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16. Abstract

Observation of the behavior of a highly expansive clay, heavily fissured by dessication, have been made over extended periods. Measurements were made of soil heave under sustained ponding as a function of time and depth and thus the effects of progressive absorption of water into the clay were directly observed. Moisture content and density were monitored by nuclear probes. Attempts to employ double-junction Spanner-type psychrometers were generally unsuccessful in field observations of changes in soil suction although future use for research purposes in less highly fissured soils remains a possibility.

With adequate calibration, the nuclear equipment was found to be quite successful in monitoring changes in moisture content and density. The simple and inexpensive heave measurements points, pushed or driven into position, are recommended for regular use in observing movements in highway embankments and foundation soils.

The records are believed to furnish a unique set of data because of swell measurements throughout the depth of the affected soil. Although the tests were not primarily intended for evaluation of pre-swelling by ponding, they show that, in the type of clay tested, the effects of one or two months of ponding are primarily limited to wetting, and stabilizing, only the top five feet. Such an effect may be quite beneficial, but the continued movement of deeper layers must also be expected.

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by

John B. Stevens
Paul N. Brotcke
Dewaine Bogard
Hudson Matlock

Research Report Number 118-9F

Study of Expansive Clays in Roadway Structural Systems

Research Project 3-8-68-118

conducted for

Texas
State Department of Highways and Public Transportation

in cooperation with the
U. S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR HIGHWAY RESEARCH

THE UNIVERSITY OF TEXAS AT AUSTIN

November 1976

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

This report is the ninth and final one of a series dealing with a variety of aspects of swelling clays. The economic significance of the problem has been discussed in the fifth report in which it is pointed out that the total cost of pavement maintenance in Texas due to expansive clay soils, though difficult to evaluate, may be on the order of 10 million dollars per year.

The initiation of Project 118 in 1967 was in large measure due to the interest of Dr. R. L. Lytton in the development of comprehensive analytical methods for prediction of swelling clay behavior. Several different people collaborated in this effort and produced the first four reports.

The analytical procedures were essentially deterministic and were at a disadvantage because of the lack of well-documented cases where the performance of swelling clay was carefully observed and measured. Most of the documented knowledge of swelling clays either dealt with measurements and theory relating to physical and chemical properties of clay samples or with a few surface measurements and very general descriptive or statistical correlations of gross effects on which some very approximate methods of design have been recommended.

One very significant field study had been made at Waco, Texas, starting in 1957. The large volume of information had been stored and was in danger of being lost. Arrangements were made whereby Mr. Chester McDowell and his coworkers summarized this information as the seventh report of Project 118.

In the original planning of the project it had been intended that improved methods of field observation would be developed and evaluated before use on actual construction projects. However, some field-related needs arose earlier and considerable effort was diverted toward installations related to ongoing construction projects. The fifth, sixth, and eighth reports document the results.

The contributions of a large number of people have been identified in the various reports of the project. Throughout the program the administrative support of Dr. Clyde Lee has been invaluable. Particular thanks go to

Mr. Art Frakes and his staff in the Center for Highway Research for their excellent work and cooperation in prepartion of the reports.

John B. Stevens
Paul N. Brotcke
Dewaine Bogard
Hudson Matlock

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 partical 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.

Report No. 118-6, "Measurements of a Swelling Clay in a Ponded Cut," by W. Gordon Watt and Malcolm L. Steinberg, reviews the use of ponding as a solution to the problem of swelling clays and presents the procedures used and results obtained to date from a ponding project conducted in 1970 in San Antonio, Texas.

Report No. 118-7, "The Waco Ponding Project," by Robert L. McKinney, Jr., James E. Kelly, and Chester McDowell, presents results of 1957-72 field studies concerned with the effectiveness of ponding and lime stabilization of clay subgrade in minimizing volume change beneath portland cement concrete pavements.

Report No. 118-8, "Continuing Measurements of a Swelling Clay in a Ponded Cut," by Malcolm Steinberg, brings up to date the results of studies of a ponded cut in an expansive clay in Bexar County.

Report No. 118-9F, "Observation of an Expansive Clay Under Controlled Conditions," by John B. Stevens, Paul N. Brotcke, Dewaine Bogard, and Hudson Matlock, discusses experiments made to develop and evaluate techniques for field measurements of expansive clay behavior.

SUMMARY

Observations of the behavior of a highly expansive clay, heavily fissured by dessication, have been made over extended periods. Measurements were made of soil heave under sustained ponding as a function of time and depth and thus the effects of progressive absorption of water into the clay were directly observed. Moisture content and density were monitored by nuclear probes. Attempts to employ double-junction Spanner-type psychrometers were generally unsuccessful in field observations of changes in soil suction although future use for research purposes in less highly fissured soils remains a possibility.

With adequate calibration, the nuclear equipment was found to be quite successful in monitoring changes in moisture content and density. The simple and inexpensive heave measurements points, pushed or driven into position, are recommended for regular use in observing movements in highway embankments and foundation soils.

Total surface swell in the order of 10 inches was observed, with movement continuing because of increasing moisture contents at depths greater than 10 feet below the soil surface. Very rapid infiltration of moisture occurred to depths of 5 feet because of open cracks and apparently greater permeability due to debris in closed fissures. These effects precluded correlation with prior analytical developments.

The records are believed to furnish a unique set of data because of swell measurements throughout the depth of the affected soil. Although the tests were not primarily intended for evaluation of pre-swelling by ponding, they show that, in the type of clay tested, the effects of one or two months of ponding are primarily limited to wetting, and stabilizing, only the top five feet. Such an effect may be quite beneficial, but the continued movement of deeper layers must also be expected.

IMPLEMENTATION STATEMENT

The records of performance of a highly active central Texas clay under sustained ponding should be of considerable benefit in furnishing general appreciation and understanding of extreme soil movement due to swell. The observations of progressive moisture change at various depths gives a clear picture of the process. Limitations of the benefits of pre-construction ponding are better understood.

The data will be useful for future correlation with predictive techniques that may be developed; the methods developed to date are hampered by the extreme variations in effective permeability of a highly fissured soil.

Nuclear observations of changes in moisture and density will be of continuing potential use and the simple and inexpensive soil movement measuring techniques should find frequent application for observing swelling clays as well as settlements and movements under other circumstances.

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CHAPTER 1. INTRODUCTION

Several methods for predicting moisture movement and potential heave of expansive soils have been proposed. However, there has been a significant shortage of field data for evaluation of these predictive methods. Therefore, a field test involving observation of an expansive clay under controlled conditions was initiated at Lake Long, Austin, Texas, in June 1973.

The Lake Long experiment was developed for two purposes: (1) to facilitate development and evaluation of methods of field observation and (2) to provide data on the behavior of a swelling clay under controlled conditions. It was hoped that such data might serve in evaluation of predictive methods developed earlier in this project (Refs 1 and 2). In any event, the observation of changes as a function of time and depth would provide a more thoroughly documented case history than had heretofore been available.

Laboratory testing of the clay was performed to obtain information required by the predictive methods. This report presents the methods and results of the field observations, laboratory testing, and an evaluation of the applicability of predictive techniques to the type of clay studied.

The experimental site and properties of the soil at the site are described in the next chapter. In Chapter 3, the instrumentation developed for this project and the methods of installation are presented. Various methods were employed for the measurement of water content and density changes at depth, surface and subsurface heave, and suction changes. Instrumentation layout and depths, site history, and preparation are presented in Chapter 4.

Chapter 5 discusses the experimental results. These results are believed to be unique in their completeness and in the length of time of observation. A comparison with McDowell's Potential Vertical Rise Method is also given. Recommendations are given in Chapter 6 for future utilization of the techniques developed.

CHAPTER 2. DESCRIPTION OF TEST SITE

Earlier in this project some field measurements were made at Atlanta and San Antonio, Texas, in cooperation with the State Department of Highways and Public Transportation (Ref 3). Although useful information was obtained, the sites were not satisfactory for development of experimental methods because of their distance from Austin and problems associated with making measurements during construction operations. To avoid these problems a search was initiated for a more suitable site, preferably within twenty miles of Austin. Other characteristics desired were a homogeneous expansive soil, flat terrain, shallow or nonexistent overburden, long-term availability without interference, and an ample water supply.

Location

The site selected was at Lake Long, seven miles east of Austin. Lake Long is a man-made cooling-water reservoir for a City of Austin electrical cooling plant. The site was on a peninsula to which the overland access was controlled by the City of Austin. Borings for construction of the power plant and the impounding dam showed the soil to be a thick deposit of expansive clay. After obtaining permission from the City of Austin to use the area, an exploration program was initiated at several potential sites. The site shown in Fig 2.1 was selected and further borings were made to provide information on in-situ conditions and samples for laboratory testing.

Geology

The expansive clay at the site is the weathered portion of a member of the Taylor Group. The Taylor Group is part of the Gulf series and was deposited in Late Cretaceous time in a relatively shallow sea. Named "Taylor Marl" by R. T. Hill in 1891, the Taylor Group was subsequently divided into three formations from oldest to youngest, the Sprinkle, Pecan Gap, and Bergstrom formations. The last is the formation at the test site.

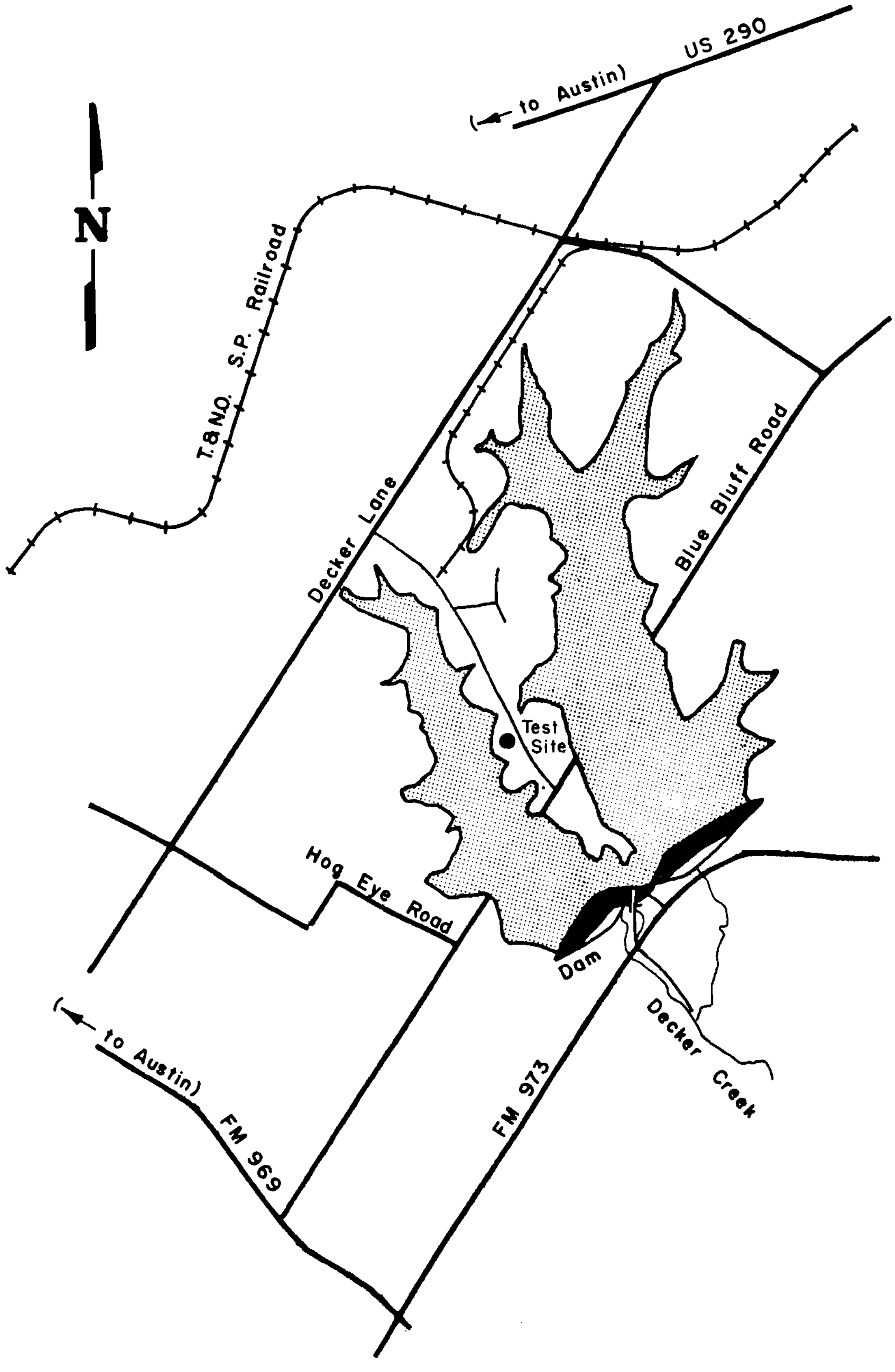


Fig 2.1. Test area at Decker Creek power generator plant.

The Bergstrom formation ranges from 330 to 380 feet in thickness. Unweathered, the formation is a gray to greenish-gray, unctuous, slightly fissle, montmorillonitic clay. Near the surface, the formation weathers to a yellowish-tan to tan clay. The color change is caused by the oxidation of the iron content of the clay. Large, repeated changes in water content and chemical alteration have developed a very stiff, highly-fissured material.

In the weathered zone, the approximate composition is 60 percent clay, 30 percent calcium carbonate, and 10 percent quartz. The approximate composition of the clay portion is 7 percent kaolinite, 5 percent illite, 69 percent Ca-montmorillonite, and 19 percent Na-montmorillonite (Ref 4).

In-Situ Soil Conditions

A typical soil profile is presented in Fig 2.2. The top 4 to 5-foot layer is a heavy, dark-gray topsoil with imbedded gravel and cobbles. Below this to a depth of approximately 40 feet is the weathered Taylor Marl. Underlying this is the hard, gray, unweathered Taylor Marl. There is no observable ground water table in the Bergstrom formation.

In-situ water content of the upper 3 to 5 feet of the profile is dependent on the very recent weather conditions. The water content fluctuates from less than 10 percent to more than 40 percent near the surface. During extended periods of dry weather, extensive shrinkage cracking takes place. Cracks 3 inches wide have been observed. Excavations have shown that the cracking in severe droughts has possibly extended to 20 feet. At the time of the reported field experiments, the in-situ water content beyond a depth of 5 feet ranged from 18 to 22 percent.

Soil Properties

Properties of the weathered Taylor Marl are presented in Table 2.1 and Fig 2.3. The suction versus water content relationship was determined using a suction-plate apparatus, vacuum dessicator, and psychrometric techniques.

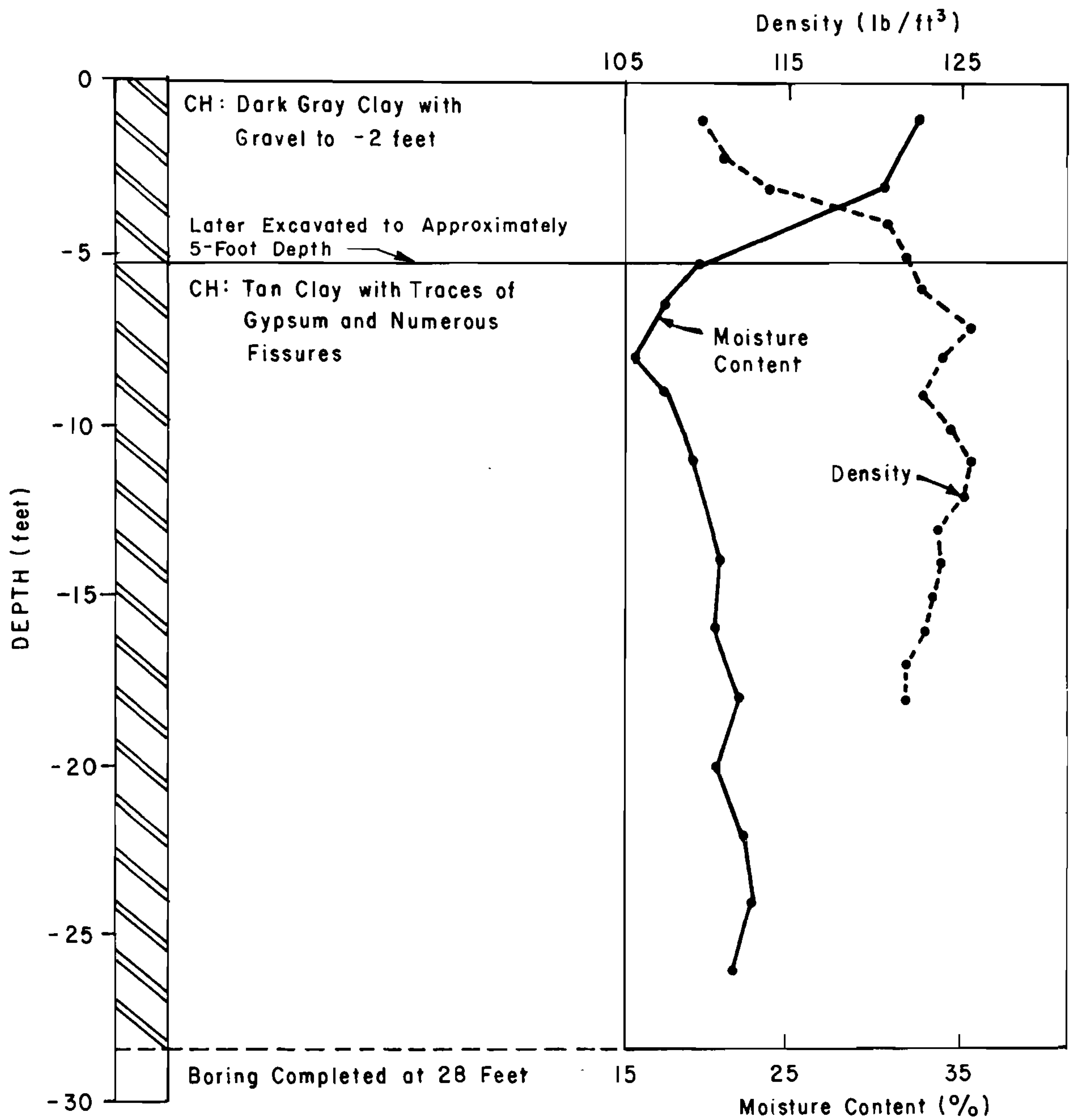


Fig 2.2. Soil profile of test area: moisture contents are from samples taken during boring for bench mark 1, densites are from nuclear access tube 3.

TABLE 2.1. PROPERTIES OF TAYLOR MARL

	Range, %
Liquid limit	57-70
Plastic limit	17-29
Shrinkage limit	10-14
Passing #200 sieve	98-99+

Climate

The following comments are taken from "A Climatological Summary of Austin, Texas" (Ref 5).

> The climate of the Capitol City may be characterized as a warm, humid and subtropical one. Weather records of the National Weather Service (formerly the U.S. Weather Bureau) for Austin for the past 50 years show that a wide range of temperatures frequently occurs in the winter, while summers are typically hot.
>
> * * *
>
> Rainfall is fairly evenly distributed throughout the year. The most abundant rainfall occurs in the spring of the year, with May being the wettest month. Driest weather usually occurs during the summer and late winter. Most of the rainfall during the warm season (April through September) results from thundershower activity, some of which produces heavy amounts over short periods of time. Consequently, a significant portion of the yearly rainfall is usually lost to the soil because of too rapid run-offs. Although thunderstorms occur frequently during the cooler half of the year, much of the winter precipitation occurs as light rain.
>
> * * *
>
> In some years during the late summer and early fall, dissipating tropical storms bring strong winds and heavy rains to the city.

Temperature and rainfall data during the period of the test program are given later, in Fig 4.6. Extremely heavy rains, such as mentioned in the last sentence quoted above, occurred during preparation of the test site in the fall of 1973.

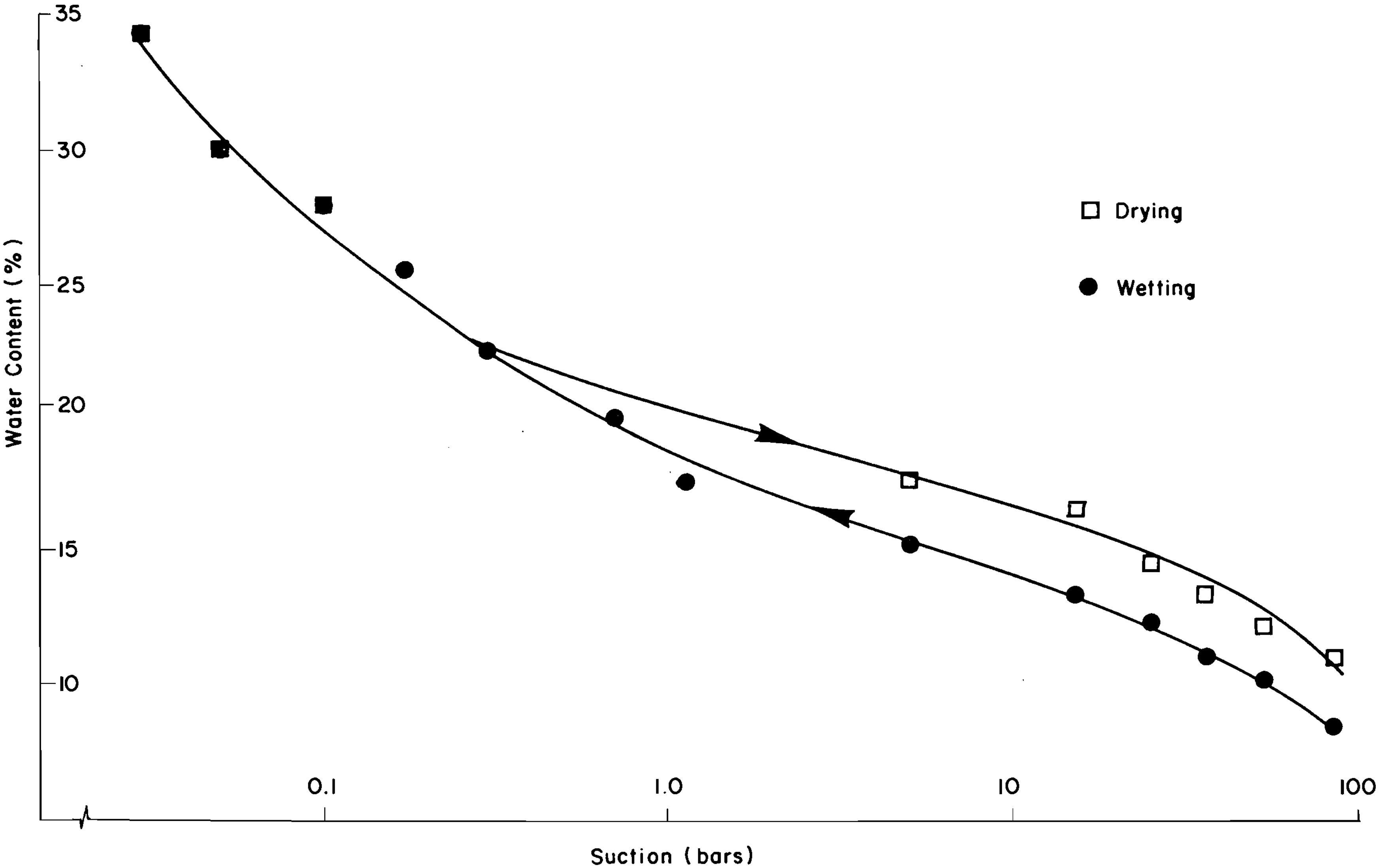


Fig 2.3. Suction versus water content.

CHAPTER 3. INSTRUMENTATION

Instrumentation was developed and installed to measure changes in water content, density, and suction at various depths in the soil for comparison with the corresponding heave as it occurred with time under controlled and known conditions.

Conventional soil sampling and moisture-content determinations were made to provide initial values and to serve as a basis for comparison. Nuclear methods were used to monitor changes in moisture content and density and psychrometer techniques were employed in suction observations. Movements of the clay were monitored with heave indicators driven into the soil by a follower tube. This chapter describes the instrumentation and its method of installation and calibration.

In-Situ Moisture Content and Density Measurements

In-situ moisture content and density measurements were made with nuclear equipment manufactured by the Troxler Electronics Laboratory, Inc., of Raleigh, North Carolina. The equipment used was

(1) a Troxler Model 1651 portable scale-ratemeter with a five-decade readout,

(2) a Troxler Model 104 depth-moisture gauge with standard, and

(3) a Troxler Model 504 depth-density gauge with standard.

To determine a water content or density profile, the appropriate nuclear gauge is connected to the scale-ratemeter and inserted into a previously installed access tube.

Nuclear Access Tube

Class 150 aluminum irrigation tubing with a 2.00-inch outside diameter and a 1.90-inch inside diameter was used as access tubing as recommended by Troxler Electronics Laboratory. It is readily available and has uniform nuclear properties.

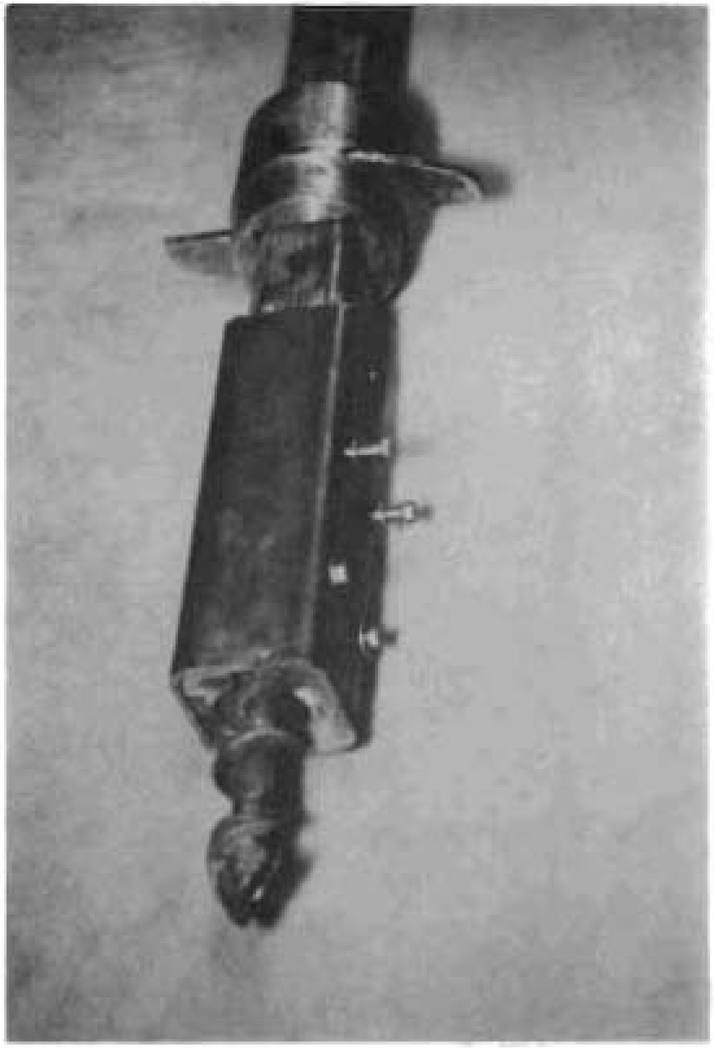
For reliable readings from the nuclear gauges, no air voids can be tolerated between the soil and access tubing. Thus the hole augered for the access tube must be straight and initially only slightly larger in diameter than the tubing. Experience at other test sites had shown that a 2.1-inch auger in 3-foot flights bored an oversized hole. This was caused by the lack of rigidity at the joints.

To overcome this problem, a special 2.1-inch auger flight, 15 feet long, was purchased from the Haynes Manufacturing Company, Lufkin, Texas. Special equipment was fabricated to use this auger with a drilling rig. The standard kelly was replaced with one fabricated from 4-inch-square structural tubing and the auger was housed inside this kelly. To apply torque to the auger, a clamp was attached at the bottom of the kelly as shown in Fig 3.1. The clamp has one fixed and one movable jaw, which are drawn together by four bolts and grip the auger. With the auger clamped, torque can be applied via the kelly from the rotary table of the drill rig.

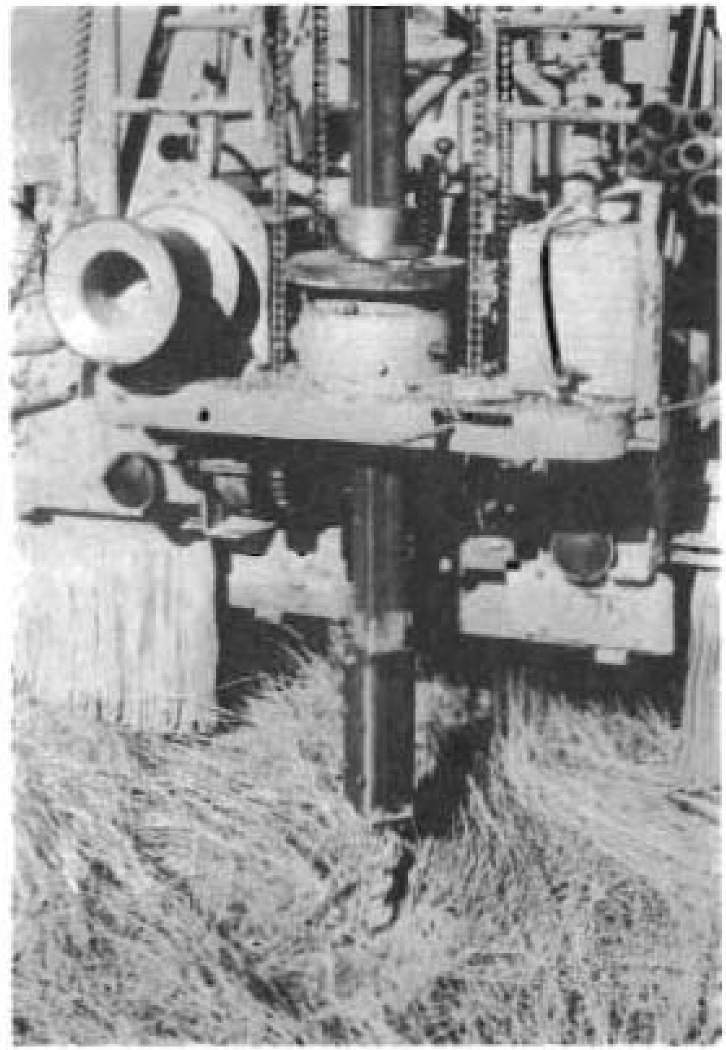
In use, the auger is gripped about 2 feet above the ground and then the boring is advanced until the clamp is near the ground surface. The auger is then unclamped, the kelly is raised about 2 feet, and the auger is gripped again. For removal of the auger, the procedure is reversed, but without rotation of the auger.

Prior to placement of the nuclear access tubes into the hole, an aluminum plug with O-ring seals was inserted in the bottom end. This was intended to prevent the entry of water into the tube and to provide a clean bottom. Even with this plug, water did enter some tubes during flooding of the test areas. This problem was corrected by inserting water-pressure test seals and expanding them at the bottom of installed tubes. On all tubes subsequently installed, the joint between the plug and the tubing was sealed with silicone aquarium sealer. Both methods have worked satisfactorily.

Nine months after the installation, the soil squeezed three of the access tubes sufficiently to prevent passage of the probe. The access tube was restored to its original inside diameter by inserting and removing a 1.90-inch-diameter mandrel. The problem did not recur.



(a) Clamp for 2-inch-diameter auger



(b) Field use of continuous auger, housed inside kelly

Fig 3.1.

Calibration

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

Also, gravimetric moisture content samples were taken from the boring for each tube. After installation of the access tube, moisture content and density measurements were made with the nuclear equipment. The nuclear measurements determine the moisture content and wet density in pounds per cubic foot. Both values are needed to compute the moisture content in the familiar gravimetric form for comparison with the soil samples. This comparison is presented in Fig 3.2. The good agreement means that, by recalibration of the nuclear equipment, an accurate picture of in-situ conditions can be obtained.

Heave Indicators

Measurement of surface heave is basic to any field study of swelling clays. For this study, subsurface heave measurements at various depths were also desired. These data were needed to determine the source of the heave at different times. A simple subsurface heave indicator system was devised to obtain these measurements and is shown in Fig 3.3. The advantages of this system are its low cost, short installation time, and minimum soil disturbance.

The heave indicator is a 9/16-inch-diameter steel point with a 23-gauge stainless steel wire or cable attached. A socket was machined into one end for insertion of a follower or push tube. The wire was attached to the steel point with a silver-soldered connection. After the soldered connection was made, 1/16-inch teflon tubing was slipped over the wire and clamped with a set screw.

Heave Indicator Installation

The technique for installing heave indicators was the same as that used to place psychrometer access tubes,discussed later. To install the heave indicator, 1/2-inch-diameter thick-walled steel tubing was used as a push

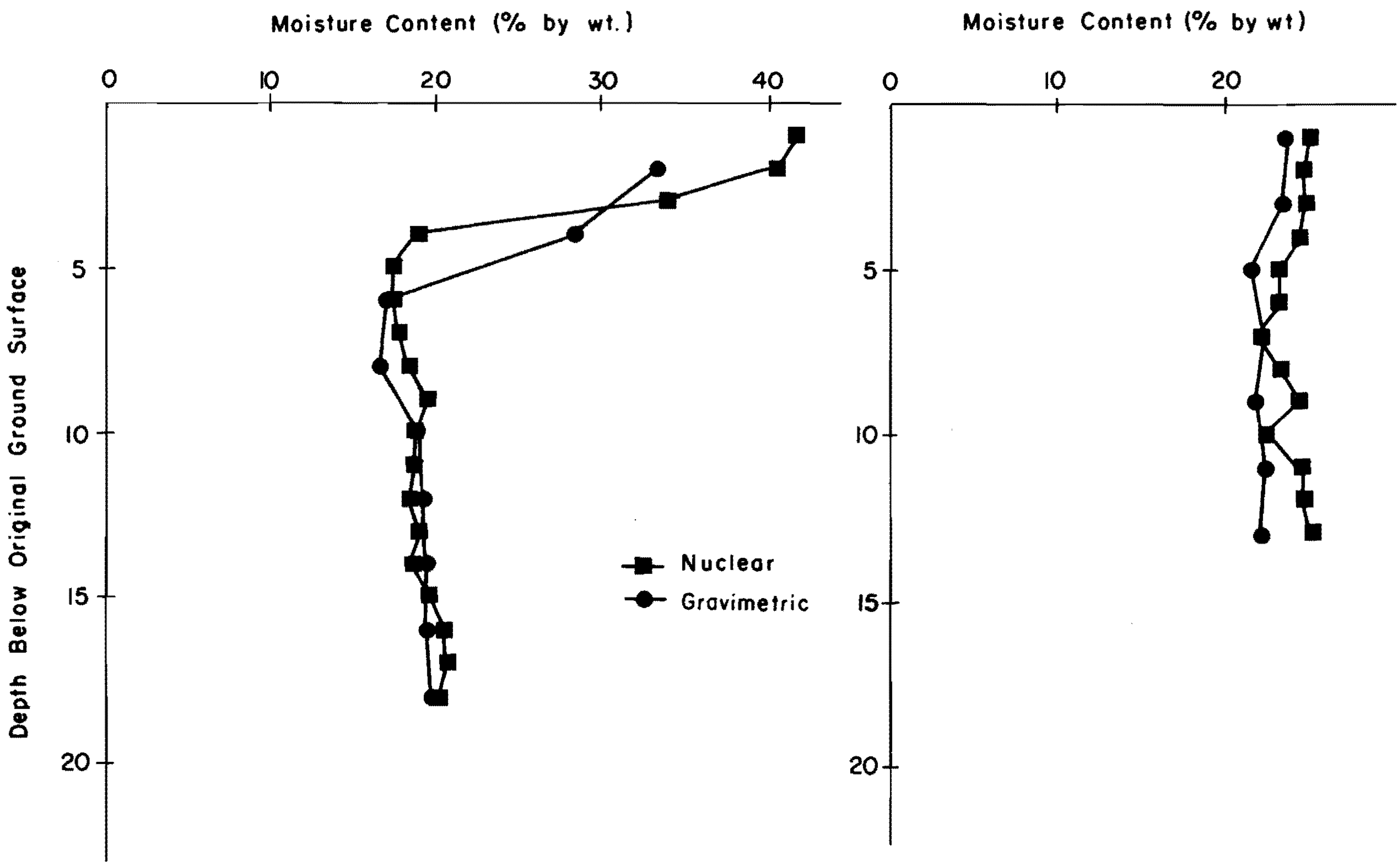


Fig 3.2. Comparison of nuclear measurements and gravimetric water contents.

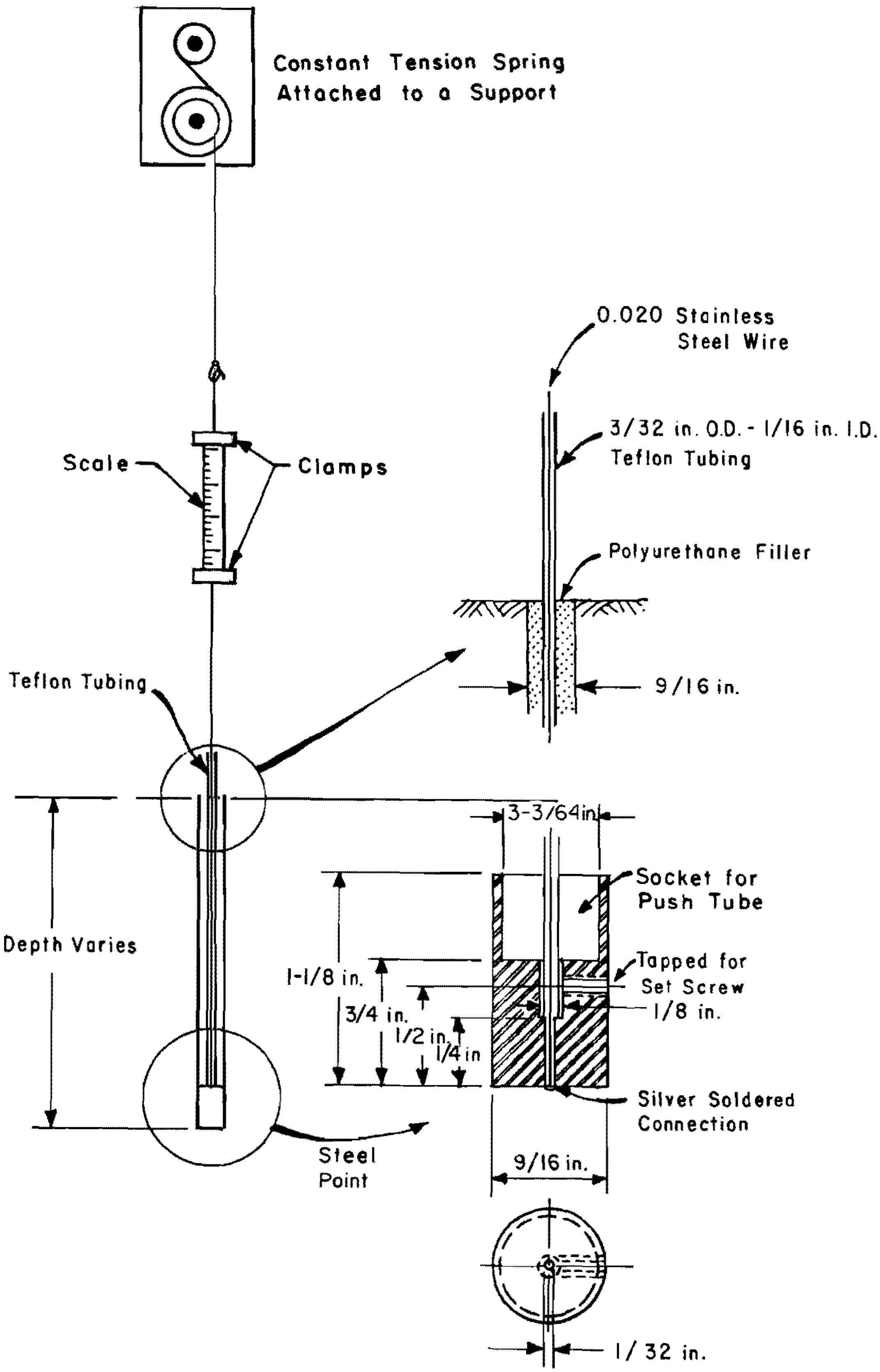


Fig 3.3. Heave indicator assembly.

tube. Before pushing the tube to depth, the wire with the teflon tubing was threaded through the push tube and the heave indicator socket was slipped over the tube. The heave indicator was then ready for emplacement.

The push-down force was supplied by either a 15-lb drop hammer or a special kelly mounted on a drill rig. This force was applied to the push tube through a clamp manufactured for gripping prestressed cables in concrete construction. These wedge-type clamps grasp the push tube wherever desired and can easily be released and shifted along the push tube as driving progresses.

For heave indicator points which were installed to depths of 5 feet or less, the 15-lb drop hammer was used. The weight, which had a 3/4-inch axial hole, was slipped over the push rod and dropped onto the cable clamp. When the heave indicator was at the desired depth, the weight was removed and the push tube withdrawn.

Heave indicator points were installed to depths of 15 feet by using the special push-down kelly shown in Figs 3.4 and 3.5a. The significant feature of this kelly is the 24-inch (0.61-m) centering sleeve which insures that the push tube is centered and remains axially aligned. The pull-down mechanism of the drill rig forces the kelly down against the prestress clamp and pushes the heave indicator point into the soil. During pushing, strokes were limited to 6 to 12 inches (15 to 30 cm) to prevent buckling of the push tube.

The kelly was also used to extract the push tube after the point had reached the desired depth. This was accomplished by using a pulling yoke which snapped over studs welded onto the kelly. Prior to pulling, the cable clamp was reversed and then the kelly was raised to pull against the clamp and extract the tube.

After removal of the push tube, the resulting hole was filled with a two-part polyurethane sealant to prevent the entry of water during flooding. Two products were used, Pecora Chemical Corporation NR-200 and Gibson-Holmans Company Eternaflex. When mixed to the manufacturer's specifications, the sealant had the viscosity of molasses, which allowed it to be poured down the hole. After approximately 24 hours, it polymerized to the consistency of very soft rubber.

After installation, the wire and teflon casing were cut, leaving approximately 10 feet (3 m) extending above the ground surface. A 12-inch (30.5-cm) machinists's rule with a hole at each end was threaded onto the wire. The

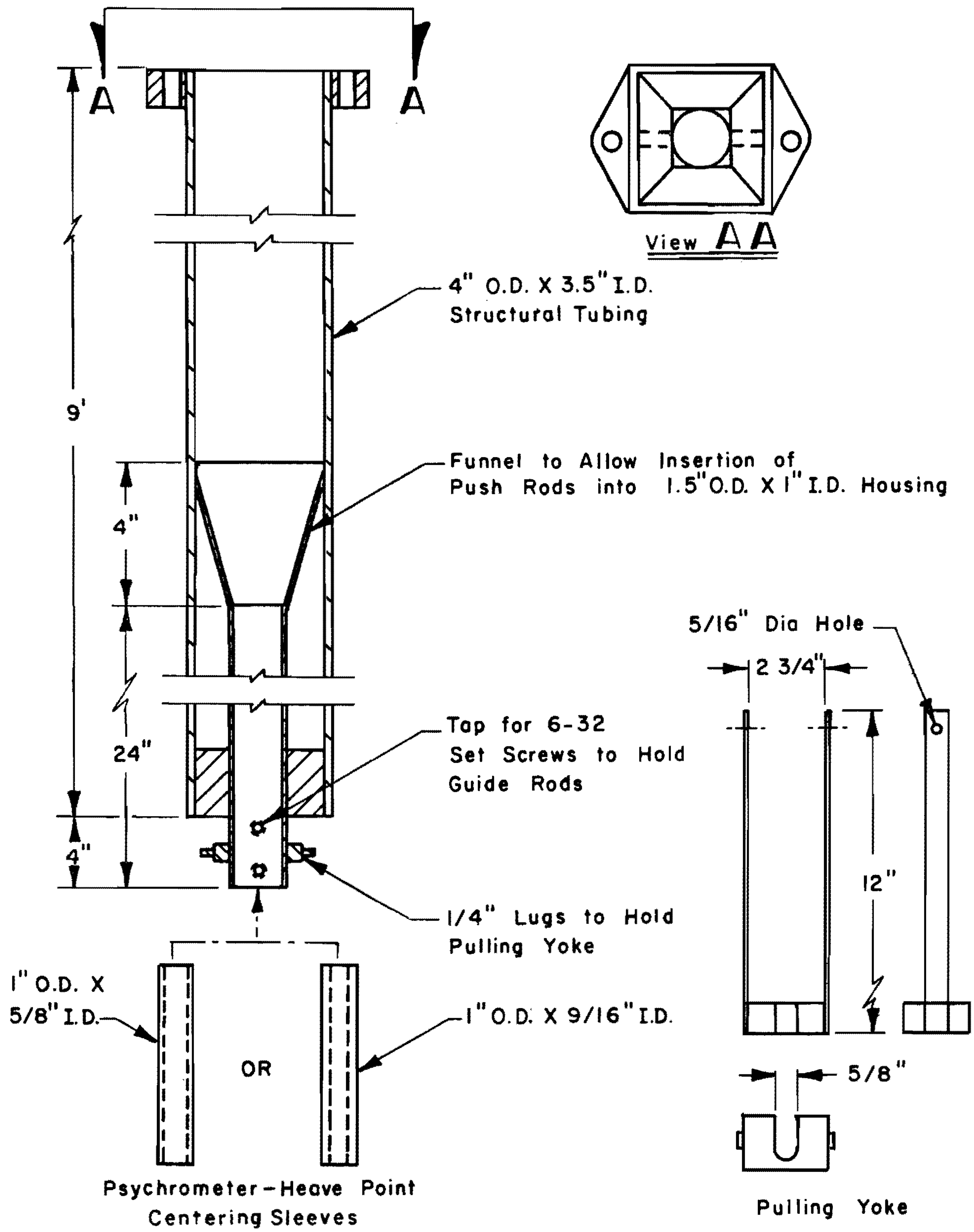
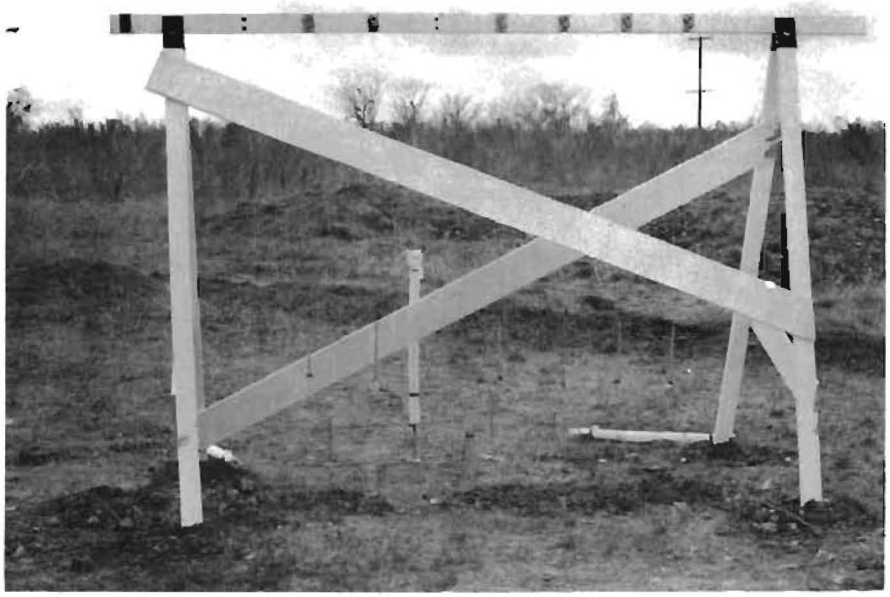


Fig 3.4. Push-down kelly assembly.

(a) Method of driving heave points and psychrometer access tubes.



(b) Complete instrumentation array.

Fig 3.5.

rule was secured to the wire by two pairs of very small clamping plates drawn together by small screws. Next a fishhook was threaded on the end and a permanent loop was formed above the rule, using a crimp connector. The fishhook was used to connect the wire to the cable from a constant tension spring on a support structure when readings were being made. At other times, the wire was disconnected to prevent breakage.

Figure 3.5b shows an array of seven heave indicators in position for reading. The support structure for the constant tension springs mounted on the crossbar is a simple 2 × 4 frame with 1 × 8 cross-bracing. Not visible in the picture are sand-cement footings molded around the bottom of the legs. These footings were found to be necessary to prevent toppling of the support structure by gusts during high winds.

Installation Pushing Forces

In one case, the pushing force necessary to install the heave point and psychrometer access tubes (discussed later) was measured with a load cell positioned between the push-down kelly and the cable clamp. As shown in Fig 3.6, the pushing force in the upper 4 feet (1.2 m) was low but increased to 4,780 pounds (2168 kg) at a depth of 14.5 feet (4.42 m). Emplacement was halted at this depth because the push tube yielded below the clamp.

Bench Marks

Four deep benchmarks were installed to provide an elevation datum for heave measurements. The benchmarks were installed in 28-ft (8.5-m)-deep, 4-inch (10-cm)-diameter borings. This depth is greater than the zone of seasonal moisture change. As shown in Fig 3.7, 2-inch (50-mm) PVC pipe open at the bottom was inserted in each boring and grouted in place with a high-slump sand-cement mortar. This casing extended to approximately 1 foot (.3 m) above the ground surface. A 1-inch (25-mm) wood auger bit welded to a pipe union was then screwed onto the bottom of a 30-ft (9-m) length of 3/4-inch (19-mm) heavy-duty steel electrical conduit. This conduit is the inner rod and was inserted into the casing and rotated so that the wood bit was fully screwed into the soil at the bottom. When installed, the top of the inner rod was approximately 2 feet above the ground level. The top of the inner rod had

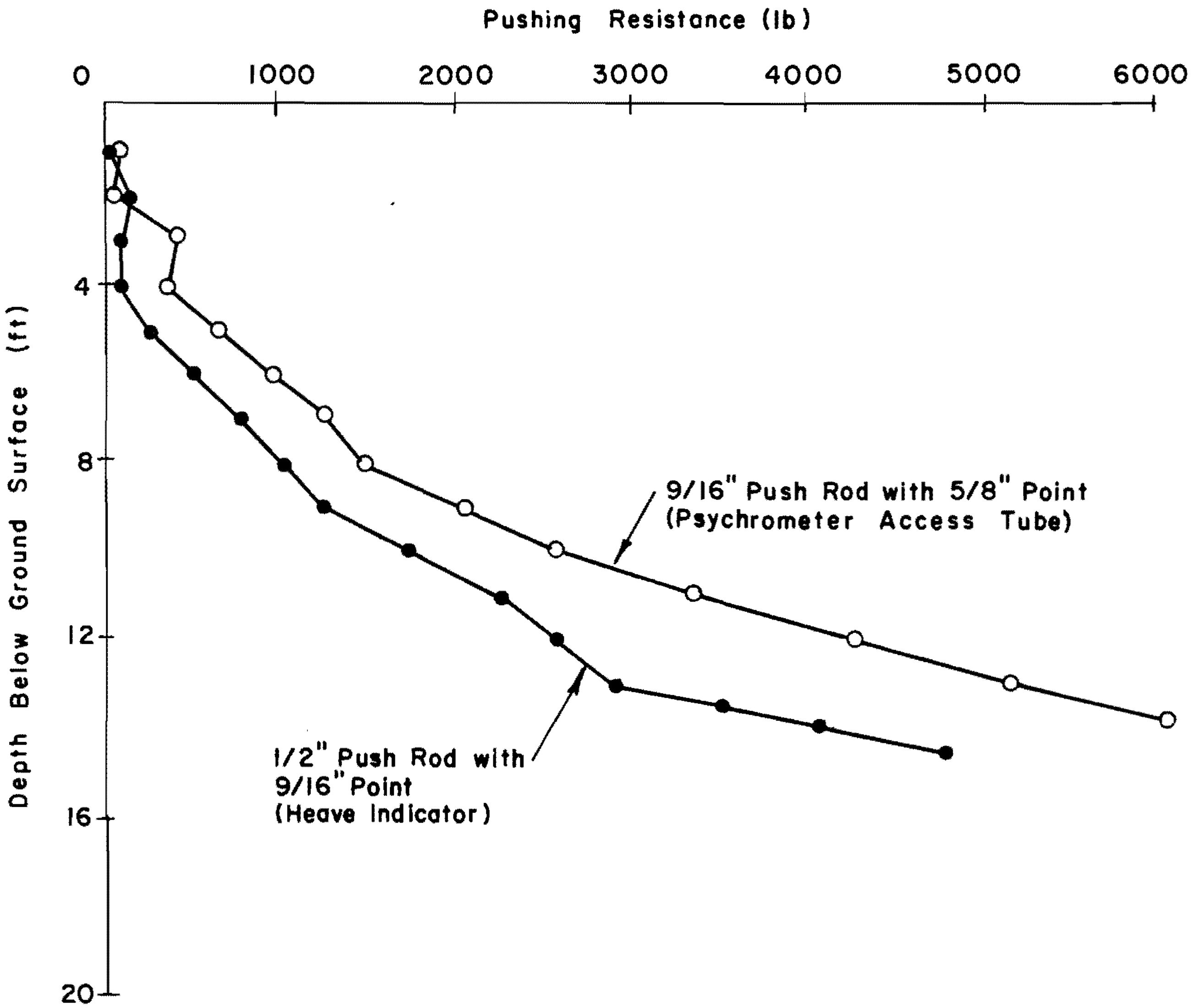


Fig 3.6. Pushing force required for 1/2 and 9/16-inch push rods.

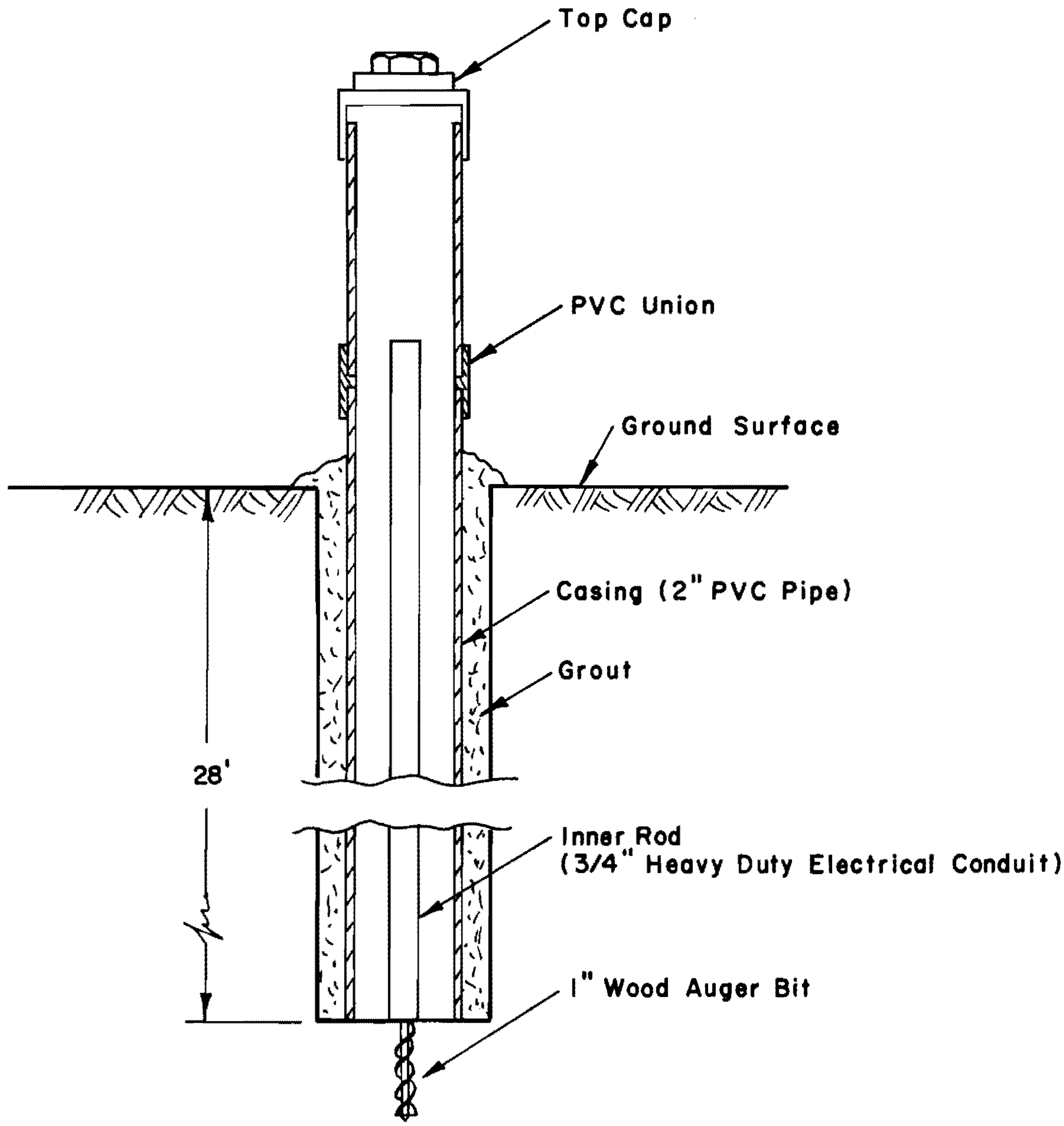


Fig 3.7. Bench mark schematic.

been turned square prior to emplacement. The casing was then extended with a 2-ft (0.6-m) section of PVC pipe and topped with a threaded cap.

To utilize the bench marks, four level rods, illustrated in Fig 3.8, were fabricated from the 3/4-inch (19-mm) electrical conduit. First the lower end was turned square and then a steel rod was inserted and held in place by Allen screws. This insured that the rods were properly mated to the bench marks during reading. A 12-inch (30.5-cm) machinist's rule like that used with the heave indicators was bolted on the level rod. One level rod was made for each bench mark.

Bench Mark Elevation

The bench marks were not tied into any existing level net. Instead, the top of the inner rod of bench mark 4 was assigned an elevation of 100 inches. Using it as a datum, the elevations of the tops of the other rods were determined in the following manner. First, the distance from the machined end of each level rod to the zero of the scale was measured; that distance is shown as a "known" distance in Fig 3.8. Next, the level rods were inserted into their respective bench mark inner rods. A series of level traverses was made, and the elevations of the other three bench marks were determined from these traverses.

Heave Measurement

Normal leveling procedures were used to read the heave indicators. Initially a Gurley Dumpy level was used, but it was not adequate to read the machinist's rule with sufficient resolution and precision. It was replaced with a Wild N-3 precise level which is capable of performing first order leveling.

For convenience, a permanent level mount was installed at the center of the instrumented area. At this location the foresights and backsights were approximately equal. The mount was a base plate with a threaded connection for a 3-inch (7.6-cm) pipe. The lower 7 feet (2 m) of a 10-ft (3-m) length of a 3-inch (7.6-cm) pipe was grouted in a 4-inch (10-cm) bore hole to serve as a support for the base plate.

When the heave indicators were read, normal leveling procedures were used. An average line-of-sight elevation was determined by backsighting on

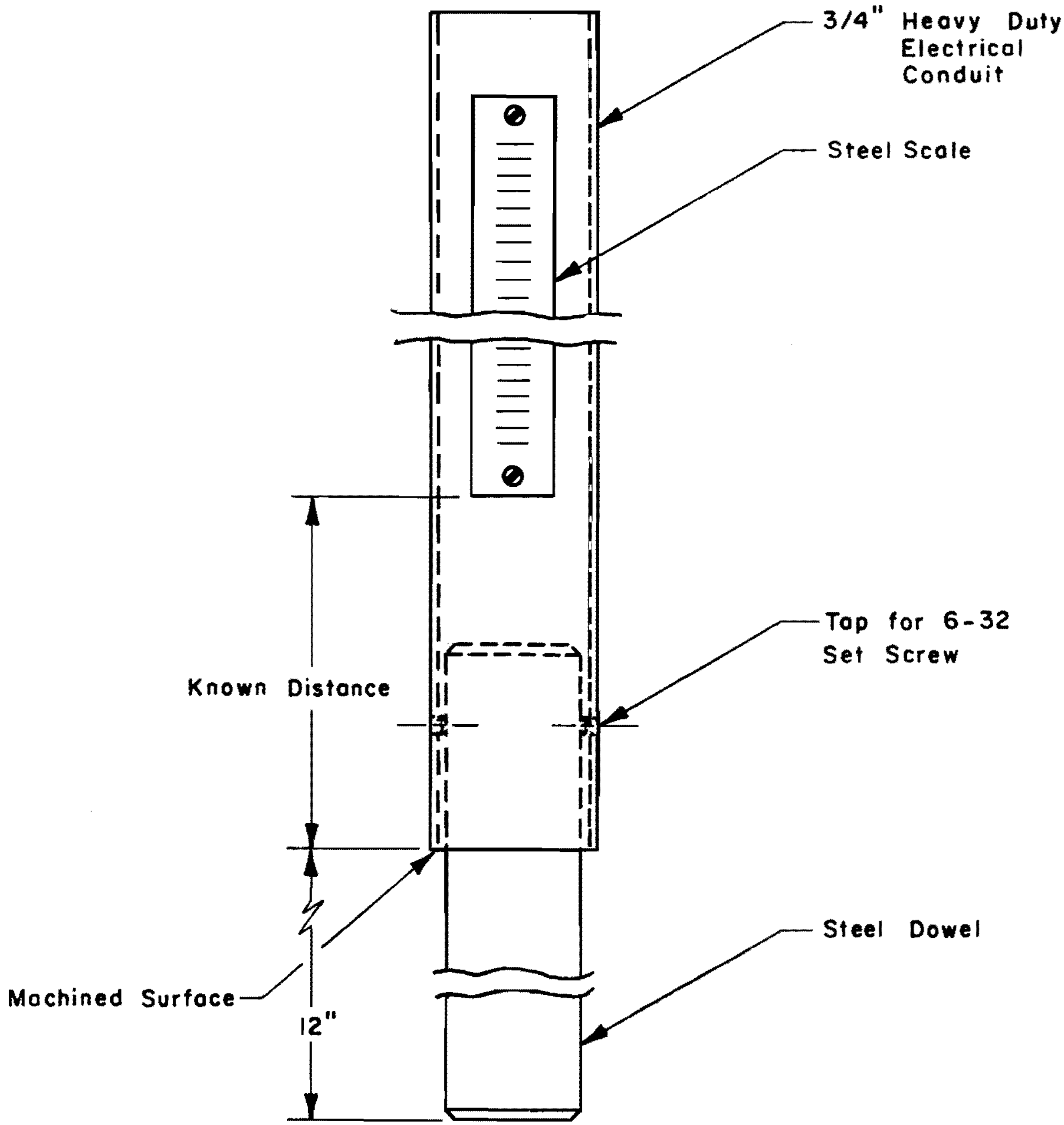


Fig 3.8. Level rod schematic.

the four bench mark level rods. Then foresights were made on each of the heave indicator rules. Rule "zero" elevations were computed and compared with the previous elevations to determine the heave.

Suction Measurements

In-situ suction measurements were made with Spanner double-junction psychrometers. A psychrometer is a humidity-measuring device, and, as shown in Fig 3.9, there is a direct relationship between humidity and suction. The equation of this curve is

$$\tau = \frac{RT}{gM} \ln H$$

where

τ = total suction, cm of water;

R = universal gas constant, 8.314×10^7 erg $^{\circ}C^{-1}$ mol.$^{-1}$;

T = absolute temperature, $^{\circ}C$;

g = gravimetric constant, 981 cm sec^{-2} ;

M = molecular weight of water, 18.02; and

H = relative humidity.

Thus, by measuring the humidity in a soil, the suction can be determined. The principal limitations of such psychrometers are that electrical output signals are extremely low, they are subject to contamination, and they are sensitive to temperature. The last is overcome by using double-junction sensors.

The Spanner psychrometer consists simply of a thermocouple made from very fine wires, plus attendant accessories, as shown in Fig 3.10. This is a single-junction psychrometer. A double-junction psychrometer is shown in Fig 3.11 and has two inherent advantages over the single-junction model. First, precise temperature control is not required since both junctions are at the same ambient temperature. Second, reading both junctions in turn provides a check on the psychrometer's condition. For this project, double-junction psychrometers were used. They were purchased from EMCO, Angola, Indiana.

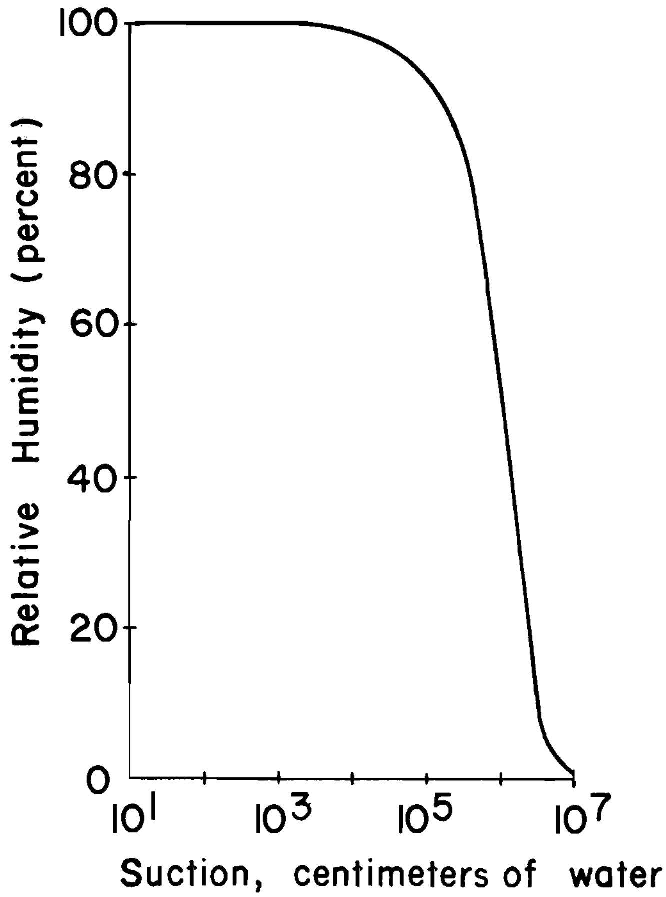


Fig 3.9. Relationship between relative humidity and suction at 20°C.

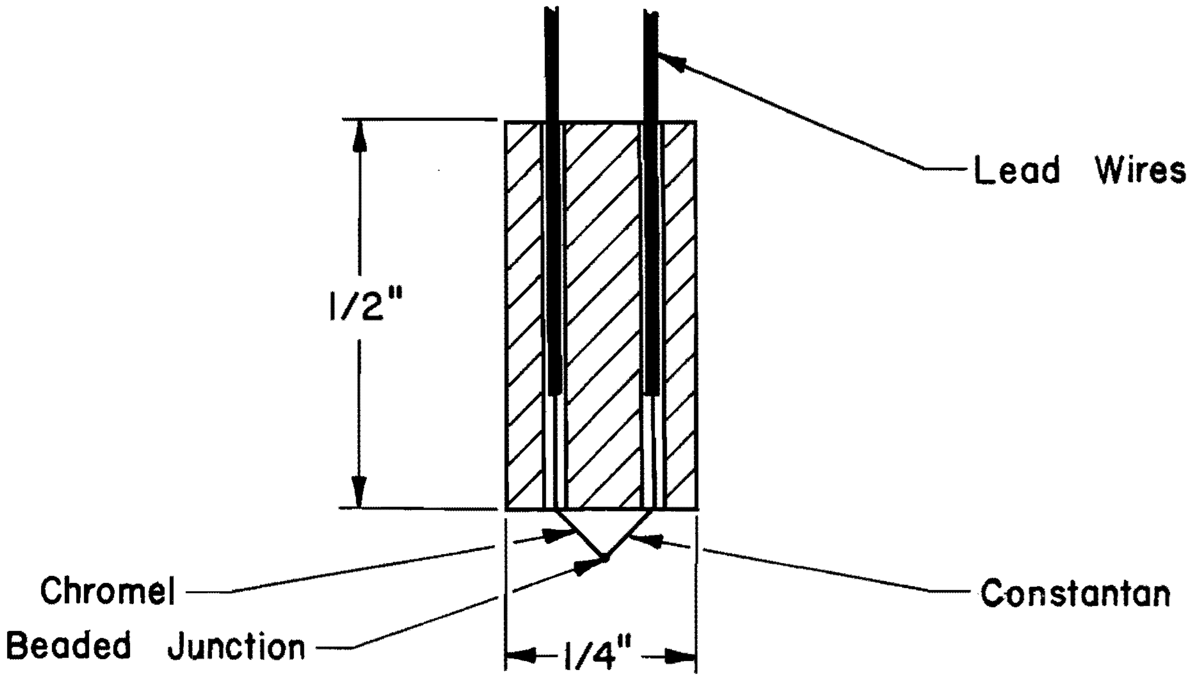


Fig 3.10. Cross-section of single junction psychrometer.

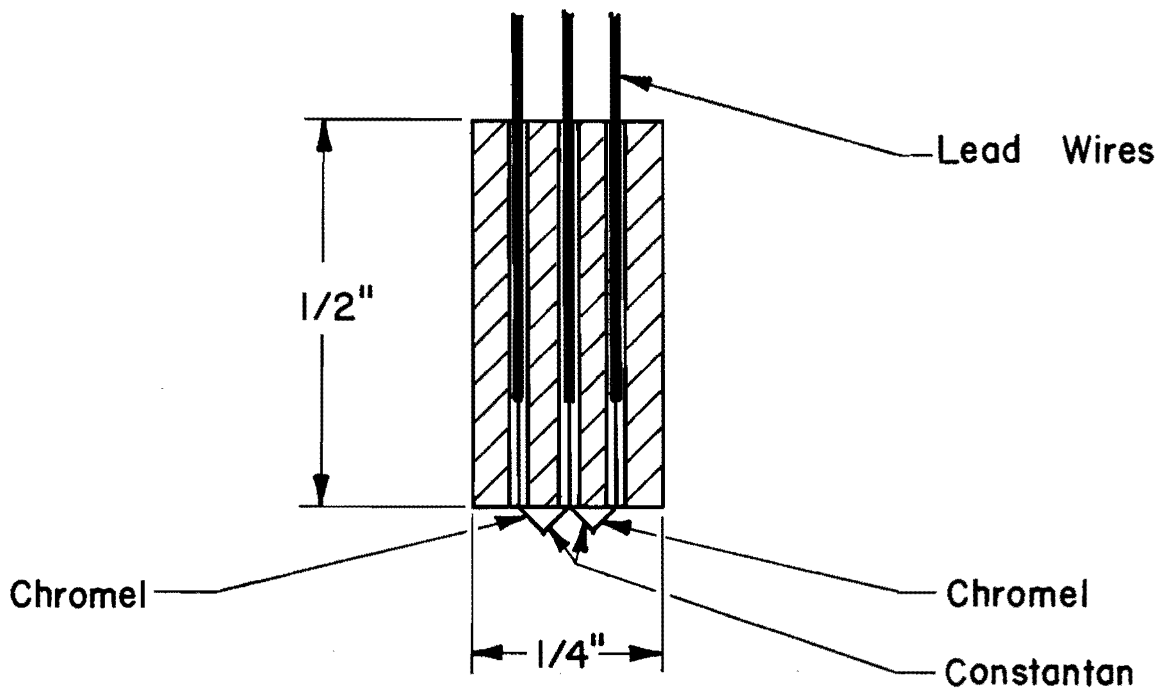


Fig 3.11. Cross-section of double junction psychrometer.

In theory, psychrometer operation is quite simple. A direct current is impressed on the thermocouple so that the current flows from the lower to the higher thermo-electromotive material. This causes the thermocouple to be cooled. Figure 3.12 shows a welding bead forming the thermocouple head, which is approximately 0.003 inch (0.08 mm) in diameter. If the ambient relative humidity is greater than about 95 percent and the current is flowing, a water droplet will form on the junction as seen in the photograph. When the current is switched off, the droplet evaporates. The rate of evaporation is inversely proportional to the ambient relative humidity. Faster evaporation cools the junction more, toward the dew point. By measuring the dry-bulb temperature and the generated emf between the dry and wet junction, a measure of the humidity is obtained.

Calibration

Spanner psychrometer calibration is performed over salt solutions of known concentration. In a closed container, a given solution will produce a known humidity. Upon receipt from the manufacturer, each psychrometer was tagged and calibrated over at least eight solutions with a range in humidity from 100 to 94 percent. This corresponds to a suction range of 0 to 100 bars at 25° C.

The calibration setup used is pictured in Fig 3.13. Up to three psychrometers are mounted in a rubber stopper which closes a 250-ml bottle containing 200 ml of the calibration solution. For temperature stability, the bottle is buried to the neck in sand in a styrofoam chest. The psychrometer is connected to a switch-box. The output is measured on a Keithley Model 150B microammeter and recorded with a Hewlett-Packard Moseley Model 7100A strip chart recorder. The entire calibration setup is maintained in a constant temperature room with temperature stability better than $\pm 1^{\circ}$ C. The same measuring and recording equipment is used in the field.

The circuit diagram for the switch box is given in Fig 3.14. At switch position 1, the psychrometer temperature is measured by comparison with the output of a reference junction in an ice bath. At switch position 2, the psychrometer circuit is shorted out and the microammeter is nulled. At switch position 3, a cooling current of 3.5 ma is applied to one psychrometer junction. This cooling current is that recommended by the psychrometer

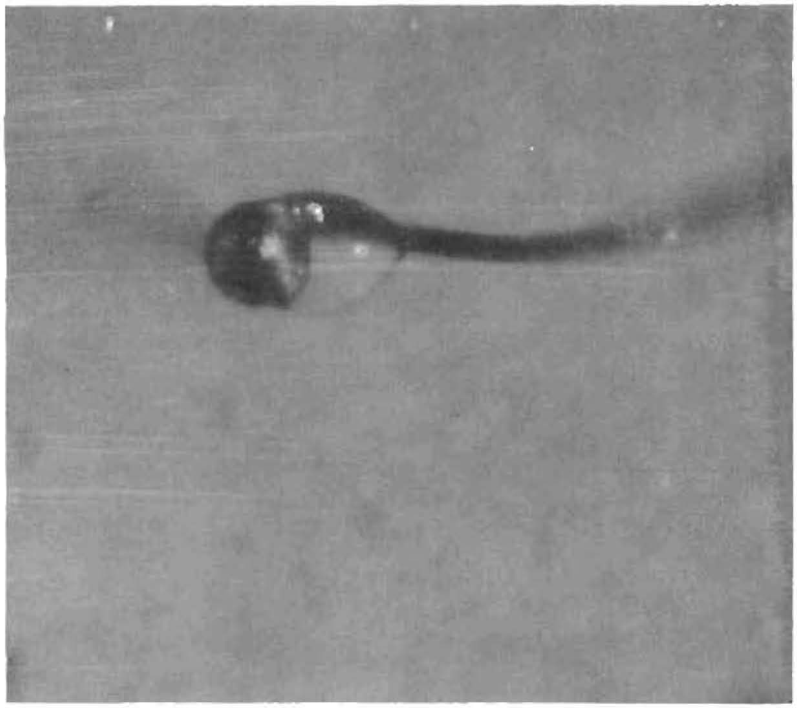


Fig 3.12. Cooled junction with water droplet.

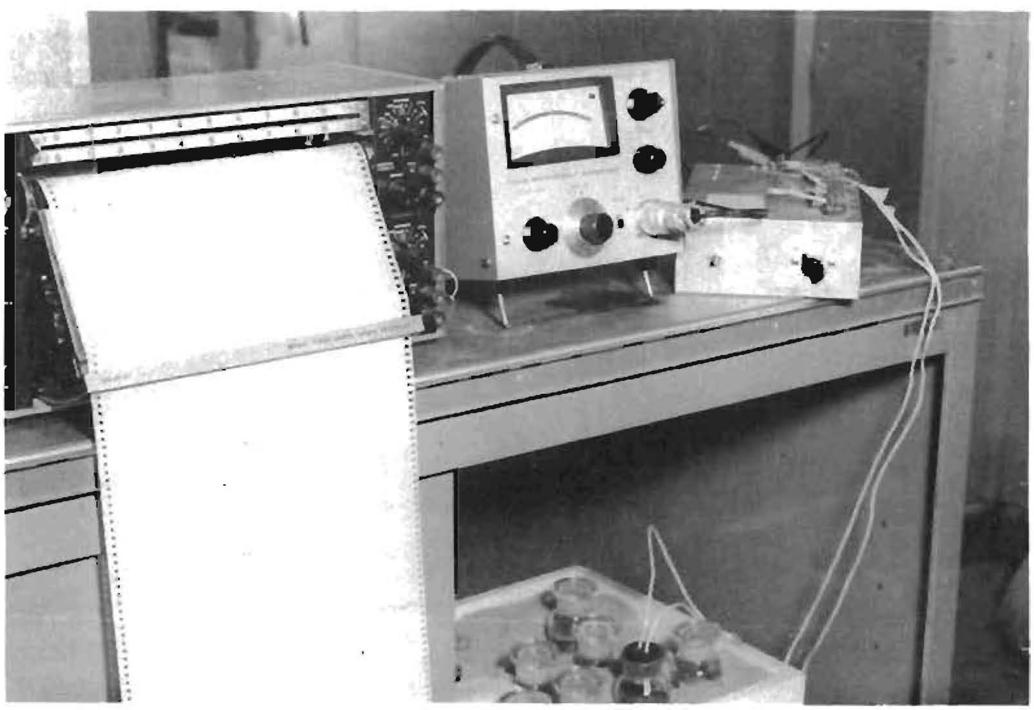
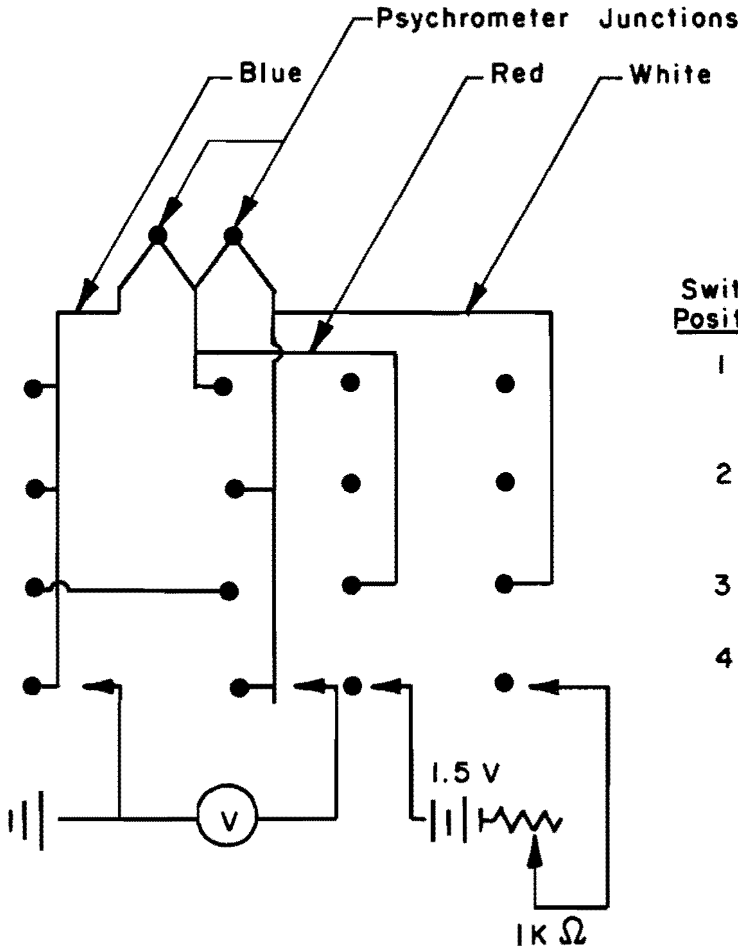


Fig 3.13. Calibration of thermocouple psychrometers.



Switch Position	Functions
1	Read Thermocouple (Temperature)
2	Open Circuit (Zero Equipment)
3	Apply Cooling Current
4	Read Thermocouple (Humidity)

Fig 3.14. Circuit diagram used for double-junction psychrometer.

manufacturer. It is supplied by a 1-1/2-volt battery and is adjusted by a 1000-ohm potentiometer. The cooling current is applied for 30 seconds. Switching to position 4 turns off the cooling current, and the emf generated by the temperature differential of the two psychrometer junctions is measured by the microammeter.

Typical recorded psychrometer outputs are shown in Fig 3.15. The series of readings over the same solution is indicative of the repeatability of the peak output under ideal conditions.

Typical calibration curves are shown in Fig 3.16a for two different cooling times. An important characteristic is the intercept at zero suction or 100 percent relative humidity. This is caused not by the evaporation of the water droplet but by the water droplet's being at a lower temperature than the uncooled junction of the psychrometer. Figure 3.16b indicates the effect of temperature. Although temperature must be known to correct the reading for this effect, the problem of temperature is not serious using the Spanner double-junction psychrometer since temperature effect at the probe is directly cancelled. This is in contrast to severe difficulties reported for single-junction psychrometers (Ref 6).

The curves in Fig 3.16a show that a given reading may correspond to two different suction values. Determining the correct one is not a problem because of the difference in the shape of the recorded psychrometer output curves. At high suction values beyond the peak of the calibration curve, the recorded curve is much more of a spike, as illustrated in Fig 3.17a. Figure 3.17b shows a cooling curve typical of the lower range of suction, which is the range of interest in the field measurements.

Field Installation

Other investigators have had varied success with psychrometers buried in the soil. The major problems have been contamination and corrosion of the microscopic thermocouple. Richards (Ref 6) reports that the calibration of psychrometers left in the soil changes significantly within a month. Johnson (Ref 7) reported they malfunction after 3 to 9 months. However, EMCO, a psychrometer manufacturer, reports no significant changes after several months.

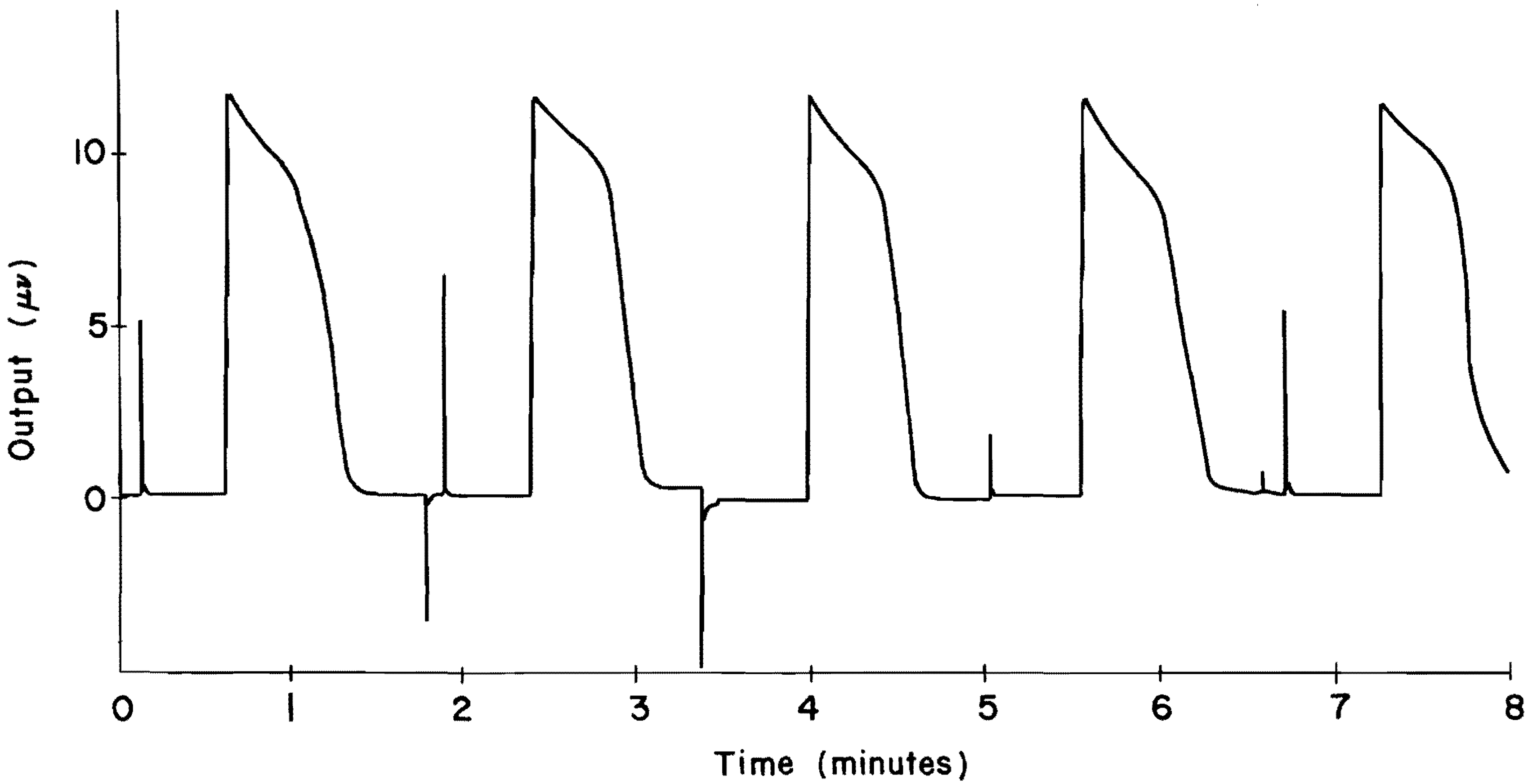
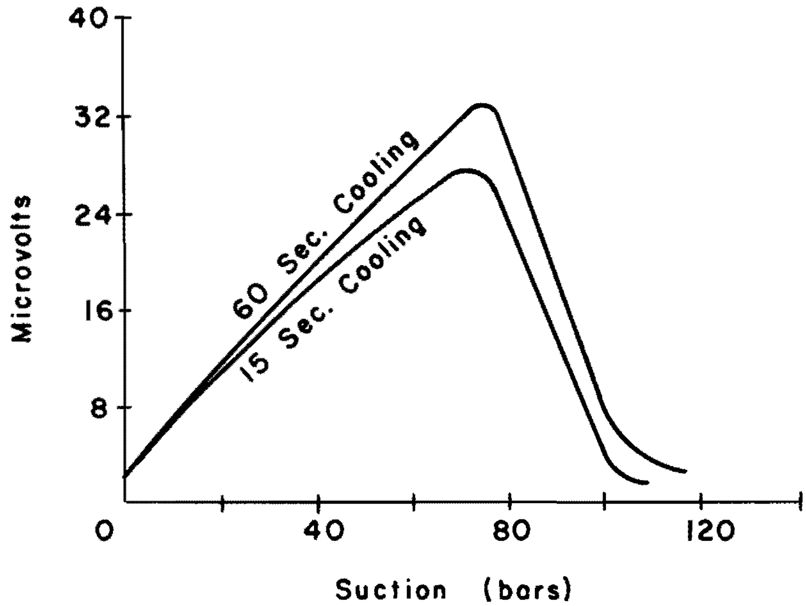
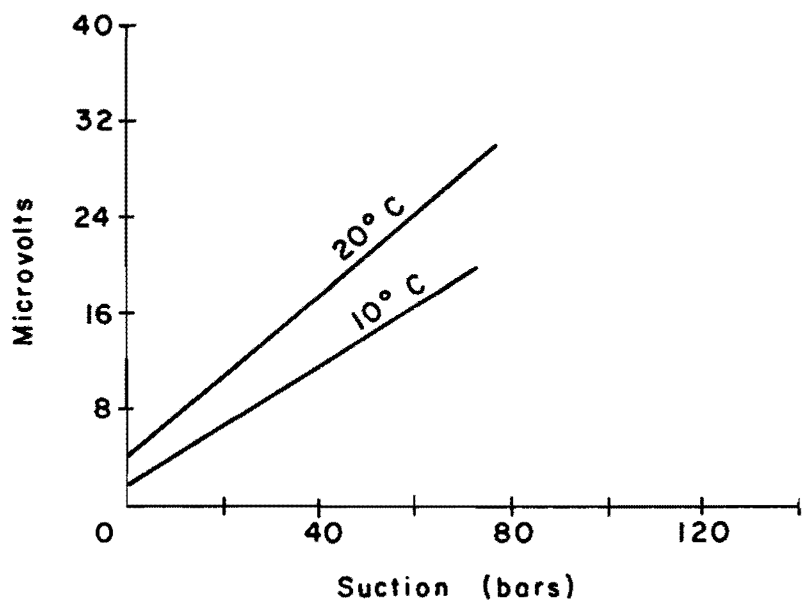


Fig 3.15. Typical curves showing repeatability

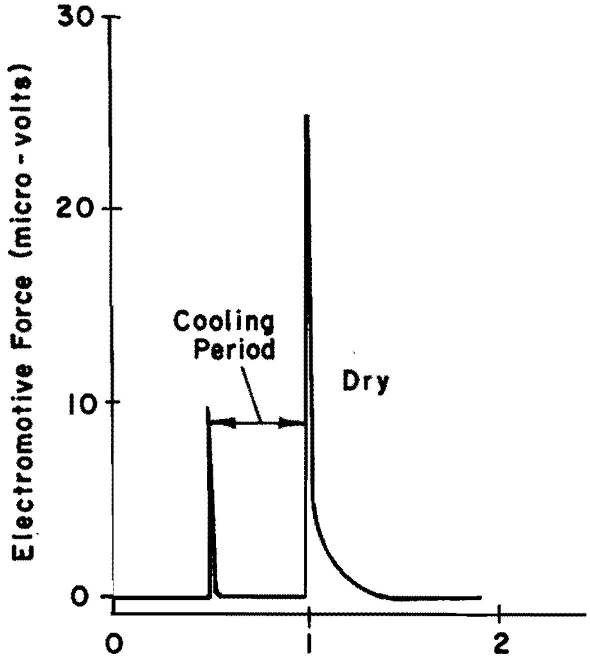


(a)

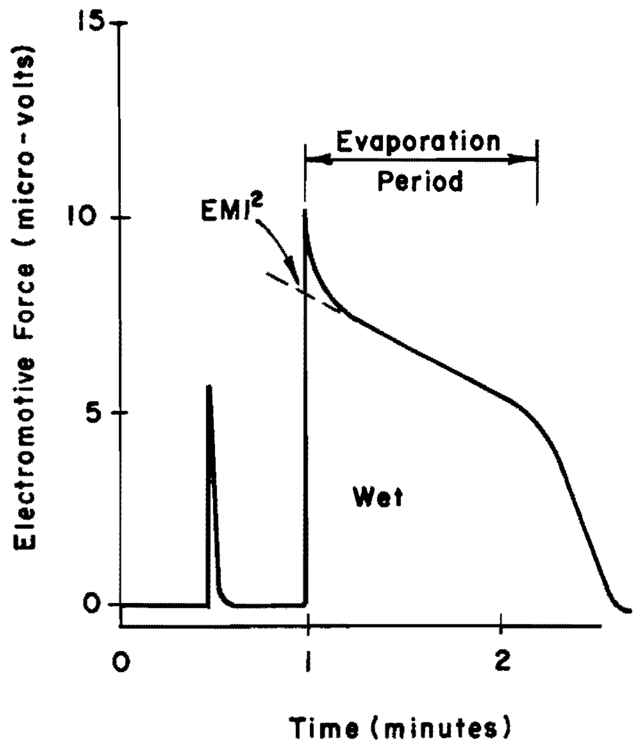


(b)

Fig 3.16. Typical calibration curves for Spanner psychrometer.



(a)



(b)

Fig 3.17. Typical cooling curves.

To avoid this problem, an alternate method was devised in which the psychrometer is removable. The details are pictured in Figs 3.18 through 3.20. A tip (Fig 3.18) is press fitted and welded onto the slotted end of an access tube (Fig 3.18) and the tube is pushed to the required depth in the same manner as a subsurface heave point. However, the access tube serves as the push tube and is not withdrawn after emplacement.

In place, soil moisture passes to the central chamber through the slots. A psychrometer fitted to the end of a 5/16-inch (7.9-mm) tube is illustrated in Fig 3.19. The tube is inserted into the access tube and pushed down until it meets the point and closes off the central chamber from the top, as shown in Fig 3.20. After temperature and humidity (suction) equilibrium is achieved between the soil and the air in the central chamber, a measurement is made. When the measurement is completed, the psychrometer is removed and the access tube is sealed.

The advantages to this system are

(1) less disturbance of in-situ conditions than the backfill method of installation,

(2) rapid installation,

(3) psychrometer recalibration or replacement is possible in case of damaged or unreliable psychrometers, and

(4) measurements can be repeated at the same point with different psychrometers.

A check of calibrations was made with the psychrometers mounted in the holder tubes. An 1000-ml Erlenmeyer flask was modified by adding a 1-inch (25-mm)-diameter, 3-inch (7.5-cm)-long, horizontal tube to the flask's neck. The psychrometer end of the tube was inserted in a rubber stopper and the rubber stopper was mounted in the horizontal tube, which sealed the flask. The flask contained 500 ml of a calibration solution. After waiting 3 to 4 hours, equilibrium was achieved and a suction measurement taken. This value of suction was then checked with the calibration curve.

If the psychrometer was found to be off calibration, it was boiled for an hour in distilled water and then dried. Another check calibration was then performed, and, if still off calibration, the psychrometer was discarded. When a psychrometer comes in contact with water containing dissolved salts, its calibration is usually altered.

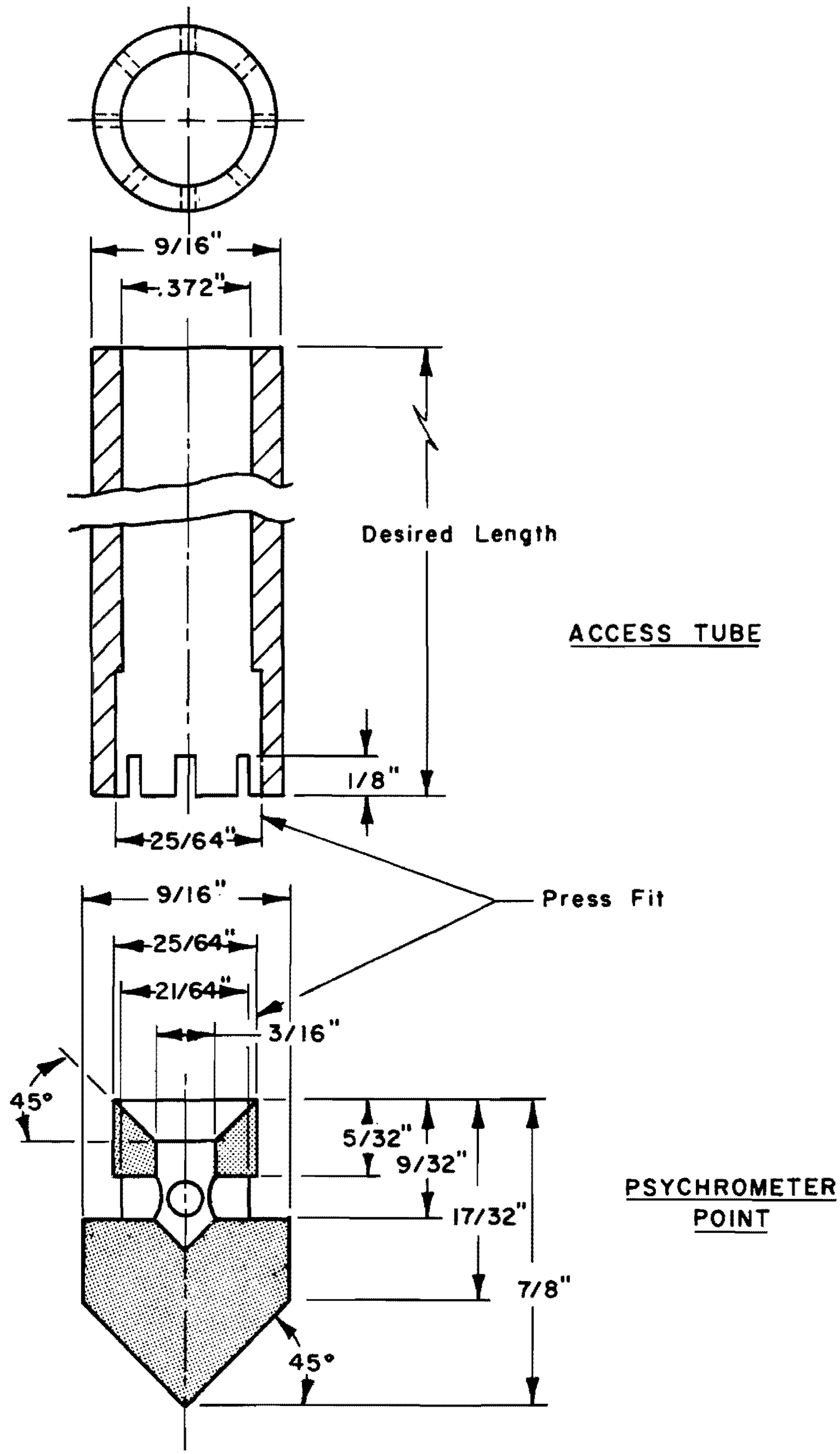


Fig 3.18. Psychrometer access tube with point.

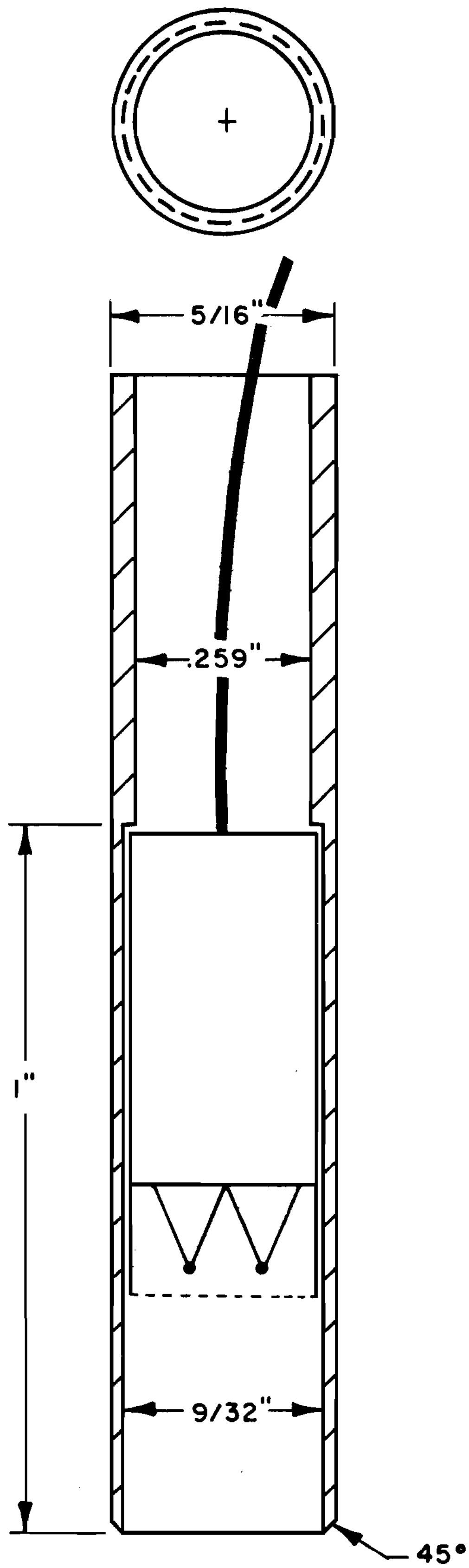


Fig 3.19. Psychrometer holder.

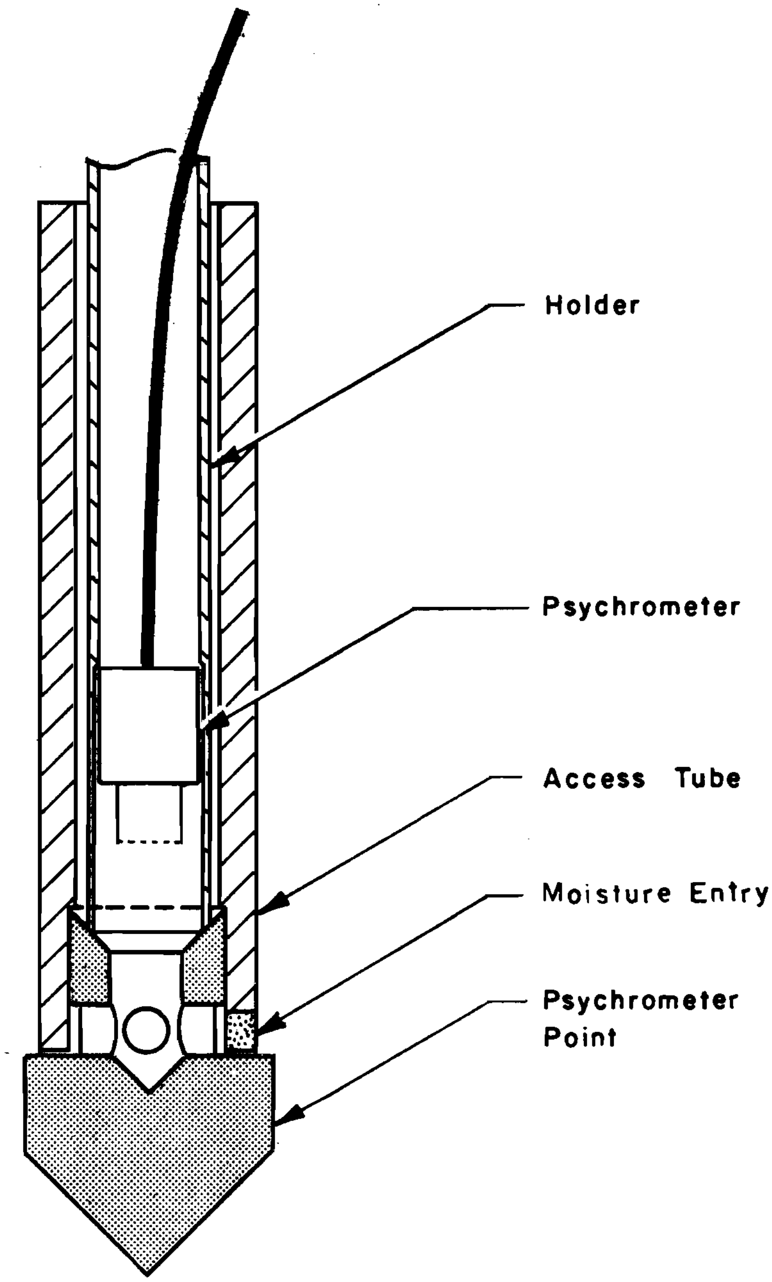


Fig 3.20. Psychrometer assembly.

Psychrometer Performance

Two major problems have developed with the psychrometer system. One is related to the highly developed system of fissures in the Taylor Marl. When the surface was ponded, the fissures conducted water down to the tip and then free water passed through the slots into the central chamber. This normally occurred with the access tubes having tip depths down to 5 feet (1.5 m). No meaningful suction measurements can be made when this infiltration occurs. With deeper tip depths, this was not a problem. However, the second problem is related to the first. Because water was found in some psychrometer tips, from either infiltration or condensation, it was necessary to check each tip prior to insertion of the psychrometer. To check for water, a rod with a bit of cloth on the end was inserted to the bottom of the tube and then withdrawn for checking. A result of this action is that the air in the access tube and central chamber is completely changed. After the psychrometer is inserted, a period of 18 to 24 hours is then required for suction equilibrium to be achieved between the soil and the central chamber.

Considerable care and attention to detail are required to perform suction measurements in-situ with psychrometers, and the delay time for equilibrium is objectionable. Further refinements are needed before such psychrometers are suitable for general field use, and it is difficult to argue that the results are competitive in quality and cost with deducing the suction from other measurements. Because of the lack of consistency and particularly of continuity of the psychrometer data, no results are shown for the field tests.

CHAPTER 4. SITE PREPARATION AND INSTRUMENTATION LAYOUT

General Site Layout

An overall view of the experimental site is depicted in Fig 4.1. There are four instrumented test areas, four deep bench marks, and a permanent level mount, as shown in Fig 4.2. Each test area is 40 × 40 feet (12 × 12 m) in plan. The locations of the level mount and bench marks were established such that the backsights and average foresight would be approximately equal. Surrounding the site is a shallow ditch to conduct excess rain water away from the test areas.

Instrumentation Layout and Installation

Instrumentation of test area 1 began in June 1973 with the installation of nuclear access tubes. During August 1973, psychrometer access tubes and subsurface heave points were installed at various depths to 10 feet (3 m) maximum below the existing ground surface. Approximately 5 feet (1.5 m) of overburden was removed from the area during 1 to 3 September 1973. The overburden was heavy gray clay with gravel and cobbles as shown in Fig 2.2. After overburden removal, test area 1 had a level bottom, which was then covered with a 6-inch (15-cm) washed sand blanket, which served as a working surface. Instrumentation installation was completed 20 September 1973, as shown in Fig 4.3.

In test area 2, instrumentation began in July 1973 with the installation of some nuclear access tubes. The overburden was removed during 4 and 5 September 1973, as in test area 1. In addition, a 35-ft-long, 4.5-ft-deep trench was excavated along one diagonal, as shown in Fig 4.4. This trench was intended to permit the entry of water horizontally into the soil. Its purpose was to determine if this provision significantly accelerated the water adsorption of the soil in the vicinity of the trench. The trench was filled with washed sand to prevent collapse of the walls. Installation of instrumentation in the configurations shown in Fig 4.4 was completed 30 December 1973,

Fig 4.1. Overall view of experimental site.

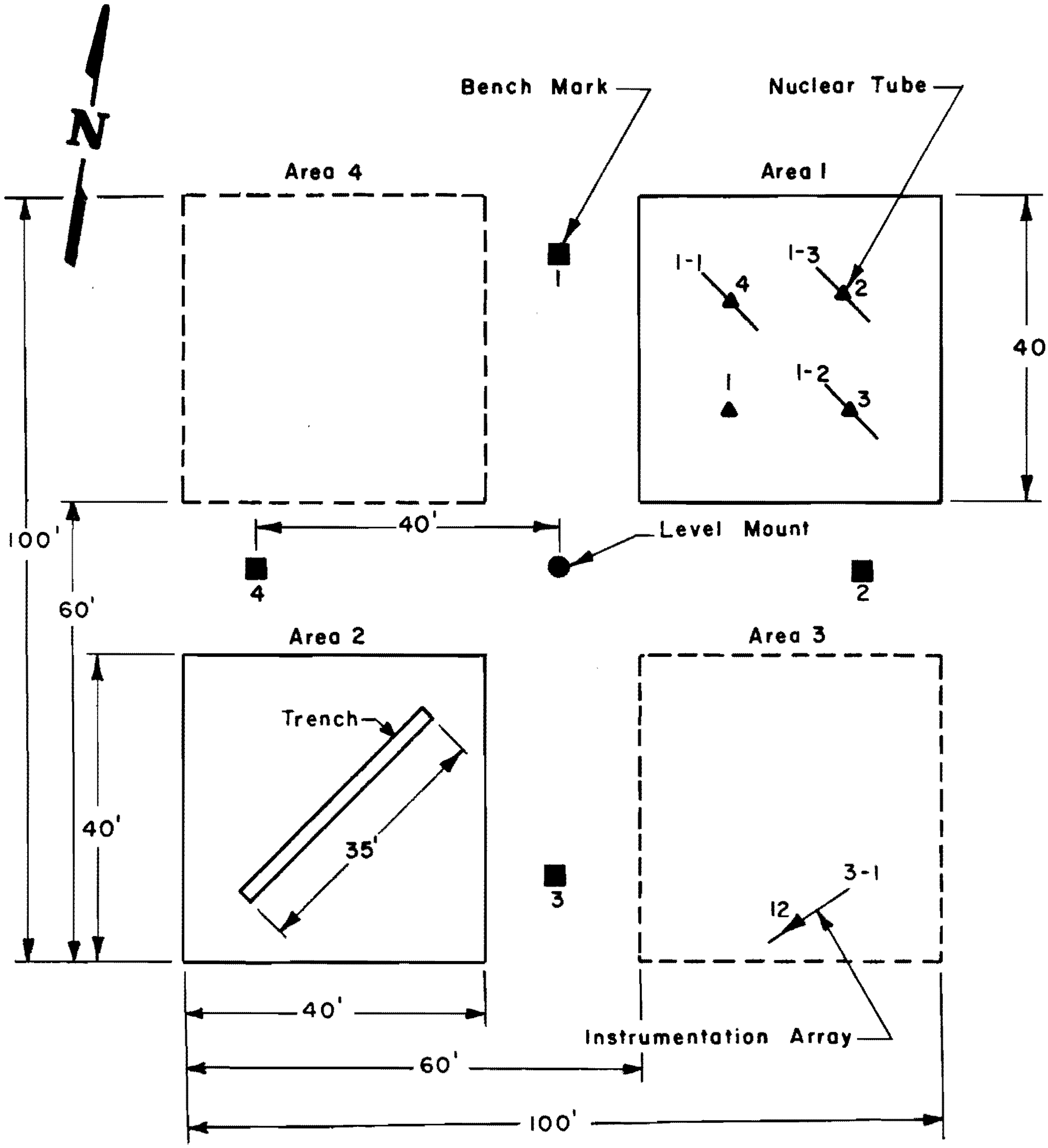


Fig 4.2. General layout of test area.

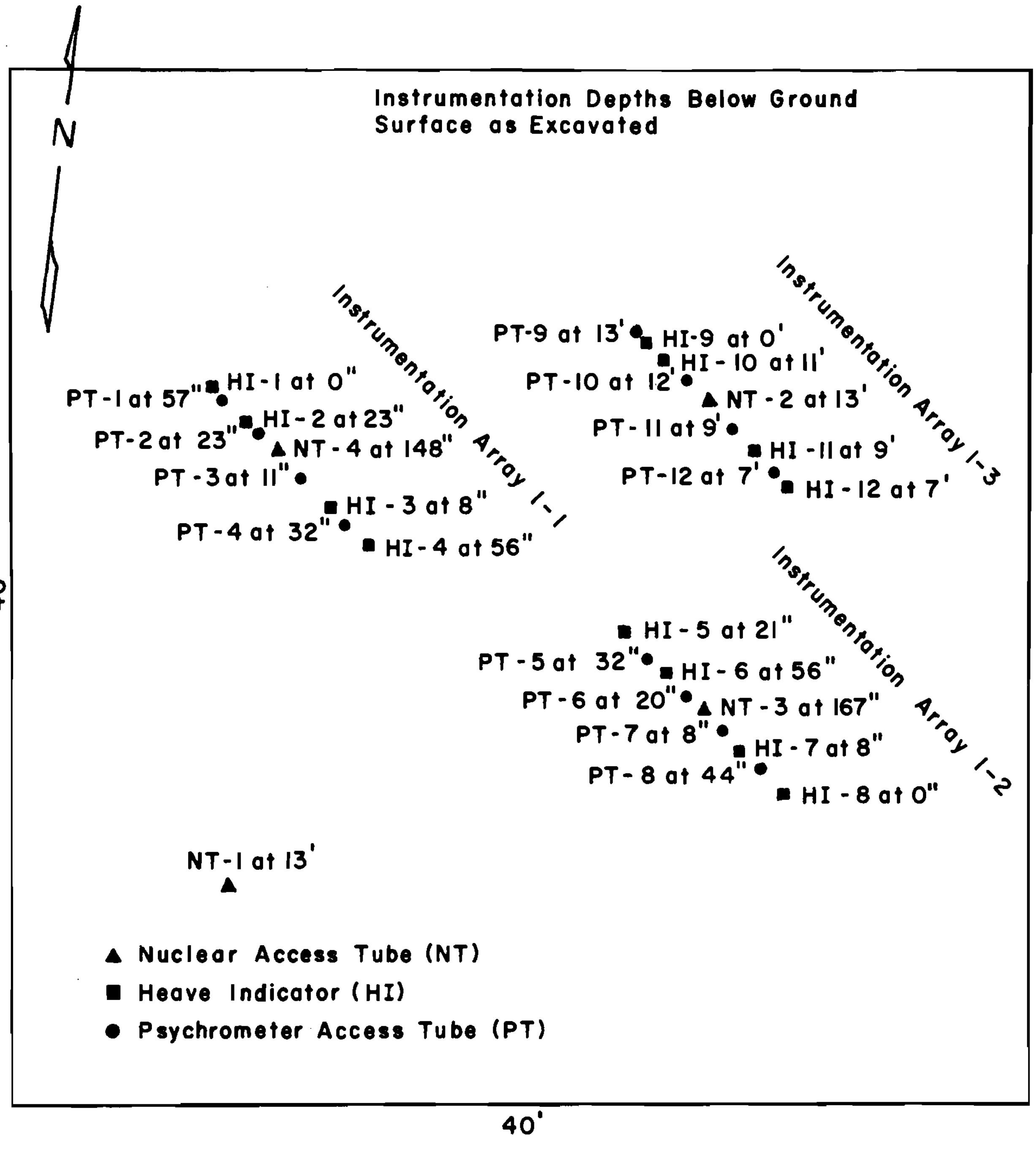


Fig 4.3. Instrumentation in test area 1.

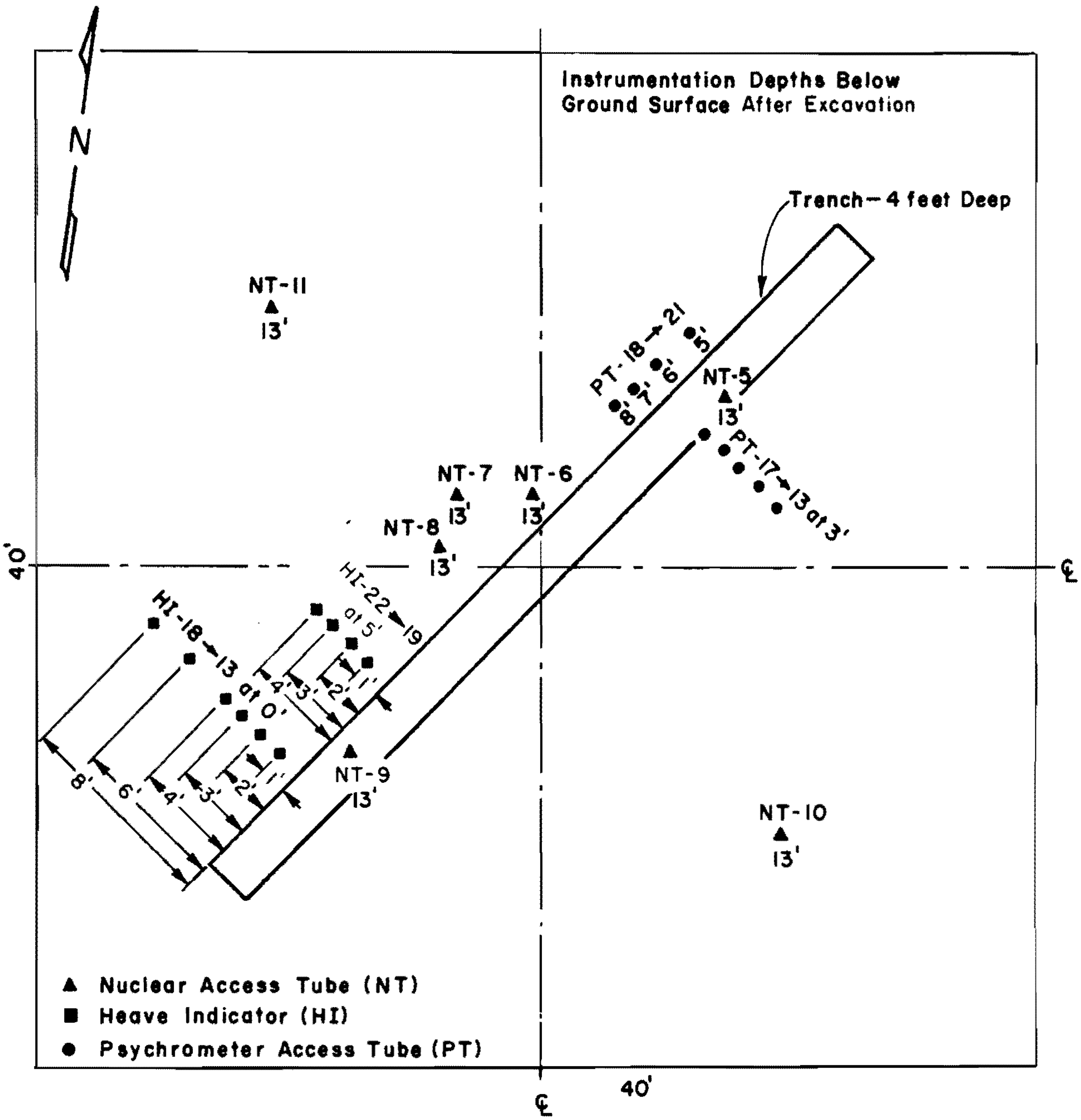


Fig 4.4. Instrumentation in test area 2.

and the bottom of the excavated area was also covered with a sand blanket working platform.

Test area 3 was initially established as a control section to observe the behavior of the natural ground under the effects of normal climate. Instrumentation was completed 16 January 1974 in the configuration shown in Fig 4.5. However, during the year it was decided to start another ponding experiment and, in October 1974, the overburden was removed to a depth of 3 feet 9 inches (1.1 m) below the original ground surface, and a horizontal bottom covered with a 6-inch (15-cm) sand blanket was provided as in test area 1. The area was ponded in October 1974 and additional instruments were then installed. Prior to overburden removal in area 3, test area 4 was established as the control section for continued observation of the behavior of the natural ground.

Site History

Prior to the presentation of results, a brief history of events at the site is necessary. Site exploration was concluded in June 1973 and the test areas were laid out. During July and August, the bench marks, some heave indicators, and psychrometer nuclear probe access tubes were installed as previously explained. In September, the overburden was removed from test areas 1 and 2, more instrumentation was installed, and sand blankets were placed. Initial readings were started at this time.

On 26 September 1973, 6.74 inches (17.12 cm) of rain fell at the site; on 4 October 1973, 2.39 inches (6.07 cm) of rain fell; and, during the period 11-15 October 1973, another 7.69 inches (19.53 cm) of rain fell. These rains repeatedly flooded test areas 1 and 2 prior to final instrumentation placement. The water was removed by pumping and extensive restoration work commenced. Instrumentation of test area 2 was finally completed in December 1973.

This series of rains disclosed several design deficiencies in the instrumentation, which, for the most part, were remedied. The problem of flow of free water into shallow psychrometer access tubes has not been solved. The premature flooding and the instrumentation problems delayed the measurements but not the movement of moisture into the clay. Thus, the data on the actual conditions in test areas 1 and 2 during the first ten months of the experiment are incomplete. Planned flooding of the site was finally done on 6 August 1974.

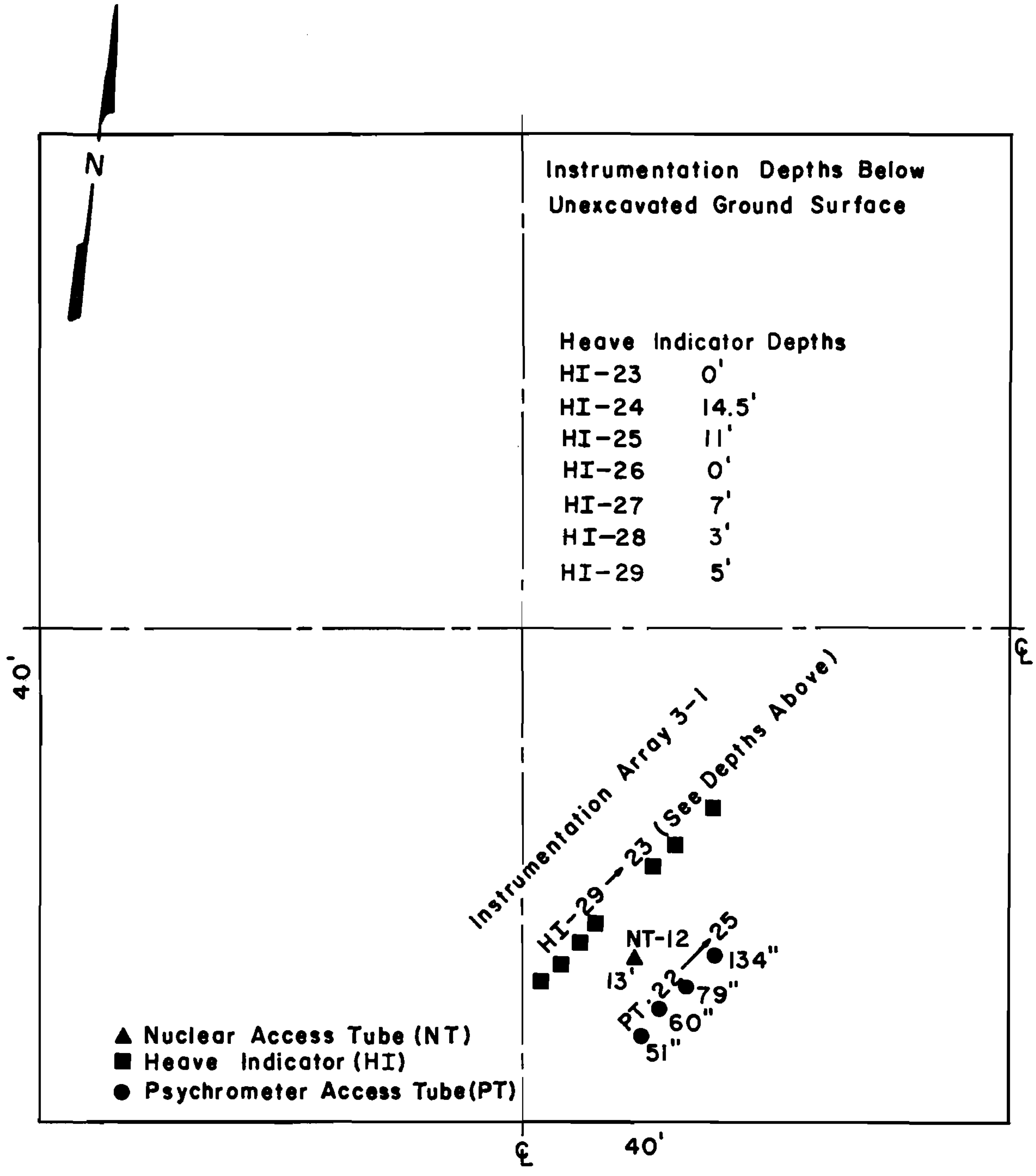


Fig 4.5. Instrumentation in test area 3.

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

Meteorological data from records for Austin, Texas, are shown in Fig 4.6, together with observed movements of the natural ground. The heavy rains of late September and early October 1973 are clearly reflected in the monthly totals.

Some correlations can be found between weather conditions and natural ground heave. For example, the extended dry weather after the fall of 1973 provided considerable drying shrinkage. In test area 4, the rapid rise of the surface in April 1975 corresponds to a heavy rain. On the other hand, some of the movements do not correspond well to rainfall records. This may be partly due to the fact that the areas are sufficiently sloped to give excellent drainage, and the behavior is probably closely related not only to rainfall amount but also to the rate and duration. Whether cracks are open also may have a dominant effect. In any event, it is interesting to note that, with the good natural ground drainage, there were almost no movements at depths beyond 5 feet; instead the movements of natural ground were confined primarily to the layer of topsoil.

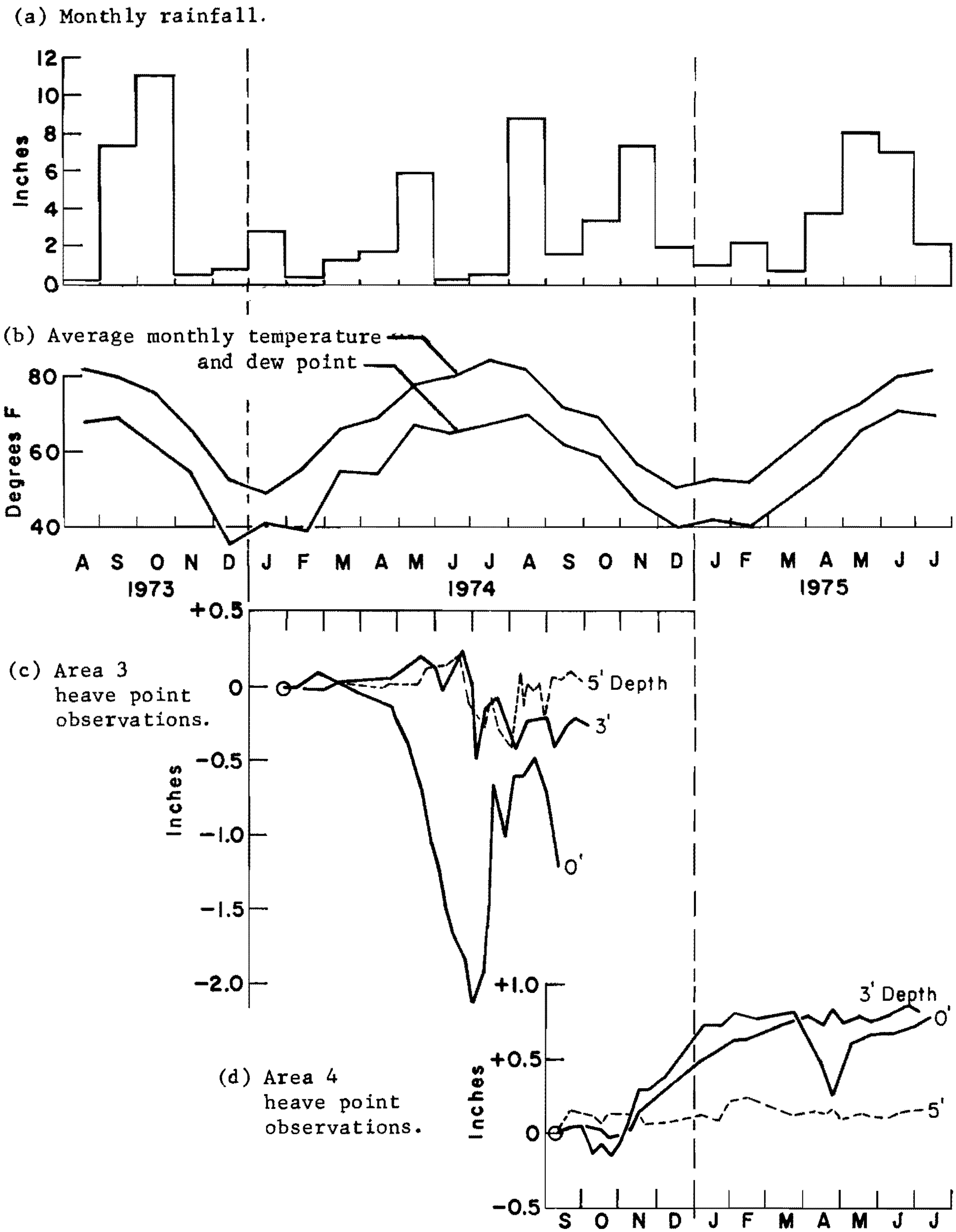


Fig 4.6. Records of weather and natural ground movements.

CHAPTER 5. EXPERIMENTAL RESULTS

This chapter presents results of the measurements made at the Lake Long test site during the two-year ponding experiment. Because of the large amount of data taken, only representative sets are included, selected from sets taken at biweekly intervals. The data clearly indicate the major trends noted during the experiment.

The measurements are also compared to presently available predictive techniques, including McDowell's Potential Vertical Rise Method, Texas Test Method Tex-124-E (Ref 8).

Test Area 1

Heave measurements were made during the excavation of this test area, using the heave points installed prior to removing the overburden. The removal of the top 4 feet of soil resulted in an upward rebound of 0.5 inch at the exposed surface and 0.25 inch at 2 feet below. No measurable rebound occurred at a depth of 4.5 feet.

The increase in heave with time is shown in Fig 5.1 for this test area. The effects of the rainfall a short time after the excavation was completed are clearly evident in the rapid initial rise at shallow depths. Although the rainwater was pumped from the excavation within 36 hours, there was sufficient penetration to cause a significant surface movement. Furthermore, the previously placed sand blanket tended to retain the moisture. The dips in the upper curves correspond to a period of severe drying during the summer of 1974.

Moisture content and density values, as measured by the nuclear device, are shown for selected times in Fig 5.2, together with corresponding heave profiles. The progressive changes in moisture content and heave show the increased penetration of moisture with time. The moisture content profiles for 10 July 1975, shown in Fig 5.2e, include both nuclear tube and gravimetric determinations, the gravimetric moisture contents extending to a depth of 28 feet.

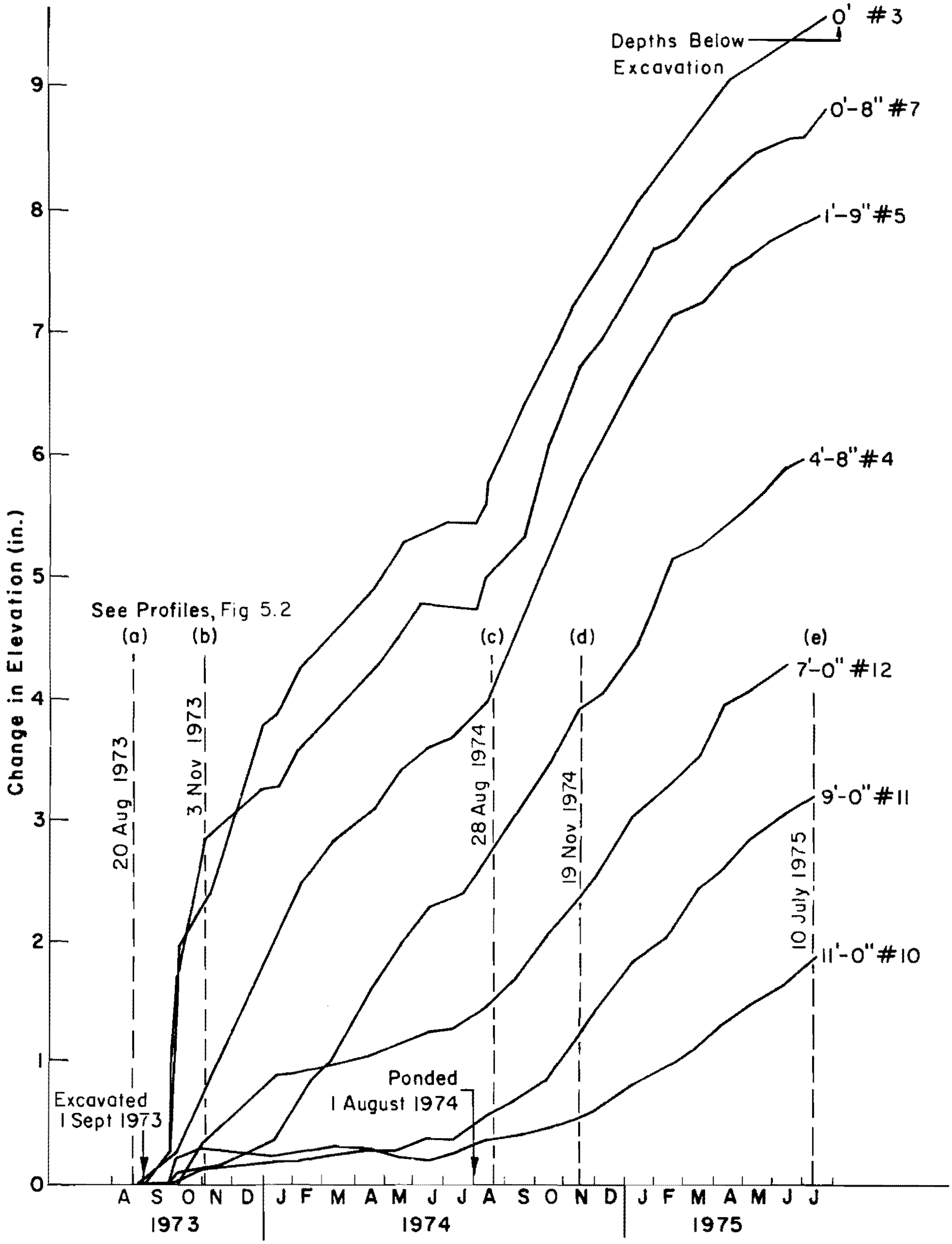
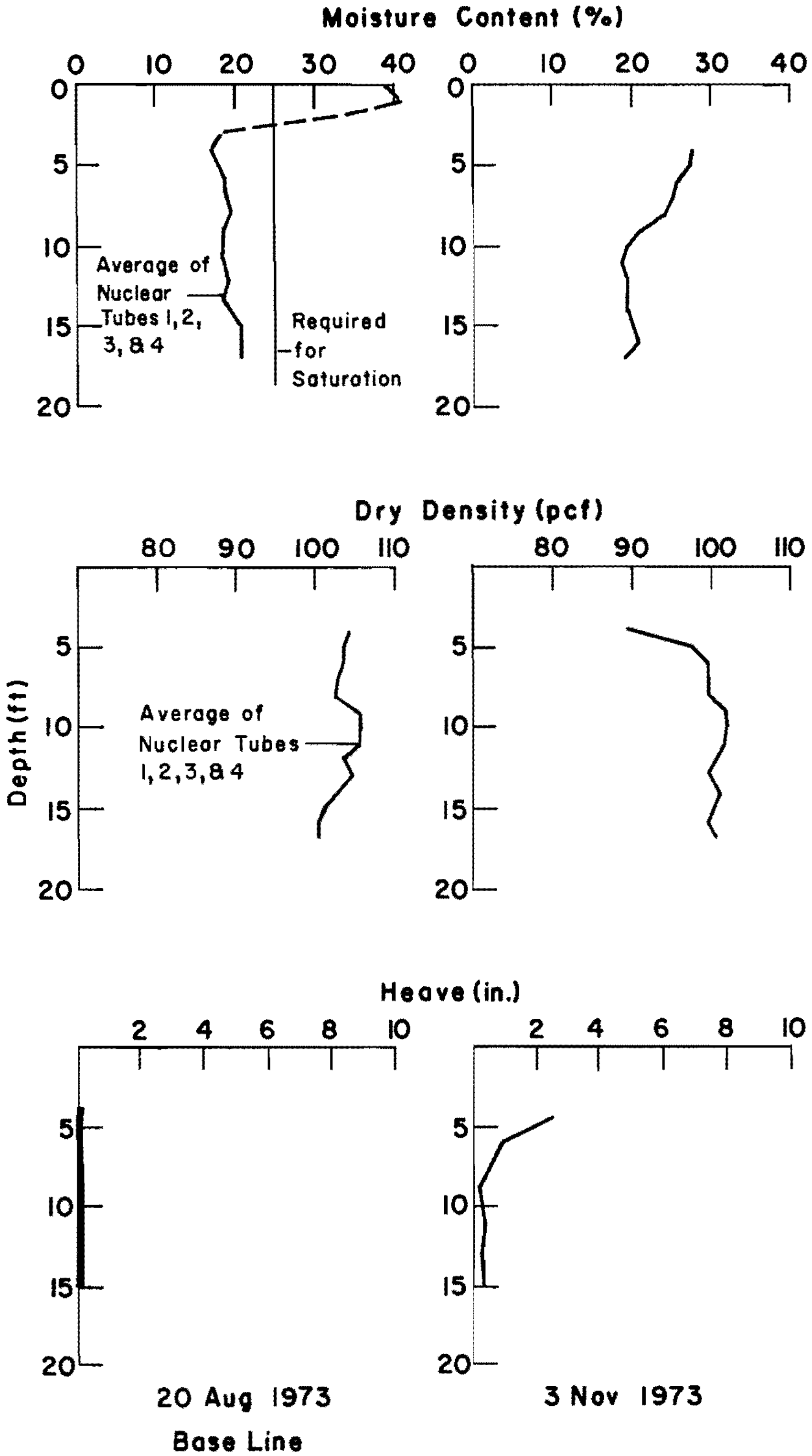


Fig 5.1. Heave records for test area 1.



(a) (b)

(continued)

Fig 5.2. Moisture, density, and heave profiles in test area 1 at selected times.

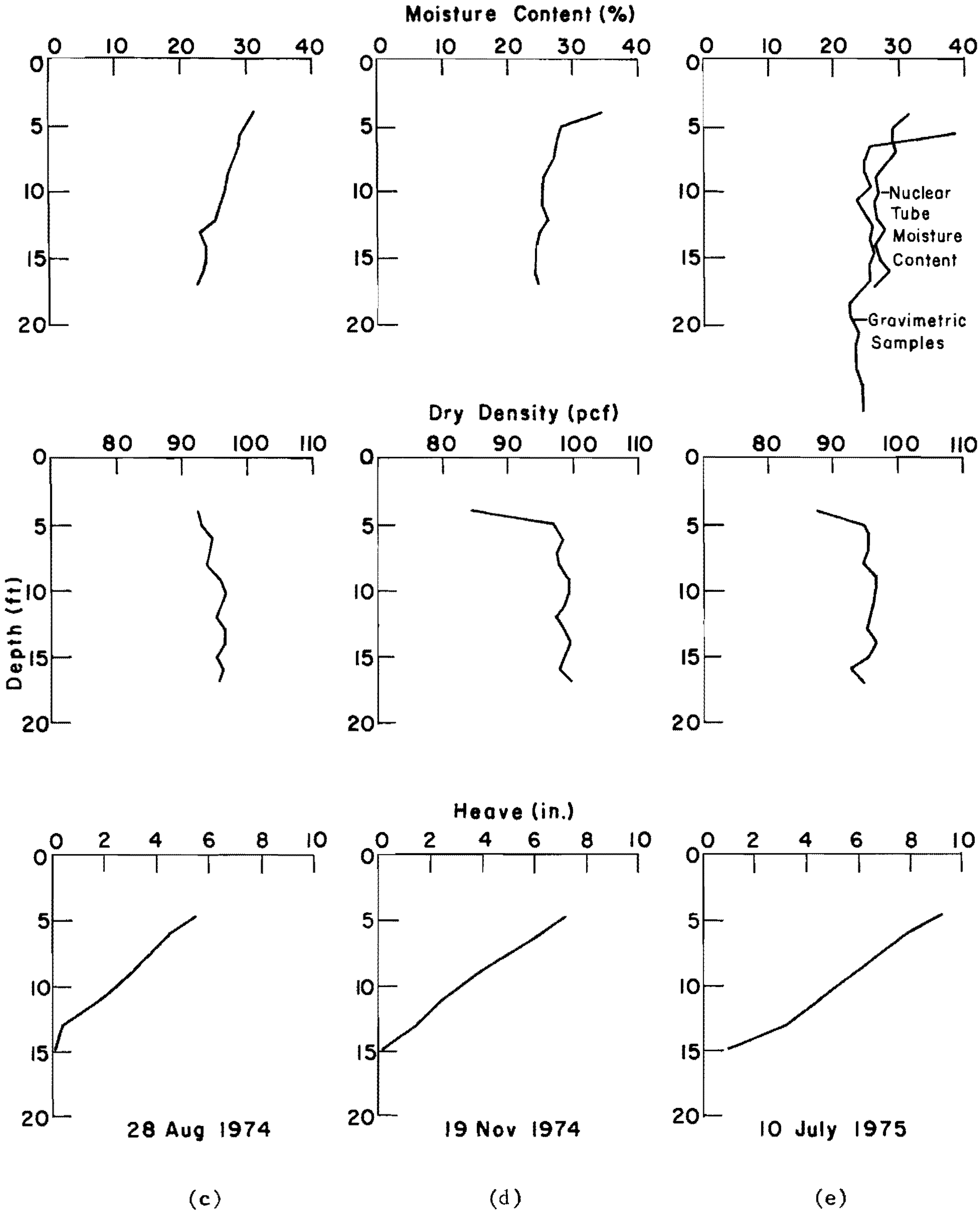


Fig 5.2. (continued)

It is interesting to note that in the first year, when the water in the pond was furnished only by rainfall, the swelling was primarily in the top 5 to 7 feet. This appears to have largely satisfied the water demand of the upper layers. The swell after intentional flooding in August 1974 occurred for the most part at depths greater than 7 feet. Notice that in Fig 5.1 the slopes of the heave curves at shallow depths are almost parallel.

It is interesting to speculate about the reasons for the behavior before and after the intentional flooding which was done on 6 August 1974. Considering the highly fissured nature of the soil to depths of 15 feet and more, it is easy to understand the rapid initial swell. The lack of swell of the deep layers may be due to the retention of water by capillary action at shallow depths where it was then rapidly absorbed. Apparently, only when a positive head was maintained by sustained ponding did the free water reach the deeper layers.

The two different phases of moisture absorption and swell are clearly shown in Fig 5.3a. These curves are very consistent with those in Fig 5.1 in showing that in the first year most of the swell was in the upper zone but after intentional ponding most of the moisture was utilized in causing swell in the lower layers.

From the heave curves in Fig 5.3b it is apparent that at the end of the observations the differential heave between the depths shown had ceased, although the surface heave was still continuing. The moisture had then penetrated below the instrumented depths, and swell was occurring below the deepest heave point. Heave occurring at such depths would not be unusual, since the swelling pressure exerted by this clay at the initial moisture content of 19 percent ranges from 30 to 100 pounds per square inch, while the total overburden pressure at the 11-foot depth is only about 10 pounds per square inch.

The data for 20 August 1973 and 10 July 1975, shown graphically in Fig 5.2, yield the following comparisons, based on average values over the instrumented depth:

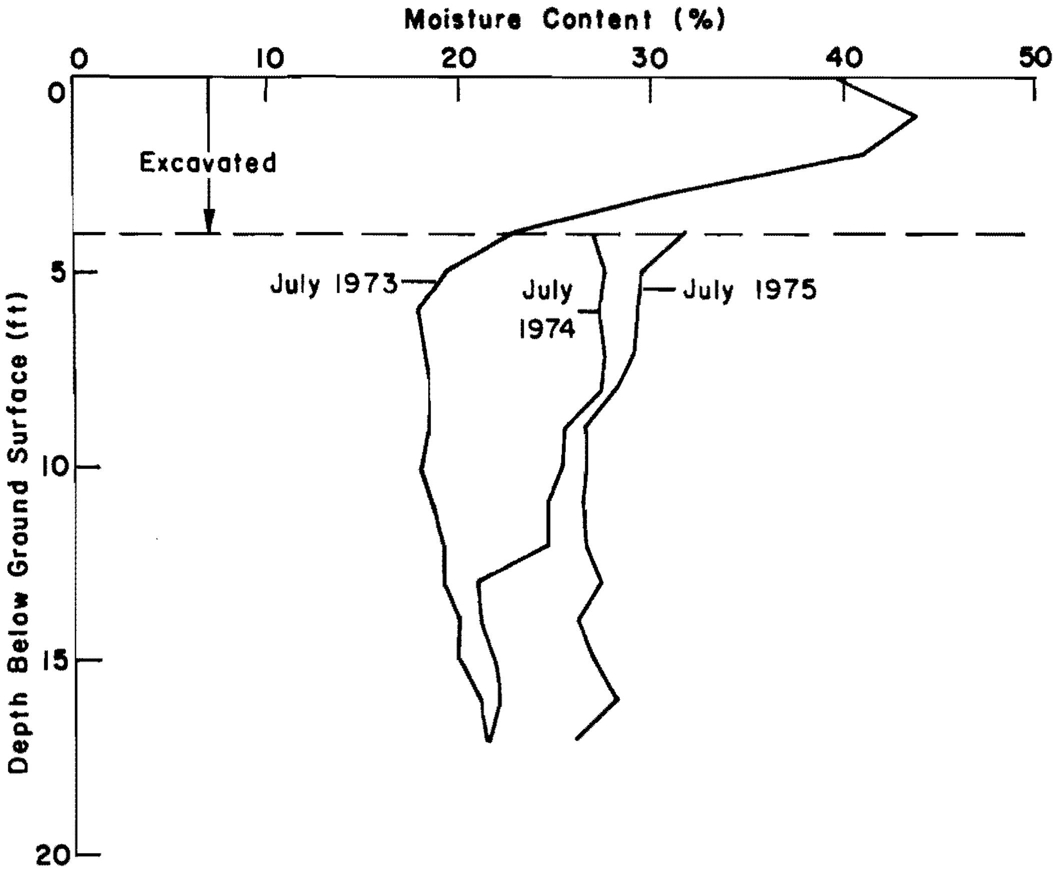
(1) an increase in moisture content from 19.2 to 27.9 percent,

(2) a decrease in dry density from 103.3 to 94.8 lb/ft^3,

(3) a total density change from 123.1 to 121.2 lb/ft^3, and

(4) a vertical heave of approximately 0.75 inch per foot of depth.

(a)



(b)

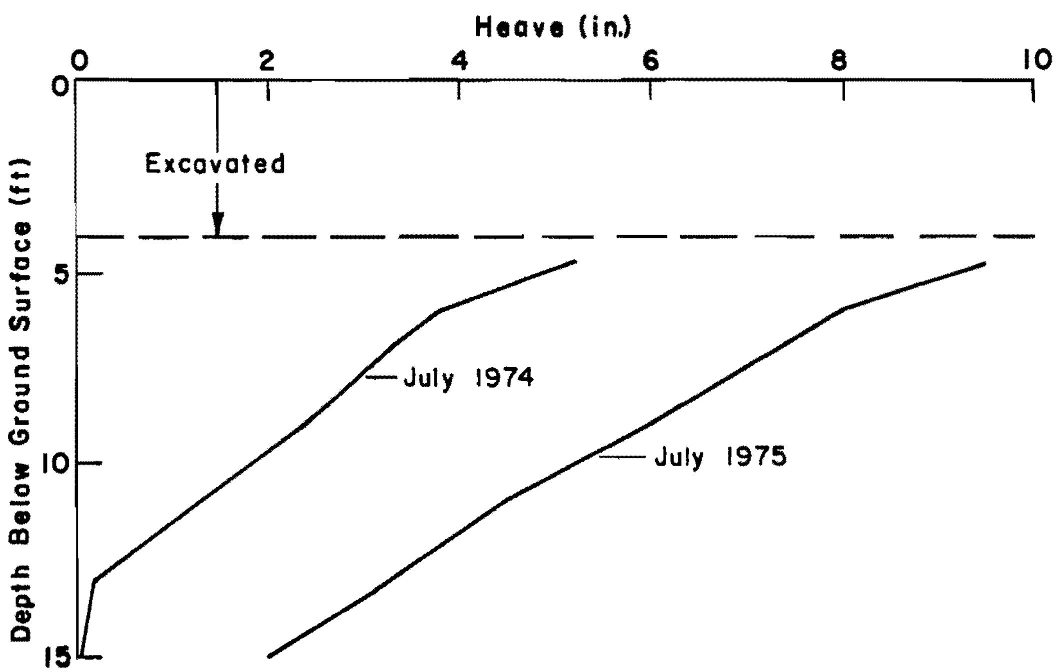


Fig 5.3. Summary of results for test area 1.

Test Area 2

Test area 2 was intended for observation of the effects of a trench on the rate of infiltration of water into the clay and thereby to provide a two-dimensional case for comparison of predictive methods developed earlier. The horizontal flow into the clay near the trench was expected to increase the time-rate of heave near the trench. The original thinking also included interest in relative permeabilities in the horizontal and vertical directions.

This test area was excavated at the same time as test area 1 and the 4-foot-deep trench were dug. The area was subjected to the same rainfall that had caused the premature heave in the first area. However, the heave points were not installed in this test area until February 1974, at which time the first test area had developed 4 inches of surface heave.

Figures 5.4 and 5.5 show the changes in heave with time at various distances from the trench. The irregular data from heave point 15 are not understood. Otherwise, it is apparent that any effect of the trench was obscured. This was due to the completion of much of the differential swell in the upper layers before the heave points were installed. A reverse effect may have occurred with heave point 22, which was the most distant from the trench at the 5-foot depth. Apparently it was least affected during the initial swell period while the excavation was open. Except for a sharp rise early in 1974, the curve is essentially parallel to the others until the area was ponded, in August 1974. Thereafter, heave point 22 rose at a considerably faster rate than the others. This may have indicated a delayed heave quite similar to that in test area 1, where the positive free water pressure head caused accelerated swelling of the previously dormant deeper layers.

Profiles of moisture content and density at selected times are shown in Fig 5.6. Sources for each of three different groups of nuclear data are shown. On 10 July 1975, the moisture contents included a gravimetric determination which is slightly lower than the previously taken nuclear values, as for test area 1 in Fig 5.2. The nuclear moisture data are very consistent, but there are some unexplained variations among the density plots. The heave data consisted of multiple determinations at two depths and the points shown are average values for each depth.

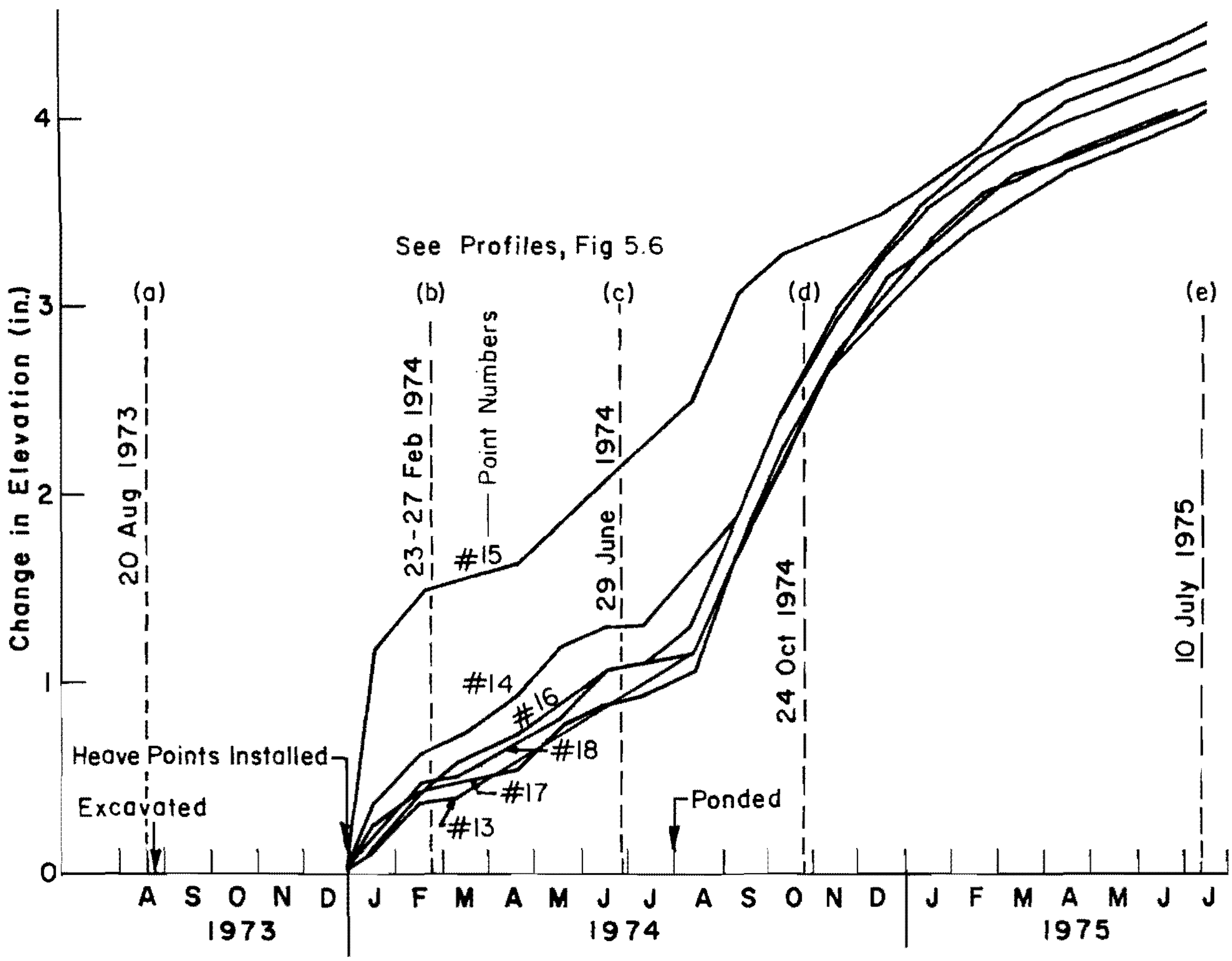


Fig 5.4. Surface heave measurements in test area 2 at varying distances up to 8 feet from the trench.

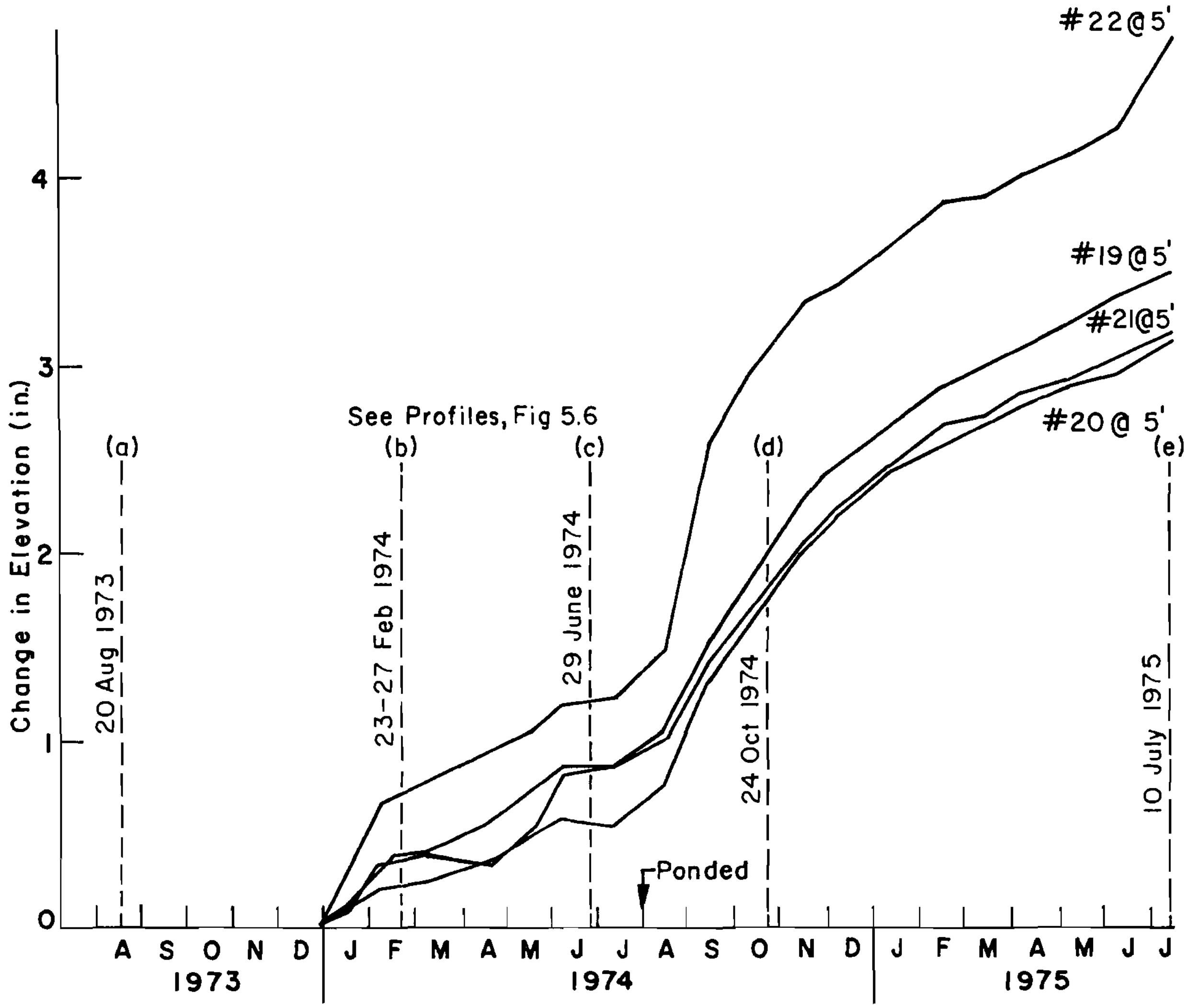
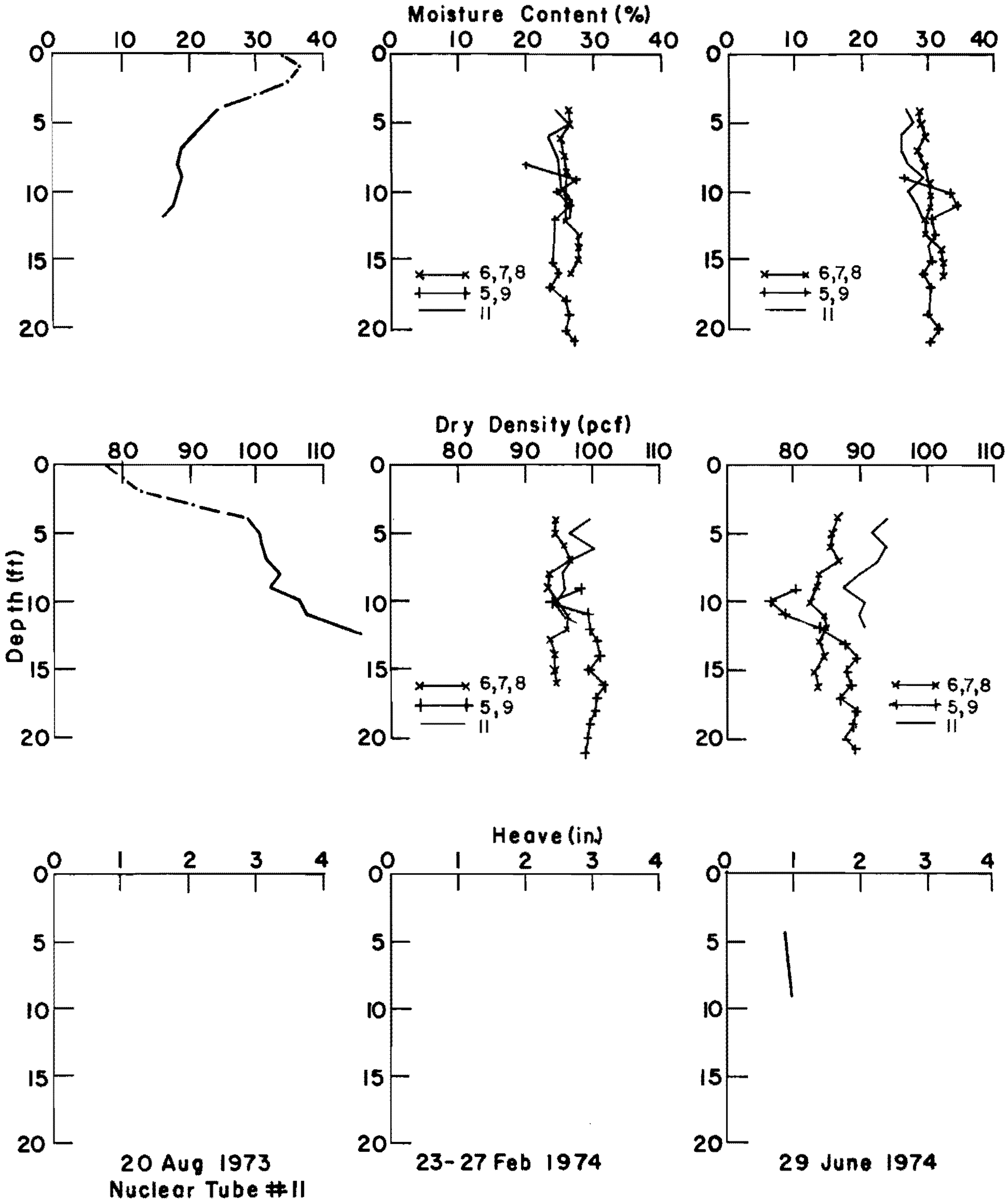


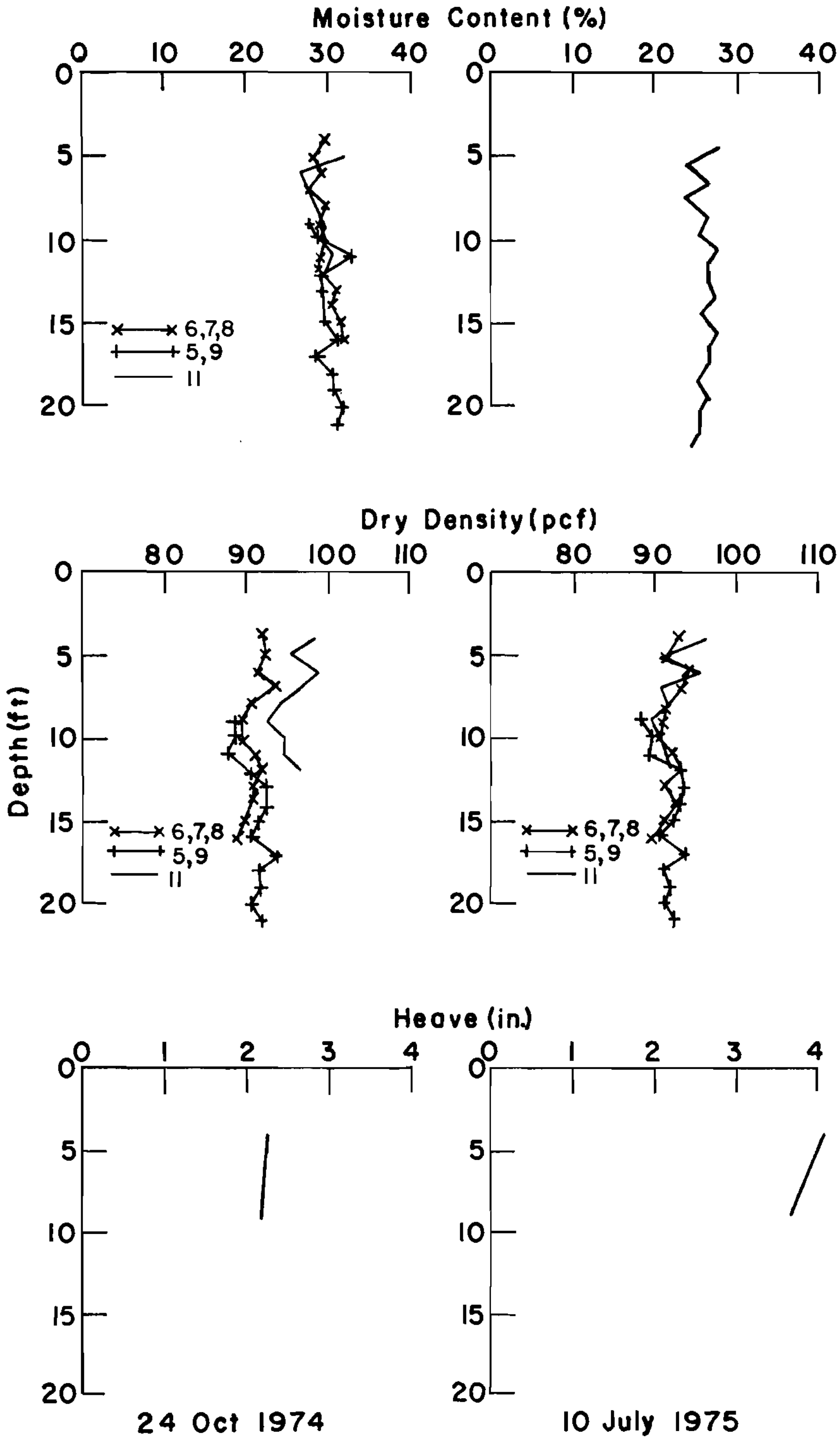
Fig 5.5. Heave measurements at a depth of 5 feet below the excavated surface of test area 2. (See Fig 4.4.)



(a) (b) (c)

(continued)

Fig 5.6. Moisture, density, and heave profiles for test area 2 at selected times. (Refer to Fig 4.3 for locations.)



(d) (e)

Fig 5.6. (continued)

The overall heave during the period of measurement is very nearly the same as that observed in test area 1 during the same period. The total heave would be the same if the 4 inches measured in the first test area were assumed to apply to test area 2 and were included in Figs 5.4 and 5.5.

Following is a summary comparison of nuclear tube and heave data for test area 2:

(1) For nuclear tube 11, beyond the influence of one trench, between depths of 4 and 12 feet from the original ground surface, the moisture content increased from 19.5 to 29.4 percent, while the dry density decreased from 104.7 to 91.6 pounds per cubic foot.

(2) For nuclear tubes 6, 7, and 8, adjacent to the trench between the depths of 4 and 16 feet from the original ground surface, the average moisture content increased from 19.5 to 30.7 percent and the dry density decreased from 104.7 to 92.0 pounds per cubic foot. The total measured heave between 23 February 1974 and 10 July 1975 was 4.1 inches, with a probable actual heave of more than 8 inches at the end of observations.

(3) For nuclear tubes 5 and 9, located in the bottom of the trench and extending to a depth of 21 feet below the original ground surface, the average moisture content increased from 19.5 to 30.9 percent and the dry density decreased from 104.7 to 91.7 pounds per cubic foot.

As can be seen by comparing the moisture content and dry density as measured by instruments in, near, and away from the trench, the condition of the clay changed fairly uniformly, with no significant difference among the measured moisture content or density changes for the three locations.

Test Area 3

Test area 3 was originally intended to be a dry land control area for the comparison of heave and moisture changes with the two ponded areas and as a reserve area for future experiments. Nuclear tubes extended to only 12 feet below the original surface. Because of the problems encountered in the first two areas due to the early flooding by rainfall, it was decided to excavate this test area and to pond it (on 21 October 1974) immediately after the overburden was removed. The curves of Fig 5.7 are assumed to have a common zero swell just prior to ponding. The heave curves are generally consistent in form with those in Fig 5.1.

Profiles of moisture, density, and heave are shown at three selected times in Fig 5.8. Nuclear tube readings are available only at a limited

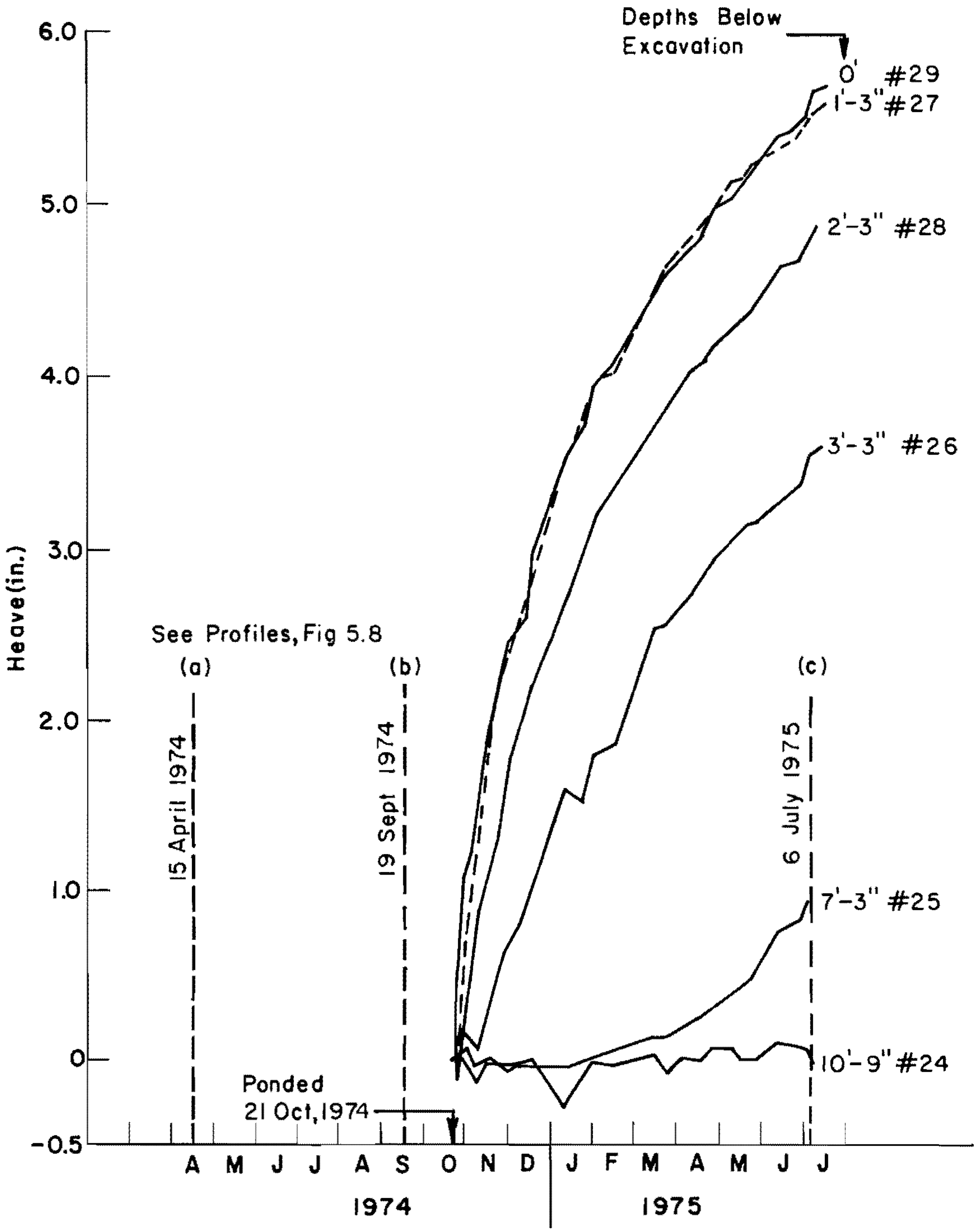
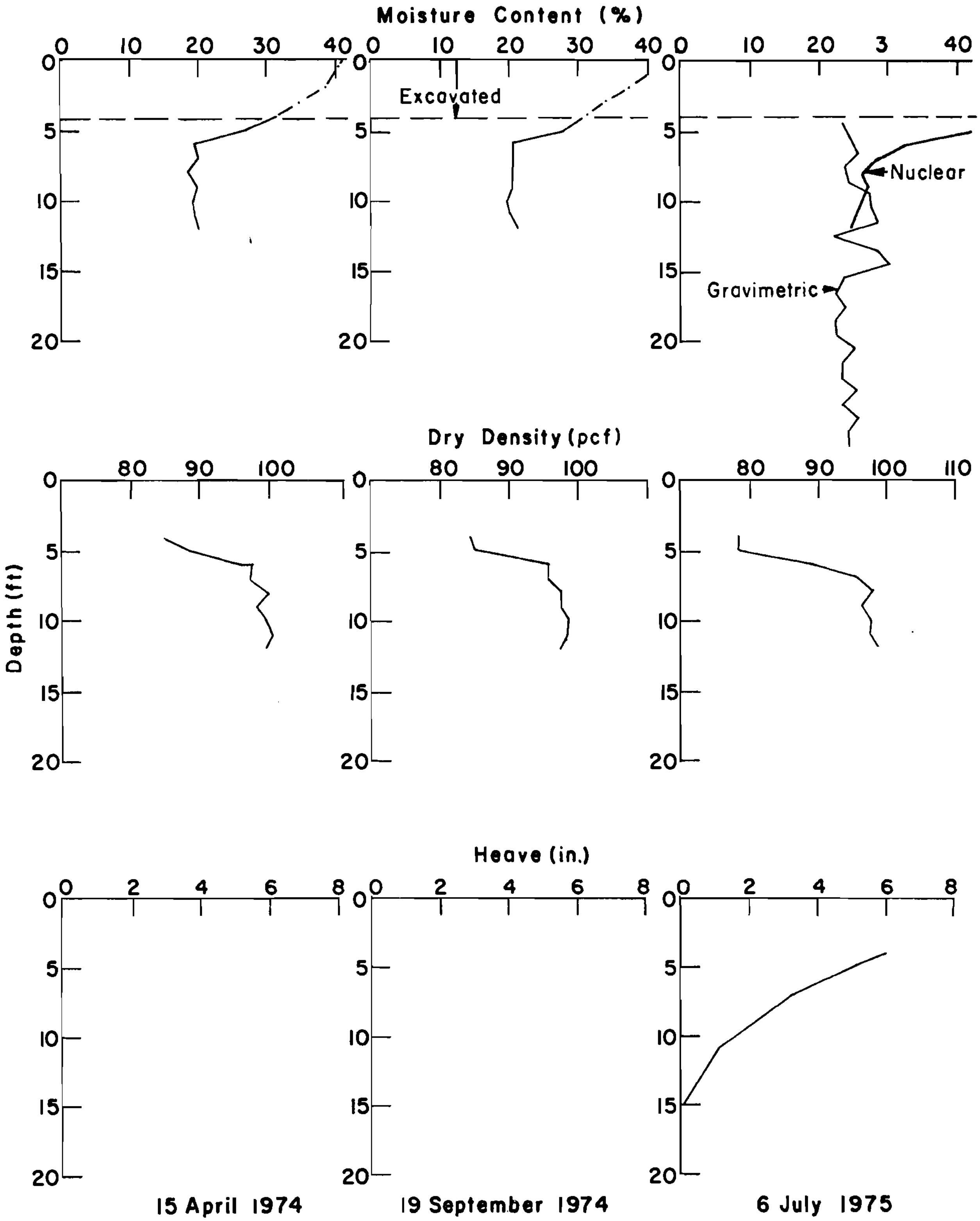


Fig 5.7. Heave curves for test area 3.



(a) (b) (c)

Fig 5.8. Selected profiles from test area 3.

number of times because of malfunctions in the equipment and extended delays required for the repair of the device by the manufacturer.

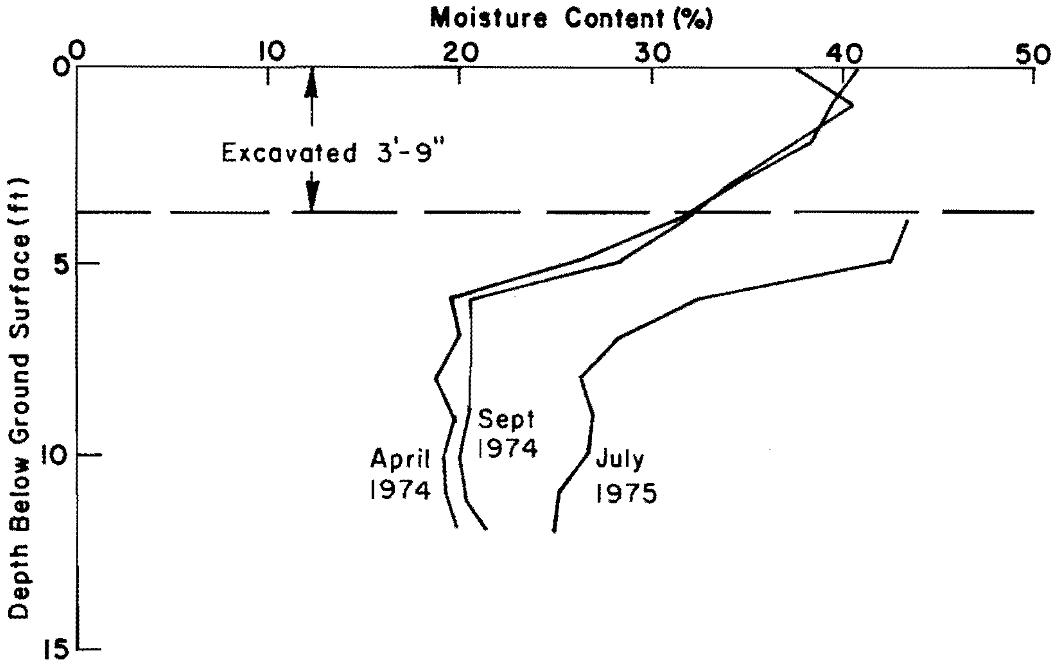
In test area 3, about 6 inches of heave occurred between October 1974, when the ponding occurred, and the end of observations, on 6 July 1975. As can be seen in Fig 5.8c, the heave was not linear with depth as had been observed in test area 1 (see Fig 5.3). This is due to the shorter time involved, which did not allow for a complete swelling process. The delayed arrival of water at the greater depths can be seen in the heave curves of Fig 5.7. At the 3-foot-3-inch depth the swell started within a couple of weeks, whereas initiation of swell at 7 feet 3 inches was delayed several months. At the completion of the observations, significant movement had not yet been detected at a depth of 10 feet 9 inches below the excavated surface. Similar behavior is evident for test area 1, except that the swell at depths below 11 feet began to develop very soon after ponding was done.

A short summary of the observations at test area 3 shows the following:

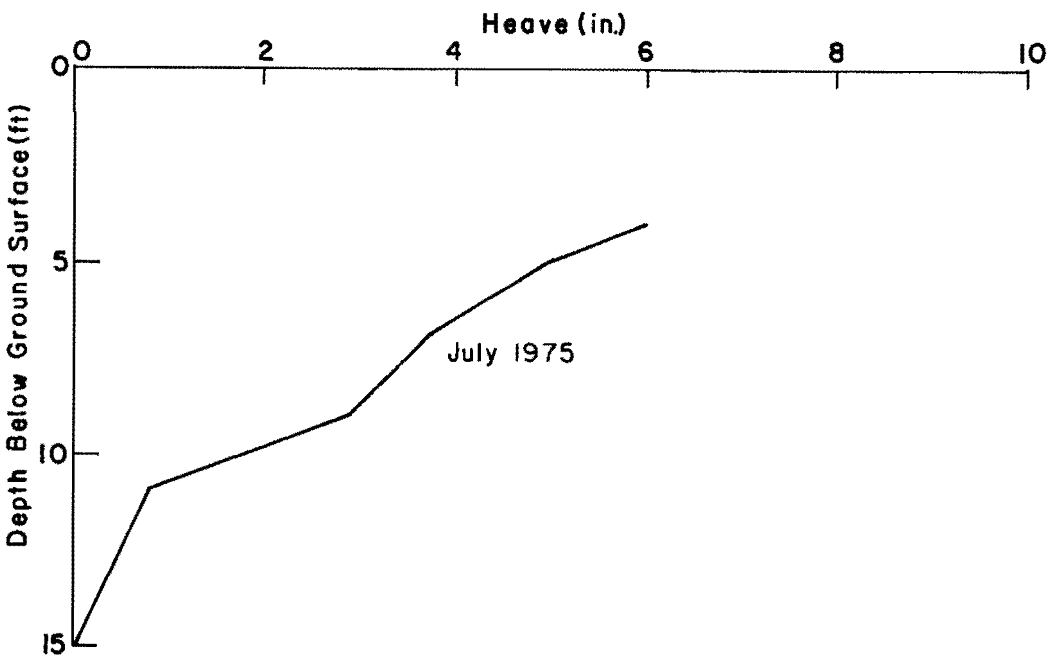
(1) Average moisture content in the top 7 feet increased from 20.6 to 33.3 percent.

(2) The corresponding dry density decreased from 97.1 to 92.2 pounds per cubic foot.

(3) There was a total heave of 6 inches because of swell beneath the depths of 4 and 15 feet, an average vertical swell of 0.55 per foot of depth, but with the swell in the lower layers incomplete.



(a)



(b)

Fig 5.9. Summary of results from test area 3.

CHAPTER 6. DISCUSSION AND CONCLUSION

Summary of Experiments

The ponding experiments at Lake Long, Austin, Texas, were conducted for the purpose of developing and evaluating techniques for field measurements of the behavior of expansive clay and for obtaining data on a case study under controlled conditions. No measurements had been found in the literature showing the progressive migration of moisture and development of swelling as a function of depth in the soil.

The experiments were not designed primarily to serve as an evaluation of the benefits of pre-wetting of highway subgrades by ponding, although a careful study of all the observations provides considerable insight for such applications. The need for an understanding of rate and depth of moisture penetration and therefore of the length of time needed for ponding was particularly evident in the planning of the field tests reported in the sixth and eighth reports of this series (Refs 3 and 9).

The ponding experiments were conducted in two phases, using four 40 $\times$ 40-ft test plots. One plot was maintained in its original condition to serve as a control. Two of the plots were cleared of surface soil and were observed for almost two years. Because of torrential rains and premature initiation of swelling, actual ponding was postponed for eleven months. One test plot had been held in reserve but, to give an improved record of the early phases of swell, was subsequently ponded and observed for approximately one year.

During the experiments, measurements of heave at various selected depths were made and nuclear and gravimetric measurements of moisture change were recorded.

Performance of Equipment

To measure heave, small metal points were pushed or driven into the clay to maximum depths of about 15 feet below the original ground surface. Trailing wires encased in very small teflon tubing and polyurethane sealant were used to observe movements. The method was found to be convenient, inexpensive, and completely satisfactory for observing movements, without the disturbance caused by drilling, and placing of more cumbersome devices. The techniques of installation are given in Chapter 3 for future use.

The nuclear probe observations of moisture and density were adequate for monitoring moisture migrations although some troubles were encountered with instrument malfunction and with delays in repair. Careful recalibration of the instruments in drums of prepared soil was found to be necessary. However, it should be pointed out that to monitor the swelling process the changes of moisture and density are of more interest than absolute values, and for this purpose the equipment was generally satisfactory. Some of the soft aluminum access tubes were deformed by lateral pressure from the clay but the inside diameter was resotred by the insertion of a mandrel.

No results are shown for the psychrometer observations of soil suction, primarily because of the infiltration of free ground water into the probes and because of times required to stabilize the humidity in the access tubes after opening to check for free water. However, the technique of inserting psychrometers into previously installed access tubes at the time of observation still holds considerable promise, at least for research-type observations. Some re-design of the access-tube technique would be in order. Otherwise, double-junction psychrometers and associated instrumentation, although delicate and sensitive, were generally satisfactory but are not recommended for routine observations of highway subgrades.

Review of Field Test Results

Three different test plots were ponded and observed over two different time intervals. The results from test area 1 show heave of the soil surface (as excavated) of almost 10 inches over a two-year period. At a depth of 5 feet the heave was 6 inches and at 10 feet more than 2 inches. Although the rate had begun to diminish, the surface heave was continuing at the end of the observation but the location of the swelling was primarily in the soil at depths greater than 10 feet.

Test area 2 was used primarily in an attempt to observe horizontal infiltration from the walls of a trench. Because of the extremely heavy rainfall during placement of instrumentation, this aspect of the test was not fruitful. However, the results are in agreement with test area 1. Furthermore, the observed strong influence of fissures in the clay in the top 5 feet would tend to override or at least supplant the effects of a trench.

Test area 3 provides an excellent record of heave during almost 9 months, following a clear and definite initiation of ponding. The surface heave reached almost 6 inches and at a depth of 7 feet was approximately 1 inch. However, at 11 feet, the swelling process had not yet started. This is in general agreement with the records of test area 1.

Detailed profiles of heave and moisture data may be found in Chapter 5. Generally, increases in moisture content were less than 10 percent during the period of observation, with corresponding changes in dry density.

Discussion of Results

Perhaps the most significant single result of the tests in the three ponded areas is that, even after a period of two years, the heave was continuing and the depth of moisture penetration was still increasing. Although the fissured nature of this particular deposit may have caused a more rapid penetration of moisture than might occur elsewhere, the infiltration rate was not reduced as much as would be expected after the upper layers had swelled an amount which should have closed the fissures. This suggests that the fissures themselves may contain permeable debris.

Attempts were made to compare the behavior observed with that predicted using the computer program (GCHPIP1) presented in Research Report 118-3 (Ref 1). It soon became apparent that some of the soil properties required for the input could not be accurately modeled. The more significant of these properties were the permeability and the changes in permeability with increasing moisture content.

It is clear that the traditional concept of permeability is not applicable to this clay. The highly fissured nature of the soil and the rapidly changing permeability as the clay swells when water is introduced do not lend themselves to the requirements of the program. From the records obtained and described in Chapter 5, it is postulated that the fissures, normally open at the beginning of the ponding process, conduct free water rapidly to considerable depths in the soil. Once the fissures are filled, the water is absorbed into the intact lumps between the fissures. As this process occurs, the intact lumps swell and close the fissures, causing a rapid reduction in the permeability. Thus the bulk permeability of the soil mass is greatly different from a laboratory sample and is dependent upon the moisture content. The variation in permeability is also time-dependent, making a quantitative description of the permeability impracticable.

In an attempt to duplicate the observed infiltration rate in test area 3, a few solutions were made assuming a saturated permeability of 1×10^{-5} inch/second. The increases in measured moisture content were greater than were predicted using this value of permeability. Larger values of permeability were then input, but the computer solutions became unstable and were inconclusive.

It was then concluded that, because of the fissured nature of the clay and the difficulties in attempting to describe the permeability, the soil model used in the computer program was not applicable in this case.

The Potential Vertical Rise for the site as predicted using the Texas Test Method Tex-124-E was 4 inches. As noted earlier, the measured heave under sustained ponding was almost 10 inches in test area 1, with only a moderate decrease in the time-rate of heave evident after almost two years.

Recommendations

The records of performance of a highly active Central Texas clay soil under sustained ponding may be used as an approximate upper bound of expected behavior of other troublesome clays. The observation of the effects of progressive moisture change at various depths gives a clear picture of the process. The continuing nature of the swell process for relatively long periods of time may be a significant factor in highway design.

A considerable amount of swell occurs in a few months and pre-wetting by ponding should be considered of substantial benefit if the absorbed moisture can be retained. However, it is not by any means a total cure, since it usually will not be practicable to pre-wet the deeper layers.

Full advantage should be taken of any future opportunities to observe moisture changes and movements of known swelling clays. The techniques demonstrated in the present project may be employed in such observations.

Only by making observations at several depths over an extended period of time will a clear understanding be developed for any given case. Knowing the performance of such clays in particular climates and particular localities will be of considerable value in roadway design.

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