MEASUREMENTS OF A SWELLING CLAY IN A PONDED CUT

by

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PREFACE

This is the sixth in a series of reports from Research Project 3-8-68-118, "Study of Expansive Clays in Roadway Structural Systems." It describes various attempts at cures for the swelling clay problem and takes a detailed look at ponding as a solution. The developments and instrumentation of the ponding of an expansive clay cut on U.S. 90 are recorded and assessed as a possible solution to the problem of limiting the destructiveness of these clays.

This report is the product of the efforts of many people. Technical contributions were made by Hudson Matlock, James Anagnos, John Wise, Thomas J. Walthall, Donald J. Frye, Charles Baxter, Henry Hardy, Paul Wright, and Eugene Baldauf. Preparation and editing of the manuscript were done by Mary Geigenmiller, Wanda Mayes, Vince Bradfield, and Gerald Hewitt.

The Texas Highway Department contact representative Larry Buttler and personnel from District 15 and its Bexar County Residency have been helpful and cooperative in the development of the work. Thanks for sponsoring this work are also due to the Federal Highway Administration, and are extended to their area representatives S. A. Ball and W. T. Kelley, who provided valuable technical and personal assistance.

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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.

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.

ABSTRACT

Ponding is the containing of water on a clay subgrade of high swelling potential in order to cause potential soil surface heaves to occur before a highway pavement is constructed. The effectiveness of the use of ponding, in Texas and elsewhere, over the last four decades is reviewed here.

A deep cut section of U.S. 90 under construction west of San Antonio was ponded for from 30 to 45 days in the spring of 1970.

Deep bench marks set at 2 feet, 3.3 feet, 4.5 feet, 10.5 feet, and 19 feet were used to record vertical movements before, during, and after ponding at three locations within and one location outside of the pond. Nuclear depth probes were used in three access tubes to measure changes in moisture content and dry density in the vicinity of one set of bench marks.

The surface water penetrated only the upper 3 feet of soil during the ponding but some wetting at depth was indicated after the area had been drained. About 50 percent of the potential vertical rise was achieved by the ponding.

KEY WORDS: clay swelling, ponding, pavement roughness, nuclear measurements, construction methods, density, ground water, highways, permeability, water content, instrumentation.

SUMMARY

The technique of flooding a clay subgrade with water to cause the soil surface to heave before a pavement is constructed rather than after was demonstrated in a research project on U.S. 90 west of San Antonio. A deep cut was made in a potentially high swelling clay, and personnel of District 15 of the Texas Highway Department and the Center for Highway Research cooperated in measuring heave and changes in water content and density.

About 50 percent of the heave which was expected occurred during the ponding period, but some uplift continued up to six months afterward.

Although the method is effective, construction equipment must be kept out of the area during the ponding and for a week after.

IMPLEMENTATION STATEMENT

Ponding is a practical method of causing a soil which may heave to do so before a pavement is placed rather than after. The method was tested successfully at Waco, Texas, in 1957 and the results reported here confirm the findings of that earlier test. The complete story of the Waco project was not published; the prime purpose of this report is publication of the procedures and results of the ponding project at San Antonio in 1970, so that they will be available to engineers involved in design and construction decisions concerning highways over swelling clays.

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CHAPTER 1. INTRODUCTION

Swelling clays have long presented construction problems throughout the world. Whether they are called erratic nonuniform swells or subsidences, swelling clays have regularly been detrimental to engineering efforts to provide a better environment for the world population.

Various techniques have been used in efforts to cope with these clays. The effort described in this report used one such technique. It involved the ponding of an area of clay extending through a considerable cut on a highway construction project just west of San Antonio. Various devices were used to measure the moisture, density, and elevation changes, and the effectiveness of the devices as well as the effectiveness of the ponding in dealing with the problem of swelling clays was noted. Of special interest is an attempt to determine the depth at which the elevation movements took place and to correlate them with the moisture and density readings.

Generally, an expansive clay is considered to be one which shows extensive changes when wetted or dried. All soils exhibit some change in volume with change in moisture content, but the troublesome soils are those which cause damage or intolerable deformation to structures overlying them or to foundations embedded in them.

To the average motorist, irregular but pronounced bumps in the roadway, asphaltic leveling on stretches of a fairly new concrete pavement, and a heater-planer at work reducing humps on long areas of asphaltic pavement are indications of the presence of swelling clays. Buildings in swelling clay areas frequently have cracks in the foundations, walls, and ceilings, and additions to these buildings often show considerable difference in elevation. Sidewalks and curbs near such buildings are frequently heaved around at different angles, and in a dry season there is considerable cracking in lawns and fields.

A comprehensive treatment of the subject of swelling clays beneath pavements has been made by Kassif et al (Ref 11). They list damage to pavements from swelling clays as

(1) unevenness along a significant length of the road surface without cracking or visible damage;

(2) longitudinal cracking parallel to the road centerline;

(3) significant localized deformation, near culverts, for example, generally accompanied by lateral cracking; and

(4) localized failure of the pavement, which is associated with disintegration of the road surface.

Maintenance of the highway system in Texas is such that usually only the first type of damage listed by Kassif et al is evident; however, within municipal limits severe breakup of pavements caused by underlying soil rather than traffic is often evident.

In Texas, troublesome soils are generally differentiated by the McDowell method of calculating a potential vertical rise (PVR) value for the soil stratum from field measurements of swell in various soils throughout the state (Ref 17). McDowell (Ref 13) suggested that when potential vertical rise values based on average moisture content exceed one-half inch, steps should be taken to reduce the detrimental effects of the swelling soil. Research Report No. 118-5 (Ref 19) contains a survey of the expansive clay problem in Texas.

Several approaches to solving problems of swelling and shrinkage of soils have been tried at various times. A drastic method is removal of all soil which could cause excess vertical movement and replacement with a nonswelling material. This was used successfully by McDowell, Senior Soils Engineer for the Texas Highway Department, to solve a problem concerning the foundation of the Highway Department laboratory building in Austin (Ref 13). He determined that there was a relatively thin layer of clay that could cause uplift to the building slab and removed the potentially destructive material. Unfortunately, it is not financially feasible to treat all clays this way; some strata, for example, are too deep to be removed and there may be insufficient nonexpansive material available for replacement.

Stabilization with lime is a solution which has many advocates in Texas and Oklahoma. When lime is mixed with clay, the swelling potential of the mixture is drastically reduced, and the region of low swelling potential is further extended when the lime is carried through the soil by infiltrating water. However, lime does not stop heave caused by wetting from a rising water table or from lateral seepage which does not pass through the lime-treated clay before it enters the untreated clay.

In the late fifties, lime stabilization of the subgrade to reduce pavement movement caused by swelling clays was reintroduced to Texas highway construction in work in Bexar County. In general only the top 6 inches of the subgrade was treated and application rates were dependent on the plasticity index (PI) and unit weights of the material. The scale varied from 2 to 4 percent and the PI was grouped from 15 to 25, 25 to 35, and 35 and up. Lime was added after the subgrade was scarified, and it was disced, plowed, rolled, and watered, and field densities were taken. Invariably, the PI declined, and the sites were operational after rains sooner than untreated sites. However, although few controlled experiments were tried, there appeared to be little reduction of swelling in the pavements that overlaid the lime-treated subgrades. In part, this work, which lasted over a decade, has given rise to the conclusion that movement comes from more than the first couple of feet of superficial subgrade.

At four Bexar County locations, slope stabilization with lime was used to retard earth slides. Holes spaced about 5 feet apart and varying from 12 to 18 inches in diameter were drilled to a depth of 20 feet. A sack of lime per foot of depth was added and mixed with water, and the holes were backfilled with base course material. These sites have continued to reflect stability several years later.

In another stabilization attempt, holes were drilled to a similar depth and the lime slurry was pumped in under pressure. Recent investigations revealed that the lime had congregated in fissures, and the effectiveness is undetermined. Lime stabilization of bases 2 feet deep with holes 5 feet apart has also proved effective at several sites in the area.

A solution to the problem of getting lime deeper into the soil has been the subject of an experiment in Oklahoma, where the soil was plowed to the 2-foot level and lime mixed with it as described in the 6-inch Texas method. These tests showed that strength was added and imply that an economic advantage could result from reduced base requirements. Considerable variation in the amount of lime thus admixed was indicated.

Lime was also used in conjunction with the underdrain system described below. Test holes in the ditch, drilled 18 inches in diameter and 20 feet deep, were backfilled with lime at the rate of a sack per 5 feet of hole depth. Most of the holes were filled with water to varying levels, which dissolved the lime, and backfill was completed with natural gravels.

Underdrains or interceptor drains are another possible solution to the swelling clay problem. They have been used in Bexar County by the Texas Highway Department on 11 projects in the last twenty years (Ref 16). The results of this method, indicated by the smoothness of the pavement and whether or not water still runs from the underdrains, suggest about a 50 percent success. On five projects, the drains function well, and the pavement remains smooth; on four, there is no discharge of water and the riding surface is rough; and on the other two, the drains do not seem to work but the pavement is smooth.

In a case described by Steinberg (Ref 16), a swell was discovered during a construction project. Investigative drilling in the ditches revealed a pronounced rise in the water table within the limits of the swell, and a double line of underdrains was installed to intercept the water. The pavement in this area has swelled about 3 inches since the drains, which continue to work well, were installed. As noted above, the test holes in the ditch line were treated with a sack of lime for every 5 feet of depth prior to backfilling. Including the almost 4 inches of swell which occurred before the underdrains were added prior to the placement of the reinforced concrete pavement, the total movement is about twice the calculated PVR. Without the lime treatment, the rise would undoubtedly have been much greater.

Another approach to solving the problem of swelling clays, which is more economical as far as direct expenditure is concerned, is being made in District 19 of the Texas Highway Department. A deep cut was allowed to come to equilibrium with its environment for a period of several months. Wetting action was encouraged by disking the surface to improve infiltration and hinder evaporation. This research, which has been reported in preliminary form (Ref 19), is continuing.

Ponding, another approach to the problem, had been used in Bexar County projects by the Texas Highway Department in the 1920's and 30's. This method was designed to get moisture into clay embankments and was used with jetting as a compactive effort. The development of water trucks and rollers and the discovery that some jetting led to severe cavitation resulted in a long hiatus in the use of ponding on highway work in Texas. However, ponding is currently being used in two jobs now underway in the western part of Bexar County. They are adjacent, but there is a significant difference between the two. In one the entire subgrade from crown to crown of the mainlane cut section is ponded, and on the other only the subgrade under the westbound lane is ponded.

CHAPTER 2. DEVELOPMENT OF THE PONDING SOLUTION

Probably ponding was used on some Texas highways as much as 40 years ago. One of the better documented projects was begun in January 1932. Extending along the present route of Interstate Highway 10, from Seguin to Cibolo Creek, a Texas Highway Department grading contract covered approximately 14 miles of Guadalupe county. The work was mostly embankment, and fills were generally a minimal foot and a half. Heavier fills were over culverts and at bridge approaches. Steam shovels were used for earthwork and mule-drawn fresnos for hauling, and compaction was accomplished mainly with mules, blades, and fresno wheels.

The Guadalupe county research project (Ref 2) was initiated by the Texas Highway Department in the late 1920's, when the movement of concrete pavements began to attract unfavorable attention. Jetting and ponding were used on the heavier embankments, but records indicate that this did not take place until the embankments had been in place for two or three months. Apparently these efforts were planned to bring the top layer of the embankment to a moisture content equivalent to the field moisture. Frequently, portland cement concrete pavements were placed directly on the clay subgrade without a base course.

The Guadalupe research project was concentrated on the paving phase of the contract, but there are many points relevant to the problems of expansive clays. It was part of a research effort to determine causes of uncomfortable riding and at times dangerous irregularities in pavement surfaces. Six-inch iron hand holes in the pavement were used to get moisture content soil samples at depths of 6, 12, 24, and 36 inches beneath the pavement. Sampling began in 1933 and continued on a regular basis through 1936 and sporadically since then. The initial conclusions from the Guadalupe research project were that there was little moisture change 36 inches beneath the pavement and that there were no appreciable pavement irregularities when a 6-inch layer of sand was placed between layers of similar clay, with the top one immediately under the pavement. Subsequent testing showed that a stable moisture content was developing after four years.

In 1942, Porter noted that clay, water, and highway had long been recognized as a difficult combination because clay soils had caused much difficulty in the past (Ref 15). Porter's series of eleven reports concluded that clays provided the least support to loads when most saturated. He noted the need to correlate laboratory and field testing and to direct pavement design toward load distribution to avoid rupturing of soil structures, a condition which tended to occur when nonuniform rapid moisture changes occurred in consolidated clays.

In the 1930's, jetting was determined to be dangerous. The cavities it created were as serious a problem as the undercompaction it was meant to cure (Ref 2), and it faded from the scene along with ponding, which, however, continued to be used to cure concrete.

The biggest highway ponding project in Texas up to its time was probably the Waco experiment, which pointed out new directions and sophistication in the field of earth sciences. Rather than attempting to compact soils by ponding and jetting as in the 1930's, the Waco project attempted to rid the riding surface of irregularities, pronounced swells that made for discomfort and could cause a driver to lose control of a vehicle.

The Portland Cement Association began the study of the problem of irregular displacement of concrete pavements in the late 1940's north of Waco along U.S. Highway 81. The resulting report discussed the "dry bulb" effect of expansive clay, which took on much more moisture than adjacent areas of higher and more uniform moisture (Ref 6) and expanded volumetrically much more than the adjacent soils, causing a hump at the surface. In the study some areas were ponded to determine if the controlled addition of water would cause the swell to occur before construction began. In one case, there was 10 inches of swell. The formation investigated was Wilson clay loam developed from the Taylor Marl parent material and had a plasticity index of 20.

Two areas were ponded. At one site, 4-inch-diameter holes were drilled to a depth of 8 feet at 5 feet on centers to allow easier access for the water. At the second site 4-inch-diameter holes were drilled to a depth of 7.5 feet at 6 feet on centers. All holes were filled with water each day except Sunday between August 25, 1948, and January 14, 1949. Because the holes tended to fill with mud after several fillings, some holes were filled with pit run sand and gravel, some were filled with pea gravel, and some were left open.

Most of the water entered the upper 3.5 feet of soil. The quantity of water added was so small compared to the quantity of soil being wetted that the swelling process was very slow and some parts of the soil were still below the shrinkage limit two months after the filling began.

On October 25, 1948, two small diked areas 20 feet square were built and flooded and left covered with water until January 14, 1949. In the 40 days prior to flooding there was no evidence of surface heave resulting from the daily filling of the holes with water, but after three days of ponding, the surface rose 1 inch. At the end of the test, swells were 2 to 4 inches, which was considerably more than the heave in the unponded section, where water was introduced only through holes. The ponding did not cause slaking of the surface soil, probably because the grass roots and blocky structure of the undisturbed soil provided stability.

Movement of water into the soil from the holes was accelerated by applying a pressure of from 25 psi to 90 psi in the sealed holes. To further check the relative effectiveness of introducing water through holes, two 15-foot square areas were ponded. One had nine 4-inch-diameter holes drilled at 5 feet on centers to an 8 foot depth and filled with pea gravel, and the other had no holes. The results showed the holes to be of little value in wetting up the soil.

The PCA report pointed out that the apparent reason for easy penetration of the ponded water was the condition of the soil, which was natural, undisturbed, block-structured dry clay. Because similar results could not be expected in a disturbed soil, it was recommended that ponding be completed before any grading was done.

Using the knowledge gained from these PCA experiments and the McDowell potential vertical rise method of predicting swell, the engineers of District 9 of the Texas Highway Department planned subsequent lanes of U.S. Highway 81 to avoid the movement there had been in the older adjacent ones. They conducted considerable coring and testing. Using the PVR calculations, they determined which areas might rise an inch or more and decided to pond all such areas in which the fill required would not be more than 6 feet. Eighteen sections, almost 27 percent of the project, ranging from 400 to 1800 feet long, were ponded in 1957 and 1958. The immediate test results indicated that moisture descended no more than 4 feet, that it began to increase 20 feet down after a few days of ponding, and that it ascended to the 4-foot level within

24 days. The ponding lasted from 22 to 41 days. By 1965, only 13 percent of objectionable swells, those easily detected when driven over at a normal speed, occurred in the ponded areas (Ref 4) if the swells at the ends of the ponded areas are discounted only 8 percent were inside the ponded areas. Thus, ponding seems justified since the areas ponded were thought to be possible trouble spots, with the remainder of the roadway having a lower swelling potential.

Another interesting result was noted three years after ponding; swelling occurred along the old pavement in the opposite lane of the divided highway parallel to the ponded areas as well as at the ends of these areas. Moisture migration was thought to be the cause and raised the question of the need for adequate barriers.

Ponding on the Waco project finally covered 82,950 square yards and used 5,220 thousand gallons (MG) of water. The contractor price included $0.75 per MG for water and $0.015 per square yard for the treated area.

In a project at Vereeniging, Transvaal, South Africa, described by Blight and DeWet in 1965 (Ref 3), ponding was used in an attempt to accelerate swelling and cause most potential surface movement to take place before the slab was placed. To support a light building, a concrete slab-on-grade was to be placed on a 19-foot-thick stratum of potentially expansive firm-to-stiff fissured silty clay in which heave was predicted to be about 2 inches.

To increase the depth of active swelling, six 20-foot holes were drilled, and the surface was flooded for 96 days. Swell continued for 18 months more as water seeped from the holes into the surrounding soil. Ninety percent of the full potential heave was achieved before the slab was placed and settlement due to reconsolidation amounted to about .4 inch.

Only surface elevation changes were measured. A coefficient of swell was calculated from soil samples placed in triaxial cells and allowed to swell under an all-around pressure, and it was concluded that by using the coefficient of swell with normal consolidation theory, well spacing could be designed which would lead to full heave within a specified time.

For a ponding project in California in 1965, reported by Gizienski and Lee (Ref 7), an 8-foot by 26-foot pit was excavated in a silty fine sand and backfilled with a moderately expansive clay. Two 4-foot by 4-foot concrete slabs were placed on the surface, with 13 inches of clay under one slab and 36 inches under the other. After allowing a one-month equilibrium period, the soil, but not the concrete, was flooded for 100 days. Elevation changes were recorded for the slabs and moisture and density were checked before flooding and after saturation. The heaves of 0.4 and 0.9 feet beneath the slabs were approximately one-third those predicted from surcharged laboratory swell tests on compacted samples.

In California, also, ponding was used for canals being built under the auspices of the Bureau of Reclamation (Ref 1). Four experimental test sections were built in the San Luis Drain, and it was found that foundation clays below 4 feet were not expansive if their natural moisture contents were preserved. The only significant expansion of soils with liquid limits of more than 50 percent occurred in soils with natural density exceeding 100 pcf near the ground surface and partially desiccated.

Test sections 500 feet long were ponded for four months to increase the natural water content to a minimum value. Moisture-density determinations were taken at 2-foot intervals to depths of 10 feet using 3-inch diameter drive-tube samplers at two, four, eight, twelve, and sixteen weeks after wetting began.

Surface elevation changes were also recorded. In one test section, 30-inch grooves were made to facilitate water penetration, and the surface of another was plowed for the same reason. Surface heaves of .14 to .26 feet were recorded. Penetration of water appeared to reach 10 feet but tell-tales at the 3-foot depth showed little movement. Sixty percent of the expansion occurred during the initial 30 days of wetting.

The ponding method proved impractical where there were lenses of pervious sand, which acted as drains. It was considered important to keep the soil above the specified minimum moist condition until construction covered it. It was specified that a vibratory roller be used twice over embankment foundations immediately after wetting operations were discontinued to heal old shrinkage cracks.

No simple statement can cover the relative effectiveness of the three methods of wetting, i.e., simple ponding, ponding after plowing, and ponding after deep grooving.

Two other Texas projects concerning concrete slabs-on-grade are of interest. One was not a ponding project, in that the soil surface was never flooded, but it is included to indicate still another way of introducing water into the soil.

In one project, the problem was to stabilize an expansive clay of variable depth and area beneath a concrete slab-on-grade so that glass partitions on the first floor of a proposed large structure would not show distress (Ref 10). Based on laboratory tests, predictions of swell over the building site varied from 2 inches to 5 inches. Preconstruction swells of the order of the predicted heaves were accomplished by infusion of water. Trenches 6 inches wide by 3 feet deep were excavated 10 feet apart and filled with 1 foot of lime and 2 feet of gravel and with a 0.25 percent solution of Kyro EO in water, which reduced the surface tension and allowed more efficient penetration. A satisfactory increase in moisture content of approximately 8 percent over a depth of 10 feet was achieved in about one month.

In the other project, ponding was used at the site for the Texas Highway Department Equipment and Shops Building at Austin (Ref 13). The Del Rio clay at the site, which had a potential vertical rise of 6 inches, was excavated to a depth of 6 feet and ponded for 30 days. At the completion of ponding the soft surface was mixed with 4 percent lime to a depth of 12 inches and within three days trucks were able to haul granular backfill onto the lime-stabilized surface. Ponding and subsequent backfill proved to be a satisfactory solution for this floor slab-on-grade.

CHAPTER 3. PONDING OF THE CUT ON U.S. 90

Recently a project involving ponding, the first in the area in about 40 years, has been underway on U.S. 90 west of San Antonio (Fig 1).

Plans prepared by the Texas Highway Department develop this section as a four-lane divided highway with controlled access, grade separations, and frontage roads that give unlimited access to adjoining property. The mainlanes have two 12-foot driving lanes with a 10-foot shoulder outside and a 6-foot shoulder inside. The median between the two mainlanes is 48 feet wide. The frontage roads vary in width from 24 to 32 feet. They serve as one-way facilities from the east end of the project through the first interchange. From there westward the frontage roads are two-way. Grade separations are located at Hunt Lane and Dwyer Road and at F.M. 1604, and there are other span structures at Medio Creek. At the east end of the project, part of the existing pavement is resurfaced and is part of the future Westbound mainlane.

Geologically, the site is a Pleistocene high-terrace deposit overlying the Taylor formation. The Taylor formation at the ponding site is a greenish-gray calcareous nodular clay which extends at least 20 feet below the bottom of the cut (Ref 9). About 5 feet of the terrace gravel was left on the site at the east boundary of the ponded area, but excavation had cut into the clay at station 250+00 and all points further west.

X-ray diffraction analyses were made of two samples within the ponded area by the Bureau of Economic Geology at The University of Texas at Austin. A sample of clay at the base of the gravel at station 242+00 was 35 percent calcium montmorillonite, 50 percent illite, and 15 percent kaolinite and chlorite. A second sample, taken from a depth of 15 feet, 4 inches at the other end of the site, was 30 percent calcium montmorillonite, 60 percent illite, and 10 percent kaolinite and chlorite. A soil profile developed from core samples taken in the area is shown in Fig 2.

Previously, highway department engineers had been aware of potential problems from swelling clays in the San Antonio area, and the District Laboratory Engineer, who was involved in the Guadalupe research project, had maintained a continuing interest in that project as well as other investigations

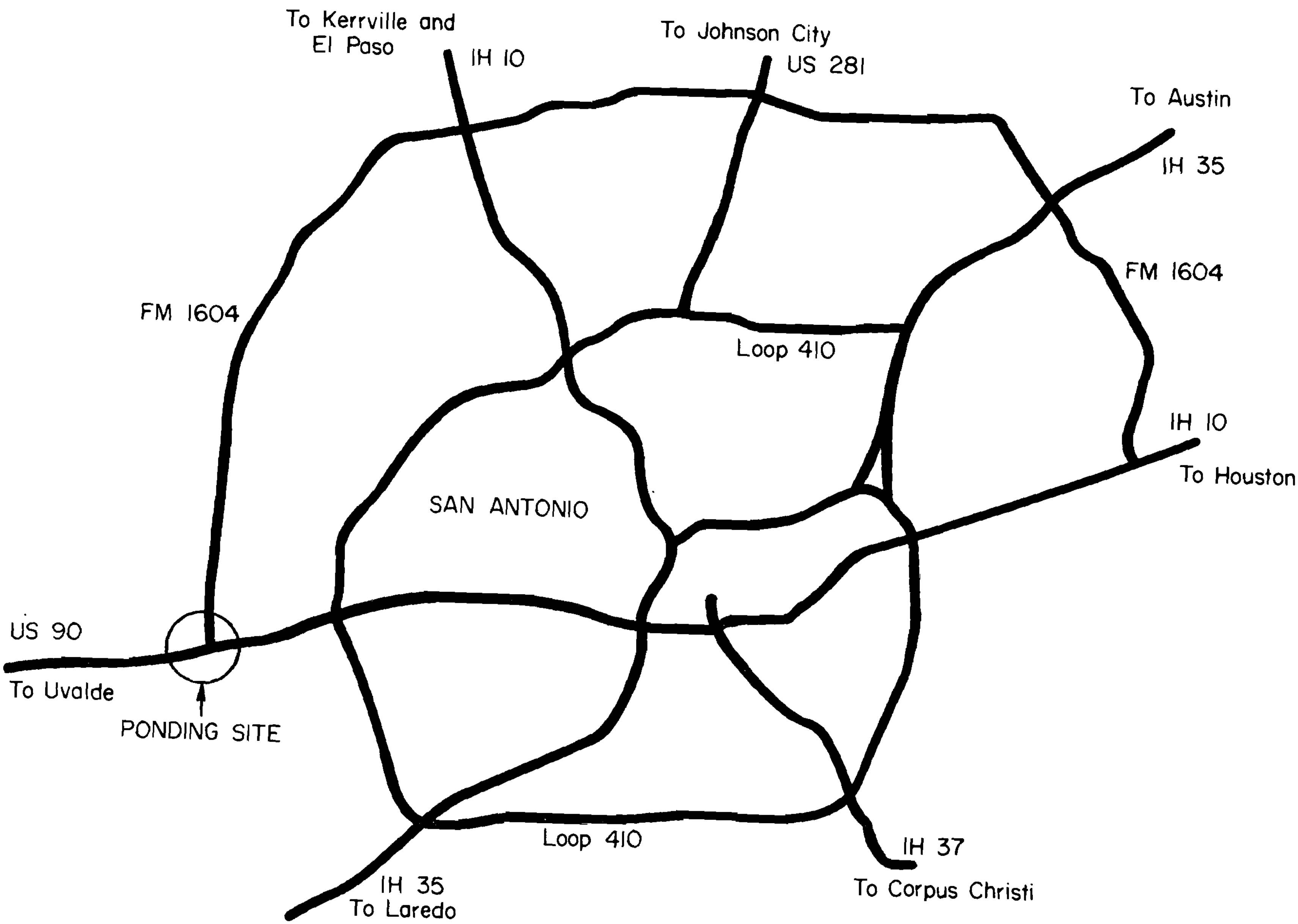


Fig 1. Location of ponding site.

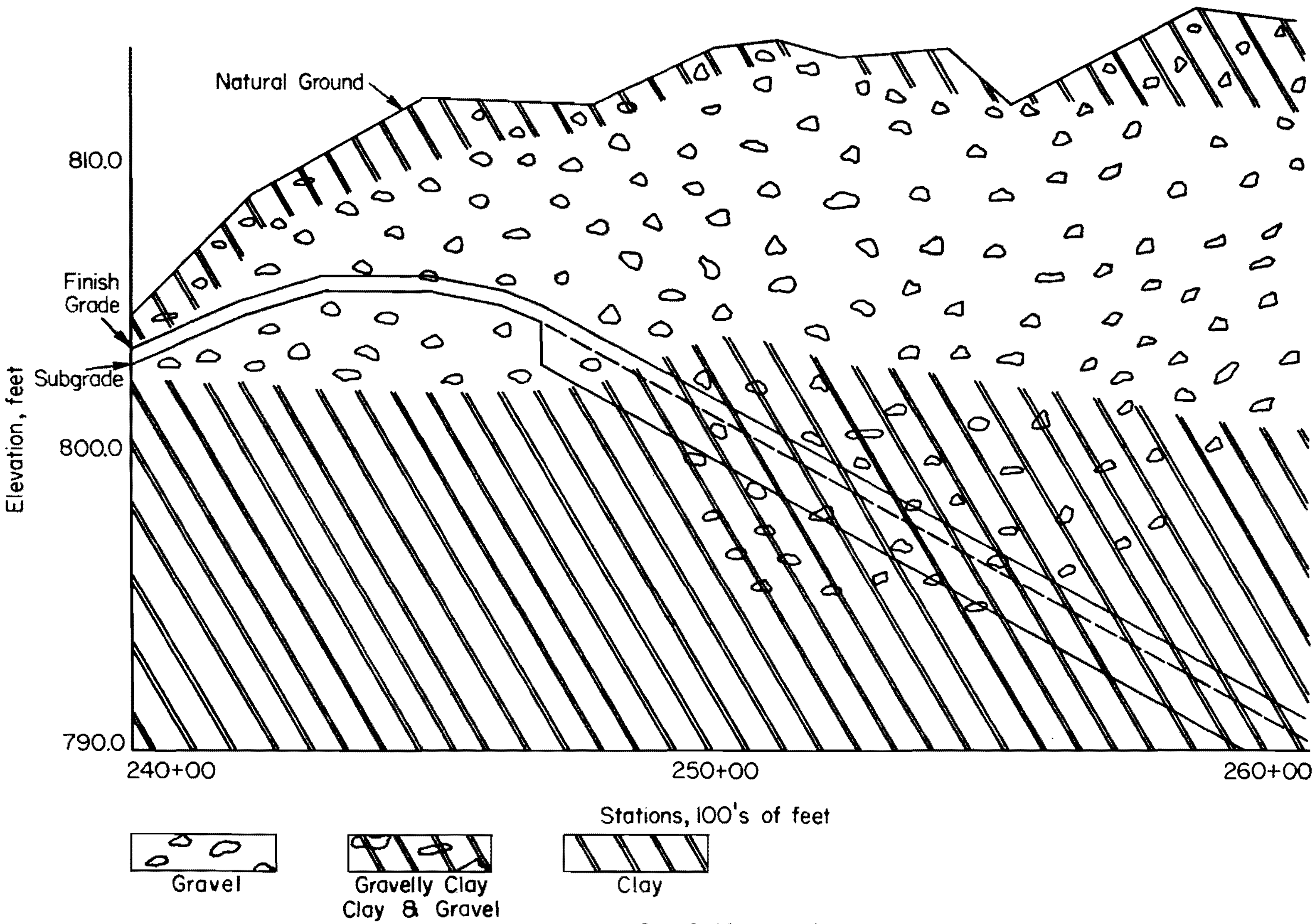


Fig 2. Soil profile.

into the problem. McDowell's work and its application to the Waco project were used as guidelines for recommendations concerning the present work.

Extensive exploratory core drilling was carried out, and calculations confirmed the anxieties concerning expansive clays. It was realized that the planned removal of considerable overburden in the FM 1604 - U.S. 90 interchange area added the problem of potential rebound to the problem of swelling clay. The cut reaches a maximum of 27 feet and the maximum PVR was calculated to be almost 6 inches. The District Laboratory Engineer recommended ponding the area from 3 feet up the back slope from one side to the other, completely covering the mainlanes, median, and shoulders. Ponding was scheduled for thirty days, after which the ponded area was to be lime-stabilized in an attempt to hold the moisture. As further protection for the riding qualities of the mainlanes in this area, select material salvaged from the cut was spread 18 inches deep across the entire section (Fig 3).

This select material is one of several items of interest in this contract. The select material has considerable gravel and a relatively low plasticity index. It is stockpiled, and leftover material is used on an adjacent project.

At the east end of the ponding area an exposed shallow gravel stratum is drained by 6-inch underdrains placed on the line of the outside mainlane ditches. They extend from station 246 back to station 236, where an 8-inch connector line brings the eastbound ditch drain to the north ditch line and discharges both into the ditch between the westbound mainlane and the north frontage road.

Contract plans permitted the contractor a maximum flexibility to plan construction operations to achieve maximum quality at minimal cost, consistent with the heavy volume of traffic. The sole stipulation on construction sequence was that the frontage roads be built and traffic routed over them or other detours before the existing pavement on U.S. 90 was disturbed. In this case, the contractor elected to remove a majority of the cut from the area to be ponded before traffic was rerouted from U.S. 90, and the underdrains were set before the ponding began.

The contract included $0.60 per thousand gallons of water for ponding. It was estimated that 9,733 MG covering 88,981 square yards would be involved. However, before any ponding was begun a study of design data indicated that some of the areas most likely to swell lay just outside the initially contracted limits of ponding. A field change was made to include this area and

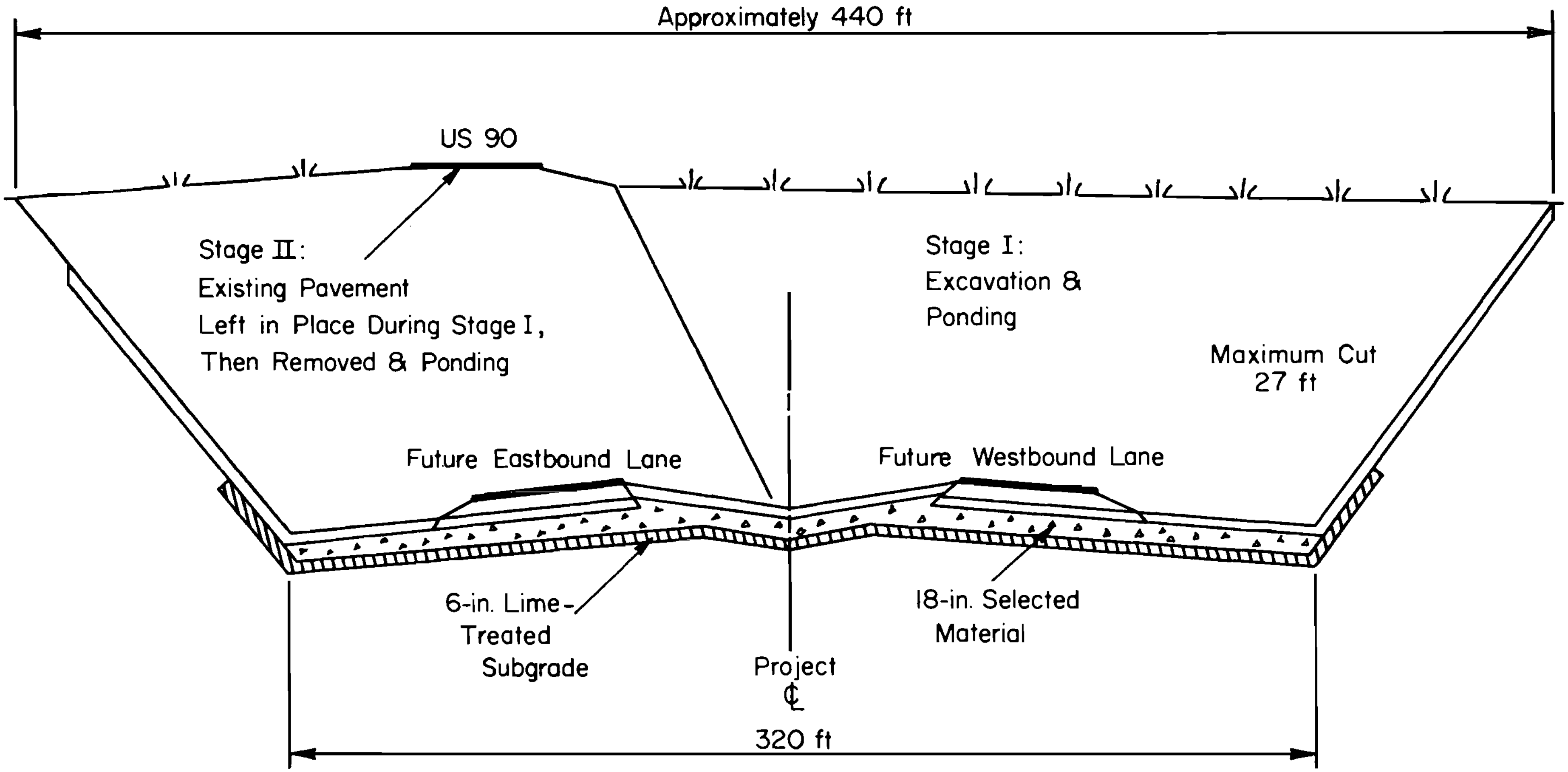


Fig 3. Cross section through cut showing construction stages.

the estimated quantity of water involved in the ponding was increased to 12,200 MG covering 110,621 square yards.

Excavation in the F.M. 1604 cut area was begun on June 9, 1969, and the first dikes were begun on February 11, 1970, and finished on February 18. Thirty-three separate dikes were in the first area to be ponded. They were spaced about 50 feet apart along the centerline and generally after leaving the median turned at a 45 degree angle to follow the contour lines of the section. There the distance at right angles to the dike was reduced to 25 feet. These first dikes extended from the north slope line to about 15 feet south of the median centerline. The width involved was about 175 feet. The remaining areas were to be ponded later.

The height of the dikes was usually between 2.5 and 3 feet. They were 4 to 4.5 feet wide at the base and tapered to 1 or 1.5 feet at the top. The ponds formed between the dikes were filled with water pumped from an 800-foot well by a 4-inch submersible electric pump. A 4-inch meter was used to measure the water quantity pumped into a 5-inch irrigation line that carried the water to the ponds. Over the first 30-day ponding period, 4,109 MG were used in this section. Ponding began with initial pumping on March 3, 1970, and all of the first series of ponds were filled by March 10. The 30-day period was considered completed on April 5.

The draining of the ponds began immediately and was finished on April 7. The initial attempt at getting the subgrade shaped failed when a dozer was bogged down, and it was not possible to get another machine to work the material for one week. Roughgrading got underway on April 14 and was completed by April 17. Finegrading was delayed by intermittant rains and was not completed until May 11. The first liming operation of the subgrade in the ponded area took place on May 12 and final mixing of these initial lands was completed on May 18 and 19.

CHAPTER 4. INSTRUMENTATION

An important aspect of this project was the checking of existing types of instrumentation. One reason that ponding has not become a general solution for the swelling clay problem is the lack of data concerning soil reaction to ponding. Information is needed on water content, dry density, soil suction, and elevation change, not only at the surface but at depths below subgrade, and should be obtained before, during, and subsequent to ponding. The effectiveness of treatment can be determined by observing the rate of heave, the reduction in soil suction, and the horizontal as well as the vertical variations in these changes.

Careful collection of these data over several time intervals would help to determine field permeability, the relationship between water content change and swell, and the relationship between water content and soil suction. Ponding can simplify the analysis of these relationships by stopping evaporation at the soil surface, by providing a constant water head at the soil surface, and by covering such a large area that lateral effects are predictable in the center of that area.

Crude measurements indicate the effectiveness of the ponding, but only careful and accurate measurements will provide data which will be useful in predicting behavior of the subgrade after the pavement is in place.

The assembly used to measure changes in elevation at depth is shown in Fig 4. Holes 9 inches in diameter were drilled to depths of 1.5, 4.0, 10.0, and 18.5 feet. An 8-inch polyvinyl chloride pipe was placed in the hole and projected 1.5 feet above ground. The space around the 8-inch pipe and inside the pipe was packed with auto chassis grease (Federal Standard Specification 791). A 1/2-inch galvanized water pipe which had a 3-inch augur plate at its bottom was placed inside the 8-inch casing. This augur plate was then advanced 1/2 foot into the ground at the bottom of the grease filled hole. At the top it projected about 1.5 feet above the PVC pipe. A cap was placed at the top of the water pipe to keep water out. The entire assembly was protected with a 4-foot-long section of 12-inch clay pipe, which was capped.

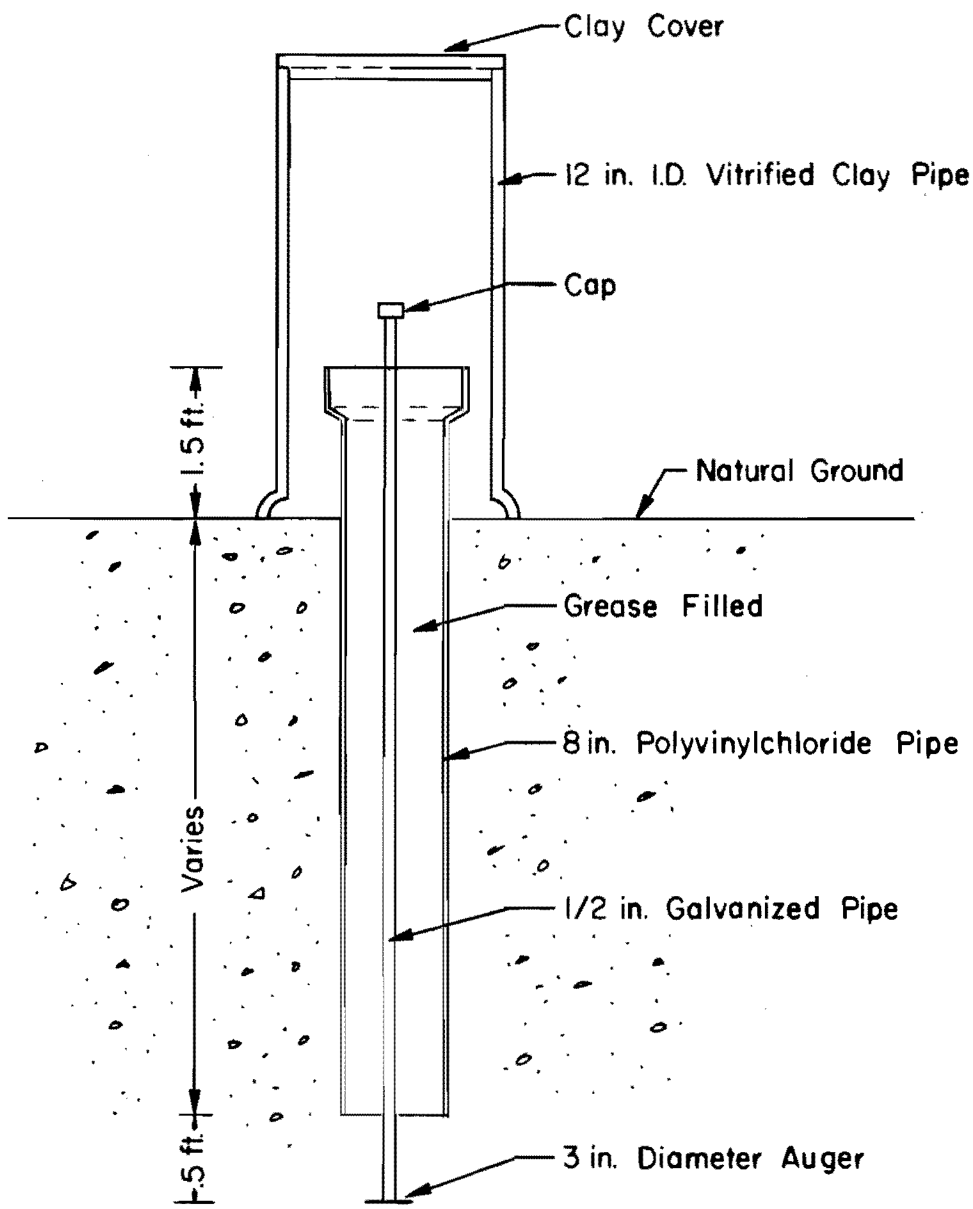


Fig 4. Section through typical elevation tell-tale.

The groups of four (Fig 5) were placed in the proposed median area of the first ponded sections. The original plan was to place each group about 500 feet apart with a control group about 200 feet beyond the east limits of the ponding area. After the field change extending the ponding area was approved, another group of elevation rods was set. The first group, drilled and set in late 1969, was placed in a box shape to centralize the location and minimize the possibility of soil differences which might affect rod movement. This was modified on the later holes when it was more important to avoid leaving a void in the limed area than to maintain proximity to avoid subsurface discontinuity.

A net of bench marks set by the District survey section was used in checking the elevation rods. These benches are 16-inch-diameter concrete drilled shafts set 22 feet in the ground (Fig 6). The net was checked periodically and a loop was run from the same net bench along the tops of the 1/2-inch water pipes of the elevation rods. Readings using a self-leveling instrument and a Philadelphia rod were recorded to .001 feet. Frequency of checking depended on work demands on the project field party and on weather conditions.

The group of tell-tales concentrated about station 250+00 was augmented on February 25, 1970, by two other devices of a different design. A tell-tale which consisted of a 5-inch-diameter single-flight augur welded to a 1/2-inch-diameter steel rod was screwed into the soil to a depth of 2 feet. It did not require boring a hole before installation and it did not have backfill of grease or soil. It is commercially available as a post for single-strand chain or cable fences and the top is formed into an eye. The tell-tale was quickly installed by using a length of pipe through this eye to manually screw the augur into the soil. The soil resistance limited the depth for hand installation to 2 feet. This device, called the eyebolt (Fig 7), was installed 2.5 feet east of the 4.5-foot tell-tale.

The other odd tell-tale (Fig 8), similar to those previously installed at a cut in New Boston, Texas, consisted of a 4-inch-diameter baseplate mounted on the bottom of a 1/2-inch-diameter conduit. A 3/4-inch-diameter conduit filled with grease prevented the soil above the baseplate from acting upon the tell-tale and was in turn fitted inside the short stub of 1-inch pipe welded to the baseplate to stop backfill from jamming between the tell-tale pipe and the sleeve as swelling took place. This tell-tale was installed by boring a hole, setting the baseplate in a thin layer of dry grout, and carefully

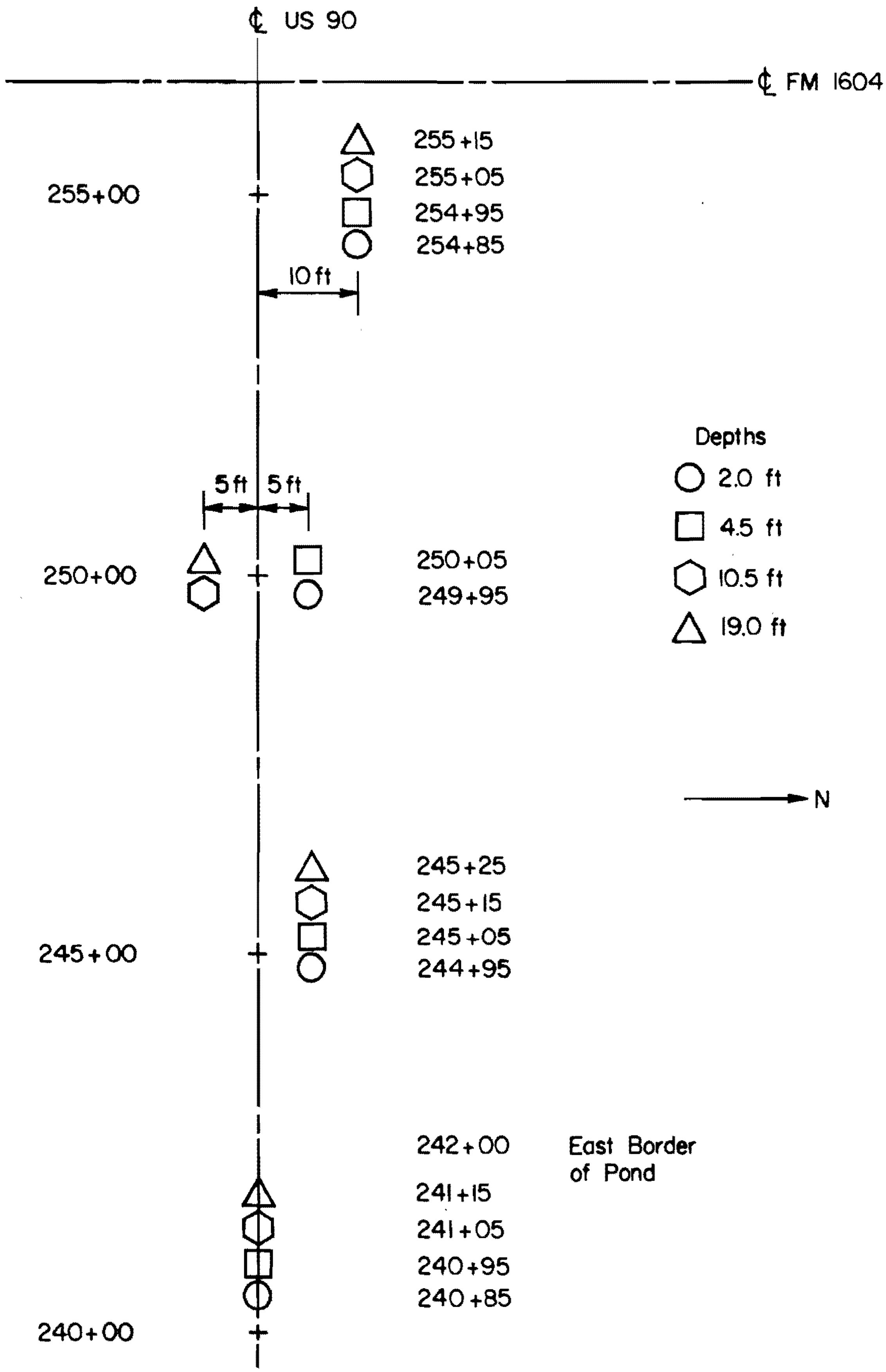


Fig 5. Location of elevation tell-tales.

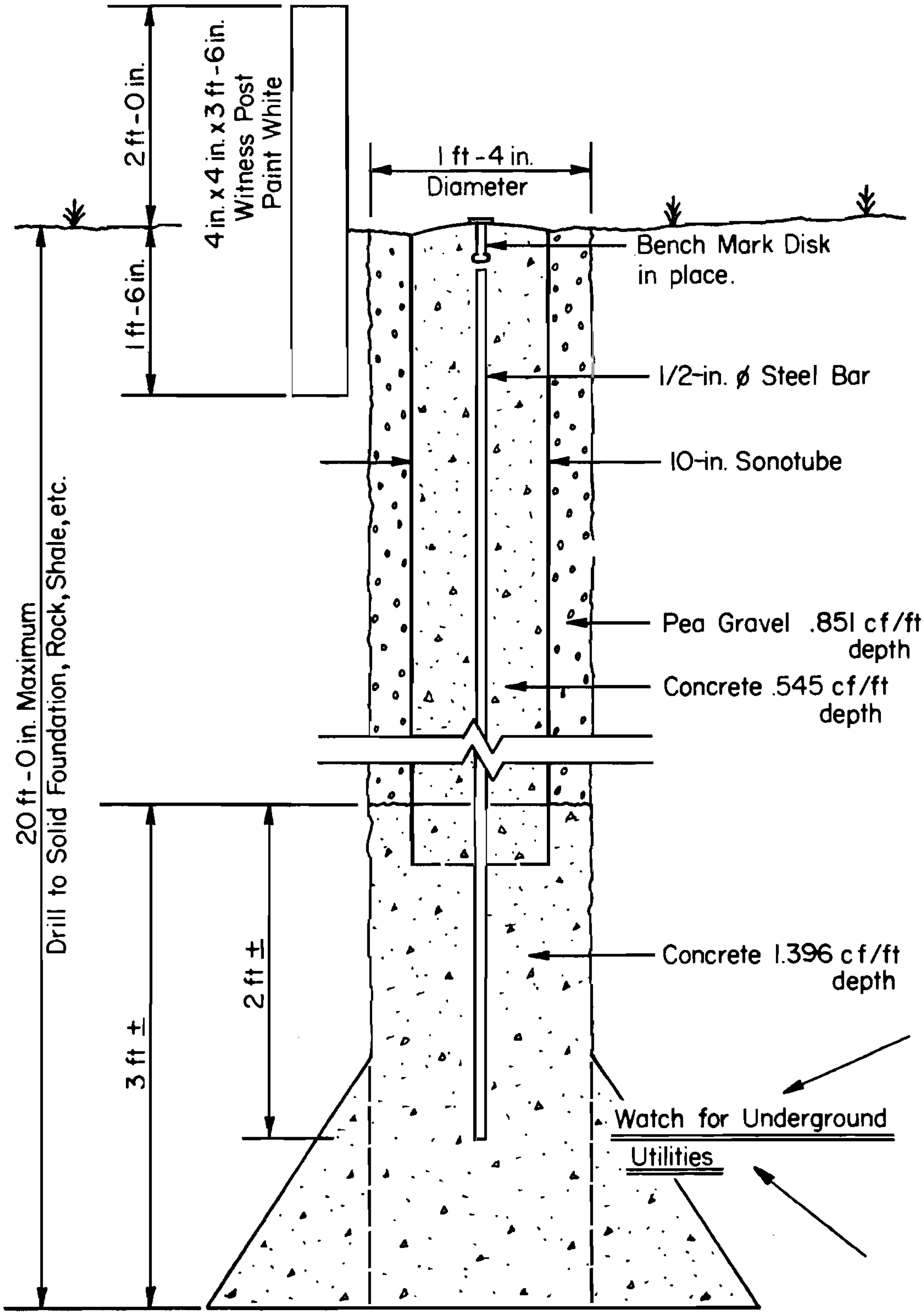


Fig 6. Detail of permanent construction bench marks.

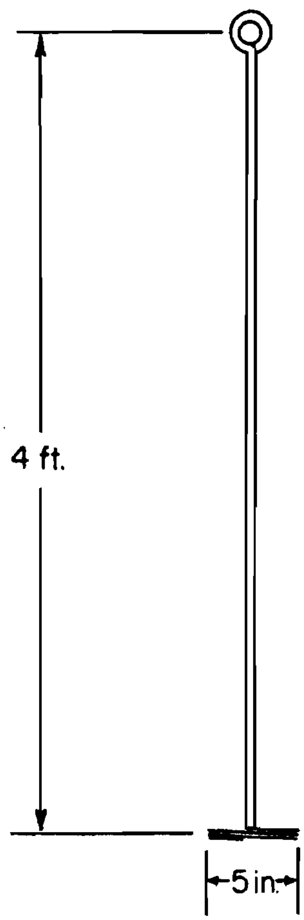


Fig 7. Eyebolt tell-tale set at station 250+2.5, 5 feet right of centerline, 2 feet deep.

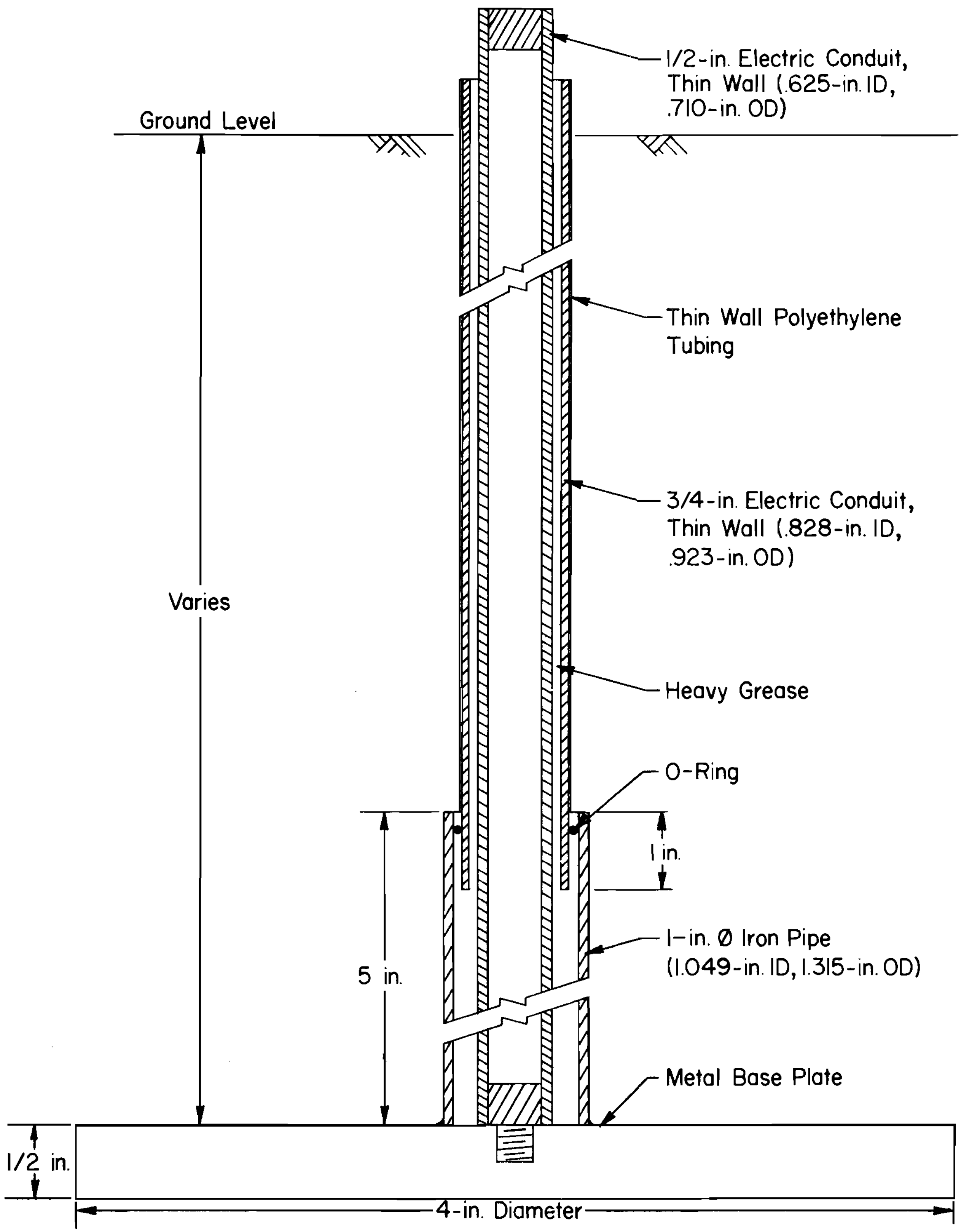


Fig 8. Tell-tale subsurface device.

backfilling around the sleeve. It was set at 3.3 feet below the soil surface, the limiting depth to which a hand augur could be advanced in the clay at the test site.

Moisture and Density Measurements

The moisture and density changes at a particular location within the ponded area were determined by nuclear depth gages purchased by the Texas Highway Department. In addition, water content determinations were taken of soil samples from boreholes and at one there was a field correlation.

The nuclear depth gage is a probe containing a nuclear source and detector tube connected by a cable to a counting device (Fig 9). The probe is lowered into an access tube set into the soil where the moisture content or density is to be measured. Separate probes are used to estimate the wet unit weight of the soil in pounds per cubic foot and the weight of water per cubic foot of soil surrounding the access tube. The counting device, called a scaler, provides an energizing voltage which transmits amplified pulses from the detector tube to digital display tubes in the scaler. The pulses from the detector tube are counted over a specified time interval, usually one minute.

When not in use the probes are stored in portable containers which act as shields to protect personnel from the nuclear source. While the probe is in the container shield a standard count can be made on the shielding, which has a known density or known hydrogen content. The count response from the soil divided by this standard count gives a count ratio for that particular point in the soil. This procedure helps to minimize instrument effects on the readings.

The count ratios for various depths can be replicated to determine the precision of the readings. The results of a set of repeated field readings for the density probe are shown in Table 1.

The density or moisture content can be estimated from calibration charts. The calculation of wet density from count ratios requires a calibration which is a straight-line relationship when the log of the count ratio is plotted against wet density. The calculation of water per cubic foot of volume is obtained from arithmetic plots of count ratio versus water density.

In this particular study the calibration curves used were obtained by inserting the probes into soil standards compacted in cut-down 55 gallon

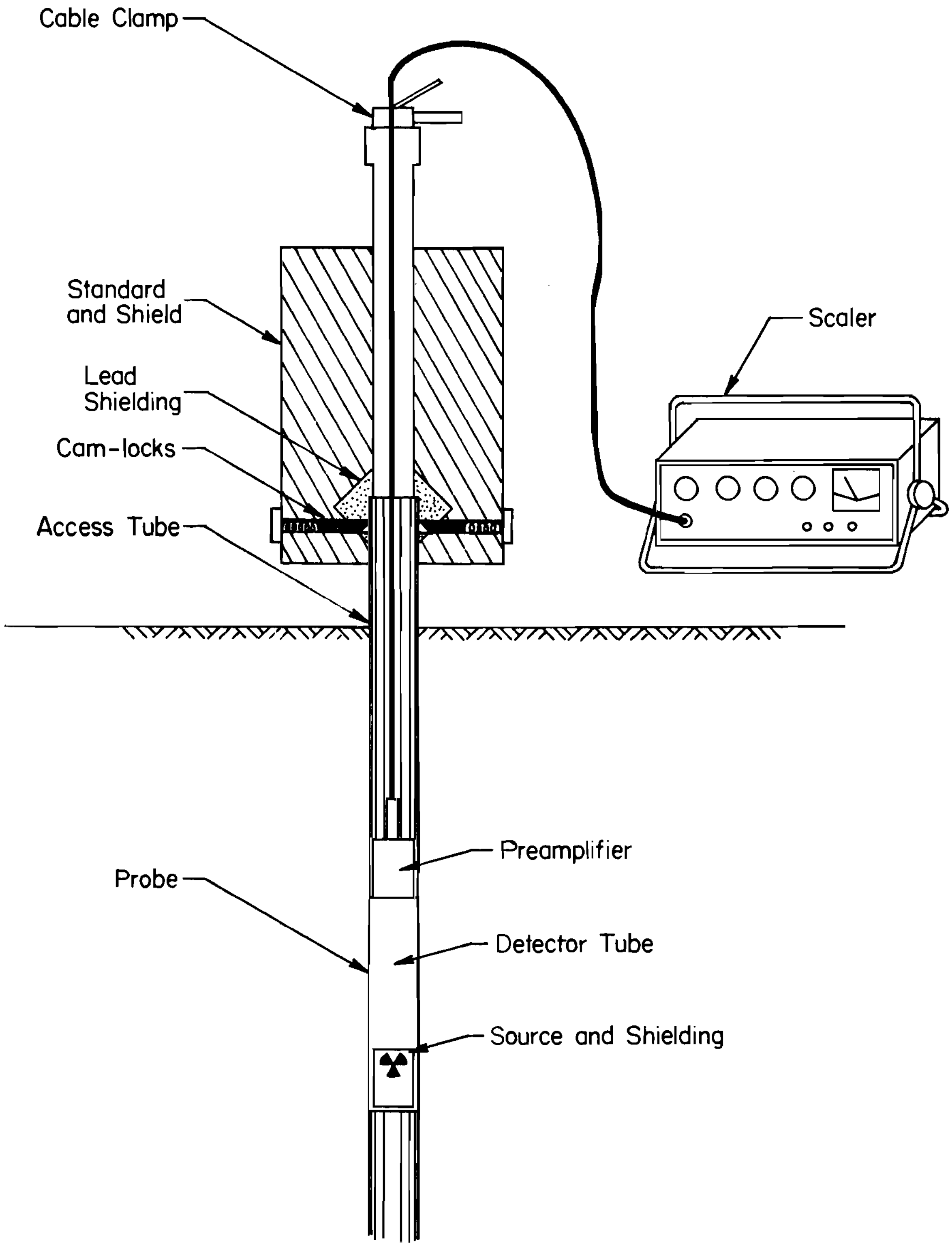


Fig 9. Nuclear depth probe.

TABLE 1. REPLICATIONS OF COUNT RATIOS FOR DENSITY

Depth, feet-inches	First Reading, 10:15 A.M.	Second Reading, 11:40 A.M.	Third Reading, 5:35 P.M.
5 - 0	1.789	1.799	1.780
10 - 0	1.809	1.815	1.805
14 - 11		1.820	
15 - 0	1.817	1.835	1.819
15 - 1		1.831	

drums. The preparation of the calibration curves and comments regarding their validity will be discussed in more detail in a future report. The density checks correlated with the manufacturer's calibration curve, but the experimental calibration curve gave water densities which were considerably less than those calculated from the manufacturer's curve. It is believed that the manufacturer's curves are not applicable to soils which contain considerable amounts of clay minerals, because the method cannot distinguish between neutron collisions with water molecules in the pore water, water of hydration, and the hydroxyl ions within the crystal lattice of the clay minerals. Thus each soil will have a unique calibration curve depending upon the percentage of each type of clay mineral in the soil. The experimental curve which was used in this project was based on laboratory standards of sand, silty clay, and a highly plastic Permian red clay from Oklahoma and a highly plastic clay from near New Boston, Texas. Because of the sparseness of data points from any one soil, a best-fit line was sketched through all of the available data points to describe a calibration curve. It is hoped that further production of standards will enable a more selective fitting to be made.

The water content of the soil can be calculated by the equation

$$w\% = \frac{D_w}{D_s - D_w} \times 100 \tag{4.1}$$

where

D_w = water density in pounds of water per cubic foot of soil aggregate, from moisture probe readings;

D_s = bulk or wet density of soil in pounds of water and solids per cubic foot of soil aggregate, from density probe readings.

The dry unit weight of the soil aggregate can be determined by

$$\gamma DRY = D_s - D_w \tag{4.2}$$

where

γDRY = dry unit weight of soil aggregate in pounds per cubic foot.

Knowing the water content, dry density, and specific gravity of the soil, one can calculate the void ratio:

$$\gamma DRY = \frac{G}{1 + e} \gamma_w \tag{4.3}$$

where

G = specific gravity of soil solids;

γ_w = unit weight of water, 62.4 pounds per cubic foot;

e = void ratio.

The degree of saturation then follows from

$$S = \frac{Gw}{e} \tag{4.4}$$

where

w = water content in percent,

S = degree of saturation in percent.

The nuclear probes measure the density or water volume of a roughly spherical zone.

The zone of measurement of the moisture probe has been theoretically worked out by van Bavel et al (Ref 18) as

$$R = 5.9 \left(\frac{100}{\text{Vol \% } H_2O}\right)^{1/3} \tag{4.5}$$

where

R = radius of sphere of influence in inches.

In the soils at this site with water contents of 38 percent by weight, the radius would be about 7.5 inches. For very moist soils this radius shrinks so that in pure water it would be only 6 inches.

The zone of influence of the density probe has an estimated radius of 5 inches (Ref 18).

Readings within 9 inches of the surface would be subject to error and are neglected in the analysis of the data.

The access tubes used at the test site were 2-inch O.D. thin-walled aluminum irrigation pipes which were cut into 10 or 20-foot lengths and embedded so that at least 4 inches of tube would remain above the water level in the pond. The tubes were sealed at the bottom by forcing solid cylindrical aluminum plugs grooved to accept two o-rings into one end of the tubes before installation. The o-rings between the plugs and the tube wall prevented water from entering through the bottom of the access tubes and rubber stoppers on the top of the tubes prevented accidental filling by rainfall or flood.

The access tubes were installed by boring a hole with a 2-inch-diameter continuous-flight auger that was designed to handle grain. The cutting head for soils was made to fit any of the 3-foot-long auger sections and a special adapter was necessary to fit to the kelly bar of the drilling rig. An essential feature of the drilling system was to have the drill stem turn in air with no more than 1/2-inch of wobble at the cutting head. The head was advanced using a slow rotating velocity and a very slight downward pressure from the drilling rig.

Auger sections were added one at a time as drilling proceeded, without retracting the embedded portion, until the prescribed depth was reached. The drill stem was then pulled straight up and disassembled section by section. This procedure left a hole which was slightly greater than the access tube. The access tube was forced into the hole by hand and over a period of days the clay soil tightened around the tubes so they could not be turned by hand.

The nuclear instrumentation at the ponding site was concentrated about the cluster of tell-tales at station 250+000 (Fig 10). Two 10-foot and one 20-foot-long access tubes were installed in December 1969, approximately 65 days before ponding commenced. The access tubes fit loosely in the holes, at the time of installation, and if water had been added then it could have run down the void space between the soil and access tube. However, the soil tightened sufficiently against the tubes so that they could not be moved by hand by the time ponding started. The two shorter tubes were used to check the moisture and density changes indicated by the readings in the longer tube.

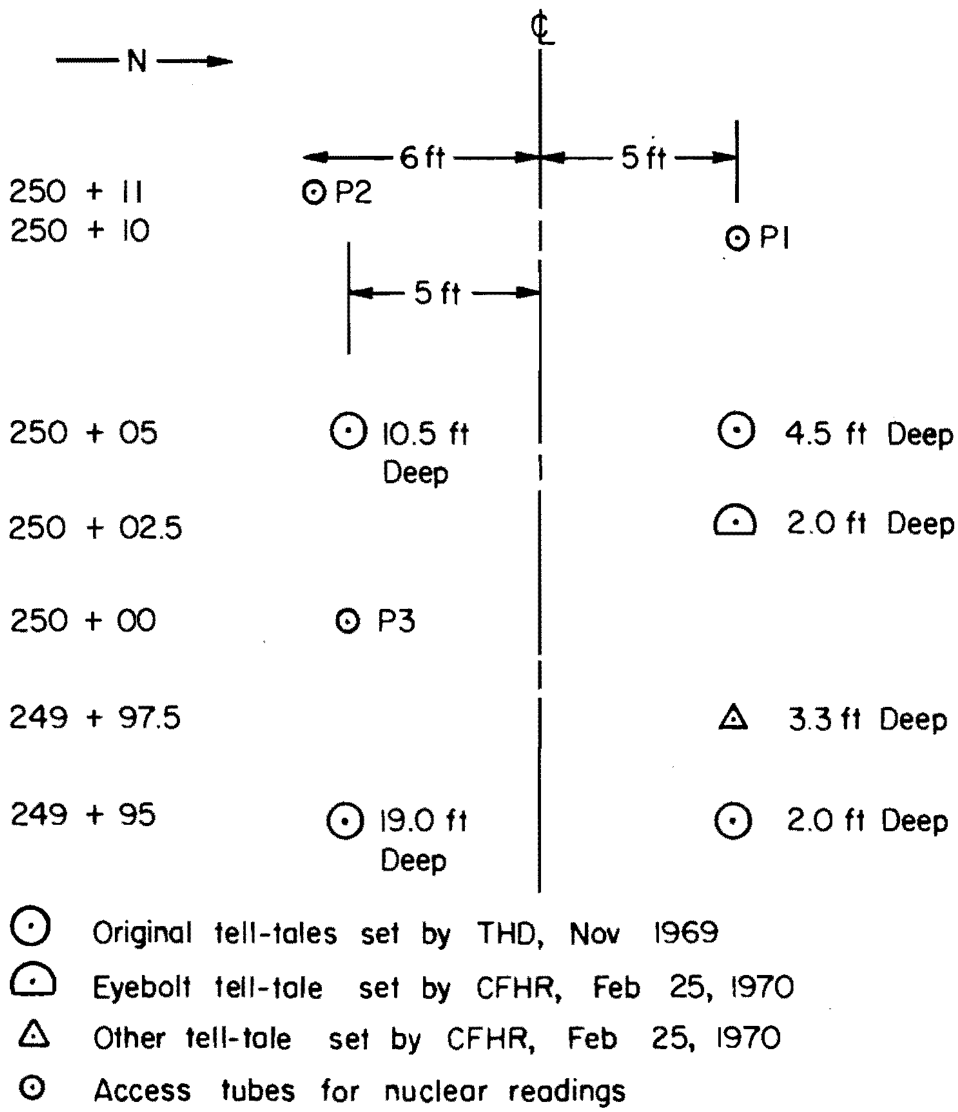


Fig 10. Instrumentation about station 250.

The test site was submerged February 20, 1970, when runoff from a heavy rain was trapped by the dikes, and remained under water until the site was drained on April 6, 1970. Thus the ponding period at station 250+00 was 45 days long.

CHAPTER 5. RESULTS

The results of the ponding study can be considered at two levels: (1) the effect of ponding in inducing swell before construction of finished roadway and (2) the ability of the various instrumentation to accurately measure heave at depth and changes in water content and dry density.

The measurements showed that the swell was primarily in the upper 4 feet of the subgrade and established the general pattern of wetting and soil expansion. However, heaves computed from changes in dry density were not confirmed by measurements of the elevation changes at depth. Possibly, the measuring techniques should be revised for better accuracy.

All elevation readings were taken by the project survey crew of the Texas Highway Department. The Center for Highway Research was responsible for readings of the nuclear density and moisture depth probes. The completed schedule of readings is shown in Table 2. Groups of four tell-tales were set at stations 245, 250, and 255 inside the dikes and at station 241 outside the ponded area. The eyebolt tell-tale was set at the 2-foot depth and the other tell-tale at the 3.3 foot depth at station 250. The two short access tubes and the long access tube were also set at station 250. Station 250 was ponded February 20, 1970, station 255 on March 5, and station 245 on March 9. All ponds were drained April 6, 1970.

For comparisons, December 15 is the starting date for elevation readings and computations from the nuclear readings. A five-day period between installation and the initial reading allowed the soil to close the air gap around the access tube.

Readings were obtained on the two short access tubes on February 17, 1970, but there was no reading on the long tube between December 22, 1969, and February 25, 1970. Since the soil around the long tube was flooded February 20, 1970, the nuclear readings five days later reflected the increase in moisture in the soil near the surface.

Although the ponding ended April 6, at station 250 there were no nuclear readings between April 1, 1970, and April 15, 1970. The level crew took

TABLE 2. COMPLETED SCHEDULE OF READINGS

	Elevation Readings						Nuclear Readings		
Date	240	245	250	Eye Bolt, 2 ft.	Tell-Tale, 3.3 ft.	255	P1	P2	P3
11-20-69			X						
11-21-69						X			
12-15-69		X	X			X			
12-16-69								X	
12-17-69							X	X	X
12-22-69									X
12-23-69		X	X			X			
2-7-70		X	X			X			
2-17-70							X	X	
2-18-70	X								
2-20-70	X	X	X			X			
2-25-70									X
2-27-70	X	X	X	X	X	X			
3-5-70							X	X	X
4-15-70							X	X	X
6-1-70	X	X	X	X	X	X			
6-5-70									X
7-2-70	X	X	X	X	X	X			
7-13-70									X
8-3-70	X	X	X			X			
9-1-70	X	X	X			X			
10-2-70	X	X	X			X			

readings on February 20, 1970, and April 4, 1970, and thus recorded the heave during ponding. The additional tell-tales, although set before ponding, were initially read 7 days after flooding.

A typical log of a level circuit is presented in Table 3. Four instrument settings were required to close the circuit. Readings were to .001 feet and the closure error was .017. The readings on all of the tell-tales at station 250 were taken from the same instrument setting. The closure error ranged from zero to 0.020.

The leveling procedures were sufficient to detect sources of heave and to record magnitudes of heave of tenths of feet, which had been predicted for the soil surface. However, they were not sufficiently precise to detect small vertical movements at the 10.5 and 19 foot depths.

Possible sources of leveling error were in the height of the instrument and in unbalanced foresight and backsight distances. Double rodding is reccommended as a check of the instrument elevations. Although the elevation of any tell-tale is somewhat in doubt, the accuracy of the readings of the relative movements of the tell-tales in a group is limited only by the ability of the instrument man to read to the nearest .001 on the level rod.

The changes in elevation of each tell-tale over the period December 15, 1969, to October 2, 1970, are plotted in Figs 11, 12, and 13. Some of the irregular vertical movements can be attributed to errors in leveling. Since the purported movements at the 19-foot level were in most cases within the error of closure, a set of corrected elevations was calculated assuming no movement at this level. The elevations were taken as the average of 12 elevations, determined from the leveling notes. The difference between the average and any measured value at the 19-foot depth was assumed to be a common error for the measured elevation data of the 2.0, 3.3, 4.5, and 10.5-foot tell-tales of the same group. The corrected elevations based on no movement at the 19-foot level are also shown in Figs 11, 12, and 13.

The difference between the uncorrected and corrected readings is the probable reading error at a particular station on a particular date. All of these errors are small.

The soil movements followed a slightly different pattern at each station. At station 245 the readings showed an expected pattern with larger movements at the surface becoming less at depth. The indication that swelling began February 27 (Fig 11) is misleading since no reading was taken between then and

TABLE 3. TYPICAL LOG OF LEVEL READINGS

February 27, 1970
Cloudy and Rainy

Location	Backsight Rod Reading	Height of Instrument	Foresight Rod Reading	Elevation	Elevation
BML-36	2.940	820.697			817.757
1.5' 5'L 249+95			5.840	803.067	
18.5' 5'R 249+95			6.190	802.717	
10' 5'L 250+05			5.780	803.127	
4' 5'R of 250+05			6.220	802.687	
Eyebolt 5'R of 250+02.5 2' below ground level			7.135	801.772	
Tell-Tale 5'R of 249+97.5 4' below ground level			6.170	802.737	
2" 5'L of 250+00 P3			6.830	802.077	
2" 5'R of 250+10 P1			7.195	801.712	
2" 6'L of 250+11 P2			6.900	802.007	
	0.550	808.947	0.510	808.397	
	4.850	813.057	0.740	808.207	
	8.990	821.207	0.840	812.212	
BM-36			3.430	817.777	817.757

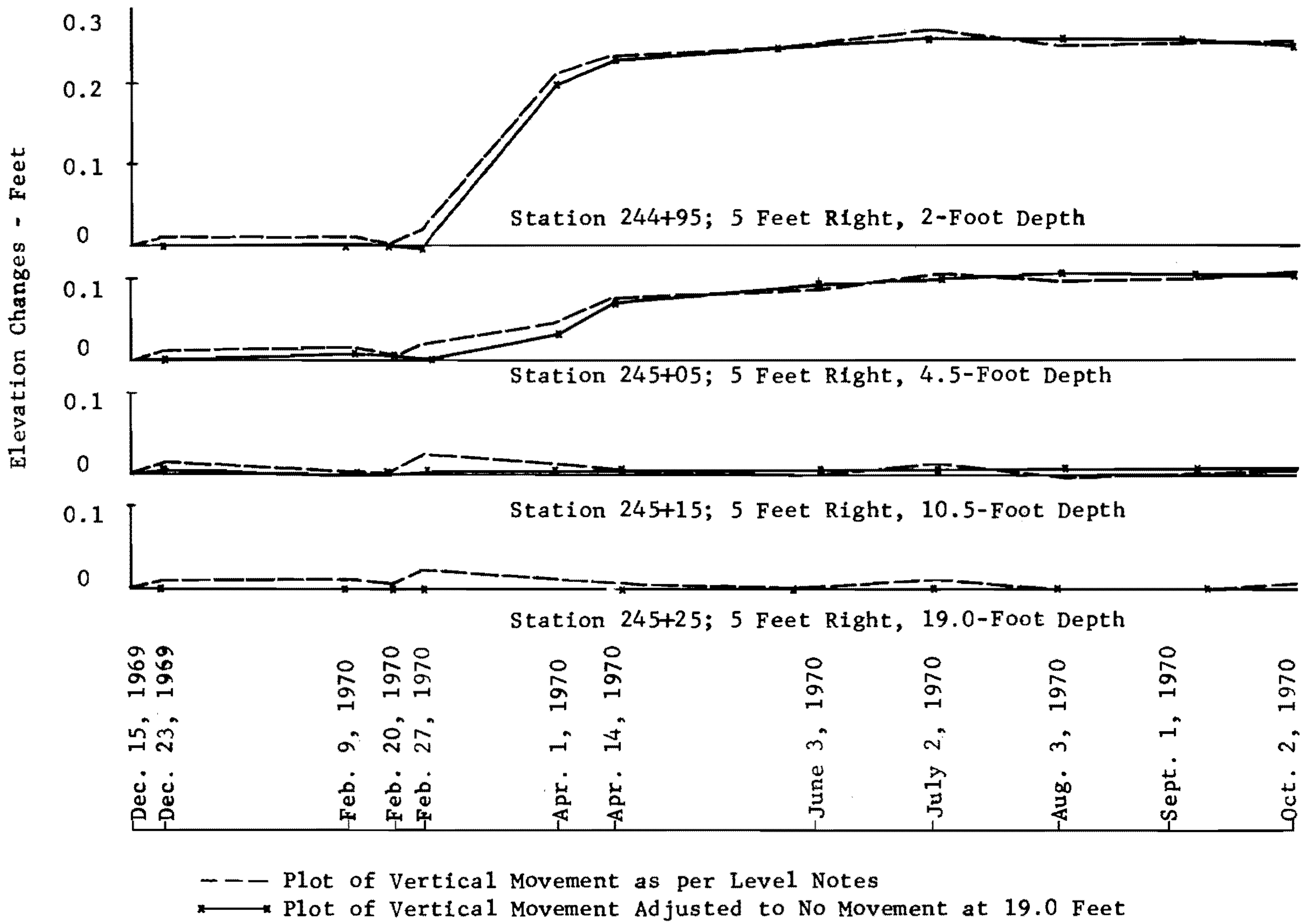


Fig 11. Vertical movement at station 245.

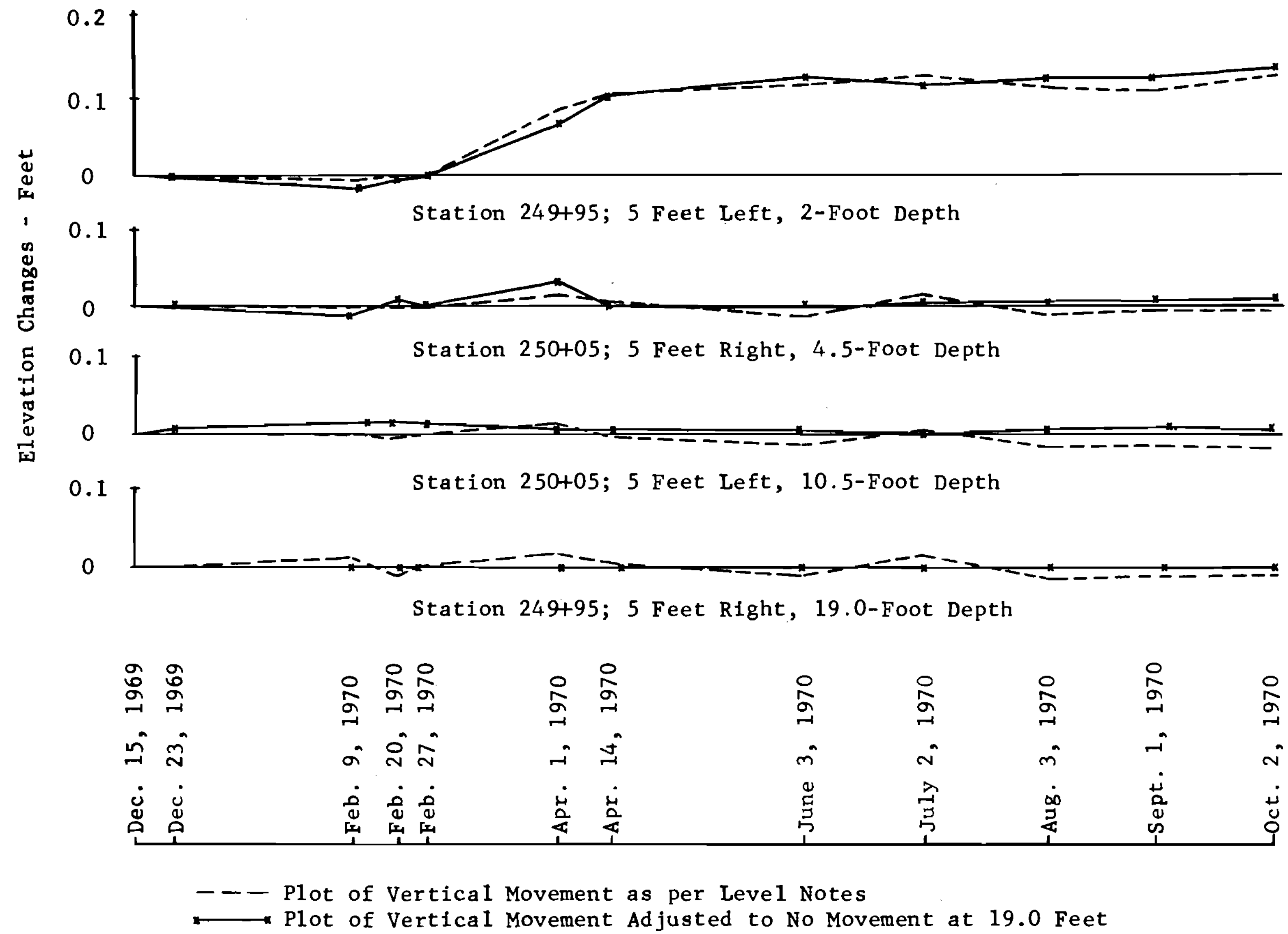
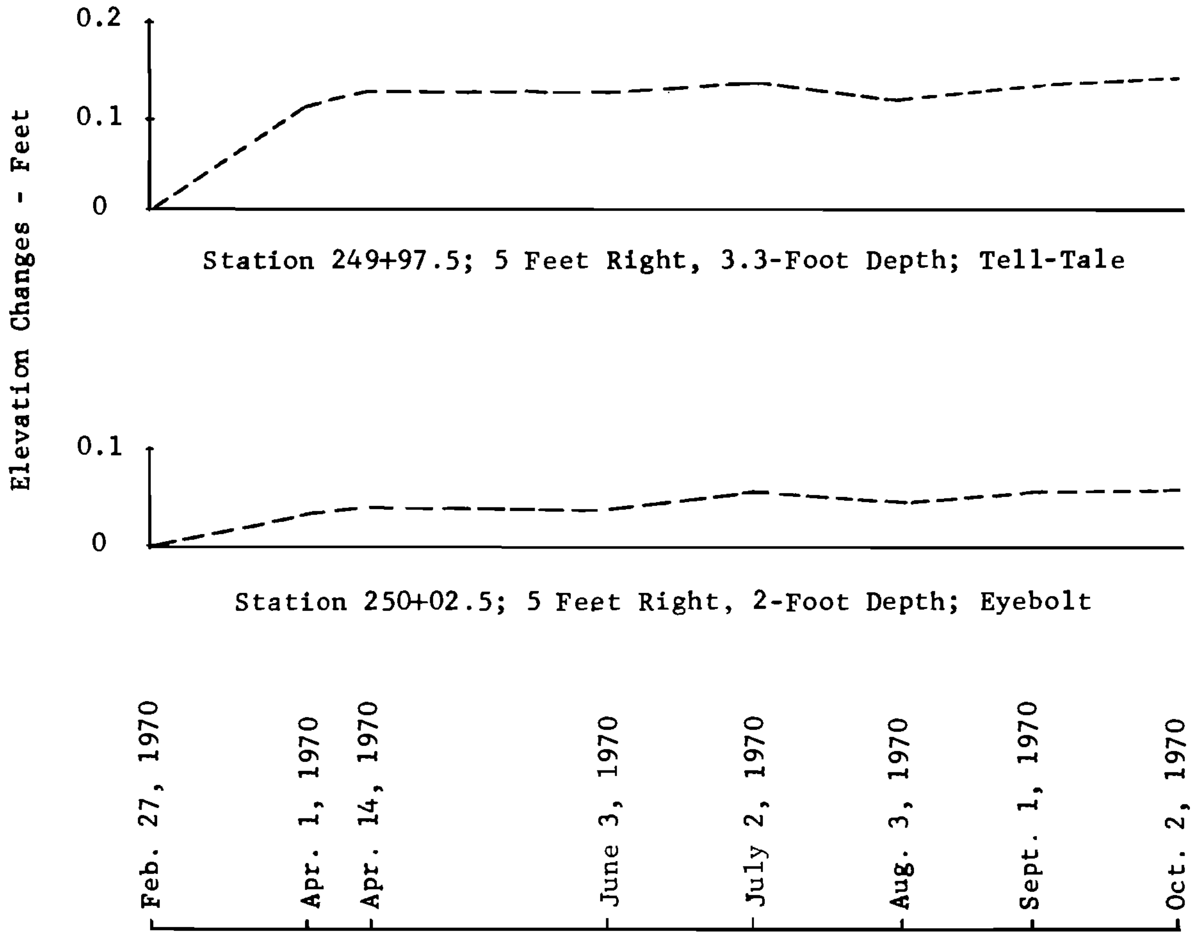


Fig 12. Vertical movement at station 250. (Continued)



--- Plot of Vertical Movement as per Level Notes

Fig 12. (Continued).

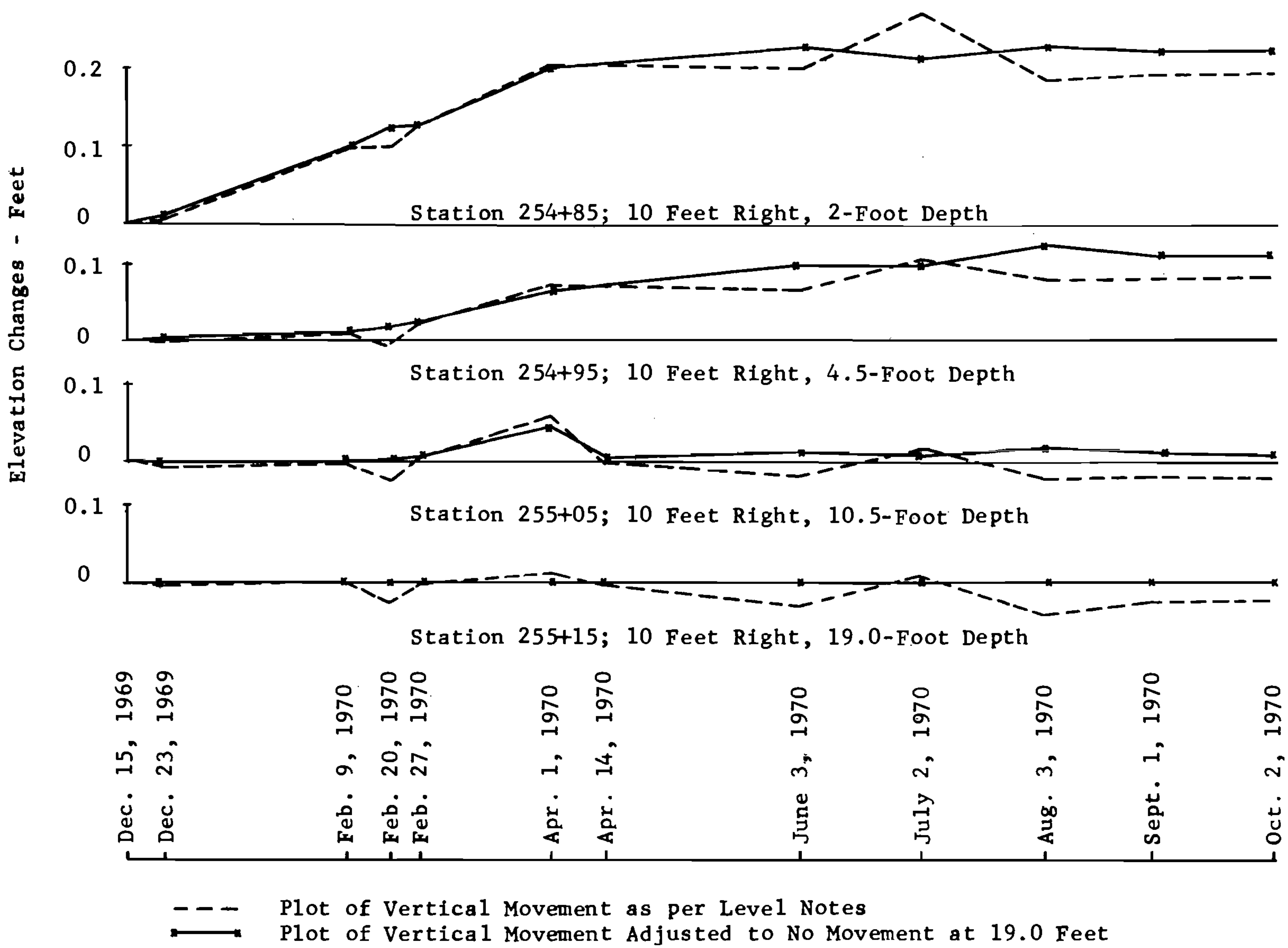


Fig 13. Vertical movement at station 255.

April 1, 1970. Most of the movement was probably due to the ponding, which began on March 9. There was little change after the ponds were drained but swelling continued at a reduced rate at the 2 and 4.5-foot levels. There was no significant movement at the lower levels.

At station 250 the tell-tales generally behaved in a similar manner except that movements are of a smaller magnitude. There were no significant movements at the 4.5 or 10.5-foot level. The two tell-tales at the 2-foot depth showed similar heaving patterns but the uncased auger heaved slightly more. The difference between the readings of the two tell-tales is attributed to a difference in soil character rather than a difference in the tell-tale installation procedure.

The readings of the tell-tales at station 255 differed in pattern from those at the other stations. At the 2-foot depth, heave of 0.12 inch was recorded before ponding began. Water from surface runoff probably caused localized wetting of the soil beneath the auger plate. A definite swell was read at the shallower depths after ponding and has held in the succeeding months. The slight swelling indicated at the 10.5-foot depth is inconsistent with the subsequent behavior of that tell-tale. Each tell-tale rod had a pipe cap which was to be removed before each reading. Placing the rod on top of the cap on April 1, 1970, would have indicated a quasiheave of 0.035 feet. Future tell-tales should either have fixed caps or none at all.

The moisture and density readings were transformed into water content and dry density. Table 4 is a typical data sheet for readings taken with the moisture depth probe in the longer access tube. Data sheets for the density probe are similar. Figure 14 is the calibration curve for the moisture and Fig 15 is the calibration curve for the density. These curves are the most likely source of error in the nuclear method of calculating water content and density because the calibrations are dependent on the soil type.

As previously noted, the calibration curve is a best fit line through sand, silty clay, highly plastic clay, and water, none of which were in the soil at the ponding site. It is assumed that if only the highly plastic clay from the site were used in the calibrations the points would fall either on the line or parallel to it. Although the absolute water and soil densities calculated from the curve may be in error, the changes calculated over a period of time are accurate.

TABLE 4. TYPICAL NUCLEAR DATA SHEET

Nuclear Moisture Determination

Site Location: P3 — Operator: G. Watt

Date: April 1, 1970 — Operating Voltage: 1250 HV

Standard Count:	Beginning 3:10	End 4:45
	14910	15119
	15076	15136
	15034	14976
	15027	15003
	14979	15012
Total	75026	
Average		15027.2

Cable Reading	Depth, feet	1st	2nd	3rd	Total	Average	CR
4	1.0	15111	15281	15335		15242	1.0143
10	1.0	15112	15142	15451	45705	15235	1.0138
4	2.0	15060	15169	15096	45325	15108	1.0054
4	3.0	14310	14053	13802	42165	14055	.9353
4	4.0	13653	13517	13809	40979	13690	.9110
4	5.0	13308	13506	13574	40388	13463	.8959
4	6.0	12918	12647	12723	38288	12763	.8493
4	7.0	13586	13406	13429	40421	13474	.8966
4	8.0	13322	13543	13631	40496	13499	.8983
4	9.0	13546	13431	13394	40371	13457	.8955
4	10.0	13828	13851	13781	41460	13820	.9197
4	11.0	13664	13566	13837	41067	13689	.9109
4	12.0	13727	13866	13635	41428	13809	.9189
4	13.0	13508	13327	13357	40192	13397	.8915
4	14.0	13192	13416	13249	39857	13286	.8841
4	15.0	13221	13265	13296	39782	13261	.8825
4	16.0	14204	14168	14001	42373	14124	.9399
4	17.0	13798	13842	13734	41374	13791	.9177
	18.5						
	19.5						

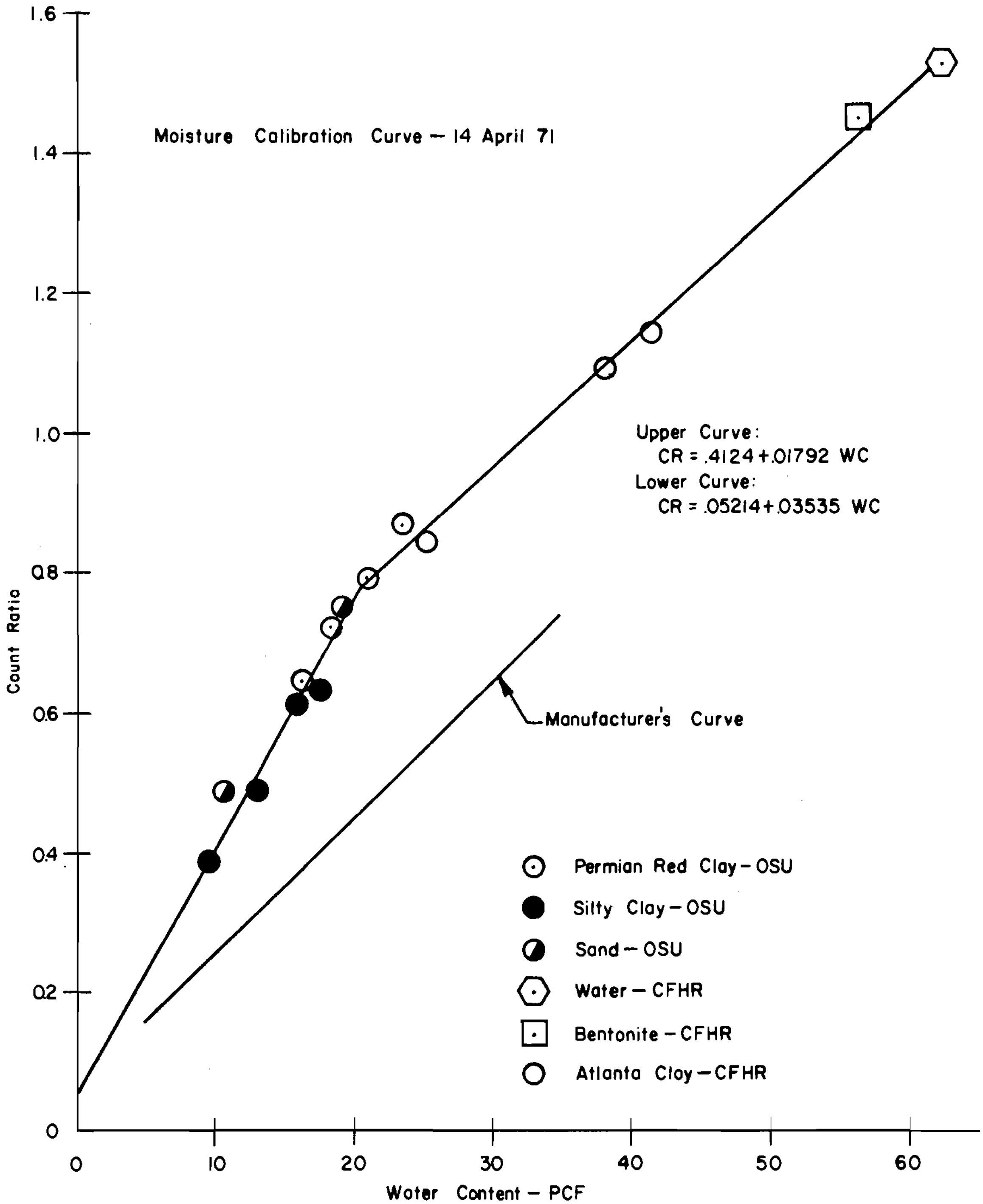


Fig 14. Calibration curve for moisture.

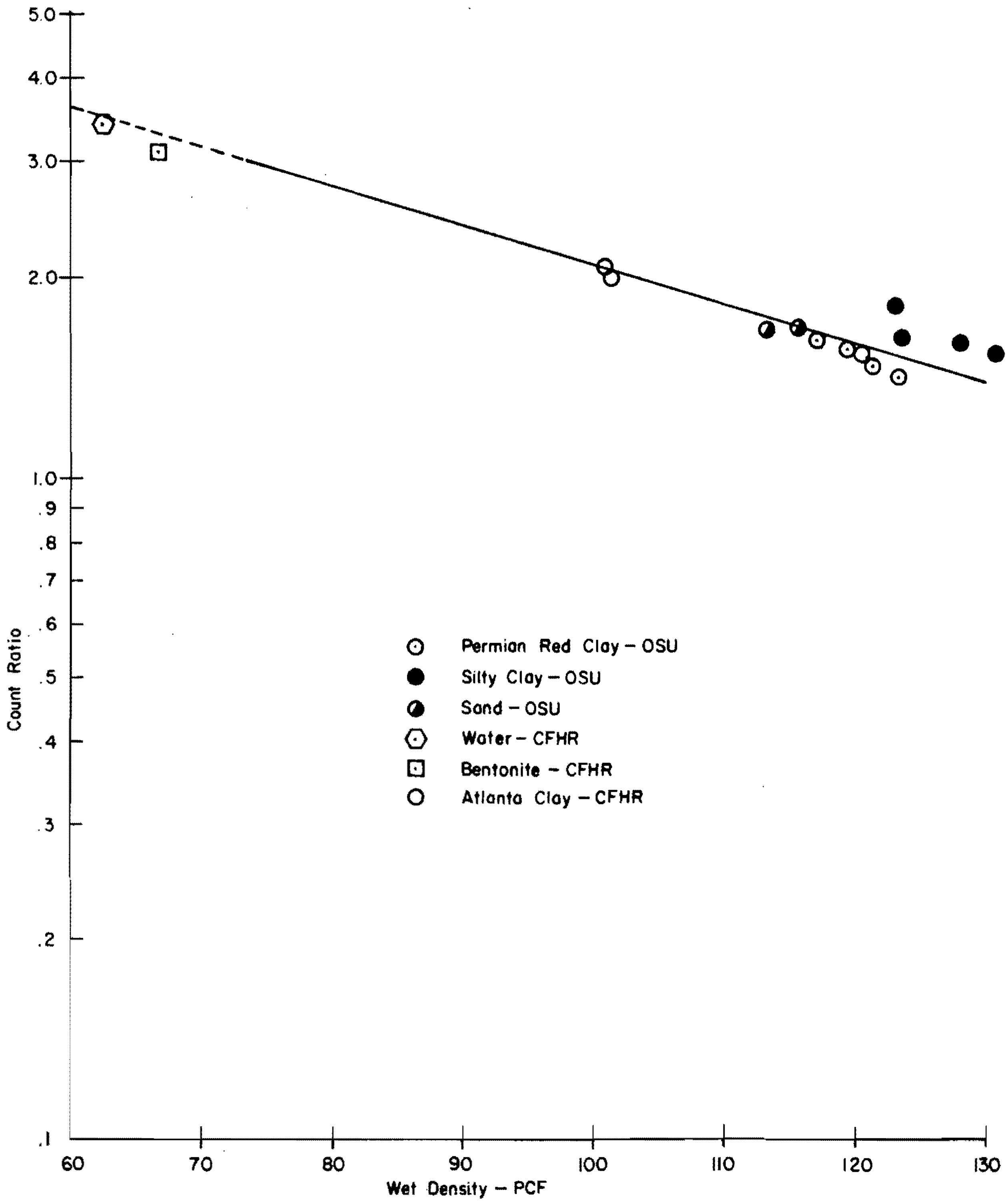


Fig 15. Calibration curve for density.

The calculations of the water content and dry density were described in Chapter 4. A field check of water contents calculated from nuclear readings was made by sampling at an access tube set in the soil at station 241+95 at the east border of the ponded area. The results are shown in Table 5. Atterberg limits at depths of 8 feet 4 inches, 9 feet 4 inches, and 13 feet 4 inches show common soil properties, with a liquid limit of 83 and a plasticity index of 50.

Figures 16 to 21 show changes in water content and dry density before ponding, during ponding, and after draining the ponds. Changes were shown because they were more accurate than the absolute values and because the changes in density can be directly related to heave.

The water content increased in the upper 3 feet in the two months before ponding and apparently there was a decrease in the water content below 7 feet in the same period. The soil columns surrounding the shorter access tubes indicated the changes in the upper 6 feet, and the soil around the longer tube showed conditions below 7 feet.

The wetting of the upper 2 feet of soil during the first five days of ponding is quite evident from the readings in the longer access tube (Fig 19). Just as noticeable, however, is the different behavior of the soils around the shorter tubes although they were only 10 feet apart. The scatter in water content changes ranges from 1 percent to 4 percent while the changes in dry densities are different by up to 4 pounds per cubic foot.

From 7 to 16 feet of depth the slight increase in dry density from 0.5 to 2 pounds per cubic foot corresponds to decreases in water content of less than 2 percent and a slight decrease in the degree of saturation of about 4 percent. Such variations in moisture and density are apparently caused by differences in intraseasonal rates of evaporation and precipitation.

Ponding for 45 days had the effect of wetting the upper 3 feet of soil only during the period of inundation. Decrease in the dry density of 6 pounds per cubic foot and increase in the water content of up to 9 percent are shown.

The region below 3 feet showed some changes but these were of the same order as the changes prior to ponding and could not be attributed to the flooding.

Figures 18 and 21 show changes which took place subsequent to draining the ponds. The upper 6 feet of soil was still wet two months after draining. The large change at the 2-foot depth of the longer access tube is probably in

TABLE 5. COMPARISON OF NUCLEAR AND GRAVIMETRIC MEASUREMENTS OF WATER CONTENT

	From Nuclear Tests			Gravimetric Test
Depth, feet-inches	Dry Density, lbs/ft^3	Degree of Saturation, Percent	Water Content, Percent	Water Content, Percent
8 - 4	88.9	102	33.9	31.5
9 - 4	83.2	101	38.2	33.4
10 - 4	81.5	92	36.2	30.3
11 - 4	83.1	88	33.6	27.1
12 - 4	83.8	91	33.9	29.8
13 - 4	81.5	90	35.6	28.5
14 - 4	83.0	89	33.6	30.2
15 - 4	82.4	89	34.5	30.0

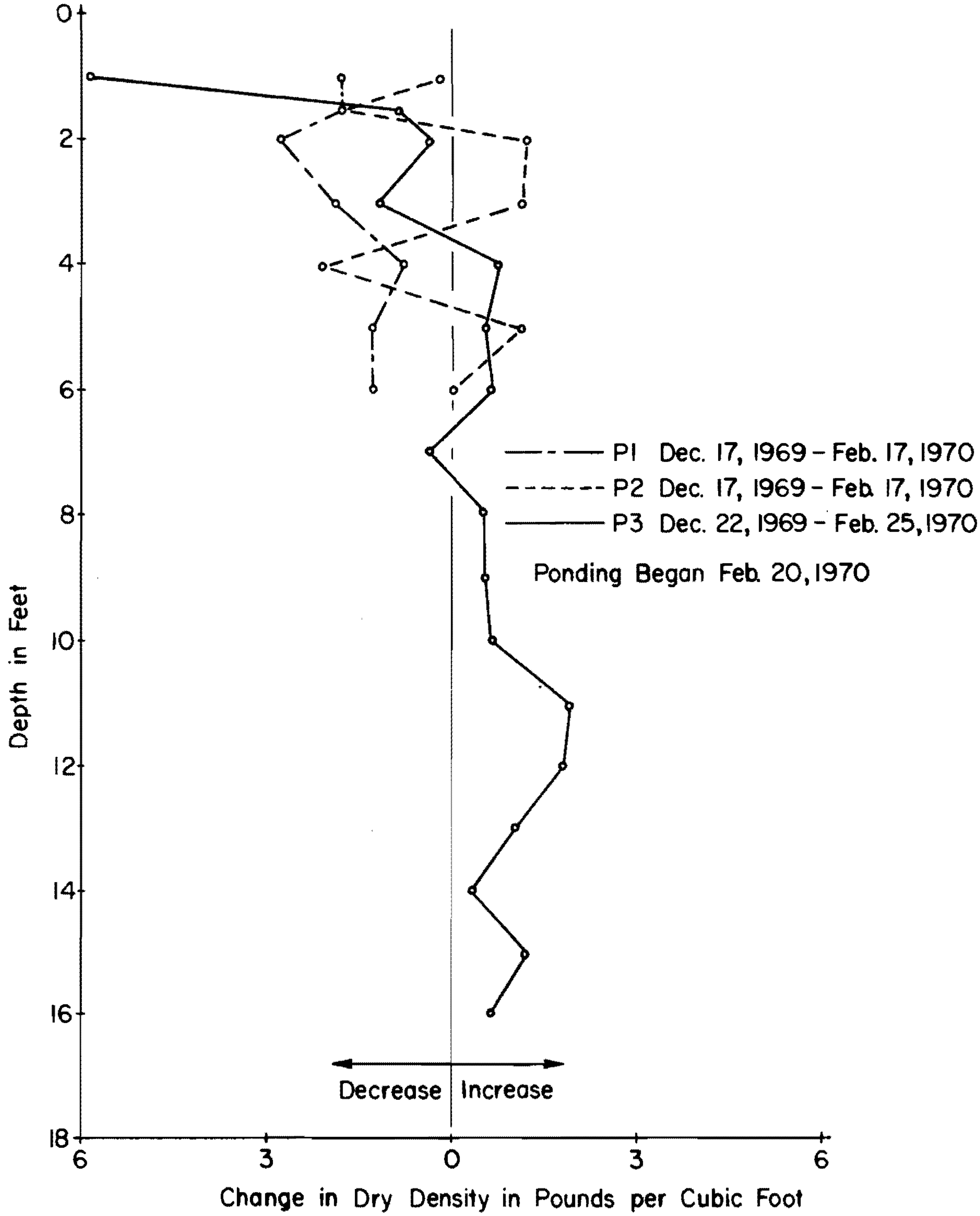


Fig 16. Changes in dry density prior to ponding.

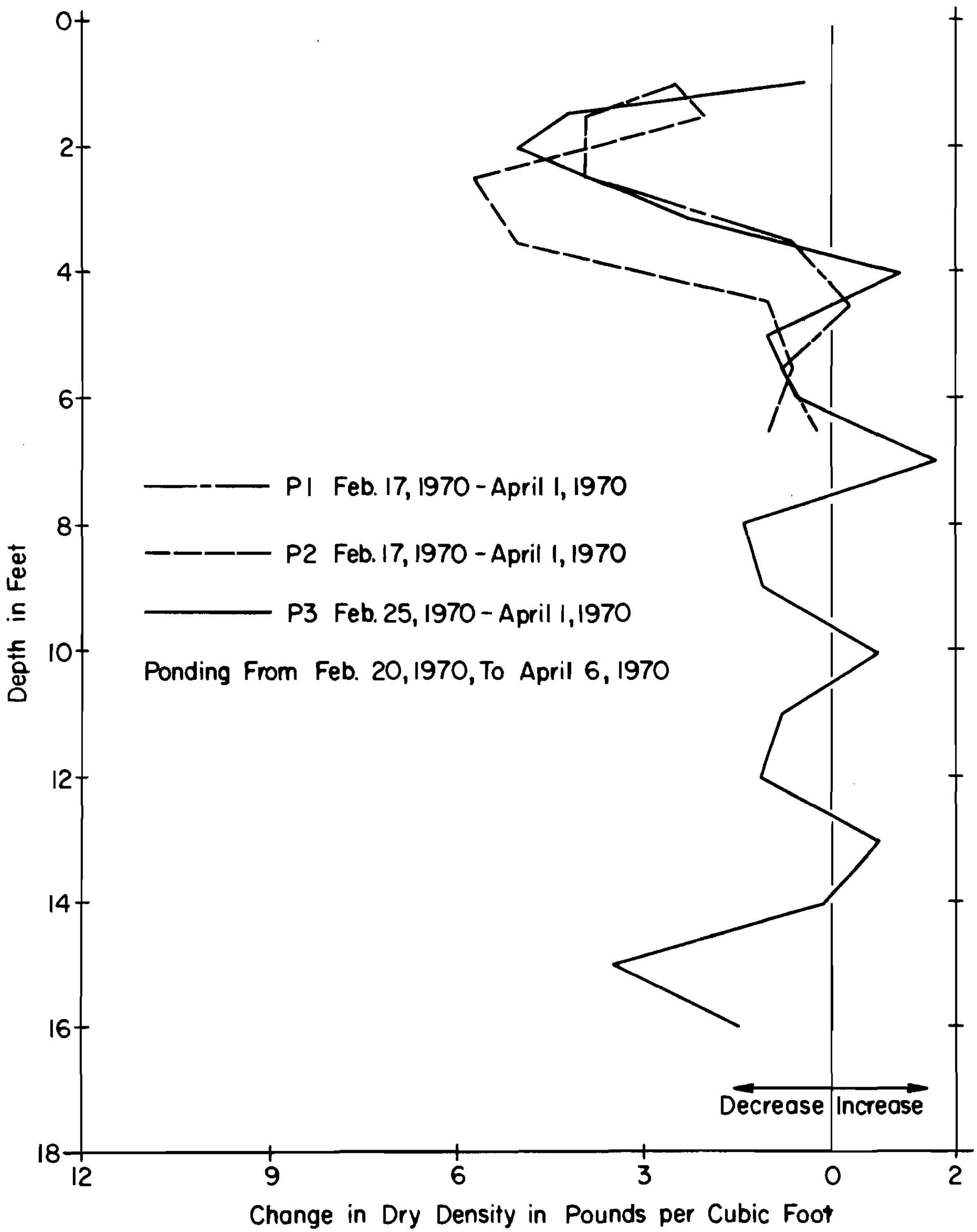


Fig 17. Changes in dry density during ponded period.

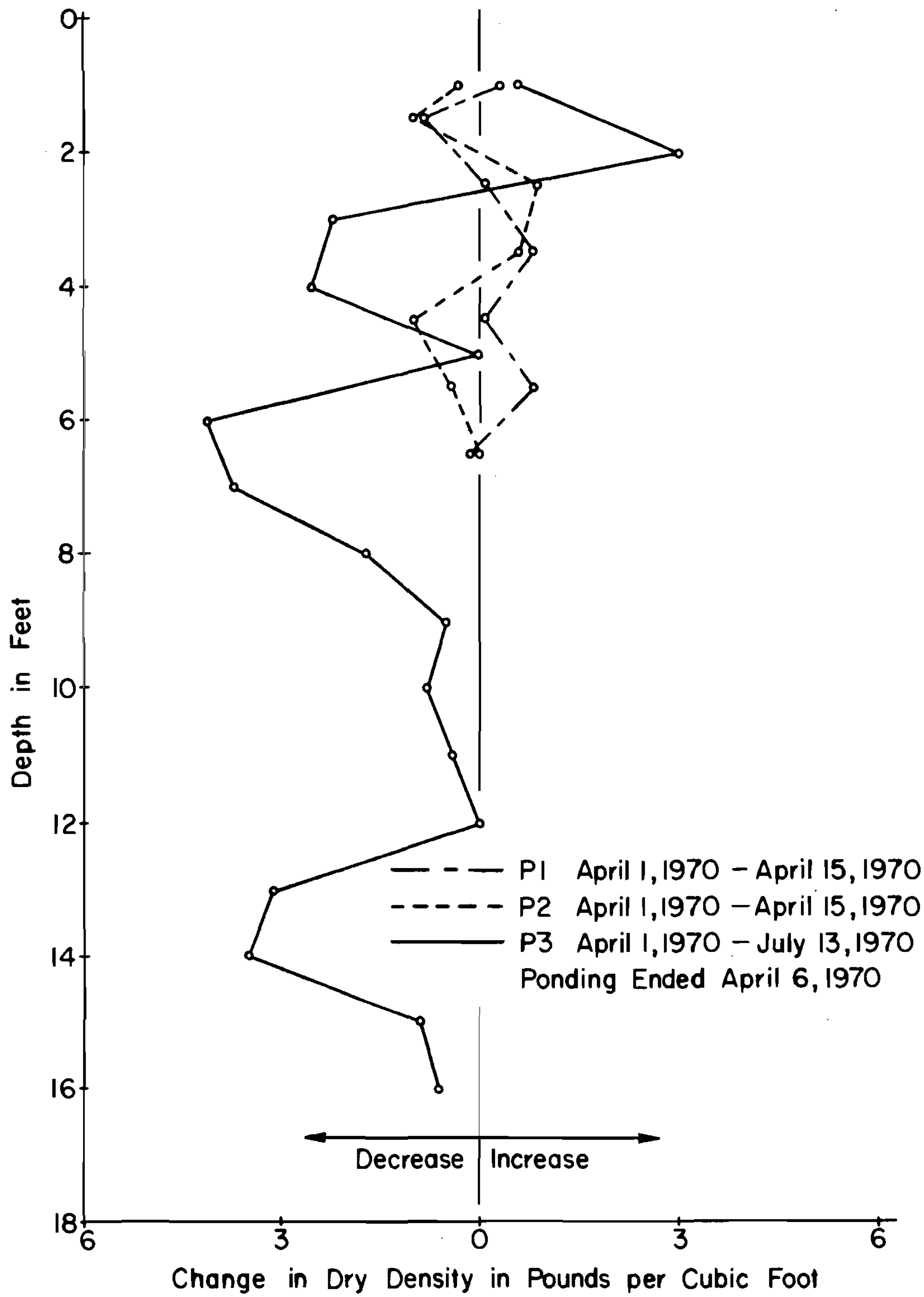


Fig 18. Changes in dry density after draining pond.

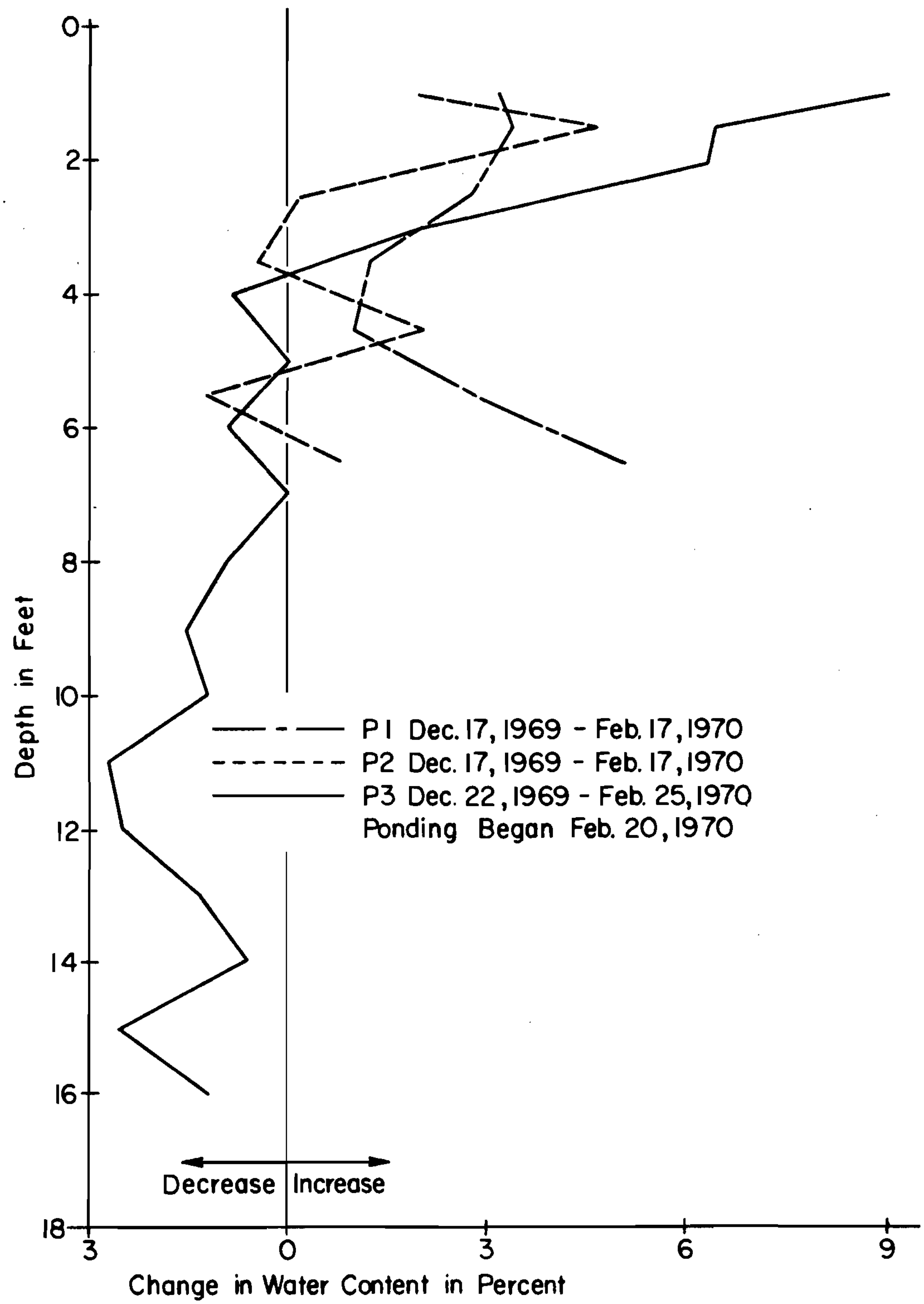


Fig 19. Changes in water content prior to ponding.

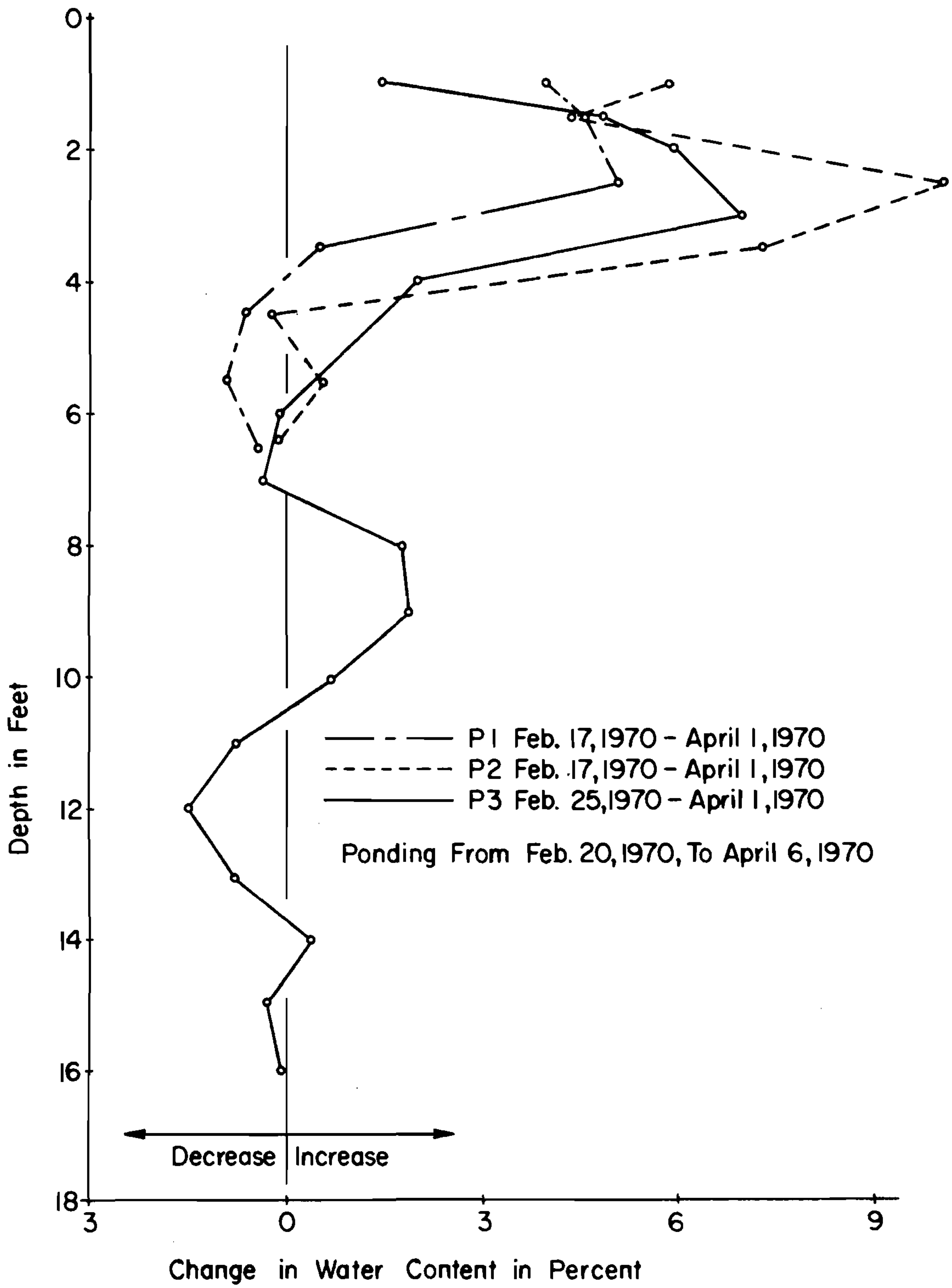


Fig 20. Changes in water content during ponding.

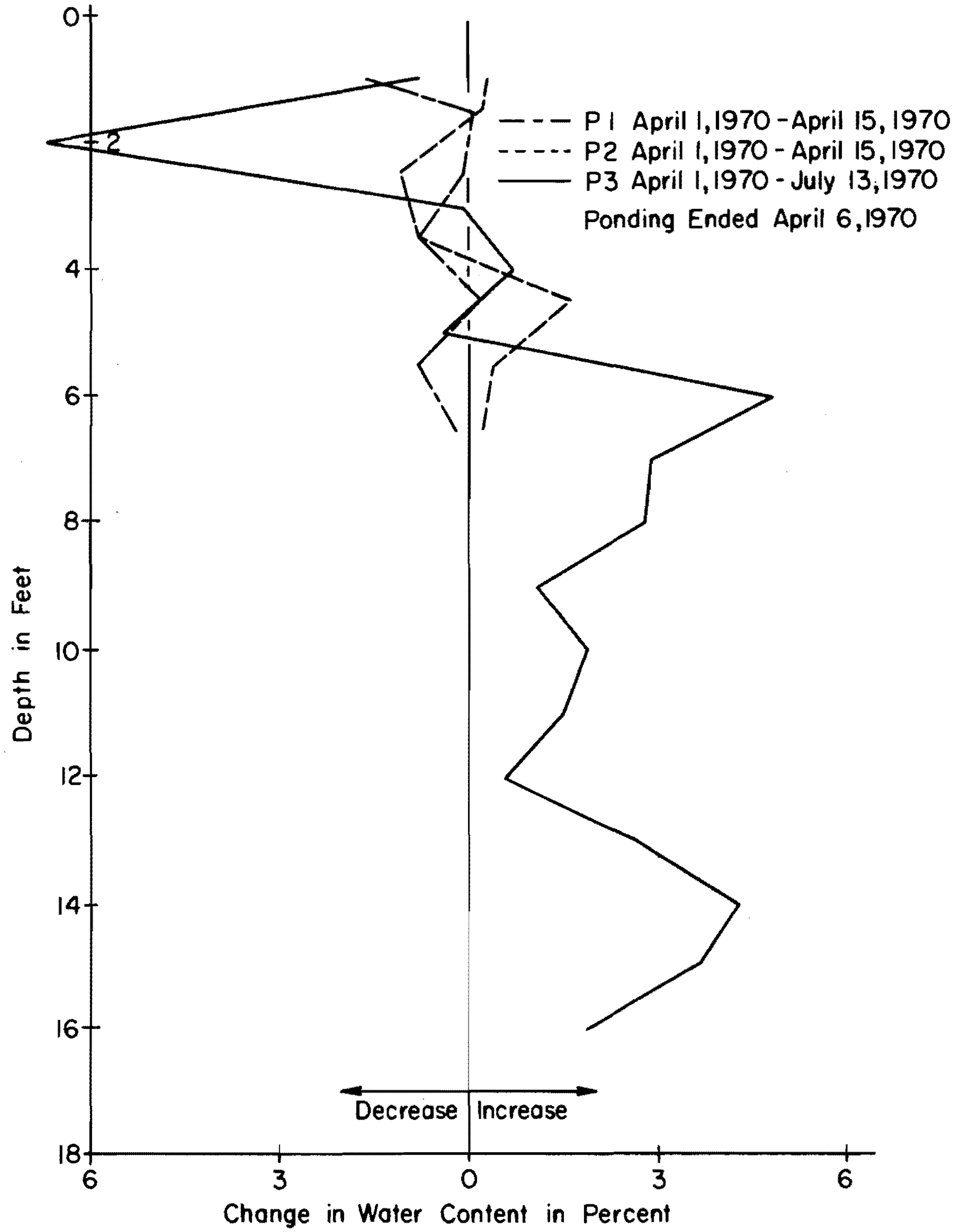


Fig 21. Changes in water content after draining pond.

error as no change in water content had taken place at that depth by June 1, 1970, and no significant drying appeared at the 1-foot depth.

The readings of the shorter tubes from April 1 to April 15 show that the final day of draining, April 6, was not a significant factor in the behavior of the surface soils.

There is an apparent wetting up below 4 feet at the 6 and 14-foot depths. If changes in water content during and after ponding are added together the increase in water content from 3 percent to 5 percent is indicated for all depths between 6 and 10 feet and between 14 and 15 feet. This is the same sequence of wetting that occurred in the ponding experiments at Waco except that the wetting at depth was evident after 24 days in Waco.

Assuming that 100 percent of the volume increase was in the vertical direction the changes in dry density can be transformed into vertical deformations. The computed vertical deformations were added between the 2.0-foot, 4.5-foot, 10.5-foot, and 19.0-foot levels and compared with the tell-tale measurements. The results are shown in Table 6, and the correlation is only fair.

The values are of the same order between 2.0 feet and 4.5 feet but at the 10.5 to 19.0-foot levels the vertical expansion calculated from the nuclear tests shows it to be about 0.3 foot whereas the values measured in the field are about 0.01 foot.

The correlation was based on the following six assumptions, one or all of which were wrong:

(1) All tell-tales and access tubes were placed in identical soil profiles.

(2) All volume change was in the vertical direction.

(3) The soil between the 16.0 and 19.0-foot depths had the same characteristics as the soil at 16.0 feet.

(4) The tell-tales correctly measured vertical movements of the soil.

(5) Changes in dry density were calculated accurately from readings of nuclear density and moisture depth probes.

(6) Specific gravity of the soil solids is 2.70.

The lateral variability of the soil is evident when the heaves of the 2-foot tell-tales are compared or when the wetting curves of the short tubes are compared. When two values were available for the same depth the average was used for the calculations for Table 6.

TABLE 6. DEFORMATIONS IN SOIL AT STATION 250

Depth in Feet	Deformation: Before Ponding in Feet		Deformation: During Ponding in Feet		Deformation: After Ponding in Feet	
	Calculated	Measured	Calculated	Measured	Calculated	Measured
2.0 to 3.3	.008	0.000	.062	.055	- .006	+ .012
3.3 to 4.5	.014	0.000	.026	.036	- .003	+ .025
4.5 to 10.5	- .030	+ .005	.017	- .004	.140	+ .011
10.5 to 19.0	- .102	+ .005	.076	- .007	.133	- .008

Soil variability is the chief reason for poor correlation but at depth the overburden pressure should be sufficient to encourage some lateral expansion. The variation in dry density and water content at a point in the soil before ponding, as shown in Figs 16 and 19, would encourage localized horizontal shrinkage or swell. This process would continue at the lower depths even during ponding with the exception that the suction at the surface would be controlled by the ponded water.

From the time ponding began, the total calculated vertical expansion of the soil below 16 feet was .039 foot. If shrinkage instead of heave occurred over that depth, the measured values could be matched, but it is obvious that other factors are involved above the 10.5 foot depth. It is possible that the tell-tales did not measure the vertical movements in the soil, but the auger was screwed into the natural soil and should have moved with it. The structural continuity of the 8-inch-diameter casing should have caused a slight upward pull at the base of the casing and the weight of the grease was no more than 57 pounds per cubic foot, which was less than the overburden weight. Therefore the soil around the tell-tale anchor should have heaved more than if the casing were not there. The measured values are less than the calculated movements and it is probable that the tell-tales accurately measured the soil movements. The calibration curve for the nuclear devices was probably in error but the same curve was used for all of the readings, and changes of some magnitude were shown to have taken place. In summary, the major source of potential difference between the calculated and measured values of vertical expansion was the erratic nature of the soil.

A swell of almost 6 inches at station 245+00 was calculated by the potential vertical rise method. To date, maximum movement has been about half of that, 3 inches. Most of this occurred during the ponding period and the rise has been fairly consistent for almost six months.

Instrumentation could be improved by placing the tell-tales in a more nearly vertical line. Double, extra-strong 1-inch pipe casing would be pushed into a 1-1/2-inch-diameter boring, with the bottom end of the casing cut as a helix so that a bent plate auger 1.4 inches in diameter would fit as it was pushed into the soil. The auger would be welded to a 1/2-inch rod within the pipe casing, which would be pushed or driven to the bottom of the prebored hole, and then the auger would be turned-in a few inches. Then the casing would be withdrawn, and the soil above the auger would be tamped and the hole filled with grease.

The tell-tales could be set at different depths in a circle with 15-inch radius about an access tube for a nuclear probe. A psychrometer attached to the trailing edge of the auger could measure the total suction at that point. At the end of the test, moisture and density samples could be taken within the ring of tell-tales to check the nuclear calibrations. Such instrumentation would provide correlating data on the soil before, during, and after ponding, in one soil profile and would be useful in establishing relationships between suction, water content, volume change, and heave.

The tell-tales used in this test were sufficient for measuring the effectiveness of the ponding during the construction operations.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

Tests made during the study provided information on the wide variation in movements which take place throughout the soil; shrinkage and swells are three-dimensional problems and nonuniformity of material can cause water content to increase at one point and decrease at a point only ten feet away.

Apparently, ponding was effective in increasing moisture content and the increase seems to have been maintained after ponding was ended. However, stopping evaporation from the pond did not cause the soil to wet up rapidly at depth. A source of water is needed to increase water content of the drier areas at depth, and vertical permeability was such that little water reached below 3 feet after 45 days of ponding. The difference between results from the tests reported here and those in open fields at Waco may have been caused by open cracks in the natural surface which would have allowed faster moisture penetration.

The elevation rods used in this test are sufficiently accurate to show the level of the source of swelling. Thus far it is shallow, 4 feet or less, and represents at most a vertical movement of 3 inches at a 2-foot depth, compared to a calculated potential vertical rise of 6 inches. In general, these readings indicate a stimulation of swell by ponding that remained after the ponds were drained. Control rods outside the ponded area indicate no swell at all to date.

Together with moisture and density data gathered from the nuclear equipment readings, these elevation readings could be used to show some of the effectiveness of ponding. They indicated a shallow penetration and a shallow movement upward that was maintained for the six-month period following the ponding. Thus, this study tended to confirm the review of prior studies indicating that ponding is effective in reducing pavement roughness over subgrades with a high potential swell by inducing movement before placement of a final riding surface. A recommendation to pond for a specific construction project would depend on local conditions, soils, sequence of construction, allowable time delays, and comparable cost studies of the effectiveness of other methods in removing roughness, including post-completion level-ups. However, ponding should be included in any systems pavement analysis as a possible means of ultimate cost reduction.

It is recommended that readings at this site be continued in order to get a longer record of developments, including the possibility of deeper movements in a five-year period. Beyond this, plans should be developed for getting water deeper into the subgrade. One possibility is drilling 12 to 18-inch holes 20 feet deep, backfilled with sand and ponded for period of 60 to 90 days. It is also suggested that provisions be made for setting elevation tell-tales at varying depths in a continuation of the effort to find the depths at which these movements begin. Instrumentation improvements mentioned elsewhere in this report should be carefully considered. The increasing damage to roadways by swelling clays and the rising cost of highway maintenance in both materials and manpower may make the use of water and time required in the ponding procedure the best way to conserve national resources.

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