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# MORPHOLOGY AND PEDOLOGIC CLASSIFICATION OF SWELLING SOILS

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Swelling soils occur in nature in a predictable manner. The pedologist identifies, classifies, and characterizes these unique soils and delineates their occurrence on the landscape. The concept of Vertisols, for example, is that of a soil that is unstable because of a high content of expanding lattice clay. The morphology is marked by intersecting slickensides, parallelepiped structural aggregates, and horizons that are thin and poorly expressed near microhighs but that are thick and well expressed in microlows only a few feet (meters) away. Where not destroyed by man, these soils have gilgai relief. Soils having swelling potential but lacking the other features of Vertisols are classified in vertic subgroups of other soil orders. By definition, these soils have more than 35 percent clay within a designated control section and a coefficient of linear extensibility of 0.09 or more or a potential linear extensibility of 2.4 in. (6 cm) or more. Vertisols and soils in vertic subgroups of other orders have the common property of instability because of swelling. They have a high plasticity index and a high liquid limit. They are characterized by a high content of expanding lattice clays, particularly montmorillonite. The micromorphology of swelling soils reveals a fabric of oriented clay particles along short-range shear planes.

•SOIL classification, whether developed by soil engineers or soil scientists, has the primary objective of grouping soils that have similar properties. As a result, all soils in a group exhibit similar behavior. This permits us to solve many kinds of simple soil problems and also to guide the testing program if the difficulty and importance of the problem dictate further investigation. Soil classification guides scientists or engineers by making available the results of field experience. Like soils should have similar behavior patterns, and this allows us to transfer experiences from one soil area to another like soil area.

Many kinds of soil classification systems have been developed, some to aid in the solution of specific problems. For example, in flow problems a soil engineer uses classes of permeability. The U.S. Army Corps of Engineers uses a frost susceptibility classification in which, on the basis of particle size, soils are classed according to similar frost behavior. The U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers use the Unified Soil Classification System (USCS) for all engineering problems. The AASHTO system provides a ready grouping of soil material for solving problems dealing with highway construction. Most of these systems, however, are based on laboratory analysis.

Pedologists have developed a system of soil classification based on the natural soil unit, using, in addition to measurable properties, in situ morphological features to define class limits. This system (16), adopted for general use by the National Cooperative Soil Survey in 1965, is multicategorical and has various levels, ranging from 10 orders at the most general level to more than 10,000 units at the more precise series level. In this system, predictions are made for in situ soil behavior and are used to solve both engineering and agronomic problems.

In this system, classification is based on six categories: the order, the suborder, the great group, the subgroup, the family, and the series. The 10 soil orders, which represent the kind and relative strength of the natural soil forming process, are as follows:

1. Entisols are young mineral soils that do not have genetically related horizons;
2. Inceptisols are young mineral soils that have weakly developed soil horizons;
3. Aridisols are generally dry soils when not frozen or irrigated that have started to form definite soil horizons;
4. Alfisols have a clay-enriched B horizon that is high in base saturation;
5. Ultisols have a clay-enriched B horizon that is low in base saturation;
6. Mollosols have a dark-colored surface layer, are high in base saturation, and are grassland soils;
7. Vertisols have a high clay content and shrink and swell excessively;
8. Oxisols are strongly weathered soils of tropical regions;
9. Spodosols have subsurface horizon of accumulated organic matter usually with iron or aluminum; and
10. Histosols are organic soils.

Suborders reflect either the presence of or lack of waterlogging or soil differences produced through the effect of climate or vegetation. Great groups are based on the kinds and sequence of major soil horizons and other compositional or morphological features. Subgroups represent the central or typic segment of the group or have properties of the group and one or more properties of another group. Families are based on properties important to the growth of plants or to the behavior of soils used for engineering. The soil series is the lowest category and includes soils that have profiles almost alike and a limited range in soil properties so that the expected behavior is the same. Most soil maps are made at the series level.

Five moisture regimes are defined in terms of the groundwater level and the presence or absence of water held at a tension of less than 15 bars (1500 kPa) as follows:

1. In the aquic moisture regime, the soil is saturated and the groundwater is on or close to the surface for significant parts of the year;
2. In the aridic or torric moisture regime, the soil is dry in all parts more than half of the time that soil temperature is warm enough to grow plants;
3. In the udic moisture regime, the soil is not dry in any part for as long as 90 days (cumulative), and the water moves through the soil at some time in most years;
4. In the ustic moisture regime, the soil is dry part of the time, but moisture is present at a time when conditions are suitable for plant growth; and
5. In the xeric moisture regime, the soil is dry in all parts for at least 45 consecutive days during the warm season.

The mean annual temperature at a depth of 20 in. (50 cm) is used to define the soil temperature regimes as given in Table 1.

The objective of this paper is to relate that part of this soil taxonomy system that deals with properties and behavior of expansive soils to engineering experiences and uses. The two major groups of soils in the United States that exhibit instability in the form of a high shrink-swell potential are the Vertisols and those integrated with Vertisols by their assignment to vertic subgroups. There are at least 24 million acres (9.7 million $hm^2$) of Vertisols in Texas, Oklahoma, Mississippi, and Alabama (2) and smaller acreages in several other parts of the country.

## NATURE OF EXPANSIVE SOILS

There is much known about the nature and behavior of expansive soils as a result of elaborate tests in both the soil mechanics and soil science laboratories. Clay content (1, 6, 12, 15), organic matter content (3), cation-exchange capacity (6, 7), kind of adsorbed cations (1, 3), charge density (12), amount and kind of clay minerals (1, 12), and bulk density (5, 12) have been shown to affect soil swelling. However, there is a general lack of recognition concerning the in situ testing of expansive clay soils, and there is a need for predicting a soil's behavior in its natural state. Gromko (8) recognized this deficiency and concluded, on the basis of studies on the process of in situ

**Table 1. Soil temperature regimes.**

| Regime | Temperature (deg F) | Location |
|---|---|---|
| Pergelic | <32 | Permafrost areas |
| Cryic or frigid | >32 <47 | Northern latitudes, high elevations in mountains |
| Mesic | >47 <59 | Mid-latitudes |
| Thermic | >59 <72 | Southern latitudes |
| Hyperthermic | >72 | Subtropics |
| Isothermic or isomesic | <90 | Tropics |

Note: 1 F = 1.8 (C) + 32.

**Figure 1. Microlows contain water in area of gilgai microrelief common to Vertisols in Texas.**

**Figure 2. Microhighs occupy lighter colored areas in this area of gilgai microrelief.**

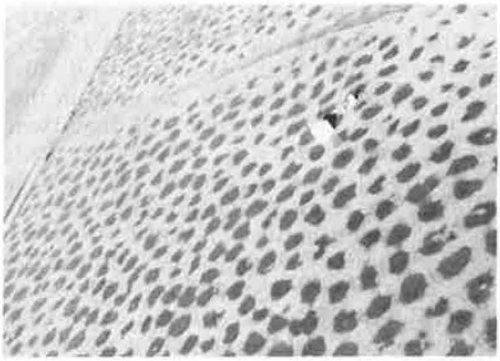
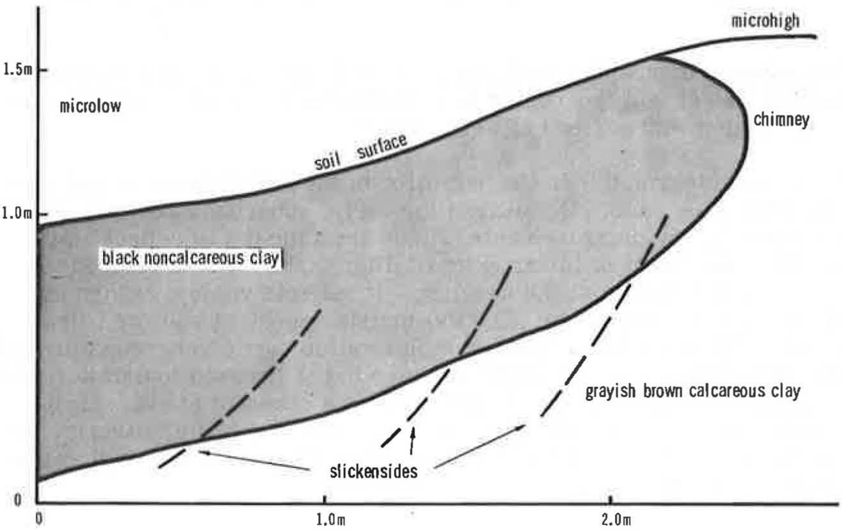


**Figure 3. Typical morphology of Houston Black soil series.**



heave, that predictions of behavior can be made by integrating several tests with climatic data. The tests he proposed are free swell index, Atterberg limits, colloid content, and one-dimensional consolidation tests. He modified these tests with climatic information. Soils having critical test values are most susceptible to expansion in those areas having lengthy drought periods followed by periods wet enough to saturate the soil.

Gromko's approach conforms well to that used by many pedologists. Actual movement in an undisturbed soil in Israel was measured by Yaalon and Kalmar (17). They measured, in a Vertisol with a clay content of more than 65 percent and dominated by montmorillonite, an annual amplitude of vertical soil movement at the soil surface of 2 in. (5 cm). The volume changes in an undisturbed soil are smaller than those measured in the laboratory. Only the incremental wetting during rainfall resulted in sufficient moisture absorption to cause volume change. The daily change during the dry season was minimal, 0.01 in. (0.3 mm), and was related to temperature changes. Yaalon and Kalmar (17) observed a rather constant rate of shrinkage during the main dry season. Swelling, however, is very marked and immediate after a deep soil wetting that occurs after the first rain. Hamilton (10) obtained maximal seasonal movements of 2.75 in. (7 cm) in undisturbed clayey soils in western Canada.

Soil moisture regimes, as used in the soil taxonomy (16), are more definitive than climatic data of the in situ soil condition. For example, all soils in an arid climate are not necessarily dry. They may be dry, moist, or saturated, depending on their position in the landscape. In any given landscape that has uniform climate, adjacent soils may have different moisture regimes. Soils in low areas may receive runoff waters in addition to rain that falls on them. However, there is a close relation between soil moisture regimes and climate. Soils with aridic or torric moisture regimes are normally in arid climates. These soils are dry in all parts more than half of the time. Soils with ustic soil moisture regimes normally occur in areas intermediate between the arid and humid zones and have moisture regimes intermediate between udic and aridic regimes. The xeric moisture regime is typified in Mediterranean climates where winters are moist and cool and summers are warm and dry. These soils have a wetting period followed by a marked dry period.

Soils that are notoriously expansive in situ have the following climatic characteristics:

1. Very high evaporation or evapotranspiration rates during some time of the year,
2. Sufficient rainfall to wet soil thoroughly to a depth of at least 30 in. (76 cm), and
3. Periods of dry weather and periods of wet weather.

This information has been integrated with the definition of the soil classes in soil taxonomy that deal with expansive soils. Definitions for vertic subgroups are based on coefficient of linear extensibility measurements (9) but are adjusted to reflect soil moisture regimes. The coefficient of linear extensibility (COLE) was developed to identify those soils with a high potential for swelling. It reflects volume change experienced by a soil sample between ⅓-bar (33-kPa) moisture content and oven-dry content. For example, a COLE value of 0.09 is definitive for vertic subgroups for soils in the eastern United States with udic and aquic regimes but is lowered to 0.05 for soils of the deserts with aridic regimes. Vertisols also reflect a climatic effect. Most of the the Vertisols have xeric, ustic, and aridic moisture regimes, but some in thermic or warmer soil temperature regimes [where mean annual soil temperature is higher than 72 F (22 C)] have a udic moisture regime.

## Morphology

Vertisols, by their definitions, have two distinct but interrelated features: the surface configuration and a unique pattern of soil horizons that underlie the surface features. Gilgai, the term used to identify the surface microrelief, is an Australian aboriginal word meaning small water hole (14) and is described as a microrelief consisting either of a succession of enclosed microbasins and microknolls in nearly level areas (Figures 1 and 2) or of microvalleys and parallel microridges that run up and down the hill in hilly areas. The height of the microridges commonly ranges from a few inches (centimeters) to about 3.3 ft (1 m). Paton (14) places those that are elongated in one direction in the linear class, and those that have no consistent preferred orientation in the nuram class. Nuram is an Australian aboriginal word meaning pockmarked. The microlows are polygonal and have microhighs surrounding them along the perimeter.

Recent fieldwork has revealed that this microrelief in the Vertisols is more complicated than a series of microhighs and microlows. The form has been characterized both in a vertical section and in a horizontal section or plan. Amplitude and wavelength are used to identify some of these features. Amplitude is determined by measuring the vertical distance between the crest of the high and the bottom of the low. Wavelength is determined by measuring the horizontal distance between the same points. Three basic types have been identified: (a) highs and lows equally developed, (b) highs of much greater extent than lows, and (c) lows of much greater extent than highs. The Houston Black series, typical of Vertisols of central Texas, is characterized by type a; that is, highs equal lows in development and have a wavelength of 6 ft (1.8 m) and an amplitude that averages 20 in. (50.8 cm).

The nature of the soil horizons reflects the features that occur in the natural soil surface. In other words, the soil profile underlying the microlow is markedly different from the profile under the microhigh. Typically, the microlow in the Houston Black series is characterized by a black to very dark gray colored $A_1$ horizon that extends to 24 in. (61 cm). This horizon changes horizontally to a much lighter color, almost grayish brown in the surface of the microhigh (Figure 3). The amplitude of the lower boundary of the dark colored A horizon ranges from about 20 in. (51 cm) to 40 in. (102 cm). The lower C horizon penetrates up into and through the overlying $A_1$ horizon to approach or reach the surface in the microhighs. These features have been referred to as chimneys through which the C horizon material surfaces. For this feature, Paton (14) has proposed the word mukara, which is an Australian aboriginal word meaning finger. Usually the soil surface is calcareous in the highs and noncalcareous in the lows.

The soil horizons in Houston Black soils, typical Vertisols, are not uniform and continuous horizontally throughout the unit of soil. The soil horizons are intermittent but recur at regular intervals that match the occurrence of the microhighs and microlows in the natural state. Vertisols are defined to have this recurrence in linear intervals of from 7 to 25 ft (2 to 7.6 m) (16).

This morphology is the result of the soil-forming processes that are unique in the Vertisols. The dominant soil-forming process is churning or self-swallowing, which tends to destroy the differential textural distinctions that are due to the soil-forming processes common in many other soils. The fine-textured soils shrink and form cracks that may extend downward from the soil surface to depths of 40 in. (1 m) or more during the dry period. During the first rains after the dry period, soil surface material is washed into the cracks. As the rain continues, the soil clays expand demanding more space. Some of the forces are lateral, but lateral movement is restricted by surrounding soil. The eventual thrust is upward along the pressure release chimneys; this gives rise to the microhighs. Soil material from the lower horizons is heaved up through the chimneys. Most water enters the soil through the lows, and thus these zones are normally more acid than the neighboring highs. Soil wetting is largely from the cracks migrating inward in response to capillary tension.

We have made numerous field observations that indicate that the extensive slickensides, whose occurrences have been recorded by most observers of the morphology of expansive soils, are oriented on inclined planes directed toward the top of the microhighs (Figure 4). The slickensides represent the surface of large blocks of soil that shift their relative position en masse in response to stresses that result from soil swelling.

Soils in vertic subgroups are similar to Vertisols in their unstable nature but lack the characteristic gilgai relief and accompanying subsurface characteristics. They do not have significant cracking in most years. For this reason, within subdivisions of

Figure 4. Vertisol profile showing wide cracking and common slickensides in a Vertisol; microhigh in upper right.

Table 2. Representative soils in fine and very fine families of typic and vertic subgroups.

| Pedologic Classification | Mineralogy | Unified Classification |
|---|---|---|
| Vertisols | | |
| Typic | Montmorillonitic | CH |
| Argiustolls | | |
| Typic | Mixed | CL |
| Vertic | Montmorillonitic | CH |
| Paleustalfs | | |
| Typic | Mixed | CL |
| Vertic | Montmorillonitic | CH |
| Paleudults | | |
| Typic | Kaolinitic | CL |
| Vertic | Mixed | CH |

great groups, soils are integrated with the Vertisols by their assignment to vertic subgroups (16). These soils, in higher categories of the system, are assigned to classes that most nearly reflect their dominant properties.

## Properties

Soils are grouped into pedologic units on the basis of similar composition and morphologies. This pedologic concept is based on the premise that similar soil parent materials, if subjected to identical environmental conditions of climate, biological activity, topography, and time, developed identical soils.

Vertisols and soils in the vertic subgroups, the two distinct soil taxa that are dominant among those soils with swelling characteristics, have the following common properties that can be learned from the definitions of the taxa:

1. Clay mineralogy is dominated by montmorillonite or other 2:1 lattice active clays,
2. COLE values are high (16),
3. Clay content is 30 percent or more in the upper part of the soil, and
4. Unified classification is dominantly group CH.

Mineralogical data of 203 soil series from the southern United States indicate that those in vertic subgroups are evenly divided between montmorillonite and mixed mineralogy (4). Limited data indicate that those with mixed mineralogy have a relatively high proportion of montmorillonite compared to other clay minerals. None of these soils are dominated by 1:1 lattice clays. All but a few series of the Vertisols have montmorillonite as the dominant clay mineral.

COLE values for Vertisols and soils in vertic subgroups range from 0.05 in arid regions to more than 0.09 in humid regions. Potential linear extensibility can be determined by multiplying the COLE times the thickness of the layer involved. Most Vertisols and soils in vertic subgroups have a potential linear extensibility of 2.4 in. (6 cm) or more.

In soils with similar clay mineralogy, clay content explains a high proportion of the variation in soil swelling. Studies of soils from the southern United States show that the clay content in representative pedons of soils in vertic subgroups exceeds 40 percent. Most of these soils qualify for group CH in the USCS. Most of the Vertisols have a clay content that exceeds 45 percent and are classified in CH. These soils have a high plasticity index and a high liquid limit. Several soils of the southern United States are given in Table 2.

Other soil properties measured by the pedologists also are highly correlated with swelling behavior. Relatively high values in such tests as moisture held at 15-bar (1500-kPa) tension and cation exchange capacity characterize the Vertisols and soils in vertic subgroups.

The micromorphology of these soils also is unique. McCormack and Wilding (13) have identified a lattisepic fabric from thin section studies of soils high in clay. They postulated that this fabric has formed as a result of stress and short-range movements of finite soil masses in response to swelling in situ. As a result of these movements, the fabric is oriented along microshear planes. Undisturbed samples of argillic and other soil horizons can be characterized by studies of the micromorphology that identify the amount and kind of soil movement that the natural soil has experienced.

## CONCLUSIONS

Expansive soils are most troublesome when used as base for roads or buildings because of their behavior patterns. Jones and Holtz (11) report that heaving, cracking, and breakup of pavements, building foundations, and channel and reservoir linings in the United States cost $2 to 3 billion annually.

Expansive soils received much consideration in the development of soil taxonomy. Soil scientists are studying these soils, both in the laboratory and in the field. In addition, selected soils are being characterized by both pedologic and engineering tests. As a result, these unique soils are recognized in situ by their characteristic morphologies, and behavior predictions are based on the engineering tests that also characterize the discrete soil units. Studies have shown that a soil series or a group of soil series has consistent engineering properties, especially those related to kind and amount of clay and particle-size distribution, throughout the area of their occurrence. The science of pedology provides a comprehensive procedure for study of soils in situ. The soil is identified in place, and the composition of the material and its variation with depth are noted. Engineering behavior patterns of soils in situ may be quite different from what laboratory tests indicate they will be. Laboratory tests, such as COLE, Atterberg limits, PVC index, consolidation tests, and free swell index, however, can be used to characterize soil units recognized in soil taxonomy. Predicting soil behavior in situ requires an integration of soil test data with those soil features that are related to the environment of the soil. For example, an expansive soil not only has a sufficient quantity of swelling clays but also must experience wetting and drying periods. Soil moisture regimes and soil temperature regimes, which characterize the moisture and temperature status of soils throughout the year, are used to characterize this environment. Soils in place have a way of recording this experience in their morphologies. Deep cracks, gilgai relief, surface A soil horizons interrupted by protruding subsurface C soil horizons, and a lattisepic microfabric in the subsoil reflect the natural soil-forming processes active in expansive soils. These effects of the environmental factors are then automatically included within the classification system.

The soil survey treats the soil classification unit as a three-dimensional body. This is represented on the soil map by its aerial boundaries. The soil unit becomes a landscape unit. Prediction of behavior or performance made for the soil taxonomic unit applies to the delineated landscape unit. Thus the prediction of soil heave made for a soil series in soil taxonomy applies reasonably well to the identified mapping unit in the soil survey. The degree of reliability for predicting this behavior pattern is quite high. It is estimated that, if one digs a test pit anywhere in the delineated area, there is an 85 to 95 percent chance of revealing a soil that has the engineering properties as given for that soil series.

Stabilization techniques of swelling soils also can be made more effective by applying the knowledge of the soil environment implied by the pedologic soil unit. Prewetting and other water control measures can be devised to fit the soil drainage and the movement characteristics of the soil water. Chemical stabilization can be more effective if devised to match the soil chemistry of the pedologic unit. In addition, structures can be designed to reflect both the soil and the site conditions. This information and other data, such as an estimate of vertical heave, depth to water table, wet-dry cycles, and water contents of different soil layers, can be learned from the soil survey.

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# CHARACTERISTICS OF EXPANSIVE CLAY ROUGHNESS OF PAVEMENTS

Robert L. Lytton, Ronald L. Boggess, and J. W. Spotts, Texas A&M University

The patterns of pavement roughness caused by expansive clay appear to be predictable from the mineralogical and pedologic properties of the clay deposit. Surveying measurements made in two gilgai fields with similar mineralogy reveal a high degree of statistical similarity in wavelengths and amplitudes of the surface waves. Field measurements of water content, suction, density, and horizontal and vertical displacements of the soil with depth throughout one weather cycle beneath the mounds and the depressions have revealed the mechanism of differential heaving of these expansive clay deposits. Pavement roughness was measured on pavements adjacent to the gilgai fields and on other pavement sections by the GM profilometer. Digital magnetic tapes of the profilometer data are analyzed by a series of digital filters especially designed for expansive clay wavelengths. Statistical data on the wavelengths and amplitudes of these typical expansive clay roughness patterns are determined and compared with those measured in the gilgai fields where these patterns have not been tampered with. The riding characteristics of these pavements as measured by the Mays meter are analyzed with a computerized model of a vehicle that is programmed to accept GM profilometer data. Dynamic load factors for the actual pavements are determined. Equations are presented that give Mays meter readings and dynamic load factors for any combination of expansive clay wavelengths and amplitudes. The effect of the dynamic load on reducing the service life of the pavement is discussed, and methods of predicting level-up and overlay quantities from the characteristics of expansive clay roughness are given.

•THE major effects of expansive clay roughness on pavements are a loss of riding comfort and road-holding ability, a reduction in pavement service life, and the costliness of the rehabilitation that is required. In those areas where expansive clays exist, they take up a substantial portion of the annual maintenance budget. Identification of expansive clays in the field is simple compared with determination of the most economic strategy to follow in preserving acceptable riding quality. This complexity is based on the fact that there have been no consistent measures of roughness that can be used to estimate the quantities of reworking, releveling, and overlaying that will be required to restore riding quality. This paper presents a hypothesis about expansive clay roughness that is based on the natural morphology of the soil surface and the cracking patterns that expansive clays tend to form when they are exposed to wetting and drying influences. A study of natural soil shapes reveals pavement roughness patterns that may be expected to develop over a period of time when a pavement has been built on expansive clay. Roughness is based on crack spacing, which, in turn, is based on the mineralogy of the soil. The natural roughness patterns are called gilgai, and these are compared with the roughness patterns measured on adjacent pavements. Amplitude versus wavelength and wavelength distributions for both pavement and natural soils are compared for two different geological formations within Texas. Although they are separated by 50 miles (80.5 km) in distance and hundreds of thousands of years in deposition history, their mineralogy is almost the same, and, consequently, it is expected that their roughness patterns are statistically similar. From the pavement roughness data, a relation among serviceability, amplitude, and wavelength is derived and is used to analyze riding quality with a computer-simulated Mays meter response as the indicator of riding quality. The expansive clay roughness of the pavement will also cause dynamic loads to be applied, thus conceivably shortening the pavement service life. The size and importance of dynamic loads caused by expansive clay roughness on service life and methods of predicting future rehabilitation requirements are discussed. The potential usefulness of this kind of characterization of expansive clay roughness is in permitting the designer to calculate the costs of future rehabilitation and compare the present value of those costs with the costs of preconstruction treatments that may be used to reduce the development of future roughness.

## GILGAI—NATURAL EXPANSIVE CLAY ROUGHNESS

Gilgai is an Australian aboriginal term for water hole, which refers to the water that collects in the depressions formed in these natural wave-like surface patterns. Figure 1 shows a hypothesis about the seven stages of development of a normal gilgai, which are discussed below.

1. Deposition. The clay is laid down in roughly horizontal layers.
2. Drying. Primarily horizontal and vertical shrinkage cracks develop in the upper soil layers in contact with the atmosphere.
3. Major shrinkage cracks. The spacing is largely determined by the type of mineral in the clay. The more active clay minerals will crack more frequently. These cracks become the determining factor in the future development of the gilgai.
4. Heaving around major cracks. Since gilgai are rarely found outside of a climatic region where around 6 to 60 in. (15.2 to 152 cm) of rain fall each year, it is apparent that their development requires an abundant supply of moisture alternating with drying influences. When the soil is cracked and water becomes available again, the water runs to the roots of each crack and the soil there swells upward and causes conical shear failures to occur around each crack. The swelling soil at the roots of the cracks begins to extrude upward within the crack. Mounds that are characteristic of the gilgai form at this stage.
5. Infiltration and leaching. Rainwater and surface runoff are trapped in the depressions between the mounds. Soluble salts are carried with the water as it percolates downward, generally following the cracks opened by shrinkage (stages 2 and 3) and heaving (stage 4). As a result of this leaching, it is not uncommon to find the soil pH tending toward neutral in the depression. There is a possibility that an alteration of the soil type may take place beneath the depressions as fresh water replaces the salt or brackish water that was present when the soil was deposited. The soil beneath the depression is darker than the soil under the mound possibly because of the replacement of saltwater and possibly because of the infiltration of organic materials.
6. Pressure zones and evaporation. The gilgai now begins to take on its mature form as expansion pressure zones begin to form at the root of the shear failure cracks on the boundary between the mound and depression soils. These boundary cracks are formed by the expansion and contraction of the soils in the depressions. Water from the depression feeds in beneath the mounds and evaporates there leaving behind calcium carbonate concretions that are remnants of the salts that have been leached and redeposited. These nodules are generally found only under the mounds. The soils beneath the depressions remain wet throughout the year.
7. Maturity. The mature gilgai profile is shown in Figure 1, stage 7. Its surface features are the mound and the depression. Below the surface, there is the saturated zone beneath the depression, the pressure zone, the intrusion zone at the root of the major genetic cracks, and the evaporation zone where carbonate concretions are found. Seasonal moisture variations cause an increase in the relative elevation of the mound and the depression, both of which rise during wet weather and subside in the dry seasons. There is a greater change of moisture content beneath the mound than in any other location.

**Figure 1. Stages of genesis of normal gilgai.**

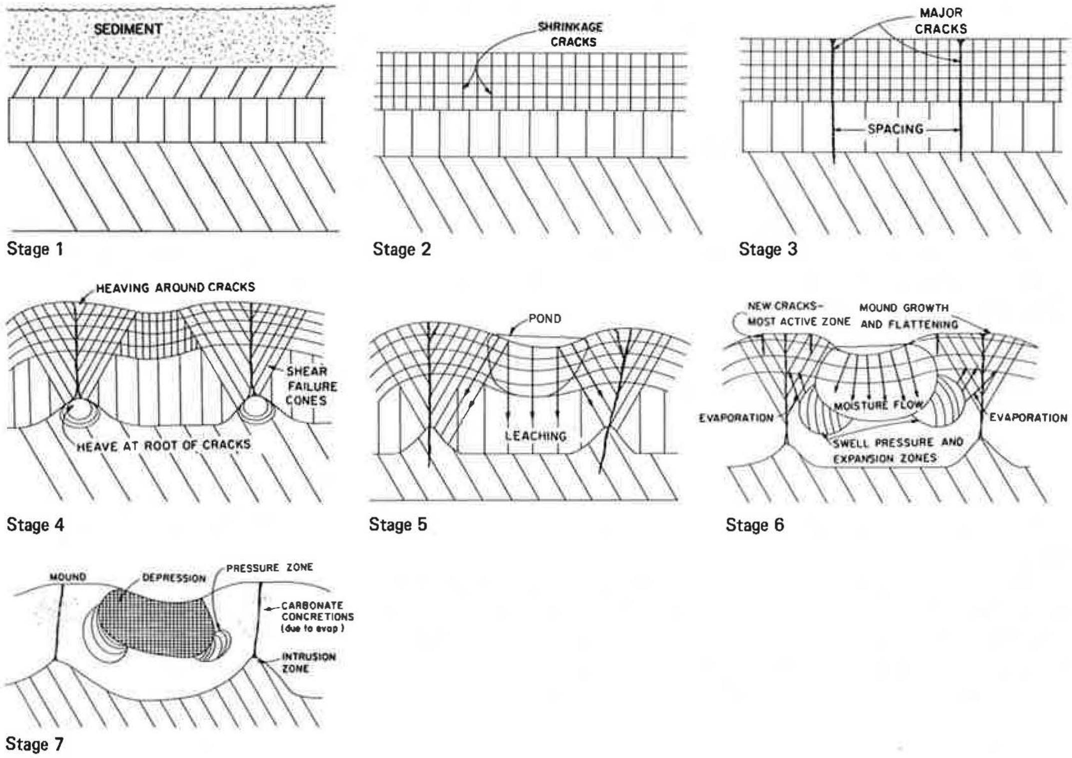



**Figure 2. Variations in gilgai soil profile.**

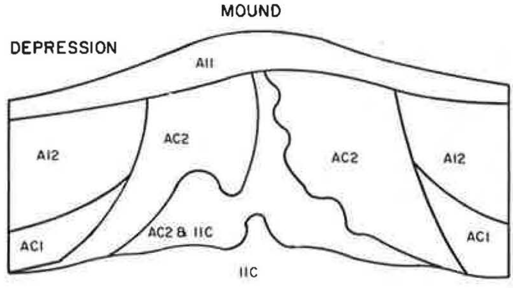



**Table 1. Laboratory characterization of gilgai samples.**

| Horizon | Sand:Silt:Clay (percent) | USDA Texture | pH | Soil Bulk Density ($g/cm^3$) | At-Field Volumetric Water Content (percent) | Liquid Limit (percent) | Plastic Limit (percent) | Electrical Conductivity (m℧/cm) |
|---|---|---|---|---|---|---|---|---|
| A11[a] | | | | | | | | 0.351 |
| A11D[a] | | | | | | | | 0.234 |
| A12 | 11:32:57 | Clay | 7.3 | 1.47 | 44.18 | 57 | 27 | |
| AC1 | 12:36:52 | Clay | 7.8 | 1.51 | 37.46 | 60 | 27 | 0.681 |
| AC2 | 15:34:51 | Clay | 7.6 | 1.52 | 37.37 | 52 | 28 | |
| AC2 and IIC | 10:37:53 | Clay | 7.6 | 1.59 | 38.90 | 67 | 27 | |
| IIC | 5:48:47 | Silty clay | 7.8 | 1.69 | 39.17 | 62 | 28 | |

Note: 1 m℧/cm = 1 mS/cm.

[a]Mount depression.

The most important element in the genesis of a gilgai is the formation of major cracks. The spacing of the cracks depends on the mineralogy, and the cracking patterns that form are responsible for all subsequent behavior of the gilgaied profile. Even though the soil may be reworked to a depth of several feet (meters) during construction, the cracking patterns remain in the natural soil below the reworked zone and, as such, constitute a source for the reestablishment of the roughness patterns. Gilgai fields that have been plowed have been found to reestablish the same patterns in the same locations as before within 2 to 11 years after they were disturbed (1). Figure 2 shows the vertical profile of a gilgai that was observed in a field study site in Snook, Texas. Various soil types were identified within the section primarily by color although their natural water content and pH values were major distinguishing characteristics. Table 1 gives several measured characteristics of each of these soils. There are four major points in Table 1:

1. Bulk density. The bulk density of the A2 soil is lowest of all and is closely followed by the AC1 and the AC2 soils. All of these are in the depression area indicating that they have swelled more than the AC2 and IIC soils. This is in accordance with the field observation that the soils beneath the depression remain wet while the soils beneath the mounds fluctuate in moisture content with the seasons.
2. pH. The pH of the A12 soil is lower than all others and close to neutral (pH = 7.0). This indicates the effect of a long period of leaching that dissolves calcium carbonate salts.
3. Electrical conductivity. The electrical conductivity of the surface soils, A11, is different between the mound and the depression. The higher conductivity over the mound indicates the presence of a higher salt concentration in that soil. This indicates that it may be a zone of accumulation of the dissolved salts and leaching water.
4. Atterberg limits. Although there are some distinctions to be made between the soil types, the Atterberg limits for all the soil types are roughly the same. The sole exception is the AC2 soil that has a lower liquid limit, presumably because of its higher sand content. The point to be observed here is that Atterberg limits could not be used to identify the presence of a gilgaied profile.

## Seasonal Movements

Gilgai will usually develop in a field that has low slopes and is poorly drained. This was the case with the field near Snook, Texas. Despite their unusually wet condition, the gilgai still experience shrinking and swelling as water enters and leaves the soil. Spotts (2) instrumented several of these gilgai and measured the vertical strain with depth, horizontal strain at the surface, soil suction (with a tensiometer), and volumetric water content with a nuclear, moisture depth probe. Various other climatic and hygrometric data were recorded and presented in Spotts' dissertation. Measurements were begun when the soil was at its driest during August 1973 and continued on until December of that year by which time the soil had become thoroughly wet. As shown in Figure 3, the major vertical strain was measured beneath the mound. Figure 4a shows the initial suction contours as measured by tensiometer, and Figure 4b shows the final suction contours. The largest change of suction took place beneath the mound. Changes in volumetric water content showed roughly the same patterns. One major finding of this field study was that water moves rapidly through the soil even in such a wet and swollen condition. This is a strong indication that the cracking fabric of the clay plays a major role in transporting moisture within the clay mass. These data call for a careful review of our concepts of field mass permeability.

## Characteristic Shapes of Gilgai

As shown in Figure 5, the surface profile of a gilgai in Snook closely resembles that of a sine wave. In fact, regression analysis of the surface elevations against a sine wave

Figure 3. Maximum vertical strain in Snook gilgai.

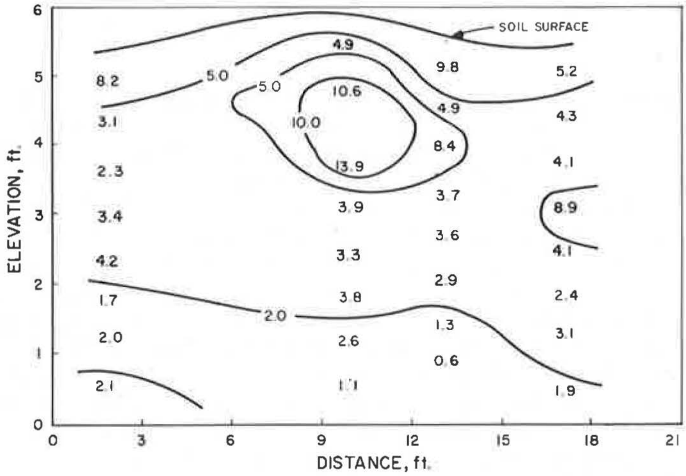



Figure 4. Contours of (a) initial suction and (b) final suction.

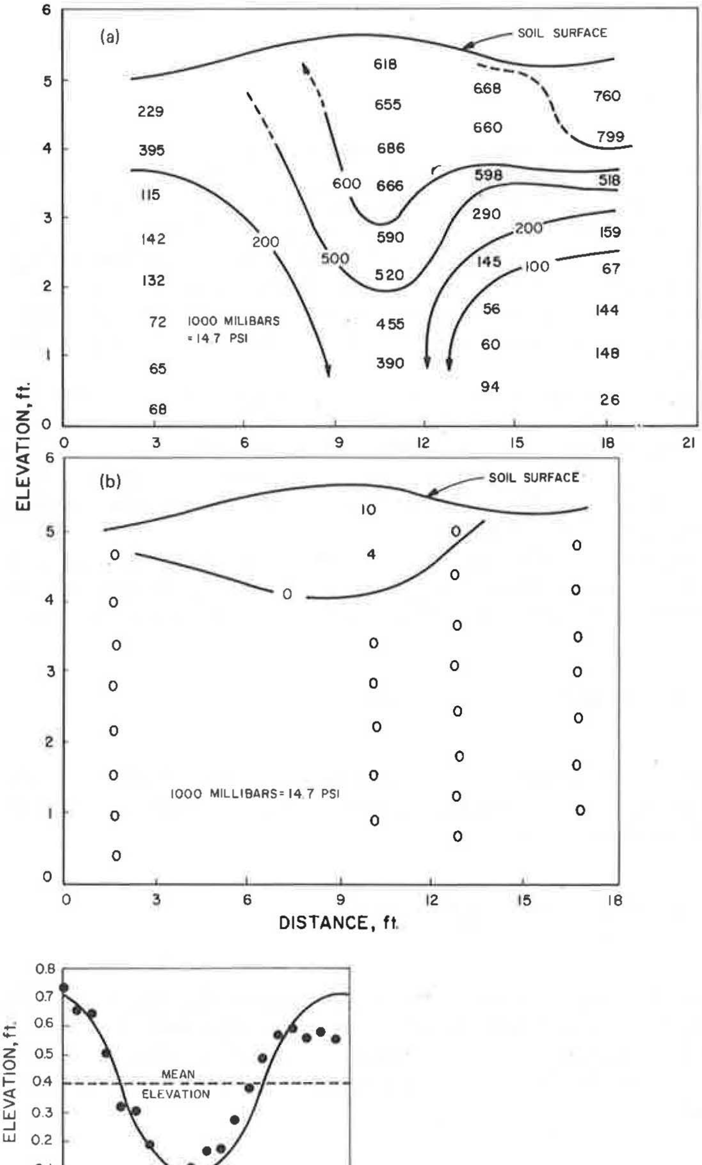
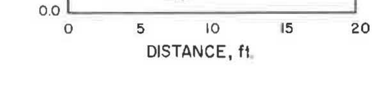



Figure 5. Vertical profile of Snook gilgai.



fitted through the data, showed correlation coefficients $R^2$ between 0.86 and 0.98. Figure 6 is an aerial view of the field at Snook, showing the overall gilgai pattern. The white building in the photograph is the shed housing the instrumentation used by Spotts. Pedologically, the Snook field is in the Burleson clay and the Thrall, Texas, field is in the Houston Black clay. Geologically, the Snook field is in the Brazos River floodplain and is a recent alluvium. The Thrall field is in the Taylor formation. Both fields have about the same mixture of minerals (>40 percent montmorillonite and <10 percent kaolinite); the only distinction between them appears to be in the percentage of clay and the pH. The Snook field has between 40 and 66 percent clay, and the Thrall field has between 55 and 62 percent. The pH is between 7.0 and 8.1 at Snook, and it is between 7.7 and 8.0 at Thrall. There is slightly more calcium carbonate in the Thrall soil. Based on the minor variations of soil properties in these two formations it was expected that the distributions of wavelengths and amplitudes that could be measured in these fields would be very nearly identical. This was shown to be the case. Two kinds of surveys were run to determine these distributions: a field survey and a grid survey for use in a Fourier analysis.

## Field Survey

Radial lines were laid out from the leveling instruments set at random in each field. A steel tape was used to measure distances to the tops of the mounds along each radial line. Relative elevations of each of these were recorded. A wavelength was taken as the distance between two successive high points. An amplitude was taken as one-half of the difference between the low point and the average of the high points. A total of 182 points were collected in the Snook field along rays that varied in length from 50 to 650 ft (15.2 to 198 m). A total of 83 points were collected at the site near Thrall. Table 2 gives the statistics of the amplitude and wavelength frequency distributions for Snook and Thrall.

Analysis of variance of the two distributions of wavelengths and amplitudes shows that neither their means nor their standard deviations are significantly different at the 95 percent confidence level. Thus, whatever differences may exist between the Snook and Thrall soils, their effect does not make much practical difference in the gilgai wavelengths or amplitudes. The wavelength frequency distributions are shown in Figures 7a for Snook and 7b for Thrall.

Because of such wide coefficients of variation for both wavelength and amplitude, any simple relation between amplitude and wavelength would be expected to have a low correlation coefficient. This was the case for the Snook gilgai,

$$a = 0.031\,\lambda^{0.58} \quad (R^2 = 0.10) \tag{1}$$

where a = amplitude and $\lambda$ = wavelength in feet (meters). The Thrall gilgai had a somewhat better correlation coefficient of 0.46, and its equation relating amplitude to wavelength is as follows:

$$a = 0.028\,\lambda^{0.77} \quad (R^2 = 0.46) \tag{2}$$

Despite the fact that there were no statistically significant differences between the amplitude and wavelength distributions, the exponents on each of these two equations differ by 0.19. This is an indication of how widely scattered the data are. The coefficient of variation of the Snook data is around 60 percent, and that of the Thrall data is about 40 percent.

Figure 6. Aerial view of Snook gilgai field.

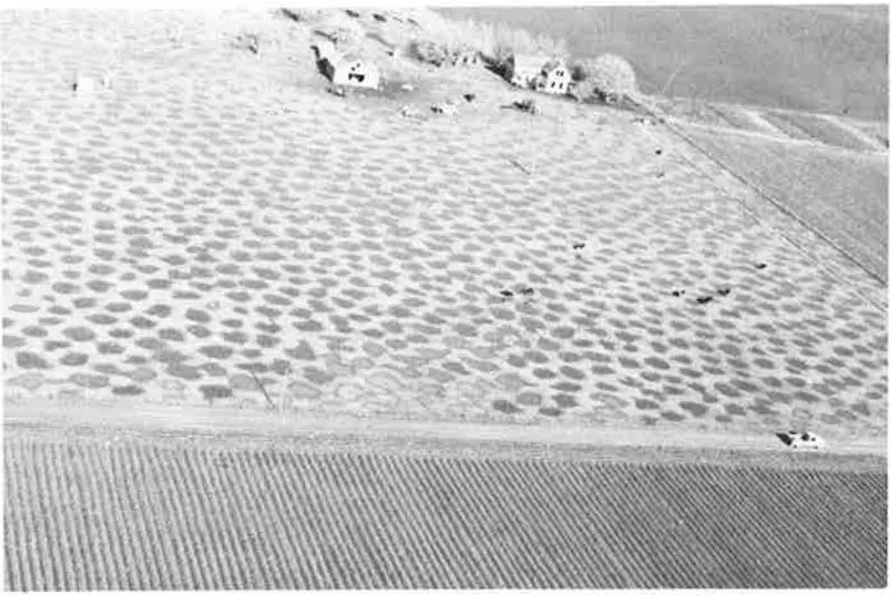
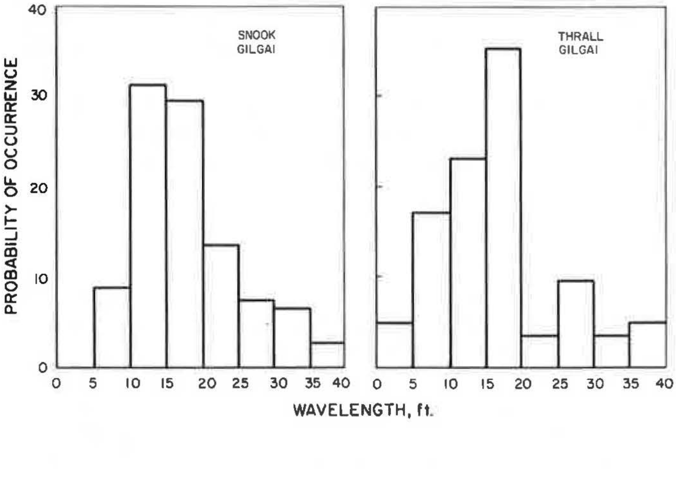


Table 2. Gilgai wave characteristics.

| | Snook Gilgai | | Thrall Gilgai | |
|---|---|---|---|---|
| Item | Wavelength | Amplitude | Wavelength | Amplitude |
| Mean, ft | 19.0 | 0.200 | 17.7 | 0.272 |
| Standard deviation, ft | 7.63 | 0.099 | 8.37 | 0.138 |
| Skewness | -0.66 | -0.71 | -0.56 | -0.30 |
| Coefficient of variation, percent | 40 | 50 | 47 | 51 |

Note: 1 ft = 0.3 m.

Figure 7. Wavelength probability density function.



Grid Survey

At Snook, elevations were measured every 5 ft (1.52 m) within a 100 by 100-ft (30.5 by 30.5-m) square grid, and each of the rows and columns of the grid was analyzed separately to determine its characteristic roughness patterns. The vertical profile of each line was assumed to be made up of sums of cosine waves of the following form:

$$u(x) = \sum_{i=1}^{n} a_i \cos \frac{i\pi x}{L} \qquad (3)$$

where u(x) is an elevation measured relative to the average elevation of the section. Solution of simultaneous equations resulting from equation 3 gave amplitudes that corresponded with the various specific wavelengths. Regression analysis of the mean amplitudes for each given wavelength resulted in the following equation:

$$a = 0.030\ \lambda^{0.20} \quad (R^2 = 0.32) \qquad (4)$$

The exponent on equation 4 is 0.20, which is a factor of 2.9 lower than the exponent, for the same field, based on directly measured data. A certain amount of this discrepancy can be explained by the scatter of the data. The rest of the difference may be sought in the different approaches taken by the field survey and the grid survey. The field survey assumed each gilgai shape to be sinusoidal but not periodic. Thus, each distance peak-to-peak was taken at the single wavelength with a single amplitude. The Fourier analysis of the grid data assumed each shape between peaks to be made up of the sum of several sine waves of varying wavelengths and amplitudes. Thus, what is listed as an amplitude in the field survey is the sum of several amplitudes in the Fourier analysis. Thus, it is reasonable to expect that the exponent relating amplitudes and wavelengths in a Fourier analysis will be smaller than that taken from survey data; in this case, the factor relating the two exponents is 2.9.

Mineralogy-Wavelength Relations

The proof that mineralogy determines the spacing of the cracks and thus the wavelengths of expansive clay roughness consists of two parts. First it must be shown that fields in different geologic formations but with similar mineralogical mixtures have similar amplitude and wavelength distributions. Secondly, it must be shown that fields with different mixtures of minerals have significantly different amplitude and wavelength distributions. The first part of this proof has been verified for the Snook and Thrall fields. Further work on the second part of this proof is presently under way.

## EXPANSIVE CLAY ROUGHNESS OF PAVEMENTS

The GM profilometer was used to measure the profile of pavements in the vicinity of the Snook and Thrall fields. One other set of profilometer runs was made on the Old San Antonio Road, a rough pavement that was built on the Burleson clay, about 20 miles (32.2 km) from Snook. This pavement section was chosen because of its similarity in mineralogy to both the Snook and the Thrall fields and because it was desirable to obtain data on low-serviceability-index pavements. There are two ways of analyzing the profilometer data: (a) by a fast Fourier transform (3, 4) of both the right and left wheel paths (5, 6) and (b) by profile filters. The filters were designed according to a procedure developed by Gimlin, Cavin and Budge (7) for the bands of wavelengths given in Table 3.

The profile data are in digital form, sampled every 2.03 in. (5.15 cm) along each wheel path. The filters are digital filters that are designed to pass only those amplitudes of waves within the wavelength band specified. The filter consists of a set of coefficients, in this case, 172 of them. So that the ordinate of the filtered data at station i can be obtained, the ordinate of the unfiltered data at station i-j is multiplied by the coefficient $c_j$ and added together with 172 other such terms as shown in the following equation:

$$\alpha_i = \sum_{j=1}^{172} c_j a_{i-j} \tag{5}$$

where $\alpha_i$ is the ordinate of the filtered data, and $a_{i-j}$ is the ordinate of the unfiltered data at station i-j.

Figure 8 shows the results of the application of these filters to pavement sections at Snook, Thrall, and the Old San Antonio Road. It is significant that these sections, all on expansive clay subgrades, show higher amplitudes for wavelengths around 13 and 30 ft (3.9 and 9.1 m), the same range of dominant wavelengths that were indicated by the field surveys in Snook and Thrall. Figure 8 also shows the distinctive growth of amplitude as the pavement gets rougher and the serviceability index decreases. A total of 14 pavement sections were measured with serviceability indexes (SI ranging between 1.7 and 4.5). A best fit polynomial that relates SI to wavelength and amplitude was as follows:

$$\begin{aligned} SI = {} & 5.00 + 0.1774\lambda - a(126.4 - 0.1665\lambda^2) \\ & + a^2(1{,}688.4 - 21.99\lambda) \quad (R^2 = 0.92) \end{aligned} \tag{6}$$

The standard error on this equation is 0.22 serviceability index units. Amplitudes are in inches (centimeters), and wavelengths are in feet (meters).

A special filter was designed to scan through the profilometer data, find the high points of the bumps, and measure the distance between the high points. This filter copies the effect of a field survey such as conducted in the gilgai fields at Snook and Thrall and was intended to determine whether the frequency distribution of wavelengths on the pavements was the same as those in the adjacent fields. Results of the filter runs at the Old San Antonio Road, Snook, and Thrall are shown in Figure 9. These distributions are similar to those determined by the field survey, indicating that the same expansive clay roughness patterns are developing beneath the pavement. Especially noticeable is the dominance of peak-to-peak distances of around 15 to 30 ft (4.6 to 9.1 m). These distances have emerged in every analysis of pavement or field as most characteristic of these expansive clay deposits.

## RIDING QUALITY OVER EXPANSIVE CLAYS

There are various mechanical means of measuring riding quality, but the Mays meter was selected for this study since its measurements have been shown to correlate well with panel ratings of serviceability index (3). Its ruggedness of construction and its ease of use recommend it for rapid surveys of roughness of pavement sections. The response of the Mays meter to excitation from roadway roughness varies with the frequency of the vibration. Computer program DYMOL (9), which has been modified for use on IBM computers, was used to obtain frequency response of the Mays meter. DYMOL solves the equations of motion of a vehicle mass and the four tire-axle masses.

Table 3. Digital filter characteristics.

| Filter No. | Frequency Range[a] (cycles/ft) | Wavelength Band[a] (ft/cycle) | Median Wavelength (ft) |
|---|---|---|---|
| A | 0.20 to 0.30 | 3.3 to 5.0 | 4.15 |
| B | 0.219 to 0.164 | 6.1 to 7.75 | 6.80 |
| C | 0.069 to 0.0808 | 12.3 to 14.4 | 13.35 |
| D | 0.054 to 0.0645 | 15.5 to 18.5 | 17.0 |
| E | 0.0441 to 0.0505 | 19.8 to 22.6 | 21.2 |
| F | 0.0315 to 0.0347 | 28.8 to 31.6 | 30.2 |
| G | 0.0221 to 0.025 | 39.6 to 45.2 | 42.4 |
| H | 0.0126 to 0.0157 | 63.4 to 79.4 | 71.4 |

Note: 1 ft = 0.3 m.

[a]For maximum filter response. Each filter will also respond to waves with frequencies slightly outside of its band.

Figure 8. Serviceability index relations for wavelength versus amplitude.

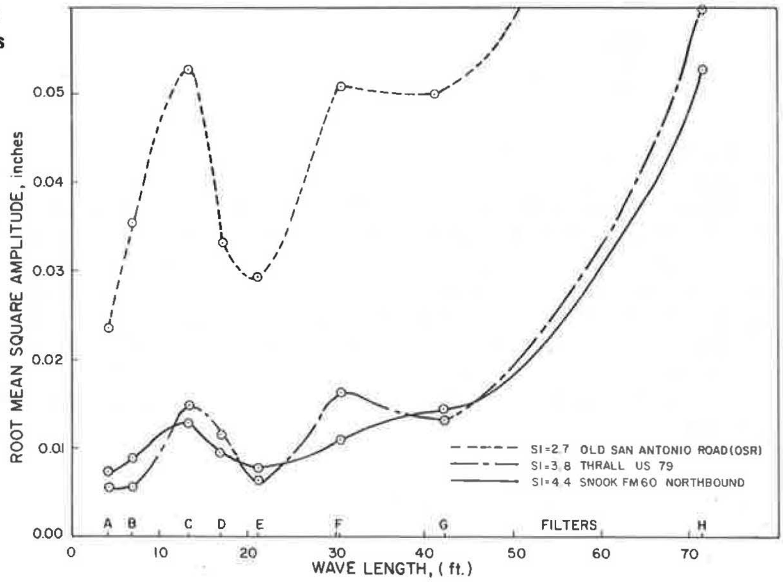



Figure 9. Probability density functions of peak-to-peak distances for left wheel path.

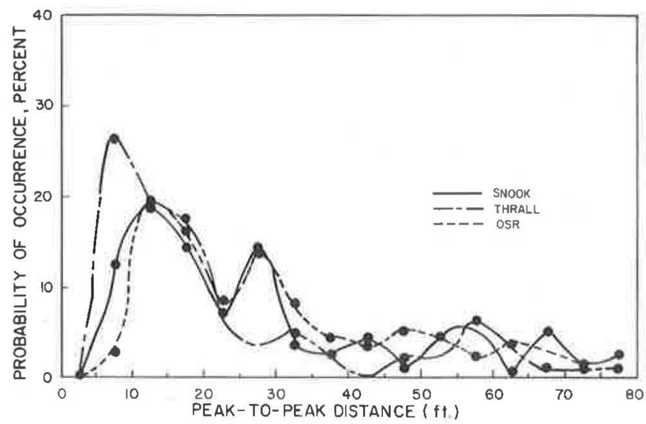
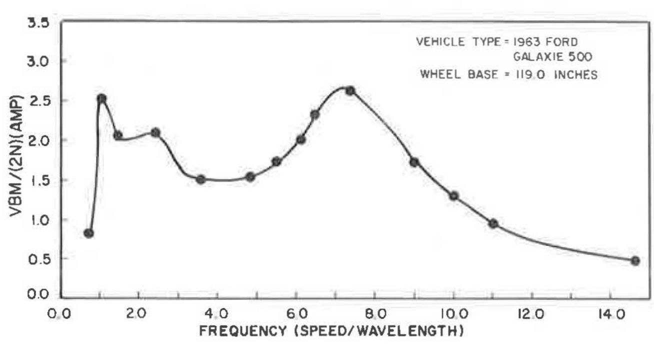



Figure 10. Frequency response curve for computer-simulated vehicle body movement.



A 1963 Galaxie Ford whose mass and suspension characteristics are published in the literature (10) was the simulated vehicle carrying the Mays meter. The Mays meter accumulates all movements of the rear axle toward and away from the vehicle body. This response of the Mays meter may be compared with the actual up and down movement of the pavement surface, and the resulting Mays meter response curve is shown in Figure 10. These data were developed by simulating runs of the 1963 Galaxie Ford over series of sine waves of varying wavelengths. The Mays meter reading is recorded in inches per mile (centimeters per kilometer), and if the road roughness is considered to be composed of a series of sine waves of varying wavelengths $\lambda_i$ with a probability of occurrence $p_i$, then the Mays meter reading for any given section of pavement will be

$$M = b\left[3{,}300 \sum_{i=1}^{n} p_i g_i\left(\frac{v}{\lambda_i}\right) a_i\right]^m \tag{7}$$

where $a_i$ is the amplitude of the wavelength $\lambda_i$ previously mentioned, v is the speed of the vehicle, and $g_i(v/\lambda_i)$ is the Mays meter response in terms of the frequency of excitation $v/\lambda_i$. The constants b and m are needed to account for the effect of the different mass and suspension systems built since 1963. For a typical 1972 Ford, b is 4.0 and m is 0.56. The results of the application of this formula to estimating the riding quality of the section of pavement are shown in Figure 11. Since the Mays meter is normally run at 50 mph (80.5 km/h), the values of serviceability index shown on those curves were determined by the ordinate of each curve at 50 mph (80.5 km/h), using correlations of Mays meter readings with the serviceability index such as the following (1 in./mile = 1.6 cm/km):

| Serviceability Index | Mays Meter Reading (in./mile) | Serviceability Index | Mays Meter Reading (in./mile) |
|---|---|---|---|
| 4.5 | 6 | 2.5 | 25 |
| 4.0 | 10 | 2.0 | 32 |
| 3.5 | 15 | 1.5 | 42 |
| 3.0 | 20 | | |

The wavelength distribution chart for the Snook gilgai (Figure 7) was used to establish the probability of occurrence of each wavelength.

## DYNAMIC TRAFFIC LOADING DUE TO EXPANSIVE CLAYS

Dynamic load factors were determined by DYMOL simulation of a dump truck running across a simulated pavement with sine-wave roughness. The dump truck had a rear axle load of 17.62 kips (78.3 N) (9), which was for all practical purposes the same as an 18-kip (80-N) single-axle design load. The dynamic load factor was defined as follows:

$$d = \frac{F_{max} - F_s}{F_s} \tag{8}$$

where $F_{max}$ is the maximum dynamic load resulting from the sinusoidal excitation, and $F_s$ is the static load on the tire. If this dynamic load factor is divided by the amplitude

Figure 11. Serviceability index relations for vehicle body movement versus speed.

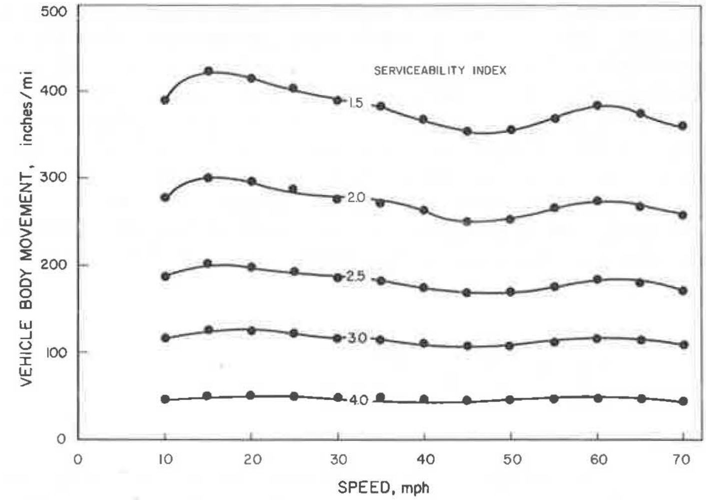



Figure 12. Frequency response curve for vehicle stiffness.

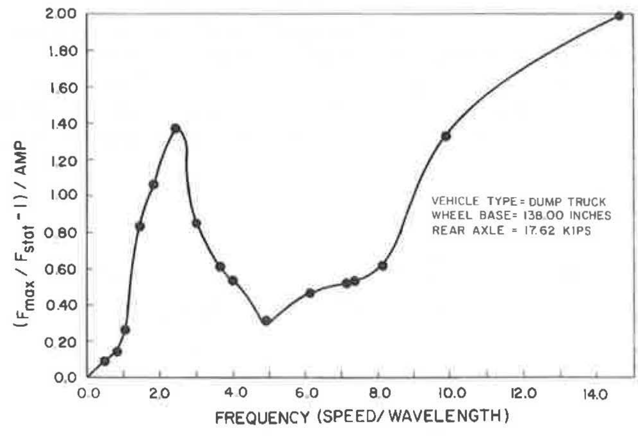
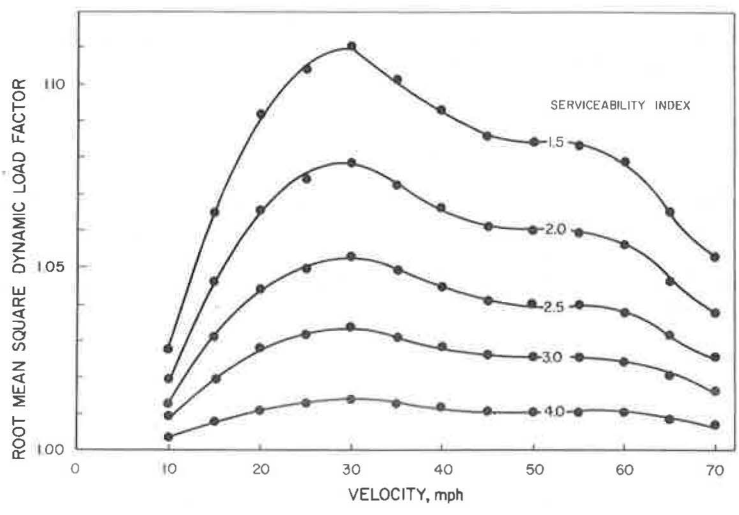



Figure 13. Serviceability index relations for root-mean-square dynamic load factor versus speed.



of excitation a to produce a system stiffness number k,

$$k = \frac{d}{a} = \left(\frac{F_{max}}{F_s} - 1\right)/a \tag{9}$$

then it is found that there is a single curve that relates k to the frequency of vibration. This is shown in Figure 12. The shape of the frequency response and the frequencies at maximum response for this dump truck are similar to those calculated by analog computers (11, 12) and measured in field tests (13). The root-mean-square total dynamic load is a measure of the total load applied to the pavement. The equation for the total dynamic load factor including a root-mean-square dynamic component is

$$d_t = 1 + \frac{1}{\sqrt{2}}\sum_{i=1}^{n} p_i\, k\left(\frac{v}{\lambda_i}\right) a_i \tag{10}$$

where, as before, $p_i$ is the probability of occurrence of the wavelength $\lambda_i$, and $a_i$ is the amplitude of that wavelength, which is a function of serviceability index. This equation was used to determine the dynamic load factor versus speed relations shown in Figure 13. The maximum dynamic loads are applied to the pavement surface when the vehicle is traveling at about 20 to 40 mph (32.2 to 64.4 km/h).

The dynamic load factor term was incorporated into the performance equation of the Texas flexible pavement design system to determine by how much the dynamic loads due to expansive clay roughness will shorten pavement service life. It was found that including the dynamic load factor made practically no difference in the service life regardless of the level of roughness. This is not necessarily the case where cracking is concerned; however, the speed of crack propagation varies as the stress intensity factor to the fourth power (14). The stress intensity factor is, in turn, directly proportional to the applied stress. If the applied stress is 1.05 times greater than the static stress, as is the case with an SI of 2.5, then the rate of propagation of a crack due to traffic loading will be 50 percent faster when the pavement is rough than when it has an SI of 4.0. This is a major incentive to maintaining high riding quality on pavements and overlays.

## PREDICTION OF REHABILITATION REQUIREMENTS

A reliable prediction of the rehabilitation requirements for a pavement that has been roughened by expansive clay depends on at least three factors: (a) the percentage of the road to be reworked, (b) the amount of releveling to be done by heater-planer or other methods, and (c) the amount of level-up and overlay material to be used. The kinds of calculations that would be required to estimate the amount of material are shown in equation 11 for the simple case for which no releveling is to be done. In this case, the amount of level-up material required to fill the hollow between two successive peaks is the product of the amplitude and the wavelength. Then the existing peaks will be overlaid to depth b above the peaks. The relation between amplitude and wavelength for a serviceability index of 2.5 is taken from equation 6 and substituted into equation 11 below. The serviceability index of 2.5 is chosen here, for it is common practice to overlay at around that level. If $p_i$ is the probability that a given wavelength $\lambda_i$ will occur, and $p_r$ is the percentage of the roadway that must be reworked, then the total amount of material required for rehabilitation is

$$M = p_r L \sum_{i=1}^{n} p_i a_i \tag{11}$$

where n is the number of significant amplitudes in the pavement, and L is the project length. Once it is possible to predict when the pavement will need to be reworked in the future, this quantity of material M is multiplied by its unit cost and may be used in estimating the present value of the rehabilitation work. This cost may then be compared with the cost of various kinds of preconstruction treatment such as ponding, wet compaction, chemical treatment, deep-plowed lime or pressure injection at critical points along the pavement such as intersections, grade-crossings, and underpasses. If the cost of pretreatment is higher than the rehabilitation cost, then the designer may normally decide not to use pretreatment. A more usual case may be that a certain amount of pretreatment will greatly reduce the future rehabilitation work to be done or will delay it considerably. In either case, a systems approach to rehabilitation will be able to arrive at the right mixture of preconstruction treatment and rehabilitation.

## CONCLUSIONS

1. There is a strong indication that cracking patterns determine roughness patterns.
2. Mineralogical information such as can be provided by pedologic surveys conducted by the Soil Conservation Service and state agricultural universities may give valuable indications of the kind of roughness to expect. One can conclude that the mineralogical mixture of a clay soil may provide valuable information for roughness predictions because there was not a great difference in the mineralogical makeup of the soils at Snook and Thrall and because there were no statistical differences between the means and standard deviations of both amplitude and wavelength distributions in each of the fields.
3. The rate of development of roughness cannot be predicted reliably at present; further study is indicated.
4. Riding quality is greatly affected by expansive clay roughness, but the pavement service life is not altered greatly by the added dynamic load.
5. Digital filters are useful in determining serviceability index relations for amplitude versus wavelength.
6. Field amplitude-wavelength relations are a practical upper limit of the roughness that will develop on a pavement.
7. The development of amplitude-wavelength relations and the prediction of the rate of change of amplitude and the rate of reduction due to preconstruction treatment of the soil will permit planning and estimation of rehabilitation work to be done on a pavement roughened by expansive clay.

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# PRESSURE-INJECTED LIME FOR TREATMENT OF SWELLING SOILS

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The pressure-injected lime technique for treating swelling soils is described and evaluated. Basic mechanisms of soil-lime reactions and pressure-injected lime are considered, and the effects of treatments with pressure-injected lime are discussed. Typical field experiences with pressure-injected lime are summarized, and the factors that appear to influence the effectiveness of the technique are identified. There are conflicting reports concerning the effectiveness of pressure-injected lime treatment of expansive soils. The condition most favoring the achievement of successful pressure-injected lime treatment of expansive soils is the presence of an extensive fissure and crack network into which the lime slurry can be successfully injected. The proposed treatment mechanisms (prewetting, development of soil-lime moisture barriers, and effective swell restraint with the formation of limited quantities of soil-lime reaction products) have validity. The relative significance of the prewetting and soil-lime pozzolanic reaction aspects of pressure-injected lime treatment has not been established. The various statements and reports in the literature and the information presented in the paper suggest that pressure-injected lime may not be effective under all circumstances but that in appropriate conditions it can be satisfactorily and economically used. It is indicated that appropriate guidelines and principles should be developed for evaluating (on a site-by-site basis) the potential effectiveness of pressure-injected lime treatment.

•EXPANSIVE clays in the design and construction of transportation facilities present a major problem. Many techniques (compaction control, prewetting, heating, and various additive stabilization procedures) have been used for controlling volume changes in expansive soils (1).

Pressure injection of lime-water slurry into the expansive soil deposit is a recently developed procedure that is attracting considerable attention. The purpose of this paper is to describe and evaluate the pressure-injected lime (PIL) technique based on information currently available.

## BACKGROUND

Lime has been widely and successfully used as a stabilizing agent for fine-grained soils. When lime is added to a fine-grained soil and intimately mixed, several reactions are initiated. Cation exchange and agglomeration-flocculation reactions take place rapidly and produce immediate changes in soil plasticity, workability, and swell properties. Plasticity and swell are reduced, and workability is substantially improved because of the low plasticity and the friable nature of the mixture. A soil-lime pozzolanic reaction may commence depending on the characteristics of the soil being stabilized. The pozzolanic reaction results in the formation of various types of hydrated calcium silicate and calcium aluminate cementing agents or both. The cementing agents increase mixture strength and durability. Pozzolanic reactions are time dependent, and strength development is gradual but continuous for a long period of time (several years in some instances).

Extensive studies (2) have shown that practically all fine-grained soils react with lime (cation exchange, agglomeration-flocculation) to effect beneficial changes in workability, plasticity, and swell properties. The extent to which the lime-soil pozzolanic reaction proceeds is influenced primarily by natural soil properties (3). With some soils, the pozzolanic reaction is inhibited, and cementing agents are not extensively formed.

Thompson (3) has termed those soils that react with lime to produce substantial strength increase as reactive and those that display limited pozzolanic reactivity as nonreactive. Properties of compacted and cured soil-lime mixtures will be different for reactive and nonreactive soils.

For the nonreactive soils, plasticity, workability, and swell properties are altered, but strength increases are nominal. Reactive soils initially experience similar plasticity, workability, and swell changes and will attain substantial strength because of the pozzolanic reaction.

In the treatment of subgrade soils with high swell potential, the major objective of lime stabilization is swell control. If the soil and lime can be intimately mixed, it is highly probable that the soil swell potential can be drastically reduced. Holtz (4) summarized the beneficial effects of lime treatment relative to the control of volume change in expansive clay soils. Not all soils respond favorably to lime treatment as a control of swell potential. Plummer (5) reported that certain of the Red River, North Dakota, clays will expand around 10 percent even after lime treatment. Mitchell (1) indicated that lime was the most effective additive for stabilization.

Even though the efficacy of lime for swell control treatment has been conclusively demonstrated, the reality of field conditions has limited the use of conventional lime stabilization. Holtz (4) has suggested that for effective swell control the expansive soil should be stabilized to a depth of approximately 5 ft (1.5 m). The costs associated with using conventional soil-lime stabilization procedures for treating the subgrade of a typical roadway section to a 5-ft (1.5-m) depth are substantial and in many instances prohibitive.

Thus, other techniques such as drill-hole lime and pressure-injected lime have been devised in an attempt to lime stabilize clays in their in situ state.

## DRILL-HOLE LIME

The drill-hole lime technique basically consists of introducing quicklime or hydrated lime into a soil mass by placing the lime in holes drilled in the soil mass. When placed in the holes, the lime (usually hydrated lime in a slurry form) migrates or diffuses into the soil system thereby initiating the soil-lime reactions. If sufficient migration or diffusion occurs, it is possible that the properties of a sizable quantity of the soil mass around the drill hole will be improved. However, lime migration or diffusion is a very slow process, and substantial time may be required before a substantial quantity of soil is affected.

In highway applications, small-diameter [6 to 12 in. (15 to 31 cm)] holes are advanced through the pavement into the subgrade soil by using a suitable apparatus such as a power post-hole digger equipped with a continuous flight auger. In highway applications, provision must be made to construct a hole in the pavement structure to gain access to the subgrade. Typically, the depths of the drill holes range from 30 to 50 in. (76 to 127 cm). The exact hole depth depends largely on the depth and nature of material to be treated. Typical hole spacings are about 4 to 5 ft (1.2 to 1.5 m) center-to-center.

After the hole has been made, it is partially filled with either quicklime or hydrated lime. In some cases, water is added to the lime to create a slurry, or a lime-water slurry is placed directly in the drill hole. However, dry lime (especially quicklime) is thought to act as a drying agent that absorbs soil moisture, thereby reducing the moisture content of the surrounding soil. The use of a lime-water slurry may, however, tend to increase the mobility of the lime since water acts as a medium for migration.

Backfilling the hole and patching the pavement are normally required. Both soil (from the drill hole) and aggregate have been used. The backfill should be tamped into the hole. The holes in the pavement may be patched with portland cement concrete or asphaltic concrete.

Several studies (6, 7, 8, 9) have considered the drill-hole procedure. Although in many instances the major stabilization objective was not swell control, the background information developed in the studies is still applicable.

In general, drill-hole lime treatment results have been erratic. Some report success, but others indicate that little or no improvement has been achieved. The various investigations have shown that the zone of influence in which soil-lime reactions have taken place is limited to the areas immediately adjacent to the drill hole.

It is apparent that the major factor limiting the effectiveness of the drill-hole lime procedure is the inability to achieve lime distribution throughout the soil mass.

## PRESSURE-INJECTED LIME

In an attempt to achieve better lime distribution in the soil mass, the PIL procedure was developed. In this procedure, a lime-water slurry is pumped under pressure through hollow injection rods into the soil. Generally, the injection rods are pushed into the soil in about 12-in. (31-cm) intervals. At each depth, the lime slurry is injected to refusal. Refusal occurs when

1. Soil will not take additional slurry,
2. Slurry is running freely on the surface either around the injection pipe or out of previous injection holes, or
3. Injection has fractured or distorted the pavement surface.

Although there is substantial variability in the amount of slurry that can be injected, a normal take is about 10 gal/ft (124 liters/m) of injected depth. Obviously, the nature of the soil being treated will influence the quantity of slurry that can be injected.

The normal lime-water slurry composition is $2\frac{1}{2}$ to 3 lb of lime/gal (0.3 to 0.4 kg of lime/liter) of water with a wetting agent added in accordance with the manufacturer's recommendation. Based on extensive field experience, the above slurry composition has proved to be satisfactory.

Although injection pressures as high as several hundred pounds per inch$^2$ (pascals) can be developed with most lime slurry injection equipment, the majority of the work is injected in the pressure range of 50 to 200 psi (345 to 1380 kPa). In this pressure range, it is normally possible to disperse the maximum amount of slurry into the soil.

Spacings of 3 to 5 ft (0.9 to 1.5 m) on centers are common in pressure injection treatment for building foundation work. Spacings of 4 ft (1.2 m) were used in the deep-layer stabilization flexible pavement test sections at Altus Air Force Base, Oklahoma. Spacings of 5 ft (1.5 m) are also typical for PIL treatment of railroad subgrades. Various pressure injection contractors in the Dallas-Fort Worth, Texas, area indicated that the amount of lime slurry that could be injected per unit volume of soil was independent of injection probe spacing within the range of 3 to 6 ft (0.9 to 1.8 m).

Injection depths are variable, but current equipment is capable of injecting to depths of approximately 10 ft (3 m). Wright (10) has indicated that a treatment depth of 7 ft (2.1 m) is normally sufficient for foundation treatments. This depth compares reasonably well with the 5-ft (1.5-m) depth suggested by Holtz (4). The general guide is to inject to a depth sufficient to be below the zone of critical moisture change in the expansive soil deposit.

If the surface of the PIL-treated soil deposit is exposed, it is common practice to mix the free surface lime available into the soil to a depth of 6 to 8 in. (15 to 20 cm). The stabilized layer further contributes to the process of retarding moisture loss from the underlying soil. Teng, Mattox, and Clisby (11) have shown the effectiveness of a lime-treated Yazoo clay layer to act as an effective capillary barrier for preventing desiccation. Similar results have been found in a wide variety of soil-lime stabilization applications.

Field studies (12, 13) in which PIL-treated soils have been excavated show that the PIL slurry is forced along fracture zones, cracks, fissures, bedding planes, root lines, coarse-textured seams in varved clays, seams and fractures effected by the pressure slurry injection process, or other passages in the soil mass. The field observations and a recent laboratory investigation (14) have conclusively demonstrated that the lime-water slurry will not permeate an intact fine-grained soil mass.

Hillel (15) states,

> The hydraulic conductivity (of the soil mass) is obviously affected by structure as well as by texture, being greater if the soil is highly porous, fractured, or aggregated than if it is tightly compacted and dense. The conductivity depends not only on the total porosity, but also, and primarily, on the sizes of the conducting pores.

Lytton (16) in characterizing the geomorphological aspects of expansive clay indicated that in gilgai land forms the "soil is fractured to great depths." In the same paper, Lytton also discusses the crack pattern formation process in expansive soils. The fact that expansive soil deposits are typically cracked or fractured in the near surface depths that are of concern in swell control procedures is helpful when the problem of trying to pressure-inject lime-water slurry into the soil mass is considered.

It is apparent that the presence of openings in the soil mass is requisite for obtaining adequate slurry distribution. Wright (10) has described the final lime distribution pattern frequently obtained in swelling clays as "a network of horizontal, sheet lime seams, often interconnected with vertical or angular veins."

## TREATMENT MECHANISMS

From the previously presented information, it is apparent that there are two major treatment mechanisms of concern relative to PIL. The first is the ability to permeate the soil mass with the stabilizing additive (in this case a lime-water slurry), and the second is the process whereby, following PIL treatment, the lime translocates and modifies the soil adjacent to the lime seams.

### Lime Injection

When the basic theory of permeability is combined with Darcy's law of fluid flow in a soil mass, the total quantity of fluid that can be forced into a soil mass during a given interval of time can be approximated by equation 1:

$$Q = \frac{\rho g}{\eta} \left(\frac{p}{\ell}\right) K A t \tag{1}$$

where

- $A$ = cross-sectional area over which pressure acts;
- $\eta$ = viscosity of fluid;
- $g$ = acceleration due to gravity;
- $p$ = pressure head;
- $K$ = $Cd^2$, intrinsic permeability of medium, where $C$ is a shape factor and $d$ is average pore size of mdeium;
- $\ell$ = length over which pressure head acts;
- $Q$ = quantity of fluid flow;
- $\rho$ = density of fluid; and
- $t$ = time of pressure application.

Equation 1 indicates that the following major factors control the quantity of fluid that is injected: (a) fluid viscosity, (b) injection pressure and time, and (c) intrinsic permeability of the soil medium.

Since lime slurry is not an ideal fluid but rather a particulate suspension, the pore size distribution of the soil mass is an important consideration in the permeation process. Successful injection of the lime slurry into the soil mass would require that channels larger than the lime particles be present.

The inherent pore size of most fine-grained soils is quite small relative to the lime particle size. Thus, appreciable lime slurry movement through these pores is questionable.

Johnson (17) recommends that the groutability ratio, as calculated by using equation 2, be greater than 20 to 25 for successful cement grouting.

$$\text{Groutability ratio} = \frac{D_{(15)}\ \text{soil}}{D_{(85)}\ \text{grout}} \tag{2}$$

where

$D_{(15)}$ soil = particle size for which 15 percent of the soil fraction is finer, and
$D_{(85)}$ grout = particle size for which 85 percent of the cement grout is finer.

Many commercial hydrated lime specifications for soil stabilization purposes require a minimum of 85 percent passing the No. 200 sieve. Using 0.0029 in. (0.074 mm) as $D_{(85)}$ in Johnson's equation indicates the $D_{(15)}$ for the soil must be approximately 0.059 in. (1.5 mm) or larger to meet a groutability ratio criterion of 20. It is obvious that for fine-grained expansive soils the lime slurry cannot be effectively forced through the soil pore system.

Laboratory studies conducted at the University of Illinois (14) indicate that it is almost impossible to force a typical lime slurry (30 percent by weight) into fine-grained soils even when pressures of up to 1,000 psi (6.9 MPa) are applied for 20 min. Typical results from this limited study are shown in Figure 1. In general, slurry penetrations averaged less than ½ in. (12.7 mm) into the silty materials, and almost no penetration was achieved in the clayey materials.

Teng, Mattox, and Clisby (11) found in their water flooding studies of a remolded and compacted Yazoo clay embankment that little moisture penetration was achieved. The flooding of the undisturbed Yazoo clay in the cut sections was successful. These findings caused them to conclude, "The fissure system plays a very significant role in allowing water intrusion into the soil mass thus causing swell." The significance of the fissure system would be equally important for the PIL procedure. It is important to note that the fissure structure in the Yazoo clay study (11) was present in the in situ cut sections, but the fissure structure was destroyed in the embankment construction process.

It is apparent that to successfully pressure-inject lime slurry into a fine-grained soil natural channels and passages larger than the lime particles must be present in the soil mass. Such channels may be present as a result of (a) inherent pore structure of the soil mass; (b) cracks, fissures, seams, and root holes present in the soils; or (c) jetting or tearing of the soil effected by the pressure injection process.

Even though the permeability of the soil resulting from the inherent soil pore structure may be quite low, the mass permeability or conductivity may be substantially higher as a result of the presence of seams, fissures, cracks, varves, and so on. When this condition exists, the potential for successful lime slurry injection of a soil mass is greatly enhanced.

Diffusion-Migration of Lime

Based on the preceding discussion, it is apparent that

1. Intimate permeation of fine-grained soils through PIL is virtually impossible, and
2. Lime slurry can be pressure-injected into certain fine-grained soil masses if varves, seams, fissures, and cracks exist but the distribution of the slurry is stratified or of a network type.

However, the lime will tend to be translocated in the fine-grained soil mass as time progresses because of diffusion and migration phenomena.

Space limitations do not allow presentation and discussion of diffusion-migration theory. In general, however, factors such as differences in clay content, clay minerals, density, absorbed cations, and temperature have been found to affect the rate of diffusion (18, 19, 20, 21).

In an early study, Davidson, Demirel, and Handy (22) suggested that the diffusion of calcium cations in a soil-lime water system is an example of the diffusion phenomena. The processes accompanying lime diffusion may include (a) transfer of lime into the soil, (b) chemical reaction between the lime and the soil, (c) formation of nuclei and growth of the reaction product, and (d) further diffusion of the lime into the soil from the reaction product layer.

The following equation has been suggested for determining the rate of growth of a product layer from the lime source (22, 23):

$$\ell = k_d \sqrt{t} \tag{3}$$

where

$\ell$ = distance of lime migration for a time t in inches (millimeters),
$k_d$ = diffusion constant in inches/day$^{1/2}$ [reported values range from 0.081 to 0.63 in./day$^{1/2}$ (2 to 16 mm/day$^{1/2}$) (22, 23)], and
t = elapsed time of diffusion in days.

Limited laboratory and field studies have been conducted to evaluate the rate and extent of lime migration. In a controlled laboratory study conducted by Fohs and Kinter (24), about 0.8 percent lime was found to migrate approximately $1\frac{1}{2}$ to 2 in. (3.8 to 5 cm) after 180 days. They concluded that the migration process for effecting translocation of lime in a soil system is very slow and that only very small amounts of lime can be translocated. This makes this process impractical for effecting substantial soil mass strength increases (24). Robnett, Jamison, and Thompson (14) conducted a limited laboratory lime migration study. Typical results are shown in Figure 2. It is apparent that the amount of lime translocated by the migration process is small.

In a field study, Lundy, Jr., and Greenfield (13) found that after 1 year approximately $^3/_4$ to $1\frac{1}{2}$ in. (19 to 38 mm) of lime migration had occurred away from the lime seams. A Louisiana Department of Highways study (12) found that about $\frac{1}{2}$ to $1\frac{1}{2}$ in. (13 to 38 mm) of lime migration occurred after 4 years.

It is evident that lime translocation by the diffusion-migration process is very slow. If equation 3 is used to estimate the required time for various distances of migration, the following values are found, if $k_d$ is assumed to be 0.10 in./day$^{1/2}$ (2.5 mm/day$^{1/2}$) (1 in. = 25.4 mm):

Figure 1. Typical lime-water slurry pressure penetration data for Fayette C soil [AASHO A 4(8)].

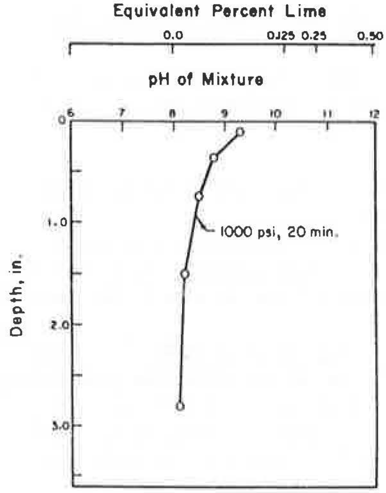
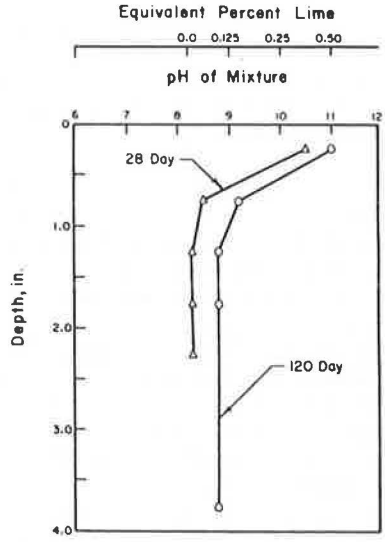



Figure 2. Typical lime migration data for Altus subgrade soil [AASHO A 6(13)].




| $\ell$ (in.) | t (days) |
|---|---|
| 1 | 100 |
| 6 | 3,600 |
| 12 | 14,600 |

Recent comprehensive studies by Stocker (25) have led to the development of an integrated theory of soil-lime stabilization reactions termed diffusion and diffuse cementation. Diffuse cementation theory as proposed by Stocker describes a process in which lime (from a PIL-deposited lime seam in the situation of interest) will diffuse into a natural soil lump. Based on his studies, Stocker stated,

> The diffused lime is shown to react with all the clay present, including that within unpulverized lumps, leading first to volume-stabilization (against wetting and drying) and increase in soaked strength, and later to remarkable increases in even unsoaked strength (for relatively high stabilizer contents). This cementation is diffuse. . . . It [diffuse cementation] is the dominant mechanism whereby included lumps of unpulverized soil are made impotent with respect to differential volume change and finally by which the lumps are increased in mechanical strength.

Stocker indicated that, in a lime-covered, soil-lump system in which diffuse cementation is occurring, the earliest physical property change effected is the "suppression of swell or wetting from the as-cured state." His studies further suggest that the development of limited cementing material is sufficient to prevent swelling but does not contribute to the development of substantial compressive or shear strength.

Stocker's diffuse cementation theory (25) suggests that in lime-reactive soils soil-pozzolanic reaction products may form in regions of low calcium concentration remote from the lime source. The applicability of the diffuse cementation theory depends on the soil being lime reactive (soil will react with lime to form calcium silicate and calcium aluminate hydrates).

## EFFECTS OF PRESSURE-INJECTED LIME

As a consequence of PIL treatment, major changes are effected in the soil (assuming that there are sufficient soil passageways to facilitate lime slurry distribution in the soil mass). The soil moisture content is increased, and an initial network of lime seams is formed in the soil.

It is well established that initial water content has a significant influence on volume change in expansive soils. In general, the lower the initial water content is, the higher the swell will be. In some PIL jobs the moisture content of the treated soil is specified. Wright (10) indicates that a typical final moisture content requirement is 1 to 2 percent above the plastic limit. It is not uncommon to achieve several inches (millimeters) of swelling in the deposit following PIL, depending on the moisture content of the expansive soil before injection (10, 26). Mitchell and Raad (1) have considered the application of prewetting as a method of controlling volume change in expansive earth materials. Many engineers are prone to overlook this important aspect of PIL treatment, which is in reality a form of prewetting.

Following PIL treatment, soil-lime reactions may occur in the areas adjacent to the lime seam network. Lime diffusion-migration takes place as previously discussed. If the soil is reactive, soil-lime cementing products will form. The stabilized zones serve as moisture barriers. Stocker (25), as previously discussed, indicated that the formation of limited accumulations of soil-lime reaction products at or near the edges of the clay particles would effectively restrain moisture-related volume expansion. Even if only the boundaries of a soil mass have reacted with lime, Stocker (25) suggests that the nonswelling shell of modified soil may constrain the relatively unaffected core. Stocker's work with lime treatment of lumps of montmorillonitic soil even suggests that the swell characteristics of the soil had been substantially modified ahead of the penetrating diffusion-reaction front.

It is important to recall that, in a practical field application, the potential for soil swell is previously reduced by the PIL treatment prewetting effect so that the preservation of the in situ soil moisture content alone will ensure volume constancy. The restraint developed from the formation of reaction products is an added beneficial effect. It was noted by Stocker that higher soil moisture contents increase the rate of development of diffuse cementation. Thus the water content increase effected by PIL will enhance the diffusion cementation process.

The preceding discussion indicates that there are several potentially beneficial processes that can occur in a PIL-treated expansive soil deposit. Under the proper circumstances, PIL treatment should serve as an effective swell control procedure. At this time it is not possible to accurately define what conditions are required to ensure a high probability of success.

## EVALUATION OF PRESSURE-INJECTED LIME

PIL has gained acceptance as a possible procedure for treating expansive soils in building foundation applications, but has not yet achieved the same degree of acceptance in transportation facility construction. PIL was not considered in any depth at the recent Denver conference, but some comments were made concerning lime treatment techniques.

1. Kelley and Kelly (27) referred to the successful use of PIL to treat the swelling soils under the Dallas-Fort Worth Regional Airport terminal buildings.
2. Krazynski (28) indicated that he did not think PIL treatments were very effective.
3. Teng, Mattox, and Clisby (11) thought that research and experience have shown that lime stabilization by pressure injection has not lessened the swell potential.
4. Blacklock (26), a PIL contractor, indicated that, based on results, PIL was definitely a solution to the swelling problem.
5. Gerhardt (29) reported on the successful use of drill-hole lime stabilization in a Colorado test section and concluded that potential swell can be reduced to almost

nothing for any depth.

6. Brakey (30) suggested that the drill-hole lime procedures were not effective in providing protection against expansion in the Mancos shale.

Ingles and Neil (31) have evaluated the effects of PIL treatment for typical Australian soil conditions. Their studies indicated that PIL treatment of expansive soils was an effective procedure, but only when the treatment was carried out at the time of maximum desiccation when the soil is fissured.

Robnett, Jamison, and Thompson in recent studies (32) of deep-layer stabilization procedures for pavement systems also considered the various aspects of PIL. Of the various procedures evaluated, the PIL procedure was determined to be one of the more promising methods.

Wright (10) cited the rapid growth of PIL building foundation treatment in Fort Worth, Texas, and indicated that the use of the procedure was spreading to other parts of Texas as well as Arkansas, Tennessee, Louisiana, and Oklahoma. It is apparent that PIL is an accepted expansive soils treatment procedure in some geographic areas.

The various statements and reports in the literature suggest that PIL may not be effective under all circumstances, but under appropriate conditions it can be satisfactorily and economically used. It appears that higher probabilities of success are achieved when the job conditions permit the satisfactory injection of lime slurry and the development of a comprehensive network of lime seams throughout the soil mass. The presence of an extensive fissure and crack system in the soil seems to be necessary.

If the lime slurry can be successfully injected throughout the fissures or cracks in the soil mass, the prewetting phase of the PIL treatment can then occur. The soil-lime pozzolanic reaction will also commence (reaction rate is influenced by time and temperature) if the soil is reactive. It is thus possible that in some PIL applications only the prewetting effect is achieved and that the soil-lime pozzolanic reaction does not occur. In such a case, the moisture barrier developed in the areas of the lime seams may not be as effective because any soil property change will be due to cation exchange and perhaps limited changes in soil structure but will not show the benefit of soil-lime reaction product formation. Whether or not the soil-lime pozzolanic reaction is essential for successful long-term PIL treatment has not been established. However, Stocker's theory (25) indicates that a lime-reactive soil is essential if effective diffuse cementation at points remote from the lime source is to be achieved. Fortunately, many expansive soils are lime reactive.

## SUMMARY

There are conflicting reports concerning the effectiveness of PIL treatment of expansive soils. The proposed treatment mechanisms (prewetting, the development of soil-lime moisture barriers, and effective swell restraint with the formation of limited quantities of soil-lime reaction products) have validity. It therefore seems logical to conclude the PIL may be an effective swell control procedure under certain circumstances. The condition most favoring the achievement of successful PIL treatment of expansive soils is the presence of an extensive fissure and crack network into which the lime slurry can be successfully injected. The relative significance of the prewetting and soil-lime pozzolanic reaction aspects of PIL treatment has not been established. If soil-lime pozzolanic reactions are essential to achieving an effective application, perhaps that fact can be used to evaluate the potential success of an anticipated treatment and explain the apparent conflicting reports on PIL experience.

It is suggested that future research and development activities focus on the following areas:

1. The relative significance of prewetting and soil-lime pozzolanic reactions in PIL treatments,
2. Consideration of Stocker's diffusion cementation theory and its application to

PIL treatment, and

3. Development of appropriate guidelines and principles for evaluating (on a site-by-site basis) the potential effectiveness of PIL treatment.

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# MEASUREMENTS BENEATH THE SURFACE OF EXPANSIVE CLAY

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Several methods for predicting moisture movement and potential heave of expansive soils are available. In June 1973, a field experiment was initiated because a dearth of integrated field data precluded evaluation of these predictive methods. Various methods were developed and used to measure changes in density, water content, and soil suction in relation to surface and subsurface heave at depths to 15 ft (4.6 m) below an expansive clay surface. This paper describes the methods used, method of installation, and preliminary results.

IN 1968, a study of expansive clays in roadways was started at the University of Texas at Austin in cooperation with the Texas Highway Department. The purpose of this research was to develop an understanding of the shrinking and swelling behavior of expansive soils in the subgrade so that the resulting loss in service life and riding quality of highway pavements could be reduced.

Four study objectives were established:

1. Review of the magnitude of the swelling subgrade problem in Texas,
2. Development of a method for predicting moisture movement in an unsaturated soil,
3. Development of a method for predicting the time rate of heave, and
4. Controlled field experiments for development of reliable case-study data for evaluating the analytical methods in objectives 2 and 3 above and for proving experimental and measurement techniques.

Work in the areas covered by objectives 1, 2, and 3 has been completed and has been reported on elsewhere (1, 2, 3, 4, 5). This paper presents some of the interim results from the field experiments conducted for objective 4. These experiments will continue through the spring of 1975.

The field experiments were conducted because the dearth of integrated field data precluded evaluation of the predictive methods developed earlier in the project. To predict the heave of a clay subgrade, one should know the soil properties, such as density, water content, and soil suction, and the interrelationships of these properties. The regular measurement of these quantities and the corresponding swell with time at various depths below the surface makes for a more comprehensive field experiment than has been previously undertaken.

A search was initiated for a site with a relatively homogeneous, expansive clay profile. A site for the experimental work was located at Lake Long, 7 miles (11 km) east of Austin, Texas. The soil at the site is Taylor marl, historically known to be highly expansive. After an exploration program at several potential sites, one location was selected, and further borings were made to provide information on in situ conditions and samples for a laboratory testing program.

# CLIMATE AND SOIL DATA AT EXPERIMENTAL SITE

## Climate

The climate of the area is mild, normally with cool, wet winters and hot, dry summers. Rainfall averages 31 to 33 in. (78 to 83 cm) annually, but variation from year to year may be extreme. More than half of the annual rainfall normally occurs during the last 4 months of the year. Little rain occurs during late spring and summer.

## Geology

The expansive clay found at the site is the weathered product of a member of the Taylor group. The Taylor group is part of the Gulf series and was deposited in the late Cretaceous period in a relatively shallow sea. Named the Taylor marls by R. T. Hill in 1891, the group was subsequently divided into three formations, from oldest to youngest: Sprinkle, Pecan Gap, and Bergstrom. The last is the formation at the test site.

The Bergstrom formation ranges from 330 to 380 ft (100 to 116 m) in thickness. Unweathered, the formation is a gray to greenish-gray, unctuous, slightly fissle, montmorillonitic clay. Near the surface, the formation weathers to a yellowish-tan to tan clay. The color change is due to the oxidation of the iron content of the clay. Large, repeated changes in water content and chemical alteration have caused the clay to develop into a very stiff, well-fissured material.

In the weathered zone, an approximate composition is 60 percent clay, 30 percent calcium carbonate, and 10 percent quartz. An approximate composition of the clay portion is 7 percent kaolinite, 5 percent illite, 69 percent Ca-montmorillonite, and 19 percent Na-montmorillonite (6).

## Soil Properties

A general soil profile is shown in Figure 1. The upper 4 to 5 ft (1.2 to 1.5 m) are a heavy, dark-gray topsoil with embedded gravel and cobbles. Below this is the weathered Taylor marl to a depth of about 40 ft (12 m). Underlying this is the hard, gray, unweathered Taylor marl.

In situ water content of the upper 3 to 5 ft (0.9 to 1.5 m) of the profile depends on the recent climatic conditions. The water content fluctuates from less than 10 percent to more than 40 percent near the surface. During extended periods of dry weather, extensive shrinkage cracking takes place. Cracks, 3 in. (7.6 cm) wide and with measured depths greater than 3 ft (0.9 m), were observed at the site during the summer of 1974. Excavations have shown that the cracking extends considerably deeper, possibly to 10 to 15 ft (3 to 4.5 m) in extreme drought periods. Below the 3 to 5-ft (0.9 to 1.5-m) zone, the in situ water content ranges from 18 to 22 percent. There is no groundwater table.

Properties of the weathered Taylor marl are shown in Figure 2 and are given in the table below:

| Property | Range (percent) | Average (percent) |
|---|---|---|
| Liquid limit | 57 to 70 | 62 |
| Plastic limit | 17 to 29 | 18 |
| Shrinkage limit | 10 to 14 | 13 |
| Percent passing No. 200 sieve | 98 to 99 + | 99 |

Figure 1. Soil profile at experimental site.

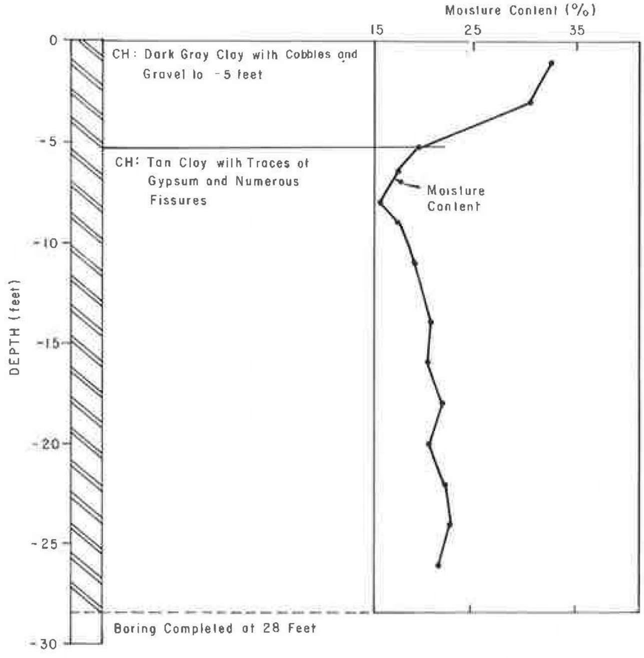



Figure 2. Suction versus water content.

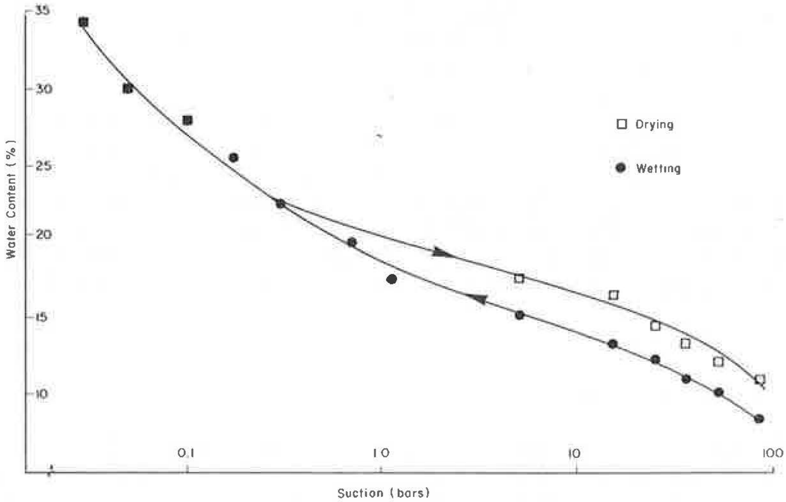



Figure 3. Overall view of experimental site.

The relationship of suction versus water content was determined by using a suction-plate apparatus, a vacuum desiccator, and psychrometric techniques.

### Potential Heave

Although Taylor marl is known to be highly expansive, some measure of the potential activity of the experimental site was needed before development. This measure was required to avoid developing a site where insignificant changes from the in situ conditions would occur. A measure of activity is the predicted heave. McDowell's method (7) of potential vertical rise (PVR) was used to make this prediction because it was developed from data on expansive clays in Texas. The PVR for the moisture content profile in Figure 1 and the soil properties in Table 1 was 4 in. (10 cm). This indicated that the site was suitable for experimental purposes.

## INSTRUMENTATION

### Layout

An overall view of the experimental site is shown in Figure 3; there are three instrumented test areas, four deep benchmarks, and a permanent level mount. The instrumentation layout in test areas 1, 2, and 3 is shown in Figures 4, 5, and 6 respectively. Approximately 5 ft (1.5 m) of overburden were removed from test areas 1 and 2 to reach the weathered Taylor marl. Each test area was 40 by 40 ft (12 by 12 m) in plan at the top of the weathered Taylor marl. The overburden was the heavy, gray clay with gravel and cobbles as mentioned previously. Both of these areas were then covered with a 6-in. (15-cm) washed sand blanket, which served as a working platform.

Test area 1 has a horizontal bottom to provide only vertical entry of water into the soil. Test area 2 has a 5-ft-deep (1.5-m), sand-filled trench on one diagonal. This trench was intended to permit horizontal entry of water. Its purpose is to determine if this provision significantly accelerates the water adsorption of the soil in the vicinity of the trench. Test area 3 is a control section. The depth and position of the instrumentation were based on the results of the prediction methods developed earlier in the project. The data for these studies were obtained from the exploration program and laboratory testing.

### Heave Measurement

Measurement of surface heave is basic to any field study of swelling clays. For this study, subsurface heave measurement at various depths was desired. These data were needed to determine the source of the surface heave at different points in time. A simple heave indicator was devised to obtain these measurements and is shown in Figure 7.

The method of installation is simple. Before emplacement is started, the Teflon-encased, stainless-steel wire is threaded through a ½-in. (12.7-mm), thick-walled steel tube that is used as a push tube. The push tube is then fitted into its socket in the steel point of the heave indicator. To transmit an axial force to the push tube, ½-in. (12.7-mm) prestress cable clamps are used. These wedge type of clamps grip the rod wherever desired but can be easily released for movement to another position. Force is applied to the clamp by a special push-down kelly mounted on a drill rig or a 15-lb (6.8-kg) drop weight. The drop weight has a ¾-in.-diameter (19-mm) axial hole that permits the weight to slide up and down the push tube.

When the heave indicator is emplaced to the desired depth, the push tube is withdrawn. Then, the hole is filled with a pourable, two-part polyurethane sealer to prevent water from entering and closing the hole. The Teflon tubing prevents the sealer from adhering to the wire.

Figure 4. Instrumentation in test area 1.

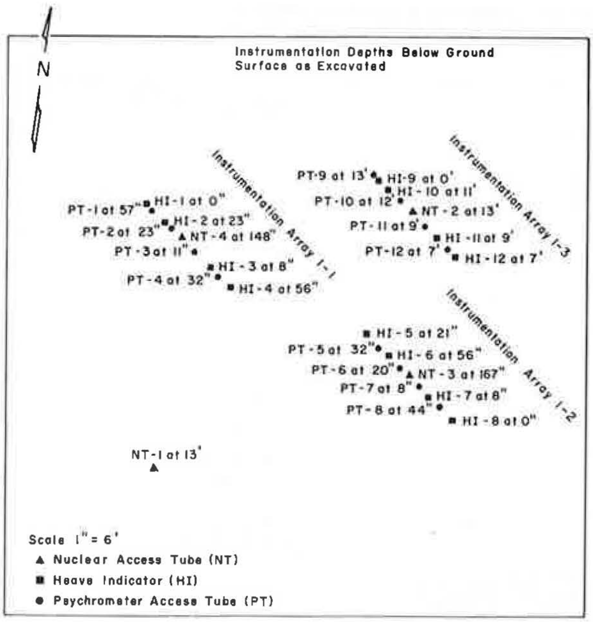
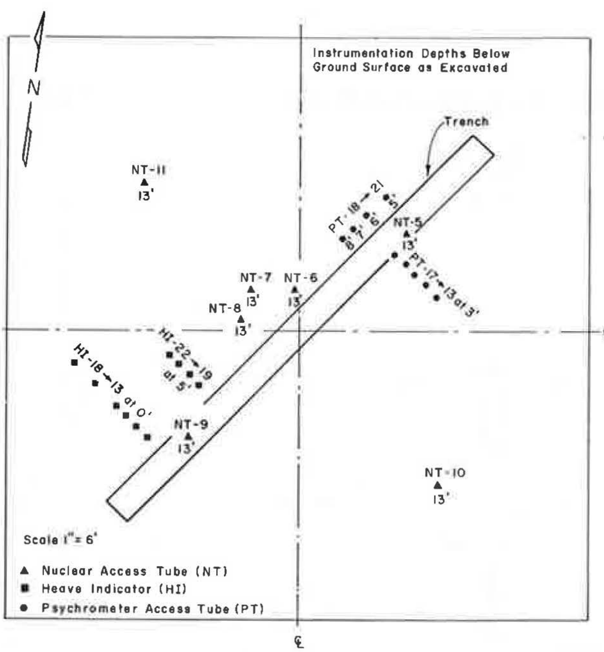



Figure 5. Instrumentation in test area 2.




Above the ground surface, the wire is attached to a constant-tension spring mounted on a support structure. A 12-in. (30-cm) machinist's rule is clamped to the wire as a reference for measuring elevation changes.

Surface heave indicators are 1-qt (0.9-liter) cans filled with concrete and have a stainless-steel wire with a rule attached.

Four 28-ft-deep (8.5-m) benchmarks were installed at the site to provide an elevation reference. Rule elevations were determined by backsighting on the benchmarks and then by foresighting on the rules. Zero elevations on the rule were then computed

Figure 6. Instrumentation in test area 3.

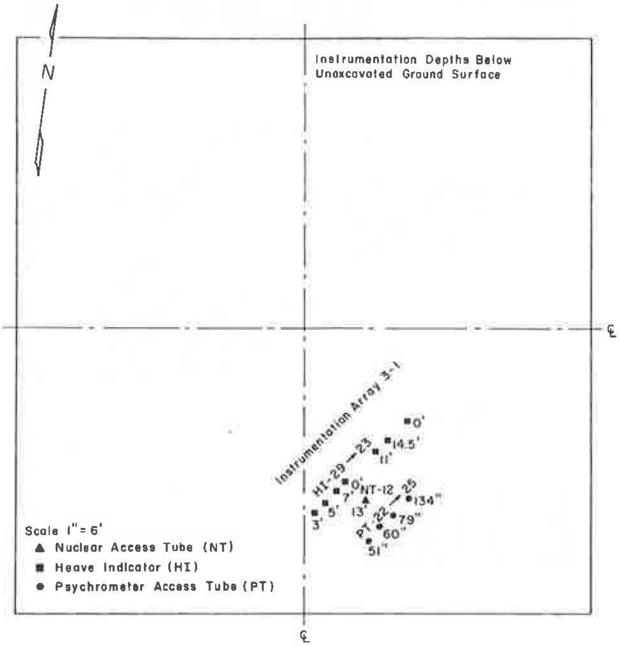



Figure 7. Heave indicator assembly.

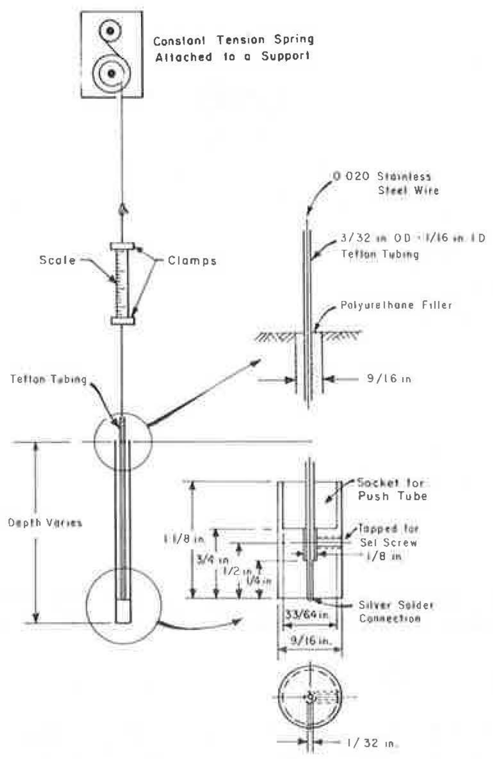
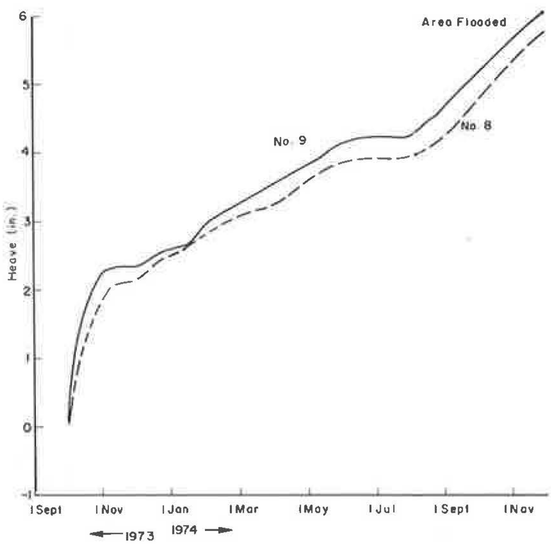



Figure 8. Records of two surface heave indicators.



and compared with the previous elevations to determine the heave.

In test area 1, there are three surface heave indicators and two subsurface heave indicators, at depths of 8, 22, and 56 in. (20, 56, and 142 cm). In that all heave indicators are subject to the same conditions, comparison of the heave records of indicators at the same depth would show the variation to be expected. If the variation is small, there is confidence that the measured heaves are reliable. Some variation is expected because of the natural heterogeneity of the soil. Figure 8 shows the heave records of surface points 8 and 9. The records essentially parallel each other, and the maximum difference between the curves is 4 percent of the total heave. Records of the other heave indicators at the same depth show the same degree of variation.

## In Situ Moisture Content and Density Measurements

In situ moisture content and density measurements were made with nuclear equipment manufactured by the Troxler Electronics Laboratory. The equipment consists of a readout device (scaler) and two nuclear probes that are inserted in previously installed access tubing. The access tubes are seamless aluminum tubing, with an outside diameter of 2.00 in. (5.08 cm) and an inside diameter of 1.90 in. (4.83 cm), sealed at the bottom end and installed in 2.10-in. (5.33-cm) boreholes. The system is shown in Figure 9.

Previous use of the nuclear equipment had shown a need for recalibration because of considerable differences between values obtained under known conditions and values from the manufacturer's calibration. Recalibration was performed in the laboratory by measuring the moisture content and density of several soils compacted at several known water contents and densities in a steel drum.

Moisture content samples were also taken from the boring for each access tube. After installation of the access tube, moisture content and density measurements were made with the nuclear equipment. The nuclear measurements determine the moisture content and wet density in pounds per cubic foot (kilograms per cubic meter). Both values are needed to compute the moisture content in the familiar gravimetric form for comparison with the soil samples. This comparison is shown in Figure 10. The good agreement means that our calibration gives an accurate picture of in situ conditions.

The nuclear equipment has been relatively trouble free. In three cases, the soil squeezed in the access tube sufficiently to prevent passage of the probe. The access tube was restored to its original size by a 1.90-in. (4.83-cm) mandrel driven through the narrowed section, and the problem has not recurred.

## Suction Measurements

In situ suction measurements were made with Spanner double-junction psychrometers. Use of these psychrometers has been described elsewhere (8). After the psychrometer was received from the manufacturer, each one was calibrated over KCl and NaCl solutions of various molalities. Recorder traces were made of all readings on a strip-chart recorder to assist in interpretation and to provide a permanent record.

Other investigators have lost psychrometers buried in the soil because of corrosion and contamination. To avoid this problem, an alternate method of installation was devised and is shown in Figure 11. A tip is welded onto the slotted end of the access tube, and then the tube is pushed to the required depth. Soil air passes through the slots to the central chamber. A psychrometer fitted into the end of a $^{5}/_{16}$-in. (7.9-mm) tube is inserted into the access tube and pushed down until it meets the point and closes off the central chamber. After temperature and humidity (suction) equilibrium is achieved between the soil and the central chamber, measurement is made. Equilibrium time is 18 to 24 hours. When the measurement is completed, the psychrometer is removed and the access tube is sealed. Following are the advantages of this system:

Figure 9. Nuclear measurements system.

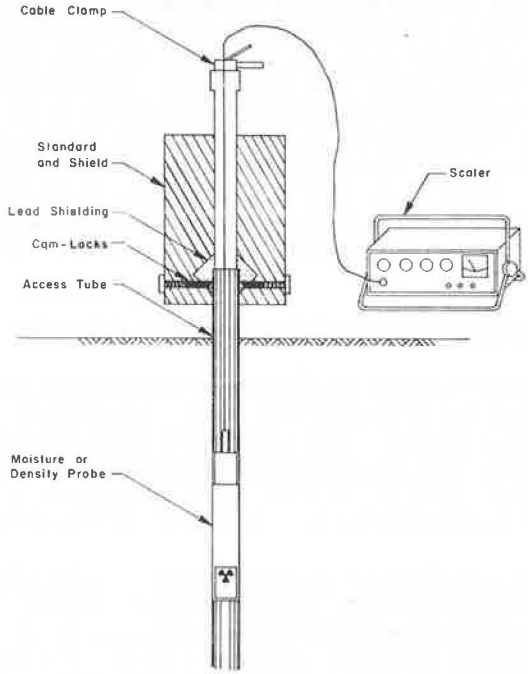



Figure 10. Comparison of nuclear measurement and soil sample water contents.

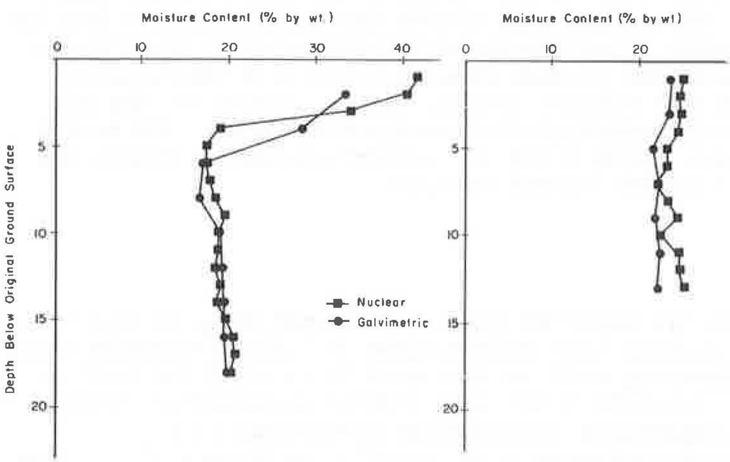
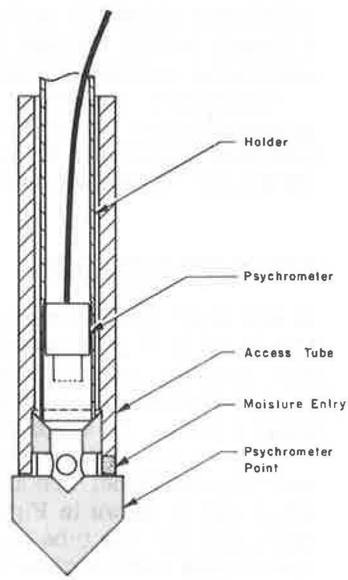



Figure 11. Psychrometer assembly.



1. There is less disturbance of in situ conditions,
2. It can be rapidly installed,
3. Psychrometer recalibration or replacement is possible in case of malfunction, and
4. Measurements can be repeated at the same point with different psychrometers.

This system has had one major problem, and it is caused by the highly developed system of fissures in the Taylor marl. When the surface is ponded, the fissures conduct water down to the tip, and then the water passes into the central chamber and access tube. This occurred only when the access tubes had tip depths down to 5 ft (1.5 m). At deeper depths, there have been no problems. No meaningful suction measurements can be made when this occurs. This problem only occurred in test areas 1 and 2 after flooding.

## HISTORY OF SITE

Site exploration was concluded in June 1973, and the test areas were laid out. During July and August, the benchmarks, some heave indicators, the nuclear probe, and psychrometer access tubes were installed. In September, the overburden was removed from test areas 1 and 2, more instrumentation was installed, and sand blankets were placed. Initial readings were started at this time.

On September 26, 1973, 6.74 in. (17.12 cm) of rain fell at the site. On October 4, 1973, 2.39 in. (6.07 cm) of rain fell, and from October 11 to 15, 1973, another 7.69 in. (19.53 cm) of rain fell. These rains flooded test areas 1 and 2 before the final instrumentation was emplaced. The water was removed, and extensive restoration work began. Instrumentation of test area 2 was finally completed in December 1973.

This series of rains disclosed several design deficiencies in the instrumentation that, for the most part, were remedied. The problem of water flow into shallow psychrometer access tubes has not been solved. The premature flooding and instrumentation problems delayed the performance of measurements but not the movement of moisture into the clay. Thus the actual boundary conditions existing during the first 10 months of the experiment can only be estimated. Planned flooding of the site was performed on August 6, 1974.

In test areas 1 and 2, the sand blanket conducted water to the clay surface but prevented its subsequent evaporation. Test area 3 was not covered with a sand blanket, and evaporation of water was not prevented. The effect of the sand blanket will be shown later, but the history must be kept in mind during the presentation of results.

## OBSERVATIONS

### Effect of Sand Blanket

The period from June 1, 1974, to August 4, 1974, was hot and dry, and there was only one occurrence of significant rainfall. The effect of the sand blanket is shown in Figure 12 by the rainfall and the heaves measured in test areas 1 and 3. In test area 1, the clay surface is covered with a sand blanket, but test area 3 is natural ground. From June 1 to August 4, 1974, extensive shrinkage cracking occurred in test area 3. On August 5 to 6, 1974, 6 in. (15 cm) of rain fell at the experimental site.

As shown in Figure 12, the heave indicators in test area 1 underwent little heave; however, all but the deepest in test area 3 show settlement because of shrinkage up until August 4, 1974. After the heavy rains on August 5 to 6, 1974, the surface heave indicators in test area 3 rose an average of 1.7 in. (4.3 cm). The heave indicator at a depth of 36 in. (91 cm) shows significant heave.

Figures 13 and 14 are water content profiles from nuclear measurements in test areas 1 and 3 respectively during this period. There is little change in the water content profile for test area 1, especially when compared with the top 3 ft (0.9 m) of the

Figure 12. Rainfall and heave test areas 1 and 3.

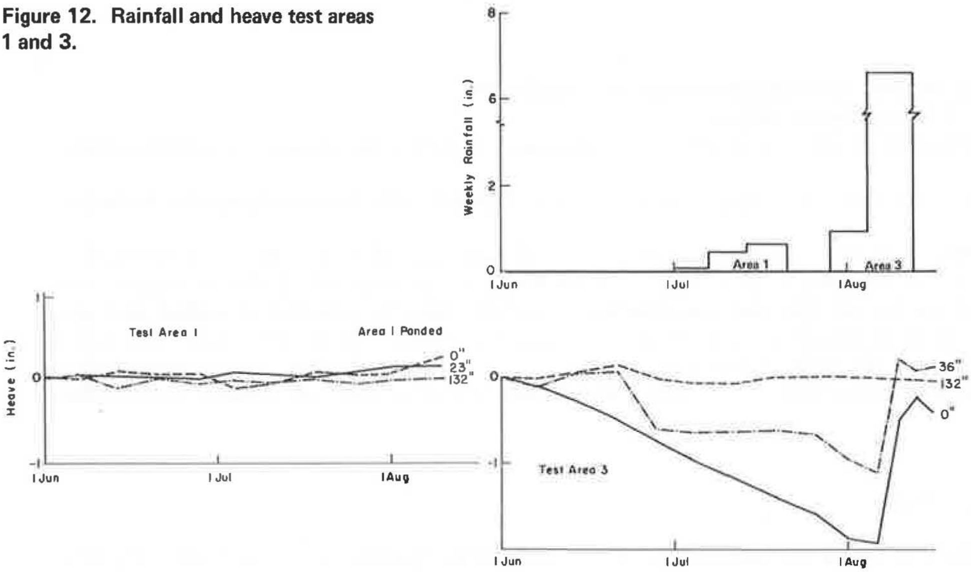



Figure 13. Water content and dry density profiles for area 1.

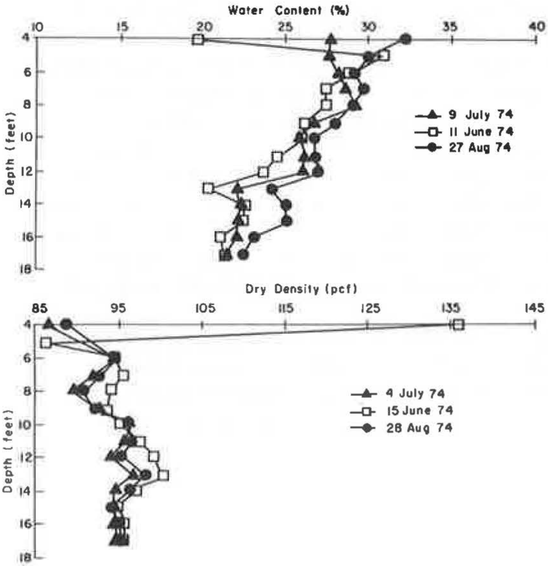



Figure 14. Water content and dry density profiles for area 3.

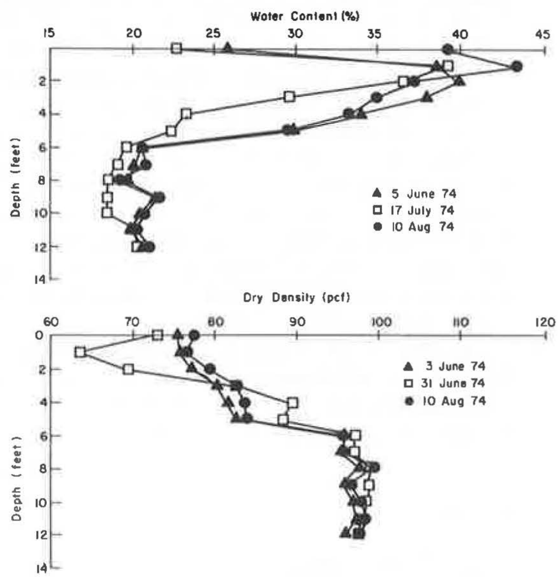



Figure 15. Suction profile for area 1.

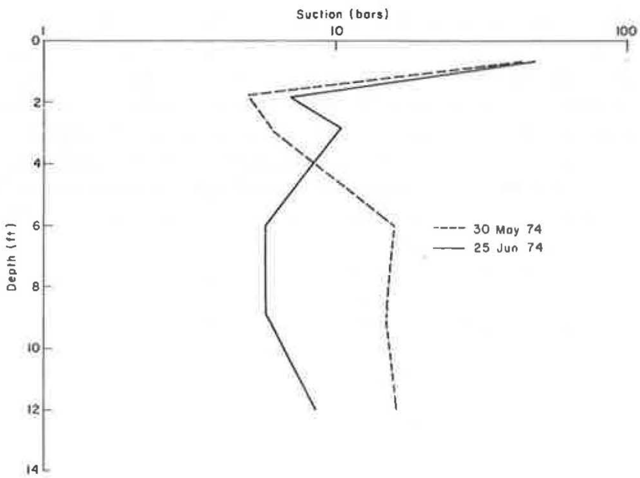
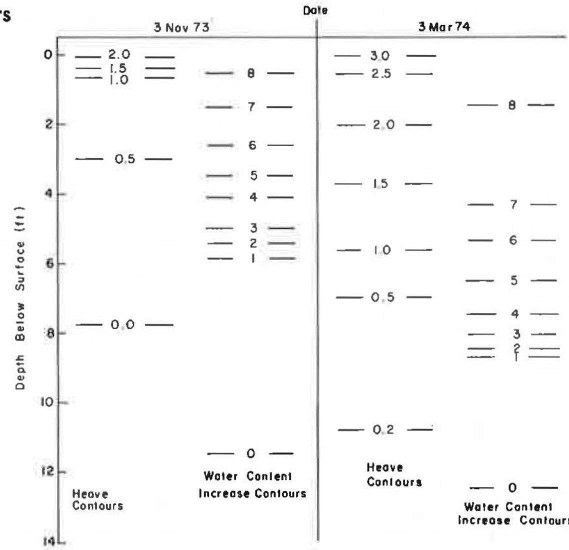


Figure 16. Heave and water content change contours for area 1.




profiles from test 3. The suction profile for test area 1 (Figure 15) also shows little change during this period. All psychrometer access tubes in test area 3 were below the zone where significant water content changes were occurring, and no significant changes in suction were observed. The sand blanket has the same effect in breaking capillary losses as the loose top layer in the agricultural technique of dry-land farming.

## Relation Between Heave and Depth of Water Infiltration

Heave contours and water content increase for November 3, 1973, and March 3, 1974, for 37 and 160 days after flooding respectively are shown in Figure 16 for test area 1. On November 3, 1973, the upper 4 to 5 ft (1.2 to 1.5 cm) of the profile show a significant increase in water content. Most of the measured heave also occurs in this zone. From November 3, 1973, to March 3, 1974, the largest water content increase is in the 4 to 7-ft (1.2 to 2.1-m) zone and, during this period, this zone is the greatest contributor to the total heave.

## Rate of Heave

After the heavy rains on August 5 to 6, 1974, the surface heave indicators in test area 3 rose an average 1.7 in. (4.3 cm) within 6 days. This corresponds well with the 2.0 in. (5.1 cm) of heave of the surface indicators in test area 1, 1 week after flooding by rains in late September 1973. The surface heave in test area 3 is less than that in test area 1 because evaporation of water started shortly after the rain. The sand blanket in test area 1 prevented evaporation from occurring.

## Effect of Trench

Surface heave in test area 2 for August 6, 1974, to October 23, 1974, is given below (1 ft = 0.3 m, 1 in. = 2.5 cm):

| Distance From Trench (ft) | Heave (in.) | Distance From Trench (ft) | Heave (in.) |
|---|---|---|---|
| 1 | 1.42 | 4 | 1.62 |
| 2 | 1.50 | 6 | 1.41 |
| 3 | 1.56 | 8 | 1.24 |

From these data it can be seen that the trench has not significantly accelerated the water entry into the soil in the vicinity of the trench. This is attributed to the highly fissured structure of the soil, which acts as a series of minitrenches. The nuclear device data from access tubes 1, 2, 3, and 12 ft (0.3, 0.6, 0.9, and 3.7 m) from the trench show differences in the order of the expected error.

## CONCLUSIONS

1. The field hydraulic conductivity greatly depends on the fissure pattern of the soil. The high rates of surface heave after the introduction of water show that the fissures rapidly conduct water to considerable depth.
2. There is considerable difference in behavior of a natural soil and that of a covered soil. In the latter, where evaporation is controlled, there is more total heave but less erratic response to climatic conditions after an initial water ingress. The covering layer need not be impermeable for these results to occur after the initial water ingress.
3. The potential heave for this test site, as predicted by the PVR method, was 4 in. (10 cm). On November 15, 1974, about 380 days after flooding, total heave in test area 1 exceeded 6 in. (15 cm). Observations indicate that the final heave may be much larger.

Measurements are continuing in test areas 1 and 2. The overburden at test area 3 was removed in October 1974; additional instrumentation was installed, and the area was subsequently flooded on October 21, 1974. This area will provide further information on the early response of the soil that was lost because of the unscheduled flooding of the other test areas.

Comparison of the experimental results with the predictive methods has only just started and will be reported on later.

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