$$

The curves in Figures 9, 10, 11, and 12 show the plots of proportionality constant "K" varying with the time in hours.

The swell of two samples at varying moisture contents is determined on the basis of a proportionality constant "K" as follows:

$$S_2/S_1 = K$$

K is a function of percent moisture and time, K_2 and K_3, respectively, only because the density K_1 is a constant at 120 pcf. The swell of the samples, S_1 and S_2, are expressed in cubic centimeters per gram of dry soil in each sample and are based on the original sample volume.

The curves are indicative of an exponential expression for a given comparison of swell characteristics of a soil. The variations of curves for a given soil may be attributed to the effects of moisture percent upon the swell characteristics of the soil and it is evident that there

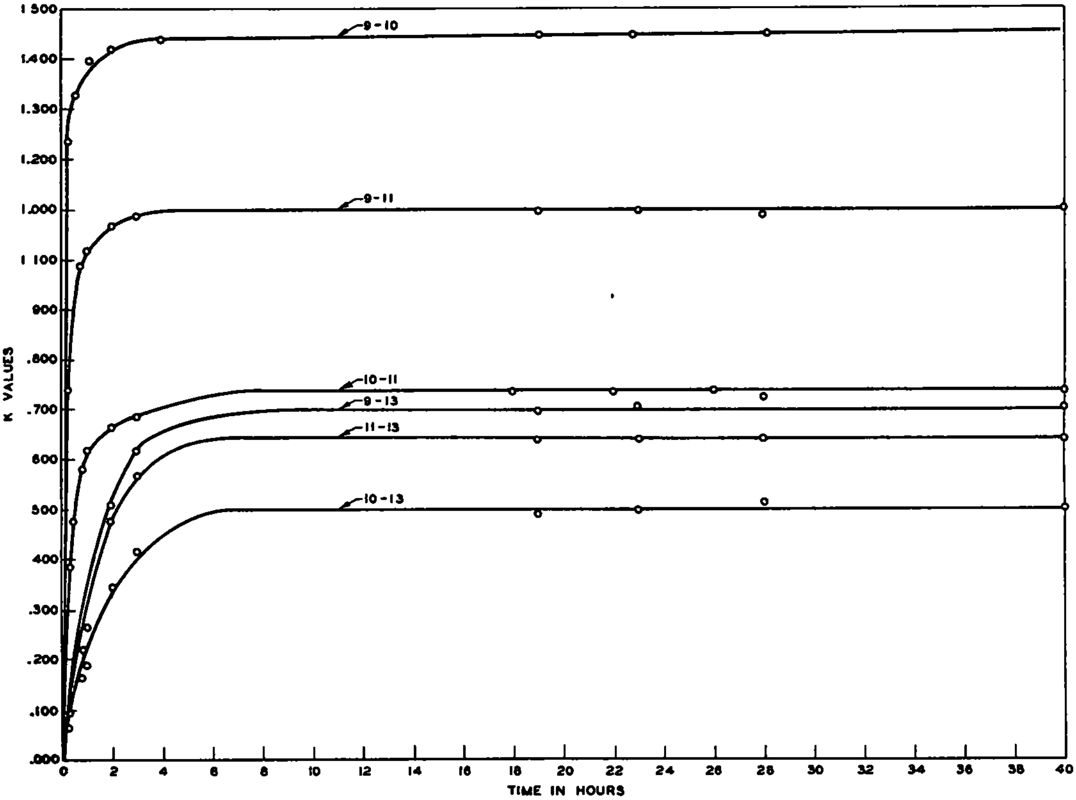

Figure 9. Swell of "hydrogen" soil at various moisture contents expressed as proportionality constant "K".

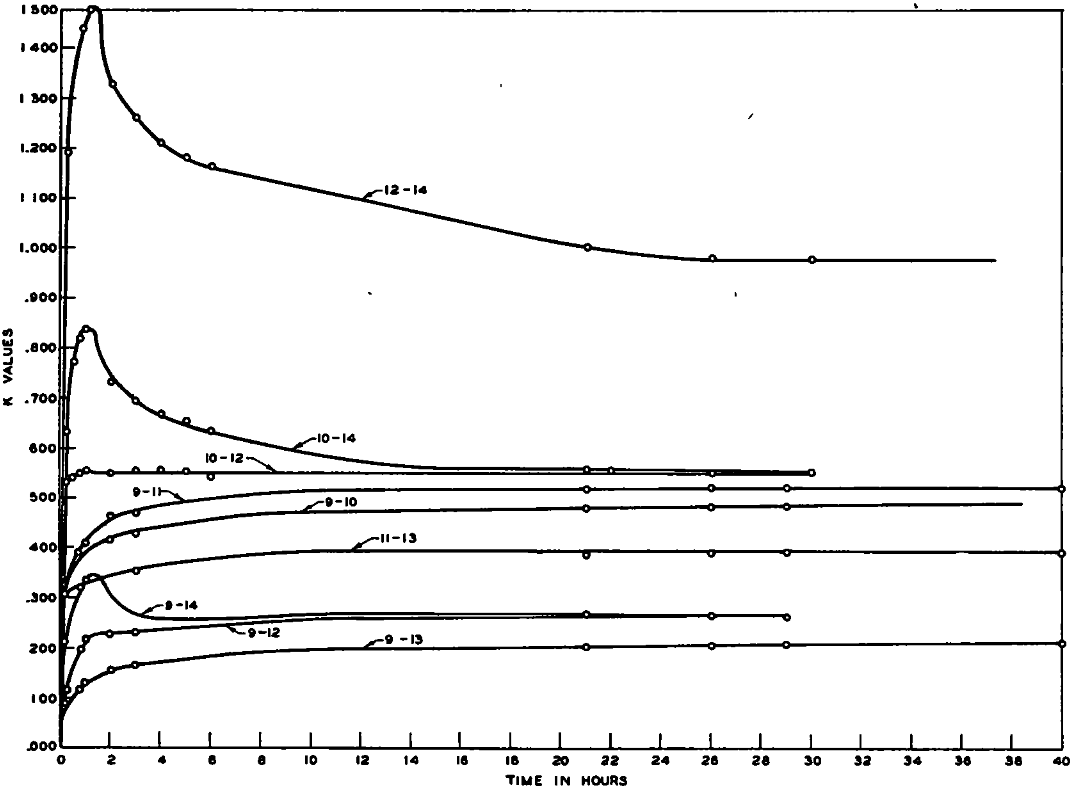

Figure 10. Swell of "aluminum" soil at various moisture contents expressed as proportionality constant "K".

is an optimum moisture content at which soils will possess minimum swell characteristics.

The "K" curves for the various homoionic soils have similarity in shape and character, but variations are attributed to the ion of the homoionic soils. For each of the homoionic soils the shape and character of the "K" curves are characteristic for all comparative moisture percents.

The "K" curves for all the homoionic soils indicate that after approximately 6 hours of swelling tests the relationship of swell of the samples at varying moisture content is constant.

CONCLUSIONS

The following conclusions have been reached at the present stage of this study:

1. An expression in exponential form can be employed to express the relationship of volume change of soil specimens compacted at the same dry density but at varying moisture contents.

2. Relationships of volume change, density, and moisture content are a function of not only the original dry density and moisture content but also of the time subsequent to compaction, type of clay mineral, and the controlling exchange ions.

3. By employing a series of controlled tests it is possible to determine for any given soil a combination of optimum dry density and

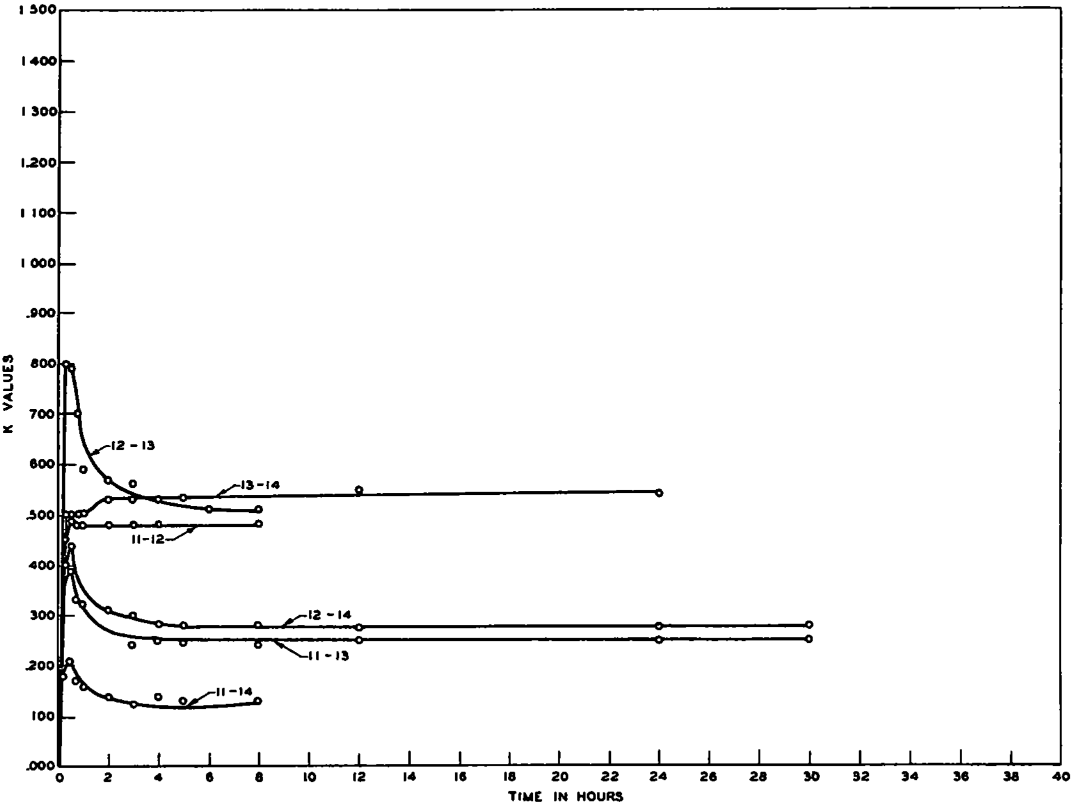

Figure 11. Swell of "calcium" soil at various moisture contents expressed as proportionality constant "K".

moisture content that will result in a minimum volume change subsequent to initial compaction.

4. The additions of organic matter in the form of lignin to the homoionic cohesive soil has a beneficial effect in reducing swell characteristics for soils having aluminum and ferric ions saturating the base exchange capacity of the clay and colloidal portion of the soil. The lignin, if applied in amounts not exceeding that necessary to provide steric hinderance to water dipoles, is effective as a moisture inhibitor, and thus reduces swell.

Further studies are being carried out to determine the following:

1. The relationship between volume changes of samples compacted at varying densities but at the same moisture content. Studies to date have indicated these volume changes may be expressed in the form of an exponential expression.

2. The swell characteristics of sodium and potassium homoionic soils as related to density and moisture content variations.

3. The effects of optimum amounts of organic additives in the form of lignin on the swell characteristics of homoionic soils at uniform density and varying moisture content, and at constant moisture content and varying densities.

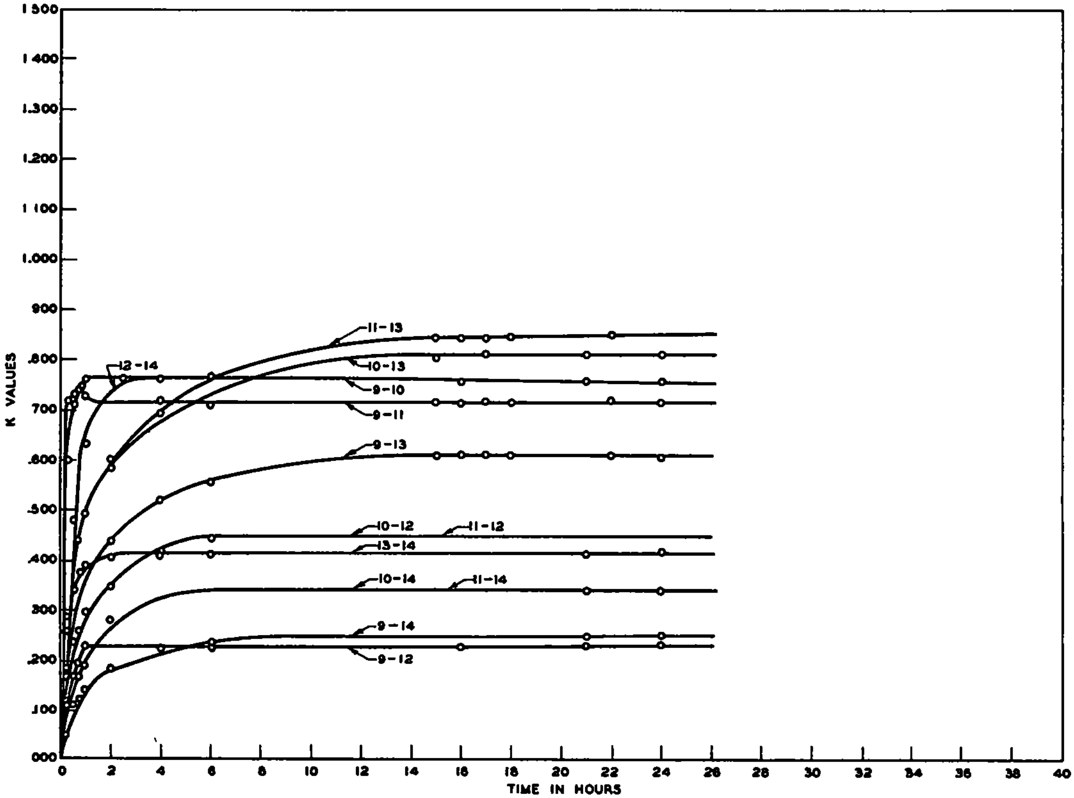

Figure 12. Swell of "ferric" soil at various moisture contents expressed as proportionality constant "K".

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