AN EXAMINATION OF EXPANSIVE CLAY PROBLEMS IN TEXAS

by

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.

PREFACE

This report is the fifth in a series of reports from Research Project 3-8-68-118, "Study of Expansive Clays in Roadway Structural Systems," a part of the Cooperative Highway Research Program of the Center for Highway Research with the Texas Highway Department and U. S. Department of Transportation Federal Highway Administration.

Professor Hudson Matlock, Principal Investigator on this project, has been involved in providing overall guidance to the field measurements and analysis of data. Thanks are due the contact representative for the Texas Highway Department, Mr. Larry J. Buttler, and a number of people on the staff of the Center for Highway Research contributed to the information and investigations contained in this report, including W. Gordon Watt, James N. Anagnos, Raymond K. Moore, and Ramesh K. Kher, who provided technical contributions and guidance. Thanks are also expressed to those who assisted with the preparation and editing of the manuscript.

The assistance rendered by Texas Highway Department District 19 personnel John W. Livingston, Weldon R. Gibson, and John E. Betts was most helpful in development of the field study.

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LIST OF REPORTS

Report No. 118-1, "Theory of Moisture Movement in Expansive Clay" by Robert L. Lytton, presents a theoretical discussion of moisture movement in clay soil.

Report No. 118-2, "Continuum Theory of Moisture Movement and Swell in Expansive Clays" by R. Ray Nachlinger and Robert L. Lytton, presents a theoretical study of the phenomenon of expansive clay.

Report No. 118-3, "Prediction of Moisture Movement in Expansive Clay" by Robert L. Lytton and Ramesh K. Kher, uses the theoretical results of Research Reports 118-1 and 118-2 in developing one and two-dimensional computer programs for solving the concentration-dependent partial differential equation for moisture movement in expansive clay.

Report No. 118-4, "Prediction of Swelling in Expansive Clay" by Robert L. Lytton and W. Gordon Watt, uses the theoretical results presented in Research Report 118-1 and the moisture distribution computer programs of Research Report 118-3 to arrive at a method for predicting vertical swelling in one and two-dimensional soil regions.

Report No. 118-5, "An Examination of Expansive Clay Problems in Texas" by John R. Wise and W. Ronald Hudson, examines the problems of expansive clays related to highway pavements and describes a field test in progress to study the moisture-swell relationships in an expansive clay.

ABSTRACT

This report examines the problems of expansive clays related to highway pavements and small structures. It describes the economic loss due to increased and more frequent maintenance of the pavement or structure which is made necessary by the heaving of underlying soils.

The following factors that affect a soil's potential or capacity to swell are considered: moisture movement in the soil; soil properties such as permeability, density, suction, moisture, and Atterberg limits; environmental conditions such as climate, rainfall, temperature, humidity, evaporation, surface cover, abnormal local sources of moisture, and vegetation; geological aspects such as clay type, clay mineralogy, parent material, folds in geologic structure, and thickness of clay deposit.

Various remedial techniques currently being attempted in Texas are discussed along with their cost effectiveness, typical usage, practicality, and effectiveness in preventing swelling.

A field test currently in progress is described; its purpose is to study the moisture-swell relationships in an expansive clay highway subgrade by observing moisture variations and volume changes with respect to depth. A description of the measuring equipment, all data collected to date, and a plan for the continuation of the study are included.

KEY WORDS: clays, permeability, soil suction, climate, moisture migration, differential heave, ponding, lime stabilization.

SUMMARY

The status of investigations and studies of expansive clay soil problems in Texas today is presented. Various soil properties that affect the expansion of clay foundation soils are described along with environmental and geological factors that influence the amount of swelling a clay soil will experience.

Several remedial techniques currently being used to pre-swell a potentially expansive clay subgrade are described. The results of these tests are not now complete enough to determine the most suitable method, but they do indicate that the swelling of clays can be controlled by proper construction methods and remedial techniques.

This report includes a field study of a deep cut in an expansive clay soil conducted to develop instruments and to gain additional information on moisture changes and differential movement of the clay soil at several depths in the subgrade. Instrumentation installed is working properly and it appears that the use of nuclear methods for determining subsurface soil moisture and density is practical and effective, particularly for measuring "changes" from an initial condition.

Problems currently encountered with expansive clay soils shorten the service life and reduce the riding qualities of highway pavements and make it essential that an economic and effective method of preventing the uncontrolled shrinking and swelling of expansive clay foundation soils be developed. This report is an initial step toward that goal.

IMPLEMENTATION STATEMENT

The field tests and literature studies discussed in this report show that the problems caused by expansive clay foundation materials are both costly and difficult to control. The service life and smoothness of highway pavements are often reduced by the swelling of their clay foundation soils. The ability to predict how much an expansive clay subgrade will swell, or if it will swell at all, is quite limited. It is important that an economical and effective method for preventing damage due to expansive clay soils be found.

This report is primarily a background study, and as such its direct implementation lies in providing a more complete knowledge of expansive clay soils and in showing the magnitude of damage caused by swelling clays. The highway engineer is presented with a most complex and difficult problem when expansive clay soil is found in a subgrade. This report, which brings together a large volume of existing information and data on expansive clay, can serve as a basis for further development in the project.

The field study described here should be continued, in order to provide more complete and documented data. In doing so, the preliminary work of installing the instrumentation and taking initial readings will be implemented and continued.

Studies of techniques to be used prior to the construction of pavement to prevent damage which might be caused by expansive clays are most important. Two of them in which the Texas Highway Department is currently involved concern methods to increase the moisture content of a clay subgrade. District 19 is using the dry land farming method described in this report, and District 15 is ponding water on the subgrade for a period of time. These studies should be continued and similar and more extensive ones should be undertaken. This report is only a beginning toward the solution of the expansive clay problem.

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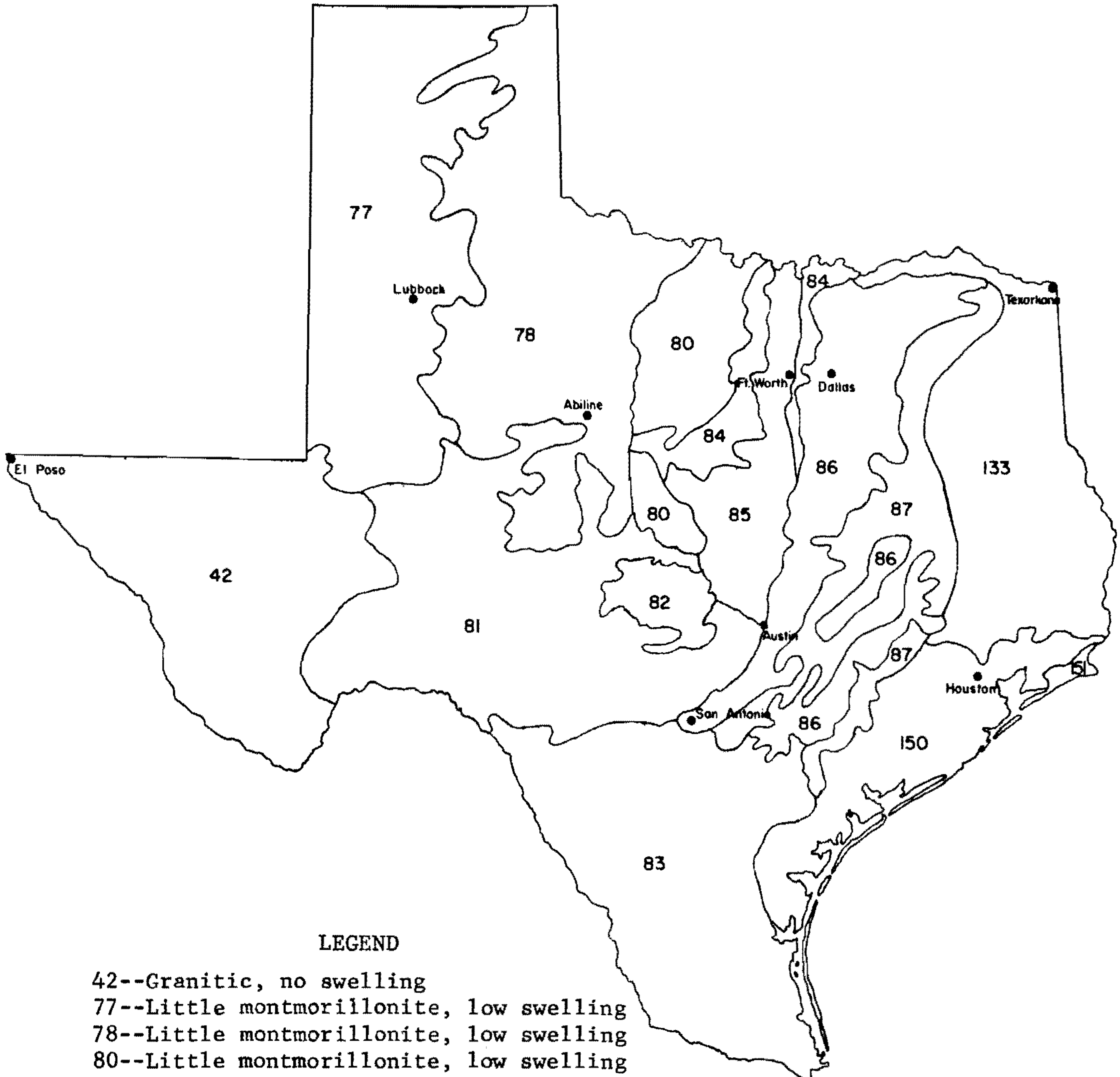
CHAPTER 1. INTRODUCTION

The distortion and cracking of highway pavements and buildings which are caused by the swelling or shrinking of expansive clay foundation soils create major engineering problems in Texas, the great plains and western states, and many other areas of the world. Expansive clays create problems involving the service life and riding qualities of highway pavements in areas where unsaturated clay soils and nonuniform rainfall occur. Clay soils with the potential to swell are found in almost any area of the world, but it is in semi-arid regions of the tropical and temperate climate zones that the environmental conditions are most conducive to the development of the problem. The presence in Texas of thick layers of expansive clay soils and a wide range of climatic conditions combine to produce some extreme cases of damage and economic loss.

The types of clay and geological formations found in Texas are shown in Fig 1.1. Although expansive clay soils containing montmorillonite and illite are found throughout the state, much of the damage caused by expansive clays occurs in the areas numbered 85, 86, 87, and 150. Expansive clay problems in other parts of the state are not as severe because the expansive clay material is not as common, weather conditions are less severe, or the clay occurs in a relatively thin layer. Thick layers of montmorillonite and illite clay soils are nearly always present when swelling of the foundation material is extensive enough to cause damage to pavements or building structures.

An expansive clay soil does not change in volume unless it undergoes a change in moisture content, and therefore, it is of prime importance to know how moisture moves in clay soil and how soil properties and climatic conditions affect the speed of moisture flow. Permeability and soil suction are the important soil properties affecting moisture movement. Climatic conditions such as rainfall, temperature, humidity, and evaporation influence the availability of moisture in the clay soil.

Presently, there is no adequate solution to the problem of how to prevent swelling in foundation soils. However, in many cases, excessive movement of



LEGEND

42--Granitic, no swelling
77--Little montmorillonite, low swelling
78--Little montmorillonite, low swelling
80--Little montmorillonite, low swelling
81--Thin montmorillonite, low swelling
82--Granitic, no swelling
83--Little montmorillonite, low swelling
84--Little montmorillonite, low swelling
85--Montmorillonite, medium swelling
86--Montmorillonite, high swelling
87--Montmorillonite, high swelling
133--Little montmorillonite, low swelling
150--Montmorillonite, high swelling
151--Gulf coast marsh, no swelling

Fig 1.1. Geological formations in Texas (from W. T. Carter, "The Soils of Texas," Texas Agricultural Experiment Station Bulletin No. 431, 1951.)

a pavement could have been prevented or at least minimized if proper construction methods, remedial techniques, and chemical stabilization of the soil had been used. The main objective of remedial techniques is to allow the subgrade soil to reach a condition of moisture equilibrium and then prevent rainfall moisture from migrating into the drier clay strata below to cause additional swelling. This may be accomplished by ponding water on the surface of the subgrade for a period of time and following it with lime stabilization, which restores strength to the wet surface clay and results in a relatively impervious barrier to moisture by reducing the activity of the clay.

In Texas, some half dozen different techniques to prevent swelling of subgrades are presently being used with varying degrees of success. These techniques are expensive, and a way is needed to predict where localized swelling will take place, so that it can be controlled economically. When a section of highway has a potentially expansive subgrade soil, damage from swelling will probably occur in only a few localized areas, thus creating roughness in a pavement.

Repairing a section of completed pavement which has heaved is expensive. The Texas Highway Department is currently using several methods for such repair: (1) mud-jacking surrounding pavement to smooth out a bump by drilling a series of holes through the pavement and pressure grouting a concrete slurry into the subgrade to raise the depressions; (2) heater-planing an asphalt surface to cut off the high spots or bumps; and (3) resurfacing or replacing sections of the pavement. It should be observed that none of these repair methods does anything to prevent the subgrade from further swelling; in fact, the mud-jacking technique adds water, which can cause further swelling of the soil.

The economic loss due to increased and more frequent maintenance required by the uncontrolled swelling and shrinking of foundation clay soils is enormous. In the 1968 fiscal year, the Texas Highway Department spent $75,700,000 for all types of maintenance, and $23,100,000 of it was spent on pavement base and surface maintenance of 60,900 miles of highways. These totals do not include major resurfacing or replacement of pavement, operations which were accomplished under separate contracts. Based on these and other maintenance costs, various research studies, and discussions with Texas Highway Department personnel, it can be seen that the total cost of pavement maintenance in Texas due to expansive clay soils may run from six to ten million dollars a year.

This report describes expansive clay problems in some detail. Various soil properties that affect the swelling of clay foundation soils are discussed along with several other factors that influence the swelling of clay soils. The remedial techniques currently being tried in Texas are described along with their cost, typical usage, and their effectiveness in preventing swelling. Included are details of a field test project currently in progress, test data collected to date, and a plan for the continuation of the study.

CHAPTER 2. DESCRIPTION OF THE PROBLEM

There are a great many interrelated factors which combine to determine the service life and riding quality of a highway pavement. A major subset of these factors is involved in the swelling clay problem. The following descriptions of the factors are brief and are intended only to define the factor in its relation to the swelling clay problem. The factors described in this chapter include differential movement; moisture movement in clays; soil properties such as permeability, soil density, and soil suction; environmental factors such as climate, vegetation, and localized conditions; and geological factors such as clay types and mineralogy. These factors combine to present the highway pavement designer with a difficult problem which must be solved if he is to provide adequate pavements at reasonable cost.

DAMAGE CAUSED BY DIFFERENTIAL MOVEMENT IN EXPANSIVE SOILS

There are several principal forms of swelling soil damage to highway pavements:

(1) A series of waves or unevenness which occurs along a stretch of pavement, usually without visible surface cracking or serious reduction in subgrade strength. Unlike frost heaving found in colder climates, which occurs in an annual cycle, these waves continue to develop until moisture equilibrium in the soil is reached and then remain practically unchanged (see Fig 2.1a).

(2) Longitudinal cracks which run parallel to the center line of the pavement. These develop because the sealing of the surface by the pavement causes the subgrade soil beneath the centerline to become increasingly wet from moisture accumulation while the moisture content of the soil in the shoulder fluctuates seasonally (see Fig 2.1b).

(3) Severe transverse cracking which may occur anywhere on a pavement where there are discontinuities in the subgrade conditions, such as at culverts, bridges, places where trees are planted too close to a pavement, or transitions from cut to fill (see Fig 2.1c).

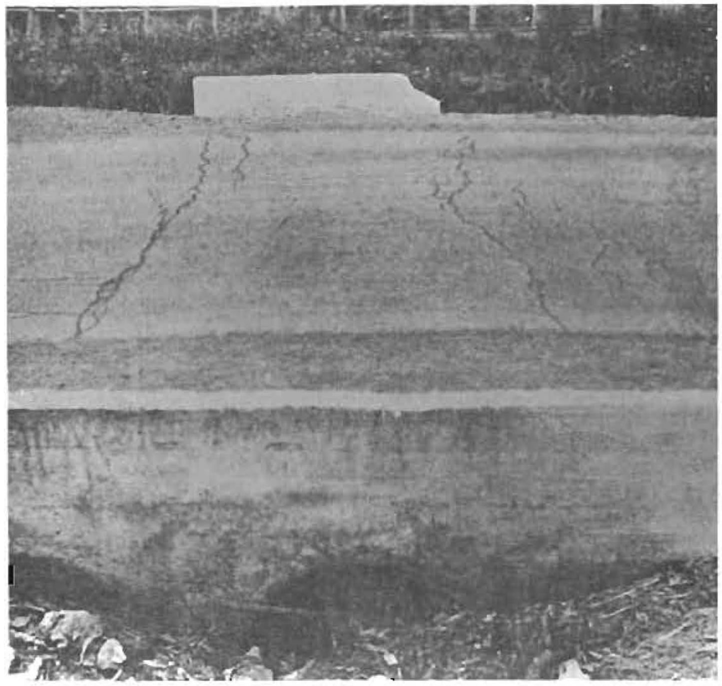
(4) Localized failure of the pavement caused by decrease in strength and bearing capacity of the subgrade (see Fig 2.1d).

(a) Series of waves or bumps.

(b) Longitudinal cracks.

Fig 2.1. Damage caused by expansive clays.

(Continued)



(c) Transverse cracks from local water source.

(d) Pavement failure due to insufficient bearing capacity.

Fig 2.1. (Continued).

Damage to highway pavements from either the first or third form is by far the most common. Combined, these two make up the most important factor in reducing the riding quality of highway pavements. Bumps in the pavement can occur in groups, in cycles, or at random, and they vary in height about 12 inches and can seriously affect driving safety, especially on high-speed highways. The heaving of a subgrade would not be a significant problem for either highway or building construction if the entire subgrade or foundation moved uniformly. The problems are primarily created by differential vertical movement, the cause of bumps in highway pavements and severe cracking in buildings and bridges.

MOISTURE MOVEMENT IN CLAYS

Moisture movement in a clay soil is the direct cause of swelling. When moisture migrates into one of the more active clay soils, the soil increases in volume and the surface will heave vertically. Unless a clay soil undergoes an increase in moisture content, it will not swell. The theory of moisture movement in a soil is complex, mainly due to the complexity of the soil system itself.

Moisture movement is generally divided or classified as either saturated or partially saturated flow, the flow mechanics being quite different in the two cases. Moisture flow in saturated soil takes place mainly in the large soil pores; in unsaturated soils the large pores are filled with air and the moisture moves primarily into the smaller capillary pores. The moisture content of saturated soil does not normally vary enough to cause serious swelling problems.

A soil-water system tends to reach an equilibrium between the soil, water, and air. Any disruption of this equilibrium will cause a movement of moisture and a corresponding change in volume until equilibrium is again reached. Rain, drought, humidity, wind, temperature, vegetation, and construction work often change the soil system's equilibrium, mainly near the surface.

Although moisture movement in clay is a very complex phenomenon, most of the theoretical aspects are rather well-known. Moisture movement in clay can be classified as hydrostatic, capillary, vapor, or osmotic flow. Each is discussed separately here, but they cannot be separated in the natural state (Ref 28).

Hydrostatic flow is caused mainly by the forces of gravity. When water is introduced at the surface of a soil, by rainfall or some other means, it is moved down through the soil by gravitational force. During irrigation or ponding hydrostatic flow is increased as the layer of water increases the pressure, causing the moisture to go deeper into the soil.

Capillary flow is moisture movement in the liquid phase caused by the surface tension forces that exist between water and soil. Water will migrate vertically downward in the capillary pores of a soil which is in contact with a source of free water. This downward migration of water will continue until the surface tension forces between the water and the soil are balanced. In addition, moisture in a clay moves upward to the surface, where it evaporates and dries out the clay. When the surface is sealed by a pavement the capillary paths are blocked and deep seated moisture will move upward and collect beneath the surface.

Vapor flow contributes greatly to the movement of moisture in soil but the full extent of its influence is not known at the present time. As the larger voids in a soil are filled with air, some equilibrium must exist between this soil-air and the moisture in the soil. The moisture contained in soil is evaporated into the soil-air and condensed out of it so that a state of equilibrium is continually maintained. If moist air reaches cooler soil, water will condense and cause an increase in water content of the cooler region (Ref 13).

Osmotic flow tends to occur between aqueous solutions having different solute concentrations. Due to the nature of the water adsorbed on clay particles (it contains a quantity of adsorbed cations which exactly balances the negative charge of the clay) the adsorbed phase will have a specific solute concentration (Ref 8). The clay type, the nature of the adsorbed cations, their position in the water structure, and other unknown factors determine the solute concentration in a soil (Ref 13).

Soil particles are negatively charged due to their molecular structure while the surrounding free moisture film is positively charged. However, since the soil particles are not free to move, the positively charged water moves towards the negative charge when there is an electrical potential difference in a clay soil.

SOIL PROPERTIES AFFECTING SWELLING

Permeability

Permeability is perhaps one of the more important factors affecting the rate of moisture movement in a clay soil. Lab tests have been developed to measure the saturated permeability of a solid soil sample in the laboratory, but the presence of extensive cracking and fissures considerably complicates the problem. In fissured and shattered soils, the primary movement of moisture is along the fissures and cracks (Ref 6). The greater the permeability of the soil, the more rapidly available moisture moves down into the soil and affects the rate of surface heave. It also seems likely that differences in permeabilities and cracking across an area will cause differential rates of heave, although the ultimate total heave may be the same. A more permeable soil is likely to have greater differential movements because of accumulation of water than a similar soil of lower permeability. The surface moisture will penetrate a permeable soil more rapidly and cause swelling to a considerable depth, while in a less permeable soil the moisture will move slowly and the source of surface moisture such as rainfall may have disappeared before penetration to any great depth. Thus permeability and the extent and depth of cracking of the soil are very important in determining not only the rate of heave of a soil but also the depth of heave and consequently the extent of differential movement.

Soil Density

There is a definite interrelation between soil density and permeability for fine-grained soils. A reduction in density will increase the permeability and porosity of a soil, causing a more rapid infiltration of moisture and a corresponding increase in vertical heave, which in some cases may equal or exceed that of a similar but denser clay.

Several research studies (Ref 17) have shown that a dense clay soil will heave more than a similar but looser soil subjected to the same conditions. As a soil increases in moisture content, the soil particles begin to increase in volume. If the soil is loosely packed the volume increase of the soil particles will fill the voids instead of causing vertical expansion. In a denser soil the volume change cannot be accommodated by a reduction of the void space and vertical heaving occurs. The density effect partly explains the frequent occurrence of differential heave at the transitions between cut and fill on a highway subgrade. A more densely compacted soil may swell more than the natural subgrade material in a cut area and thus cause differential heaving.

Soil Suction

The concept of soil suction is complex. Only a brief discussion is presented in this report. More complete descriptions of suction are presented by Lytton (Ref 21), Kassiff, Livneh, and Wiseman (Ref 18), Gardner (Ref 12), Croney and Coleman (Ref 5), and a review panel (Ref 31). Suction can be described as a soil property which indicates the tendency of a soil to attract water, or as the "thirst of the soil." It is normally defined as a negative gage pressure and is expressed in inches or centimeters of water.

The accepted definitions for soil suction, its components, and the different potentials which make up the total potential of soil water are given on page 9 of Ref 31. The main components of the total potential of the soil-water system are

(1) osmotic (or solute) potential,
(2) gravitational potential,
(3) matrix or capillary potential,
(4) potential due to external gas pressure, and
(5) potential caused by overburden pressure.

Depending on the conditions found in the field, some of the components of the total potential of soil water can be neglected. In a saturated soil below the free water table, the soil-water potential is that of free water and is given by the gravitational potential. In this particular case, a linear increase in water pressure is obtained with depth. The potential due to external gas pressure need be considered only when the gas pressure is greatly different from atmospheric pressure. Overburden pressure need not be considered for the light loading of a highway pavement unless the clay soil is very near saturation.

For highway pavement subgrades, the total soil suction is considered to be the sum of the matrix suction and the osmotic (or solute) suction. Osmotic suction is related to the free ion concentration of the soil water and is defined as the negative gage pressure which will hold pure water in equilibrium with soil water through a membrane which allows only water molecules to pass.

Matrix suction is a decrease in potential as a result of the interaction between fluid and solid or as a result of capillary forces. It is defined as the negative gage pressure which will hold soil water in equilibrium through a porous membrane with the same soil water within a sample of soil.

Variations in suction in a soil lead directly to moisture migration from areas of low suction to areas of high suction. Moisture is redistributed until a new state of suction equilibrium is reached, but the rate may be slow. Two different soils in contact may be in suction equilibrium even though their moisture contents may differ (see Fig 2.2a). It is also possible for two portions of the same soil to be in suction equilibrium but have different moisture contents. This occurs when one soil portion is becoming wetter while an adjacent portion is drying (see Fig 2.2b). A good knowledge of suction conditions will greatly aid a soils engineer to predict potential and direction of moisture flow in a clay soil.

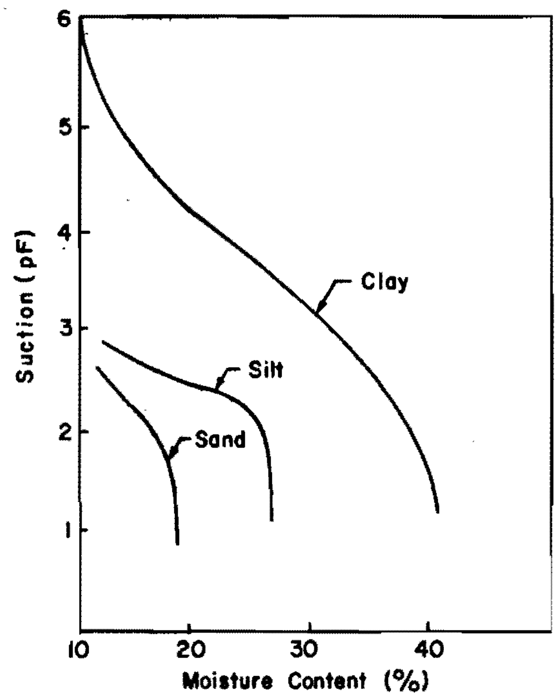
ENVIRONMENTAL FACTORS AFFECTING SWELLING

After a pavement or structure has been constructed, it is primarily environmental factors and loading conditions that affect the heaving of its subgrade or foundation. Some of the more important environmental factors are rainfall, evaporation, humidity, temperature, cover, and vegetation.

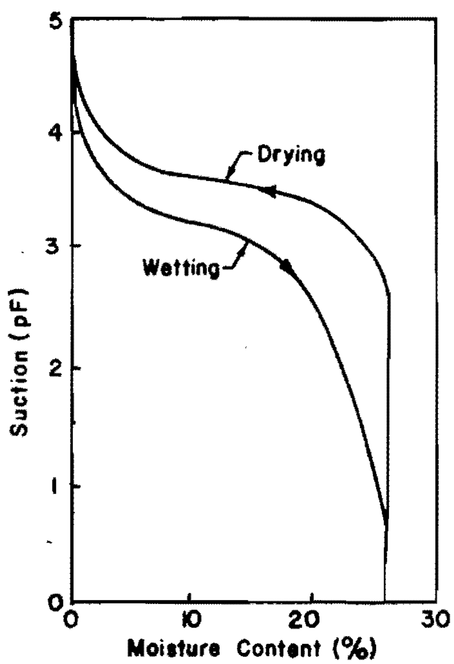
Climate

Because heaving is directly related to an increase in the moisture of the subgrade, climatic conditions, especially rainfall and evaporation, can greatly affect heaving.

In semi-arid regions where evaporation exceeds rainfall, the soil is dry enough that an increase in moisture content is accompanied by volume changes. The pattern of moisture variations within the soil is determined by the relationship of the period of maximum evaporation to the period of maximum rainfall. When there is a marked separation between a wet season and a hot dry season (maximum evaporation), there is also a large seasonal variation in soil moisture content. Shallow water tables can show seasonal fluctuations of several feet. In this type of climate there will be a definite seasonal fluctuation in soil moisture, especially under the outer walls of buildings and highway shoulders. There is usually little cumulative upward movement except possibly that due to the accumulation of moisture under the center of a large



(a) Relationship for clay, silt, and sand.



(b) Wetting and drying.

Fig 2.2. Soil suction versus moisture content curves (Ref 16).

building. In summer rainfall areas where the high evaporation rates coincide with most of the rainfall, seasonal variations in soil moisture content are not nearly so great and hence there tends to be more cumulative heave. During the wet season, moisture migrates to the underside of a building or pavement and is retained there during the dry winter season, when the evaporation is lower. There is usually a slight seasonal variation superimposed on the cumulative movement, mainly near the edges of the structure.

Vegetation

The effect of vegetation, including that existing prior to construction and that which grows on the shoulders after construction, is one of the more important factors which affect soil moisture, and yet it is usually neglected. If the construction of a pavement crosses a tree, or heavy brush, line, differential heave is likely to occur. Tree roots tend to dry out the soil to depths of 6 to 14 feet and if a pavement is constructed over such a region, differential heave will develop as capillary action tends to equalize the moisture content of the subgrade. Similarly, trees near pavements can cause localized settling as their roots dry the surrounding soil. Burrage (Ref 4) reported that the moisture content of a forested silty clay loam soil at a depth of 18 inches was reduced from 20 percent in June to 10 percent in September even though 4 or 5 inches of rain fell over that period. In contrast, the moisture content of an open soil area nearby fluctuated very little, remaining near or above 20 percent at all times.

Local Conditions

The sealing of a surface with a pavement or a building slab will greatly affect the equilibrium conditions of the subgrade soil's moisture content. According to DeBruijn (Ref 7), the subgrade beneath a sealed road surface will generally have a higher moisture content and a lower suction potential than an unsealed area nearby. Both the extent of cracking of a pavement and the type of cover on the shoulder area affect the equilibrium conditions of the subgrade moisture content.

An additional complication occurs when a sealed pavement surface begins to develop surface cracking. The cracks, in addition to affecting the riding quality of the surface, are an easy and direct access route for rainfall to infiltrate the subgrade. This ready supply of moisture to the subgrade will increase the rate of heaving and deterioration of the pavement.

Local sources of moisture supply can often cause serious heaving conditions. Added moisture from a broken water pipe, drain, or culvert, or the accumulation of surface water against a building wall may cause localized wetting of the soil and increased heaving in local areas. If such localized wetting occurs during a dry part of the year it will have a much more serious effect on differential movement than if it occurs during a wet season. In a desert climate where the moisture content of the soil is usually low, such localized sources of moisture cause very serious damage because they cause heave in an otherwise stable foundation soil.

GEOLOGY

A complete knowledge of the geological structure and history of soil is necessary for a good understanding of the problems associated with expansive clay soils. If a highway pavement is constructed so that the grade line cuts across the tops of a series of folds or arches in the soil stratum, the variations in soil type and history will likely cause differential movement of the subgrade. A knowledge of the stress history and thickness of deposit is also useful in more fully understanding the expansive clay problem.

Clay Types

In engineering usage, the term clay refers to a naturally occurring inorganic plastic material which is made up largely or completely of particles less than 0.005 mm.in diameter. Most clay contains two or more of the clay minerals, illite, montmorillonite, and kaolinite, and practically all contain nonclay constituents of both organic and inorganic nature.

Most clays are derived from rock by mechanical or chemical disintegration, chemical decomposition, or a combination of the two. Disintegration may result from the action of running water, wind abrasion, thawing and freezing, etc. Decomposition is associated with oxidation or hydration. The combined mechanical and chemical process is called weathering (Ref 18).

Residual clays are developed by the disintegration and decomposition of bedrock in place. A reasonable prediction of their texture can be made from a study of the type of parent rock and the environmental conditions. If the parent material is a sedimentary rock such as dolomite, hard limestone, or calcareous shale, the clay is frequently deep brown to red in color, is highly plastic, and contains significant amounts of clay minerals. If the clay has developed from igneous rocks such as basalt and other volcanic rocks, it is usually brown to black in color and contains a large amount of colloidal material.

Clays can also be formed from transported sediments. They are usually transported by slow moving water; are dark brown to dark grey in color, depending on the parent rock; and are found in thick deposits, with the colloidal content increasing with depth. This type of clay is highly plastic, relatively impervious, and may be heavily slickensided as a result of desiccation and cracking.

Clay Mineralogy

The mechanism of swelling clay can be described in terms of its mineralogy and chemical structure. Knowledge of the mechanism of swelling enables the soils engineer to make a reasonable estimate of the potential swelling of the clay sample tested. In a subgrade, however, nonhomogeneous soil conditions and variations in moisture and its availability, add complications to the estimating of swelling.

The following descriptions of the three basic clay minerals are presented to provide a better understanding of the behavior of expansive clays.

Montmorillonite Group. Montmorillonite is the most common of all clay minerals, especially in clays derived from weathering of volcanic ash. It has a lattice structure and the band between individual montmorillonite units is relatively weak and is dependent on the type of exchangeable cations. Montmorillonite clay has a high potential for swelling; when soaked some of it expands eight to ten times the original volume.

Illite Group. Illite is perhaps the most abundant of the clay minerals occurring in modern marine clays and is the dominant clay mineral in shales. The lattice structure is nonexpansive with addition of water. Its structure is similar to that of montmorillonite, but there is a pronounced replacement of aluminum ions by silicon ions. This results in a larger net negative charge than in montmorillonite, but most of it is balanced by adsorbed, nonexchangeable potassium ions. This causes the illite units to be more stable and to swell much less than montmorillonite.

Kaolinite Group. The kaolinite minerals are widespread in modern marine clays but are less abundant in these deposits than illite. Kaolinite is the most abundant constituent of most residual clays. Its ionic bonding is fairly tight and it is difficult to separate into layers. As a result kaolinite is relatively stable and shows little swell or wetting.

CHAPTER 3. POSSIBLE REMEDIES TO THE EXPANSIVE CLAY PROBLEM

The ability to predict whether or not a certain section of subgrade or foundation is likely to swell is just the first step in a complete solution to the expansive clay problem. A design or soils engineer must know what to do when he learns that he is working with a potentially expansive soil. Many different methods for controlling the expansive nature of certain clays (Ref 1) have been tried with varying degrees of success. The soils engineer should use the most efficient method, taking into consideration the environment, type of structure, and, most important of all, the need to establish the degree of foundation treatment needed for the structure to survive under estimated future moisture changes.

Many of the methods currently used to control expansive clay soils have been reviewed and can generally be sorted into the following three categories:

(1) prevention of moisture increase in the dry clay,

(2) removal of the dry soil and replacement with soil of desired and uniform moisture content, and

(3) wetting the dry soil in place to bring it to the same moisture content as adjacent soil.

Category 1 methods are perhaps the most difficult since they require either the use of some type of membrane or that the soil be made completely impermeable to water. Category 2 is perhaps the easiest for a small structure, but would be uneconomical for a large area such as a highway pavement. Category 3 may be a reasonable solution but much research remains to be done on such possible methods as ponding, water injection with a system of wells, and dry land farming.

LOCATION OF THE STRUCTURE

Obviously the easiest way to eliminate the problem of expansive clay is simply to avoid it, but clay makes up a great portion of the earth's soil and in areas where climatic conditions cause swelling in clays it may be impossible to locate a structure on nonexpansive soil. Often, though, the planned location of a building, culvert, or bridge may be changed slightly in order to take advantage of a thinner stratum of clay or to avoid a very dry clay. This is most difficult when it is a pavement that is being constructed, however, since highways connect towns and the route and amount of cut and fill are determined by terrain.

REMOVAL OF EXPANSIVE CLAY

This method, like relocation, avoids the problem rather than solving it, but it does consider the very important question of how thick a layer of expansive clay must be removed to effectively solve the problem. Naturally, the new soil should be nonexpansive; it should also act as a barrier to prevent moisture from reaching the more expansive soil below, which assumes that the new soil is cohesive and relatively impermeable. The new soil should be of near optimum moisture content, for ease of compaction, but it will probably lose some moisture to the surrounding expanding clay.

The thickness of the layer of expansive clay which must be removed is dependent on many factors such as clay type, soil structure, permeability, and climate, and there is no definitely established rule. McDowell (Ref 25) states that swelling in a highly expansive clay can occur as deep as 20 feet from the surface, but because moisture from rainfall or ponding does not usually penetrate more than 6 feet into the soil in a period of several months, this depth is often considered as the depth for active swelling. The following references list the results of various field tests showing little or no moisture change below a depth of 6 feet: Blight and DeWet (Ref 2), DeBruijn (Refs 6 and 7), McDowell (Ref 25), and Russam and Dagg (Ref 34).

MEMBRANES

Because swelling in expansive soils is caused by an increase in the soil's water content, an impervious membrane on or near the surface which prevents moisture access to the soil will prevent swelling. But if the moisture flow is upwards due to a relatively high water table, an impervious membrane will prove rather detrimental. Little research has been done with membranes and most of the information available is generally unfavorable from the economic and long-range point of view. A study in Colorado still in progress (Ref 3) has shown a considerable benefit of bituminous membranes in preventing the subgrade wetting from above. This method seems to work well when first installed, but after a period of time the moisture content of the soil begins to increase and some heaving occurs. Russam and Dagg (Ref 34) found that a polyethylene membrane covering a pavement shoulder effectively excluded light rain but was not effective against prolonged heavy rain. Rapid changes in moisture conditions in the shoulders were prevented and fairly wet conditions were maintained under the edge of the pavement. The polyethylene membrane was fairly thick and held up well over the two-year test period. There was some horizontal flow but in two years the moisture had only moved about 3 feet under the edge of the membrane. A grass covered control section experienced vigorous wetting and drying cycles which resulted in cracking of the adjacent road pavement. Newland (Ref 29) reports that a very thin (.005 inch) polyethylene film was used with good success to prevent the foundation of a house from excessive swelling. Lamb (Ref 20) reports that a membrane section tested did retard moisture intrusion into the subgrade.

LIME STABILIZATION

Lime stabilization of about the top 6 inches of highway subgrades is probably the most widely used method of controlling the swelling clay problem. Several extensive research projects have been conducted to find a material which will effectively reduce the swelling phenomenon of the expansive clay soils. An extensive literature survey was conducted on many aspects of lime-soil stabilization by Puleo (Ref 30).

Lamb (Refs 19 and 20) used the results of several research projects to determine what the most effective chemical stabilizing agent for clay soils would be. It was stated that the ideal stabilizing agent should possess migration properties, or in other words, the agent must diffuse and penetrate into the expansive clay soil; however, no chemical agent was found with migratory properties. Hydrated lime was by far the most effective stabilizing agent and stabilizes soil by replacing the very active sodium ions found in montmorillonite with less active calcium ions. The results of the tests reported by Lamb show that lime used as a stabilizing agent will reduce volumetric expansion from a range of 5 to 9 percent (depending on water content at compaction) to below 2 percent. Swell pressure will decrease from more than 1500 pounds per square foot for an untreated soil to about 500 pounds per square foot for specimens compacted at moisture contents above optimum and to about 800 pounds per square foot for moisture contents just below optimum (Ref 20).

The Texas Highway Department, along with highway builders in many other states, has been using hydrated lime to stabilize subgrades whenever clay is present. A mix of from 2 to 5 percent by weight has proved to be the most economical. As the lime will not migrate, it must be mechanically mixed into the clay soil. This is usually done with either a disc harrow or a small ripper. It has been this mixing technique that has limited the effectiveness of lime stabilization as pm;u abpit a 6-inch lift can be worked at a time. This 6-inch stabilized layer has generally been very effective in reducing swell since the lime-stabilized clay soil is stronger and has a higher bearing capacity than the natural clay. This stronger subgrade allows the engineer to design a thinner and less expensive pavement structure. The stabilized layer, in feet, acts as a membrane preventing the entry of water from the top. However, the practice has little effect on deep-seated heave. Studies by Chester McDowell (Ref 26) have also emphasized that extensive vertical movements due to swelling or heaving of deep layers of soils use not overcome by lime-treatment or any other treatment applied in thin layers. Also when the cycles of wetting and drying the stabilized layer make it to crumble and crack, the moisture entry from the top starts again and the stabilized layer loses its effectiveness.

DEEP LIME STABILIZATION

Deep lime treatment, to a depth of about 2 feet, was recently used by the Oklahoma Department of Highways to stabilize expansive clay soils (see Ref 15). The final results on several projects where this method was used extensively are not yet available but tests and indications to date are encouraging.

Large tractors equipped with special attachments on heavy ripper blades were used to mix the lime and soil to a depth of about 2 feet. According to Hartronft et al (Ref 15), such tractors should be rated at 200 horsepower or larger, weigh 30,000 pounds, and have a 52,000-pound drawbar pull in low gear. A 2 percent (by weight) mixture of lime was used and may be applied either dry or in a water slurry. The lime was applied after first plowing the subgrade. Several covers to a 2-foot depth with heavy rippers were necessary to adequately mix the lime and soil.

Data obtained from the Oklahoma Department of Highways shows that the deep lime stabilization method costs \$100,737 per two-lane mile. This indicates a savings of \$12,582 per two-lane mile when compared with a design using a 5 percent stabilization of only a 6-inch layer of subgrade. The initial findings by the Oklahoma Department of Highways (Ref 15) also show the following approximate changes in soil properties when comparing the untreated soil with stabilized soils; 50 percent reduction in the plastic index, 25 percent reduction in the liquid limit, 50 percent reduction in the volume change, and a 300 percent increase in CBR values.

The reduction in costs and the improved soil properties indicate that this technique may prove to be a major advance in the ability to control expansive clay soils in highway subgrades. Continued testing and use of this method is planned by the Oklahoma Department of Highways. District 2 (Ft. Worth) of the Texas Highway Department also plans to use the deep lime method for subgrade stabilization in the near future.

DRY LAND FARMING

This technique has been used by farmers for many years to stabilize water content of soil. The surface of the ground is plowed or disced at regular intervals. This breaks the capillary flow of moisture vertically and tends to prevent the loss of any significant amount of moisture. It also causes deep-seated moisture to move upward.

Rain will partially re-establish the capillary flow and thus soak into the soil normally. Therefore, after a heavy rain, usually as soon as the ground surface has dried, the area should be replowed to again break the capillary flow to the surface. If the plowing is done often enough and, especially, at the right time, the ground should lose no moisture because of evaporation but should slowly increase in moisture content if there is sufficient rainfall (Ref 30).

The main disadvantage to this method of controlling expansive clays is the long period of time (usually six to twelve months) necessary for the soil to reach its optimum moisture condition (see Ref 32).

The discing operations involved with this method of controlling expansive clays are usually relatively inexpensive if the cost of delaying construction of the highway for six to twelve months is not included. This method is

Water was applied once, at the beginning of a test, and moisture readings were taken for a week. In flooding tests with Laredo clay, water penetrated 4 or 5 feet in five days.

McDowell (Ref 22) reported remedial measures used on the foundation of a building. Extreme measures were called for as calculations indicated a "potential vertical rise" of 6 inches. In addition to several other remedial measures, the basement soil was ponded for 30 days. The application of lime to the very wet and mushy subgrade after completion of ponding permitted hauling of heavy loads in three days.

Felt (Ref 9) reported an extensive research project that used ponding to pre-wet expansive clay soils. A number of areas of localized dry clay were located on the proposed route of U. S. Highway 81. Several of these dry areas ponded for six months swelled as much as 5 to 6 inches. The soil moisture contents increased from an average of about 13 percent to an average of 19 percent in the top 7-1/2 feet of soil. The increases in moisture contents were greater near the surface with only very slight increases occurring below 7 or 8 feet.

A ponding project is currently being conducted in District 15 (San Antonio, Texas Highway Department) on a section of U. S. 90. The soil in this area is a highly plastic, montmorillonite clay. Instrumentation for measuring heave at various depths and tubes for taking subsurface moisture and density determinations with a neutron counter have been installed.

WATER INJECTION IN CLAY WITH A SYSTEM OF WELLS

Some research has been carried out to find a method to accelerate the flow of water in a clay soil. This can be done best by reducing the length of the maximum flow path, and the most obvious way is to construct a grid system of vertical wells or drains to accelerate the vertical entry of water. The water is given rapid entry deep into the clay where it can migrate laterally along a shorter flow path. As the spacing of the wells should probably be 10 feet or less, the cost of such a system of many drilled wells will be very high.

The results of pre-construction flooding in conjunction with a grid of vertical wells was reported by Blight and DeWet (Ref 2). A grid of 4-inch-diameter wells 20 feet deep was used, with a spacing of 10 feet. At the conclusion of 96 days of ponding, 90 percent of the maximum surface heave had probably one of the most natural and best since it allows time for the subgrade soil to reach the natural or desired moisture content and, hopefully, to do all the possible swelling before the pavement is constructed. The long period of time should also eliminate most of the differential heaving caused by soils that are either very dry in a localized area or are nonhomogeneous in their swelling potential.

This method is currently being used in Texas Highway Department District 19 on a 5-mile length of Interstate Highway 30 near New Boston, Texas. A section of the project has been instrumented for measuring differential vertical movement at various depths and for in situ moisture contents and wet densities. A complete description of the test site along with all data taken to date is included in Chapter 4 of this report. District 19 personnel had planned to continue the discing operations for a year but a delay in beginning caused by several months of bad weather may decrease the time available.

PONDING WITH WATER

Ponding with water has been used in Texas since 1934 to wet a natural, undisturbed, dry clay soil and thereby pre-swell the soil prior to construction of the pavement or structure (Ref 33). Ponding has had rather limited use due to its cost, length of time required to produce satisfactory results, and the mushy, wet subgrade surface which results. It may take one to three months ponding to produce satisfactory results and therefore seems economically justifiable only when extremely high volume changes of the subgrade are predicted. The loss in subgrade support caused by the mushy surface after ponding can quickly and easily be replaced by the use of lime stabilization.

The time required to satisfactorily wet a clay soil is long because of the slow migration of water down into the relatively impermeable clay soil. Extensive root structure or deep shrinkage cracks will, in many cases, greatly increase the flow of water into a clay soil.

An extensive ponding project was conducted in Guadalupe County (District 15, Texas Highway Department) from 1934 to 1938 (Ref 33). The report of it contains extensive data on a large ponding project for a section of U. S. Highway 90 near Seguin, Texas. Several years of observations of heave and moisture content along with all maintenance records are included.

Rockwell (Ref 30) reported on irrigation tests where water was applied to four types of soil by flooding or in furrows of various dimensions and spacing.

taken place. The test area was then covered with a concrete slab. The slab continued heaving very slowly for the next 18 months, probably because the wells were still full of water when the slab was constructed. The slab then started a very slow settlement which was still continuing after 7-1/2 years. Excellent results in pre-swelling the soil were obtained, but since no control section with ponding only was used, the effects of the well system cannot be isolated.

Felt (Ref 9) reported on a research project that used a system of 4-inch-diameter drilled wells to speed the flow of moisture into a clay soil. The system of wells was used (1) separately, (2) in conjunction with ponding, and (3) to pressure force water into the soil. The use of the well system only was not practical. Pressure forcing water into the drilled wells was of value, but it is very questionable whether the value would be commensurate with the added drilling and pumping expenses. Ponding, in conjunction with a system of drilled wells, did not prove to increase the rate of water migration into the soil enough to compensate for the high cost of the drilling operations.

CHAPTER 4. ATLANTA DISTRICT PILOT STUDY

Study of existing data on expansive clays has shown that few experiments have been conducted with sufficient data and soil information to verify the theory or to adequately investigate the problem. Lytton and Kher (Ref 22) and Watt (Ref 23) have developed computer programs for predicting moisture movement and swell in expansive clays. These programs are based on rather complex moisture movement and swell theories and cannot be adequately checked and proved until more complete laboratory and field data are available.

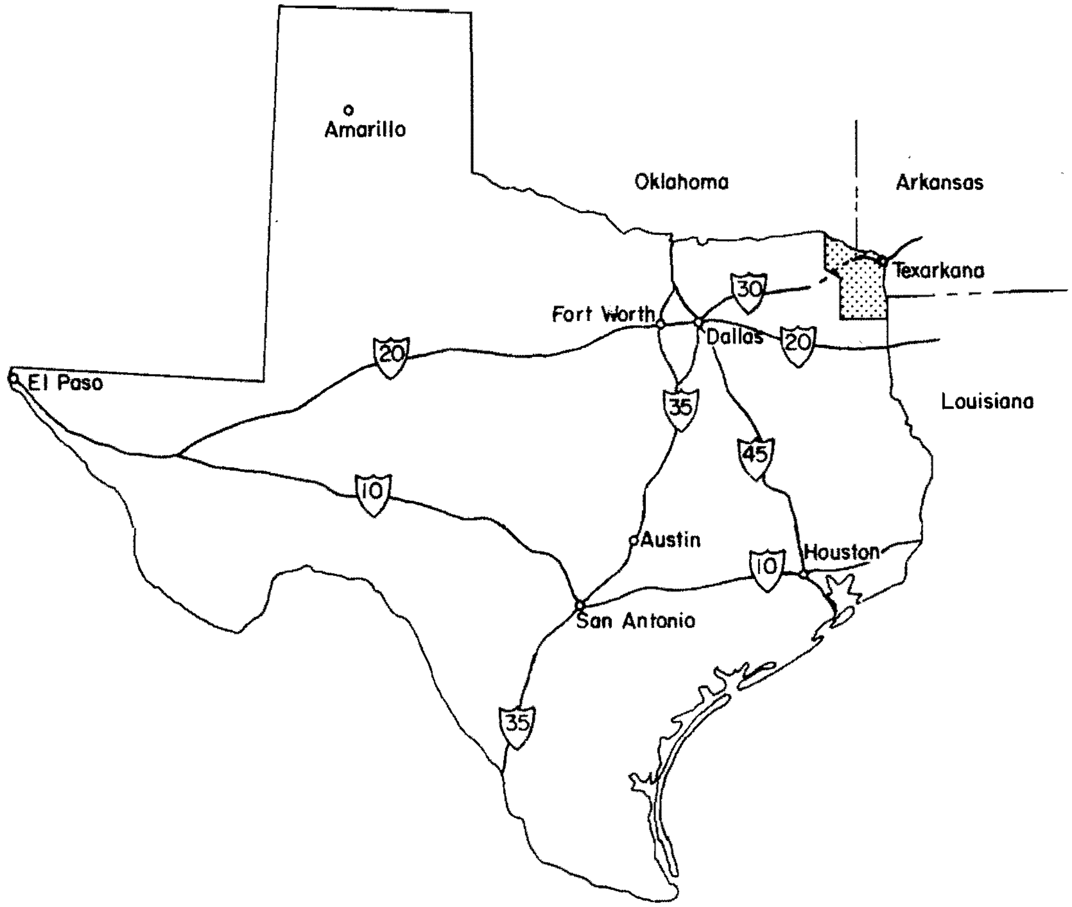
In 1968, the opportunity to initiate a field study developed in District 19 (Atlanta) of the Texas Highway Department, where a 23-foot-deep cut was to be made through an expansive clay soil. Because of previous bad experience with expansive clays in this area, attempts to allow heave to occur prior to construction of the pavement are being made. A dry land farming technique for pre-wetting and thus pre-swelling an expansive clay soil is being used for this purpose (see Chapter 3).

DESCRIPTION OF THE TEST SITE

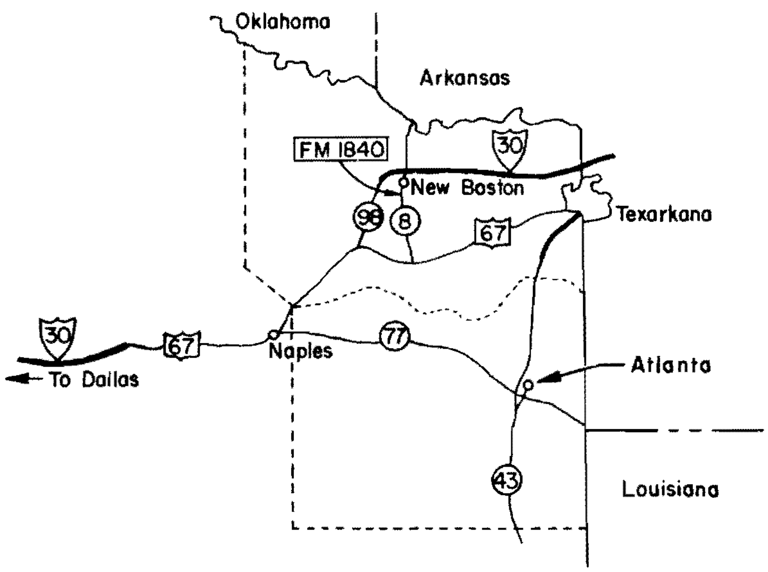
The clay cut is located in Bowie County, Texas, on a section of IH-30 which is about 10 miles west of New Boston (see Fig 4.1). The cut is about three quarters of a mile long and 400 feet wide. The depth of the cut varies but about 1900 feet of it is at least 17 feet deep with the greatest depth, 23 feet, occurring at station 1000+00 (see Fig 4.2). The cross section of station 1000+00 is shown in Fig 4.3. The only complete boring log that goes deep enough to show the underlying soil structure was taken on the centerline at station 1009+00 and gives the following soil profile:

Elevation	Soil Type
394	Original ground surface
394 - 392	Top soil
392 - 387	Sandy clay, mottled
387 - 363	Red clay

(profile continued)



(a) General location map.



(b) Vicinity map.

Fig 4.1. Location of the test site.

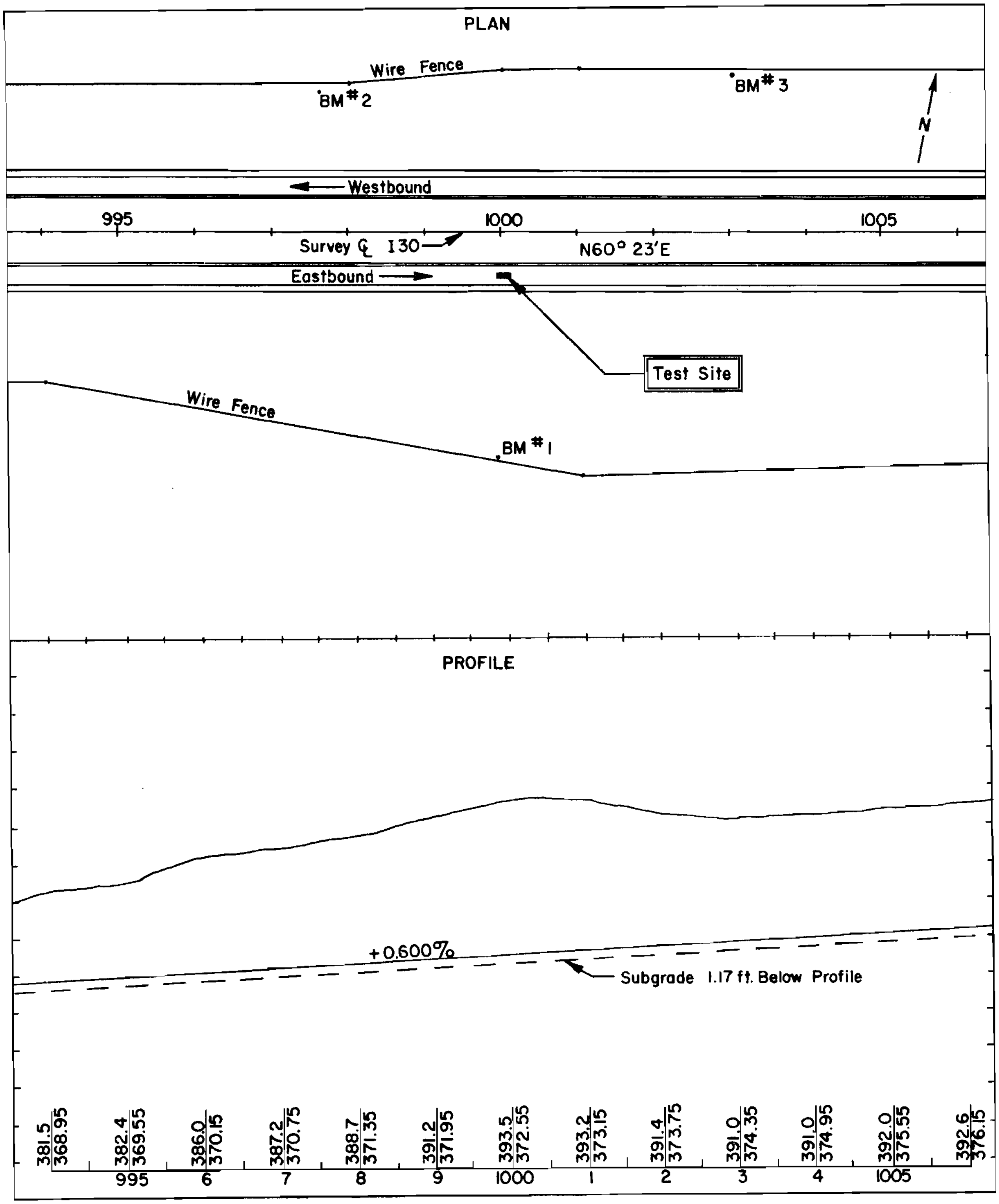


Fig 4.2. Plan-profile sheet at Station 1000+00.

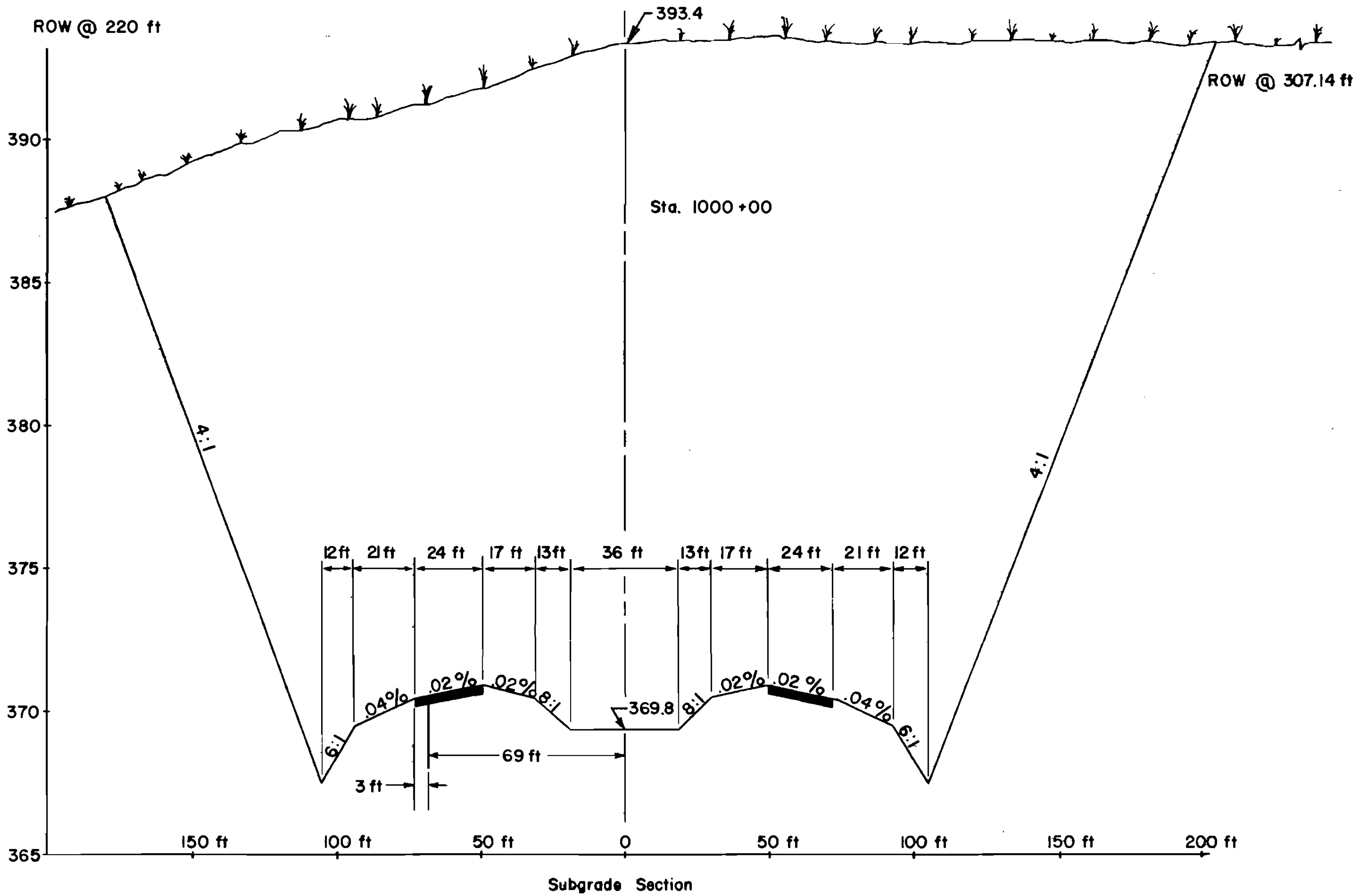


Fig 4.3. Cross section of station 1000+00.

(profile continued)

363 - 351	Yellow-grey clay
351 - 337	Yellow-grey sand
337 - 335	Gravel and clay
below 335	Laminated grey silty shale

The final subgrade at station 1009+00 will be at an elevation of 377.2 feet and the pavement subgrade will consist primarily of a 14-foot layer of red clay. Below an elevation of 377.2 feet this red clay had the following soil properties prior to the start of construction: water content of 22 percent, liquid limit of 55, plastic limit of 11, plasticity index of 34, and bar linear shrinkage of 20.0. Potential vertical rise (PVR) calculations as suggested by McDowell (Ref 24) indicate a potential for 3.5 inches of vertical heave. Core samples taken at the test site (station 1000+00, 69 ft. rt.) indicate a similar soil structure. The final subgrade elevation is 369 feet and the interface between the red clay and the yellow-grey clay occurs about ten feet below the subgrade. A chemical analysis and a clay mineralogy breakdown of the material found at the test site are given in Appendix 2.

The following is a geological description of the area: The material encountered from station 990 to station 1025 on IH-30 is a massive Pleistocene terrace deposit of the Red River, known as the Hardeman Terrace (Kansan Age). It is an extensive system that can be traced almost continuously from the High Plains to the Arkansas state line. The altitude of the terrace is 90-120 feet above the present Red River elevation. At this location the elevation is up to 400 feet, which is 100 feet above the Red River elevation.

EXPERIMENT DESIGN

The basic research objective was to study the moisture-swell relationships in an expansive clay highway subgrade. Moisture variations and volume changes with respect to depth were to be observed and daily rainfall and temperatures were to be studied also.

The location of the test site in the clay cut was selected, and devices to measure vertical movement of the soil at subgrade elevation and at several depths below it were installed before any of the overburden soil was removed. A potential vertical rise study of this soil as suggested by McDowell (Ref 24) indicated that much of the volume change would occur in the top four feet of the subgrade and almost all would occur within ten feet. Therefore, the devices to measure vertical movement, called "tell-tales" and described later in this chapter, were installed at subgrade elevation and at 4 and 10 feet below it. The initial moisture content and density of the subgrade soil to a depth of approximately 20 feet were determined. Access tubes were installed as soon as possible after removal of the overburden soil and moisture and density determinations were taken with portable nuclear equipment (the nuclear equipment is described in Appendix 1 and Chapter 4 contains a discussion of calibration methods).

Elevation readings for measuring vertical movement, along with moisture content and density determinations, will be made at intervals of about two months until after the pavement has been constructed and should be continued, at four to six-month intervals, as long as desired.

DESCRIPTION AND LOCATION OF INSTRUMENTATION

The instrumentation is located in the vicinity of station 1000+00. The clay cut before and after construction is shown in Figs 4.4 and 4.5.

The instrumentation, devices to measure elevation changes at different depths below subgrade and nuclear access tubes for moisture content and density determinations, was installed so that it will be below the 24-foot eastbound concrete pavement. Access holes will be placed in the pavement during construction to facilitate future measurements.

It was originally planned to have three complete clusters of instruments, each with a surface plate, 4-foot and 10-foot tell-tale, and nuclear tube, but installation difficulties with the tell-tales prevented this. The surface plates were installed three months before the tell-tales and installation problems caused the tell-tales to be installed 12 feet too far to the east. This presents no particular difficulties to the long-range study as the surface movement indicator will be transferred to the pavement as soon as it is placed.

Three surface plates, which were installed November 12, 1968, are located 60 feet to the right of the survey center line, in the center line of the 24-foot eastbound pavement. Three 4-foot-long and three 10-foot-long tell-tales, installed February 10-13, 1969, are located 69 feet to the right of the survey center line, about 3 feet from the outside edge of the right eastbound lane. Three nuclear access tubes were installed on August 26, 1969, each one

Fig 4.4. Clay cut with work in progress on westbound lane.

Fig 4.5. Completed clay cut. The three round paint can lids mark locations of access tubes.

located midway between a 4-foot and a 10-foot tell-tale. Locations of the instrumentation are shown in Fig 4.6.

Nuclear Access Tubes

The three aluminum access tubes, for nuclear moisture and density determinations, are 2 inches in diameter and 20 feet long. The tubing which is suitable for use with Troxler depth probes, is standard class 150 aluminum irrigation pipe with a 2.000-inch outside diameter and a 1.900-inch inside diameter. It was used because of resistance to corrosion, low cost, and ready availability.

The bottom end of each access tube was sealed with an aluminum plug and two rubber O-rings (see Fig 4.7) prior to installation in the ground. Two square grooves were machined in the plug to seat the O-rings. The plug and an inch of the inside of the access tube were given a thick coating of silastic and the plug was pushed into the tube. A heavy coat of silastic was applied to the end of the tube and plug and allowed to harden for 24 hours. Much difficulty in preventing water leakage had been reported by other researchers and it was hoped that this method would prevent the leakage, but one tube began leaking in December 1969.

The holes for the access tubes (Fig 4.8) were drilled with 2-inch continuous flight auger. It was slowly advanced into the soil, and after every foot or so was allowed to rotate a short while to clear the threads and move the soil to the surface. The auger was advanced slowly in order to let the bit cut into the clay rather than screw itself into the soil. After each 3-foot section entered the ground, that section and the adaptor used with it were unscrewed, the kelly was raised, and another 3-foot section of auger added. The hole was drilled about 8 inches deeper than required to allow for soil which fell into the hole when the auger was removed, and material dislodged from the sides of the hole as the access tube was installed. (A photograph of an installed nuclear access tube is shown in Fig 4.9).

The adaptor was used to connect the auger to the 2 1/2-inch kelly bar of a Texas Highway Department drill rig (Fig 4.8). Care was taken to be sure the threaded end of the adaptor was in perfect alignment with the kelly bar since any eccentricity or poor alignment would cause the auger to wobble and either enlarge the top of the hole or shear off the attachment bolt. An enlarged

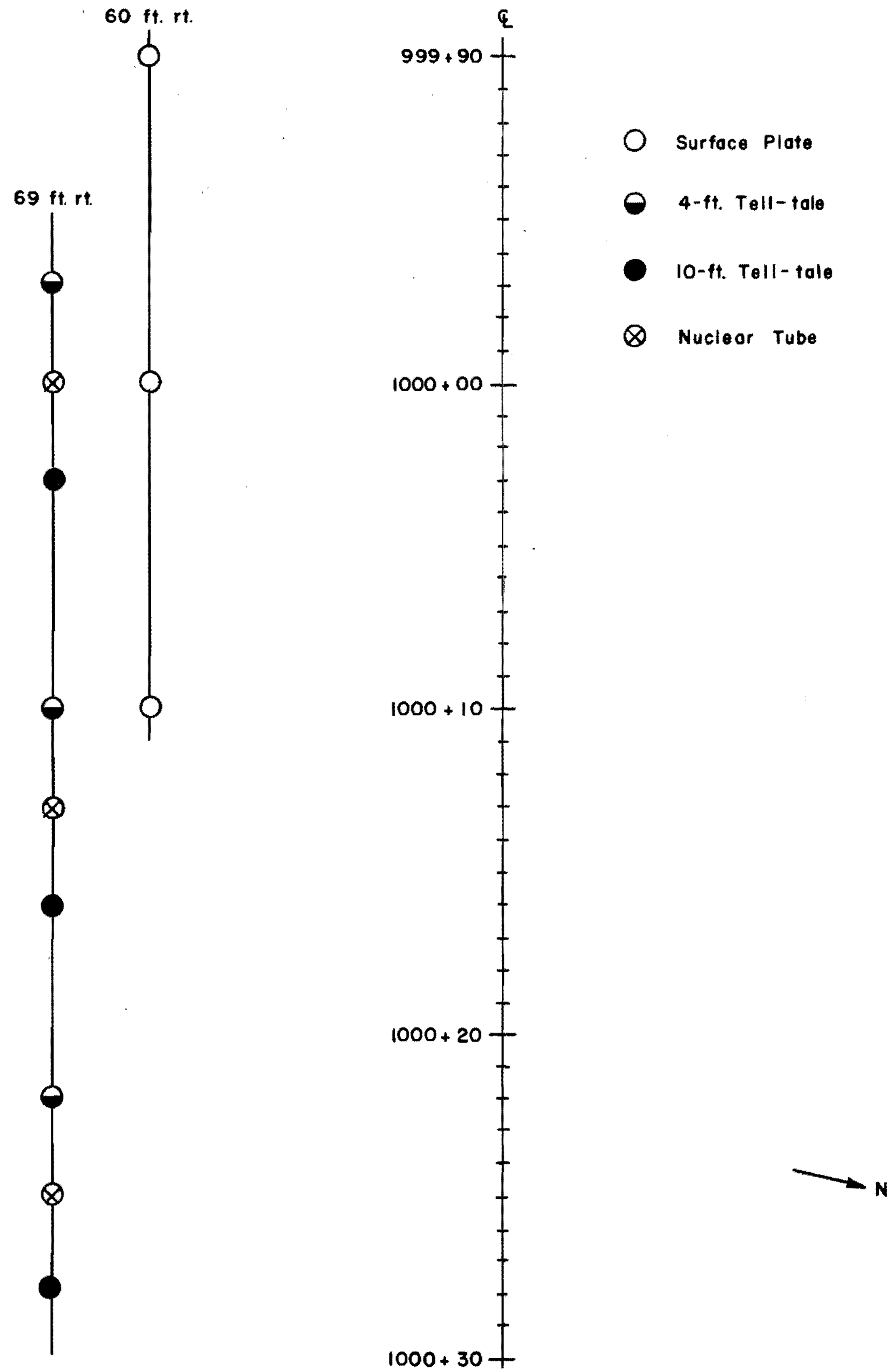


Fig 4.6. Plan view of location of instrumentation.

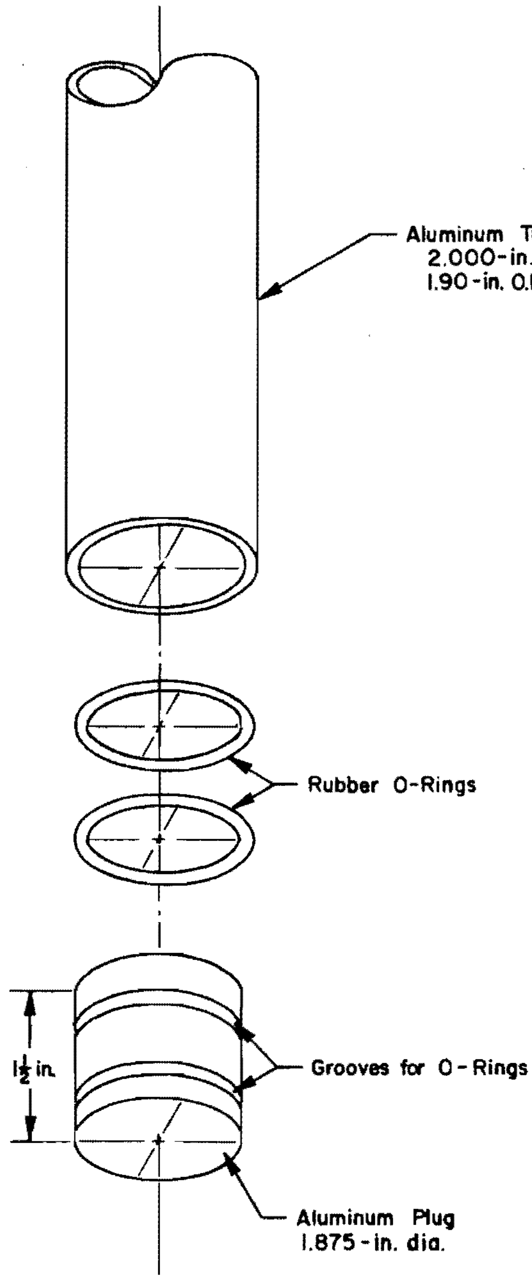


Fig 4.7. Method for sealing access tubes.

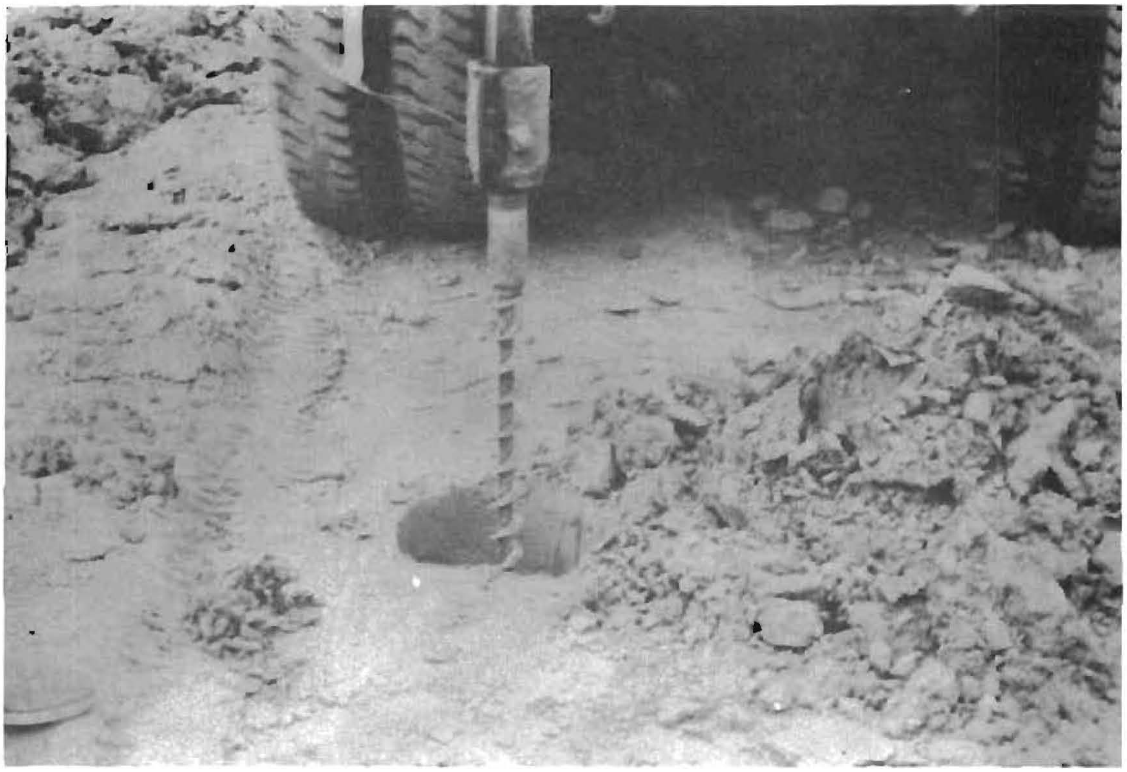


Fig 4.8. Drilling small hole for nuclear access tube.

Fig 4.9. Installed nuclear access tube.

hole would result in a loose fit for the access tubing and allow water to flow down into the hole and result in erroneous moisture and density determinations.

A number 10-1/2 rubber stopper was placed in the top of the access tube to prevent the entrance of moisture, and a small bag of desiccant was fastened to the bottom of the stopper to absorb moisture in the tube. A five-gallon paint can with the lid removed was used to cover and protect the top of the access tube.

Surface Plates

Three surface plates were installed on November 12, 1968, at the locations shown in Fig 4.6. The plates were constructed of a 10-inch diameter plate of 1-inch steel with a 6-inch length of 1-inch rod welded to them. As these were installed before any removal of overburden had started, a 30-inch-diameter hole was drilled to the desired depth (about 18 inches below final subgrade). A workman was lowered to the bottom of the hole to clean out the loose soil and prepare a solid footing for the surface plates with concrete. The plates were then installed, elevation readings taken to the top of the rod, and the 30-inch hole backfilled and compacted.

Bench Marks

Three deep bench marks to provide reference were installed at the edge of the right-of-way, two on the north side and one on the south side, as shown in Fig 4.2. Bench mark No. 1 was installed on November 6, 1968, at station 999+97, 295 feet right. Bench mark No. 2, at station 997+78, 185 feet left, and bench mark No. 3, at station 1003+02, 212 feet left, were installed on January 15, 1969. These bench marks were installed by drilling a 5-inch-diameter hole to a depth of 40 feet and inserting a 2-inch-diameter plastic water pipe with a threaded upper end. A 50-foot length of No. 5 reinforcing steel bar was put inside the plastic water pipe and driven about 2 feet into the soil. A 3-1/2-foot length of 3-inch plastic water pipe with a threaded cap on the upper end was fastened to the upper end of the 2-inch plastic water pipe.

Tell-tale Subsurface Devices

Six tell-tale devices, located as shown in Fig 4.6, were installed on February 10-13, 1969, before any removal of overburden had started. These devices were designed to measure vertical movement at different depths and were installed at both 6 feet and 12 feet below subgrade elevation.

The tell-tales were constructed as shown in Fig 4.10. Figures 4.11 and 4.12 show the tell-tale and installation. A 5-inch length of 1-inch-diameter iron pipe was welded to a 4-inch-diameter steel base plate. A length of 1/2-inch electrical conduit, varying with the depth below ground of tell-tale installation, was threaded to the base plate. This much of the assembly was the part of the tell-tale that moved up or down as the soil beneath the base plate swelled or shrank. A length of 3/4-inch electrical conduit about 6 inches shorter than the 1/2-inch conduit was slipped over the 1/2-inch conduit and coated with grease, and then a length of lay-flat, thin-wall polyethylene tubing was pulled on over the grease. The polyethylene tubing was in direct contact with the soil and insulated the center rod from soil friction. An O-ring sealed the gap between the 1-inch iron pipe on the base plate and the 3/4-inch conduit, and the space between the 1/2-inch and the 3/4-inch conduits was filled with heavy grease to prevent soil particles from penetrating and binding these parts together.

The tell-tales were installed by drilling a 30-inch-diameter hole to final subgrade elevation of approximately 23 feet and then drilling a smaller 6-inch-diameter hole to the depth at which elevation readings were desired. All loose soil was removed from the bottom of the smaller hole and several inches of concrete grout was placed on the bottom. The complete tell-tale assembly was lowered into the hole and firmly seated in the concrete footing. The soil around the tell-tale at the bottom of the 30-inch hole was carefully backfilled and compacted. During the backfilling operation the outer, 3/4-inch-diameter, pipe was temporarily held so there was about a 4-inch gap between the bottom of it and the metal base plate. This gap was necessary to give the tell-tale room to swell upward when the outer pipe was fixed by the soil. The tell-tale was allowed to stabilize for several hours and then the elevation reading was taken to the top of the inner, 1/2-inch, pipe. Finally, the 30-inch-diameter hole was backfilled and compacted to ground level.

FIELD TESTING PROCEDURE

Elevation readings for the surface plates and tell-tales were taken the same day that nuclear moisture and density determinations were made, to allow a correlation between volume change and variations in moisture content of the

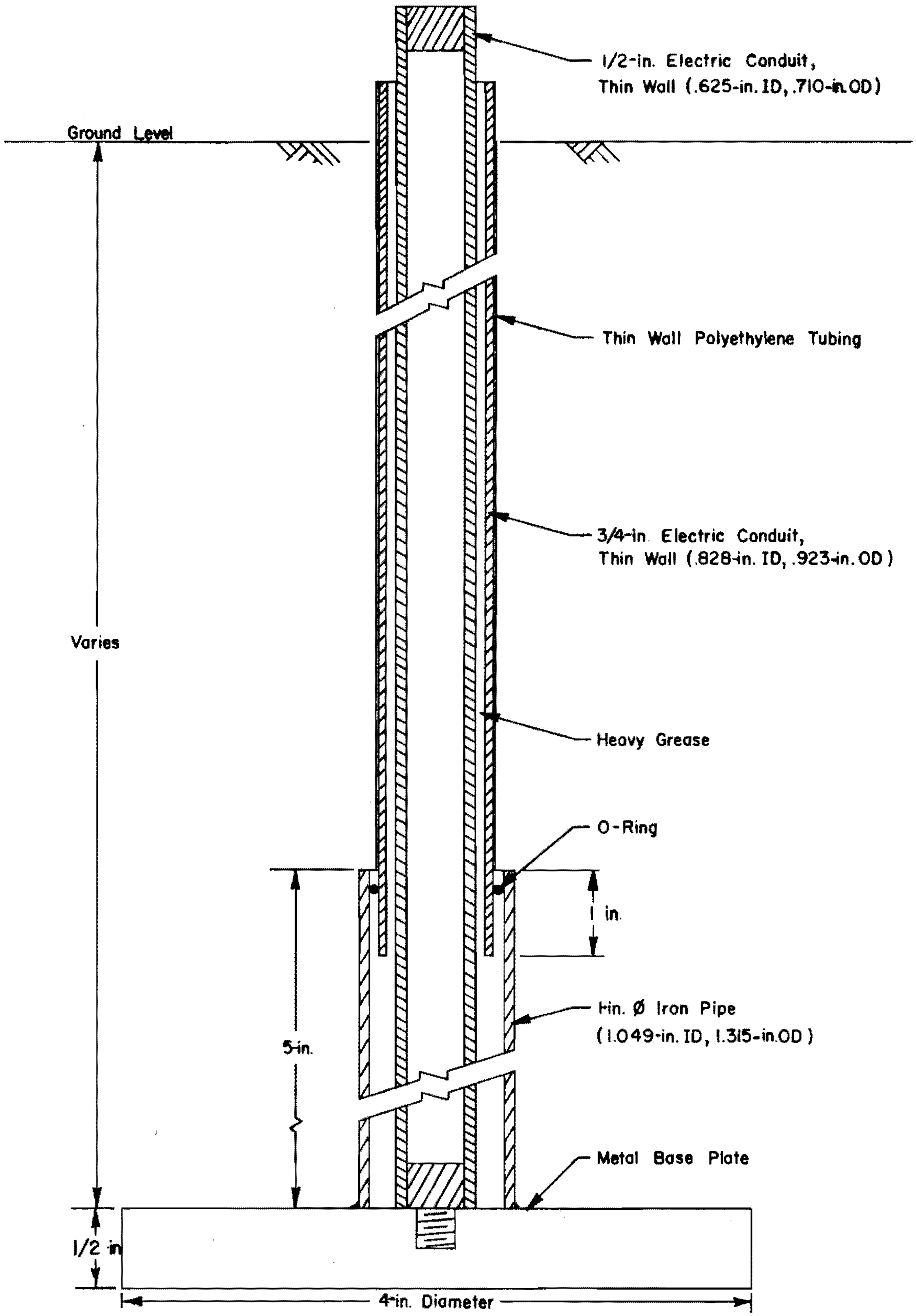


Fig 4.10. Tell-tale subsurface device.

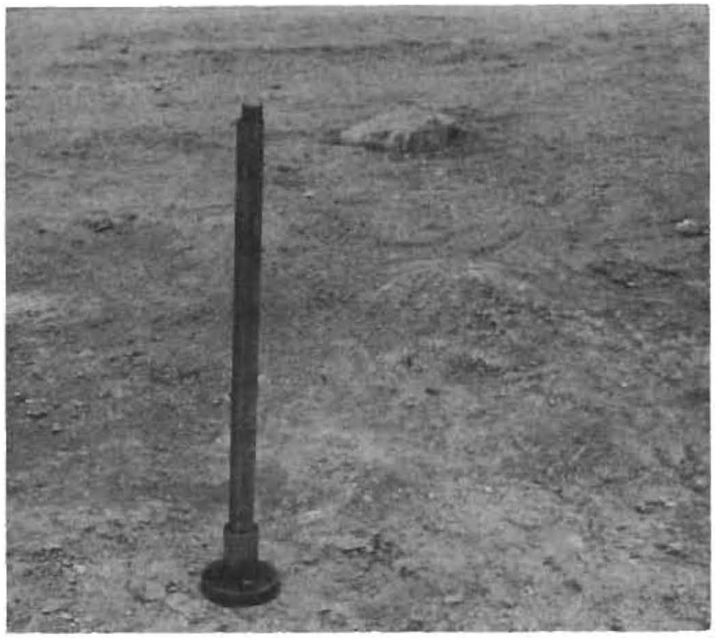


Fig 4.11. Complete tell-tale assembly.

Fig 4.12. Installed tell-tale.

soil. The instrumentation was approximately 2 feet below the present ground surface elevation so that discing operations could be conducted without damaging the instruments or slowing down construction operations.

Nuclear Moisture and Density Determinations

The nuclear depth moisture and density probes used are described in Appendix 1. Soil properties were measured indirectly by the radiation back-scatter phenomenon method in both the moisture and density gages. The calibration standards developed and constructed at Oklahoma State University by Moore and Haliburton (Ref 27) were used to obtain calibration curves for the nuclear equipment (see Appendix 3).

A complete set of nuclear readings (see Appendix 4) was taken on three consecutive days after the nuclear access tubes were installed. The readings taken on the first two days were not used because the soil disturbed during drilling holes for the tubes was still rebounding and tightening around the tube. The third day's readings were used for initial subsurface moisture content and density values.

The nuclear equipment was warmed up for approximately 15 minutes before any readings were taken. A standard count (see Fig 4.13) consisting of five 60 second counts with the probe in its shield was taken before and after taking a complete set of subsurface readings in an access tube (see Fig 4.14). This standard count was used to minimize instrument error. Three 60-second counts were taken at 1-foot intervals in the access tube, starting 6 inches from the bottom. The sum of the three 60-second counts was divided by three to determine an average reading at a given elevation. This reading was divided by the average of the before and after standard counts to arrive at a count ratio. The entire process was followed with both the moisture and density probes. The count ratios for moisture and density were then used with the appropriate calibration curve to determine the water content (pounds per cubic foot) and wet density (pounds per cubic foot) of the subsurface soil.

Elevation Readings

Elevation readings were taken with a self-leveling Carl Zeiss (No. 362013) level. Bench mark No. 1 was used as the reference for all elevation readings, and periodic level loops between all three bench marks were run to be sure that the reference had not moved. On the same day nuclear readings were made, a

Fig 4.13. Taking standard count with moisture probe.

Fig 4.14. Readings in access tube with density probe.

level loop was run to tie in bench mark No. 1 and the surface plate located at station 999+90, 60 feet right of survey center line. As only small variations in movement were expected at different depths, this reference loop had to be tied in with an accuracy of ± 0.005 feet. Using the surface plate at station 999+90 and 60 feet of survey center line as a reference, a final level loop was run to determine elevations of all the swell measuring devices. The longest level shot in this loop was about 20 feet, and therefore an accuracy of better than ± 0.002 feet was obtained. Due to the small movements expected, accuracy in all elevation readings was most important. The same level rod was to be used throughout the entire study to minimize rod error. It was also found necessary to use a target and rod level.

RESULTS OF THE FIELD STUDY

During the first 18 months of this study, five complete sets of data for swell measurement and nuclear determinations for subsurface moisture and density were obtained from the Atlanta test site. A set of moisture and density values was also obtained with a push-barrel undisturbed sampler. The data were obtained on the following dates:

Data	Date
Initial elevations of surface plates	November 12, 1968
Initial elevations of tell-tales	February 11, 1969
Elevations of surface plates and tell-tales	July 22, 1969
Push-barrel undisturbed samples	August 20, 1969
Nuclear moisture and density readings	August 27, 1969
Complete elevation and nuclear readings	August 28, 1969
Nuclear moisture and density readings	August 29, 1969
Complete elevation and nuclear readings	November 7, 1969
Complete elevation and nuclear readings	December 19, 1969
Complete elevation and nuclear readings	April 7, 1970

Tabulations of the nuclear moisture and density data are included in Appendix 4. Records of daily temperature and rainfall were kept at the test site (Appendix 5) which also includes an unloading curve for the test site.

Moisture Changes

The moisture content of the subgrade increased at all three access tube locations. Due to the necessity of locating the access tubes below the level of discing operations, no nuclear determinations were made in the top 3 feet of the subgrade (elevation of 371 to 368 feet). However, soil samples taken by hand indicate that the top 3 feet of soil had a moisture content of more than 30 percent on August 20, 1969. From 3 to 6 feet deep (elevation of 368 to 365 feet) during the period August 27, 1969, to April 7, 1970, the moisture content of the soil increased about 5 percent. The moisture content below 6 feet increased an average of 2 percent. This increase in moisture content was fairly uniform and extended to the full 20-foot-depth being tested. Moisture profiles for all three nuclear access tubes (Figs 4.15 through 4.17) indicate the moisture variations.

Density Changes

The apparent wet density of the subgrade increased at all three access tube locations. No nuclear determinations for density were made in the top 3 feet of the subgrade (elevation of 371 to 368 feet). During the seven months nuclear readings were taken, August 29, 1969, November 7, 1969, December 19, 1969, and April 7, 1970, the apparent density at a depth of 3 feet increased about 4 lb/cu ft (from an average of 88 lb/cu ft to 92 lb/cu ft). As the depth increased, the density of the soil also increased until at a depth of 20 feet the density was 98 lb/cu ft. The increase in density became smaller as the depth increased and was only 2 lb/cu ft at a depth of 20 feet. Density profiles for all three nuclear access tubes (Figs 4.18 through 4.20) indicated this trend.

Soil Sampling

The subgrade soil was sampled to assure that the nuclear moisture-density results were not reflecting a local condition induced by the access tubes. Undisturbed push-barrel samples were obtained 3 feet from each access tube on August 20, 1969.

The sampling did not give results which agreed with absolute values of the nuclear results. However, the plots of the two methods indicate similar trends in both moisture and density. The plotted peaks of the push-barrel

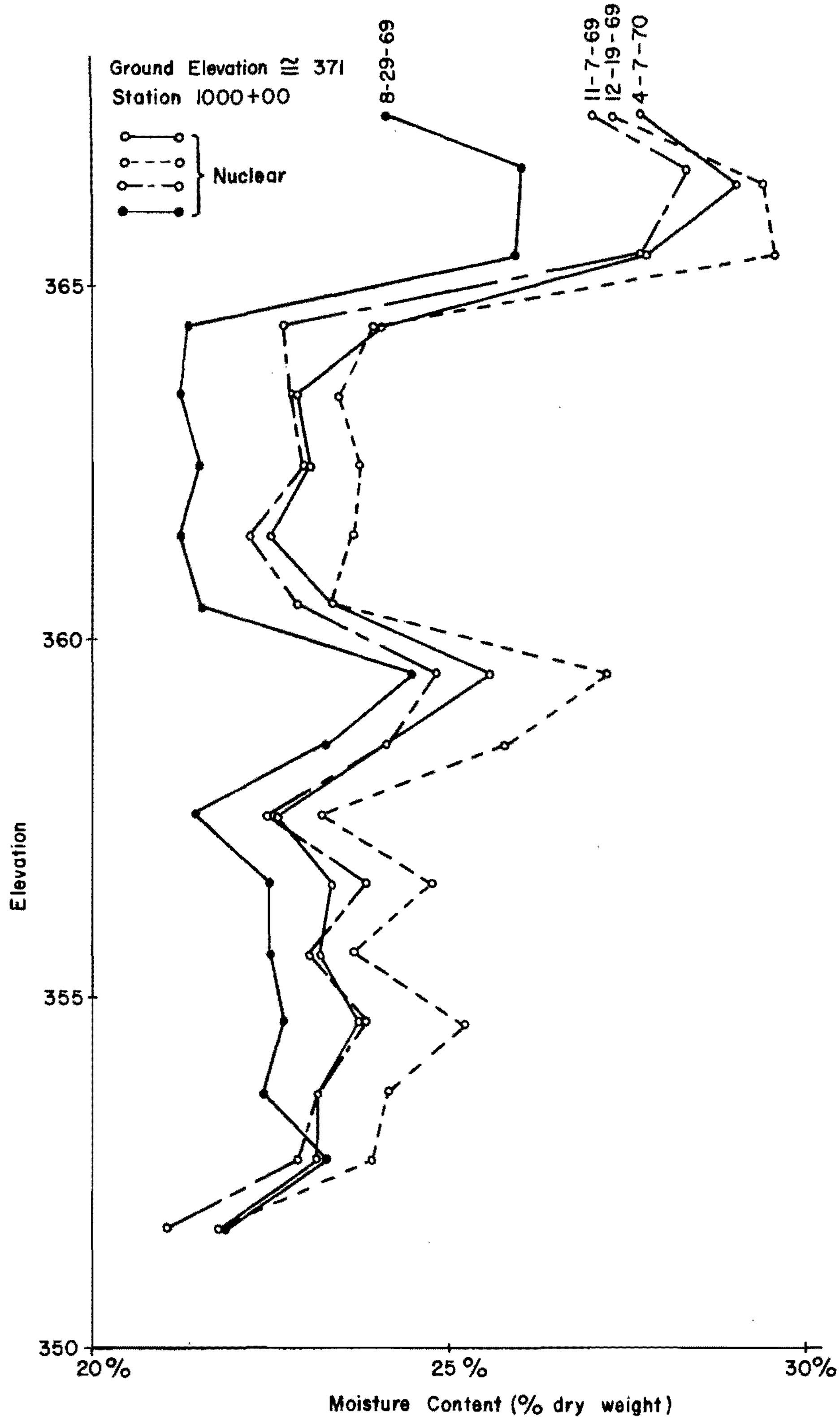


Fig 4.15. Water content at Station 1000+00, 69 feet right.

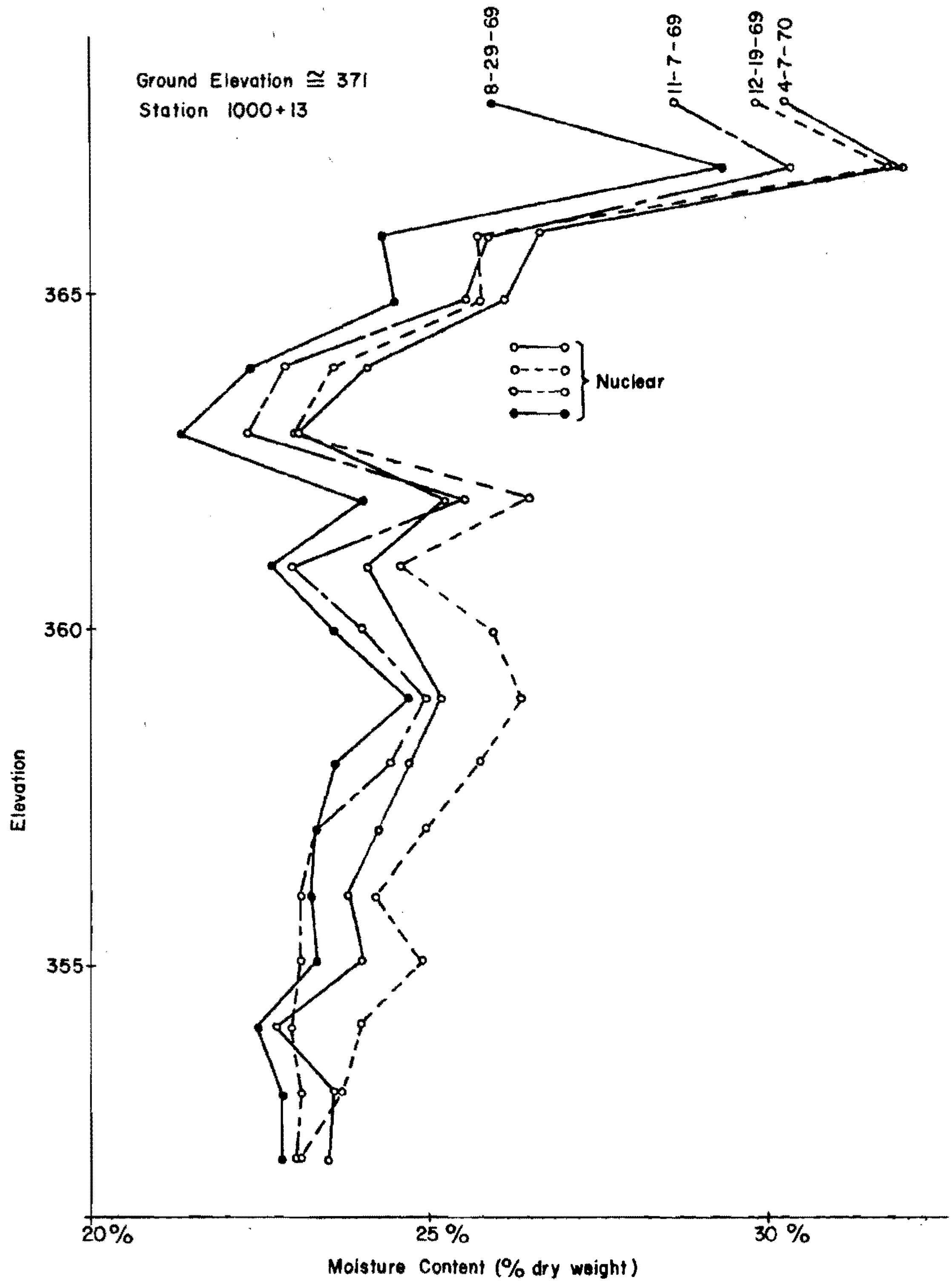


Fig 4.16. Water content at Station 1000+13, 69 feet right.

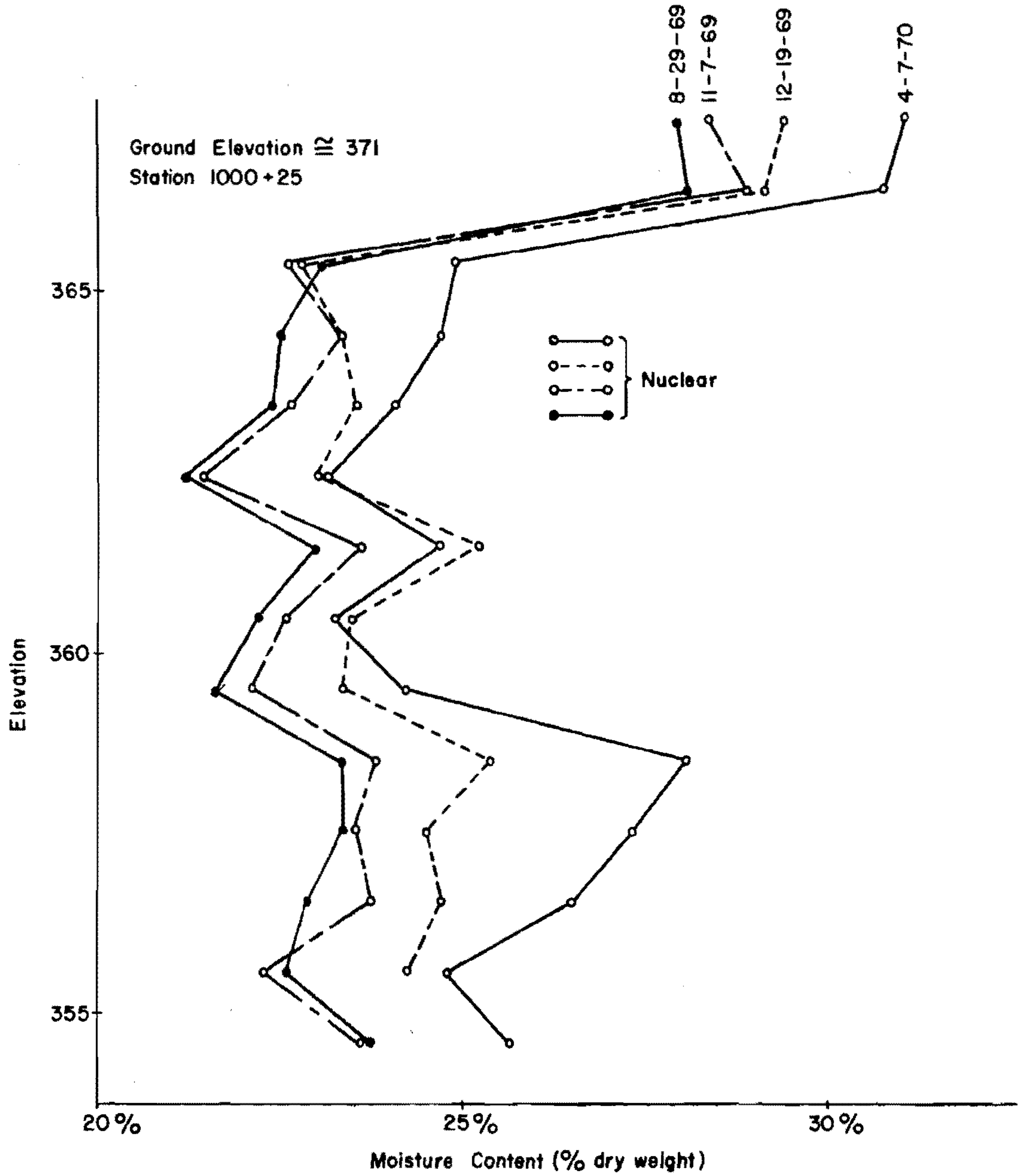


Fig 4.17. Water content at Station 1000+25, 69 feet right.

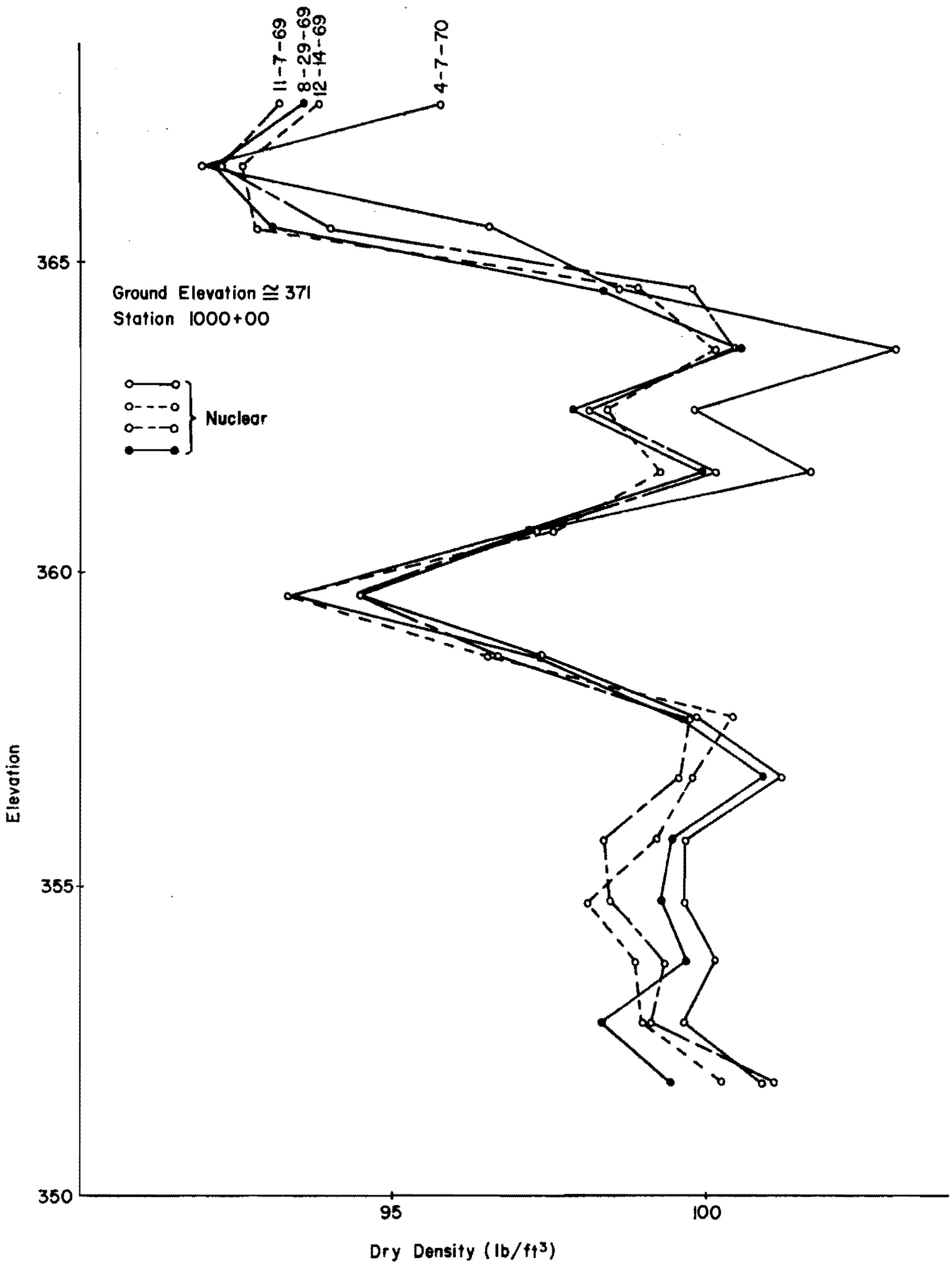


Fig 4.18. Density at Station 1000+00, 69 feet right.

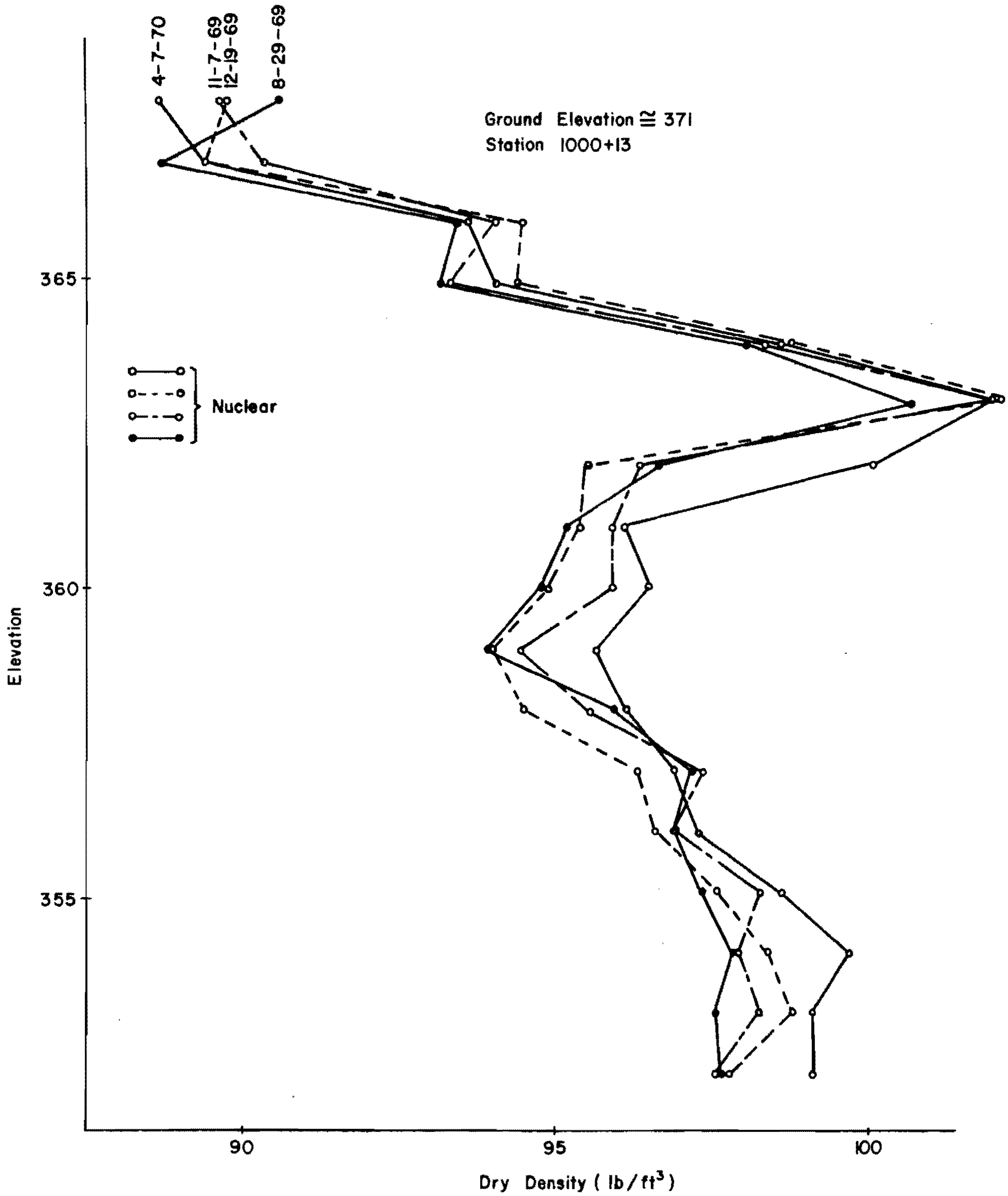


Fig 4.19. Density at Station 1000+13, 69 feet right.

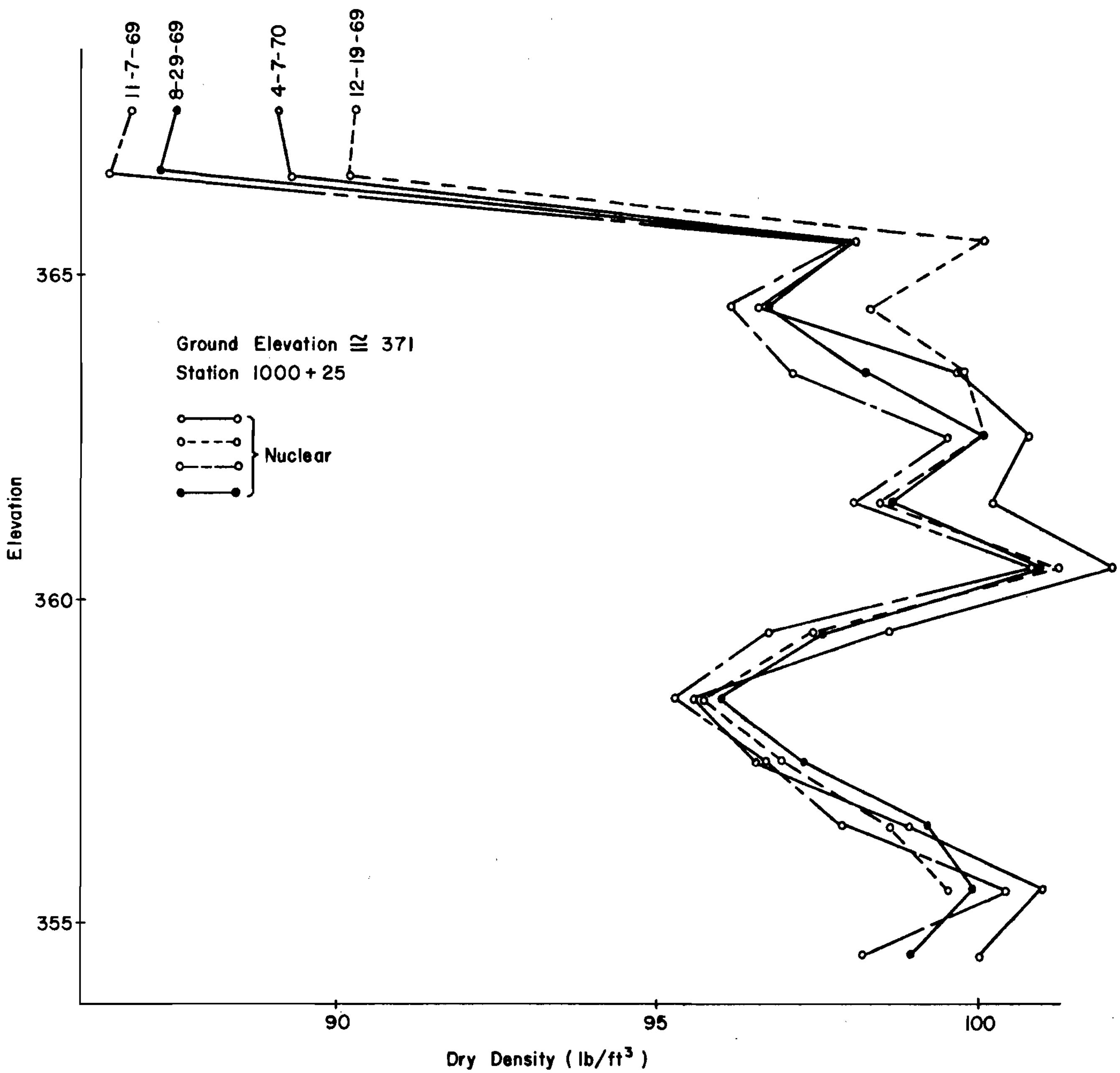


Fig 4.20. Density at Station 1000+25, 69 feet right.

results are more eratic than those of the nuclear method. This could be explained by the band averaging of the nuclear method, as against the point sampling of the push-barrel method.

Vertical Heave

All the vertical heave measuring devices were installed before any of the 23 feet of overburden had been removed. The vertical heave measured after removal of the overburden was, therefore, a combination of stress relief and swell. Interim readings on July 22, 1969, were followed by another set of elevation readings on August 28, 1969. Both of these readings were taken before the nuclear access tubes were installed and while sufficient overburden soil (3 to 5 feet) was still in place to prevent a significant change in the soil's moisture content. Push-barrel samples taken in early August and nuclear readings taken upon the installation of the access tubes verified that no moisture changes had occurred. Vertical movement that had occurred by the end of August 1969 was assumed to have been caused entirely by stress relief. Theoretical curves for stress relief using these points, the initial readings, and the total movements recorded on three subsequent dates are shown in Figs 4.21 through 4.23.

The average total vertical movement of the three surface plates on December 19, 1969, was 0.142 feet. Of this, 0.118 feet was attributable to stress relief and 0.024 feet to swell. The average total movement of two 4-foot-long tell-tales was 0.108 feet. Of this, 0.087 feet was attributable to stress relief and 0.021 feet to swell. A 4-foot tell-tale at station 1000+22 failed, probably due to loose soil left at the bottom of the hole during installation. The average total movement of the three 10-foot-long tell-tales was 0.080 feet. Of this, 0.066 feet was attributable to stress relief and 0.014 feet to swell.

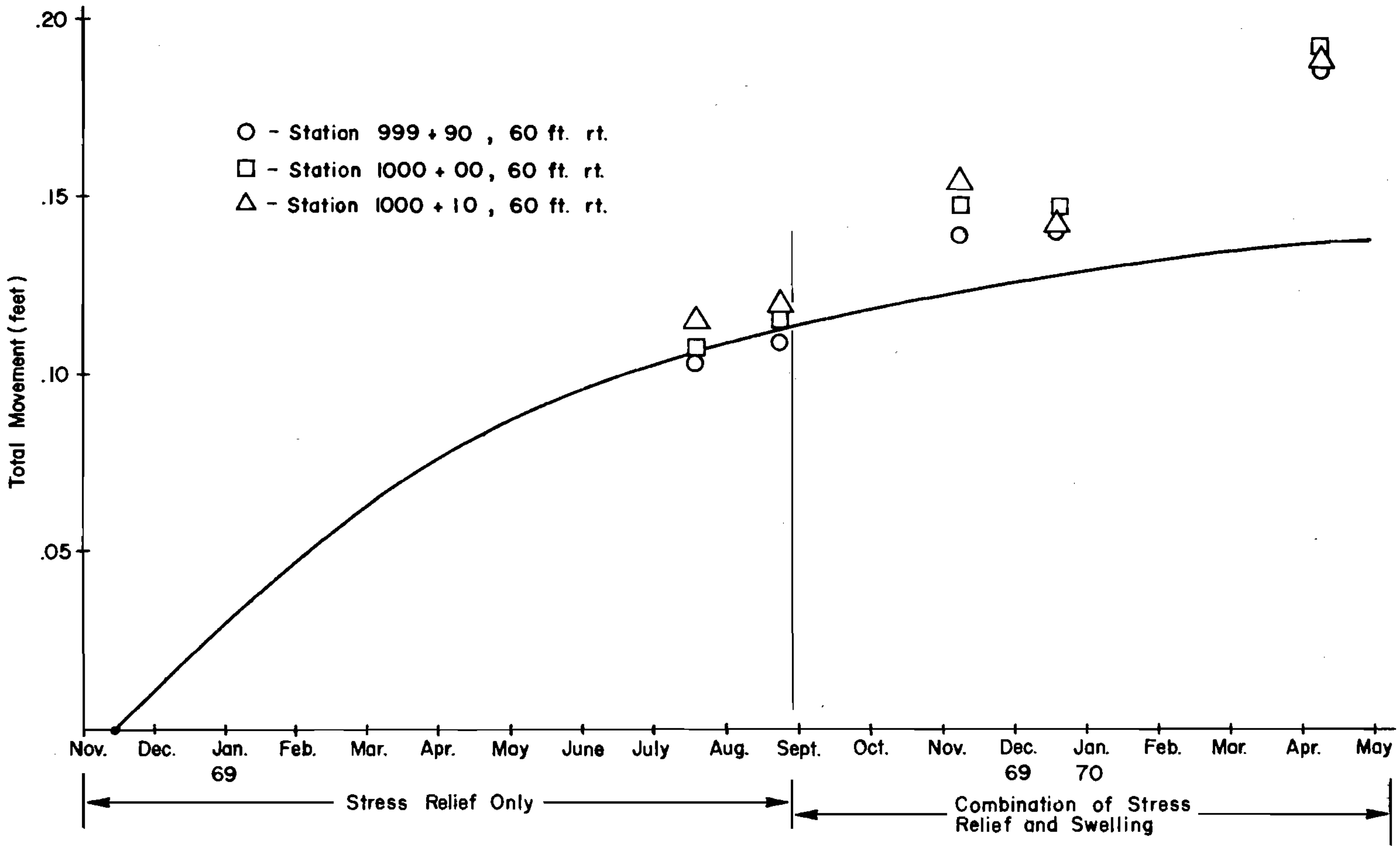


Fig 4.21. Vertical movement of surface plate.

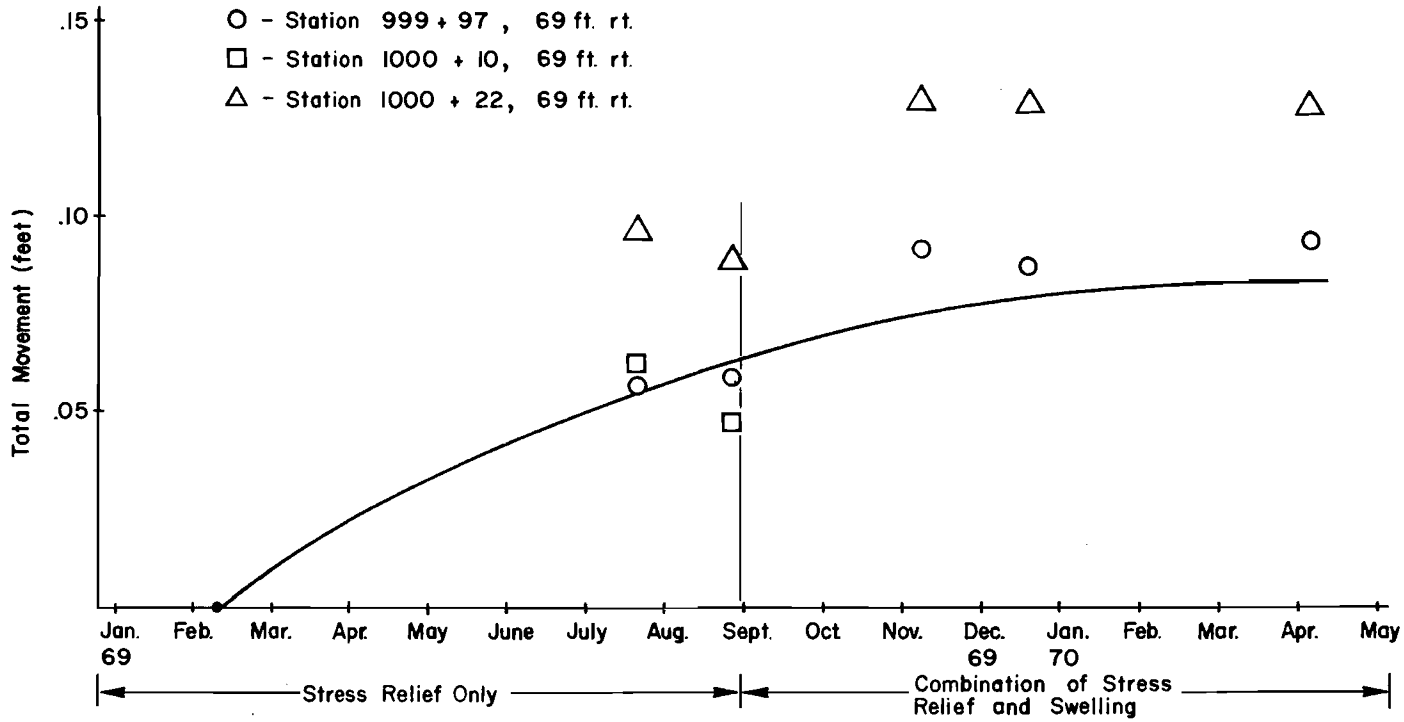


Fig 4.22. Vertical movement of 4-foot tell-tales.

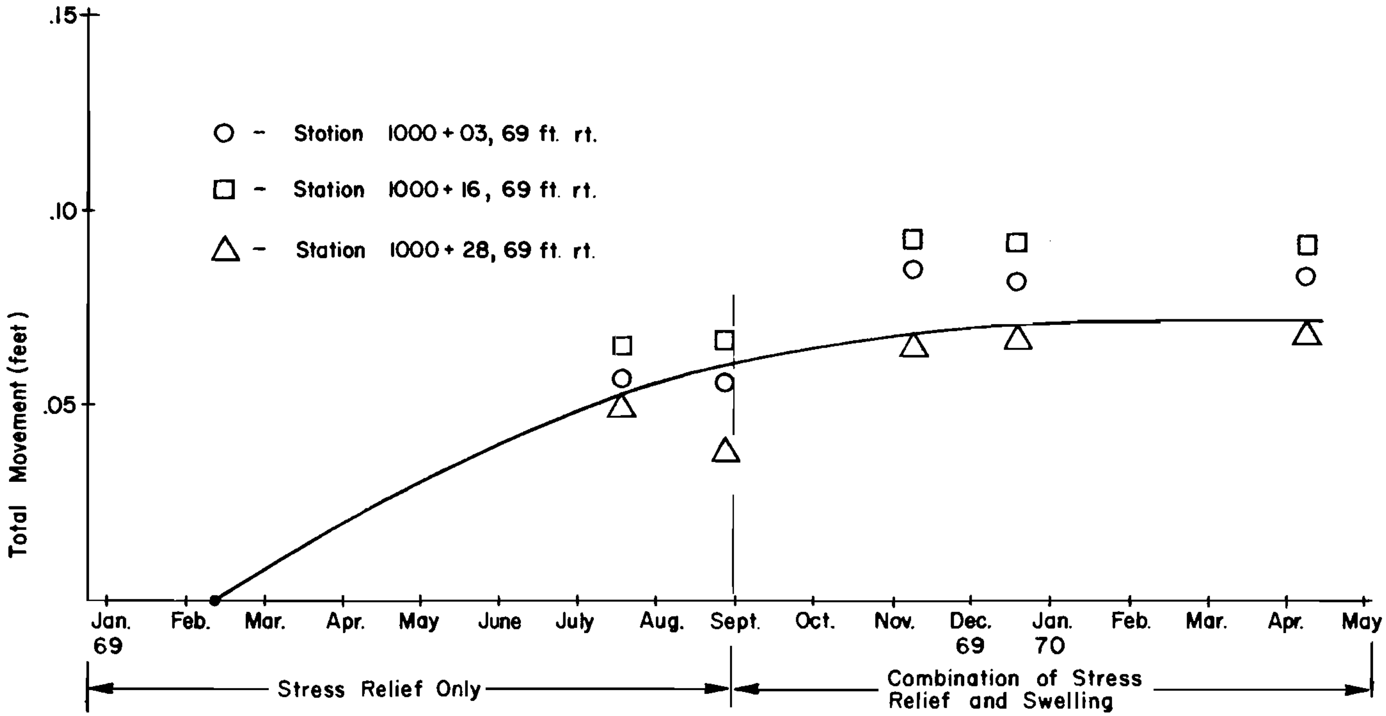


Fig 4.23. Vertical movement of 10-foot tell-tales.

CHAPTER 5. SUMMARY AND RECOMMENDATIONS

EXPANSIVE CLAY PROBLEM

Subgrade and foundation heaving caused by expansive clay soils creates major engineering and economic problems in Texas. Layers of montmorillonite expansive clays are found in wide areas of Texas where there are also climatic conditions conducive to the alternate shrinking and swelling of subgrade clays.

Presently there is no adequate and economic method available for determining how much a potentially expansive clay subgrade will swell. There are several methods for determining how much a specific clay sample will swell but variations of soil properties over the length of a highway may make it necessary to test many soil samples and such extensive testing can make the cost prohibitive.

Several remedial techniques which pre-swell an expansive clay soil are currently being investigated (see Chapter 3). At the present time the experimental results are not for determining which method is the most suitable, but in many cases, swelling can be controlled by proper construction methods and remedial techniques.

ATLANTA FIELD STUDY

This study was conducted to gather preliminary information and to develop instrumentation. The subgrade moisture and density variations and vertical movement of the soil at different depths were measured. The instrumentation and procedures for obtaining data are described in Chapter 4. The following is a summation of the field study results to date:

(1) With proper calibration procedures, nuclear methods for determining subsurface moisture and density are practical and effective.

(2) The subgrade moisture in the upper 6 feet of clay increased from 25 percent to over 30 percent while the moisture below 6 feet increased an average of 2 percent during the seven months of the study.

(3) The average wet density of the top 5 feet of soil was 92 pounds per cubic foot while the average wet density from 5 to 20 feet deep

was 98 pounds per cubic foot. This increase occurred abruptly at a depth of about 5 feet (see Figs 4.18 through 4.20). During the seven months of this study, the subgrade wet density increased about 4 pounds per cubic foot at a depth of 4 feet and about 2 pounds per cubic foot at a depth of 20 feet.

(4) For the seven months of the study, the surface soil moved upward .08 foot more than the soil at a 4-foot depth and .11 foot more than the soil at a 10-foot depth.

FUTURE STUDIES

The measurement of soil suction should be investigated and equipment necessary to measure in situ soil suction should be developed. Research work in expansive clay soils has shown that "soil water suction" may be an important factor in the movement of moisture in clays. Research Reports 118-1 and 118-3 (Refs 21 and 22) establish the theory of moisture movement and soil suction in a clay soil. A thermocouple phychrometer has been successfully used to measure soil suction (Ref 32), but little research has been done on methods of measuring soil suction in situ.

The study of nuclear methods for determining soil moisture and density should be continued, with special emphasis on calibration techniques and methods of installing and subsurface access tubes. Extensive work on this topic is being done by Haliburton (Ref 14) on soils in Oklahoma.

The study and use of various remedial techniques to prevent damage to highways and building structures should be continued.

A study should be undertaken to develop a better and easier method of measuring the vertical movement of soil at various depths below ground level.

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APPENDIX 1

DESCRIPTION OF NUCLEAR EQUIPMENT

APPENDIX 1. DESCRIPTION OF NUCLEAR EQUIPMENT

All equipment manufactured by Troxler Electronic Laboratories, Inc., U. S. Highway 70, West, P. O. Box 5997, Raleigh, North Carolina 27607.

Scaler:	Model	200B
	Serial Number	148

Density Equipment: ST-DD-2 Depth Density Gage, Model 1351, Shield and Standard Model S-7, Serial Number 99, with a Depth Density Probe Model 504, Serial Number 42, using a 4 millicurie radium 226 source, Serial Number TR-3-10.

Moisture Equipment: SY-SM-1 Depth Moisture Gage, Shield and Standard Model S-5, Serial Number 418, with a Depth Moisture Probe Model 104, Serial Number G-24812E, using a 3 millicurie radium 226 beryllium source, Serial Number N-3-73.

APPENDIX 2

CHEMICAL AND X-RAY DIFFRACTION OF SOILS

TABLE A2.1. ANALYSIS OF ATLANTA TEST SITE SOIL

Element	Semi-Quantitative Spectrographic Analysis*	
	Red Clay	Yellow-Grey Clay
Magnesium	1 to 10 %	1 to 10 %
Iron	2 to 20 %	2 to 20 %
Sodium	0.5 to 5 %	0.5 to 5 %
Potassium	Less than 0.5 %	Less than 0.5 %
Calcium	0.05 to 0.5 %	0.02 to 0.2 %
Cadmium	Less than 0.01 %	Less than 0.01 %
Berrylium	Less than 0.003 %	Less than 0.003 %
Boron	Less than 0.005 %	Less than 0.005 %
Lithium	Less than 0.02 %	Less than 0.02 %
Silicon	Major	Major
Aluminum	1 to 10 %	1 to 10 %
Copper	0.001 to 0.01 %	0.0005 to 0.005 %
Titanium	0.2 to 2 %	0.2 to 2 %
Manganese	0.02 to 0.2 %	0.02 to 0.2 %
Chromium	0.01 to 0.1 %	0.01 to 0.1 %
Vanadium	0.01 to 0.1 %	0.01 to 0.1 %
Nickel	0.001 to 0.01 %	0.001 to 0.01 %
Zirconium	0.002 to 0.02 %	0.002 to 0.02 %
Barium	0.001 to 0.01 %	0.001 to 0.01 %
Tungsten	Less than 0.02 %	Less than 0.02 %
Bismuth	Less than 0.001 %	Less than 0.001 %
Antimony	Less than 0.02 %	Less than 0.02 %
Tin	Less than 0.003 %	Less than 0.003 %
Molybdenum	Less than 0.001 %	Less than 0.001 %
Cobalt	Less than 0.001 %	Less than 0.001 %
Lead	Less than 0.01 %	Less than 0.01 %
Zinc	Less than 0.02 %	Less than 0.02 %
Niobium	Less than 0.006 %	Less than 0.006 %
Tantalum	Less than 0.1 %	Less than 0.1 %

* Analysis performed by Andrew S. McCreath Lab, Harrisburg, Pennsylvania.

TABLE A2.2. WET CHEMICAL ANALYSIS OF O.S.U. CLAY

Element/Compound	Wet Chemical Analysis* Permion Red Clay Used in Oklahoma State University Nuclear Calibration Standards
Magnesium	.37 %
Iron	2.54 %
Sodium	.58 %
Potassium	1.01 %
Calcium	.73 %
Cadmium	Less than 0.001 %
Berrylium	Less than .001 %
Boron	Less than .005 %
Lithium	.002 %
Silicon Dioxide	75.40 %

* Analysis performed by Andrew S. McCreath Lab, Harrisburg, Pennsylvania.

TABLE A2.3. X-RAY DIFFRACTION DATA

Clay Type	Percentage of Clay Minerals as Determined by X-ray Diffraction		
	Montmorillonite	Illite	Kaolinite
Red clay found at Atlanta test site	35%	50%	15%
Yellow-grey clay found at Atlanta test site	50%	40%	10%
Permian red clay used in Oklahoma State University nuclear calibration standards	60%	15%	25%

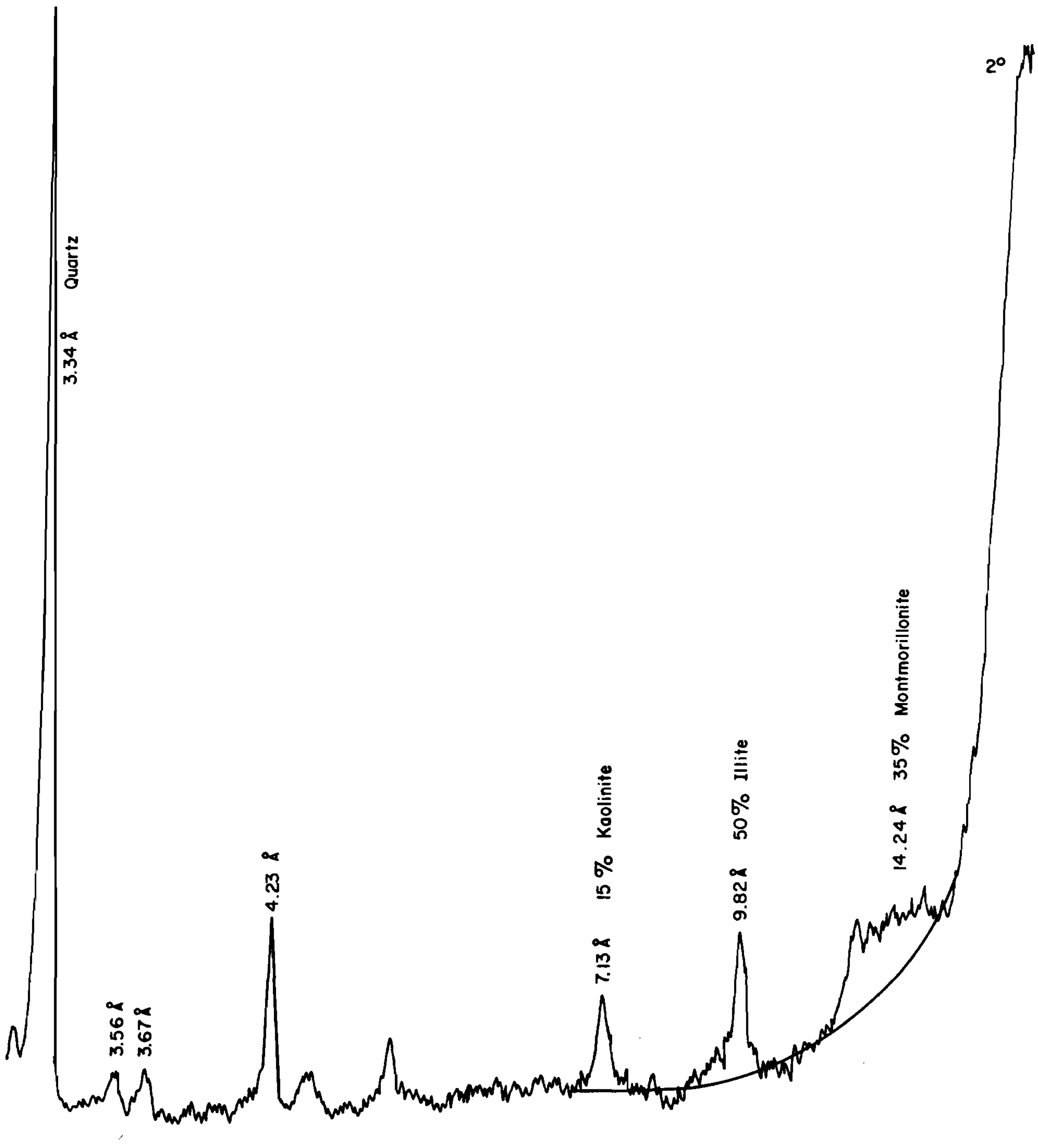

Fig A2.1. X-ray diffraction of test site red clay.

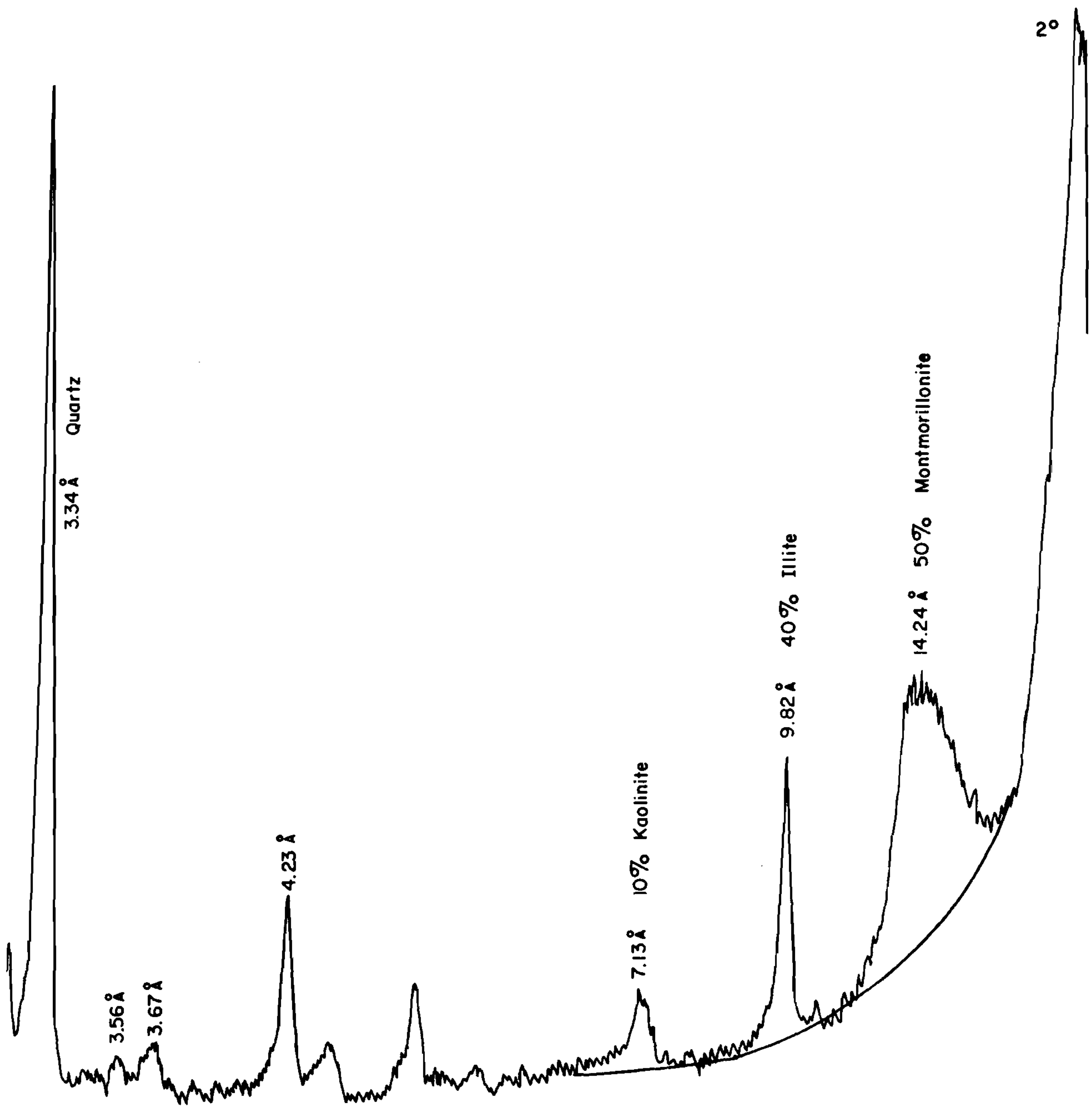

Fig A2.2. X-ray diffraction of test site yellow-grey clay.

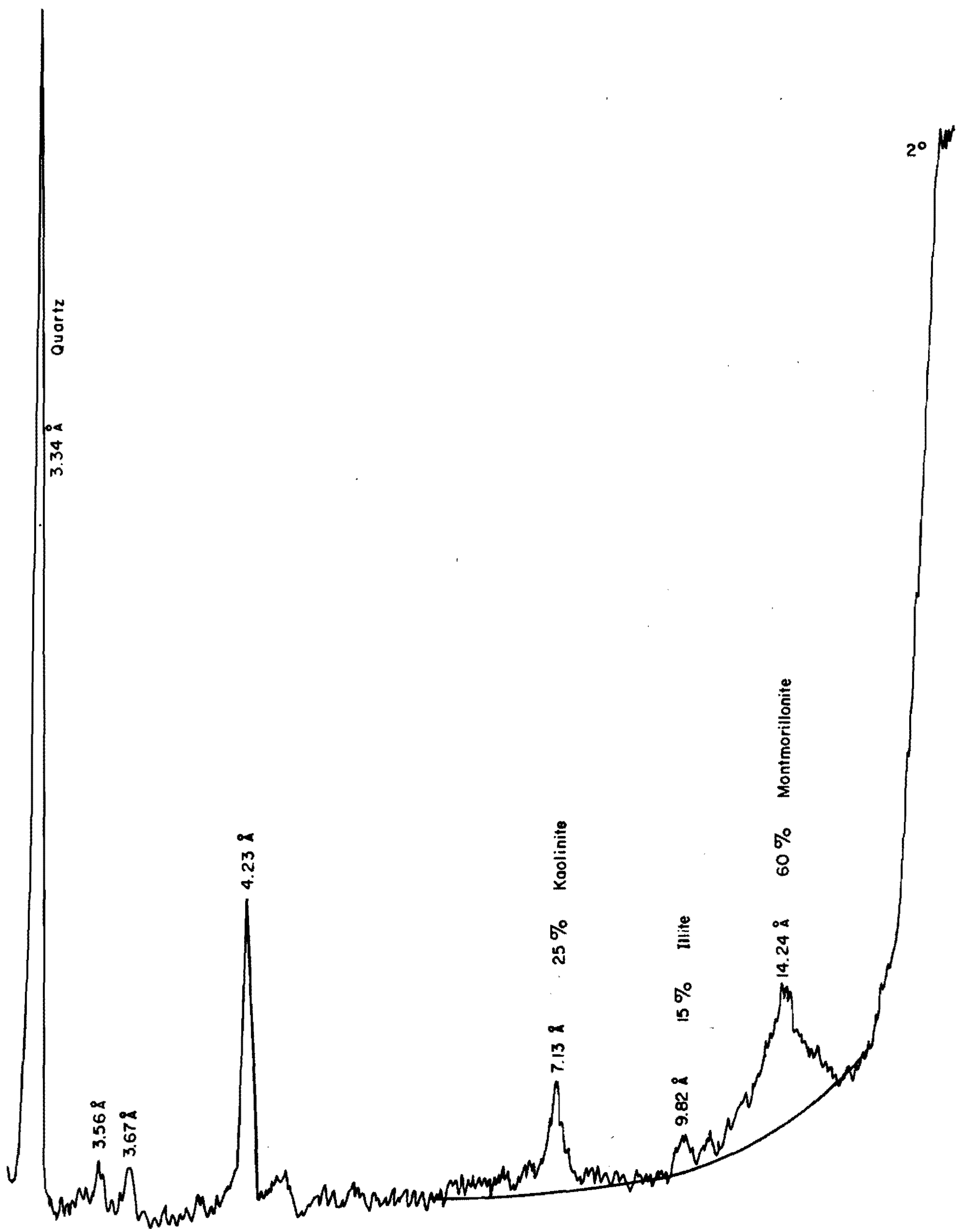

Fig A2.3. X-ray diffraction of O.S.U. clay sample.

APPENDIX 3

NUCLEAR CALIBRATION CURVES

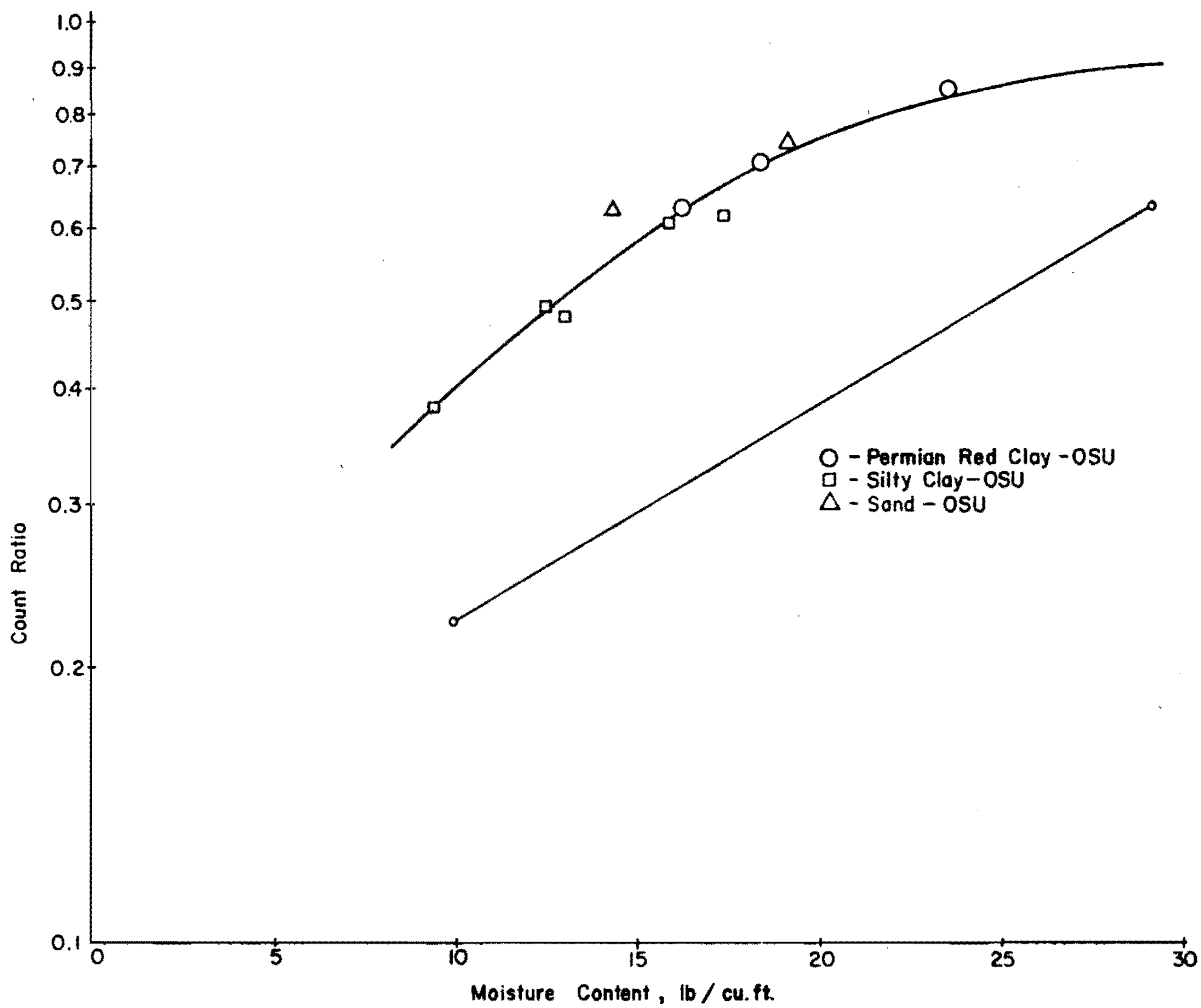

Fig A3.1. Moisture content calibration curve, April 14, 1970.

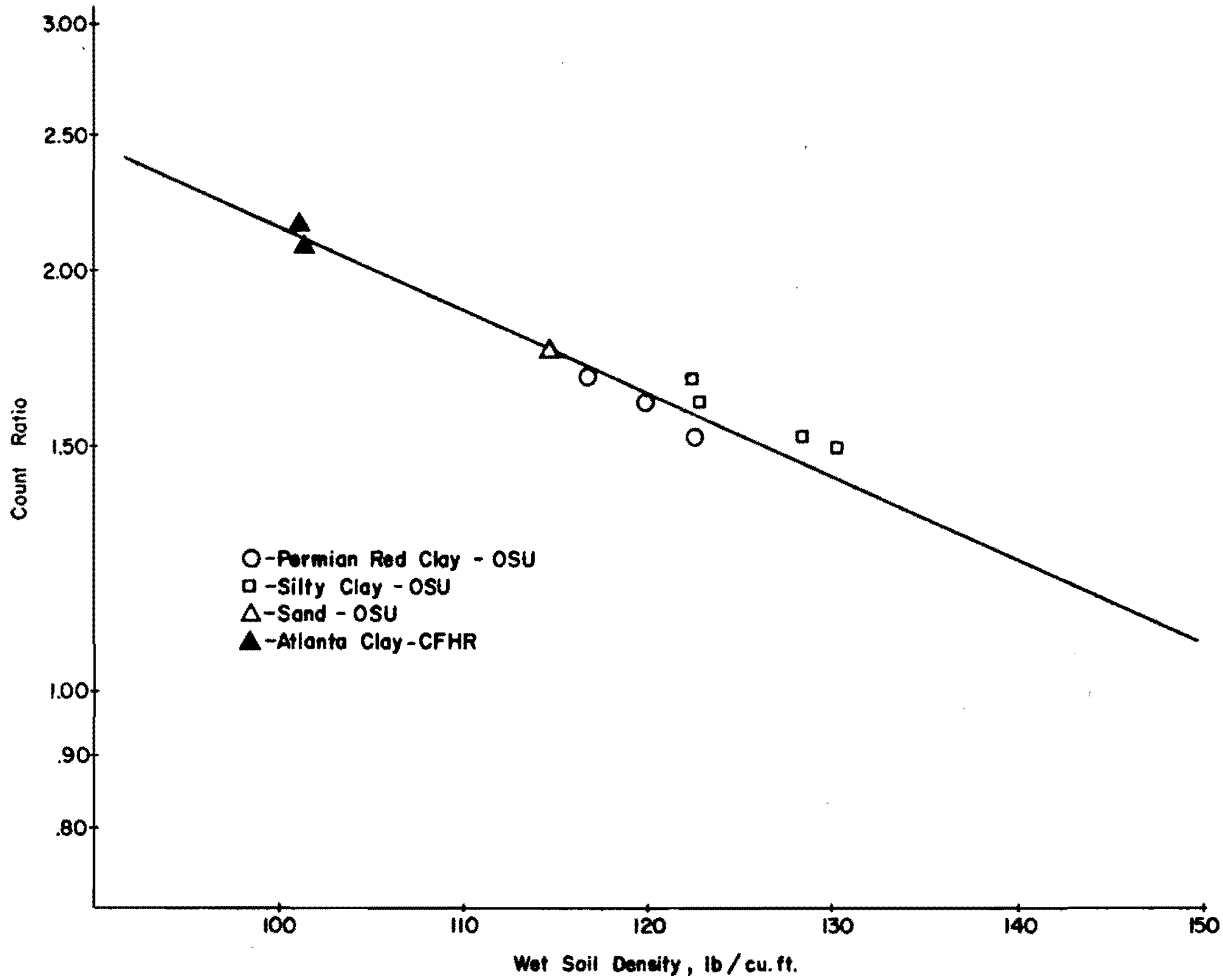

Fig A3.2. Wet density calibration curve, April 14, 1970.

APPENDIX 4

TEST SITE SUBSURFACE MOISTURE AND DENSITY DATA

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+00, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 29 Aug 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	350.70	Elev. Tube Bottom	350.70
Standard Count Used	14549.3	Standard Count Used	12806.1
Reading on Cable	10	Reading on Cable	2
Operating Voltage	1350	Operating Voltage	775

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.20	1.668	116.2	.8143	22.7	93.5	.243
15.5	366.20	1.664	116.2	.8472	24.2	92.0	.263
14.5	365.20	1.647	117.2	.8475	24.3	92.9	.262
13.5	364.20	1.595	119.5	.7750	21.1	98.4	.214
12.5	363.20	1.541	122.3	.7820	21.5	100.8	.213
11.5	362.20	1.604	119.0	.7760	21.1	97.9	.216
10.5	361.20	1.555	121.4	.7797	21.3	100.1	.213
9.5	360.20	1.615	118.6	.7740	21.1	97.5	.216
8.5	359.20	1.665	116.2	.8199	23.0	93.2	.247
7.5	358.20	1.587	120.2	.8184	22.8	97.4	.234
6.5	357.20	1.558	121.1	.7809	21.4	99.7	.215
5.5	356.20	1.504	123.9	.8183	22.8	101.1	.226
4.5	355.20	1.546	122.1	.8092	22.5	99.6	.226
3.5	354.20	1.547	122.1	.8169	22.7	99.4	.228
2.5	353.20	1.504	122.3	.8106	22.5	99.8	.225
1.5	352.20	1.556	121.4	.8202	23.0	98.4	.234
0.5	351.20	1.555	121.4	.7921	21.8	99.6	.219

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+00, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 7 Nov 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	350.70	Elev. Tube Bottom	350.70
Standard Count Used	14573.8	Standard Count Used	12660.1
Reading on Cable	10	Reading on Cable	2
Operating Voltage	1400	Operating Voltage	750

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.20	1.618	118.5	.8695	25.4	93.1	.273
15.5	366.20	1.619	118.5	.8856	26.4	92.1	.287
14.5	365.20	1.587	120.2	.8842	26.3	93.9	.280
13.5	364.20	1.530	122.7	.8185	22.8	99.9	.228
12.5	363.20	1.506	123.8	.8233	23.1	100.7	.229
11.5	362.20	1.566	120.9	.8164	22.7	98.2	.231
10.5	361.20	1.529	122.7	.8070	22.4	100.3	.223
9.5	360.20	1.598	119.4	.8016	22.3	97.1	.230
8.5	359.20	1.633	118.0	.8374	23.6	94.4	.250
7.5	358.20	1.584	120.2	.8321	23.5	96.7	.243
6.5	357.20	1.538	122.3	.8091	22.5	99.8	.225
5.5	356.20	1.510	123.6	.8405	23.9	99.7	.240
4.5	355.20	1.555	121.4	.8197	22.9	98.5	.232
3.5	354.20	1.542	122.3	.8346	23.7	98.6	.240
2.5	353.20	1.534	122.7	.8237	23.2	99.5	.233
1.5	352.20	1.547	122.0	.8185	22.8	99.2	.230
0.5	351.20	1.530	122.7	.7818	21.4	101.3	.211

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+00, 69 feet right of center line. Interstate Highway 30, Bowie County, Texas.

Date Reading Taken: 19 Dec 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	350.70	Elev. Tube Bottom	350.70
Standard Count Used	13594.2	Standard Count Used	12695.1
Reading on Cable	10	Reading on Cable	2
Operating Voltage	1400	Operating Voltage	750

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.20	1.600	119.6	.8796	25.9	93.7	.276
15.5	366.20	1.578	120.1	.9118	27.6	92.5	.298
14.5	365.20	1.577	120.5	.9145	27.8	92.7	.300
13.5	364.20	1.528	122.9	.8409	23.9	99.0	.241
12.5	363.20	1.999	124.0	.8368	23.7	100.3	.236
11.5	362.20	1.546	122.0	.8314	23.5	98.5	.239
10.5	361.20	1.518	123.1	.8366	23.7	99.4	.238
9.5	360.20	1.577	120.5	.8198	22.9	97.6	.235
8.5	359.20	1.612	118.8	.8732	25.6	93.2	.275
7.5	358.20	1.552	121.7	.8638	25.1	96.6	.260
6.5	357.20	1.503	124.0	.8279	23.4	100.6	.233
5.5	356.20	1.485	124.9	.8627	25.0	99.9	.250
4.5	355.20	1.527	122.9	.8339	23.6	99.3	.238
3.5	354.20	1.516	123.1	.8617	24.9	98.2	.254
2.5	353.20	1.517	123.1	.8445	24.1	99.0	.243
1.5	352.20	1.524	123.0	.8409	23.9	99.1	.241
0.5	351.20	1.535	122.3	.7936	21.9	100.4	.218

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+00, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 7 April 70
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	350.70	Elev. Tube Bottom	350.70
Standard Count Used	14502.7	Standard Count Used	
Reading on Cable	10	Reading on Cable	2
Operating Voltage	1350	Operating Voltage	

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.20	1.530	122.6	.897	26.8	95.8	.280
15.5	366.20	1.610	118.8	.902	27.0	91.8	.294
14.5	365.20	1.513	123.7	.904	27.1	16.6	.281
13.5	364.20	1.529	122.6	.842	23.9	98.7	.242
12.5	363.20	1.442	127.1	.839	23.8	103.3	.230
11.5	362.20	1.522	123.1	.826	23.2	99.9	.232
10.5	361.20	1.489	124.9	.820	23.0	101.9	.226
9.5	360.20	1.587	120.0	.815	22.8	97.2	.235
8.5	359.20	1.609	118.8	.849	24.4	94.4	.258
7.5	358.20	1.561	121.1	.835	23.7	97.4	.243
6.5	357.20	1.530	122.6	.811	22.6	100.0	.226
5.5	356.20	1.482	125.2	.832	23.8	101.4	.235
4.5	355.20	1.522	123.1	.828	23.3	99.8	.233
3.5	354.20	1.512	123.7	.842	23.9	99.8	.239
2.5	353.20	1.512	123.7	.828	23.4	100.3	.233
1.5	352.20	1.523	123.1	.824	23.3	99.8	.233
0.5	351.20	1.520	123.1	.797	22.0	101.1	.218

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+13, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 29 Aug 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	351.00	Elev. Tube Bottom	351.00
Standard Count Used	14549.3	Standard Count Used	12806.1
Reading on Cable	9	Reading on Cable	1
Operating Voltage	1350	Operating Voltage	775

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.50	1.727	113.7	.8323	23.5	90.4	.260
15.5	366.50	1.711	114.6	.8820	26.1	88.5	.295
14.5	365.50	1.671	116.1	.8140	22.7	93.4	.243
13.5	364.50	1.680	115.9	.8174	22.8	93.1	.245
12.5	363.50	1.586	120.1	.7938	21.9	98.2	.223
11.5	362.50	1.535	122.4	.7809	21.4	101.0	.212
10.5	361.50	1.590	120.0	.8254	23.3	96.7	.241
9.5	360.50	1.653	116.7	.7851	21.5	95.2	.226
8.5	359.50	1.643	117.2	.8071	22.4	94.8	.236
7.5	358.50	1.650	117.1	.8238	23.2	93.9	.247
6.5	357.50	1.618	118.7	.8130	22.7	96.0	.236
5.5	356.50	1.590	120.0	.8150	22.7	97.3	.233
4.5	355.50	1.596	119.5	.8087	22.5	97.0	.232
3.5	354.50	1.560	120.1	.8128	22.7	97.4	.233
2.5	353.50	1.563	120.0	.7975	22.0	98.0	.224
1.5	352.50	1.565	120.0	.8040	22.3	97.7	.228
0.5	351.50	1.561	120.1	.8033	22.3	97.8	.228

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+13, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 7 Nov 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	351.00	Elev. Tube Bottom	351.00
Standard Count Used	14573.8	Standard Count Used	12660.1
Reading on Cable	9	Reading on Cable	1
Operating Voltage	1400	Operating Voltage	750

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.50	1.689	115.3	.8787	25.8	89.5	.288
15.5	366.50	1.638	117.8	.9128	27.6	90.2	.306
14.5	365.50	1.670	118.5	.8507	24.4	94.1	.259
13.5	364.50	1.698	117.2	.8412	23.9	93.3	.256
12.5	363.50	1.559	121.1	.8109	22.5	98.6	.228
11.5	362.50	1.489	124.9	.8126	22.7	102.2	.222
10.5	361.50	1.558	121.1	.8561	24.7	96.4	.256
9.5	360.50	1.628	118.0	.7986	22.0	96.0	.229
8.5	359.50	1.607	119.0	.8218	23.0	96.0	.240
7.5	358.50	1.629	118.0	.8337	23.6	94.4	.250
6.5	357.50	1.612	118.9	.8264	23.3	95.6	.244
5.5	356.50	1.589	120.2	.8173	22.7	97.5	.233
4.5	355.50	1.601	119.4	.8050	22.4	97.0	.231
3.5	354.50	1.560	121.1	.8166	22.7	98.4	.231
2.5	353.50	1.573	120.6	.8094	22.5	98.1	.229
1.5	352.50	1.561	121.1	.8180	22.7	98.4	.231
0.5	351.50	1.585	120.2	.8093	22.5	97.7	.230

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+13, 69 feet right of center line. Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 19 Dec 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	351.00	Elev. Tube Bottom	351.00
Standard Count Used	13622.3	Standard Count Used	12695.1
Reading on Cable	9	Reading on Cable	1
Operating Voltage	1400	Operating Voltage	750

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.50	1.660	116.5	.8999	26.9	89.6	.300
15.5	366.50	1.633	118.0	.9335	28.7	89.3	.321
14.5	365.50	1.605	118.9	.8496	24.4	94.5	.258
13.5	364.50	1.610	118.8	.8509	24.4	94.4	.258
12.5	363.50	1.532	122.4	.8280	23.4	99.0	.236
11.5	362.50	1.463	125.9	.8322	23.5	102.4	.229
10.5	361.50	1.563	121.0	.8706	25.4	95.6	.266
9.5	360.50	1.607	118.9	.8312	23.5	95.4	.246
8.5	359.50	1.600	119.6	.8587	24.7	94.9	.260
7.5	358.50	1.610	118.8	.8600	24.8	94.0	.264
6.5	357.50	1.604	118.9	.8512	24.4	94.5	.258
5.5	356.50	1.579	120.5	.8442	24.1	96.4	.250
4.5	355.50	1.585	120.1	.8280	23.4	96.7	.242
3.5	354.50	1.548	122.0	.8502	24.3	97.7	.249
2.5	353.50	1.537	122.3	.8348	23.7	98.6	.240
1.5	352.50	1.537	122.3	.8292	23.4	98.9	.237
0.5	351.50	1.573	120.5	.8107	22.6	97.9	.231

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+13, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 7 April 70
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	351.00	Elev. Tube Bottom	351.00
Standard Count Used	14682.2	Standard Count Used	
Reading on Cable	9	Reading on Cable	1
Operating Voltage	1350	Operating Voltage	

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
16.5	367.50	1.691	115.4	.901	26.9	88.5	.304
15.5	366.50	1.631	118.0	.935	28.8	89.2	.323
14.5	365.50	1.616	118.6	.862	25.0	93.6	.267
13.5	364.50	1.608	118.8	.855	24.7	94.1	.262
12.5	363.50	1.555	122.6	.837	23.8	98.8	.241
11.5	362.50	1.470	125.7	.830	23.5	102.2	.230
10.5	361.50	1.468	125.7	.870	25.4	100.3	.253
9.5	360.50	1.601	119.4	.824	23.2	96.2	.241
8.5	359.50	1.580	120.4	.840	23.8	96.6	.246
7.5	358.50	1.596	119.8	.844	24.1	95.7	.252
6.5	357.50	1.590	120.0	.839	23.8	96.2	.247
5.5	356.50	1.572	120.6	.833	23.6	97.0	.245
4.5	355.50	1.568	120.6	.823	23.2	97.4	.238
3.5	354.50	1.541	122.5	.835	23.7	98.8	.240
2.5	353.50	1.533	122.6	.816	22.7	99.9	.227
1.5	352.50	1.529	122.7	.829	23.4	99.3	.236
0.5	351.50	1.537	122.6	.825	23.3	99.3	.235

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+25, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 29 Aug 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	353.50	Elev. Tube Bottom	353.50
Standard Count Used	14549.3	Standard Count Used	12806.1
Reading on Cable	12	Reading on Cable	4
Operating Voltage	1350	Operating Voltage	775

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
13.5	367.00	1.768	112.1	.8538	24.6	87.5	.281
12.5	366.00	1.782	111.8	.8530	24.6	87.2	.282
11.5	365.00	1.575	120.5	.8088	22.5	98.0	.230
10.5	364.00	1.622	118.4	.7909	21.7	96.7	.224
9.5	363.00	1.587	120.1	.7968	21.9	98.2	.223
8.5	362.00	1.557	121.2	.7732	21.1	100.1	.211
7.5	361.00	1.554	121.2	.8114	22.6	98.6	.229
6.5	360.00	1.519	123.3	.8027	22.3	101.0	.221
5.5	359.00	1.617	118.6	.7709	21.0	97.6	.215
4.5	358.00	1.625	118.4	.8059	22.4	96.0	.233
3.5	357.00	1.592	120.0	.8127	22.7	97.3	.233
2.5	356.00	1.548	121.8	.8106	22.6	99.2	.228
1.5	355.00	1.533	122.4	.8100	22.5	99.9	.225
0.5	354.00	1.537	122.3	.8296	23.4	98.9	.237

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+25, 69 feet right of center line. Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 7 Nov 69
Person Taking Readings: John Wise

Moisture Data

Elev. Tube Bottom	353.50
Standard Count Used	14573.8
Reading on Cable	12
Operating Voltage	1400

Density Data

Elev. Tube Bottom	353.50
Standard Count Used	12660.1
Reading on Cable	4
Operating Voltage	750

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
13.5	367.00	1.787	111.5	.8559	24.7	86.8	.285
12.5	366.00	1.789	111.5	.8643	25.1	86.4	.291
11.5	365.00	1.584	120.2	.8001	22.1	98.1	.225
10.5	364.00	1.616	118.5	.8055	22.4	96.1	.233
9.5	363.00	1.603	119.0	.7935	21.9	97.1	.226
8.5	362.00	1.569	120.7	.7770	21.2	99.5	.213
7.5	361.00	1.558	121.1	.8224	23.1	98.0	.236
6.5	360.00	1.512	123.6	.8125	22.7	100.9	.225
5.5	359.00	1.628	118.0	.7813	21.3	96.7	.220
4.5	358.00	1.628	118.0	.8136	22.7	95.3	.238
3.5	357.00	1.599	119.4	.8127	22.7	96.7	.235
2.5	356.00	1.559	121.1	.8264	23.2	97.9	.237
1.5	355.00	1.534	122.7	.8045	22.3	100.4	.222
0.5	354.00	1.555	121.4	.8250	23.2	98.2	.236

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+25, 69 feet right of center line. Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 19 Dec 69
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	353.50	Elev. Tube Bottom	353.50
Standard Count Used	13563.8	Standard Count Used	12695.1
Reading on Cable	12	Reading on Cable	4
Operating Voltage	1400	Operating Voltage	750

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
13.5	367.00	1.652	117.0	.8929	26.7	90.3	.296
12.5	366.00	1.659	116.6	.8877	26.4	90.2	.293
11.5	365.00	1.528	122,8	.8127	22.7	100.1	.227
10.5	364.00	1.557	121.2	.8173	22.9	98.3	.233
9.5	363.00	1.517	123.3	.8324	23.5	99.8	.235
8.5	362.00	1.524	123.0	.8200	22.9	100.1	.229
7.5	361.00	1.518	123.3	.8613	24.9	98.4	.253
6.5	360.00	1.488	124.9	.8362	23.7	101.2	.234
5.5	359.00	1.583	120.1	.8136	22.7	97.4	.233
4.5	358.00	1.588	120.0	.8483	24.3	95.7	.254
3.5	357.00	1.568	120.8	.8391	23.8	97.0	.245
2.5	356.00	1.527	123.0	.8524	24.4	98.6	.247
1.5	355.00	1.513	123.6	.8450	24.1	99.5	.242
0.5	354.00	---	---	---	--	--	--

NUCLEAR DETERMINATIONS

SITE LOCATION: Tube at Station 1000+25, 69 feet right of center line.
Interstate Highway 30, Bowie County, Texas.

Date Readings Taken: 7 April 70
Person Taking Readings: John Wise

Moisture Data		Density Data	
Elev. Tube Bottom	353.50	Elev. Tube Bottom	353.50
Standard Count Used	14682.2	Standard Count Used	
Reading on Cable	12	Reading on Cable	4
Operating Voltage	1350	Operating Voltage	

Dist. From Tube Bottom	Elev.	Density Count Ratio	Wet Density lb/ft^3	Moisture Count Ratio	Water Density lb/ft^3	Dry Density lb/ft^3	Moisture Content
13.5	367.00	1.651	117.0	.919	27.9	89.1	.313
12.5	366.00	1.654	117.0	.915	27.7	89.3	.310
11.5	365.00	1.542	122.5	.850	24.4	98.1	.249
10.5	364.00	1.576	120.5	.842	23.9	96.6	.247
9.5	363.00	1.512	123.7	.844	24.0	99.7	.241
8.5	362.00	1.498	124.1	.824	23.3	100.8	.231
7.5	361.00	1.488	124.9	.856	24.7	100.2	.247
6.5	360.00	1.472	125.7	.838	23.7	102.0	.232
5.5	359.00	1.540	122.5	.841	23.9	98.6	.242
4.5	358.00	1.538	122.6	.902	27.0	95.6	.282
3.5	357.00	1.520	123.1	.889	26.5	96.6	.274
2.5	356.00	1.481	125.2	.885	26.3	98.9	.266
1.5	355.00	1.459	126.0	.863	25.0	101.0	.248
0.5	354.00	1.471	125.7	.875	25.7	100.0	.257

APPENDIX 5

OTHER INFORMATION AT TEST SITE

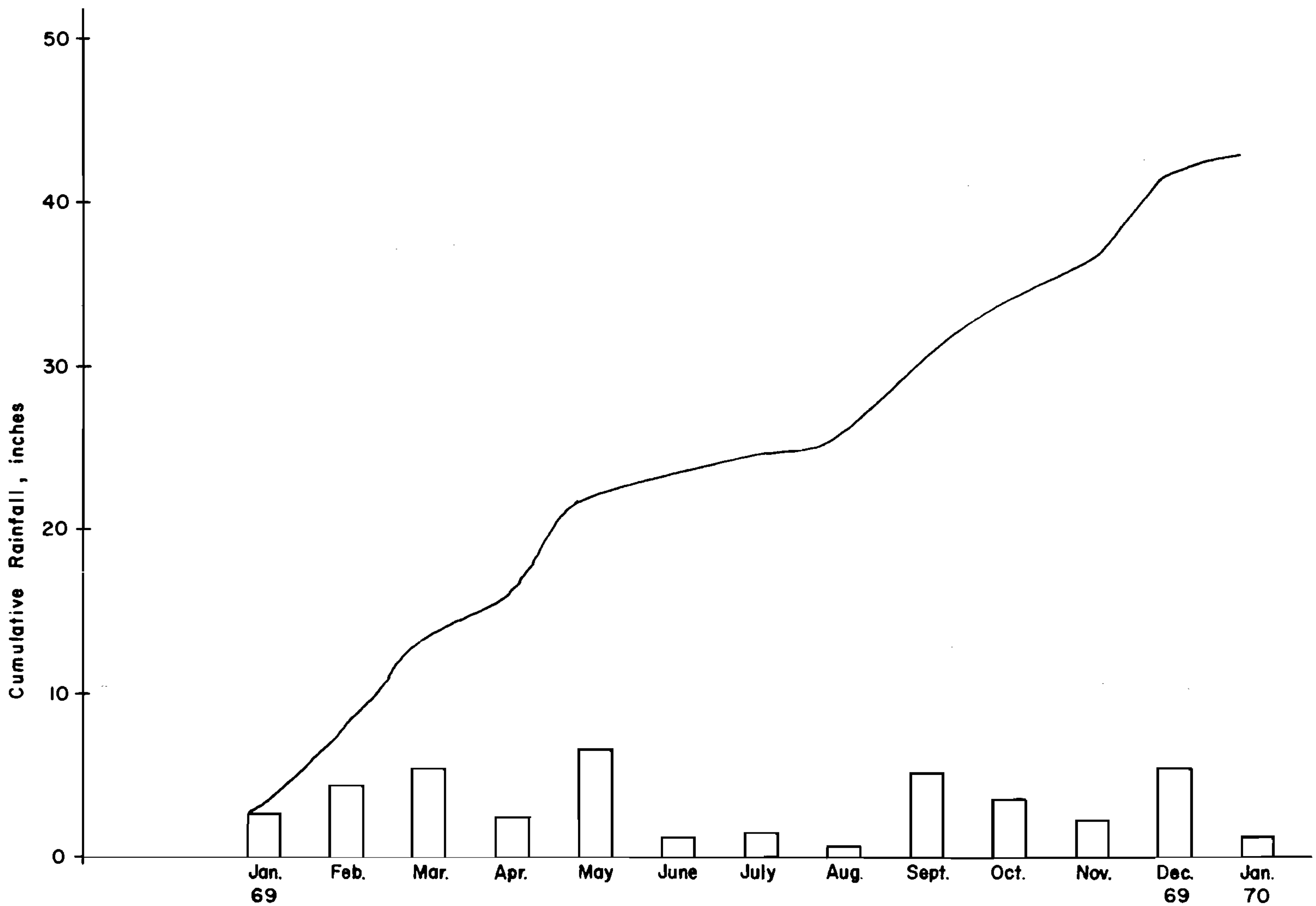

Fig A5.1. Rainfall at Atlanta test site.

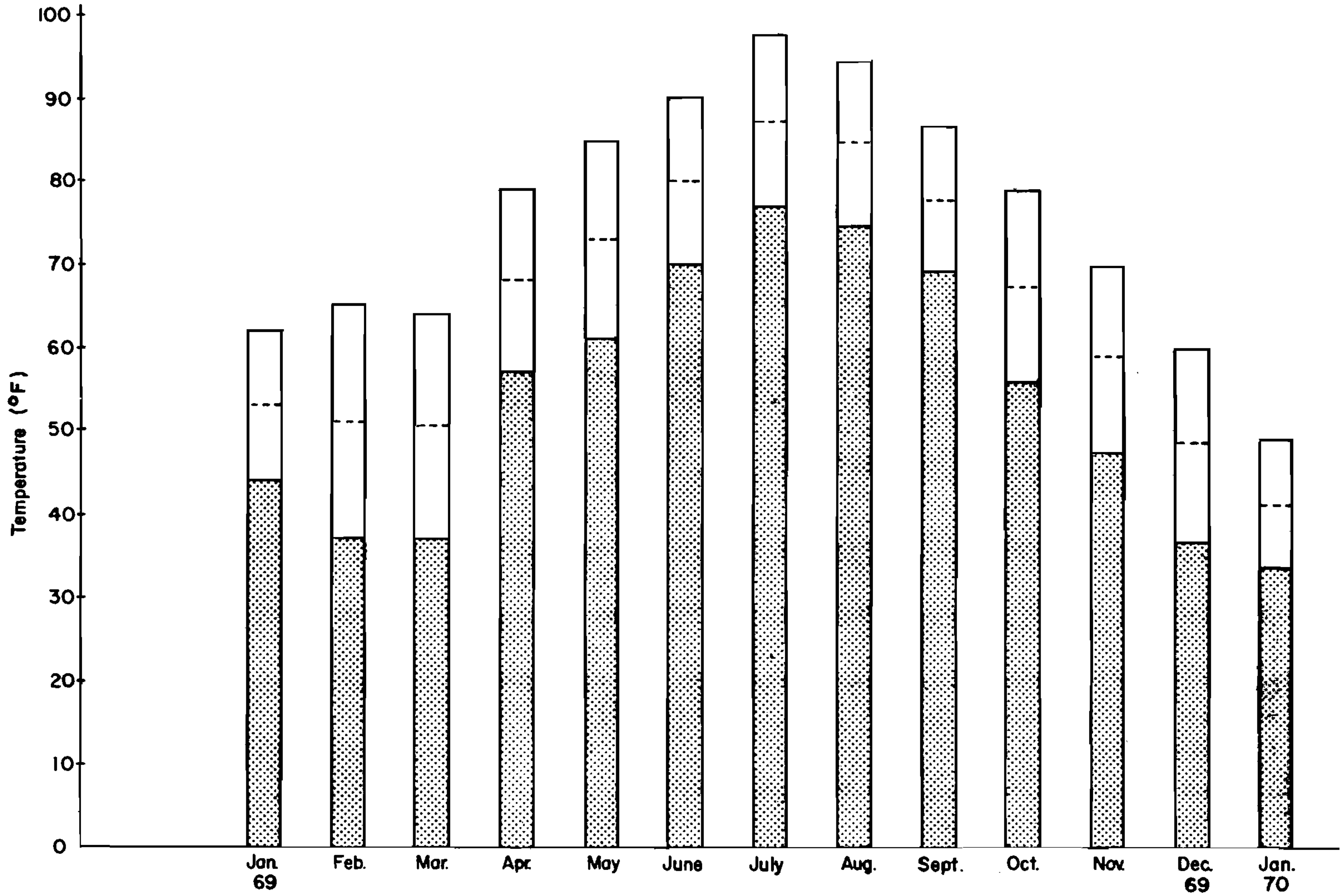

Fig A5.2. Average monthly temperatures at Atlanta test site.

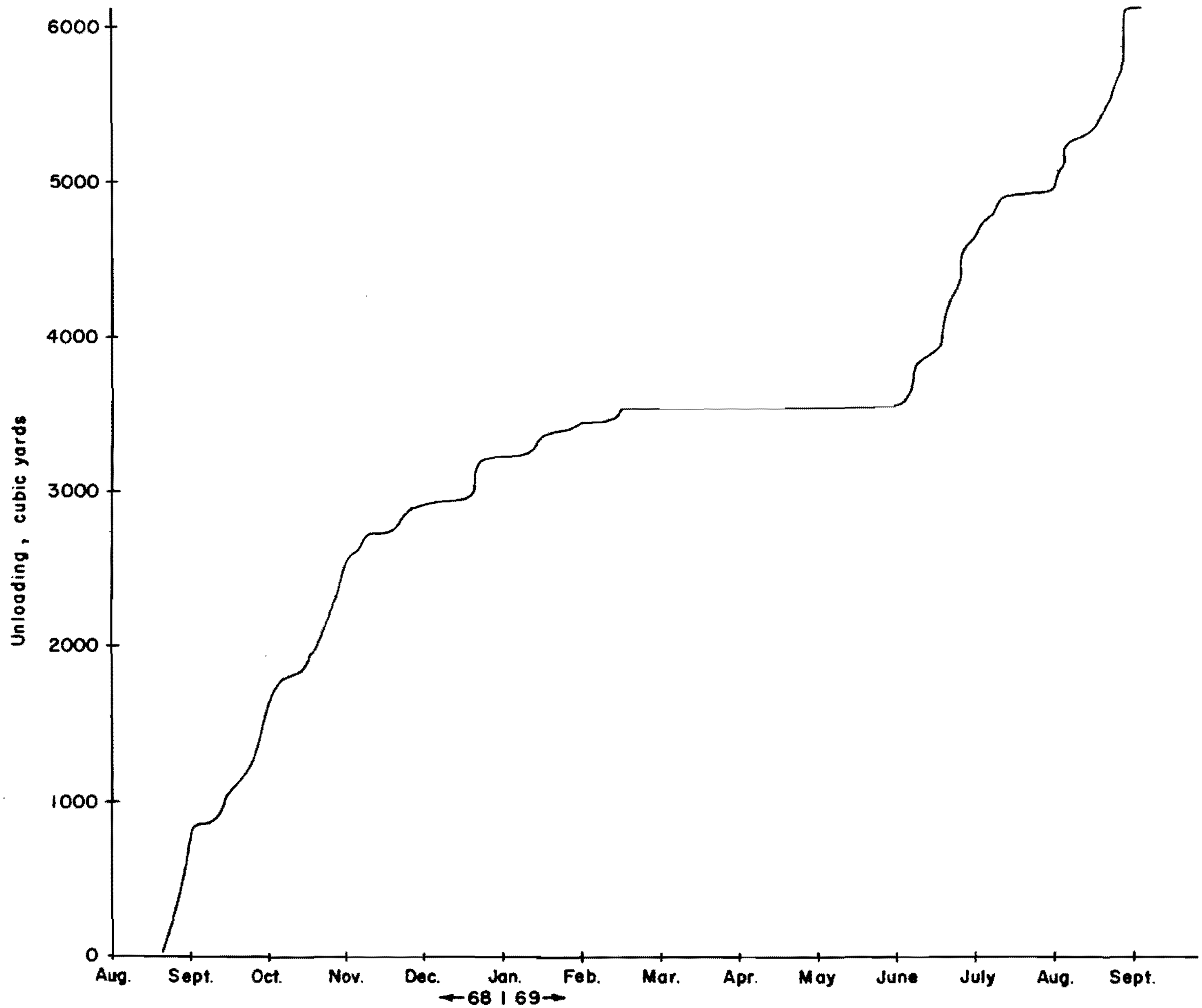

Fig A5.3. Unloading at Atlanta test site.

THE AUTHORS

John R. Wise is currently a 1st lieutenant in the U. S. Army Corps of Engineers, Fort Worth District, Texas. While at The University of Texas at Austin (1968-1970), he was a Research Engineer Assistant at the Center for Highway Research, where his research dealt with expansive clay foundation materials.

W. Ronald Hudson is an Associate Professor of Civil Engineering and Associate Dean of the College of Engineering at The University of Texas at Austin. He has had a wide variety of experience as a research engineer with the Texas Highway Department and the Center for Highway Research at The University of Texas at Austin and was Assistant Chief of the Rigid Pavement Research Branch of the AASHO Road Test. He is the author of numerous publications and was the recipient of the 1967 ASCE J. James R. Croes Medal. He is presently concerned with research in the areas of (1) analysis and design of pavement management systems, (2) measurement of pavement roughness performance, (3) slab analysis and design, and (4) tensile strength of stabilized subbase materials.