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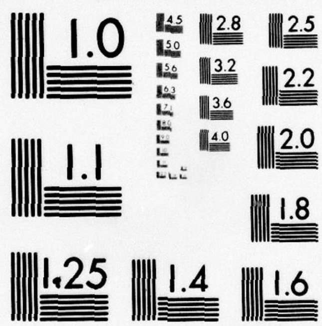

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PHASE IV REPORT

LEVEL III

VOLUME IV

RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM

EXISTING LOCKS AND DAM NO. 26
MISSISSIPPI RIVER, ALTON, ILLINOIS

Prepared for

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

By

Woodward-Clyde Consultants
Chicago, Illinois

15 July 1979

Contract No. DACW43-78-C-0005

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PHASE IV REPORT

VOLUME IV

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RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM.

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10 Jean-Yves Perez   Stanley F. Gizienski

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Vol III Results and Interpretation of Pile Driving Effects Test Program
Vol IIIA Appendices H through T, Results and Interpretation of Pile Driving Effects Test Program
Vol IV Results and Interpretation of Drilled-In Pile Test Program
Vol IVA Appendices A through E, Results and Interpretation of Drilled-In Pile Test Program
Vol V Results and Interpretation of Rock Anchor Test Program
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)
A series of tests examining various foundation systems and construction techniques were conducted on Ellis Island near Locks and Dam No. 26 in alluvial sand deposits underlain by glacial deposits and limestone. The chemical grout test consisted of grouting the upper 20 feet of the alluvial sand by injecting a number of different silicate and cement-bentonite grout types, while varying the grouting method, hole spacing, and injecting rates. Heave, lateral displacement, and pore pressure were monitored during grout injection. The in situ properties of the sand were measured before and after grouting by standard

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20. penetration tests, static cone penetration tests, pressuremeter tests, bore hole permeability tests, and shear wave velocity tests. Concurrently laboratory tests were conducted to investigate the strength and creep behavior of the grouted sand. After completion of grouting, the site was excavated to examine and evaluate the grouted sand. In the rock anchor test, inclined rock anchors were installed in limestone through 130 feet of alluvial and glacial deposits using a pneumatic down-the-hole hammer with an offset reamer. Load tests were conducted on three instrumentated rock anchors and the feasibility of installation of the rock anchors was determined by evaluating loss of ground during installation, performance of the installation equipment, and rate of installation. The drilled-in pile test consisted of installation of large diameter high capacity pipe piles by the Benoto method. The feasibility of installing these piles was determined by evaluating loss of ground during installation, performance of the Benoto equipment, and rate of installation. In the pile driving effects test, pile founded monoliths were constructed, supported on either one, eight or twelve timber piles jetted and driven in alluvial sand to a depth of 35 feet. After applying lateral and vertical load to the monoliths, steel piles were driven at varying distances from the monoliths while monitoring movement of the monolith and supporting piles; shear, moment, and axial load in the timber piles; and pore pressure, movement, and particle velocity, in the soil. Parameters examined were pile type being driven (sheet, pipe, or H-pile), pile driving hammer (diesel, air-steam, or vibratory), distance of driven piles from monolith, driving of multiple piles at the same distance from the monolith, load level applied to the monolith, and soil properties (grouted and ungrouted). Vertical and lateral load tests were conducted on each pile founded monolith. Tests were also conducted to assess what effect grouted soil has on piles. Piles were driven in both grouted and ungrouted sand to examine driving characteristics and lateral load tests were conducted on H and pipe piles in both grouted and ungrouted sand.

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PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM

SECTION 0

SUMMARY AND CONCLUSIONS

0 SUMMARY AND CONCLUSIONS

0.1 DRILLED-IN PILE TEST PROGRAM

The feasibility of constructing batter piles by a drilling method was investigated near Locks and Dam No. 26 on the Mississippi River. The piles had to be drilled through approximately 130 ft of submerged alluvial and glacial soil and socketed into limestone bedrock. The tests were designed to assess whether or not drilled-in pile construction has adverse effects, such as loss of ground and loosening, on the surrounding soil mass. They were also intended to investigate the feasibility of constructing drilled-in piles at angles of batter flatter than usual (that is, flatter than 4 vertical to 1 horizontal).

The tests were designed from November 1977 to May 1978. The test area was prepared from April to September 1978. The test instrumentation was installed in September and October 1978. The field tests were conducted from November 1978 to January 1979. Three test drilled-in piles at a 4 to 1 batter were installed, and the effects of construction were assessed by various measurements. No pile was installed at a batter flatter than 4 to 1; batters flatter than 4 to 1 were deemed unfeasible on the basis of observations made during construction of the first three test piles.

0.2 TEST AREA SUBSURFACE CONDITIONS

The test area was located on Ellis island, about one mile downstream of Locks and Dam No. 26 on the Missouri side of the Mississippi River. The subsurface profile at the locations of the test area consisted of approximately 100 ft of sand and gravel of alluvial and glacial origin, overlying limestone bedrock. The sand and gravel was overlain by about 25 ft of cohesive flood plain deposits. The subsurface conditions were investigated at the design stage and again immediately before the tests; the conditions were reassessed after the tests to detect changes caused by construction of the test piles. The subsurface investigations relied primarily on the use of in situ testing methods (dynamic and static penetration tests, pressuremeter tests, and density masurements using a nuclear probe).

0.3 TEST PROGRAM DESIGN

0.3.1 Test Area Selection

The test area was selected from four candidate test sites preselected by the Government. At the location of the selected test area, the subsurface conditions matched best the conditions at Locks and Dam No. 26. To a variable degree, these conditions are also representative of those at other navigation structures on the Mississippi River. The major difference was that the cohesive flood plain deposits forming the upper portion of the test area subsurface profile are not present under the existing structures.

0.3.2 Selection of Construction Method and Equipment

The requirements for the test piles were as follows. The piles had to be:

(1) installed by drilling techniques, as opposed to driving techniques, to minimize construction vibrations;

(2) installed through over 100 ft of submerged sand, gravel and occasional cobble zones, and socketed into bedrock;

(3) cased with a 20-in.-dia steel casing; and

(4) at a batter of 4 (vert) to 1 (hor) or flatter.

On the basis of literature review and discussions with specialized contractors and consultants, the Benoto method or some variation of it, was selected. Among available methods, only the Benoto method, which entails advancing a steel casing by applying oscillatory twisting motions and axial thrust to the casing while removing the soil inside the casing with hammergrabs, appeared to be capable of meeting the test requirements with some degree of success.

0.3.3 Effects of Pile Installation

The effects of drilled-in pile installation on the surrounding soil mass were assessed by monitoring ground instrumentation installed before the tests, measuring the quantity of soil excavated from inside the advancing casing, and measuring in situ soil properties after the tests. The ground instrumentation consisted of surface and subsurface settlement monitoring devices (surface reference points, Borros gages, and Sondex rings), and inclinometer casings.

The soil excavated from inside the casing was collected in bins and weighed on scales. On the basis of water content and in-place unit weight measurements, the volume that the excavated soil occupied in the ground was calculated and compared to the theoretical volume of the casing.

The in situ properties of the soil after drilled-in pile installation were inferred from in situ borehole tests; these properties were compared to the initial soil properties.

0.3.4 Angle of Batter

It was planned to attempt installation of as many as three test piles at batters of 3 (vert) to 1 (hor), to 2 (vert) to 1 (hor).

0.4 TEST RESULTS

0.4.1 Effects of Pile Installation

Very little ground loss, if any, was experienced during drilling of two test piles at 6 ft centers. The volume of excavated soil exceeded the theoretical volume of the casing only near the bottom of the two piles. Elsewhere, the volume of excavated soil was equal to or less than the theoretical volume. Ground surface settlement was primarily caused by consolidation of the cohesive flood plain deposits under the weight of the fill placed to prepare the test area working platform. In general, the observed ground settlement at depth was small and proportional to the thickness of alluvial and glacial sand underlying the measurement point. The maximum settlement attributed to the installation of two closely-spaced test piles was 0.26 in., measured at el 395, about 100 ft above bedrock surface.

Horizontal ground deformation was also small, generally less than 0.5 in. and not exceeding 1.5 in. The soil deformation generally did not indicate soil movement toward the piles. Slight reductions in soil properties (density, in situ stresses, and stiffness) were inferred from in situ tests. The reductions, however, were small, compared to the data scatter inherent in the soil; for all practical purposes, it can be concluded that the soil properties did not significantly change due to drilled-in pile installation.

0.4.2 Angle of Batter

On the basis of the very slow rate of progress experienced in the three piles drilled at 4 (vert) to 1 (hor) batter, it was concluded that flatter angles of batter would not be economically feasible. Installation of test piles at batters flatter than 4 to 1 was not attempted.

0.4.3 Evaluation of Equipment and Techniques

The Benoto boring machine was generally capable of performing the functions specified for installing piles at 4 to 1 batter. Various modifications to the original equipment had to be implemented to increase the productivity. Despite these modifications, the rate of progress remained very slow: 2.2 ft/hr from 0 to 50 ft depth; 1 ft/hr from 50 ft to 100 ft depth; 0.6 ft/hr below 100 ft depth. The rock socket was readily drilled using a rotary drilling technique and a tricone roller bit.

Water level and depth of soil plug inside the casing were shown to affect the rate of progress to a large extent. The water level inside the casing had to be maintained above the outside groundwater surface to avoid blows at the bottom of the casing. As a general rule, it was found that the soil plug inside the casing should not be less than 1 ft to avoid creep of the soil into the casing nor more than 4 ft to be able to advance the casing by oscillation and axial thrust.

0.5 COST INFORMATION

On the basis of the production rates experienced during the tests, corrected for major atypical delays (such as equipment modifications), but uncorrected for testing interruptions, the cost of constructing one drilled-in pile was estimated at $42,500. This estimated cost assumes that equipment and crew requirements would be the same as those experienced during the tests. It is likely that this estimated cost could be decreased by some unknown amount and be more representative of large scale production work.

0.6 SUMMARY OF CONCLUSIONS

In summary:

(1) the Benoto method is feasible to install 20-in.-dia piles at 4 to 1 batter;

(2) the method is extremely slow and does not appear to be economical to install 20-in.-dia piles at 4 to 1 batter;

(3) the method results in very small or no loss of ground provided the water level inside the casing is maintained above groundwater level;

(4) ground deformation associated with drilling is small;

(5) soil properties are not significantly affected by drilling; and

(6) the method is not feasible for angles of batter flatter than 4 to 1 under the conditions prevailing at the test area.

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF
DRILLED-IN PILE TEST PROGRAM

SECTION 1

PURPOSE AND OBJECTIVES

1 PURPOSE AND OBJECTIVES

1.1 PURPOSE

The drilled-in pile tests described in this Volume IV were part of an investigation and test program designed to provide technical bases for the evaluation of various overwater construction schemes and techniques, that could be used to strengthen existing navigation structures, such as Locks and Dam No. 26. Batter piles installed by drilling methods are among potentially attractive schemes. The purpose of the drilled-in pile test program was to assess the feasibility of constructing relatively long, high-capacity batter piles by a drilling method, through submerged alluvial and glacial soil typically found in the Mississippi River Valley.

The investigation and test program was conducted on Ellis Island, approximately one mile downstream of Locks and Dam No. 26 on tne Missouri side of the Mississippi River. In addition to the drilled-in pile tests, the test program also included an assessment of chemical grouting in alluvial sand (Volume II), an assessment of pile driving effects (Volume III), and an evaluation of the construction feasibility of rock anchors (Volume V). Summaries of conclusions for each of these tests are presented in Volume I.

The overall foundation investigation and test program was performed under contract DACW43-78-C-0005 between the US Department of the Army, Corps of Engineers, St Louis District and Woodward-Clyde Consultants, Chicago, Illinois.

1.2 OBJECTIVES

The objectives of the drilled-in pile test program were to assess:

(1) the feasibility of constructing 20-in.-dia, steel-cased piles at a 4 to 1 batter through submerged sand and gravel and into rock, using drilling methods;

(2) whether or not pile drilling has an adverse effect on the surrounding soil mass, such as loss of ground; and

(3) the feasibility of constructing drilled-in piles at angles of batter flatter than 4 to 1.

1.3 ORGANIZATION OF VOLUME IV

The concept of the drilled-in pile test program, including the selection of the equipment, test variables, and procedures are presented in Section 2. The subsurface conditions of the test area are discussed in Section 3. Section 4 describes the instrumentation installed and monitored during the tests. The test results concerning the effects of drilling on the surrounding ground are presented in Section 5. An evaluation of the drilled-in pile equipment and techniques is given in Section 6. Cost information isre summarized in Section 7.

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF
DRILLED-IN PILE TEST PROGRAM

SECTION 2

CONCEPT AND DESCRIPTION OF TESTS

2 CONCEPT AND DESCRIPTION OF TESTS

2.1 SELECTION OF EQUIPMENT

2.1.1 Requirements

Numerous methods of installing drilled-in piles and piers have been used (Carson 1965, Chellis 1961, Woodward et al 1972, and Weinhold 1970). Few were considered suitable for the subsurface conditions of the test area and the batter requirements of the test piles. An adequate method must permit installation of 20-in.-dia casings on a batter, through 130 ft of medium dense to dense, submerged sand, gravel, and cobble. It must allow drilling of a socket into hard limestone and through occasional obstructions, such as boulders and buried wood.

2.1.2 Available Equipment

In developing the test program design, a review was made of available drilled-in pile or pier installation methods, including:

(1) Prestcore piles (Chellis 1961) involving advancing a steel casing by well-boring methods;

(2) uncased drilled-in piles;

(3) Presscrete piles (Chellis 1961) involving sinking a sectional steel casing into the ground by jetting, jacking, or other means;

(4) auger piles;

(5) drilled-in caissons by Thornley patent (Thornley 1978); and

(6) Benoto-type piles.

Among these, only the Benoto method, or some variation of it, was expected to be capable of achieving with some degree of success the objectives of the tests. The Benoto EDF boring machine, described by Chellis (1961), was developed in France in the 1950's and has been used worldwide. In the US, this type of equipment was used to install caissons, particularly in Chicago in the 1960's. In fact, the very equipment used for the test had been active in Chicago until a few years ago. Modern powerful augers generally supplanted the Benoto machine for drilling through the Chicago clay and hardpan in the 1970's. A sketch of the Benoto machine as modified for this project is shown in Fig. 2.1.

A relatively recent variation of the Benoto method, the Hochstrasse method was also considered at the design stage. The Hochstrasse method, which involves twisting a steel casing using two air-activated pistons, was not selected for the tests for two reasons. The alternating rotation movement of the casing imparted by the air pistons was considered to be too abrupt, as opposed to the continuous oscillations of the Benoto machine; the sudden casing movement would result in excessive ground vibrations. Also, the Hochstrasse method was not known to have been used to install piles on a batter.

The general testing contractor initially proposed to use a casing oscillator that operates on the same principle as the Benoto machine, but is of a more recent design. The proposed casing oscillator was similar to that used by the general testing contractor on a dam repair project for the Corps of Engineers at Wolf Creek, Tennessee. As illustrated schematically in Fig. 2.2, the equipment consists of two major parts; the hydraulic casing oscillator and thruster, and a service crane with wire rope guide and clamshell. Unfortunately, the configuration of the equipment was such that the service crane, during installation, would be located over the numerous items of instrumentation installed to measure ground movements, thus making them inaccessible for measurements during progress of the work. The Contractor was given the option to design and construct an elevated working platform that would permit access to the instruments under the crane and the casing oscillator. He elected to mobilize a Benoto machine which would not interfere with the instrumentation.

2.2 DESCRIPTION OF TESTS

2.2.1 General

The intent of the drilled-in pile tests was to install 20-in.-od steel pipe-cased piles at a batter of 4 to 1 through saturated sands, gravels and cobbles into bedrock while at the same time observing ground movements and loss of ground. Another aspect of the test was intended to investigate the feasibility of installing the drilled-in piles on flatter angles of batter than 4 to 1. To accomplish the tests, an area was selected where the subsurface conditions consisted of saturated alluvial and glacial granular materials extending to limestone bedrock. The relationship of the drilled-in pile test area to those of other tests of the foundation investigation and test program is shown in Fig. 2.3. The area was raised to about el 422 by placing moderately compacted clay and silt fill obtained from on-site excavations.

2.2.2 Test Area Configuration

The layout plan of the drilled-in pile test area, which was surfaced with crushed rock over an area of 100 by 150 ft, is shown in Fig. 2.4. The area was divided into two sub-areas.

Tests for construction feasibility and effects of installing drilled-in piles on a 4 (vert) to 1 (hor) batter were made in the southern half of the site (test piles DP1, DP2, and DP3). Test pile DP1 was to be installed on a west to east batter 19 ft away from DP2 and DP3 which were to be installed on a line parallel to DP1. The northern half of the area was set aside for installation of drilled-in piles on flatter angles of batter with piles separated by a distance of 12 ft.

2.2.3 Installation of Cased Drilled-in Piles

Three 20-in.-od, cased, drilled-in piles were to be installed with the Benoto machine at the designated locations. DP1 was to be installed first on a 4 to 1 batter for the purpose of establishing detailed operating procedures and identifying potential construction difficulties. It was not intended to place concrete in DP1.

Piles DP2 and DP3 were to be installed on 6 ft center to center also on a 4 to 1 batter as shown in Fig. 2.5. After drilling the shaft and socket, a steel H beam stub core was to be placed and each pile filled with tremie concrete. Construction operations were to be observed and documented to gain information on production time, control of water level in the casing, and removal of cobbles and boulders.

Measurements of in situ soil properties were to be made by standard penetration tests (SPT), cone penetrometer tests, and pressuremeter tests in four borings before and after installation of the drilled-in piles. Inclinometers and subsurface settlement probes (three-dimensional deformation gages), Borros settlement points, and surface settlement points were to be installed prior to installation of the piles and then monitored during and after installation of the piles to detect ground deformation. The cuttings removed from the pile casing were to be measured and compared to the theoretical volume of the hole to assess ground loss.

2.2.4 Angle of Batter

It was planned as part of this test program to attempt to excavate (but not concrete) three drilled-in piles at batters of 3 to 1, 2.5 to 1, and 2 to 1, as shown in Fig. 2.6. The piles would consist of advancing 20-in.-od steel casings through the alluvial and glacial deposits and at least 1 ft into rock and drilling of a 4-ft socket in rock. The installation operations were to be observed and documented to gain information on excavation efficiency at flat angles of batter and to assess the flattest practical angle of batter. No measurement of soil movements was planned for these tests. These tests were not made because sufficient information was obtained from the first three piles at a 4 to 1 batter to conclude that flatter angles of batter would not be feasible.

2.3 DESCRIPTION OF EQUIPMENT

2.3.1 General

The equipment used to install the drilled-in piles consisted of the special drilled-in pile equipment, as described below along with weighing bins and scales and miscellaneous hand tools. Materials included steel casing steel H beams, and ready-mix concrete.

2.3.2 Special Drilled-In Pile Equipment

Specifications. The project specifications set forth that "the Contractor shall provide and use a special boring machine which is capable of advancing a 20-in.-od steel pipe casing by combination of reciprocating circumferential oscillations and axial thrust and of excavating materials consisting of wet firm clays, dense sands and gravels below the water table containing cobbles and possibly boulders, and of limestone rock. The machine shall be capable of advancing steel pipe casings at a batter and on the alignments specified to depths of at least 150 ft. It shall be capable of installing piles at a batter of at least 4 vertical to 1 horizontal and of experimenting with batters as flat as 2 vertical to 1 horizontal. It is the intent in this test program to use equipment similar to the Benoto EDF 55 machine or one of more recent design. The special drilled-in pile equipment shall include all necessary pumps, airlifts, bailing buckets, tremie pipes as well as rotary drilling equipment capable of drilling an 18-in.-dia hole in hard rock at a depth of at least 150 ft below ground surface on a batter. The equipment shall be complete with all accessories required for the test including excavating tools, scales, and weigh bins, as described herein or on applicable drawings".

Benoto Boring Machine. The Benoto EDF 55 machine is a self-propelled, self-powered diesel-hydraulic unit capable of installing casings from 20- to 40-in.-dia to a depth of 350 ft. Other variations of the Benoto machine have reportedly sunk 5-ft-dia shafts up to 230 ft in depth. Photographs of side and frontal views of the EDF 55 equipment are shown in Fig 2.7. The machine advances a steel casing by applying oscillatory twisting motions and axial thrust to the casing. The soil inside the casing is removed by various tools, usually hammergrabs.

The steel casing is gripped by a circular collar a few feet above ground surface. To prevent slipping, steel beads are welded on the inside face of the collar. When the collar is closed, the steel beads bite into the surface of the steel casing. The opening and closing of the collar is actuated by a hydraulic piston. Below the gripping collar is the collar guide which, together with the bonnet located on the traveling carrier at the top, maintains the casing in alignment during installation. Reciprocating pistons located on a horizontal plane immediately in front of the engine apply lateral forces alternately to the end of the oscillator lever arm, which in this case is 9.25 ft long. The lever arm, which is connected to the gripping collar, thus applies oscillatory movement to the gripping collar. Initially, the equipment was capable of rotating the casing up to about 10 degrees total at a maximum torque of 300,000 ft-lb. The equipment was modified during the tests to increase the applied force and the maximum rotation to 17 degrees by installing longer reciprocating pistons. The machine became then capable of developing a maximum torque of 600,000 ft-lb and a circumferential movement of 3 in.

The collar and the casing gripped by the collar receive an axial thrust from two large axial pistons mounted on the sides of the machine mast. The thrust pistons are capable of applying a downward force up to 98,600 lb and an upward force of 78,125 lb. The total weight of the equipment is 64,000 lb. To be able to utilize the full axial thrust of the Benoto machine during the test, the machine was anchored down to a large concrete pad which was cast around the test piles. Tiedown threaded rods, 2-in.-dia anchored the boring machine to 1.5 in. thick steel plates embedded in the concrete pad which weighed about 75,000 lb at DP1 and 117,600 lb at DP2 and DP3.

The original Benoto machine was capable of installing piles on 15 degree batter (3.73 vertical to 1 horizontal). The machine was modified by the general testing contractor prior to mobilization to accomodate flatter angles of batter up to 2 to 1.

Hammergrabs. Cable-operated single-line hammergrabs manufactured by Casagrande & Company of Italy were used during the test to remove soil from inside the advancing casing. Two sizes of hammergrabs were used. The characteristics of each are given below:

Single Line Hammergrab BLP

	TypeA	Type B
Nominal dia (mm)	400	500
Grab dia (mm)	370	435
Grab dia (in.)	14.45	16.99
Length, open grab (mm)	3040	3040
Length, open grab (in.)	118.75	118.75
Length, closed grab (mm)	2750	2810
Length, closed grab (in.)	107.42	109.77
Weight (kg)	1100	1250
Weight (lb)	2420	2750
Grab capacity (kg)	10	13

Two hammergrabs (type B) are shown in Fig. 2.8. One of the hammergrabs was equipped with two-pronged jaws; the other had three-pronged jaws. Both were used during the tests. The grab body is rectangular in shape and contains the mechanism which controls the opening and closing of the jaws. The automatic opening of the grab is obtained by means of a hooking and releasing device. The release jig is located in the carrier head of the Benoto machine and the closing device is located at the top of the hammergrab.

Rotary Drill. Drilling of the rock socket in limestone was accomplished using a Driltech rotary drill rig set at a 4 (vert) to 1 (hor) batter and using a tricone roller bit. The roller bit, which was designed to drill an 18-in.-dia hole, was attached to a follower designed to maintain the alignment of the drilled socket on the centerline of the steel casing.

Water Storage Tank. A requirement of the test program was to maintain the water level within the pipe casing at least 2 ft above the groundwater table. A 6000-gal tank was used to maintain a ready source of water. The water was pumped from the tank through a hose into the casing as needed. The tank was filled periodically by pumping from the Mississippi River about 500 ft away.

Miscellaneous Equipment. Various other tools or pieces of equipment were included with the special drilled-in pile equipment. The rock chisel shown in Fig. 2.8 was fabricated by the contractor for the purpose of breaking up cobbles and boulders and the limestone bedrock as necessary to advance the pile.

For the purpose of cleaning out the rock socket, a bailing bucket consisting of heavy-wall steel pipe approximately 10 ft long and 10 in. od, and fitted with a flap near the bottom was provided. A 15-t hydraulic crane was used for placing the successive sections of steel casing into the Benoto machine and for other tasks. Welding was accomplished with 250-amp, gas-engine driven, electric arc welders.

2.3.3 Weighing Bins

Bins. Two trough-shaped wooden bins each capable of containing 32 ft^3 of excavated material were built by the contractor for the purpose of collecting and weighing the cuttings during excavation. At one end, a No. 10 mesh screen opening, as shown in Fig. 2.9, allowed excess water to drain out of the soil. The bins were provided with hooks at the corners for lifting.

Scales. Two 2000-lb capacity platform scales were used to weigh the bins and soil. Special wooden racks were placed on the scales to support the weighing bin at an inclination for drainage of the soil. The photograph in Fig. 2.9 shows a weighing bin on the scales.

2.3.4 Miscellaneous Tools

Alignment of the installed casing and the Benoto pile leads were checked using a surveyor's transit. A 4-ft long carpenter's level and a wood batter board were used for checking the batter of the casing and the verticality of the Benoto leads.

2.3.5 Materials

Casing. The casing consisted of 18-ft sections of 20-in.-od, 1.031-in. thick steel pipe. The pipes conformed to ASTM A252, Grade 2, longitudinally welded.

The leading edge of the first length of casing was serrated as shown in Fig. 2.10. The 2-in.-deep teeth were surfaced with hard, abrasive weld beads. The contractor proposed this type of cutting edge instead of the cast-steel, inside cutting shoe (Associated Pile and Fitting Corporation, No. 0-14001) specified. The top end of the first and subsequent lengths of casing were cut square as shown in Fig. 2.10. The original design called for casing lengths to be 25 ft. The length was reduced to 18 ft at the time the casing was ordered because this was the maximum that could be handled in the Benoto leads. The casing lengths were spliced by electric arc, fully-penetrating, bevel welds, the lower end of each casing length being beveled at 45 degrees, as shown in Fig. 2.10. A special clamp was used to hold the newly added section to the previously installed section of casing; after confirmation of alignment the clamp was secured and welding accomplished.

Stub Core. The stub core installed in the rock socket was a section of steel 10-in. H beam weighing 57 lb/ft and having a length of 9 ft. It was of ASTM A36 steel and was centered with the aid of No. 4 reinforcing bars welded to the beam. To facilitate dispersion of tremie concrete around the H beam, holes were cut in the web and flanges.

Concrete. Concrete was placed in the socket and casing through a 6-in.-id, 6.38-in.-od tremie pipe. Concrete was made with Type I cement, two parts river sand to one part 3/8 in. pea gravel; the concrete had a 6-in. slump, and 6.1-percent air content. Average seven-day strength was 2500 lb/in^2 and average 28-day strength 3900 lb/in^2.

2.4 TEST VARIABLES

2.4.1 General

The drilled-in pile test program was designed to observe the effects of certain variables and parameters, such as types of excavating tools and techniques, varying the water level inside the casing, group effects on soil disturbance and deformation, and angle of batter.

2.4.2 Excavation Tools and Techniques

As described in Section 2.3.2, two sizes of hammergrab were used, one (Grab A) weighing 2420 lb and having a diameter of 14.45 in., the other (Grab B) weighing 2750 lb and having a diameter of 16.99 in. Grab B, which almost completely filled the 18-in.-id pipe, did not fall freely when submerged because of the restricted annular space. Measurements of rate of fall in water indicated that it was of the order of 4.5 ft/s which is considerably less than the rate of free fall in water. The hammergrab in effect was behaving, during fall, as a piston in a cylinder of fluid and the reduced impact at the soil surface did not allow full penetration of the jaws of the hammergrab into the soil. Consequently, the amount of soil recovered within the jaws was less than full capacity. This was true when the excavation was being performed below depths of the order of 35 ft below the groundwater surface.

Modifications were made to the techniques to determine whether better soil recovery could be achieved. These modifications included varying the rate of fall and the rate of withdrawal, varying the rate of closing of the jaws, delaying closing of the jaws after penetration, and varying the depth of soil plug in the pipe.

Additional variations or modifications included using a smaller diameter hammergrab (Grab A), adding weight to Grab B by welding on steel plates, cutting away portions of the body of Grab B to reduce its cross-sectional area, using three-pronged jaws and two-pronged jaws on the same hammergrabs, and loosening the soil with the rock chisel.

Other variations during the test included varying the water surface level within the casing with respect to the groundwater surface. The interior water level varied from as low as 36 ft below to as high as 39.5 ft above the groundwater surface. Also varied was the depth of soil plug within the pipe. That varied from as much as 10 ft to no plug at all.

The procedure for oscillating and advancing the casing was also varied, as were the procedures for advancing into rock. They included chopping with the two-pronged and three-pronged hammergrabs, rock chisel, roller bit, and bailing.

Some of these modifications and variations did improve production, but generally, the improvement was small. In particular, adding weight to the type B hammergrab and maintaining a small soil plug at the bottom of the casing (1 to 1.5 ft) appeared to slightly improve production.

2.4.3 Group Effects

The effects of group installation were investigated by constructing two drilled-in batter piles 6 ft on centers in an instrumented area and monitoring the instruments before, during, and after drilling.

2.4.4 Angle of Batter

It was planned, as stated in Section 2.2.4, and set forth in the test program plans and specifications, to install piles at four angles of batter as follows:

Angle degree	Approximate Inclination (vertical to horizontal)
14	4 to 1
18.5	3 to 1
22	2.5 to 1
26.5	2 to 1

Three piles were installed at a batter of 4 to 1. Due to the slow rate of installation progress, it was decided that installation of piles at the flatter angles of batter would not be feasible within a reasonable time. The flat batter piles were eliminated.

2.5 SEQUENCE OF ACTIVITIES

The sequence of performance of the drilled-in pile test program was as follows:

(1) measurement of initial soil properties at the location by standard penetration tests (SPT), cone penetrometer tests, and pressuremeter tests;

(2) installation of instrumentation on lines parallel to the centerline of drilled-in piles DP2 and DP3;

(3) installation of a 20-in.-od pile on a 4 to 1 batter into bedrock at location DP1 and the making of observations and appropriate modifications; (the purpose of this installation was to establish detailed operating procedures of the rig and identify potential construction difficulties);

(4) at locations DP2 and DP3, where instruments were installed, two drilled-in pipe piles were installed on a 4 to 1 batter as shown on Fig. 2.5 and filled with concrete; instrumentation was monitored at intervals during progress of drilling as described in Section 4; material excavated was weighed at intervals in the weighing bins;

(5) it was originally planned to install piles at locations DP4, DP5, and DP6 at batters of 3 to 1, 2.5 to 1, and 2 to 1. As noted in Section 2.4.4, due to the slow progress of excavating the 4 to 1 piles, it was deemed potentially unproductive to install piles at the flatter batters. That work was not done.

2.6 INSTALLATION PROCEDURES

After installation of instrumentation in the test area, the Benoto boring machine was aligned along the axis of the batter pile and centered over the pile location at the ground surface. The leads were adjusted to the specified batter and the machine anchored to the concrete anchor pad. A steel casing with the serrated teeth at the bottom was lifted into position with a hydraulic crane and held in place near the bottom by the gripping collars and collar guide and at the top by the bonnet attached to the lower frame of the carrier. Also attached to the carrier at bonnet level, were the working platform and above that the hammergrab guide carrier. The entire carrier was capable of riding up and down the leads as necessary to follow the top of the casing.

A transit positioned about 300 ft from the pile and along the centerline of the batter pile, was used to check the verticality of the casing. A 4-ft carpenter's level and a beveled batter board were used to check the batter of the casing. The first 18-ft length of casing was lifted into posiition with a 15-t hydraulic crane and after alignment was clamped by the gripping collar and held in alignment by the collar guide and bonnet. It then was forced into the upper clay to a depth of 14 ft, thus leaving 4 ft protruding from the ground. It was found that a 4-ft protrusion was necessary to allow for the gripping collars and welding. Then a second 18-ft-long casing was lifted into position and placed on top of the first section. A special welding clamp was loosely attached to both casing sections and the bonnet lowered onto the top of the casing thus allowing the casing alignment to be checked. Deviations were corrected by adjusting the tilt of the leads. It was found that, if the first casing was properly aligned, the following casings were usually properly aligned. Spacers 1/8-in. thick were placed between the ends of the two casing sections to prevent trapping of gas bubbles during welding and the welding clamp was tightened. The upper end of the first casing was flat and the bottom of the second casing was beveled.

The weld was then made as a single-bevel, full-penetration, electric arc weld all around the circumference of the casing. The weld was ground flush with the outside of the casing.

The hammergrab held by the hoisting cable and with the jaws open was dropped down the inside of the casing. After digging into the bottom, the jaws were closed by the hoisting cable and the hammergrab lifted out of the casing up into the grab guide carrier. The grab guide carrier, which was hinged at the top, was swung outward over the hopper opening in the platform of the carrier and the jaws opened thus discharging the contents into the weighing bin (Fig. 2.11). The grab guide carrier containing the hammergrab was then returned to position over the casing for the next cycle of excavation. To lock the jaws into an open position, the hammergrab was hoisted sharply upward into the release jig mounted in the upper end of the grab guide carrier.

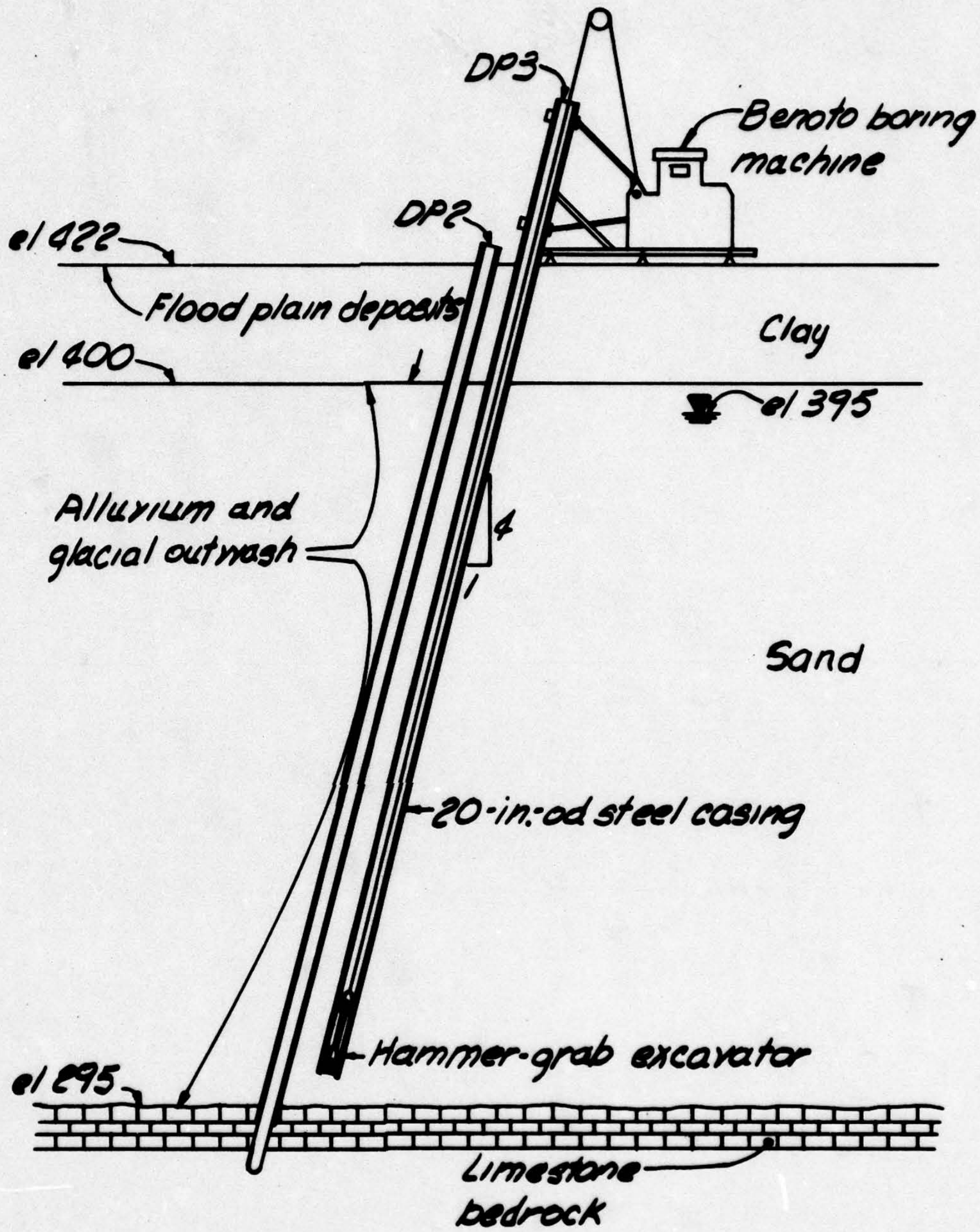
Periodically during excavation, the casing was pushed into the ground while at the same time being oscillated circumferentially. The relationship between excavation, pushing, and oscillation was varied as described later.

Excavated material, during installation of DP2 and DP3, was discharged into weighing bins as shown in Fig. 2.11 for the purpose of estimating the volume of material being removed. The measurement procedures and results are described in Sections 4.3 and 5.1, respectively.

Periodically, the depth to the excavated surface and to the water level in the casing was mesured with weighted tapes and electric probes. The depth of casing in the ground was monitored by marking the casing in 1-ft increments and measuring from welded joints or from the top end of the casing.

When bedrock was encountered, an unsuccessful attempt was made to excavate the rock, first using the two-pronged and then the three-pronged hammergrab. The rock chisel was likewise unsuccessful in breaking up the rock efficiently for excavation by the hammergrab. The bedrock was readily excavated to the design depth using a tri-cone roller bit operated by the rotary drill described in Section 2.3.2. Most of the cuttings were removed by the circulating water without resorting to the use of drilling muds such as bentonite. The remaining cuttings were removed using the bailing bucket described in Section 2.3.2. Bailing continued until the water was clear of cuttings and measurements to bottom of the excavation confirmed the depth to rock surface was the same as indicated by the rotary drilling.

After cleaning the hole, the H beam stub core was inserted into the rock socket and concrete placed by tremie method. Details of the stub core and rock socket are shown in Fig. 2.12.



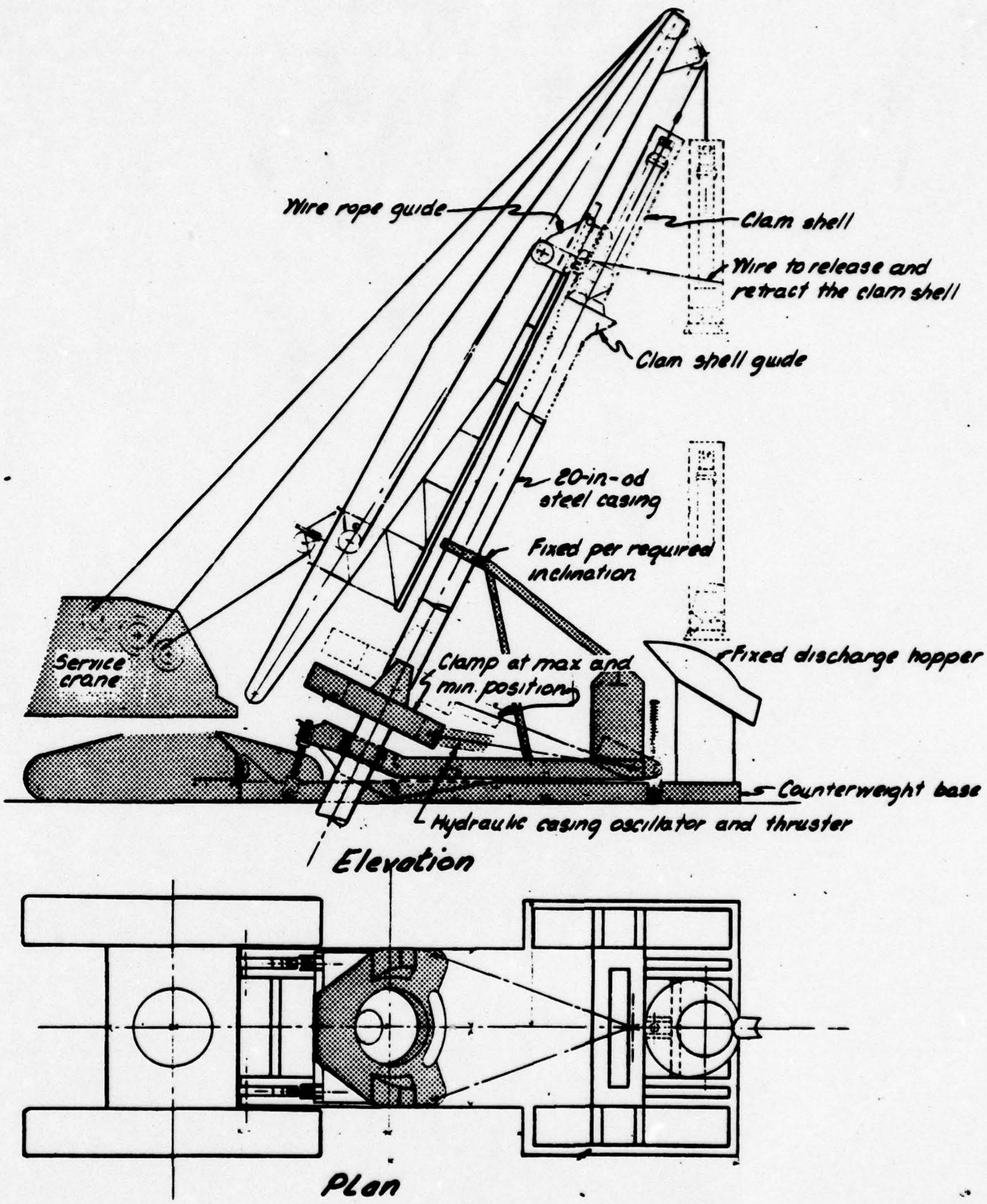
DRILLED-IN PILE TEST PROGRAM

INSTALLATION OF DRILLED-IN PILES WITH BENOTO MACHINE

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

Woodward-Clyde Consultants
Y7C026 Phase III

Fig. 2.1



DRILLED-IN PILE TEST PROGRAM

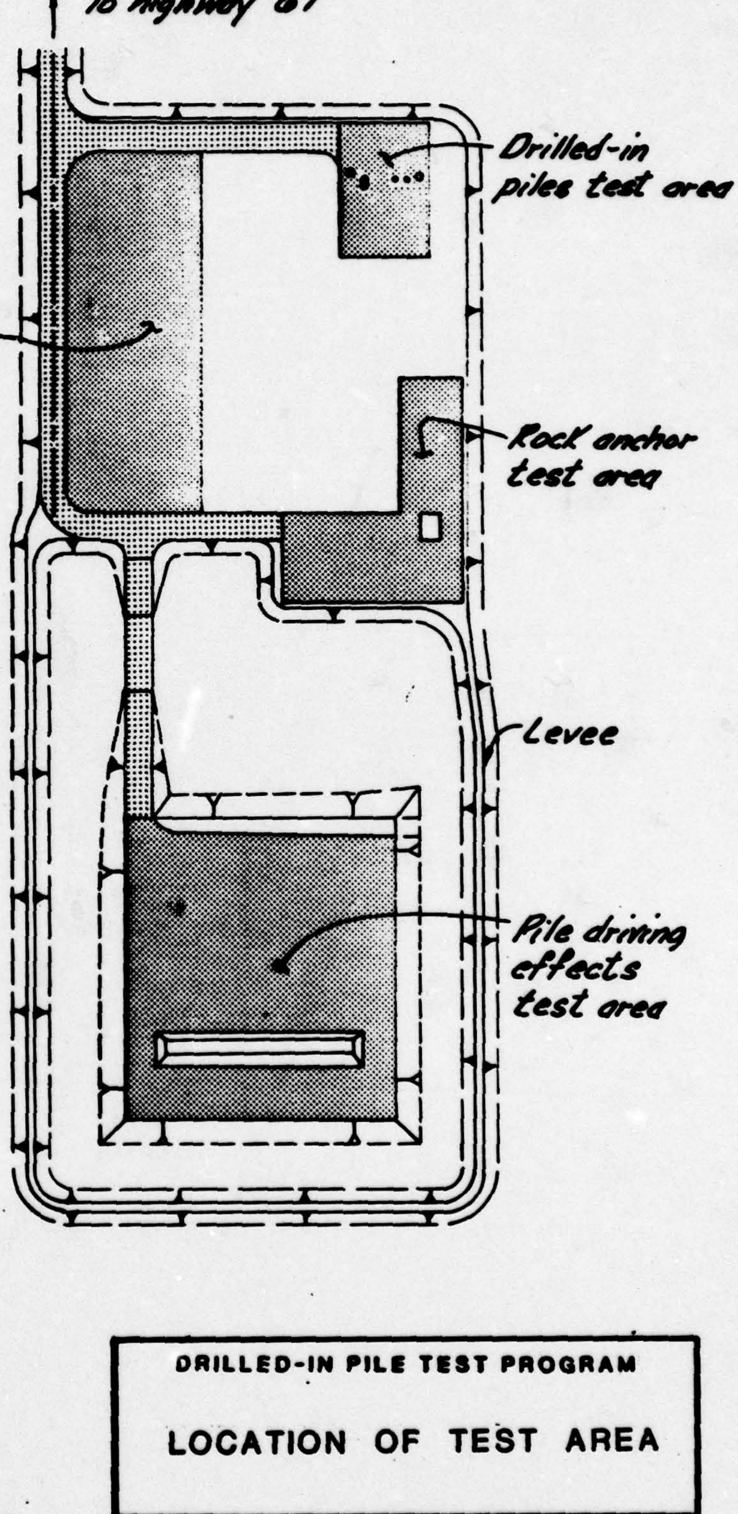
CASING OSCILLATOR INITIALLY PROPOSED BY CONTRACTOR (NOT USED)

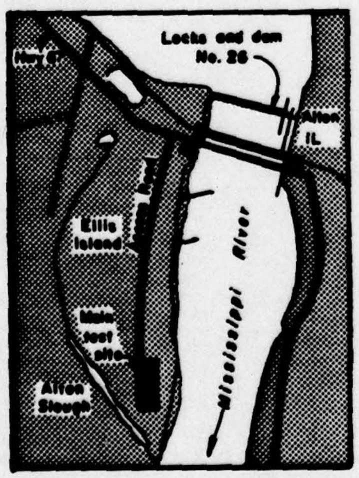
FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-79-C-0066

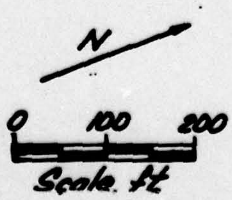
Woodward-Clyde Consultants

Fig. 2.2

Y7C025 Phase III







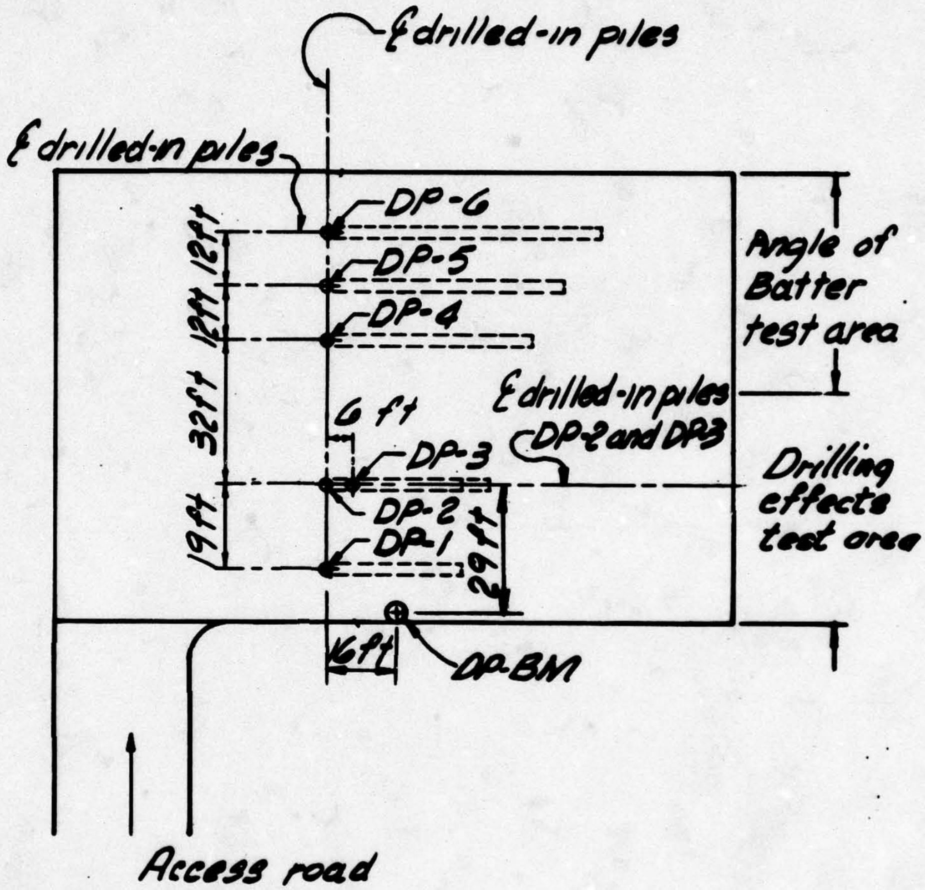
DRILLED-IN PILE TEST PROGRAM

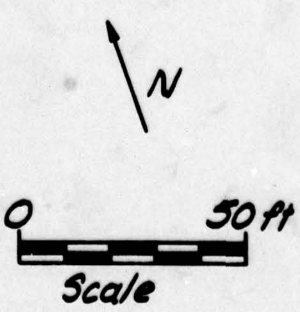
LOCATION OF TEST AREA

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

Fig.2.3





DRILLED-IN PILE TEST PROGRAM

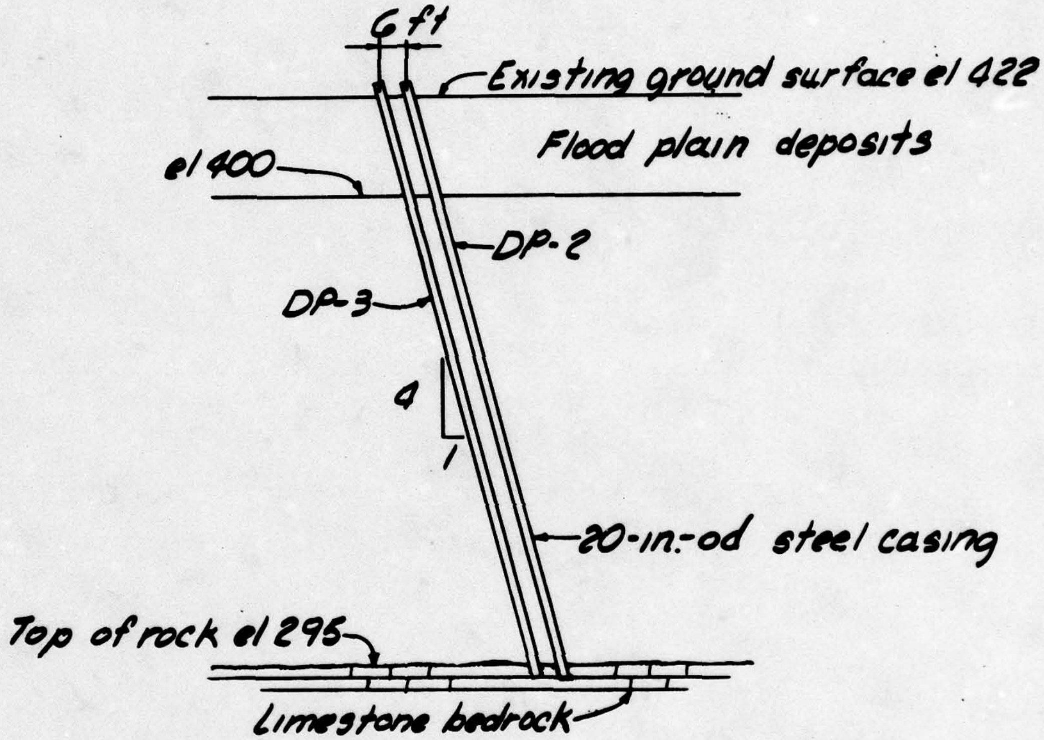
PLAN OF DRILLED-IN PILE TEST AREA

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0066

Woodward-Clyde Consultants

Y7C825 Phase IX

Fig. 2.4



Note:
See fig 2.4 for location of piles

DRILLED-IN PILE TEST PROGRAM

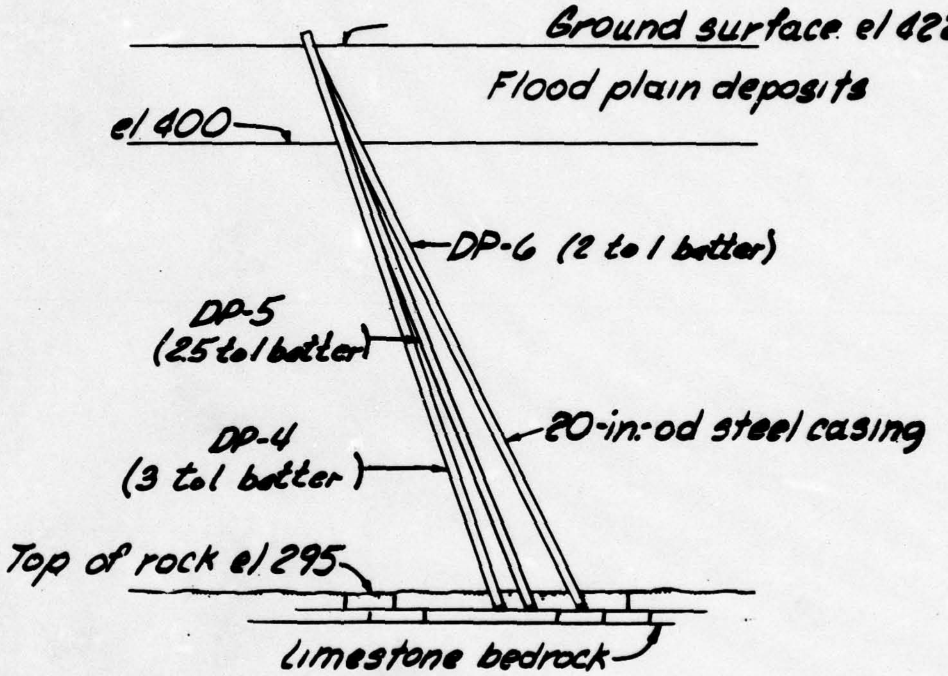
PROFILE OF DRILLED-IN PILES DP2 AND DP3

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants
Y7C628 Phase II

Fig. 2.5

0 50 ft
Scale



Note:
See fig 2.4 for location of piles

DRILLED-IN PILE TEST PROGRAM

PROFILE OF PROPOSED DRILLED-IN PILES DP4, DP5 AND DP6

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

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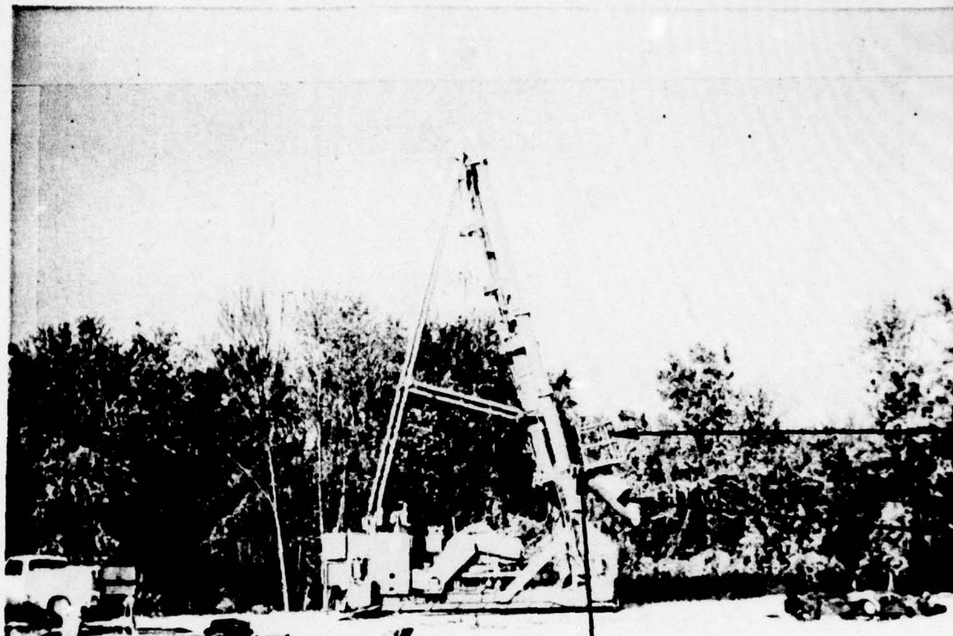
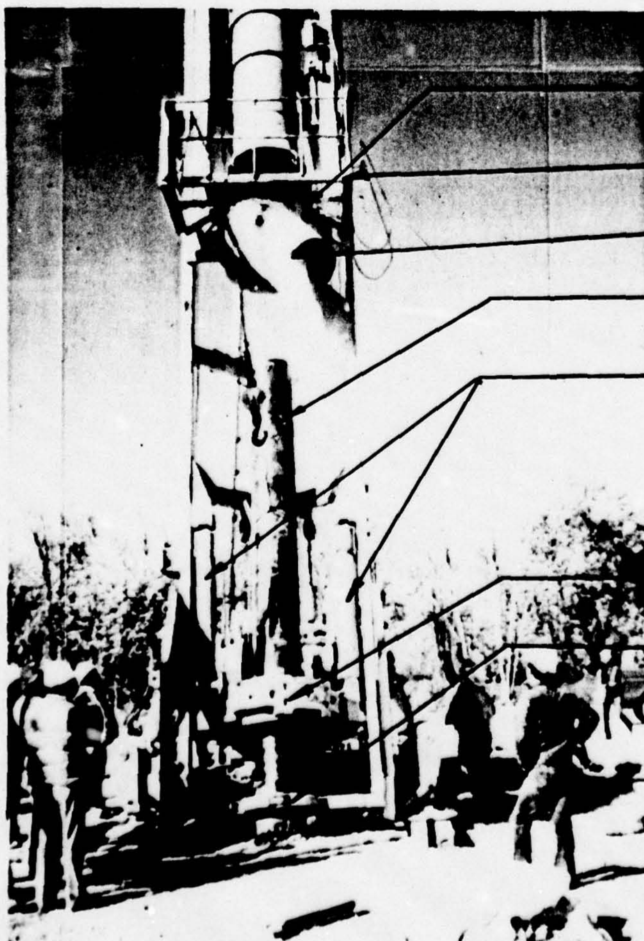
Y7C626 Phase IX

Fig. 2.6





DRILLED-IN PILE TEST PROGRAM

BENOTO EDF 55 MACHINE

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

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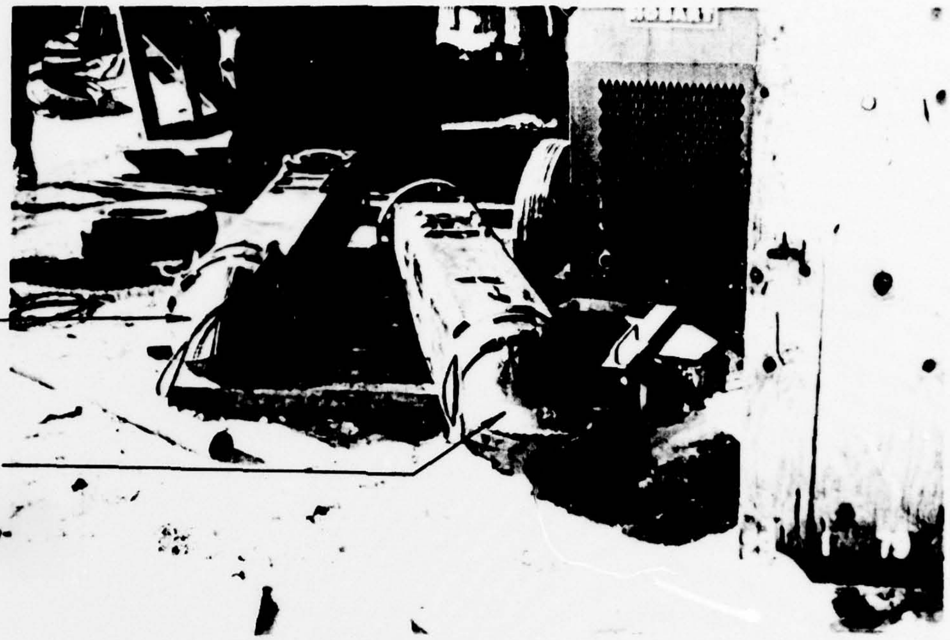
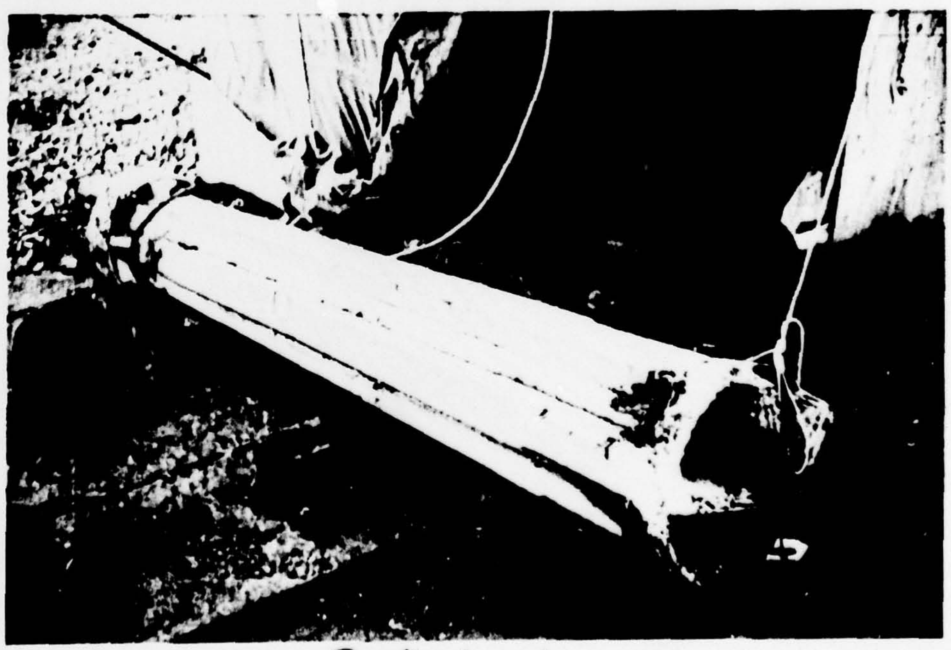
Y7C825 Phase IX

Fig. 2.7



Hammergrabs



Rock chisel

Note:
Specifications for hammer-grabs are in section 2.3.2

DRILLED-IN PILE TEST PROGRAM

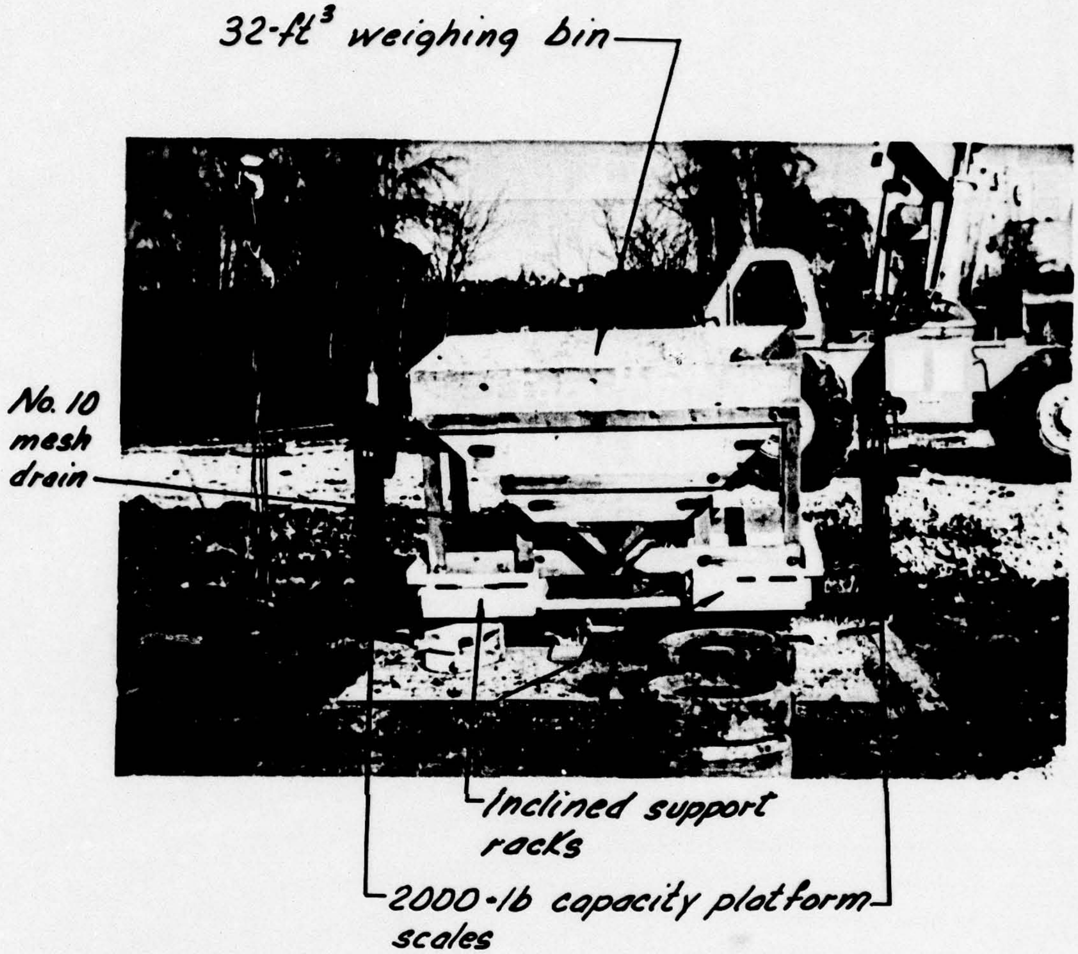
DOWN-HOLE TOOLS USED DURING TEST PROGRAM

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0005

Woodward-Clyde Consultants

Fig. 2.8

Y7C825 Phase IX



DRILLED-IN PILE TEST PROGRAM

WEIGHING BIN AND SCALES

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

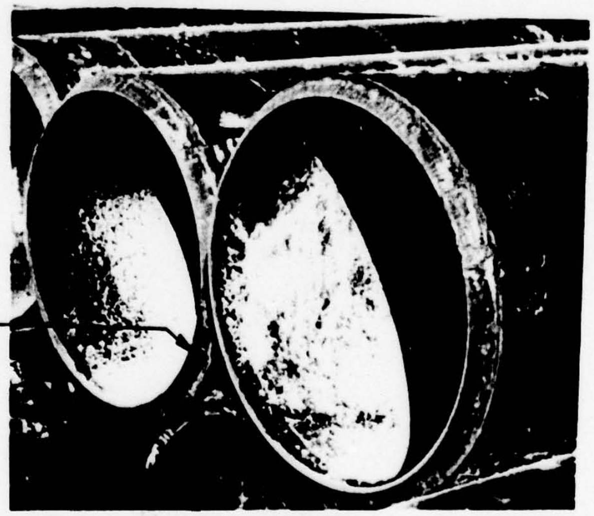
Y7C025 Phase IX

Fig. 2.9

Leading edge of casing

Square-cut top end of casing

45° bevel



Beveled end of casing

DRILLED-IN PILE TEST PROGRAM

20-IN.-DIAMETER STEEL CASING

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

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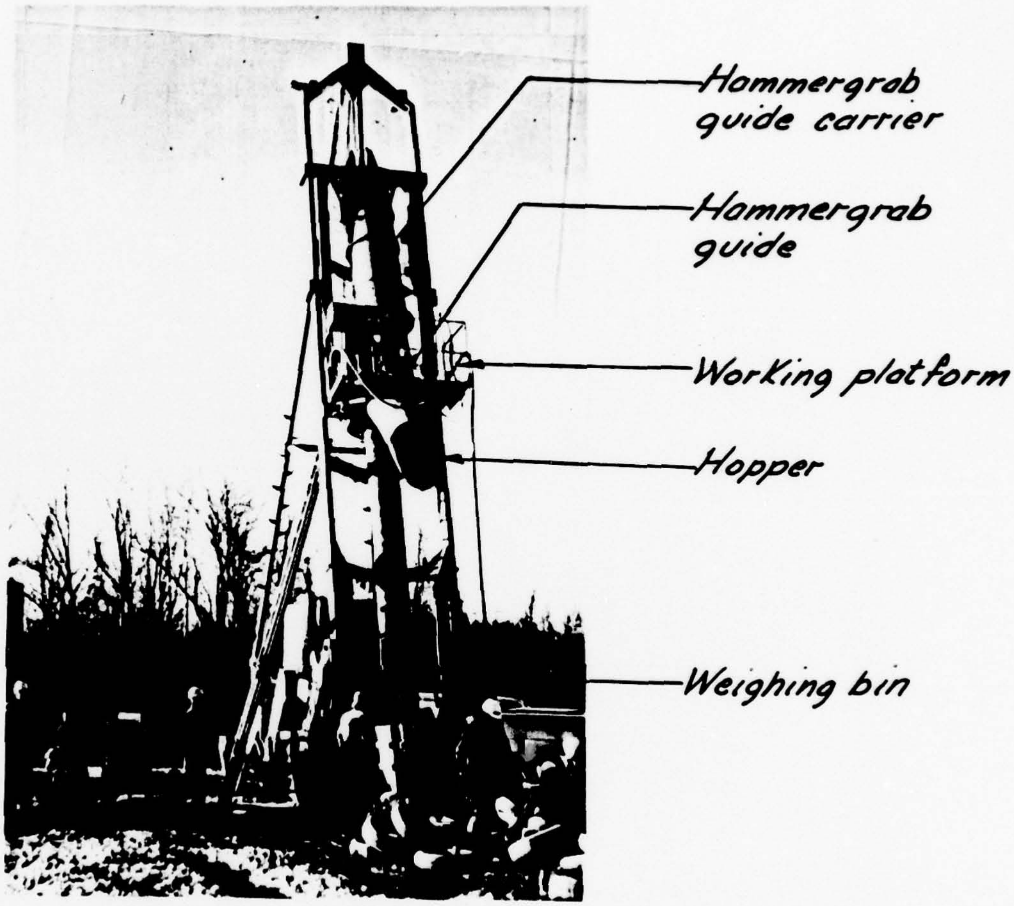
Y7C825 Phase IX

Fig. 2.10



DRILLED-IN PILE TEST PROGRAM

BENOTO MACHINE SET UP FOR DRILLING

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

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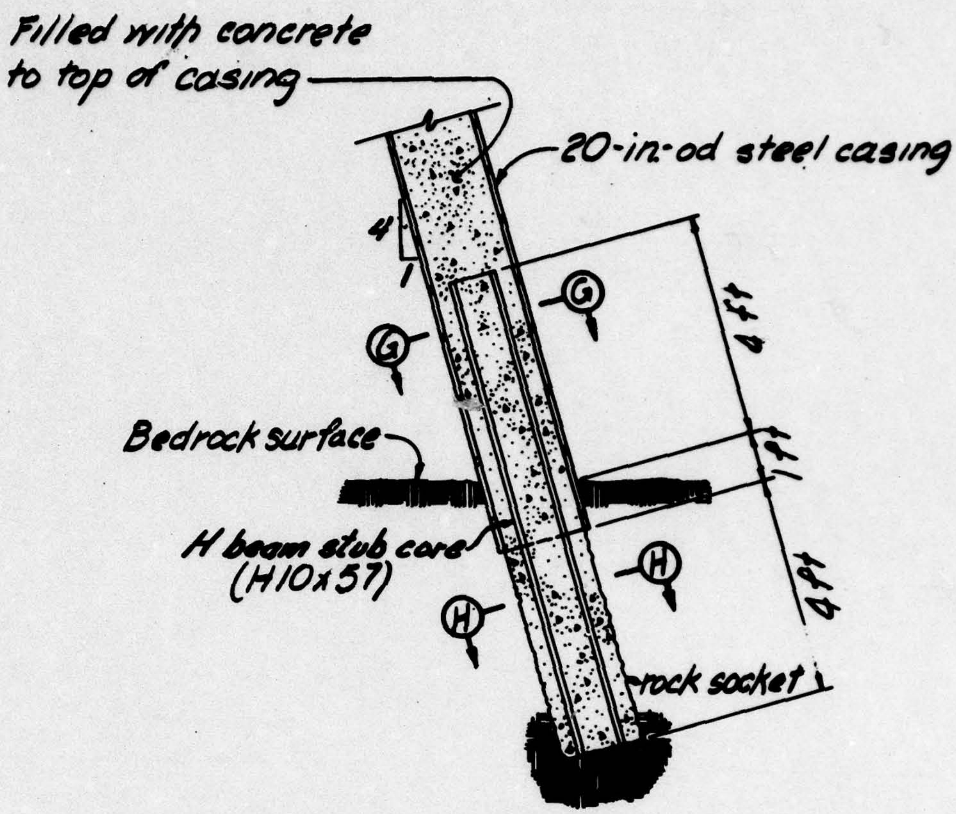
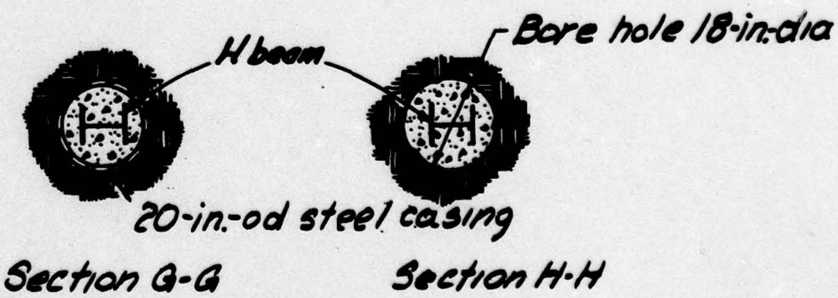
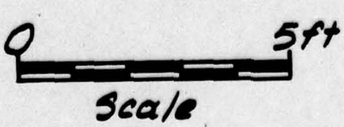
77C625 Phase IX

Fig.2.11







DRILLED-IN PILE TEST PROGRAM

ROCK SOCKET DETAIL

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT. CORPS OF ENGINEERS.
DACW43-78-C-0008

Woodward-Clyde Consultants

Y7C826 Phase IX

Fig. 2.12

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM

SECTION 3

TEST AREA SUBSURFACE CONDITIONS

3 TEST AREA SUBSURFACE CONDITIONS

3.1 TEST AREA SELECTION

At the conclusion of the Phase II subsurface investigation program, the area for the drilled-in pile test program was selected to be in the northeast corner of the main test area. The main test area was located near the downstream end of Ellis Island. The subsurface conditions were evaluated during the Phase III field exploration program and they were found to suitably represent the alluvial and glacial profile under Locks and Dam No. 26. Specifically, glacial till was not found overlying bedrock at the selected location; glacial till is not present under the dam, but was found under other candidate test area locations.

3.2 SUBSURFACE INVESTIGATIONS

3.2.1 Purpose and Scope

Subsurface investigations for the drilled-in pile test were performed during the Phase II, III, and IV field exploration programs to locate a suitable site and measure the properties of the natural soil prior to pile installation. These properties served as a basis for comparison with future measurements made after the piles were installed. The program of borings, sampling, and in situ and laboratory testing undertaken during these investigations in summarized below.

Boring	Date	Purpose
C-4, 7	Jan 1978	Phase II confirmation of Ellis Island as test site
S-15, 16	Jan 1978	Phase II location and confirmation of individual test areas
D-1, 2	Feb 1978	Phase III subsurface investigation of drilled-in pile test area
DP-C1, PM1, PM2 PM3, D1, 3D4A	Sept-Oct 1978	Phase IV subsurface investigation and in situ testing for piles DP2 and DP3

Phase IV in situ testing was performed in close proximity to the anticipated location of piles DP-2 and DP-3. This testing was part of a program to measure the influence of pile installation on adjacent soil. To limit the number of boreholes and minimize disturbance to the soil through which the piles were to be installed, boreholes for ground instrumentation installation were also used for the in situ tests.

Boreholes were drilled at locations shown in Fig. 3.1. In situ testing performed in these boreholes included dynamic and static penetration tests, pressuremeter tests, and nuclear density measurements. Grain-size analyses and natural water content measurements were made on disturbed samples. Laboratory maximum-minimum unit weight determinations and triaxial compression tests were performed on undisturbed samples. Triaxial compression tests were also performed on laboratory reconstituted samples.

3.2.2 Borehole Sampling

Undisturbed and disturbed borehole samples were obtained for laboratory index and strength property determination. Undisturbed samples were obtained in borings S-15, C-4, D-1, D-2, and DP-D1 using Osterberg, Hvorslev, and Pitcher samples. The Osterberg and Hvorslev samplers generally resulted in sample recoveries of 65 to 100 percent in the upper alluvium. Below el 355 to el 357, where the glacial deposits were encountered, these piston samplers could not penetrate the coarse-grained deposits without damaging the sampling tube. In the deeper deposits, a Pitcher sampler was used; the sample recovery was only 10 to 50 percent. Disturbed soil samples were used for laboratory grain-size analyses and natural water content determinations.

3.2.3 Dynamic Penetration Testing

These tests consisted of driving a split spoon into the soil. Two split spoons were used: a 2-in.-od spoon driven with a 140-lb hammer falling 30-in. (standard penetration test, ASTM D1586-67), and a 3-in.-od spoon driven with a 350-lb hammer falling 18 in. (a procedure commonly used by the St Louis District in alluvial deposits). The dynamic penetration resistance was recorded as the number of hammer blows, N or N_3, required to drive the 2-in. or 3-in. spoon, respectively, 12 in. into the soil.

Standard penetration tests were made in borings DP-PM1 and DP-PM3. Standard penetration resistance is a commonly used index to engineering properties, and it was correlated with static cone and pressuremeter test results performed in borings DP-C1, DP-PM1, and DP-PM3, respectively. Dynamic penetration tests using the 3-in. spoon were made in borings C-7, C-4, S-15, S-16, D-1, D-2, and DP-D1. The non-standard larger spoon was used to provide a correlation with prior borings performed by the St Louis District. This spoon also provides a larger sample which may be more representative of the actual subsurface materials in the glacial deposits which frequently included coarse gravel.

The results of dynamic penetration tests performed in Phase IV before pile installation are shown in Fig. 3.2. Above el 348, N_3-values fall at the low end of the range of N-values obtained in borings DP-PM1 and DP-PM3. This is consistent with previous results obtained at or near the site. Below el 348, N_3-values exceed or equal the corresponding N-values. The high resistance at el 345 is due to local stratigraphic differences, but apparently in the deeper glacial deposits, the two spoons give similar resistances.

3.2.4 Static Cone Penetration Testing

Continuous static cone penetration tests were made in boring DP-C1. The cone has a 10-cm^2 cross-sectional area, and an angle of 60 degrees. The load applied on the cone to push it into the soil at a constant rate of penetration of 4 ft/min was measured by a load cell and was recorded on a strip chart. Continuous cone penetration profiles were obtained by alternately pushing the cone 5 ft to 10 ft into the soil at the bottom of the borehole, and reaming the borehole after each cone run by rotary drilling.

The results of the static cone sounding made before pile installation are shown in Fig. 3.2, together with the dynamic penetration resistances. The correlation between cone penetration resistance, q_c, and standard penetration resistance, N, is presented in Fig. 3.3. This correlation can be approximated by:

$$q_c \ (t/ft^2) = 4.4 \ N \ (blow/ft) \text{ above el } 380$$
$$q_c \ (t/ft^2) = 6.3 \ N \ (blow/ft) \text{ below el } 380$$

A q_c/N factor of 4 is consistent with published correlations for fine to medium sand, and a q_c/N factor of 6 is consistent for a coarser sand with gravel.

3.2.5 Pressuremeter Testing

Pressuremeter tests were made in two borings (DP-PM1 and DP-PM3). The Menard type GAm pressuremeter used for the measurements consisted of a BX-size probe that was expanded at the bottom of the borehole. The volume change of the probe was measured as a function of the applied pressure. The data was used to obtain elastic and plastic characteristics of the soil. The boreholes were carefully prepared by slow drilling with a drag bit and thick bentonite drilling fluid.

An idealized pressuremeter volume-change vs applied pressure curve is shown in Fig. 3.4. At the beginning of the test, the probe begins to expand through the drilling fluid with little lateral restraint, until it makes contact with the borehole walls. This corresponds to the steep initial portion of the volume change curve. As the probe continues to expand, the soil resistance is mobilized and the volume change curve is linear (pseudo-elastic response). At higher pressure, plastic deformation occurs. The soil then sustains large deformations for small pressure increases. The asymptote of the volume change vs pressure curve corresponds to the ultimate strength of the soil (or limit pressure).

The results of actual pressuremeter tests performed in borings DP-PM1 and DP-PM2 are presented in Appendix A, Volume IVA. Bentonite stabilizing fluid was used in these borings because of the long time during which the hole had to remain open to complete the large scope of testing and instrument installation in each borehole. Use of bentonite increased the level of disturbance in these tests as compared to higher quality tests performed in the chemical grouting and pile driving effects test programs, where Revert stabilizing fluid was used. This disturbance increases measured horizontal stresses and decreases soil stiffness and strength.

3.2.6 Nuclear Density Measurements

Nuclear density measurements were performed in boring DP-D1 using a Gearhart-Owen high-resolution gamma density probe serial No. 5650 which uses two Americium sources and a scintillation crystal receiver. Roberts Geophysical Services made the measurements in a 6-in.-dia borehole advanced by rotary drilling with bentonite drilling fluid. Three-inch split-spoon samples were taken at 5-ft depth intervals to rock. Visual classification and natural moisture content determination were performed on each sample. The unit weights of three undisturbed Hvorslev samples taken in the upper 70 ft of the boring were also determined. The precise diameter of the borehole was measured by a continuous electronic caliper sounding. The caliper located washouts and voids in the borehole walls which would yield erroneous unit weight readings. The density probe was lowered to the bottom of the borehole after the caliper sounding, and then raised at a speed of 2 ft/min. The output of the probe provided a continuous analog recording of total unit weight vs depth. The probe was calibrated with two blocks of known density, one plastic and one aluminum.

Results of nuclear density measurements yielded total unit weights somewhat higher than known in situ unit weights measured or calculated by other methods. Comparison with the undisturbed sample unit weights and relative densities determined from static cone penetration tests correlated with laboratory maximum-minimum unit weights tests indicated that the nuclear density measurements were high by a constant factor of 20 percent. Results of striking precision and resolution were obtained, however. These results will be discussed further in Section 3.4.3. Future use of this in situ test should be accompanied by calibration measurements on site-specific soils compacted to various known densities to increase measurement accuracy.

3.2.7 Laboratory Testing

Grain-size analyses were made on disturbed split-spoon samples and undisturbed Osterberg and Pitcher samples obtained in borings C-4, C-7, S-15, S-17, D-1, and D-2. Grain-size distribution curves for these samples are presented in Appendix A, Volume IVA.

Maximum-minimum unit weight determinations were made in the laboratory on undisturbed Osterberg and Pitcher samples obtained in borings C-4, S-15, D-1, and D-2. The maximum unit weight was determined by the Modified Providence Method using an electromagnetic jack hammer instead of the standard ASTM ballpeen hammer. Minimum unit weights were determined using the tube method developed by Lucks (1970), funnel method in 0.1 ft^3 mold (ASTM), and small 432 cm^3 mold and cylinder tilt method (Kolbuszewski 1948). Test results are discussed in Section 3.4.3.

Consolidated-drained and consolidated-undrained triaxial compression tests were performed on undisturbed Osterberg samples from borings D-1 and D-2. Confining stresses of 1, 2, and 4 t/ft^2 were used for each test series, and the samples were loaded at a strain rate of 0.15 percent per minute. Test results are presented in Appendix A, Volume IVA. A consolidated-drained triaxial compression test series was also performed on laboratory reconstituted specimens obtained using the 3-in. split spoon. The samples were reconstituted to a relative density of 70 percent as is typically found in situ. Results of this triaxial test series are presented in Appendix A, Volume IVA.

3.3 STRATIGRAPHY

3.3.1 General Geology

The test site is located within the Mississippi River flood plain near Alton, Illinois, at the southwestern edge of the central lowland physiographic province. In the vicinity of Locks and Dam No. 26, the flood plain surface is generally flat at el 410 to el 420.

In the vicinity of Locks and Dam No. 26, bedrock rises uniformly from el 270 on the Missouri side to el 330 on the Illinois side. The bedrock is overlain by soil deposits of glacial, alluvial, and colluvial origin. Five major soil strata and one bedrock unit have been identified; various stratigraphic units were inferred within some of the major soil strata. The inferred subsurface profile A-A along the centerline of piles DP-2 and DP-3 is shown in Fig. 3.5. The soil strata are, in decending order, flood plain deposits, recent alluvium, alluvial outwash (reworked alluvium), Wisconsinan outwash, and Illinoian ice contact deposits. Occasionally, glacial till pockets are intercalated between the ice contact deposits and bedrock surface. Till was not present at the test area location. The rock units underlying the soil consist of Mississippian limestone of the Meramecian (Valmeyer) Series. The upper formation is the St Genevieve. The following is a description of each soil stratum and rock unit.

3.3.2 Flood Plain Deposits

Flood plain deposits consist primarily of high to low plasticity clay, with varying amounts of silt, fine sand, and organic material. The source of the flood plain deposits is the active and abandoned channels, back swamps, and flood basin areas of the Mississippi River flood plain. The river materials, mainly silt and clay, are deposited in relatively quiescent waters. New material is carried in during river flood stages, or forms as colluvial deposits.

Unit A1 (Fig. 3.5) is a 5- to 7-ft-thick layer of fill placed during site preparation activities. The fill was borrowed outside the drilled-in pile test area from unit A2. The fill was placed in relatively thick layers and compacted only by the movement of the scrapers during placement. This unit consolidated under overburden weight throughout the duration of the tests. Unit A2 (Fig. 3.5) is the natural flood plain deposit. The unit is a soft to firm, gray clayey silt, with a trace of fine sand (ML-CL).

3.3.3 Recent Alluvium

The recent alluvium originated during aggrading and meandering of the Mississippi River across its flood plain during post-Wisconsin (Holocene to Recent) time. The recent alluvium is a relatively uniform deposit because of common depositional environment and history, and because of the large scale of the Mississippi-Missouri fluvial system. It is uniform in such characteristics as depositional structures, and abundance and distribution of carbonaceous material. The recent alluvium ranges from coarse silt to fine gravel, but is predominantly fine to medium sand. These sediments are characteristically clean, well-sorted (poorly graded) sand composed of at least 70 percent, and frequently more than 80 percent, quartz grains. They also contain abundant concentrations of carbonaceous material including wood, charcoal, and lignite, which range in size from coarse silt to large tree trunks.

Five minor stratigraphic units were inferred within the recent alluvium (Fig. 3.5). Unit B consists of a loose to medium dense, brown to gray, fine to coarse sand with a trace of silt and a trace of fine gravel (SP). Unit C is more uniform than Unit B, and is predominantly fine to medium sand (SP). Unit D is similar to Unit C, except that it is better graded, coarser-grained, and contains local concentrations of fine gravel. Unit E1 is a thin layer or pocket of firm, gray, silty clay with a trace of fine sand. Unit E1 is typically underlain by a silty fine sand layer, Unit E2. The deepest unit (Unit F) identified in the recent alluvium is a medium dense, gray, fine to coarse sand with thin clay seams and a trace of fine gravel (SP).

3.3.4 Alluvial Outwash

The alluvial outwash consists of coarse- to fine-grained, poorly graded sand, with some silty sand and gravel zones. The alluvial outwash is considered to be an intergrading of recent alluvium and the underlying Wisconsinan outwash. The deposits may have formed contemporaneously with Wisconsin glaciation. The major portion of this alluvial outwash deposit, however, is believed to have formed during the in situ reworking of glacial outwash in post-Wisconsin to Recent time. Variations in stream flow, channel form and width, and obstructions led to renewed scouring of previously deposited sediment (Wisconsinan outwash) and redeposition elsewhere.

3.3.5 Wisconsinan Outwash

The Wisconsinan outwash consists of coarse to medium grained, poorly graded sand, silty sand, and gravel. The Wisconsinan outwash was deposited in the Mississippi Valley during the Wisconsin glacial advance into areas west, north and east of the St Louis area. Major streams that carried outwash material included the Illinois and Missouri Rivers, as well as the Mississippi River.

Three minor stratigraphic units were inferred within the Wisconsinan outwash (Fig. 3.5). At the top of the Wisconsinan outwash deposit, a 3-in.-thick layer (Unit H) of medium-dense, gray-black, silty fine sand (SM) was encountered. This layer was deposited during a quiescent period during the alluvial or glacial depositions. This layer was not generally encountered in the other test areas. Unit I is a very dense, gray, fine sand with a trace of silt (SP). This was the densest and most uniform stratum encountered in the drilled-in pile test area subsurface profile. Significant concentrations of black basaltic minerals were also observed in the sand particle matrix. Finally, Unit J is a medium -dense to dense, gray, fine to coarse sand with a trace of silt and fine gravel (SP). This unit is poorly graded and contains subrounded to subangular particles.

3.3.6 Illinoian Ice Contact Deposits

The Illinoian ice contact deposits (Unit K) consist generally of fine-to coarse-grained, poorly graded sand with numerous boulder, cobble, gravel, and occasional silty sand zones. The ice contact deposits are generally dense. The ice contact deposits formed immediately adjacent to the glacial ice front, resulting in an extremely variable particle size. The deposits are discontinuous and may not be undisturbed glacial deposits. Large particles from upstream glacial materials, along with alluvium, may have been placed in some areas as channel lag deposits. Till-like material (flow till) resulting from superglacial mud or debris flows are often found in ice contact deposits. Large fragments of chert were often encountered toward the bottom of Unit K.

3.3.7 Limestone: St Genevieve Formation

The St Genevieve formation is a light-colored, sandy, oolitic, cross-bedded calcarenite. The upper beds are often separted by thin, shaley limestone and thin shale partings. Exposed on the bluffs north of Alton, portions of the St Genevieve massive cross-bedded oolite are abruptly replaced by thin-bedded shaley or slabby oolitic limestone. Small faults and some solution activity have been noted in this limestone on the bluffs.

3.4 INITIAL SOIL PROPERTIES

3.4.1 Grain-Size Distribution

The laboratory grain-size curves were evaluated in light of the stratigraphic profile information to develop ranges of grain-size distribution for each sand unit. Grain-size distribution ranges for units A2 through D, E2 through G, and I through K are given in Fig. 3.6.

3.4.2 Stresses

The in situ state of stress was evaluated from pressuremeter, density test results, and field groundwater table observations. The horizontal total stress was measured during pressuremeter testing as the cell pressure at which the undisturbed elastic resistance of the soil was mobilized; that is, the stress at which the pressure-volume change curve becomes linear. The horizontal effective stress was obtained by subtracting the static pore pressure because the tests are assumed to have been fully drained. The inferred horizontal effective stress profile before pile installation from borings DP-PM1 and DP-PM3 is shown in Fig. 3.7. Superimposed on these measured stresses is a profile of stresses which correspond to certain values of K_o. The coefficient of earth pressure at rest, K_o, is the ratio of horizontal to vertical effective stress.

Results from borings DP-PM1 and DP-PM3 indicated a relatively large horizontal effective stress in the upper 80 ft of the subsurface profile (K_o = 0.9), implying some overconsolidation. This overconsolidation is unlikely in light of the geologic history of these deposits. The results of boring DP2-PM1 which was performed after pile installation, but was relatively unaffected by the installation (Section 5.2), showed a smaller horizontal stress with K_o ranging from 0.4 to 0.7, indicating that the deposit was primarily normally consolidated. A possible explanation for the large horizontal stresses measured in borings DP-PM1 and DP-PM3 may be the hole preparation method. Bentonite drilling fluid was used to keep the hole open for the long period of time required for testing and ground instrumentation installation. Revert was used in boring DP2-PM1 and has been shown in the chemical grouting tests (Volume II) and pile driving effects tests (Volume III) to produce a lower degree of disturbance. This distubance masked the start of the undisturbed elastic response during the pressuremeter tests and thereby yielded higher horizontal stresses. The level of disturbance was, however, relatively small as the remainder of the tests below el 340 yielded reasonable results.

3.4.3 Density

A profile of total unit weight with depth was measured in boring DP-D1 using a nuclear density probe. The natural water content of the soil was determined from disturbed split-spoon samples obtained at 5-ft depth intervals in this boring. From these measurements, a dry unit weight profile with depth was developed. This profile is shown on Fig. 3.8. As mentioned in Section 3.2.6, measured total weights had to be reduced by 20 percent to match results of measurements made on undisturbed samples; these undisturbed sample unit weights are also shown on Fig. 3.8. The unit weight profile is discontinuous because erroneous readings corresponding to borehole washouts were deleted. Many of the peaks in the Illinoian ice contact deposit correspond to cobbles and boulders.

Maximum and minimum dry unit weights determined in the laboratory are presented in Fig. 3.9. There is some scatter in the results from different samples and borings because of differences in stratigraphy and sample quality, and because of the inherent variability in the testing procedures.

Relative density profiles were determined from the results of static cone and standard penetration tests. Relative density was calculated from static cone penetration point resistance using an empirical correlation (Schmertmann 1976) established with an electrical cone (similar to that used here) in normally consolidated fine to medium sand (SP). The correlation takes into account the effect of vertical effective overburden stress. The relative density profile for boring DP-C1 is shown in Fig. 3.12. Standard penetration resistances were related to relative density using the Gibbs and Holtz' (1953) correlation. The Gibbs and Holtz' correlation was chosen because the sands tested in that study were similar to Ellis Island sand. Relative densities calculated from N-values for borings DP-PM1 and DP-PM3 are plotted on Fig. 3.10. Relative density can be calculated from the in situ and laboratory maximum-minimum unit weights using:

$$D_r = \frac{\gamma_{d\,max}}{\gamma_d} \left(\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right) \times 100$$

where: D_r = relative density, percent

$\gamma_{d\,max}$ = maximum dry unit weight;

$\gamma_{d\,min}$ = minimum dry unit weight; and

γ_d = dry unit weight.

Relative densities calculated from the data in Fig. 3.8 and 3.9 are plotted in Fig. 3.10. Relative densities determined using static cone and standard penetration resistances, and direct measurement agreed well.

3.4.4 Stiffness

Soil stiffness is characterized by an elastic deformation modulus E_s which relates the stress-strain response up to a stress level where the shear strength of the soil is exceeded. The elastic deformation modulus (Young's modulus) was inferred from the results of static cone penetration tests, pressuremeter tests, and laboratory consolidated-drained triaxial compression tests. Each of these moduli represent a drained modulus; however, the strain amplitude and plane of deformation was different for each test.

Static Cone Modulus. Elastic deformation modulus values were determined from the static cone penetration tests using a empirical correlation first suggested by Vesic (1970):

$$E_s = 2\,(1 + D_r^2)\,q_c$$

where: D_r = relative density; and

q_c = cone penetration resistance.

There are many other correlations, primarily derived from plate load tests in various types of sands, that give similar results. This modulus is, therefore, representative of three-dimensional (deviatoric) compression. Modulus values calculated from static cone penetration tests in borings DP-C1 are presented in Fig. 3.11.

Pressuremeter Modulus. Elastic deformation modulus values were calculated from pressuremeter tests using the slope of the linear pseudo-elastic portion of the pressure-volume change curve. An equation for cylindrical cavity expansion of a linearly elastic material under conditions of axial symmetry and plane strain was used:

$$E_s = 2V_o (1 + \nu) \Delta P/\Delta V$$

where: V_o = initial volume of measuring cell;
ν = Poisson's ratio;
ΔP = pressure increment; and
ΔV = volume increment resulting from ΔP.

Modulus values calculated from pressuremeter tests in borings DP-PM1 and DP-PM3 are also presented in Fig. 3.11.

Laboratory Modulus. Elastic deformation modulus values were calculated from laboratory $\overline{\text{CID}}$ triaxial compression tests on undisturbed borehole samples and laboratory reconstituted samples. Initial tangent moduli and secant moduli at peak deviator stress (failure) were obtained from stress-strain curves. These moduli are plotted on Fig. 3.11 at a depth corresponding to that where the sample was obtained.

The static cone resistance yields modulus values one to three times larger than the pressuremeter derived moduli. The pressuremeter modulus is a measure of horizontal properties, whereas the cone applies its load in a vertical plane. Anisotropy may account for some difference in modulus values derived from the two in situ tests. Laboratory triaxial secant moduli at failure appear to match the pressuremeter moduli, whereas cone moduli approximate the laboratory initial tangent modulus values. The pressuremeter may, therefore, be measuring a secant modulus and the cone modulus may reprsent an initial tangent modulus. Differences in sample size, strain amplitude, stress duration, and disturbance would also affect measured soil stiffness.

3.4.5 Shear Strength

The shear strength of the subsurface soil can be characterized by the Mohr-Coulomb failure criteria:

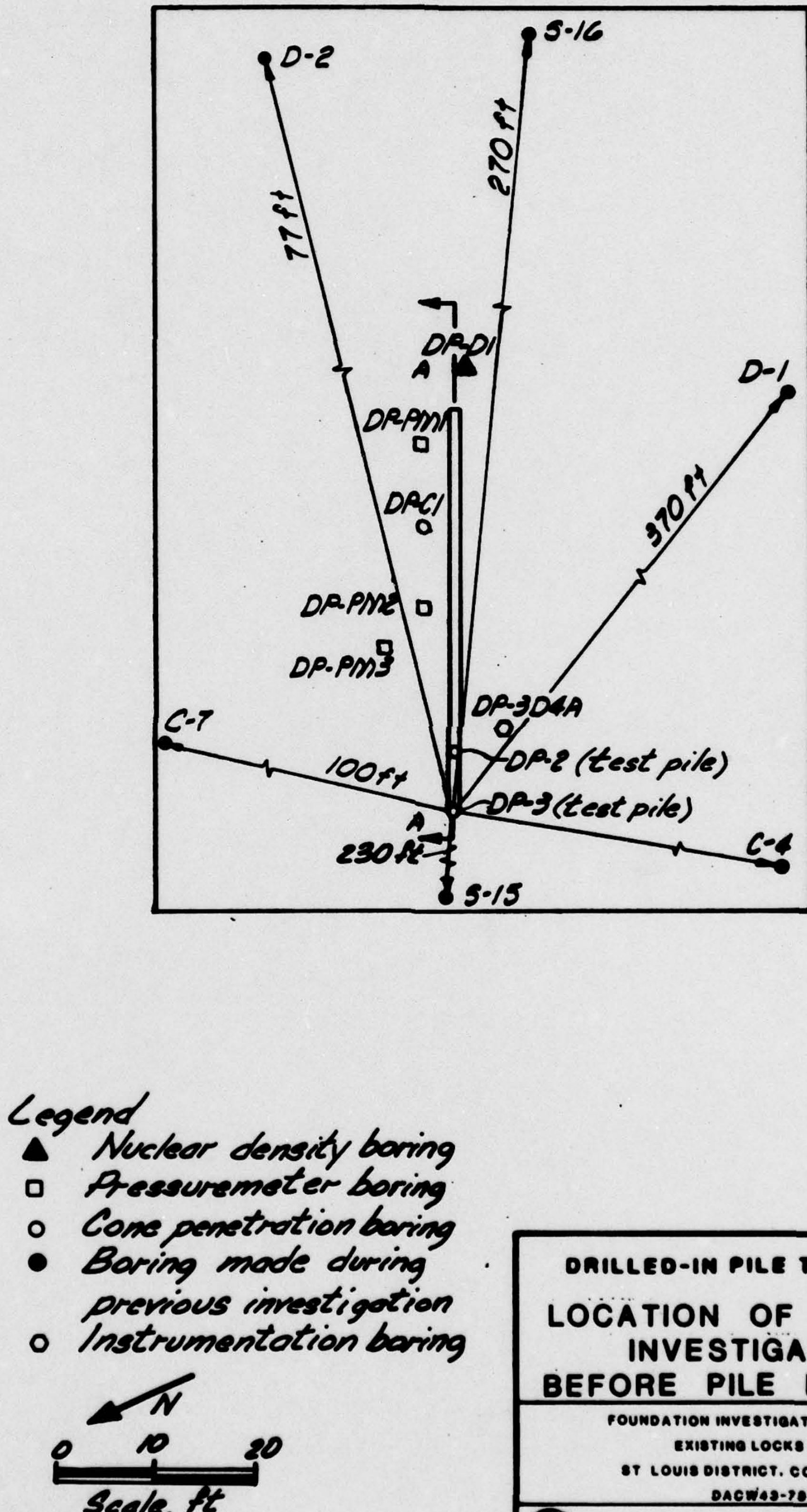
$$\tau_f = \bar{c} + \bar{\sigma}_f \tan\bar{\phi}$$

where: τ_f = shear strength;
$\bar{c}$ = cohesion;
$\bar{\sigma}_f$ = effective stress on failure plane at failure; and
$\bar{\phi}$ = angle of internal friction.

The laboratory triaxial compression tests provide these parameters as determined from the Mohr circles presented in Fig. A.31 through A.33, Appendix A, Volume IVA. These results indicate that the sand is cohesionless and develops its shear strength from the frictional component between sand grains. The drained angle of internal friction derived from these tests on undisturbed borehole samples and laboratory reconstituted sample is 39.5 degrees.

The drained angle of internal friction was determined from the results of static cone penetration and pressuremeter tests. Static cone penetration resistance were correlated to friction angle $\bar{\phi}$ using an empirical chart developed by Meyerhoff (1974). This correlation is independent of the in situ stress conditions. Friction angles calculated from the cone point resistance are plotted with depth in Fig. 3.12. The drained angle of internal friction was also determined from pressuremeter tests using a method developed by Hughes, Wroth, and Windle (1977). Friction angles calculated using this method are also plotted on Fig. 3.12.

In the sand, friction angles derived from static cone penetration tests ranged from 36 to 44 degrees with an average value of approximately 40 degrees. Friction angles determined from pressuremeter test results ranged from 37 to 44 degrees and averaged 39.5 degrees. The laboratory triaxial friction angle of 39.5 degrees confirmed the interpretation of the in situ testing results.



DRILLED-IN PILE TEST PROGRAM

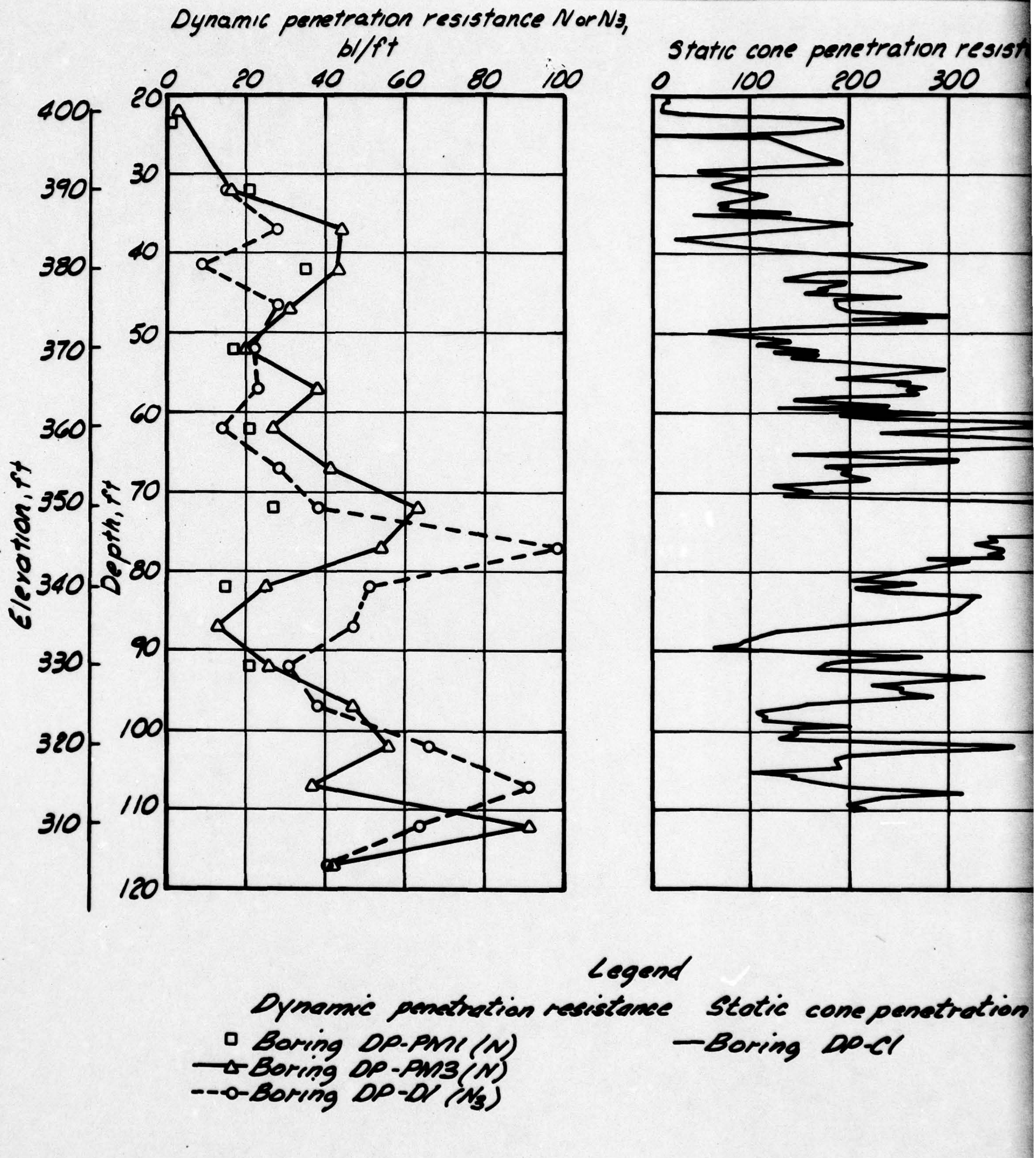
LOCATION OF SUBSURFACE INVESTIGATION BEFORE PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

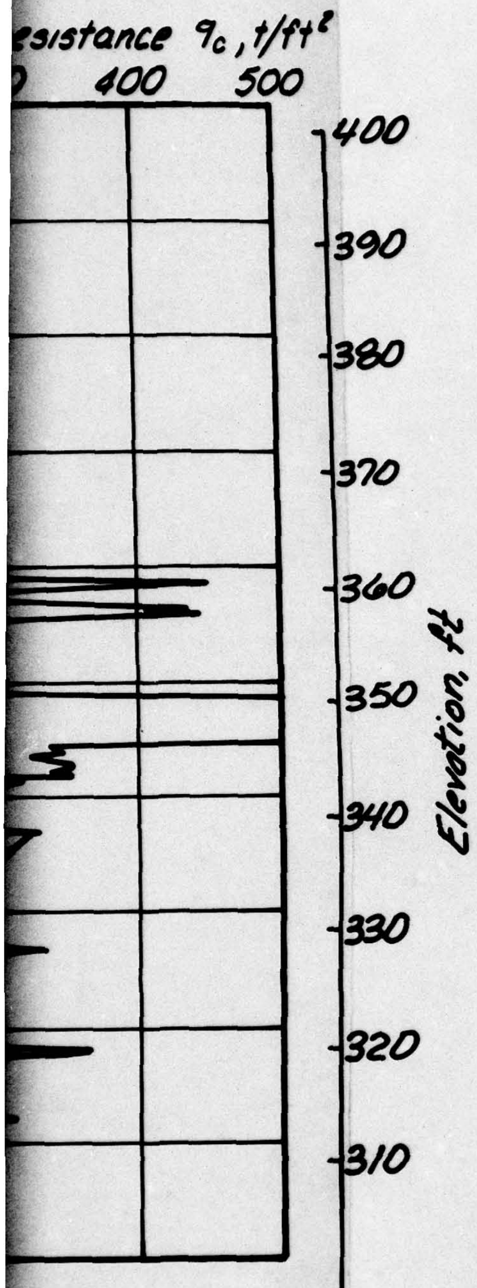
Woodward-Clyde Consultants

Y7C525 Phase IX

Fig. 3.1



Dynamic penetration resistance N or N3, bl/ft
0 20 40 60 80 100
Static cone penetration resist
0 100 200 300
Elevation, ft
400 390 380 370 360 350 340 330 320 310
Depth, ft
20 30 40 50 60 70 80 90 100 110 120
Legend
Dynamic penetration resistance
Boring DP-PM1 (N)
Boring DP-PM3 (N)
Boring DP-D1 (N3)
Static cone penetration
Boring DP-C1



ation resistance

DRILLED-IN PILE TEST PROGRAM

STATIC CONE AND DYNAMIC PENETRATION TEST RESULTS BEFORE PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
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DACW43-78-C-0006

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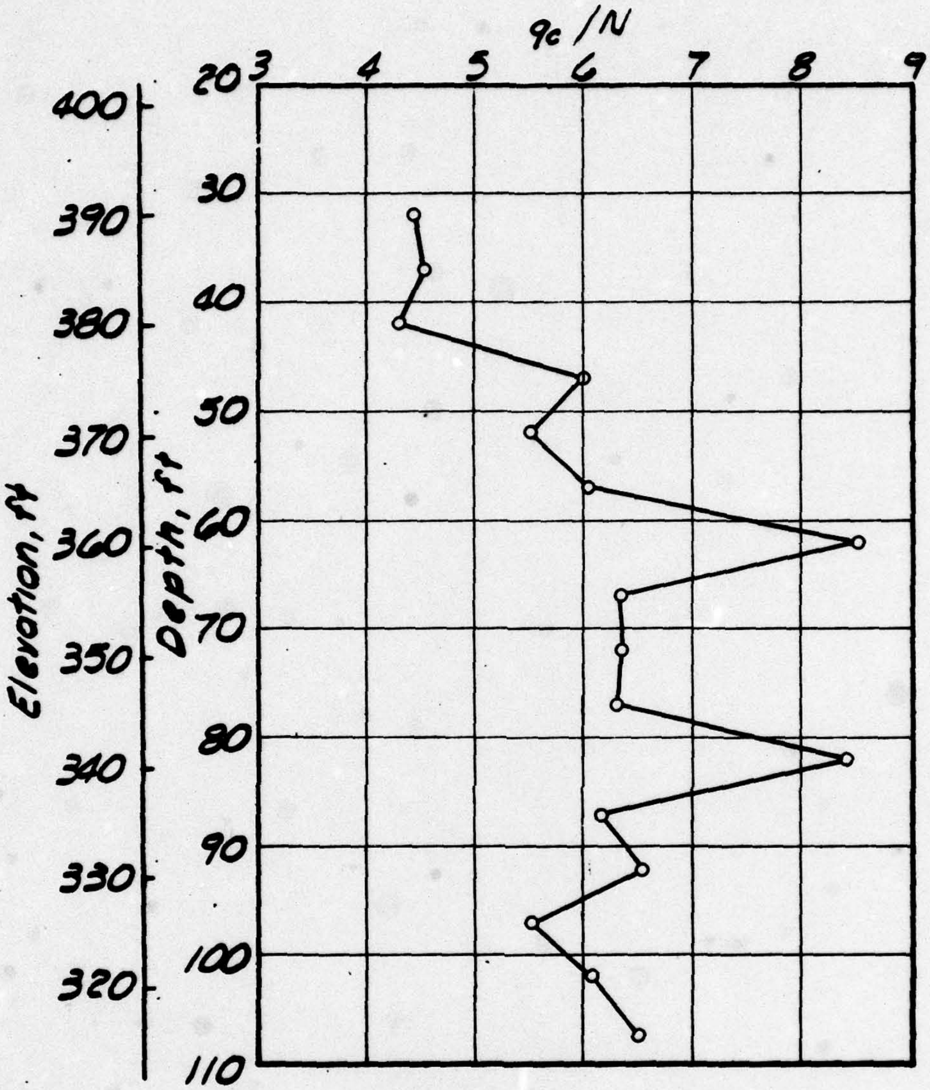
Y7C626 Phase II

Fig. 3.2



Note:
q_c = static cone penetration resistance, t/ft^2
N = standard penetration resistance, bl/ft

DRILLED-IN PILE TEST PROGRAM

CORRELATION BETWEEN STATIC CONE AND STANDARD PENETRATION RESISTANCE BEFORE PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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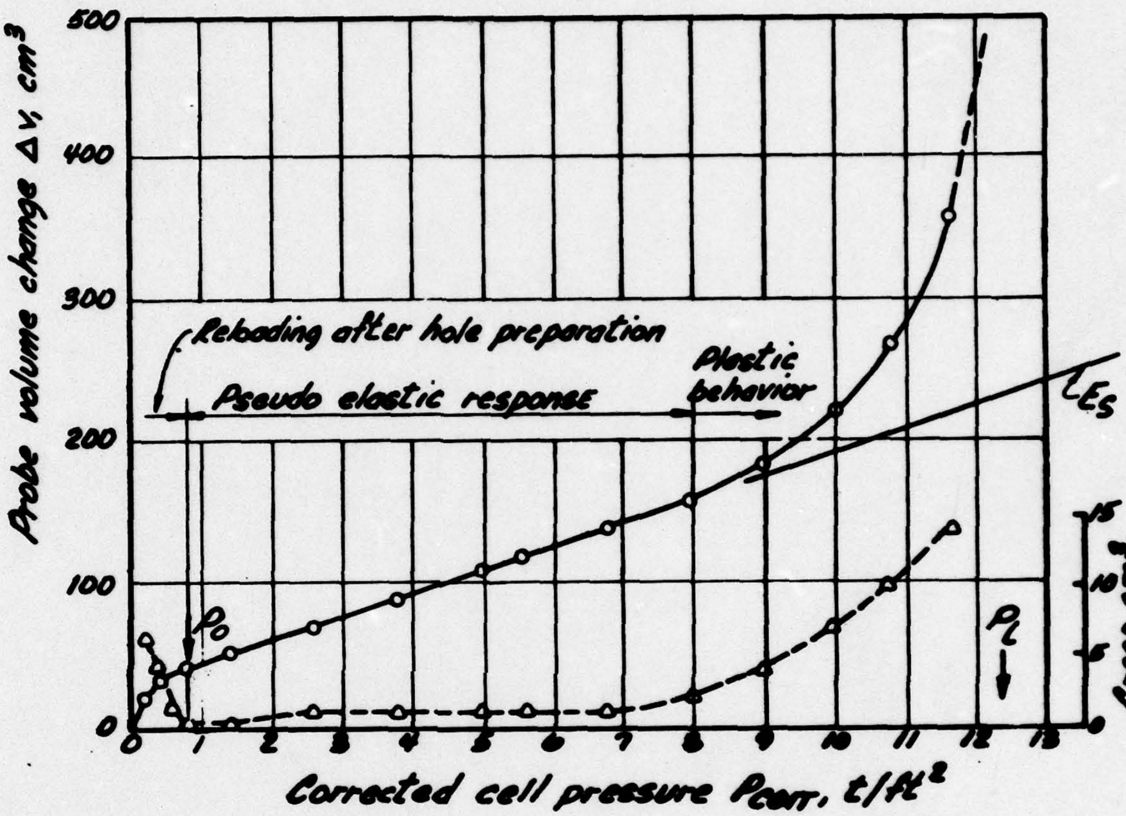
Y7C028 Phase III

Fig. 3.3



Legend

—o— Probe volume increase vs P_{corr}

--△-- Creep vs P_{corr}

P_o In situ horizontal stress

P_l Limit pressure

E_s Deformation modulus

DRILLED-IN PILE TEST PROGRAM

IDEALIZED PRESSUREMETER TEST RESULTS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-76-C-0065

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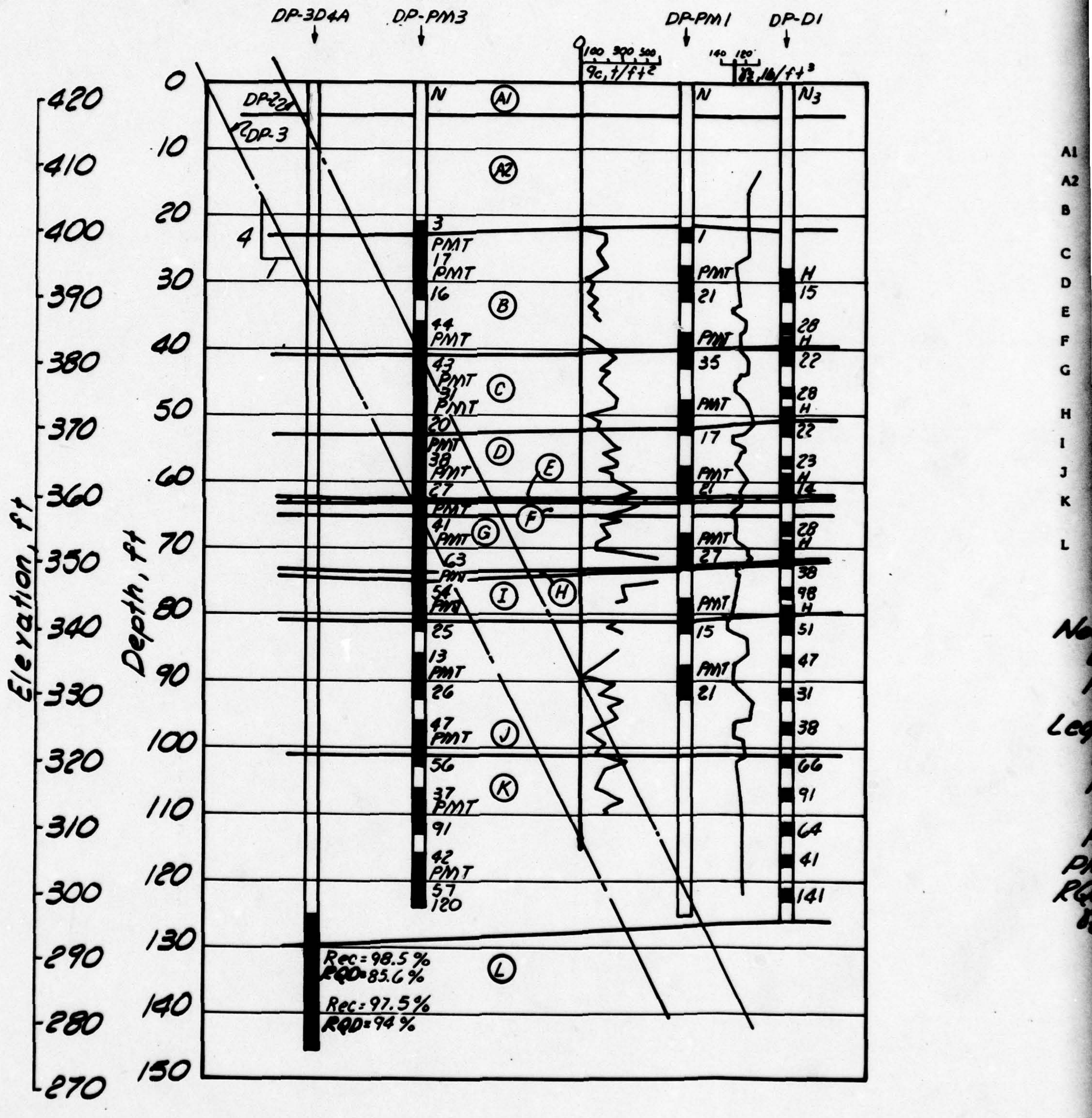
Y7C626 Phase II

Fig. 3.4



DP-3D4A
DP-PM3
DP-PM1
DP-D1
q_c, t/ft²
100 300 500
140 120
γ_d, lb/ft³
Elevation, ft
Depth, ft
N
N₃
DP-22
DP-3
A1
A2
B
C
D
E
F
G
H
I
J
K
L
PMT
Rec=98.5%
RQD=85.6%
Rec=97.5%
RQD=94%

Profile A-A

Drilled-In Pile Test Area

Subsurface Stratigraphy

A1 Soft grey clayey SILT, trace fine sand (ML-CL), FILL

A2 Soft to firm grey clayey SILT, trace fine sand (ML-CL), FLOODPLAIN

B Loose to medium dense brown to grey fine, fine to medium or fine to coarse SAND, trace silt, trace fine gravel (SP)

C Medium dense to dense grey fine to medium SAND, trace silt, trace fine gravel (SP)

D Medium dense grey fine to coarse SAND, trace silt, trace fine gravel (SP)

E Firm grey silty CLAY, trace fine sand (CL)

F Medium dense grey fine to coarse SAND with thin clay seams, trace fine gravel (SP)

RECENT ALLUVIUM (A2–F)

G Medium dense to dense grey fine to coarse SAND, trace silt, trace fine gravel, grading with occ. shell and lignite fragments (SP), ALLUVIAL OUTWASH

H Medium dense grey-black silty fine SAND (SM)

I Very dense grey fine SAND, trace silt (SP)

J Medium dense to dense grey fine to coarse SAND, trace silt, trace fine gravel (SP)

WISCONSINAN OUTWASH (H–J)

K Dense to very dense grey fine to coarse SAND, trace silt, trace fine to coarse gravel, occ. grading with cobbles and boulders (SP), ILLINOIAN ICE CONTACT DEPOSITS

L St Genevieve Limestone (LS)

Note

Location of profile A-A is shown in Fig. 3.1

Legend

N= Standard penetration resistance, blow/ft

N_3= Penetration resistance for 3-in spoon driven with 350 lb hammer, falling 18 in.

H= Hvorslev sampler

PMT= Pressuremeter test

RQD= Rock quality designation (Deere 1963)

γ_t = Total unit weight

DRILLED-IN PILE TEST PROGRAM

SUBSURFACE PROFILE
DRILLING EFFECTS TEST AREA

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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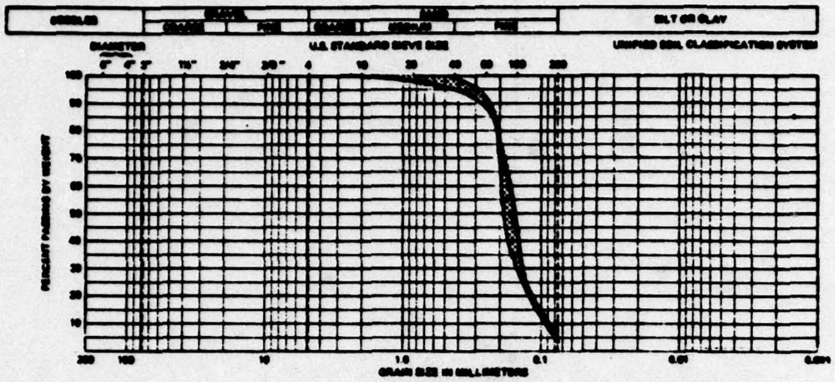
Fig. 3.5

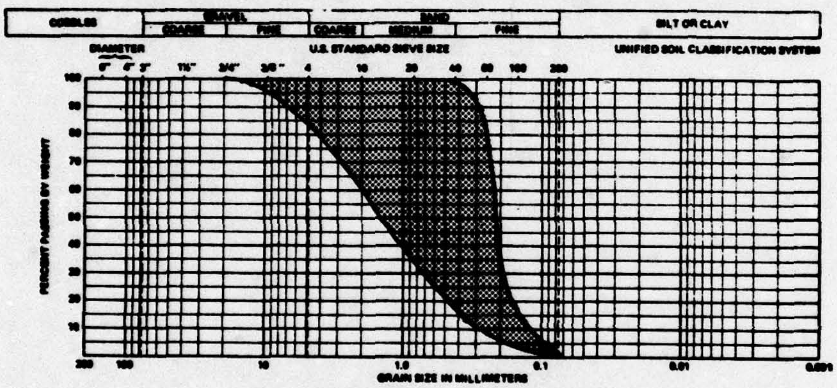
Y7C829 Phase IX

1

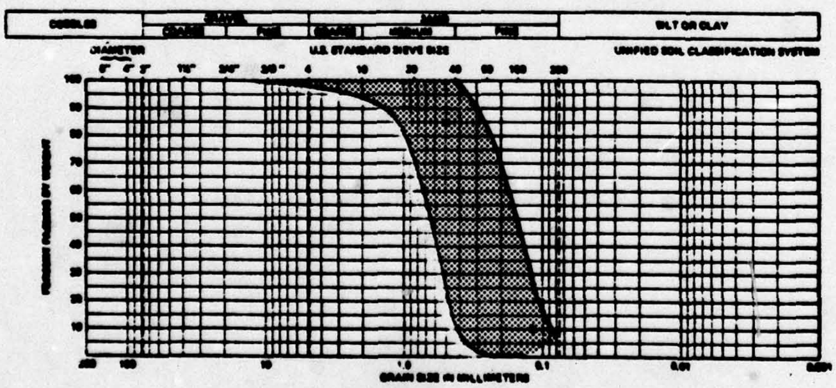
2



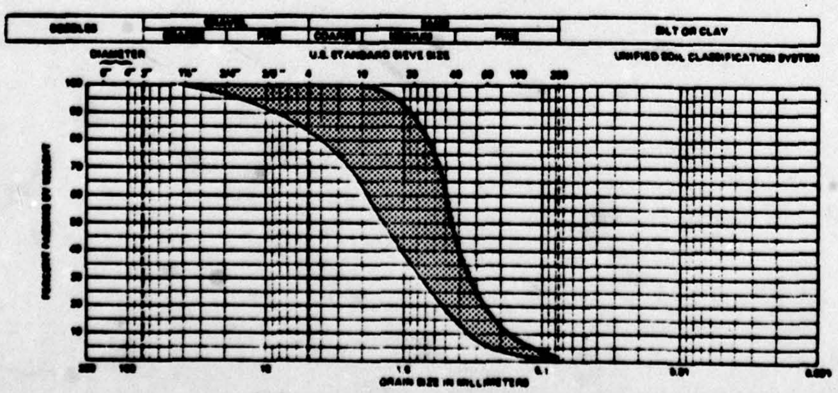
Sand seam within Stratigraphic unit A2



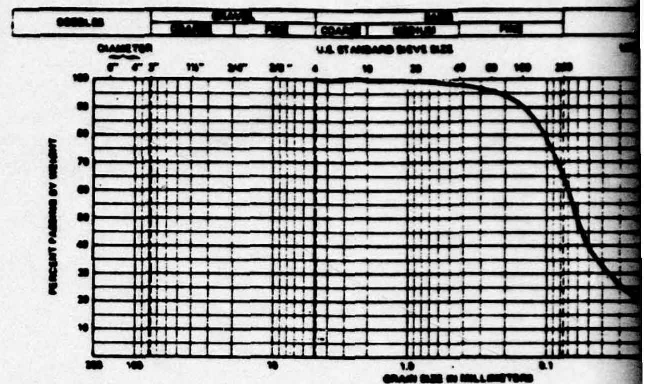
Stratigraphic unit B



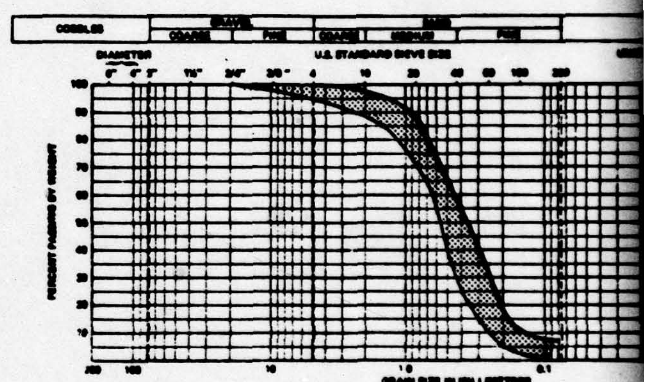
Stratigraphic unit C



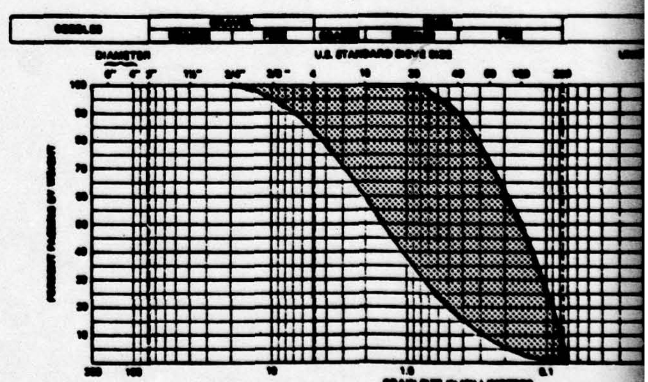
Stratigraphic unit D



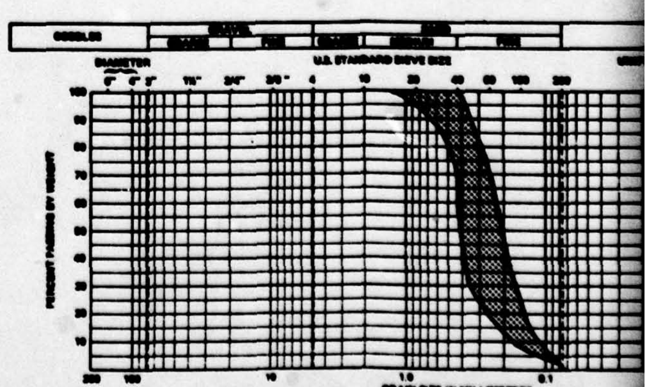
Stratigraphic unit E2



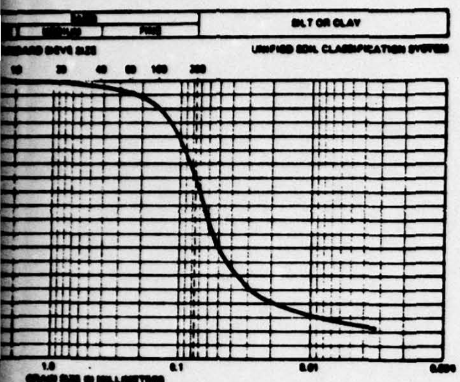
Stratigraphic unit F



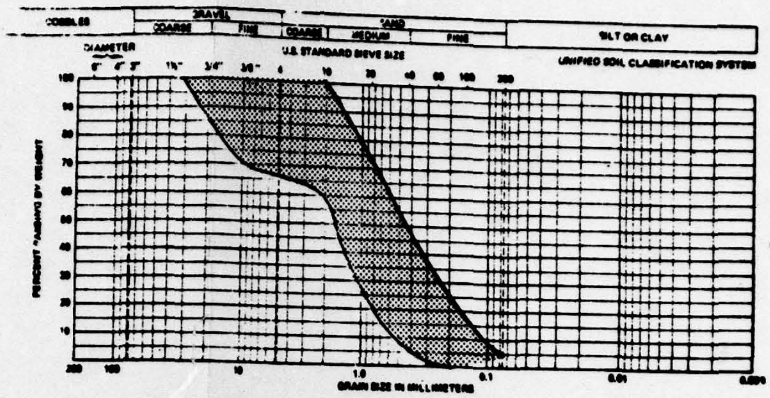
Stratigraphic unit G



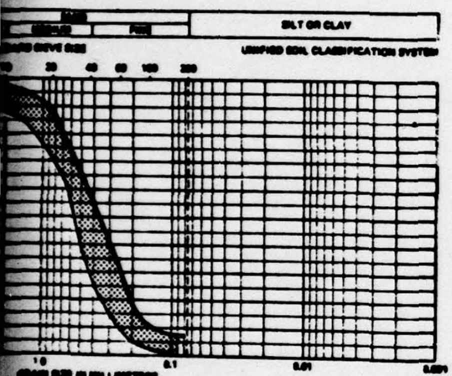
Stratigraphic unit I



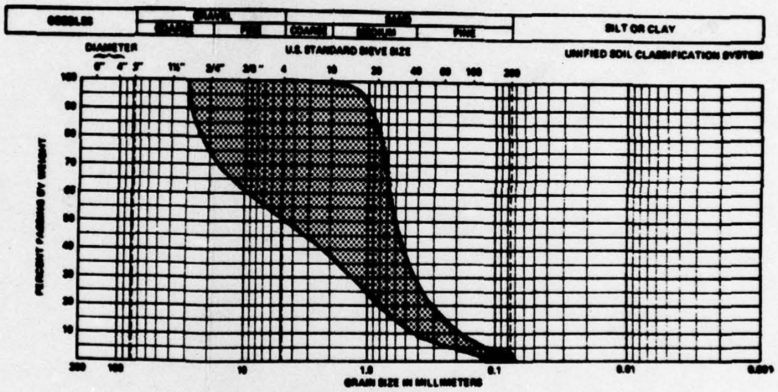
...raphic unit E2



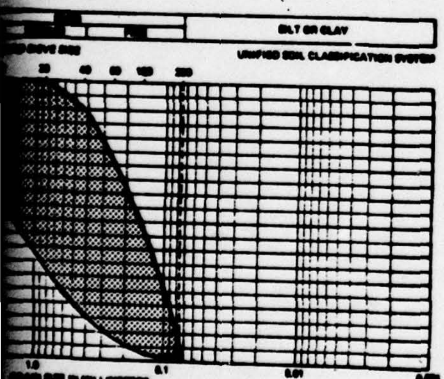
Stratigraphic unit J



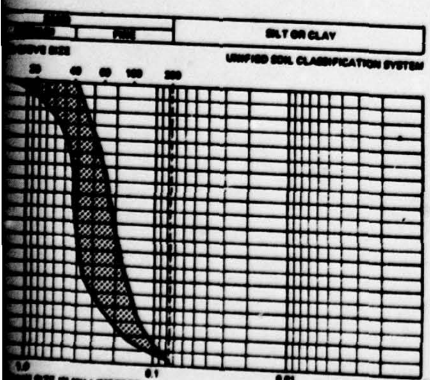
...raphic unit F



Stratigraphic unit K



...aphic unit G



...ahic unit I

DRILLED-IN PILE TEST PROGRAM

GRAIN-SIZE DISTRIBUTION OF VARIOUS STRATIGRAPHIC UNITS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
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DACW43-78-C-0066

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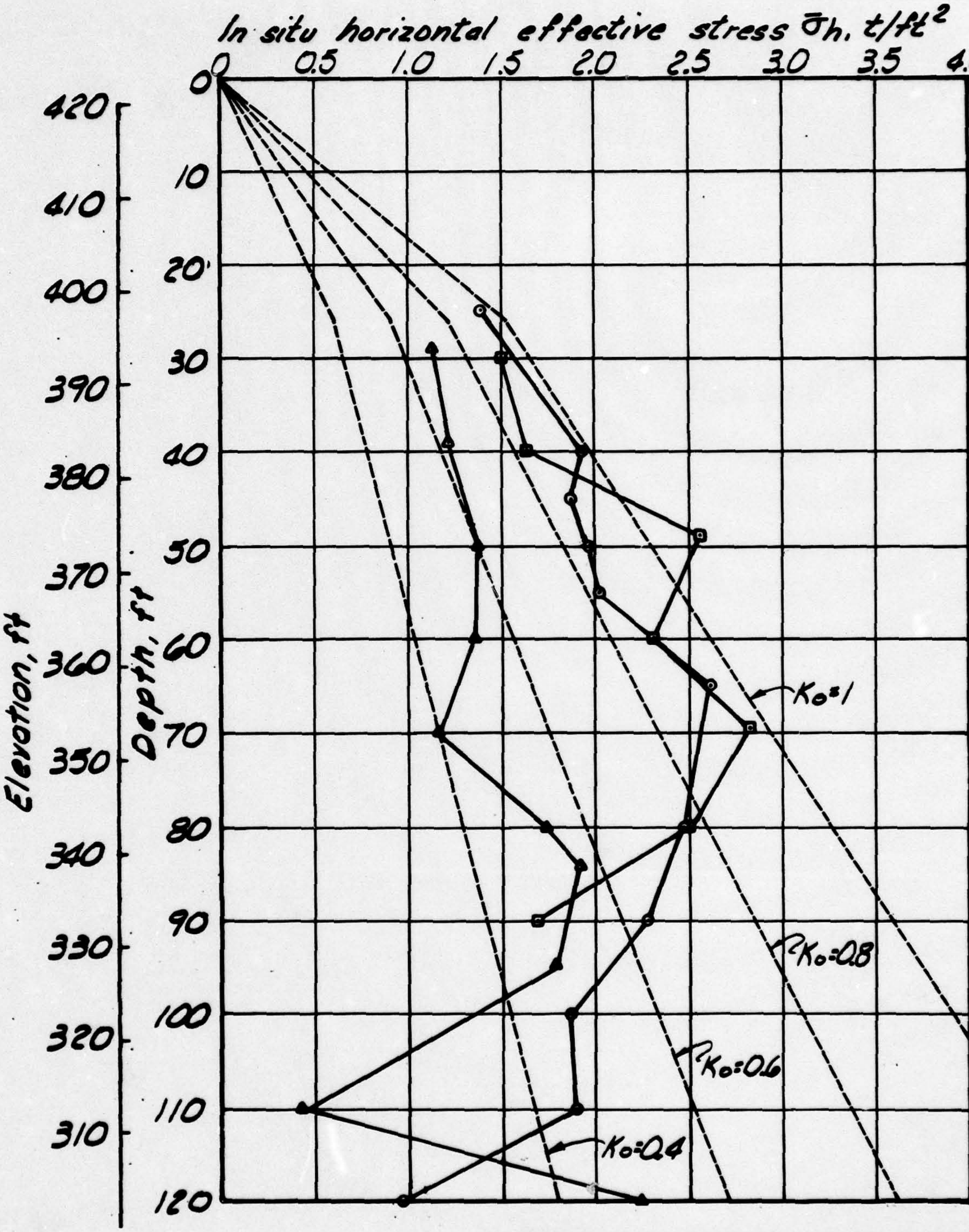
Fig. 3.6

Y7C828 Phase IX



Legend
- o Boring DP-PM-1
- □ Boring DP-PM3
- △ Boring DP-2-PM1

$K_o = \bar{\sigma}_h / \bar{\sigma}_v$

Note:
In situ horizontal effective stress inferred from pressuremeter test

DRILLED-IN PILE TEST PROGRAM

INFERRED HORIZONTAL EFFECTIVE STRESS PROFILE BEFORE PILE INSTALLATION

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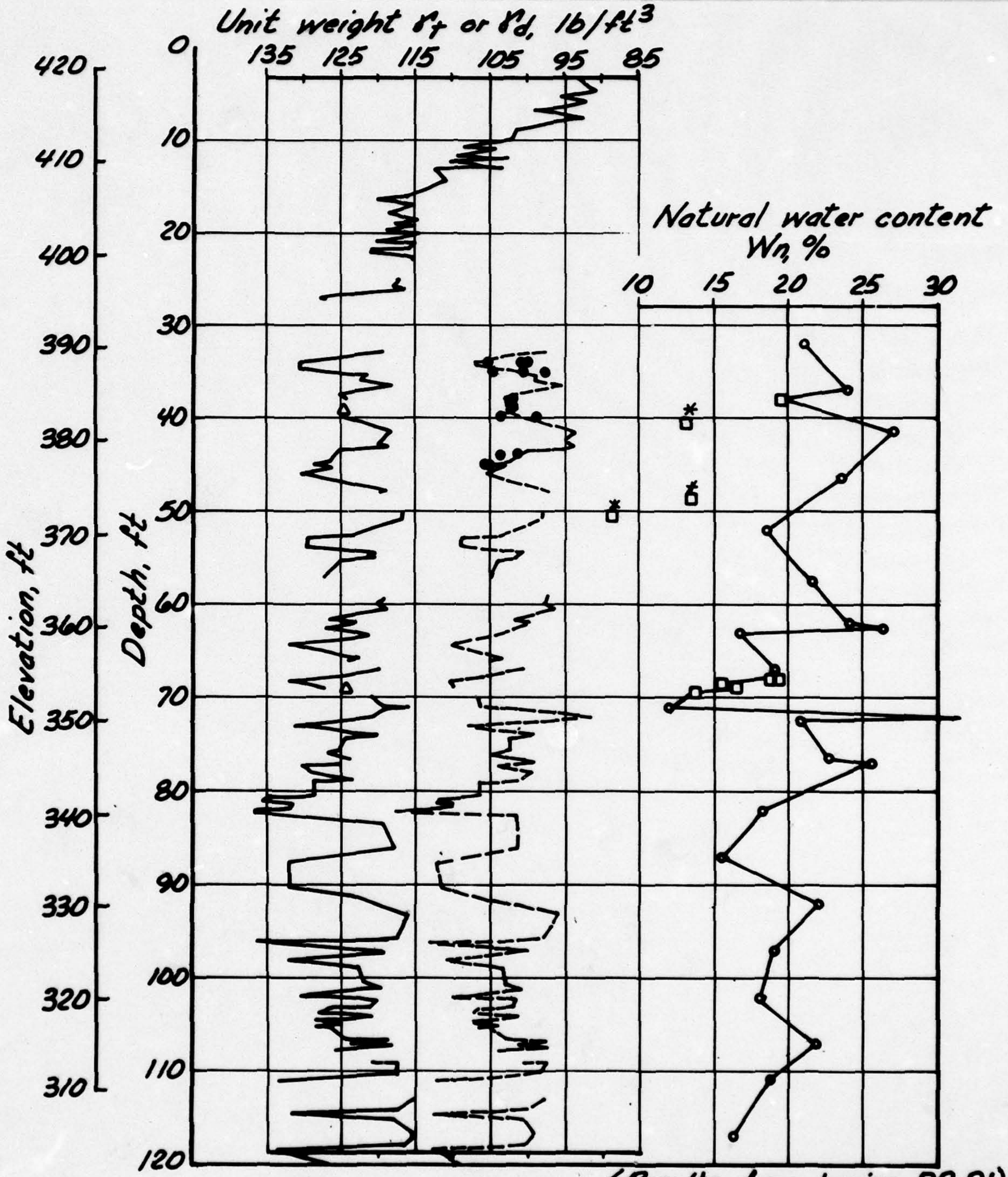
Y7C825 Phase IX

Fig. 3.7



(Results from boring DP-D1)

Legend

Unit weight

— Total-(nuclear density probe)

△ Total-(Hvorslev sample)

--- Dry-(nuclear density probe)

● Dry-(Osterberg sample)

Natural water content

○ Split spoon sample (disturbed)

□ Hvorslev sample

* Drainage allowed in tube

DRILLED-IN PILE TEST PROGRAM

PROFILE OF NATURAL UNIT WEIGHT AND WATER CONTENT BEFORE PILE INSTALLATION

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DACW43-78-C-[illegible]

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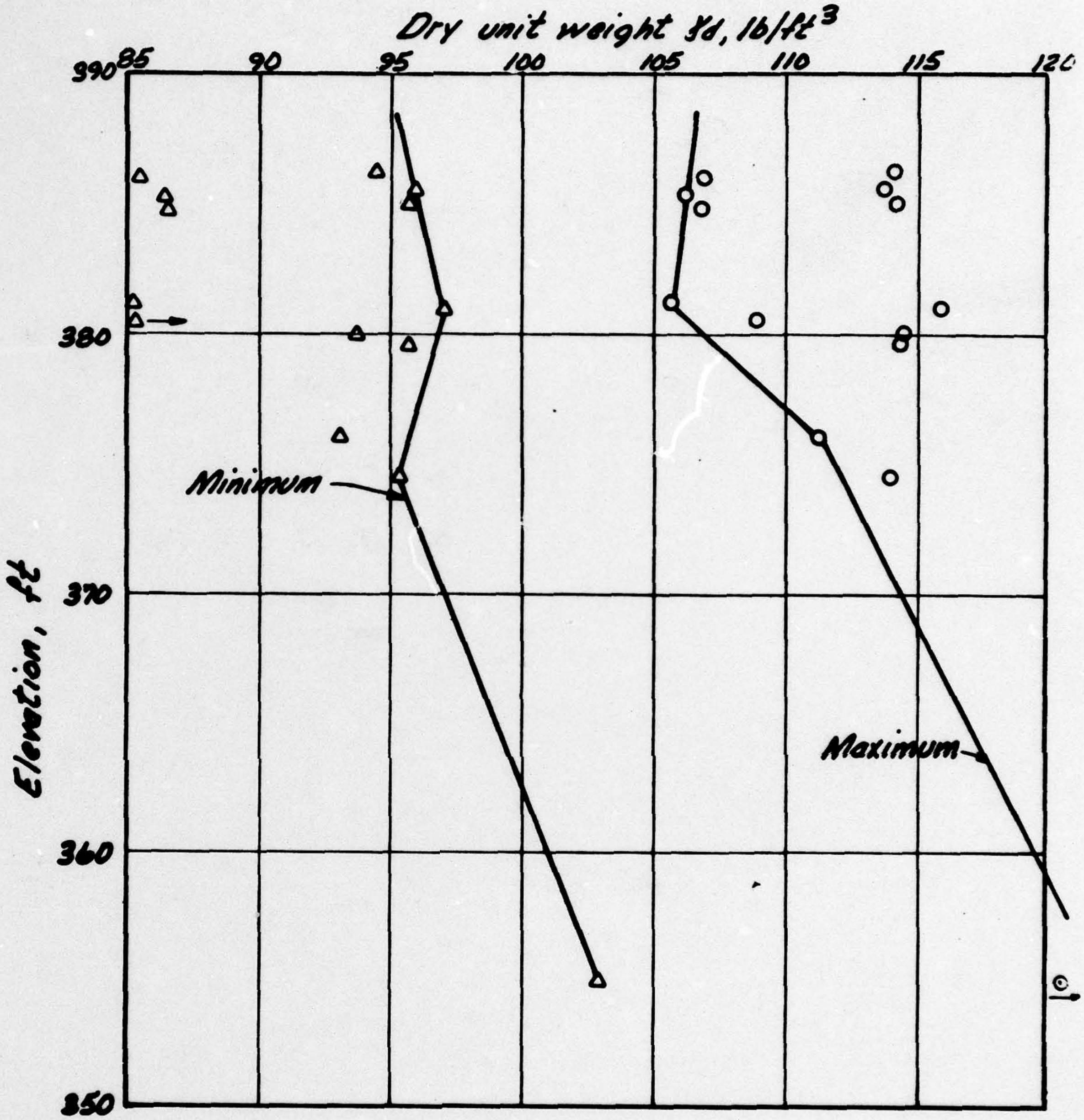
77C[illegible] Phase III

Fig. 3.8



Legend

○ Maximum dry unit weight

△ Minimum dry unit weight

Note:

Maximum and minimum dry unit weight determined on Osterberg samples using the modified Providence, funnel, tube, and cylinder tilt method

DRILLED-IN PILE TEST PROGRAM.

LABORATORY MAXIMUM-MINIMUM DRY UNIT WEIGHTS

FOUNDATION INVESTIGATION AND TEST PROGRAM

EXISTING LOCKS AND DAM No. 26

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DACW43-78-C-0009

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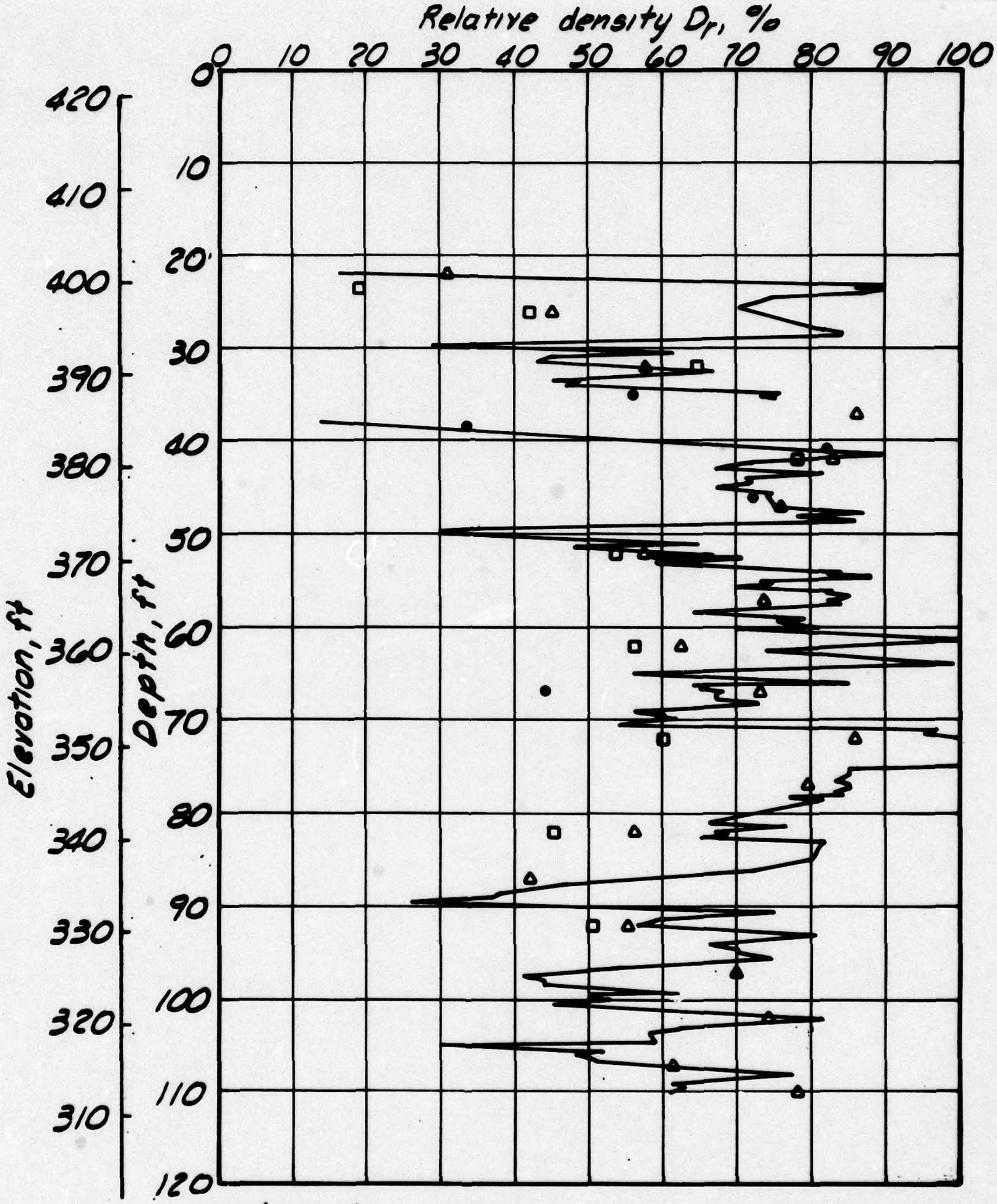
77C028 Phase III

Fig. 3.9



Legend

Static cone penetration test (Schmertmann 1976)

—— Boring DP-C1

Standard penetration test (Gibbs & Holtz 1953)

□ Boring DP-PM1

△ Boring DP-PM3

• Calculated based on in situ unit weight and laboratory max-min unit weight

DRILLED-IN PILE TEST PROGRAM

RELATIVE DENSITY PROFILE BEFORE PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
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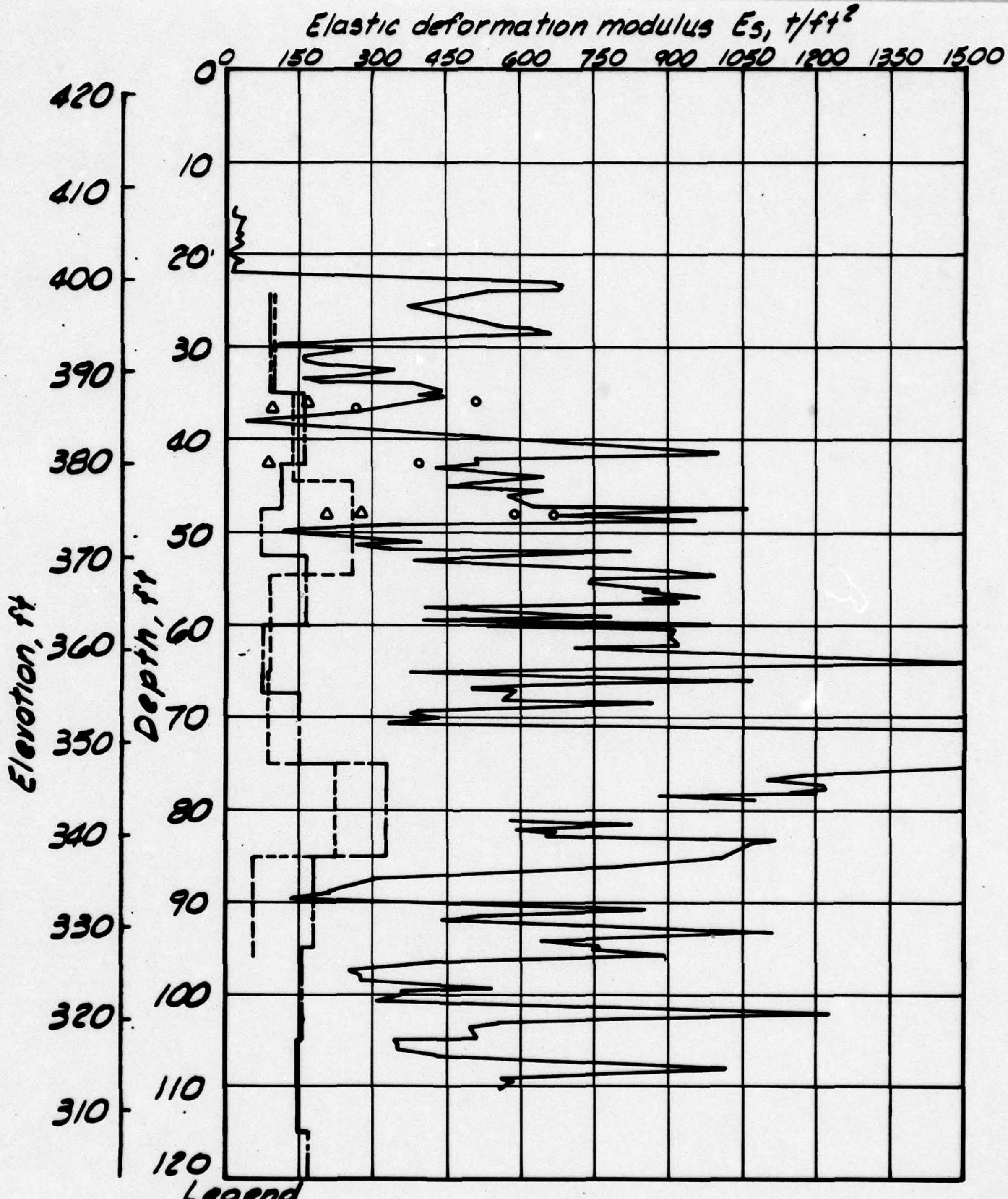
Fig 3.10



Legend

Static cone penetration tests (Vesic 1970)

— Boring DP-C1

Pressuremeter tests

-- Boring DP-PM1

-·- Boring DP-PM3

CID triaxial tests

△ Secant modulus at failure

○ Initial tangent modulus

DRILLED-IN PILE TEST PROGRAM

ELASTIC DEFORMATION MODULUS PROFILE BEFORE PILE INSTALLATION

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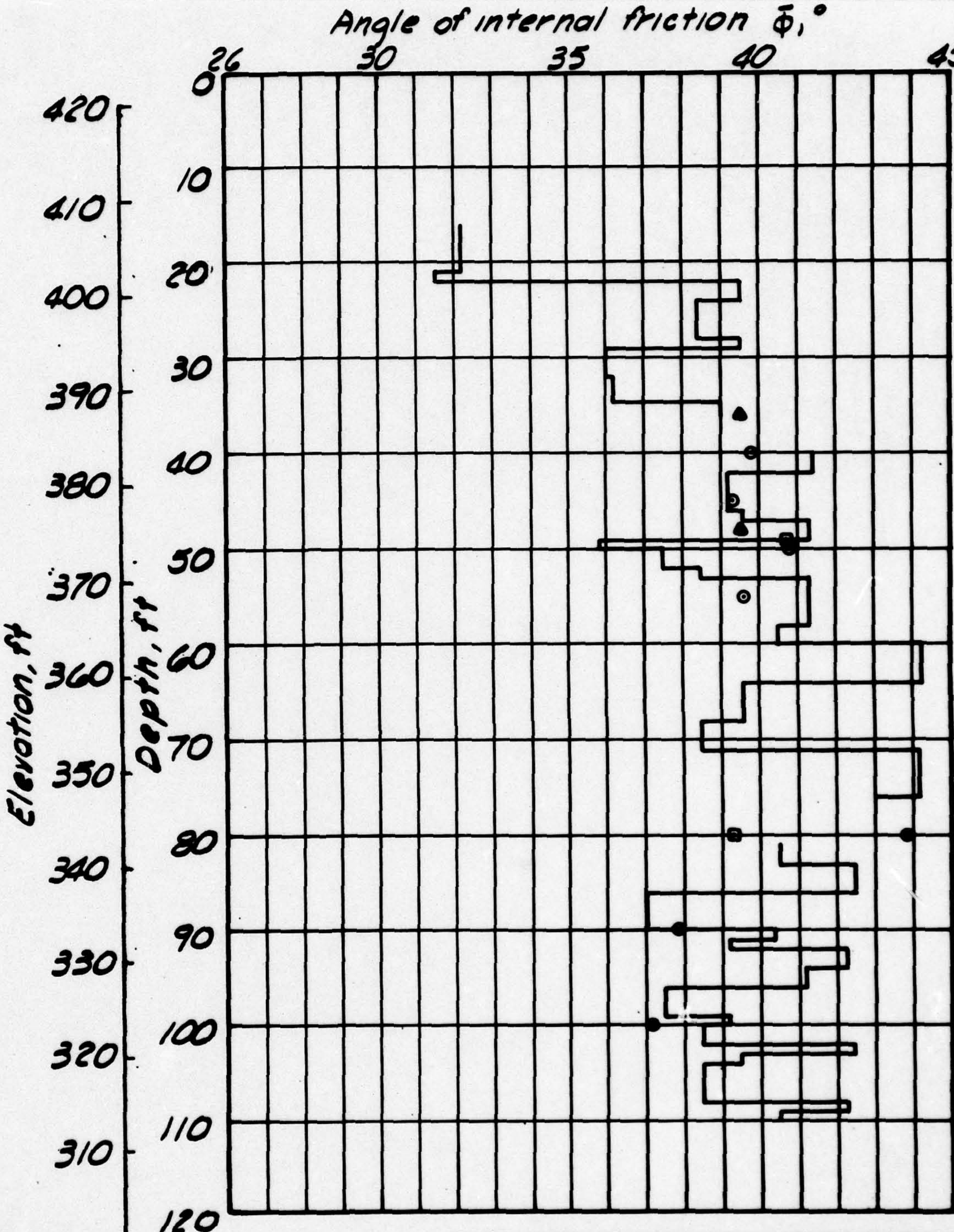
Fig. 3.11



Legend
Static cone penetration tests (Meyerhoff 1974)
—— Borings DP-C1
Pressuremeter tests (Hughes et al 1977)
□ Boring DP-PM1
○ Boring DP-PM3
▲ CID triaxial test

DRILLED-IN PILE TEST PROGRAM

ANGLE OF INTERNAL FRICTION PROFILE BEFORE PILE INSTALLATION

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Fig.3.12

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF
DRILLED-IN PILE TEST PROGRAM

SECTION 4

INSTRUMENTATION

4 INSTRUMENTATION

4.1 INSTRUMENTATION REQUIREMENTS

4.1.1 General Requirements

The instrumentation required for the drilled-in pile tests was simple, but extensive. It was designed to satisfy the primary objectives of the test program that were to:

(1) measure the soil mass and ground surface response to the installation of the piles; and

(2) measure the quantity of material excavated from within the casing during installation.

A list of instrumentation installed and monitored during the program is given in Section 4.2.2. The locations of all instruments and profile of those most readily referenced to the centerline of piles DP2 and DP3 are shown in Fig. 4.1 and 4.2, respectively. The instrumentation is discussed in detail in the following sections.

4.1.2 Soil Mass Response to Pile Installation

During the installation of two batter drilled-in pil (DP2 and DP3) the permanent deformation of the soil mass and ground surface was measured as follows:

(1) settlement at various depths and locations was measured using Borros settlement gages installed as shown in Fig. 4.1 and 4.2;

(2) lateral displacement and settlement were measured using three-dimensional (3-D) deformation gages, consisting of inclinometer casings extending the full depth to rock and magnetic settlement devices (Sondex rings) installed at various intervals along the inclinometer casings; and

(3) surface settlement was measured using surface settlement points intalled at a depth of about 2 ft below the ground surface.

4.1.3 Quantity of Material Excavated

During the drilling operations, the quantity of material excavated from inside the casings was measured using:

(1) weighing bins, as described in Section 2, to contain the excavated material; and

(2) large capacity scales to weigh the excavated materials.

4.2 DESCRIPTION OF INSTRUMENTATION

4.2.1 Optical Instrumentation and Controls

Settlement measurements were referenced to a fixed vertical benchmark. This was done by direct measurements of control points using a Wild NAK1 self-leveling level.

Vertical Benchmark. An as-built sketch of the vertical benchmark used for the drilled-in pile test is shown in Fig. 4.3. The benchmark was located near the central west edge of the drilling effects test area (Fig. 4.1). Surveying rods were used for the vertical surveys. The accuracy of the measurements was 0.005 ft.

4.2.2 Ground Instrumentation

The following instruments were installed in the ground at the locations shown in Fig. 4.1 to measure soil mass and ground surface response during pile installation.

Type of Instrument	Number of Instruments	Comments
Surface Reference Point	15	Rebar installed 2 ft below ground surface
3-D deformation gage	8	Inclinometer casings fitted with Sondex rings
Borros settlement gages	18	Installed at various elevations

Surface Reference Points. The surface reference points consisted of a No. 6 reinforcing steel bar with a diagonal saw cut at the top. An as-built sketch is shown in Fig. 4.4. The system accuracy was ±0.005 ft.

Three-Dimensional Gages. The three-dimensional (3-D) deformation gages consisted of Sondex settlement rings slipped over a PVC inclinometer casing (Sinco 2.75-in.-od casing) and fixed to the surrounding soil using pea gravel backfill. The Sondex rings were placed at approximately 2.5-ft, 5-ft, and 10-ft intervals along the casing, depending upon location of the drilled-in pile axis with respect to the inclinometer casing. They consisted of stainless steel wire loops attached to 12-in.-long segments of thin, corrugated polyethylene casing. The segments of polyethlene casing were attached to the PVC casing using plastic tape. A bentonite paste was inserted between the two casings to reduce friction. This installation method was developed in the field on the basis of observations made earlier during the chemical grouting test program (Section 8, Volume III). An as-built sketch of a 3-D deformation gage is shown in Fig. 4.5. A photograph of one of the 3-D deformation gages at the drilled-in pile test location is shown in Fig. 4.6. The inclinometer casings were surveyed for horizontal movements with a Sinco Model 50309 digital, manual readout (operating range of inclinometer: 0 to

30 degrees from vertical; sensitivity: ±0.005 ft/100 ft of casing). The Sondex rings were surveyed with a Sinco Model 50812 Sondex settlement probe (operating range: 250 ft; sensitivity: 0.01 in.). The Sondex system accuracy was rated at ±0.05 in., which was the accuracy of the tape.

Borros Settlement Gages. The Borros gage consisted of an anchored point attached to a 0.25-in.-dia steel riser which was isolated from friction of the surrounding soil by a 1-in.-dia steel pipe. An as-built sketch is shown in Fig. 4.6. The system accuracy was 0.005 ft.

4.3 DESCRIPTION OF MEASUREMENT OF EXCAVATED SOIL QUANTITY

4.3.1 Weighing Bins and Scales

The weighing bins and scales are described in Section 2.3.3. The bins were designed to allow drainage of free water and to be able to store at least 1500 lb of cuttings.

4.3.2 Procedures

After about 1600 lb and 2000 lb of excavated soil were placed in a bin by the hammergrab, the bin was hoisted with a 15-t hydraulic crane and placed on tilting racks on two 2000 lb scales. On the racks, the tilted bin allowed free water to drain out of the screen at the base of the end panel. By making periodic weighings on the lever arm of the scales and when the weights remained constant over several weighings, it was possible to determine when all free water was drained. This was further confirmed by observations at the screen. At first, a cloth bag was attached below the screen to collect fines discharged with the drainage water. The first bin showed the amount of fines being lost was negligible compared to the weight of the material in the bin and the use of the bag was discontinued.

After the free water was drained, three soil samples were taken, two at a depth of about 3 in. below the surface and one about 9 in. below the surface, for water content determination. For pile DP3, four samples were taken at each bin because of difficulties with freezing of the soil.

The tare weight of the bin and racks was subtracted from the total weight and the weight of dry soil was calculated from the resulting weight of wet cuttings and average water content. The dry weight was divided by the in-place dry unit weight to obtain the volume that the soil occupied in the ground for the interval excavated. The in-place dry unit weight was determined in the field with the nuclear density probe as described in Section 3.2.6. The total volume was then divided by the drilled depth interval, resulting in the average volume of soil excavated per foot of excavation.

4.4 INSTRUMENT LIMITATIONS

4.4.1 Vertical Benchmarks

This benchmark was grouted into bedrock at the locations shown in Fig. 4.3. The center pipe extended about 127 ft from bedrock to the surface and could have been affected to a small degree by thermal expansion between the time initial measurements were made (October 1978) and when test measurements were made (December 1978 and January 1979). It is estimated the thermal contraction could have been on the order of 0.1 in. Allowance was made for this in the case of settlement measurements by using measurements made in December 1978 as the initial data. Otherwise, the benchmark, as installed, was an excellent reference for making precise elevation measurements.

4.4.2 Surface Reference Points

These points were installed to a maximum depth of about 2 ft below the surface of 5 ft to 6 ft of recently placed fill overlying normally consolidated soft to firm silt and clay of the flood plain deposits. The weight of the fill caused the underlying flood plain deposit layer, which was about 16 ft thick, to gradually consolidate. Therefore, the surface reference points were measuring consolidation of shallow soils rather than any subsidence that may have been caused by installation of the drilled-in piles.

4.4.3 Three-Dimensional Deformation Gages

Sondex Rings. These multi-level devices, which were intended to measure soil settlement, have certain limitations which need to be addressed. All measurements are made with reference to the top of the casing. Therefore, it is desirable to measure the elevation of the top of the casing each time measurements are made and it is important to use the same spot at the top of the casing as the reference point. Ground freezing can raise the casing or construction activity can move the top of the casing, thus upsetting the elevation of the top of the casing. Errors in measurements can be minimized by clearly marking the casing at the measurement point, making optical surveys of the top of the casings at each set of readings, and providing protective covers.

The Sondex settlement probe is suspended by an electrical cable which has a modest degree of stiffness in cold weather. This results in some kinking of the cable and erroneous measurements. The difficulty was solved by adding weights to the settlement probe which maintained a substantial tension in the cable. The initial readings were made using this procedure as were the subsequent readings; hence, possible errors due to that source were minimized.

Measurements were made by reading the markings on a tape measure extending from the probe to the top edge of the casing. A possible error resulted from the angle at which different operators read the tape and how they lined up the tape with the top of the casing. This source of error can be minimized by using the same operator to make all readings. It took about 20 min to make measurements on all Sondex rings at each 3-D gage casing and about 3 hr were required to measure all eight gages. In subfreezing weather, and especially when snowing or sleeting, it is possible some measurements were not as accurately made as they would have been in more ideal conditions. During the time that the Sondex measurements, and the inclinometer measurements described below, were being made installation of the drilled-in piles was stopped. These delays and the presence of the instrumentation had an effect on the rate at which the piles were installed. This is discussed in Section 6.5.

As described in Section 4.2.2, the Sondex rings were fixed to the surrounding soil by pea gravel backfill. The pea gravel was dropped into the bentonite fluid in the annular space between the PVC inclinometer casing and the surrounding soil, displacing the bentonite to the surface. As the gravel fell to the bottom, the casing was tapped to reduce the possibility of bridging and creation of gaps. In spite of these precautions, it was found that the pea gravel backfill would settle several inches overnight. Additional gravel was added accompanied by additional vibrations of the casing. There is a definite possibility that later ground vibrations caused by construction operations and the dropping of the heavy hammergrab into the water column in the pipe casing caused further settlement of the pea gravel backfill in some of the gages. It is impossible to know which Sondex rings were affected by such settlement of the backfill. Such settlement could probably have been minimized by backfilling with a weak grout rather than pea gravel; however, grout was not used because of concern it might spread laterally in pervious soil zones and modify the soil characteristics. Furthermore, there is no assurance that the grout column would not be stiffer than the surrounding soil thereby inhibiting downward movement of the Sondex rings as the adjacent soil settled. The backfilling problem is an inherent weakness of the Sondex system for measuring very small movements.

Inclinometer. The inclinometer casings, with Sondex settlement rings attached, were installed in 10-ft segments for the full depth of the borehole to bedrock. Because the casings were grooved, extreme care was required to assure the grooves were lined up. Even small misalignments would cause the wheels of the inclinometer probe to jump the grooves and slip into the second set of grooves. If the probe jumps in and out of the grooves, erroneous measurements result. Misalignment was minimized by careful installation with a special alignment key.

Another problem experienced with most inclinometer casings was spiral ling of the casing. Such spiralling can be a result of some twisting of the PVC casing lengths, of slight misalignment of grooves and/or because the boring itself may be spiralled by drilling. Errors due to spiralling were corrected by using a special spiral checking instrument. This instrument measures the rotation of the grooves at 5-ft intervals over full depth of the casing. The instrument was used on the eight inclinometer casings at the drilled-in pile test site to gather data for the necessary corrections due to spiralling.

It required about 30 to 40 minutes to make inclinometer measurements at each 3-D gage and about 5 hours for all eight gages. These measurements were made immediately before or after Sondex settlement ring measurements; the shutdown of the pile installation was usually governed by making inclinometer measurements. Usually, measurements were scheduled during welding of another section of pipe. As in the case of the Sondex measurements, but to a lesser degree, adverse weather conditions had an effect on the accuracy of inclinometer measurements.

4.4.4 Borros Settlement Gages

Because the points consisted of a steel rod inside a 1-in.-dia pipe extending from several inches above the ground surface to various depths in the ground, thermal expansion due to considerable variations in temperature probably affected the accuracy of measurements. Groundwater and air temperature in the annular space were not monitored and it was not possible to calculate the amount of thermal expansion. Reasonable estimates, however, can be made. As an example, assuming a change in temperature of 30 degrees Fahrenheit in the air and 5 degrees Fahrenheit in the water in the annual space, a Borros gage extending to el 315 could contract or expand 0.15 in. This would represent an upperbound value for thermal expansion. To minimize the possible effects of thermal expansion (contraction) between the warm days of October and the cold days of December, January, and February, elevations taken on 21 December 1978 were used as base data. At that time, very little excavation had been made at DP2.

The Borros gage were the most reliable settlement measuring instruments installed below the ground surface at the drilled-in pile test site.

4.4.5 Measurement of Quantity of Soil Excavated

The procedures for measuring the quantity of soil removed by excavation from the inside casing were designed to minimize errors by working with large volumes of cuttings for each weighing. Possible sources of error are loss of soil in transferring from the casing to the weighing bin, variations in tare of the weighing bin, errors in weighing, errors in water content of soil, and errors in the in-place dry density.

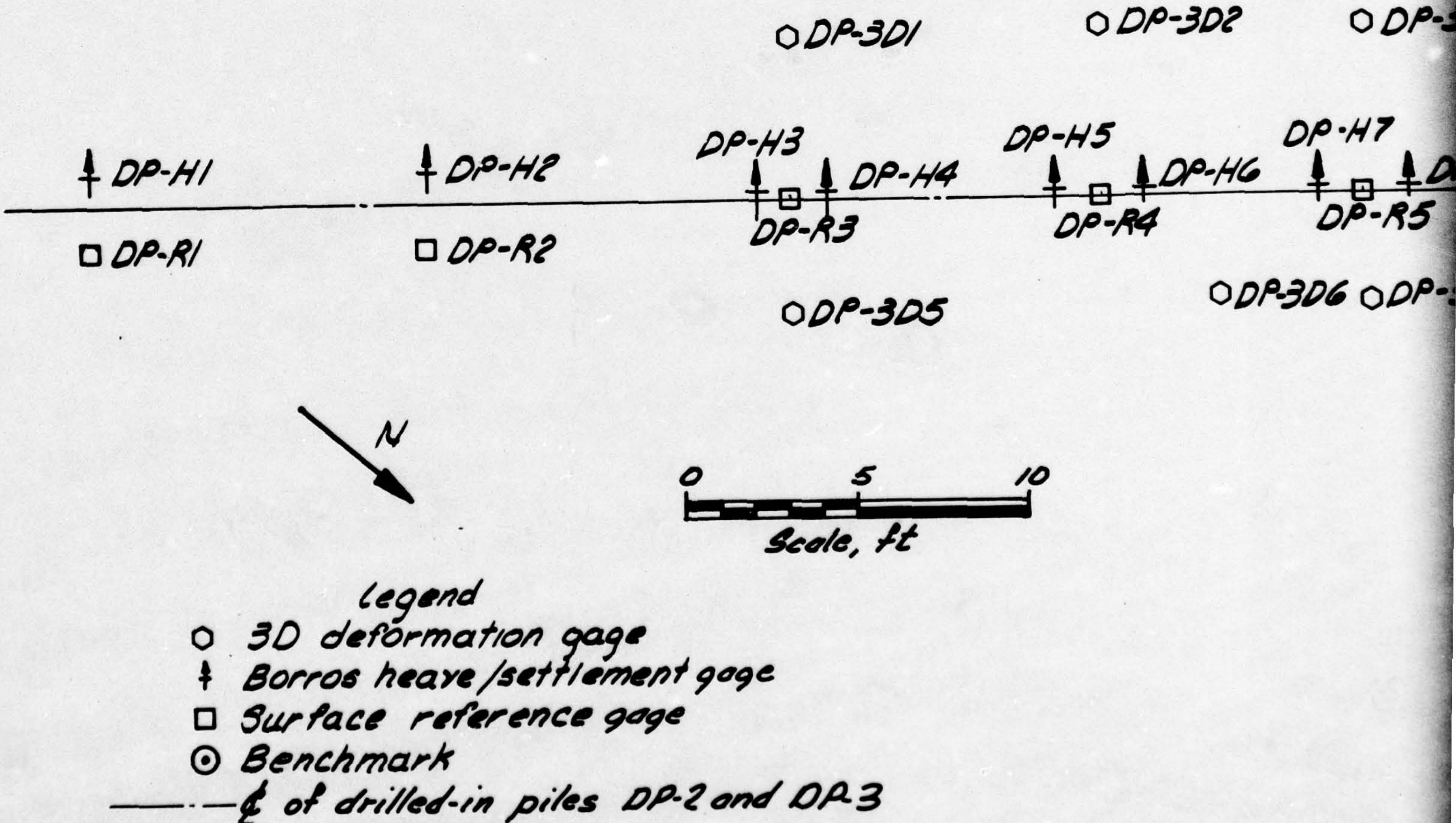
Experience showed that the loss of soil in transfer from the casing to the weighing bin was less than about 20 lb out of about 1600 lb. That is a reduction in the indicated quantity removed of about -1.3 percent. Because of the severely freezing weather, especially during installation of DP3, some difficulty was experienced with some frozen soil sticking to the weighing bin after weighing and emptying. Likewise, it is possible that the amount of moisture absorbed by the wooden bins between various weighings could vary. It is estimated that the amount of soil adhering to the bin surface could be of the order of 20 lb; hence, the actual soil weight would appear to be about 20 lb larger than actually excavated and the error would be +1.3 percent. The bins were originally weighed clean and when the wood moisture content was less. It is estimated that in subsequent weighings, during subfreezing weather, as much as 10 lb additional moisture could be absorbed by the wood. This would result in an additional error of +0.6 percent. Errors in weighing should be small inasmuch as the accuracy of the scales is ±0.1 percent.

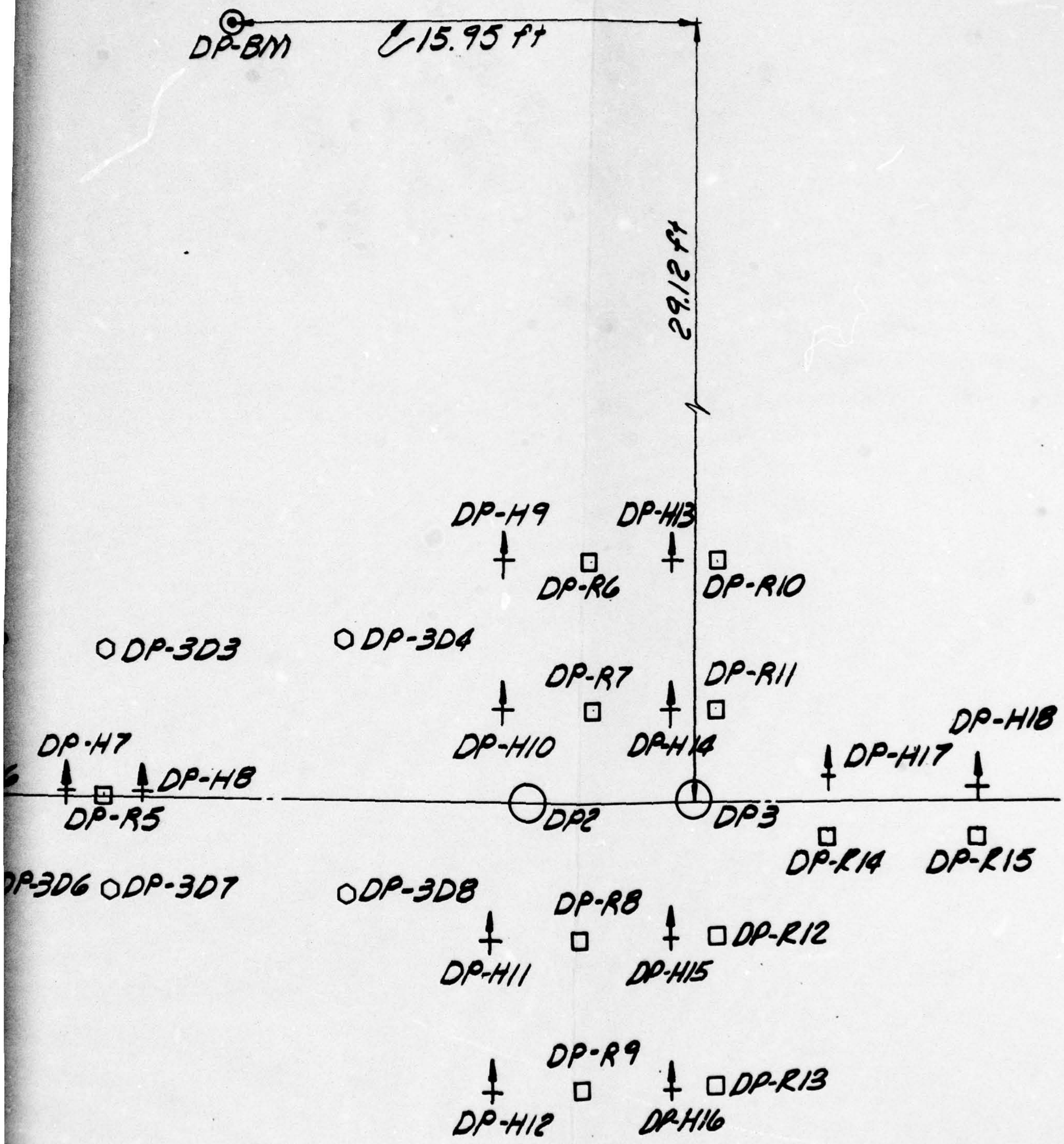
Analysis of the procedures used for taking soil samples for water content determination indicates the samples might underrepresent the average water content of the soil by about 2 percent and result in an error of +2 percent in weight of the cuttings removed. The estimated maximum error in the in-place unit weight assumption is ± 5 percent.

Combining all the possible errors indicates that the weighing bin procedure could result in estimated loss per foot of casing of the order of 7.7 percent too high to 2.5 percent too low. As an example, if the calculated volume of soil removed from the casing was 87 percent of the theoretical volume, the actual volume could vary between 94.7 percent (error on the high side) to 84.5 percent (error on the low side).

4.4.6 Construction Activity and Weather

All instrument locations were marked and protected with tires. This was generally sufficient to avoid damage to the instruments. Only Borros gage H7 appeared to have been jarred during the tests. As previously discussed, cold weather affected the accuracy of instrument measurement.


DP-3D1
DP-3D2
DP-
DP-H1
DP-H2
DP-H3
DP-H4
DP-H5
DP-H6
DP-H7
DP-R1
DP-R2
DP-R3
DP-R4
DP-R5
DP-3D5
DP-3D6
DP-
N
0
5
10
Scale, ft
Legend
3D deformation gage
Borros heave/settlement gage
Surface reference gage
Benchmark
℄ of drilled-in piles DP-2 and DP-3



DRILLED-IN PILE TEST PROGRAM

INSTRUMENTATION PLAN

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
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DACW43-78-C-0005

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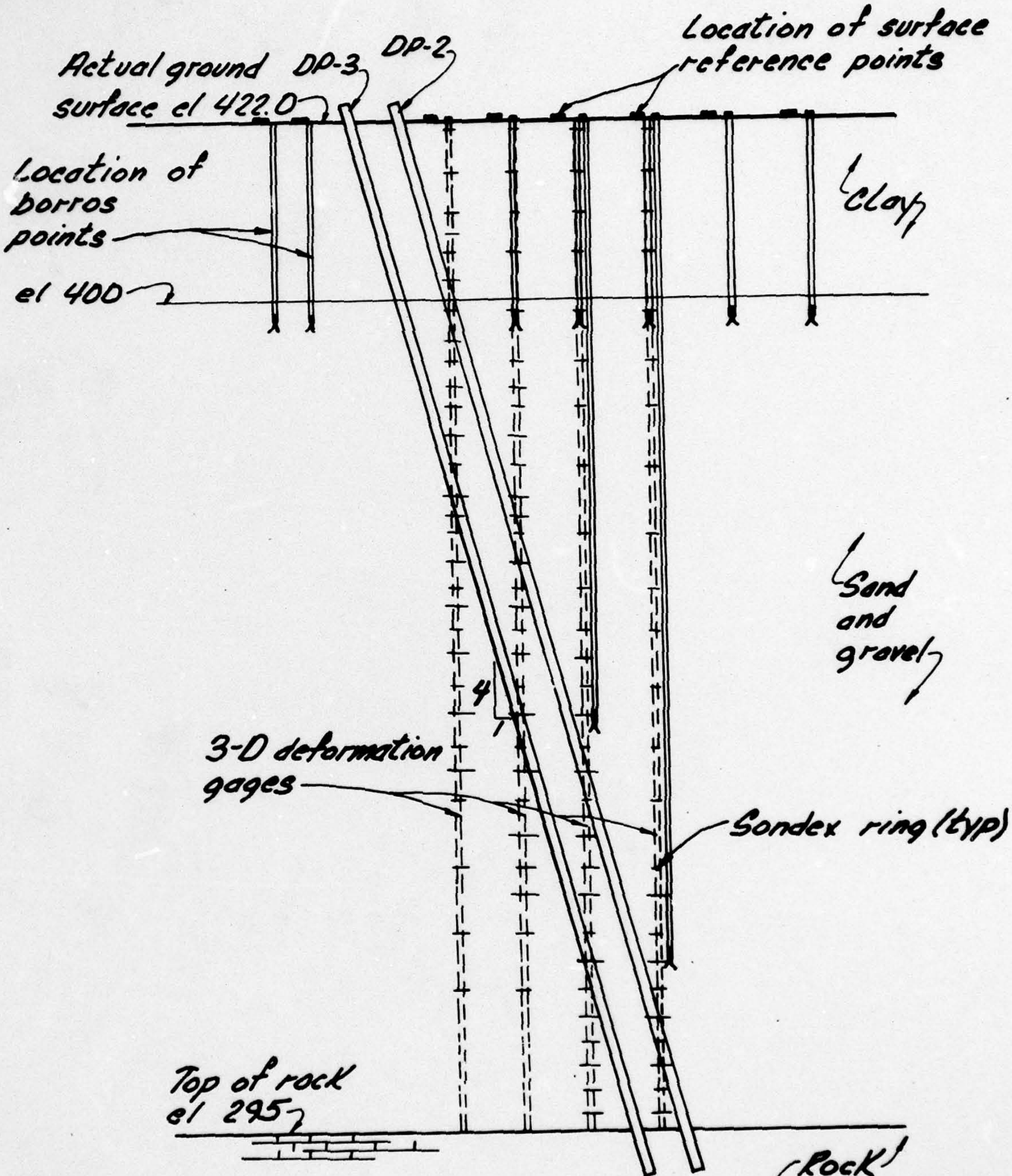
Fig. 4.1

2

V2689 Phase III



Notes:
(1) Surface reference points and borros heave / settlement gages shown are those approximately on centerline of DP2 and DP3
(2) 3-D deformation gages shown are gages 3D1 - 3D4 offset (See plan of instruments, Fig. 4.1)

DRILLED-IN PILE TEST PROGRAM

PROFILE OF INSTRUMENTATION FOR GROUND LOSS MEASUREMENTS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
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DACW43-78-C-0006

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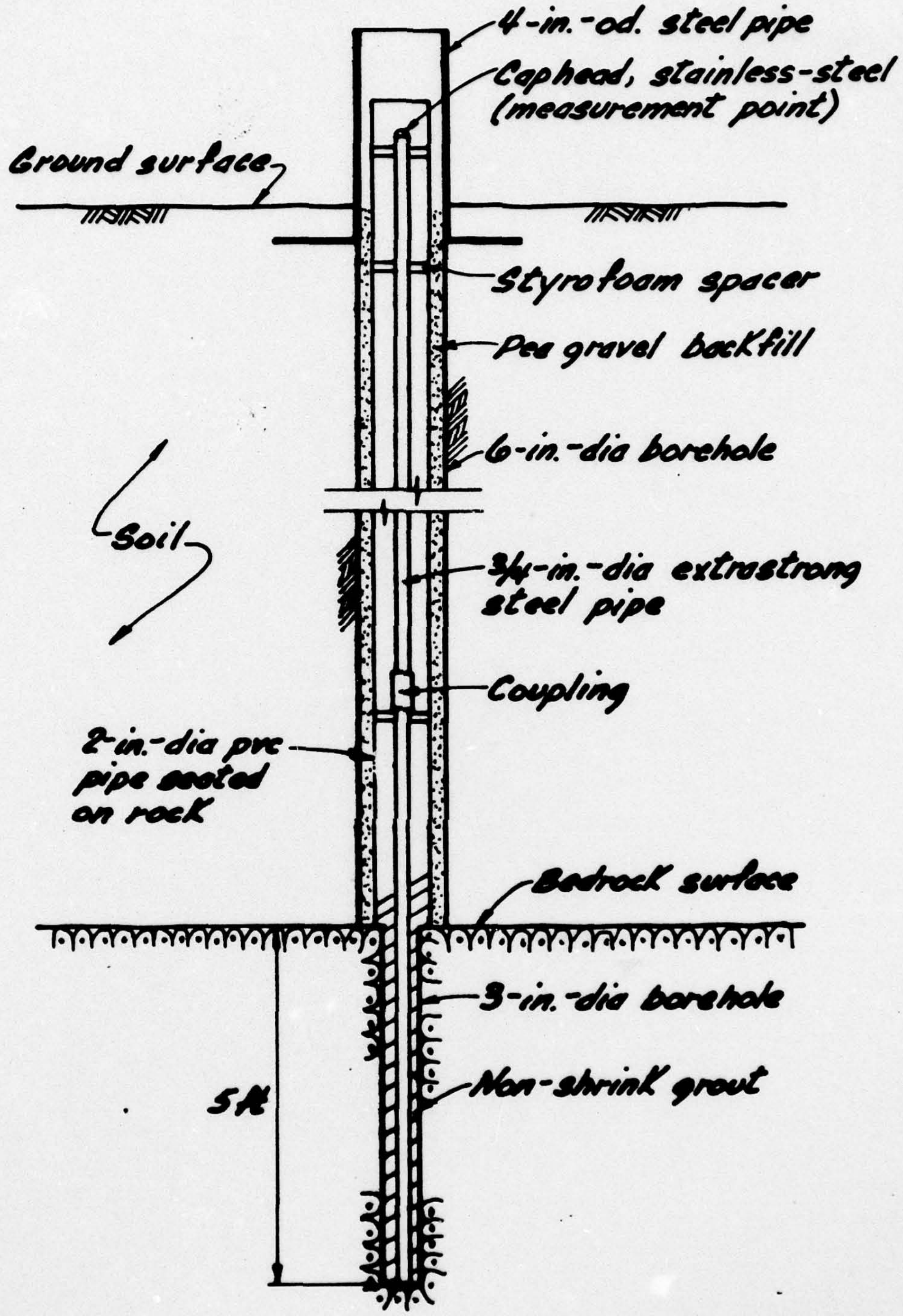
Fig. 4.2



Not to scale

DRILLED-IN PILE TEST PROGRAM

VERTICAL CONTROL BENCHMARK

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
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[illegible]

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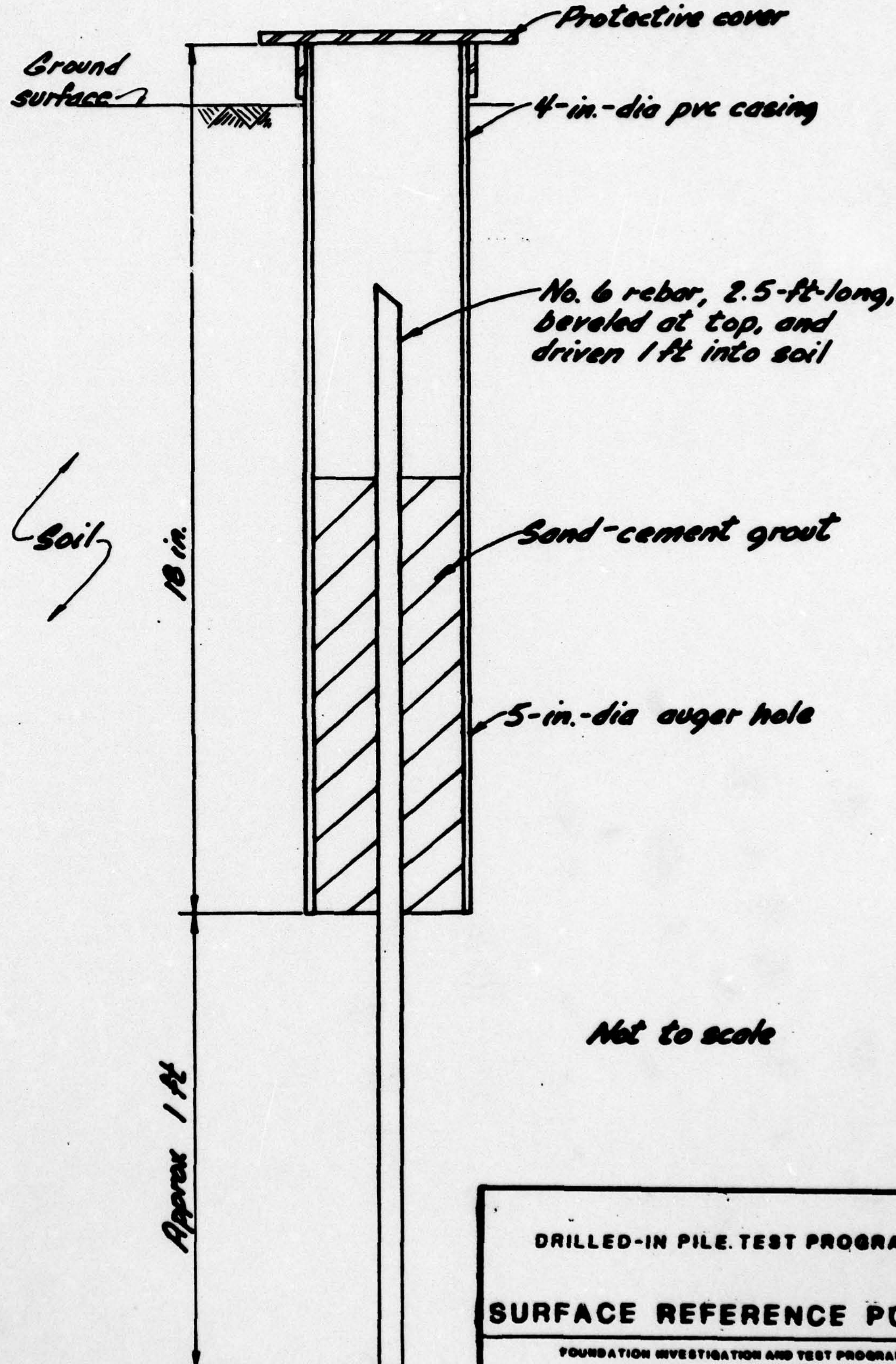
Fig. 4.3



DRILLED-IN PILE TEST PROGRAM

SURFACE REFERENCE POINT

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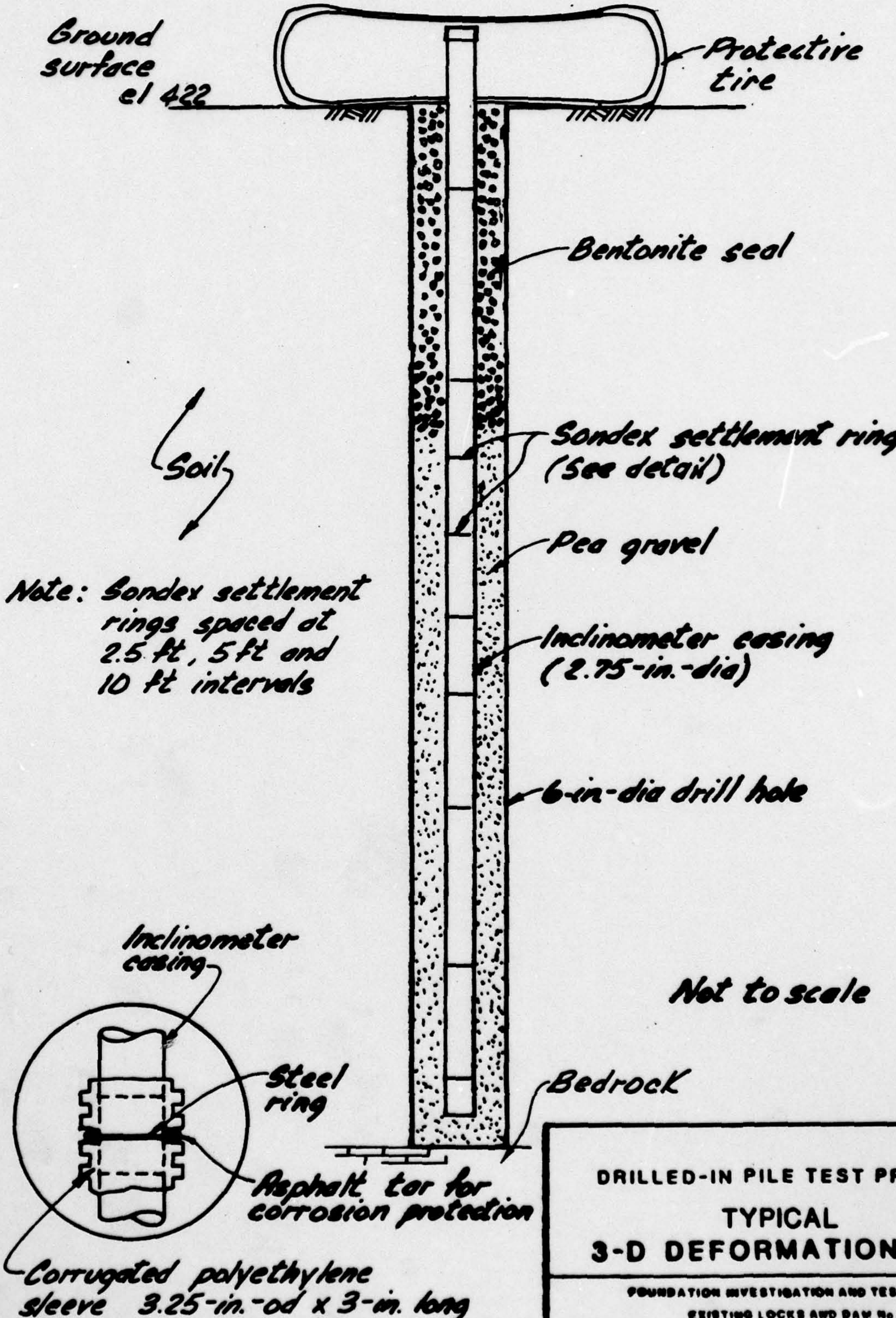
Fig. 4.4



DRILLED-IN PILE TEST PROGRAM

TYPICAL
3-D DEFORMATION GAGE

FOUNDATION INVESTIGATION AND TEST PROGRAM
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DACW43-75-C-0005

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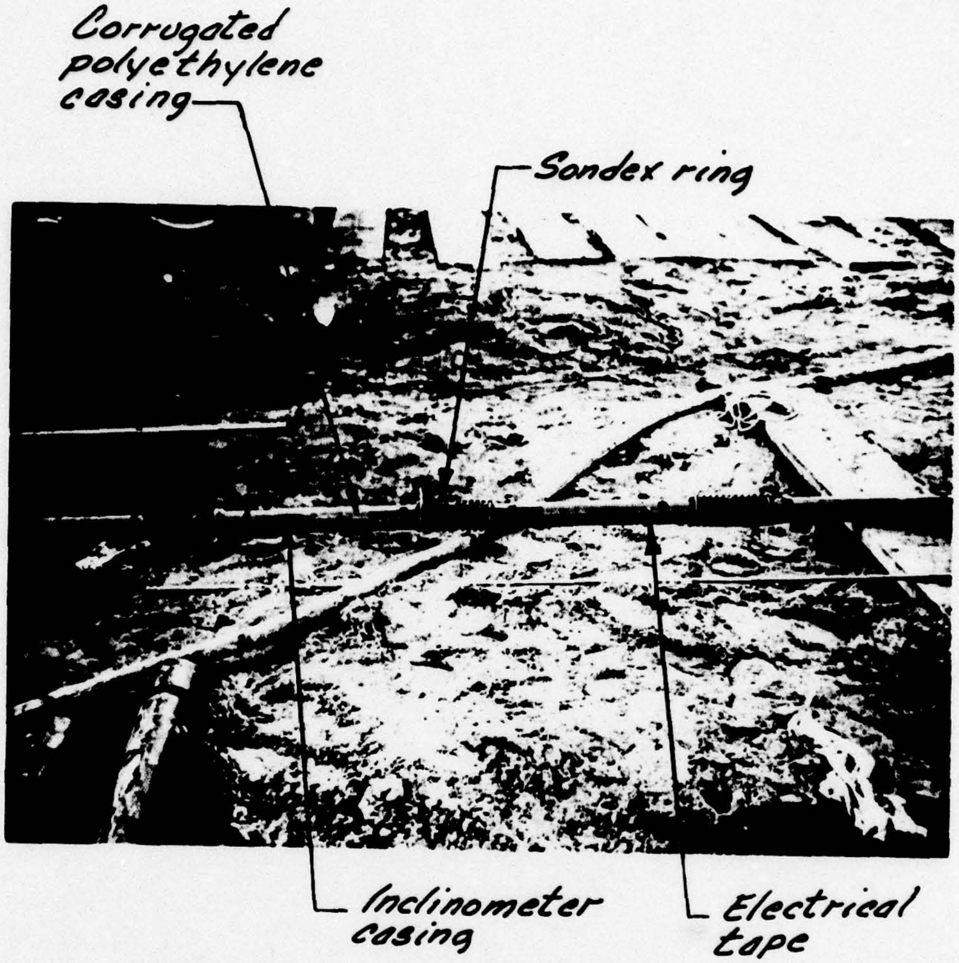
Fig. 4.5

VTC026 Phase II



DRILLED-IN PILE TEST PROGRAM

3-D
DEFORMATION GAGE

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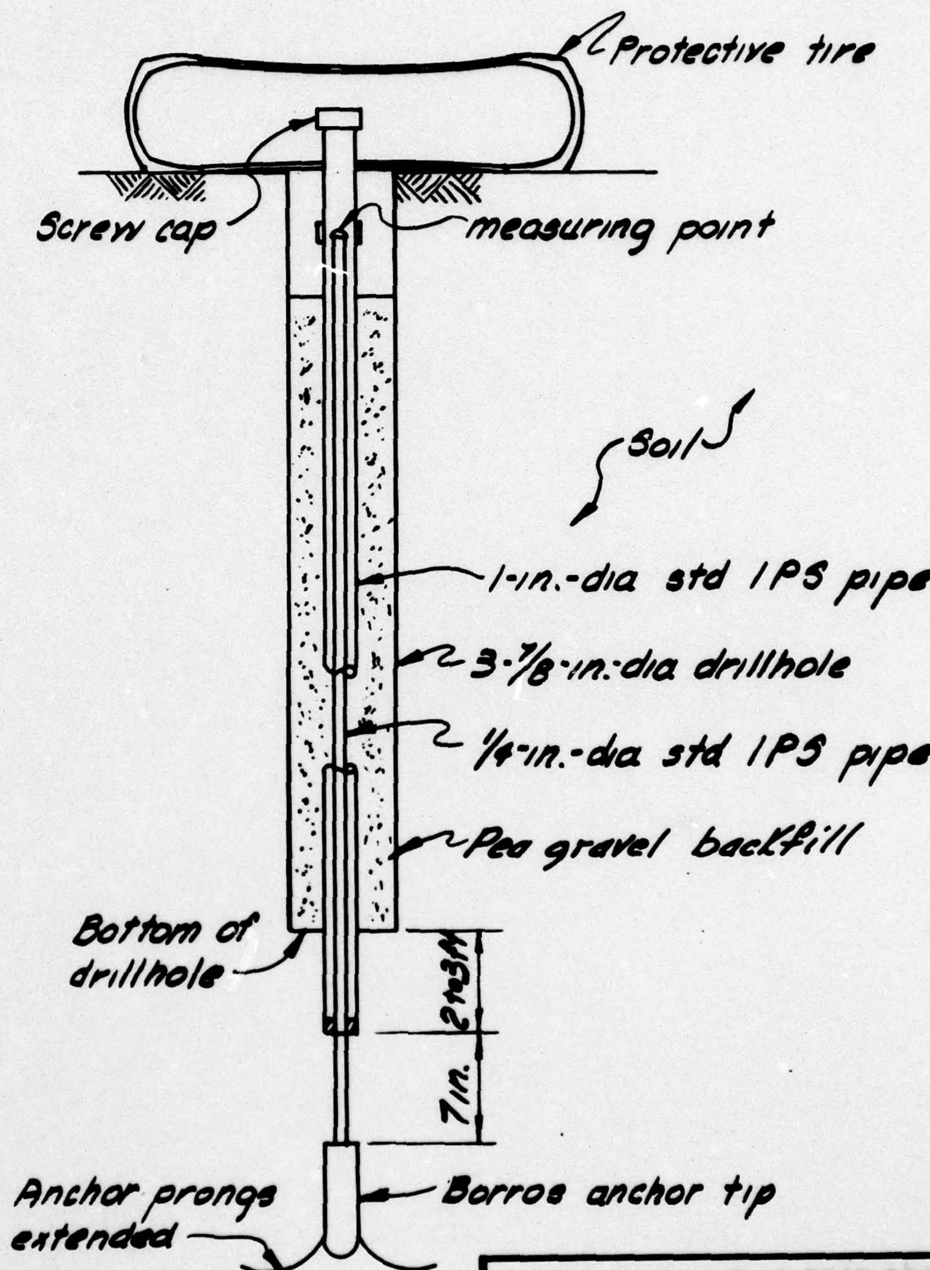
Fig. 4.6



DRILLED-IN PILE TEST PROGRAM

BORROS SETTLEMENT GAGE

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77C825 Phase III

Fig. 4.7

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM

SECTION 5

EFFECTS OF DRILLING

5 EFFECTS OF DRILLING

5.1 GROUND LOSS

5.1.1 Measurements of Quantity of Excavated Soil

The procedures for measuring the quantity of soil excavated from the casing are described in Section 4.3. The resulting data are shown in Tables 5.1 and 5.2. The theoretical volume of the 20-in.-od pipe casing is 2.18 ft^3/ft and the volume excavated for DP2 was generally less than the theoretical volume, except near the bottom of the pile. The cuttings measurement data are plotted against elevation as shown in Fig. 5.1 and 5.2 for DP2 and DP3, respectively. Figure 5.1 shows that the volume excavated is less than the theoretical volume for a 20-in.-dia hole, but more than the theoretical volume of an 18-in.-dia hole. The inside diameter of the casing was 18 in. It appears that some soil under the outside edge of the 1-in.-thick pipe wall was pushed to the outside; however, most was pushed to the inside where, together with the inside plug, it was excavated. Near the bottom of pile DP2, more material than the theoretical volume was excavated and there was some loss of ground. This is not surprising because the chopping and surging activity connected with driving the casing about 1 ft into bedrock on a batter undoubtedly caused soil disturbance and consequent movement into the pipe. The schematic in Fig. 5.3 illustrates how such loss of ground could occur. The excavation into rock was attempted with both types of hammergrabs.

Some loss of ground was also experienced near the bottom of DP3, as shown in Fig. 5.2. At the interval of depth of 119.3 ft to 125.1 ft, a zone of sand, gravel, and cobble was encountered and substantial difficulty was experienced in advancing the casing. To facilitate advancement of the casing, the soil plug was allowed to be less than 1 ft at a depth of 122 ft and again at 125.6 ft. In both cases, the bottom blew in as reported later in Section 6.4.3 and recorded in Table 6.1. Such blow-in resulted in lost ground.

As indicated by the data shown in Tables 5.1 and 5.2 and plotted in Fig. 5.1 and 5.2, ground loss was experienced in only two areas and for the reasons given above. Logs of the drilling operations are given in Appendix D, Volume IVA and provide information on the construction details and difficulties which caused the loss of ground.

5.1.2 Ground Settlement

General. The instrumentation used to measure ground settlement consisted of surface reference points, Borros settlement gages, and Sondex rings, as described in Section 4.2.2. The limitations of these instruments are discussed in Section 4.4. Measurements were made when the Benoto equipment was shutdown and scheduling of measurements was arranged so as to gather pertinent data after stages of progress. Figure 5.4 shows the depth of excavation in piles DP2 and DP3 when measurements were being made on the instruments.

Surface Reference Points. As described in Section 4.4.2, settlement reflected by these instruments represented consolidation of the fill and flood plain deposits, rather than any subsidence caused by loss of ground due to pile installation. A summary of surface settlement measurement data is given in Appendix B, Volume IVA, and a surface settlement profile is presented in Fig. 5.5. The initial measurements were made on 24 October 1978, not long after the fill was completed, and the next measurements were made on 21 December 1978 when DP2 was installed to about el 389. Surface reference points R1, R2, R3, and R4 should not have been affected by the drilling of the first 33 ft of pile DP2, yet they all showed a fairly uniform settlement of 0.032 to 0.037 ft (0.38 to 0.44 in.). This was clearly consolidation of the fill and flood plain deposits. Since the first section of casing of pile DP2 was pushed into the flood plain deposits to a depth of 24 ft on 20 December 1978 without excavation (Appendix D, Volume IVA), it is very possible this activity might have affected R5 which settled 0.049 ft (0.55 in.).

In Fig. 5.5, the 15 and 16 January 1979 measurements represent the end of installation of pile DP2 insofar as surface movement measurements are concerned. The incremental settlements between the 21 December 1978 and 15/16 January 1979 measurements of monuments R1 through R4 range between 0.012 and 0.016 ft (0.14 and 0.19 in.). Surface reference point R5 settled another 0.02 ft (0.24 in.).

Near the completion of installation of DP3 another set of measurements was made (26 January 1979). The incremental settlement for this period was as follows:

Surface Reference Point No.	Settlement ft	Settlement in.
R1	0.004	0.05
R2	0.002	0.02
R3	0.001	0.01
R4	0.007	0.08
R5	0.008	0.10

The settlements measured on R1, R2, and R3 were less than the degree of accuracy of measurement of the surface monuments which is ±0.005 ft.

The total settlement for all surface reference points between 24 October 1978 and 26 January 1979 was as follows:

Surface Reference Point No.	Settlement ft	Settlement in.
R1	0.050	0.60
R2	0.055	0.66
R3	0.048	0.58
R4	0.053	0.64
R5	0.074	0.89

Although surface reference point R5 shows more settlement than the others, it is unlikely it reflected loss of ground due to installation of the piles. It could have reflected, however, because of its proximity, some subsidence of the underlying granular alluvium due to ground vibrations caused by the construction activities.

Borros Settlement Gages. The Borros gages installed at various elevations beneath the ground surface were designed to measure settlement of local zones. The basic measurements made on certain dates are summarized in Appendix B, Volume IVA. The data for gages H1 through H10 only are shown. Though initial measurements were made on all 18 gages installed, certain gages (H11, H17, and H18) were obliterated by construction operations and the others, with the exception of H13, were not measured often enough to derive settlement trends. The presence of construction equipment and materials made access to gages H11, H12, and H14 through H18 extremely difficult. H17 and H18 were located underneath the Benoto rig. The net settlement measured by gage H13, which was set at el 395, was 0.06 in.

As discussed in Section 4.4.4, the Borros gages appeared to have been affected to some degree by temperature charges; to minimize temperature effects, the measurements made on 21 December 1978 were used as the base data. At that time, as shown in Fig. 5.4, the excavation in pile DP2 had progressed only to el 389, therefore, most of the Borros gages should not have been affected by the excavation.

The settlements with respect to the base elevation for each gage were calculated for various depths of excavation and plotted as shown in Appendix B, Volume IVA. As can be seen, there was a scatter of data, but generally within the accuracy of the system. A few gross deviations, such as Borros gagesH7 and H10, were possibly a result of the instruments being displaced by construction equipment. Best fitting curves were drawn and estimated settlement calculated. The net settlement between 21 December 1978 and completion of installation of pile DP2 and of pile DP3 is shown in Fig. 5.6. The settlements were very small and represent subsidence of the alluvial and glacial deposits due to vibration caused by construction equipment rather than pile excavation. When the hammergrab impacted the water inside the casing, the shock could easily be felt at the ground surface. It is believed that these vibrations caused localized pockets of loose, granular materials to densify, resulting in the settlement profile shown on Fig. 5.6.

In support of the above conclusion are data which show some downward movements of Borros gages located below the depth of excavation. For example, Borros gage H3 experienced about 0.08 in. of settlement before excavation reached the depth of that gage. Likewise, gage H4, which was set at el 395, but about 30 ft away from the point of excavation, indicated a settlement of about 0.07 in. when DP2 was excavated to el 389. Since both these gages were well outside the influence zone of drilled-in pile excavation, it appears settlement was caused by other factors.

As indicated in Fig. 5.6, the maximum measured ground settlement was 0.26 in. after completion of installation of piles DP2 and DP3; this maximum settlement was measured at el 395. The least settlement was measured at gage H3, which was set at el 315 or about 20 ft above the bedrock. In general, settlement measured by the Borros gages located in the proximity of the drilled-in piles was proportional to the depth of soil underlying the points.

Borros gage H3 was located at el 315, which is about 15 ft above the lost ground zone experienced in DP2 and 9 ft above the point at which the blow-in occurred in DP3. Neither of these had an apparent effect on settlement of this Borros gage.

Sondex Rings. These instruments and their limitations were discussed in Sections 4.2.2 and 4.4.3. The basic field measurements data are given in Appendix B, Volume IVA. These data, corrected for discrepancies in surface elevations, were used to calculate the amount of settlement experienced by each Sondex ring. The net settlement for gages 3D1 through 3D4 is shown in Fig. 5.7 and for gages 3D5 through 3D8 in Fig. 5.8. The vertical displacements are plotted on a horizontal scale (+) for heave and (-) for settlement. The solid lines represent the apparent movement between 21 December 1978 and 12 January 1979 (end of DP2) and the dashed lines between 21 December 1978 and February 1979 (end of DP3).

In the upper zones, from el 395 to el 420, the Sondex rings indicated settlement occurred in the flood plain deposits and the fill. Below el 395, the data have such a random scatter that no conclusion can be drawn. Gage 3D5 shows essentially no settlement. The apparent heaving shown by the dashed line reveals the inherent inaccuracy of the system. No apparent settlement was indicated by gage 3D5 between el 295 and el 306; some ground loss was measured by weighing of cuttings at that level (Section 5.1.1). Gage 3D5 was located 3.1 ft from the centerline of the line of piles, whereas gage 3D2 was located 5 ft away. The latter seems to indicate some settlement between el 300 and el 320 that could be associated with ground loss; however, similar movements were measured beteen el 370 and el 395 where no such settlement should be expected; gage 3D6 showed settlement below the area of excavation. Gage 3D2 shows settlement increasing with depth which is not consistent with possible loss of ground due to excavation. It could be consistent, however, with settlement of the pea gravel backfill as discussed in Section 4.4.3. Gage 3D7 shows some settlement which might be related to excavation, but gage 3D3 on the opposite side of the line of drilled-in piles did not. Gage 3D8 indicated some possible movement, but gage 3D4 showed practically none (at 3D8, the data for 2 February 1979 were obviously incorrect as they showed a reversal of the downward movement measured on 12 January 1979). The 26 January 1979 measurements were made close to the completion of installation of pile DP3 (Fig. 5.4); a second plot of settlement was prepared using those data.

In view of the wide scatter of conflicting measurements, it is not recommended to place reliance on the small ground settlement recorded using the Sondex rings.

5.1.3 Horizontal Ground Deformation

As described in Section 4.2.2, horizontal ground deformation was measured by inclinometers included in the 3-D deformation gages. Limitations were discussed in Section 4.4.3.

The apparent inclinometer casing deflections between initial installation (21 October 1978) and 11 January 1979 (end of DP2), and 21 February 1979 (end of DP3), are plotted for each gage in Appendix B, Volume IVA. From these data, the magnitude, direction, and depth of maximum ground deformation were calculated. They are shown in Fig. 5.9. The upper plot shows the maximum horizontal ground deformation after DP2 and the lower shows that after DP3. It is apparent that there was no horizontal deformation that that would indicate a movement of the soil towards the piles.

5.2 CHANGES IN SOIL PROPERTIES

5.2.1 Purpose and Scope

A program of in situ testing was performed in close proximity to drilled-in piles DP-2 and DP-3 after installation of the piles to evaluate the influence of pile installation on soil properties. Four static cone penetration test soundings DP2-C1, C2, C3, and C4 and two pressuremeter borings DP2-PM1 and PM2 were accomplished at the locations indicated in Fig. 5.10. Standard penetration tests were performed at 5-ft depth increments in borings DP2-PM1 and DP2-PM2. From these test results, changes in in-situ stresses, relative density, elastic deformation modulus, and angle of internal friction can be inferred.

5.2.2 Standard Penetration Tests

The results of standard penetration tests made after drilled-in pile installation are compared in Fig. 5.11 to results obtained before pile installation. The data shows a large scatter inherent in the alluvial and glacial sediments. A slight decrease in N-values was noted between approximately el 365 and el 345, near the piles; the decrease is so small and the data scatter so large that no definite conclusions can be drawn.

5.2.3 Static Cone Penetration Tests

The results of static cone penetration tests made after drilled-in pile installation are compared in Fig. 5.12 to results obtained before pile installation; Fig. 5.12a should be compared to Fig. 3.2 depicting cone resistance before pile installation. As for the standard penetration tests, the data scatter is large, making definite conclusions difficult or impossible. Qualitatively, a slight consistent decrease in cone resistance was noted above the pile shafts; the decrease is so small, however, that no definite conclusion can be drawn.

5.2.4 Pressuremeter Tests

Volume-change vs pressure curves for pressuremeter tests made after drilled-in pile installation are given in Appendix C, Volume IVA. Boring DP2-PM1 (after pile installation) was drilled using Revert drilling fluid; borings DP-PM1 and DP-PM3 (before pile installation), and boring DP2-PM2 (after pile installation) were drilled with bentonite. Use of bentonite generally resulted in larger borehole disturbance.

In Situ Stresses. In situ horizontal stressed derived from the pressuremeter tests after drilled-in pile installation are compared in Fig. 5.13 to stress values before pile installation. Qualitatively, a decrease in horizontal stress was noted immediately above and between the two pile shafts. A slight stress increase was apparent at the point where boring DP2-PM2 is closest to pile DP3. This increase may be due to the presence of the stiff pile shaft which reacted against the pressuremeter probe, or to a bulb of highly stressed soil immediately surrounding the pile. Elsewhere, the stress remained relatively unchanged, within the normal range of data scatter.

Stiffness. Soil stiffness was characterized by the elastic deformation modulus derived from pressuremeter tests. Pressuremeter modulus is a horizontal modulus which matches the secant modulus at failure determined from laboratory CID triaxial tests (Section 3). Elastic modulus values derived from the pressuremeter tests after drilled-in pile installation are compared in Fig. 5.14 to modulus values before pile installation. A small relaxation of the soil and a reduction of modulus was noted in the zone above and below the piles.

Shear Strength. Shear strength of the soil was characterized by the drained angle of internal friction determined from pressuremeter tests. Friction angle values from these tests after drilled-in pile installation are compared in Fig. 5.15 to values obtained before pile installation. Except for a few cases, no change in friction angle values was observed; the few higher values were not readily explained.

DATE INTERVAL	EXCAVATION INTERVAL DEPTH[1] ft	EXCAVATION INTERVAL ELEVATION[2] ft	DRY WEIGHT OF CUTTINGS lb	WATER CONTENT %	IN PLACE[3] DRY UNIT WEIGHT lb/ft^3	VOLUME EXCAVATED ft^3/ft	GAIN[4] OR LOSS %
21 Dec 78	27.7 to 37.5	395.1 to 385.6	1875	3.8	102	1.88	-13.8
21 Dec	37.5 to 46.8	385.6 to 376.6	1953	5	100	2.1	-3.7
27 Dec	46.8 to 58.3	376.6 to 365.4	2201	8.3	102	1.88	-13.8
27 Dec	58.3 to 66.3	365.4 to 357.7	1576	11.3	102	1.93	-11.5
27-28 Dec	66.3 to 76.2	357.7 to 348.1	2037	11	105	1.96	-10.1
28-29 Dec	76.2 to 86.4	348.1 to 342.1	1379	11.8	103	2.16	-0.9
29-30 Dec	82.4 to 91.7	342.1 to 333	1924	7.2	105	1.97	-9.6
30 Dec	91.7 to 99.2	333 to 325.8	1500	4.4	105	1.9	-12.8
2-3 Jan 79	99.2 to 108.5	325.8 to 316.7	1700	10.6	105	1.74	-20.2
3-4 Jan	108.5 to 117	316.7 to 308.5	1809	6.3	105	2.03	-6.9
2-5 Jan	117 to 126.3	308.5 to 299.5	2125	10	105	2.18	0
5-12 Jan	126.3 to 131.3	299.5 to 294.6	1405	7.5	105	2.68	+22.9

Notes

(1) *Depths are measured along inclined axis of the pile*

(2) *Elevations are on a vertical axis*

(3) *Average measured in-place dry unit weight over interval being measured (Fig. 5.3)*

(4) *Theoretical unit volume of a 20-in.-dia cylindrical opening is 2.81 ft^3/ft; gain or loss is with respect to that volume*

DRILLED-IN PILE TEST PROGRAM

RESULTS OF MEASUREMENTS OF QUANTITY OF EXCAVATED SOIL PILE DP2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

Woodward-Clyde Consultants

V7C026 Phase III

Table 5.1

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RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM. EXI--ETC(U)
JUL 79 J PEREZ , S F GIZIENSKI DACW43-78-C-0005

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2 OF 2
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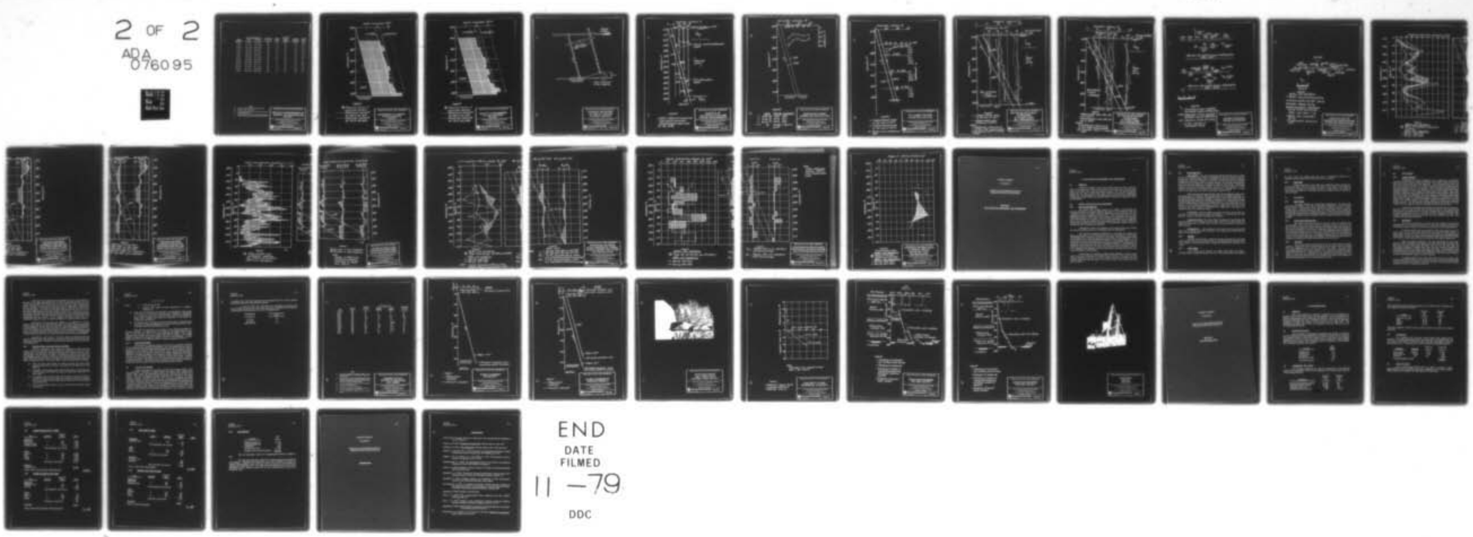

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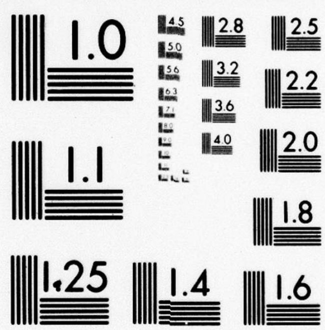


MICROCOPY RESOLUTION TEST CHART

DATE INTERVAL	EXCAVATION INTERVAL DEPTH(1) ft	EXCAVATION INTERVAL ELEVATION(2) ft	DRY WEIGHT OF CUTTINGS lb	WATER CONTENT %	IN PLACE(3) DRY UNIT WEIGHT lb/ft³	VOLUME EXCAVATED ft³/ft	GAIN(4) OR LOSS %
17-18 Jan	24.1 to 33	398.6 to 390	1407	18.6	102	1.55	-28.9
18 Jan	33 to 42.5	390 to 380.8	1439	12.3	100	1.51	-30.7
18-19 Jan	42.5 to 51.1	380.8 to 372.4	1472	13.3	102	1.68	-22.9
19 Jan	51.1 to 60.8	372.4 to 363	1711	7.8	102	1.73	-20.6
19-20 Jan	60.8 to 69.6	363 to 354.5	1749	10.2	102	1.95	-10.6
20 Jan	69.6 to 78.3	354.5 to 346	1683	9.1	105	1.84	-15.6
20 Jan	78.3 to 83.8	346 to 340.7	947	16.9	103	1.67	-23.4
20-22 Jan	83.8 to 92.9	340.7 to 331.9	1851	10.7	105	1.94	-11
22-23 Jan	92.9 to 100.7	331.9 to 324.3	1350	5	105	1.65	-24.3
23-25 Jan	100.7 to 110.3	324.3 to 315	1751	9.9	105	1.74	-20.2
25-26 Jan	110.3 to 119.3	315 to 306.3	1918	5.3	105	2.03	-6.9
26-27 Jan	119.3 to 125.1	306.3 to 300.6	1518	7.5	105	2.49	+14.2
27-29 Jan	125.1 to 130.5	300.6 to 295.4	1186	11.8	105	2.09	-4.1

Notes

(1) *Depths are measured along inclined axis of the pile*

(2) *Elevations are on a vertical axis*

(3) *Average measured in-place dry unit weight over interval being measured (Fig. 5.3)*

(4) *Theoretical unit volume of a 20-in.-dia cylindrical opening is 2.81 ft³/ft; gain or loss is with respect to that volume*

DRILLED-IN PILE TEST PROGRAM

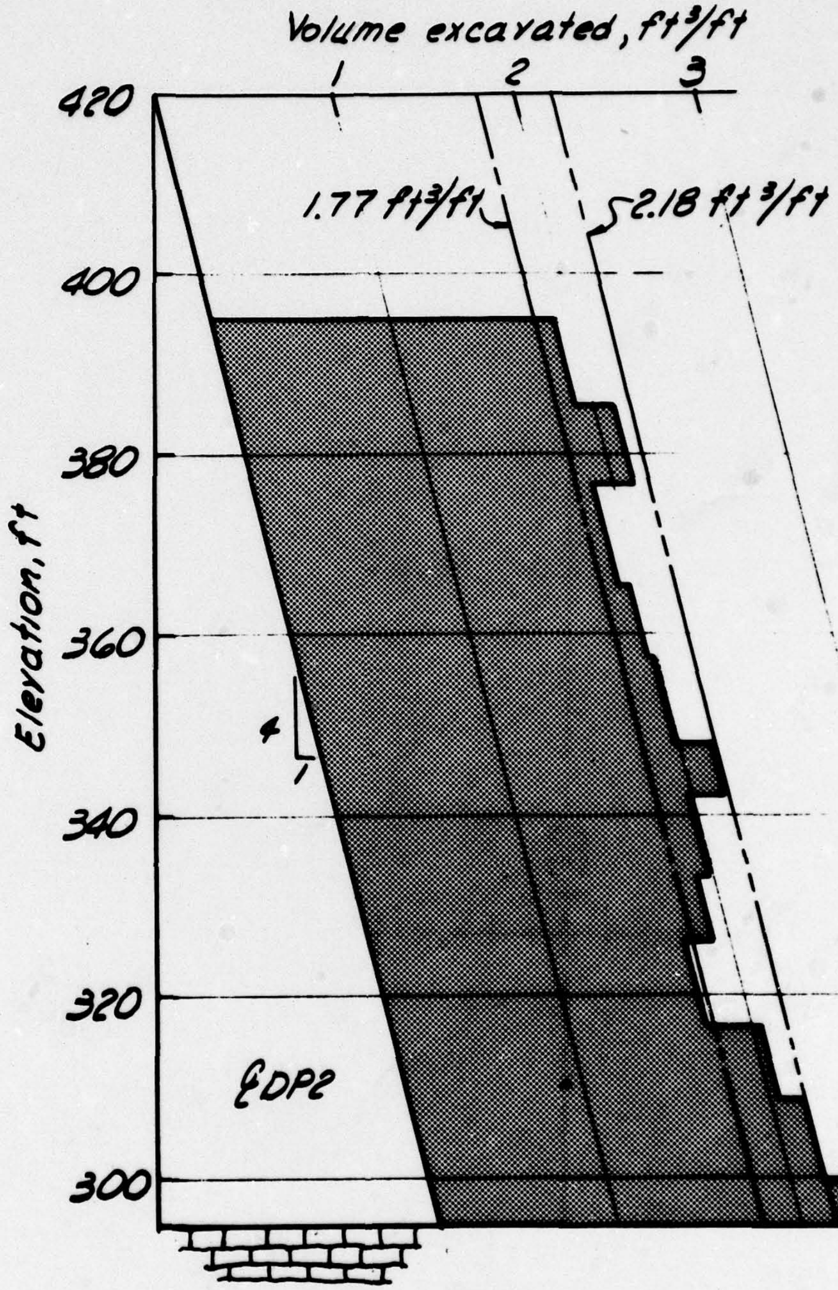
RESULTS OF MEASUREMENTS OF QUANTITY OF EXCAVATED SOIL PILE DP 3

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

77C026 Phase II

Table 5.2



Legend

- Measured volume of excavated materal
- ----Theoretical volume/ft for 20-in.-dia. hole
- ——--Theoretical volume/ft for 18-in.-dia. hole

DRILLED-IN PILE TEST PROGRAM

MEASUREMENT OF QUANTITY OF SOIL EXCAVATED PILE DP2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0090

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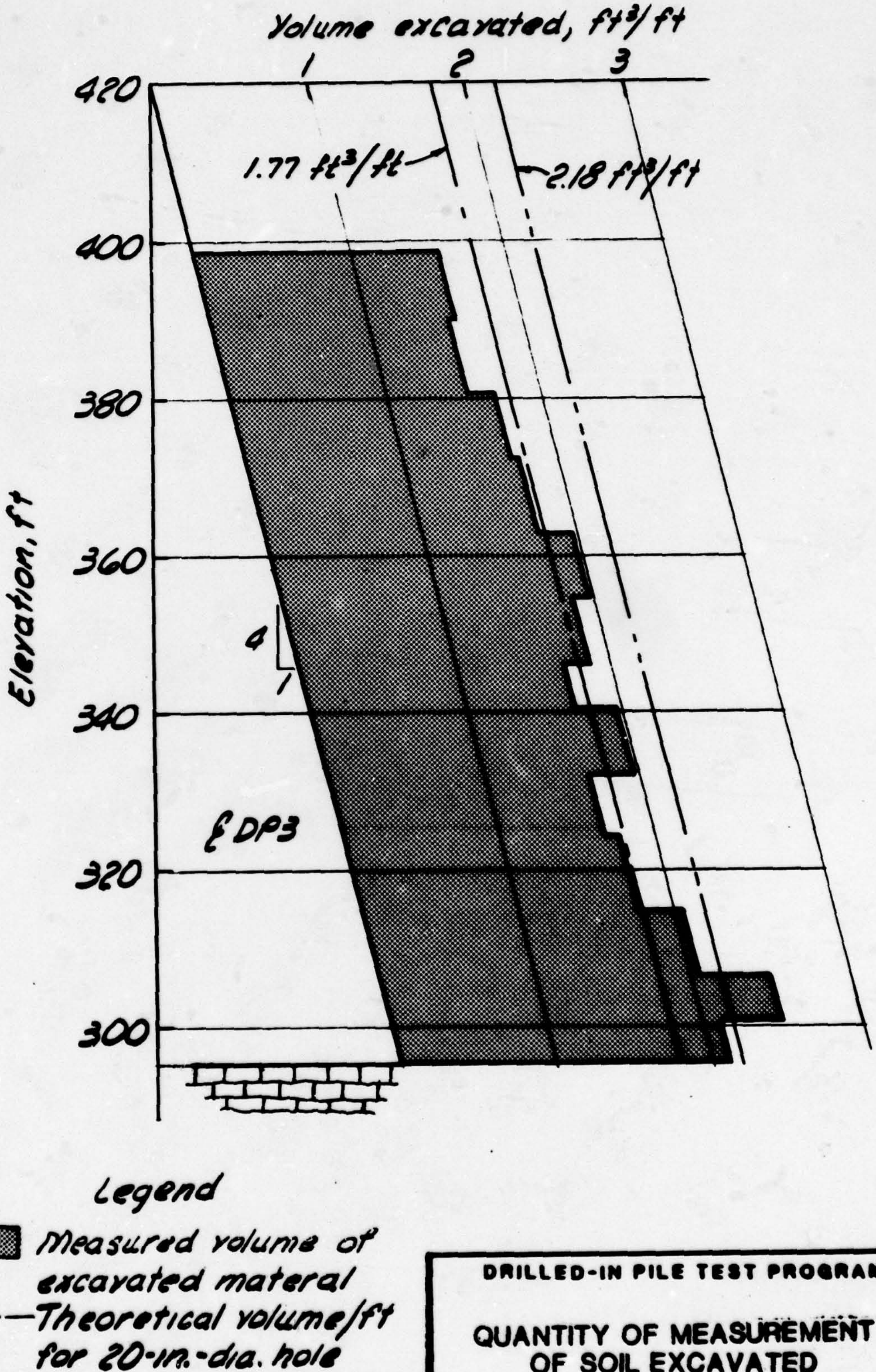
77C026 Phase II

Fig.5.1



Legend

- Measured volume of excavated materal
- ---- Theoretical volume/ft for 20-in.-dia. hole
- — - — Theoretical volume/ft for 18-in.-dia. hole

DRILLED-IN PILE TEST PROGRAM

QUANTITY OF MEASUREMENT OF SOIL EXCAVATED PILE DP3

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-76-C-0005

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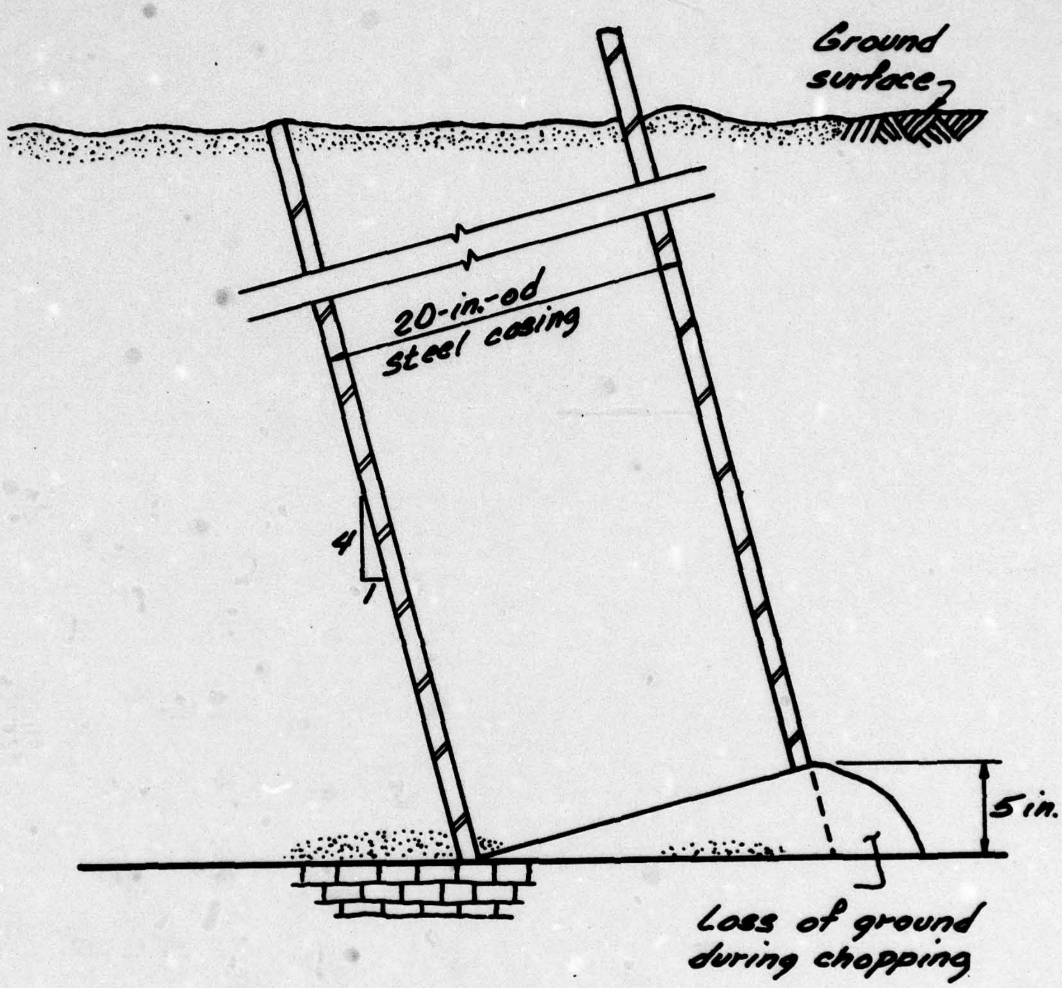
Fig. 5.2

VTC... Phase II



DRILLED-IN PILE TEST PROGRAM

POSSIBLE MECHANISM OF LOSS OF GROUND AT ROCK CONTACT

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0095

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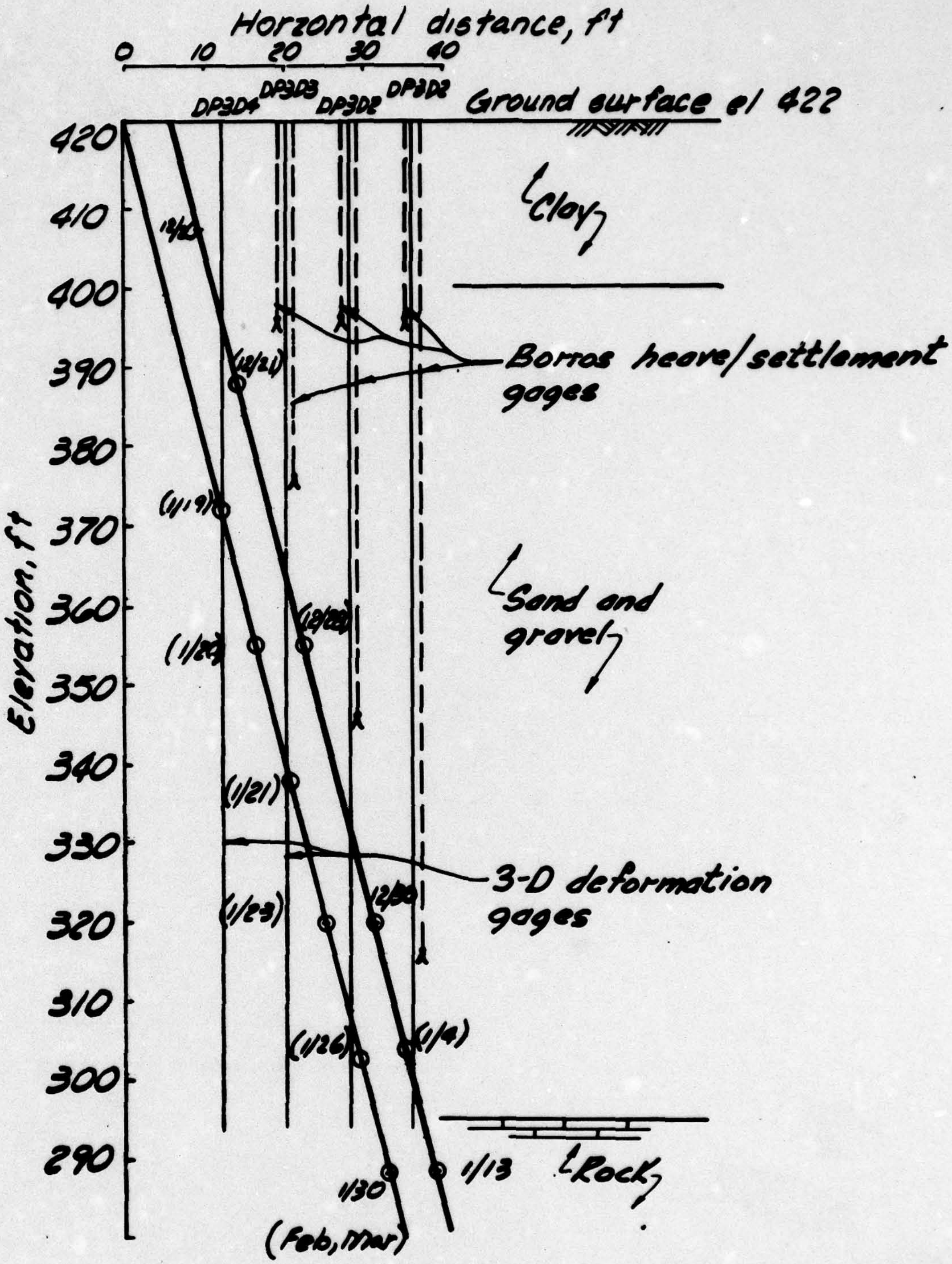
Y7C826 Phase IX

Fig. 5.3



Legend

○ (12/21) Status of excavation when ground movement measurements were made (21 Dec 1978)

DRILLED-IN PILE TEST PROGRAM

SCHEDULE OF PILE INSTALLATION AND GROUND MEASUREMENTS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
[illegible]

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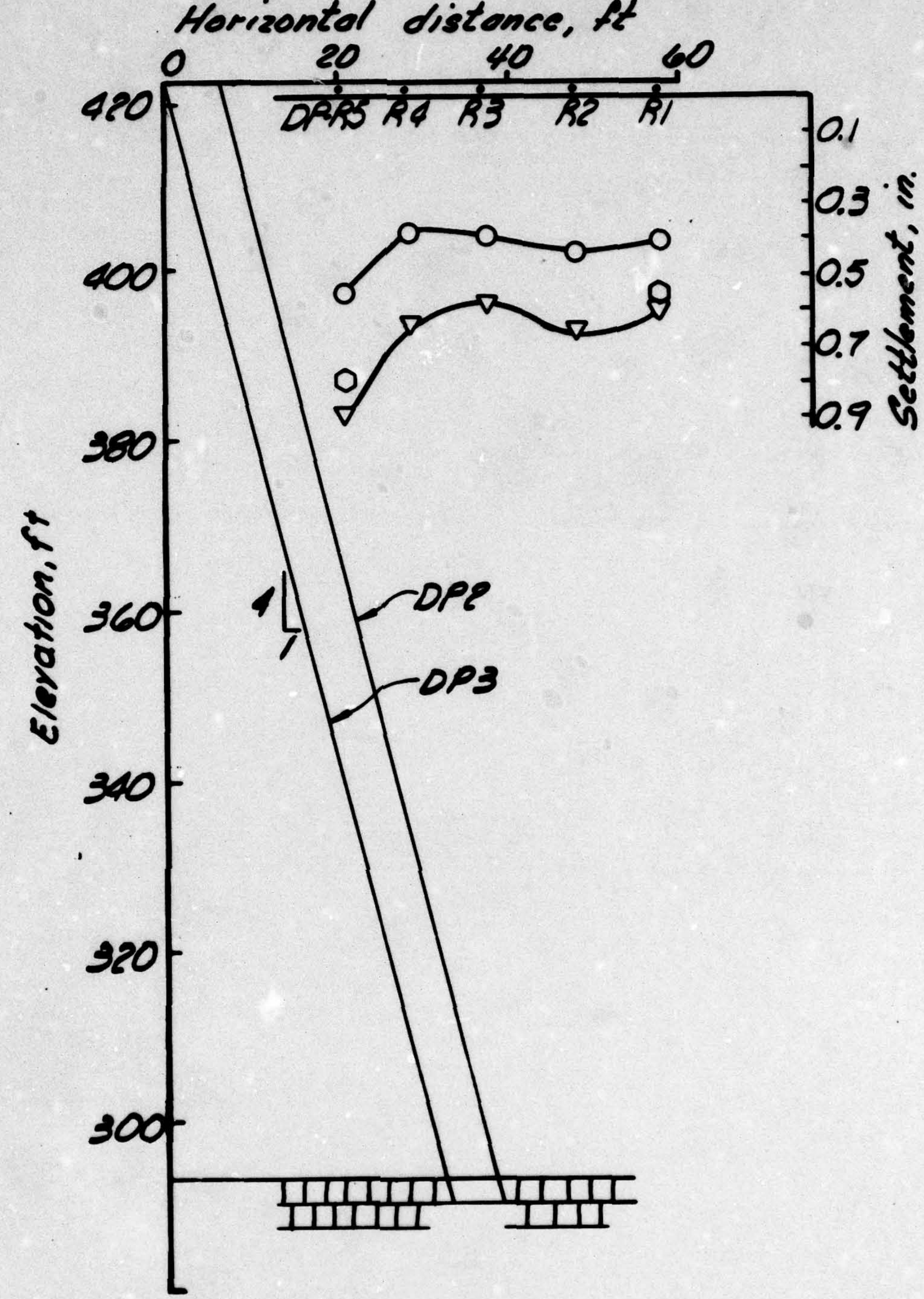
Fig. 5.4



Legend

	Date	Pile No.	Drilled el
○	22 Dec 78	DP-2	389
⬡	16 Jan 79	DP-2	290
▽	26 Jan 79	DP-3	304
——	R1	Surface reference point	

DRILLED-IN PILE TEST PROGRAM

SURFACE SETTLEMENT DURING PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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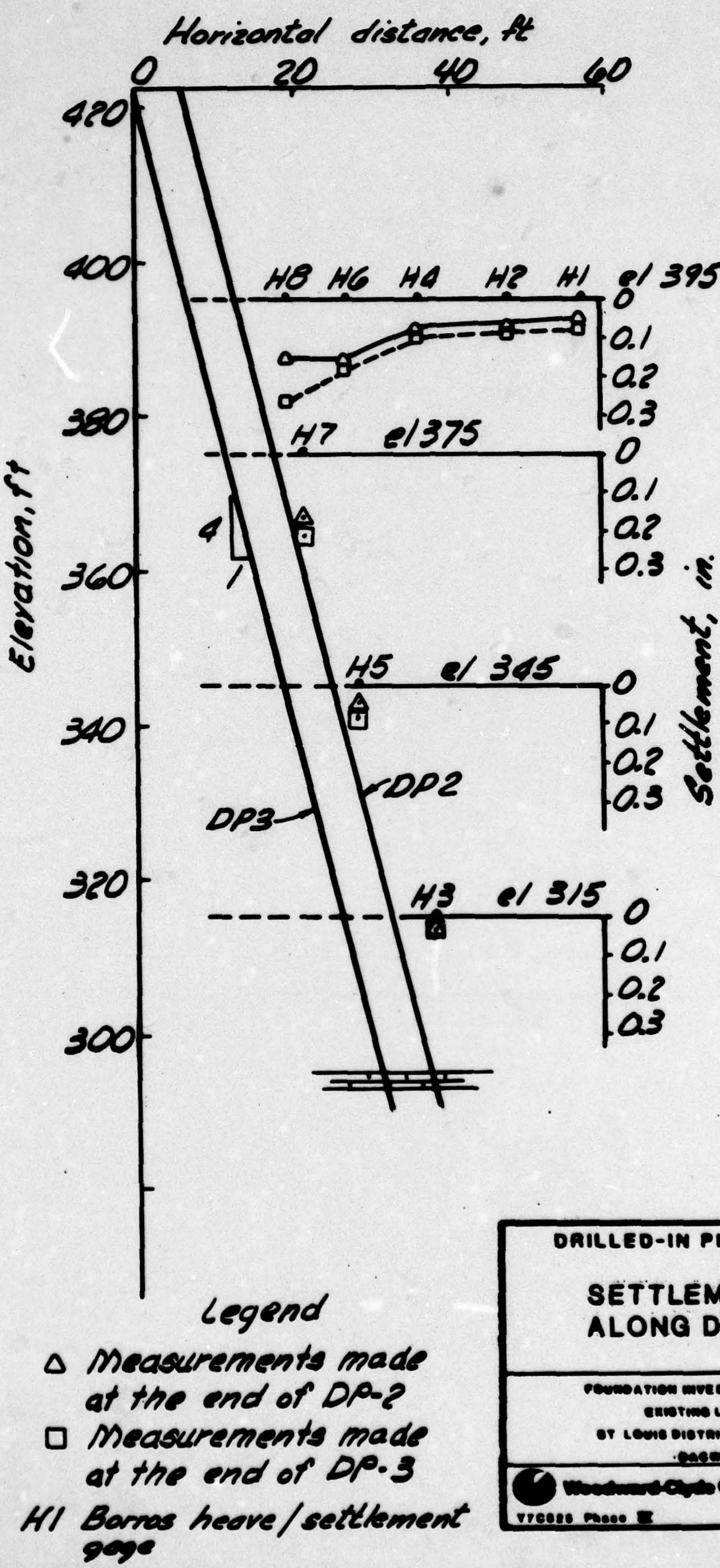
Fig.5.5

V7C826 Phase III



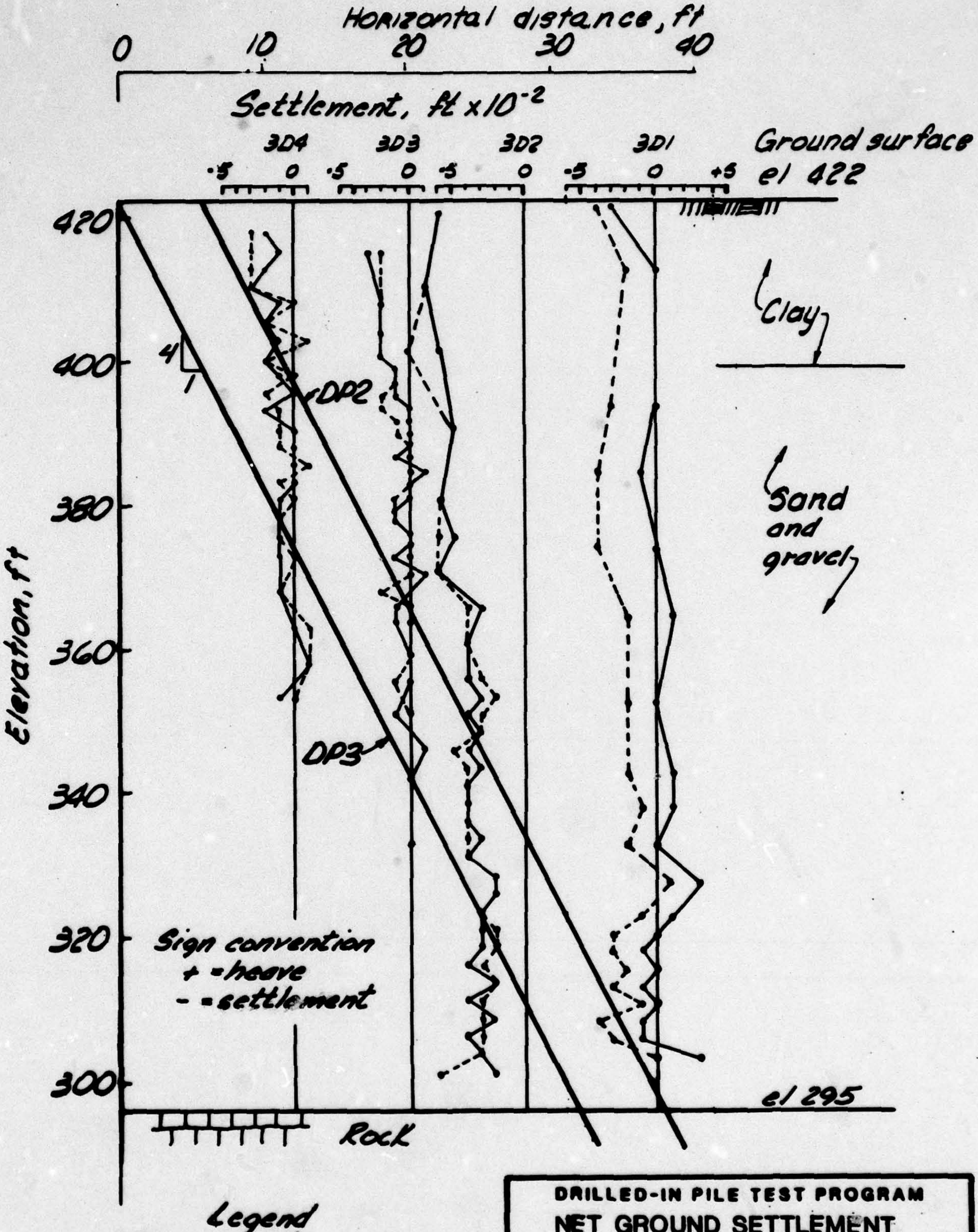
DRILLED-IN PILE TEST PROGRAM

SETTLEMENT PROFILE ALONG DRILLING AXIS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
[illegible]

Woodward-Clyde Consultants
77C626 Phase [illegible]

Fig. 5.6



Legend
—— Measurements made at the end of DP-2
- - - Measurements made at the end of DP-3

Notes:
(1) Settlement referenced to 21 Dec. 1978 measurements
(2) Horizontal scale = 2 x vertical scale

DRILLED-IN PILE TEST PROGRAM

NET GROUND SETTLEMENT AS MEASURED BY SONDEX GAGES 3D-1 THROUGH 3D-4

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-[illegible]

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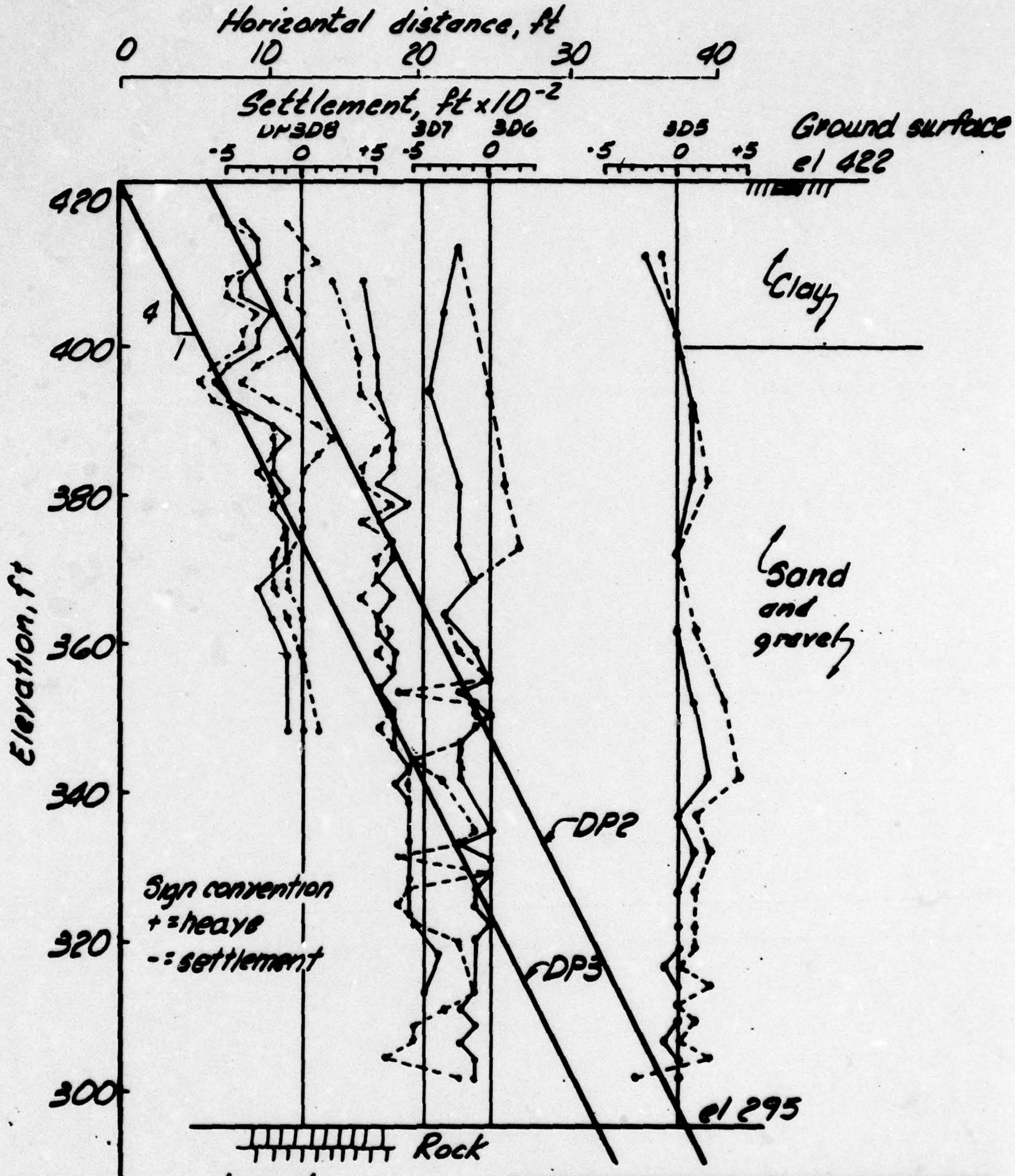
Fig.5.7



Legend

Measurements made after the end of DP-2

Measurements made after the end of DP-3

Notes:

(1) Settlement referenced to 21 Dec 1978 measurements

(2) Horizontal scale = 2 x vertical scale

DRILLED-IN PILE TEST PROGRAM

NET GROUND SETTLEMENT AS MEASURED BY SONDEX GAGES 3D-5 THROUGH 3D-8

FOUNDATION INVESTIGATION AND TEST PROGRAM

EXISTING LOCKS AND DAM No. 26

ST LOUIS DISTRICT, CORPS OF ENGINEERS.

DACW43-78-C-0088

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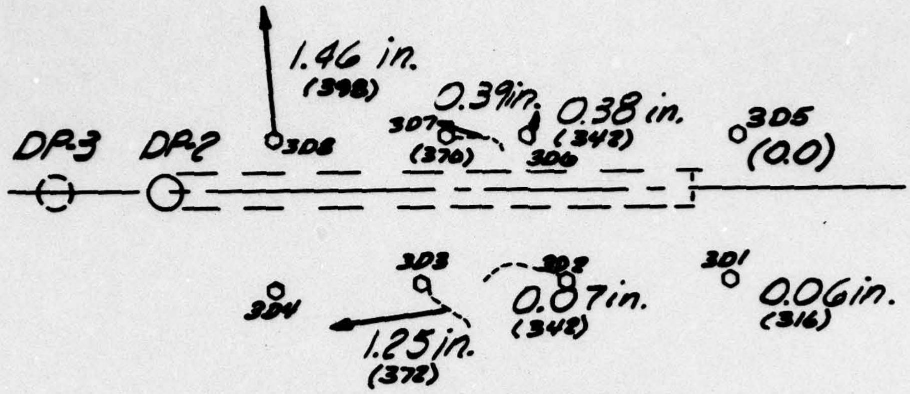
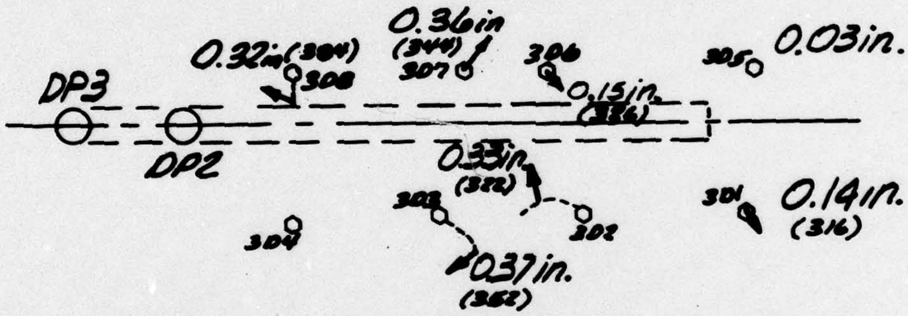
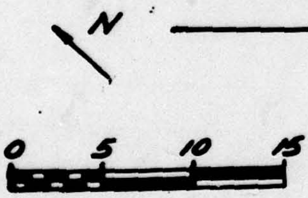
Fig. 5.8



Maximum horizontal ground deformation after DP-2



Maximum horizontal ground deformation after DP-3



Legend

Direction of max resultant horizontal ground deformation

0.37in. Magnitude of max resultant horizontal ground deformation

(362) Elevation of axis of drilling

------ Initial location of inclinometer casing

DRILLED-IN PILE TEST PROGRAM

MAXIMUM HORIZONTAL GROUND DEFORMATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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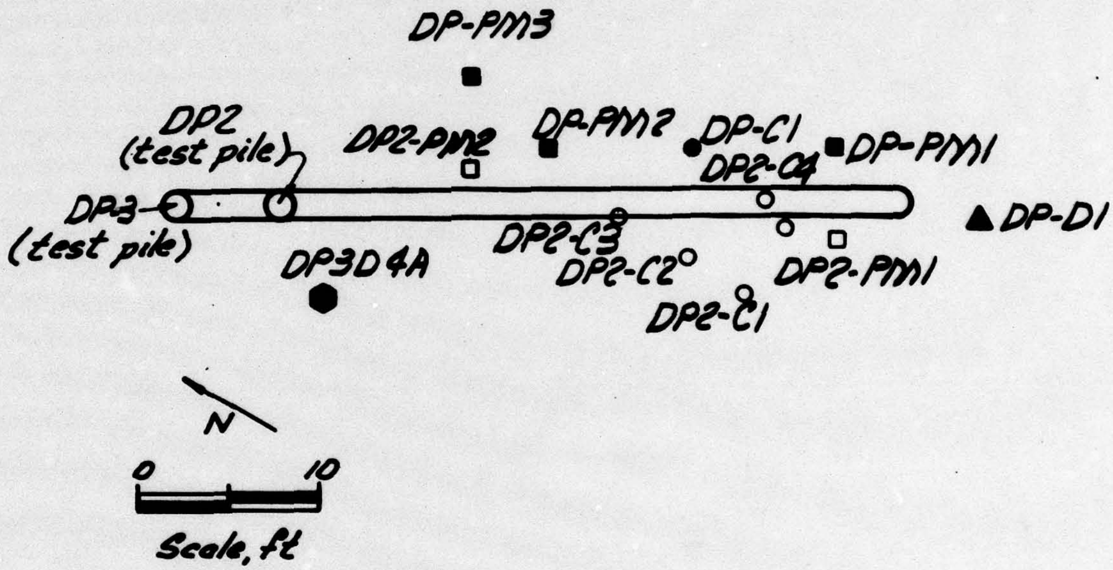
Fig.5.9

77C826 Phase III



Legend

Before pile installation

● Static cone penetration boring

■ Pressuremeter and SPT boring

▲ Nuclear density boring

⬢ Instrumentation installation boring with rock coring

After pile installation

○ Static cone penetration boring

□ Pressuremeter boring and SPT

DRILLED-IN PILE TEST PROGRAM

LOCATION PLAN OF SUBSURFACE INVESTIGATION AFTER PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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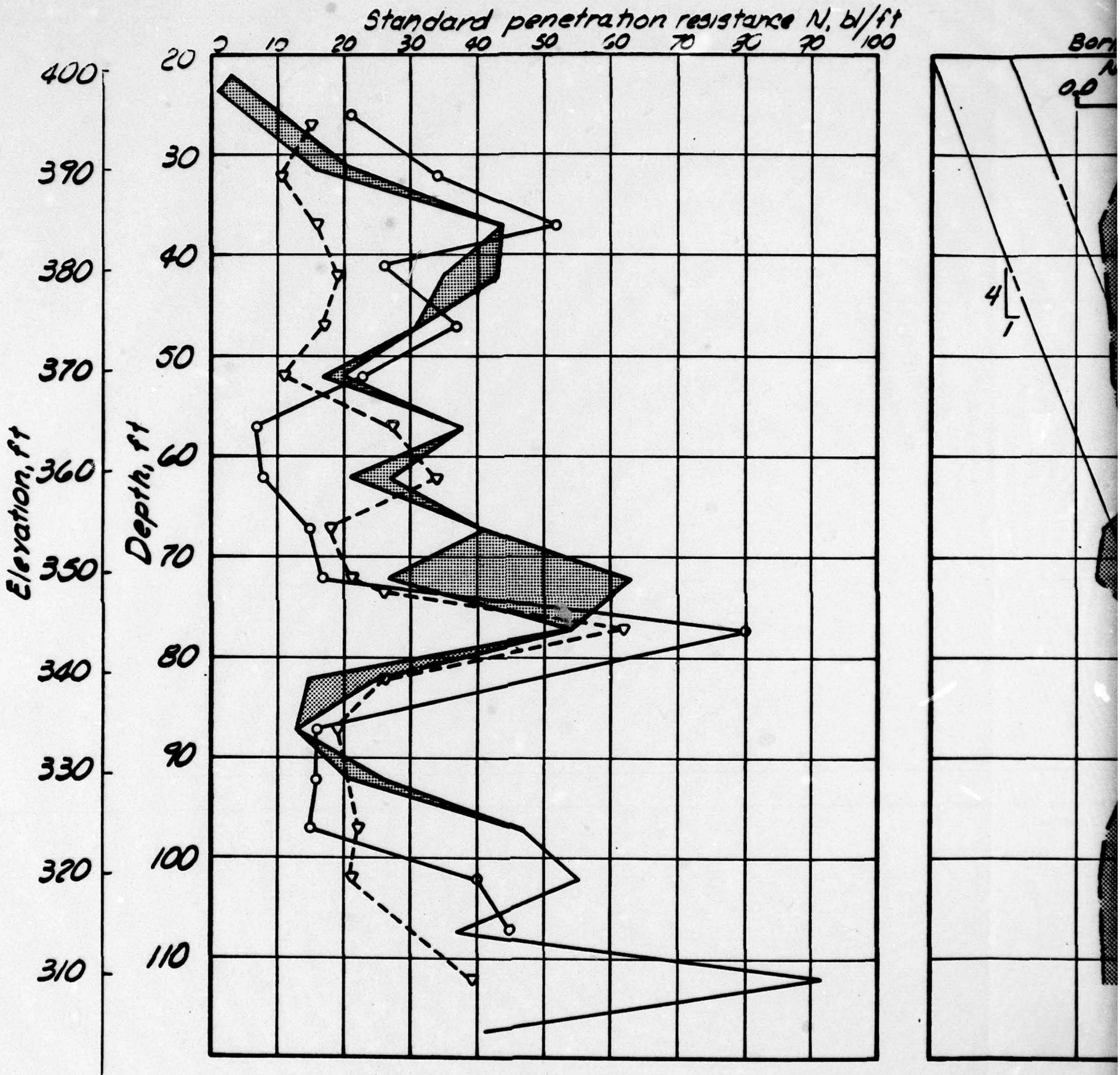
Y7C525 Phase II

Fig.5.10



Legend

Before pile installation

Range of standard penetration resistance

After pile installation

○ Boring DP2-PM1

△ Boring DP2-PM2

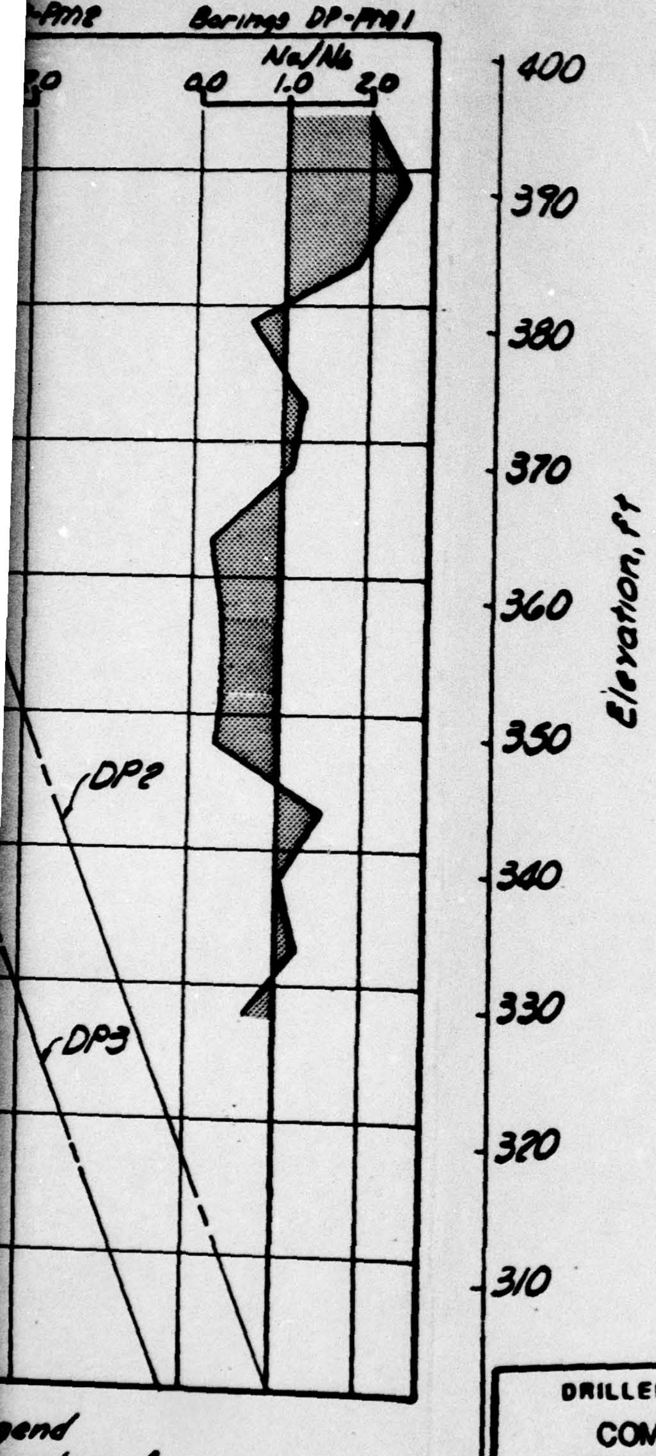
Ratio

--- DP2-

—— DP2-P

Nb; N-valu

Na; N-valu



gend
-values from
and DP-PM3
nd DP-PM1
fore pile installation
r pile installation

DRILLED-IN PILE TEST PROGRAM

COMPARISON BETWEEN STANDARD PENETRATION RESISTANCES BEFORE AND AFTER PILE INSTALLATION

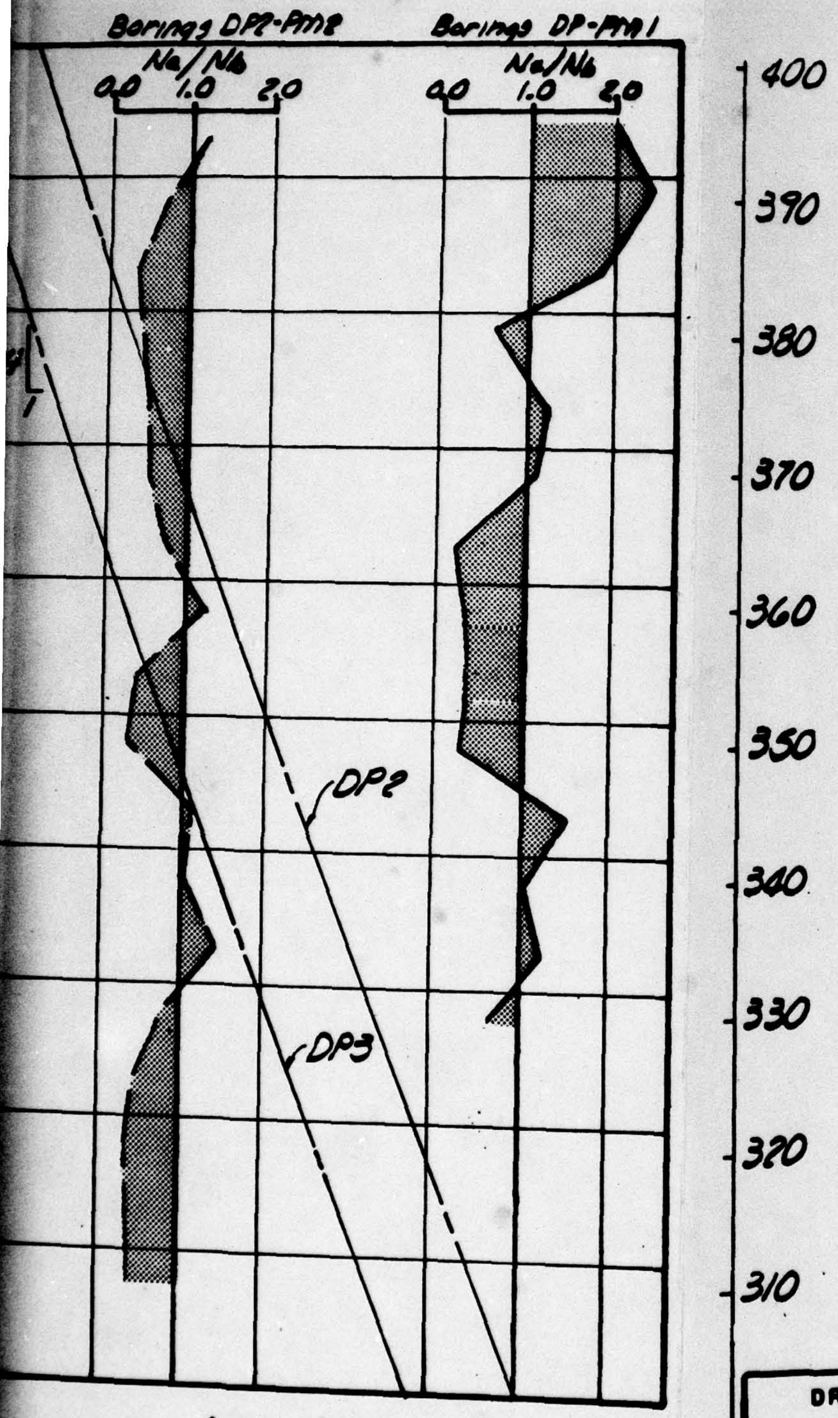
FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0006

Woodward-Clyde Consultants

77C029 Phase II

Fig.5.11

2



Legend
Ratio of N-values from
DP2-PM2 and DP-PM3
DP2-PM1 and DP-PM1
N-value before pile installation
N-value after pile installation

DRILLED-IN PILE TEST PROGRAM

COMPARISON BETWEEN STANDARD PENETRATION RESISTANCES BEFORE AND AFTER PILE INSTALLATION

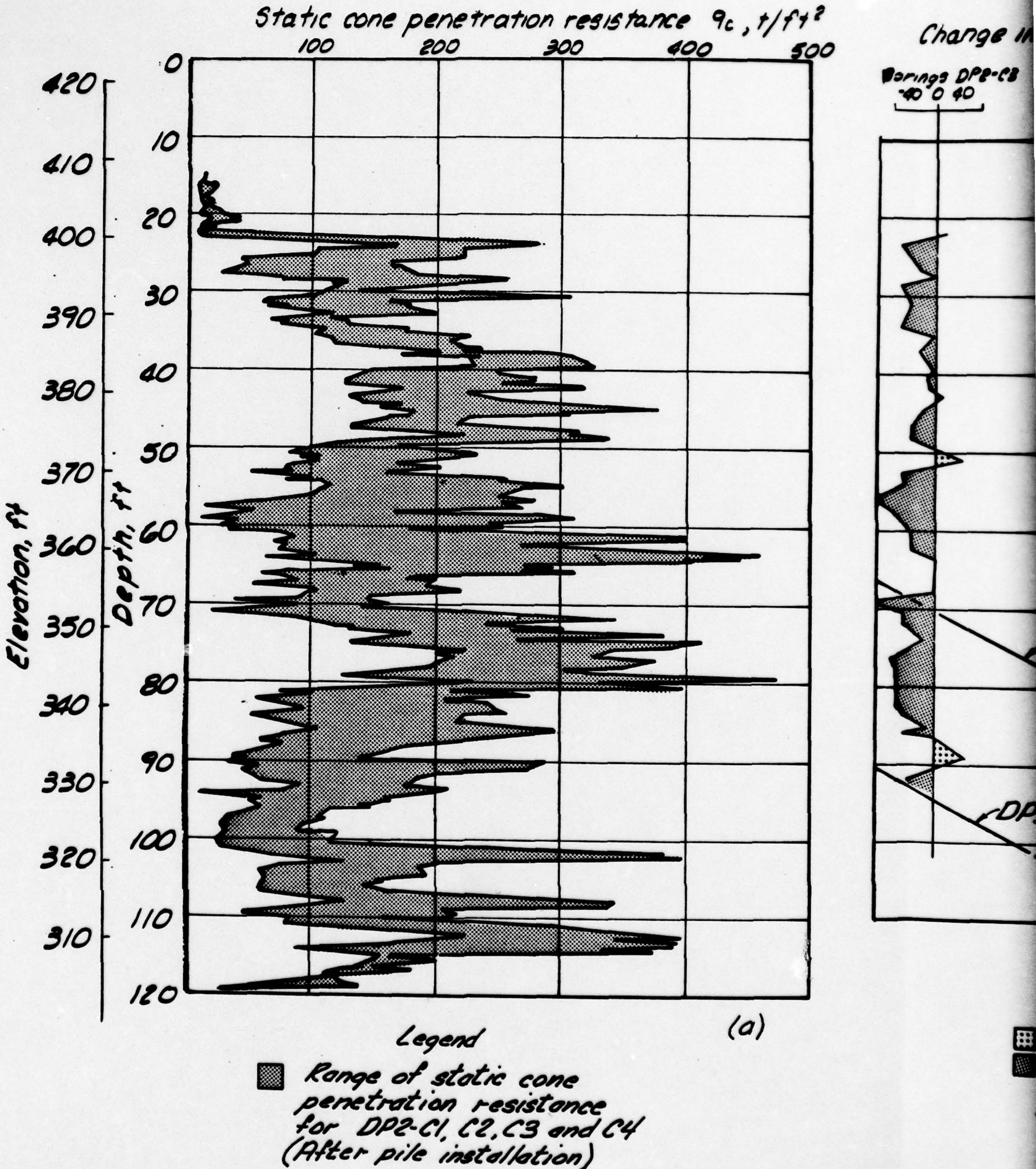
FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0088

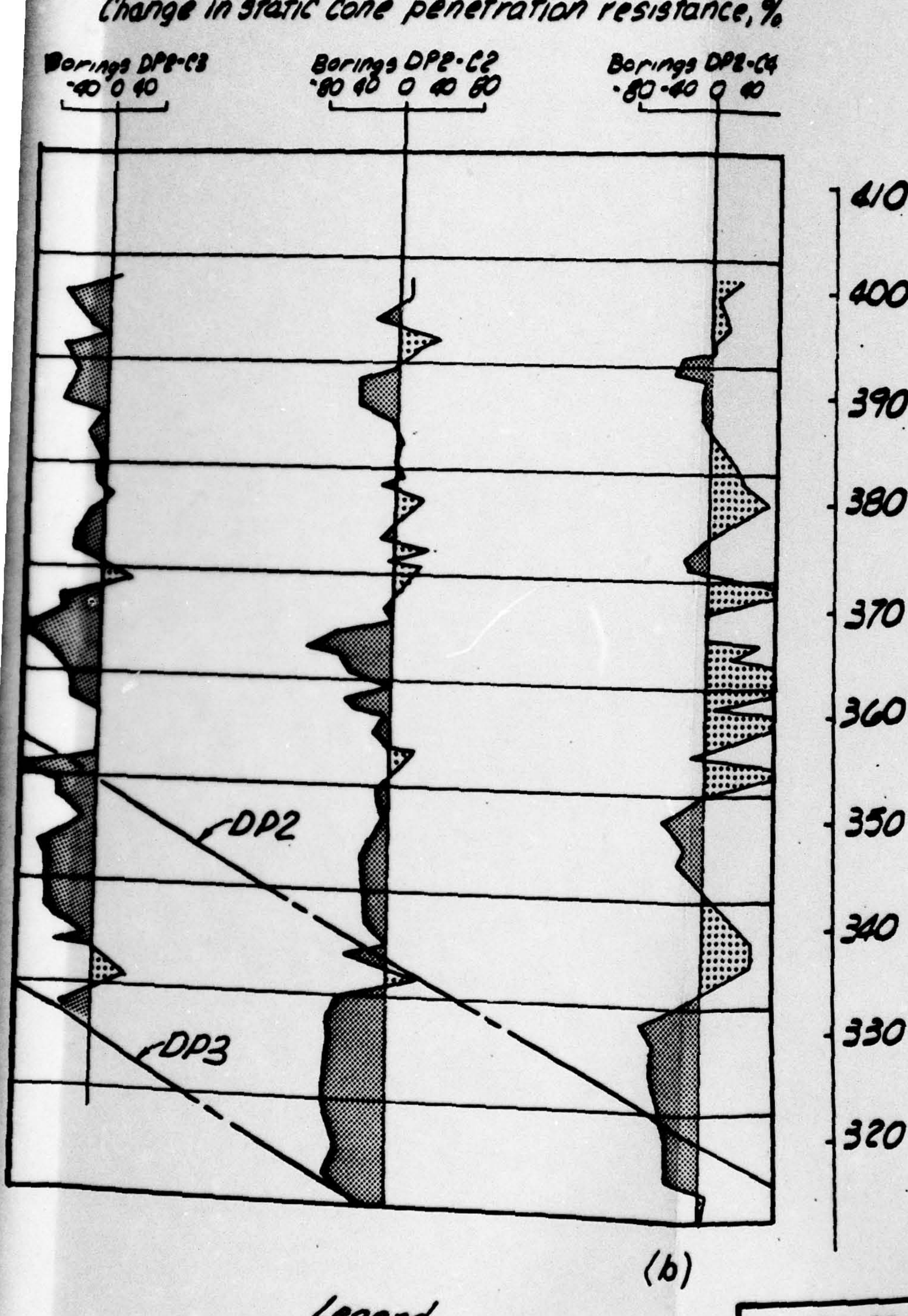
Woodward-Clyde Consultants | Fig.5.11

Y7C525 Phase II

1

2


Static cone penetration resistance q_c, t/ft²
0
100
200
300
400
500
10
20
30
40
50
60
70
80
90
100
110
120
420
410
400
390
380
370
360
350
340
330
320
310
Elevation, ft
Depth, ft
Change in
Borings DP2-C8
-40 0 40
DP
(a)
Legend
Range of static cone penetration resistance for DP2-C1, C2, C3 and C4 (After pile installation)



(b)

Legend

Increase in cone resistance

Decrease in cone resistance

Note

Change in static cone penetration resistance referenced to results from DP2-C1

DRILLED-IN PILE TEST PROGRAM

CHANGE IN STATIC CONE PENETRATION RESISTANCE AFTER PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-75-C-0008

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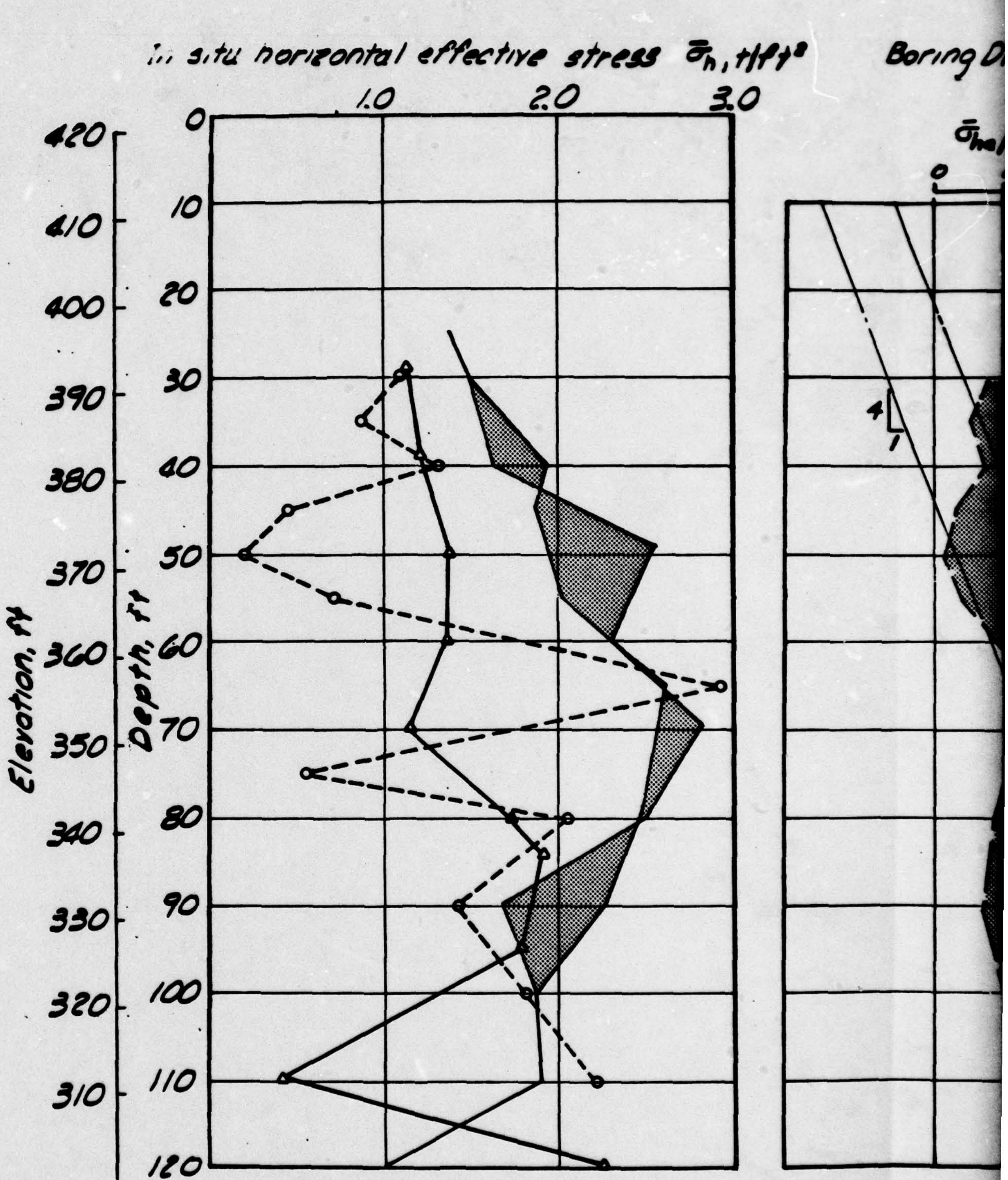
Fig. 5.12

1

2



Legend

Before pile installation

Range from borings DP-PM1 and DP-PM2

After pile installation

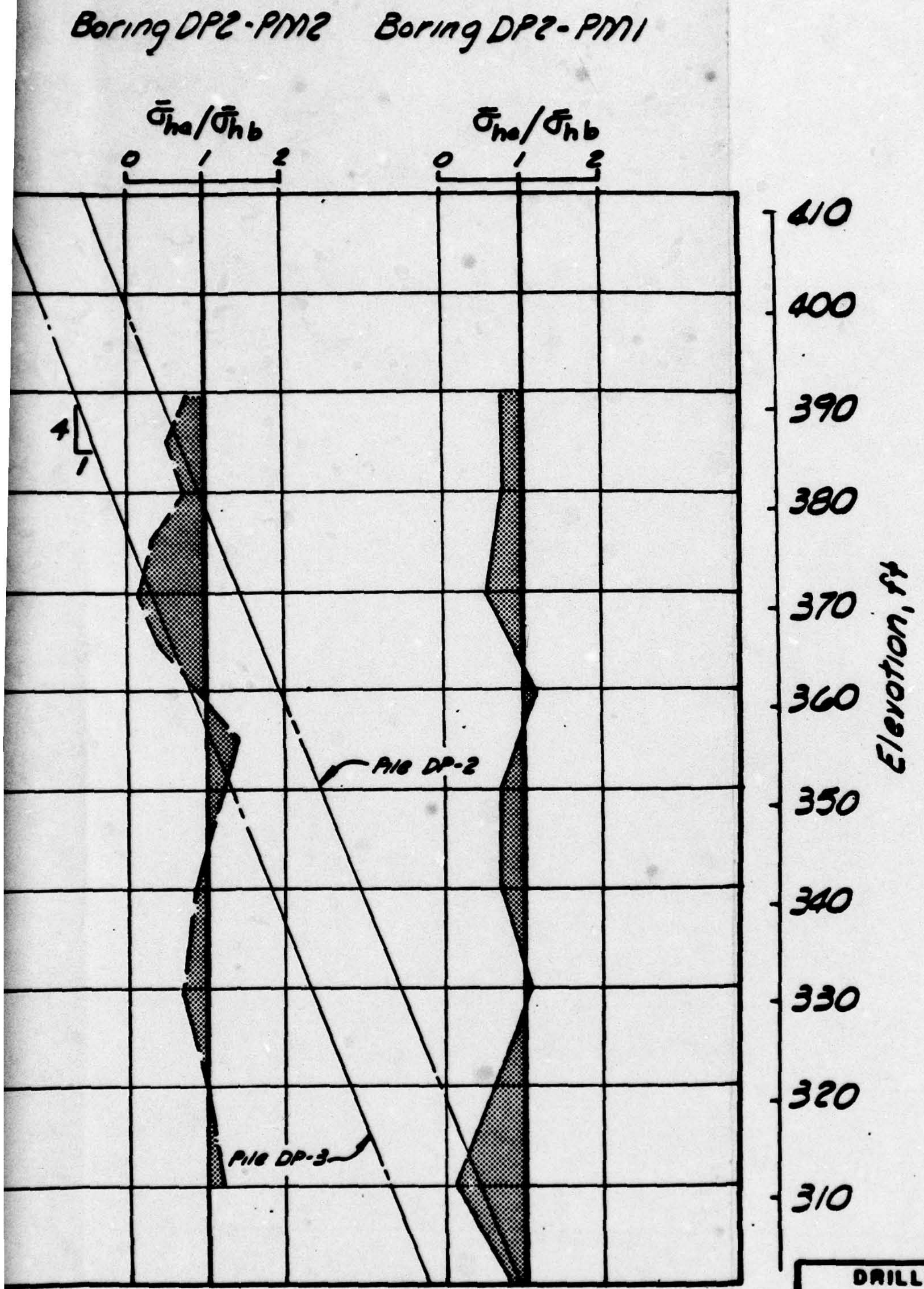
—△— Boring DPZ-PM1

—○— Boring DPZ-PM2

Note: In situ horizontal effective stress inferred from pressuremeter tests

$\bar{\sigma}_{hb_j}$

$\bar{\sigma}_{ha_j}$



Legend
Ratio of $\bar{\sigma}_h$

PM2 --- Borings DP2-PM2 and DP-PM3

—— Borings DP2-PM1 and DP-PM1

$\bar{\sigma}_{hb}$; In situ horizontal stress before pile installation (DP-series borings)

$\bar{\sigma}_{ha}$; In situ horizontal stress after pile installation (DP2-series borings)

DRILLED-IN PILE TEST PROGRAM

COMPARISON BETWEEN IN SITU HORIZONTAL STRESS PROFILE BEFORE AND AFTER PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-75-C-0000

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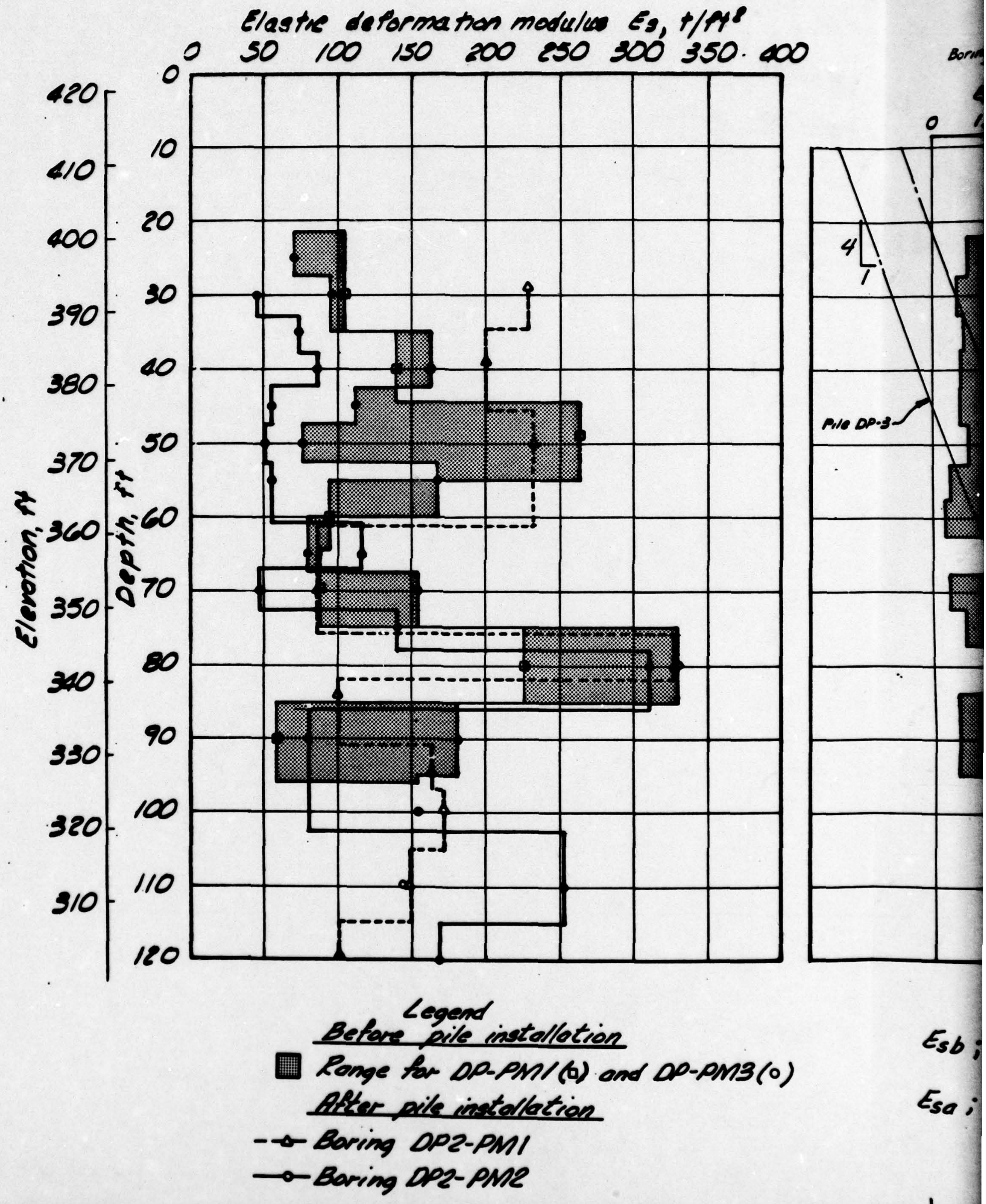
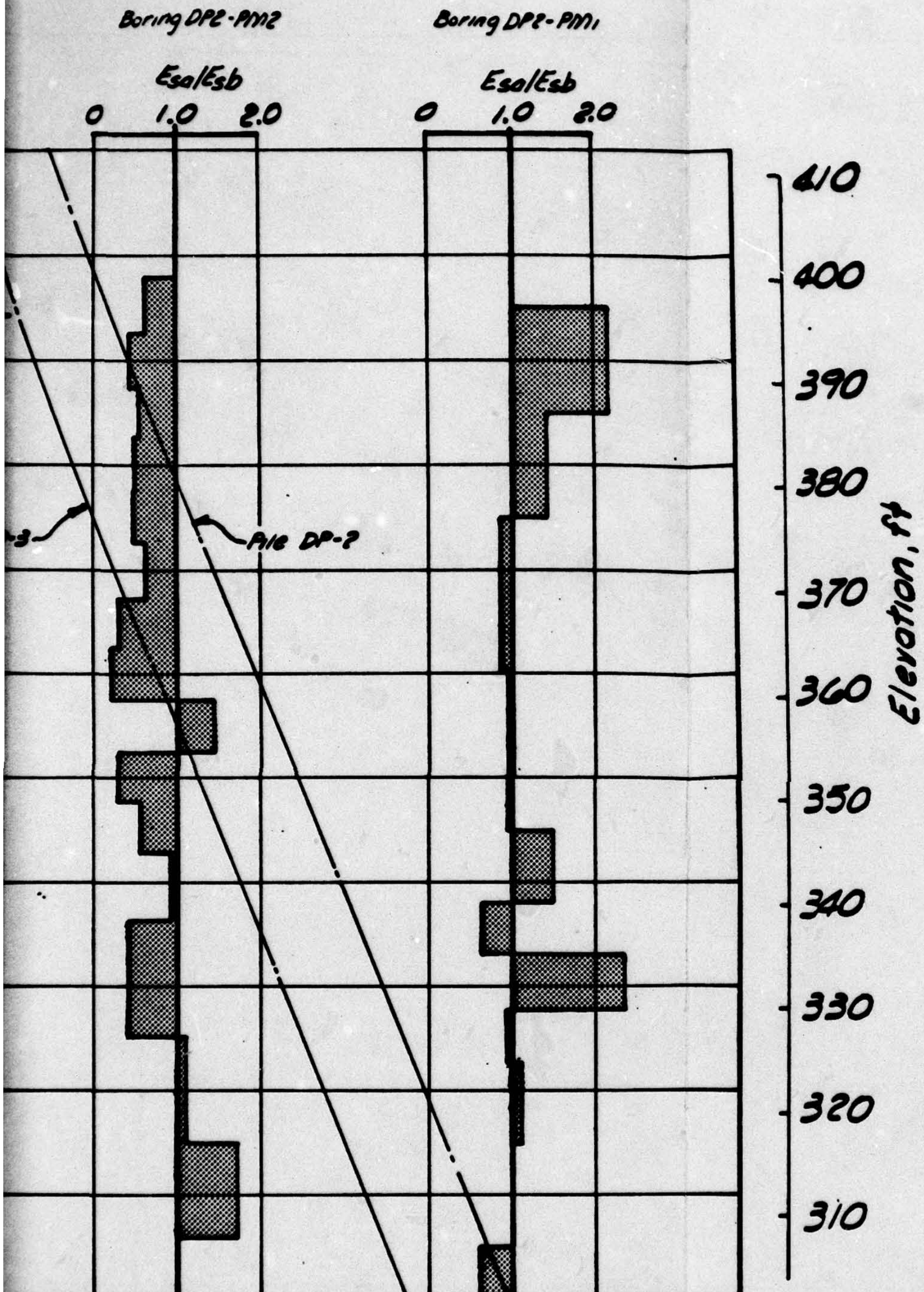
Y7C026 Phase II

Fig.5.13

2


Elastic deformation modulus Es, t/ft²
0 50 100 150 200 250 300 350 400
Elevation, ft
420 410 400 390 380 370 360 350 340 330 320 310
Depth, ft
0 10 20 30 40 50 60 70 80 90 100 110 120
Pile DP-3
4
1
Legend
Before pile installation
Range for DP-PM1(◻) and DP-PM3(○)
After pile installation
Boring DP2-PM1
Boring DP2-PM2
Esb ;
Esa ;



Note:
Elastic deformation modulus inferred from pressuremeter tests

Legend
Esb ; Modulus before pile installation (DP series borings)
Esa ; Modulus after pile installation (DP2 series borings)

DRILLED-IN PILE TEST PROGRAM

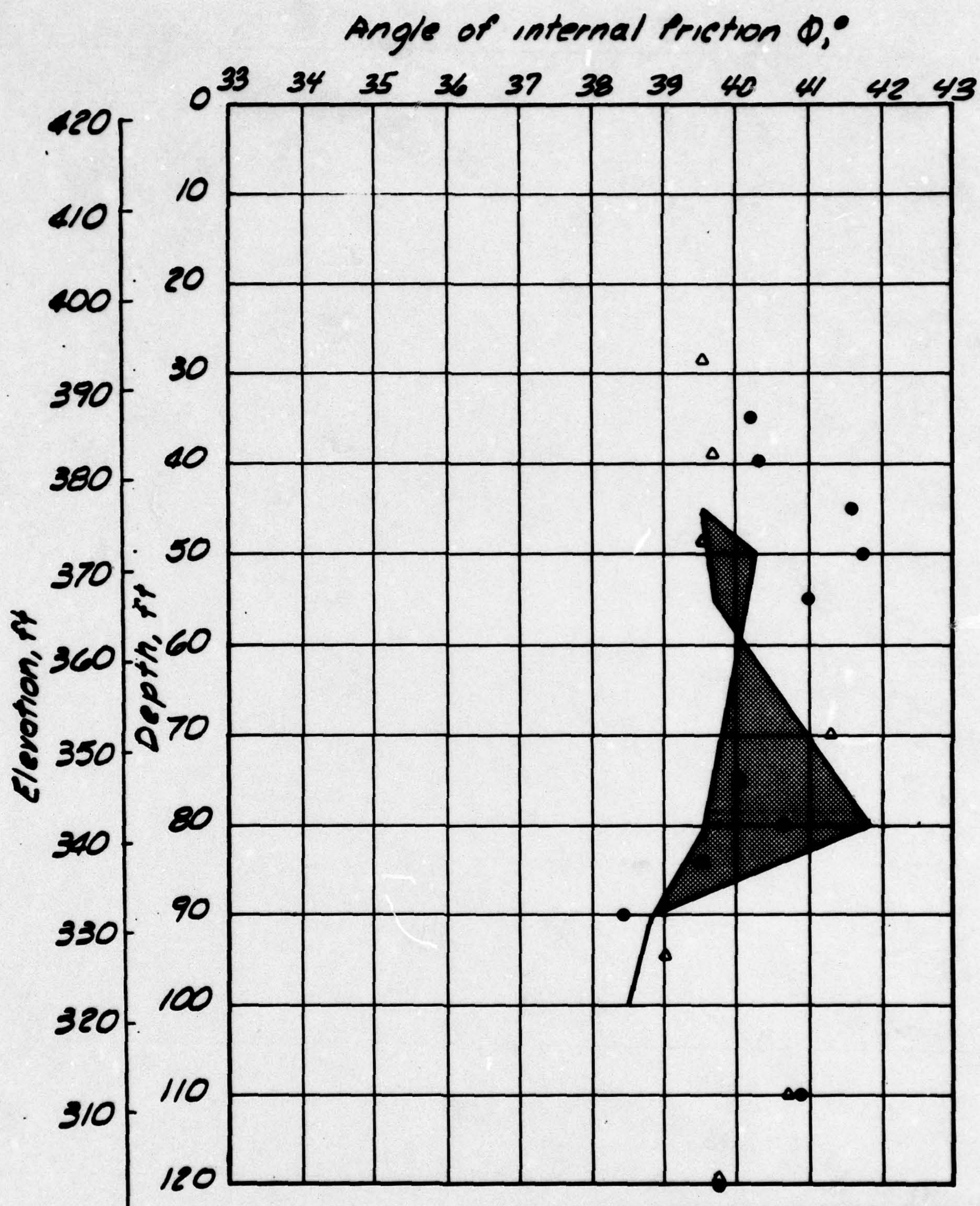
COMPARISON BETWEEN ELASTIC DEFORMATION MODULUS BEFORE AND AFTER PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0085

Woodward-Clyde Consultants
Y7C829 Phase III

Fig. 5.14

1 2



Legend

Pressuremeter tests before pile installation

Range of borings DP-PM1 and DP-PM2

After pile installation

△ Boring DP2-PM1

● Boring DP2-PM2

DRILLED-IN PILE TEST PROGRAM

COMPARISON BETWEEN ANGLE OF INTERNAL FRICTION BEFORE AND AFTER PILE INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM

EXISTING LOCKS AND DAM No. 26

ST LOUIS DISTRICT, CORPS OF ENGINEERS.

DACW43-78-C-0088

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Y7C826 Phase III

Fig. 5.15

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM

SECTION 6

EVALUATION OF EQUIPMENT AND TECHNIQUES

6 EVALUATION OF EQUIPMENT AND TECHNIQUES

6.1 GENERAL

During the progress of the work, detailed notes were kept of observations of the operations. These records consist of drilled-in pile test logs, daily report of activities, and excavation progress reports. The test logs and progress reports for the work on piles DP1, DP2, and DP3 are given in Appendix D, Volume IVA. The information in the progress reports provides a direct relationship between time, depth of pile in the ground, depth to water in the casing, and depth to groundwater.

6.2 SPECIAL DRILLED-IN PILE EQUIPMENT

6.2.1 Benoto Boring Machine

The Benoto EDF boring machine was capable of performing the functions specified for installing drilled-in pipe piles on a pre-selected batter with a minimum of loss of ground. Initially, difficulties were encountered with the equipment because of inexperienced operators; local labor rules did not allow experienced operators from other areas to operate the equipment and therefore, local operators had to be trained to deal with the peculiarities of the equipment such as rate of fall and withdrawal of hammergrab and other features which are learned over extended periods of operation. The inexperience resulted initially in breakdowns which could have been avoided.

Although the machine was equipped to lift casing sections into position, the contractor elected to use a 15-t hydraulic crane for this purpose.

The machine experienced difficulties in advancing the casing after depths of about 100 ft. Due to the substantial frictional resistance at those depths, the gripping collar sometimes would slip on the casing. The welding of steel beads on the inside of the collar was generally effective in preventing slippage. It was also found that the circumferential movement of the gripping collar was not sufficient to break the frictional stress on the pipe adequately. Consequently, the equipment was modified near the end of installation of the second test pile DP2 to double the torque capacity and increase the circumferential movement to about 3 in. Slippage of the gripping collar continued and occasionally the problem was solved by welding it to the casing.

Even though the equipment weighed about 64,000 lb, it was not sufficient to force the casing into the ground while oscillating. The concrete anchor blocks described in Section 2.3.2 were helpful, but did not entirely solve the problem, especially when the plug of soil at the bottom of the casing was more than 2 ft deep.

6.2.2 Excavating Tools

Hammergrabs. The large hammergrab provided initially was too large for the 18-in. diameter pipe; when it was dropped into the water inside the pipe it behaved like a piston striking a fluid. In addition to slowing the velocity of fall and consequently the impact needed to penetrate into the soil, the high pressure exerted on the fluid compacted the soil thus making it more difficult to penetrate. Later, the large hammergrab was modified to reduce its cross section. This helped somewhat; however, excavation progress was still less than expected. It was expected initially that excavation would proceed at an average rate of about 6 ft/hr. The actual rate was 1 ft/hr or less below a depth of 50 ft.

When excavating through gravelly sand, the hammergrab would sometimes grip a gravel in the jaws, thus preventing them from closing completely. As the hammergrab was withdrawn through the water, all the sand would wash out through the partially open jaws resulting in an empty grab. In very dense fine sand, the hammergrab had difficulty in removing material. It is believed part of the problem was insufficient penetration and when the jaws were closed the rush of water out of the double spoon-shaped enclosure would wash out most of the contents. Many times when the hammergrab was withdrawn, it was empty even though the jaws were completely closed.

Rock Chisel. The rock chisel was used in the bottom of the third test pile DP3 to loosen and breakup cobbles and boulders. It was effective for that purpose. It was not effective in breaking up the bedrock.

Rotary Rock Drill. The tricone roller bit operated by the Driltech drill rig was very effective in drilling the rock sockets. The 4 ft 2 in. socket at DP1 was drilled in 33 min, the 5 ft 1/2 in. socket at DP2 in 74 min, and the 5 ft socket at DP3 in 48 min.

Bailing Bucket. After drilling the rock socket with the rotary drill about 3 ft of cuttings remained in the socket. The bailing bucket readily removed these in 4 to 6 buckets full.

6.2.3 Welding

Welding each successive casing section was slow because of the large weld required. Two welders working with two welding machines required about 2 hr to make the weld and another hour to grind the weld flush with the wall of the casing.

6.2.4 Water Supply

After trying various schemes for adding water inside the casing, a 6000-gal tank was eventually brought to the site. The tank provided a ready source of water, which was pumped from the river. Considerable difficulty was experienced with freezing water lines during very cold weather.

6.2.5 Concreting

During concreting of pile DP2, some difficulties arose because the initial tremie bucket used for concrete placement was too small. It was difficult to maintain a full pipe and a blockage occurred. A larger bucket was brought in which was more effective. In both piles the tremie equipment did not function as well as it should.

6.3 MATERIALS

6.3.1 Steel Casing

The casing fulfilled the specification requirements. The heavy wall performed as intended. It was able to withstand twisting back and forth without torsion failure. Observations during installation at depths in excess of 100 ft revealed that in several instances the pipe was twisted about 3/4 in. elastically with probably no movement occurring at the tip. It is likely that had one-half inch wall thickness pipe been used as originally intended, it would not have been possible to install the casing because of excessive elastic torsional deformation or possible yielding.

6.3.2 Rock Socket Reinforcement

The steel H beam stub installed in the rock socket was modified from that specified. There was concern that the placing of tremie concrete in the socket would be extremely difficult because of the limited space for concrete to flow around the H beam. The contractor proposed cutting some holes through the web and in the flanges of the H beam. After considering the cross sectional area of steel required to mobilize 100-t tension capacity, the contractor was allowed to cut two 6-in.-dia holes and one 6-in.-square opening in the web, and eight half circles of 3-in.-radius in the edges of the flanges. No difficulty was experienced in placing and centering the H beam reinforcement in the rock socket.

6.3.3 Concrete

The concrete used for placement in the rock socket did not perform properly. Coring of pile DP2 revealed that most of the cement paste had washed out of the aggregate in the concrete placed in the socket. The core barrel recovered pea gravel rather than a concrete core. This will be discussed in more detail under Section 6.4. It appears that in a restricted area such as the subject socket, it would have been preferable to use pumped-in sand-cement grout to a depth of several feet above the top of the H beam. The water in the casing could have then been pumped out and concrete placed in the dry.

6.4 TECHNIQUES

6.4.1 Casing Alignment

Maintaining the proper alignment of the casing was not difficult. Verticality of each section of pipe was checked with a transit and the leads of the Benoto machine adjusted for correct alignment. A 4-ft-long carpenter's level was also used to check verticality. The batter was checked with the carpenter's level and a wood template cut on a 4 to 1 batter. These procedures were adequate for checking alignment; however, the alignment was not checked frequently enough. During the oscillaton of the casing, very large torques were applied. These often resulted in distortions and movements of the Benoto boring machine which could throw the casing out of alignment. After the casings were installed, measurements were made with a Sinco inclinometer probe lowered in an inclinometer casing placed in the casings. In DP2, the inclinometer casing was installed in a core hole drilled in the concrete placed in the casing. Piles DP1 and DP3 were measured without concrete. The inclinometer was lowered along the lower edge of the pipe casing; there is no assurance that the casing did not drift up the side wall.

The profiles developed by the inclinometer measurements are shown for pile DP1 in Fig. 6.1 and for piles DP2 and DP3 in Fig. 6.2. The measurements indicate that, as installed, none of the piles complied with the specification limit of maximum deviation of 1/4 in. in 5 ft (that is, 6.5 in. deviation at el 290). With frequent checking and careful techniques, the piles probably could have been installed within the specified limits.

6.4.2 Excavation

The rate of excavation was much less than expected and generally unsatisfactory. As described in Section 6.2.2, much of the problem was due to the hammergrab being too large in cross section, with respect to the casing diameter.

At shallow depths when excavating the flood plain deposits above the water table, the wet clay would stick to the jaws when they were opened. A laborer had to scrape the clay out of the jaws at each bite thus delaying progress substantially. Later it was found that by keeping a high water level in the casing which would wet the inside surfaces of the jaws, the clay would no longer stick.

When excavation continued in the sand underlying the flood plain deposits and the hammergrab was falling in air rather than water penetration was good and full jaws were being excavated. This was not the case when the hammergrab was dropped through water. When very slow excavation progress was experienced, different excavating techniques were tried: slow lowering of the hammergrab into the water; delayed jaw-closing after impact; slow closure of jaws; and slow lifting through the water. It was difficult to control the rate of closing of the jaws and waiting after impact was not successful.

At greater depths in DP1 when dense silty sand was excavated, trials were made with shallow depth of water inside the casing. This did not significantly improve excavation rate; however, when the soil plug in the casing was too small, the plug would blow in causing loss of ground.

When very slow excavation was experienced in DP1, some portions of the body of the larger hammergrab were cut away to partially decrease the cross section area and about 130 lb of steel plate were welded to the grab to increase its weight. Because the cross section area was still too large, these modifications were unsuccessful. The smaller hammergrab (grab A) described in Section 2.3.2 was then brought in. This was not more successful because the jaws were small. The larger hammergrab was further modified off-site so it would not act as a piston in the water and was then used to perform most of the remaining excavation in piles DP2 and DP3. The final technique involved relatively slow lowering of the hammergrab into the water so as to minimize the piston effect and slow withdrawal through the water to minimize washing out of fine sand from the jaws.

Excavation of the gravels and cobbles in the Illinoian ice contact deposits above bedrock was slow because single rocks would keep the jaws from closing completely. In pile DP3 when both two- and three-pronged hammergrabs were not effective, the rock chisel was used; by raising and dropping the chisel 3 ft to 5 ft, the cobbles were sufficiently broken so the two-pronged hammergrab could remove the smaller particles. This procedure, however, was slow. The photograph in Fig. 6.3 shows the size of some of the materials removed with the hammergrab.

Excavation of the socket in rock was easily accomplished using the roller bit driven by the rotary drill rig. Attempts to chisel the rock with the rock chisel were unsuccessful. Removal of the rotary drill cuttings was easily done with the bailing bucket.

6.4.3 Control of Water Level and Depth of Soil Plug

Table 6.1 summarizes data collected concerning soil blows into the casing. Most of the blow-ins occurred during the installation of pile DP1 while experimenting with different excavation techniques and varying depths of water and soil plug. The data shown in Table 6.1 are plotted in Fig. 6.4 to evaluate the relationship between the depth of plug and the excess hydrostatic head on the soil plug. When the water level inside of the pipe was above the groundwater table, the head is considered positive. The data in Table 6.1 and Fig. 6.4 show the following:

(1) when the water level inside the casing was below the water table (negative head) blow-in occurred, even when the soil plug was as deep as 8.5 ft;

(2) experiments with negative heads showed that plugs 4 ft deep blowed in when the water level inside the casing was more than 14 ft below the water table;

(3) as a general rule, when the plug was less than 1.5 ft deep, soil had a tendancy to creep in even if there was a positive head in the casing; in two cases creep-in occurred under positive heads of 25 ft when the plug was less than 1 ft;

(4) in the case of negative heads, creep-in occurred when the depth of plug was less than:

$$d = 1.5 - 0.61\ h$$

where: d = depth of plug, ft; and

h = negative head inside the pipe (expressed as a negative number), ft;

(5) when work was stopped for several days, it was desirable to have a plug of at least 5 ft to prevent creep-in under hydrostatic equilibrium ($h = 0$); for future work, if it is not possible to advance the casing that deep, either a higher head should be maintained or soil should be dumped into the pipe; and

(6) as a general rule, if the plug of soil was not less than 1 ft and the water level in the casing was maintained at or above the ground surface blow-in during excavation progress did not occur.

When the soil plug was 6 ft or more, it was virtually impossible to advance the casing. As a general rule, it was found that the plug should not be less than 1 ft and not more than 1.5 ft when advancing the casing. Once casing oscillation had reduced the friction between the soil and casing, it was usually possible to advance the casing until a plug of about 4 ft was developed. At large depths (over 110 ft), it was seldom possible to develop a plug greater than 3 ft.

6.4.4 Concrete Placement

The procedure of placing tremie concrete was not carefully controlled by the contractor, thus resulting in segregation. It is also possible that the holes in the stub core web may have helped cause segregation because the concrete mixture spilling out of the openings would fall freely in water. Even though pea gravel aggregate was used, the concrete would not flow readily through the tremie pipe. After the concreting techniques were modified by using a laydown bucket to place concrete into the tremie hopper, the concrete flowed more freely. Initially, concrete was placed in increments out of the ready-mix truck between removal of tremie pipe sections. The laydown bucket allowed continuous concrete placement because it was not necessary to break the tremie pipe in short sections.

6.5 RATE OF PROGRESS

Excavation progress for test piles DP2 and DP3 is shown in Fig. 6.5 and 6.6, respectively. Elevation of casing bottom, soil surface inside casing, water surface inside casing, and groundwater surface are indicated for given date at 1900 hr. The rate of progress generally decreased as the depth to the casing bottom increased. Between el 420 and el 375, the rate of progress was about 30 ft/day. Between el 375 and el 320, the rate of progress averaged 13 ft/day for DP2 and 15 ft/day for DP3. Below el 320, the rate of progress was approximately 10 ft/day for both DP2 and DP3. These rates of progress are net of any major work stoppage (holidays, major repairs to the equipment); however, they include stoppages for instrumentation measurements and severe weather conditions. In many instances, only a few hours of work were accomplished in any one day due to snow or extreme cold. The Benoto machine was winterized (Fig. 6.7), but the canopies and shelters interfered with efficient operation.

At test pile DP3, the excavation and installation procedures had become fairly routine for the crew, when they were not delayed by bad weather. Accounting for weather delays, the net rate of progress was:

Depth Range ft	Rate of Excavation ft/hr
0 to 50	2.2
50 to 100	1
more than 100	0.6

DATE	TEST PILE NO.	DEPTH TO CASING TIP ft	DEPTH OF SOIL PLUG ft BEFORE BLOW-IN	DEPTH OF SOIL PLUG ft AFTER BLOW-IN	WATER HEAD IN CASING ft
17 Nov 78	DP1	50	2	8.5	- 7.5
20 Nov	DP1	56	4	6	-17
20 Nov	DP1	57	4	6	-14
21 Nov	DP1	65	3.5	17.1	-36
21 Nov	DP1	67.5	8.5	10.0	-12.5
25 Nov	DP1	72.5	2.8	3.3	-11.8
25 Nov	DP1	73.5	2.2	4	- 2.5
12 Dec	DP1	75	4	6 (1)	0
13 Dec	DP1	81	0	1.5	2.5
15 Dec	DP1	98	2.6	5.25 (2)	13.9
17 Dec	DP1	116	1	2.50 (6)	13
18 Dec	DP1	116	0.25	1.2	16.5
18 Dec	DP1	116.5	0.40	2.25	6
11 Jan 79	DP2	131.2	0.9	1.7 (3)	28
12 Jan	DP2	131.7	0	1	29
18 Jan	DP3	30	0.7	3.9 (4)	28
22 Jan	DP3	86	2.2	3.8 (5)	14
26 Jan	DP3	122	0.9	4	25
27 Jan	DP3	125.6	0.7	2.1	25

Notes

(1) Creep-in and deposition of suspended sediment of 1.5 ft occurred between 25 Nov and 12 Dec during a period of no activity at DP1

(2) Loosening of 0.9 ft occurred during advancing of the casing from 98 ft to 99.75 ft over a period of 20 minutes. Due to difficulty of advancement, pipe was lifted several times about 2 inches to 3 inches before advancing

(3) Creep-in and deposition of sediment of 0.8 ft between 6 Jan and 11 Jan during rig repair

(4) Loosening of 0.4 ft during casing advance

(5) Creep-in and deposition of fine suspended sediment of 1.6 ft from 2400 on 20 Jan to 0800 on 22 Jan

(6) Creep-in and deposition of sediment of 1.5 ft over 28-hr period

DRILLED-IN PILE TEST PROGRAM

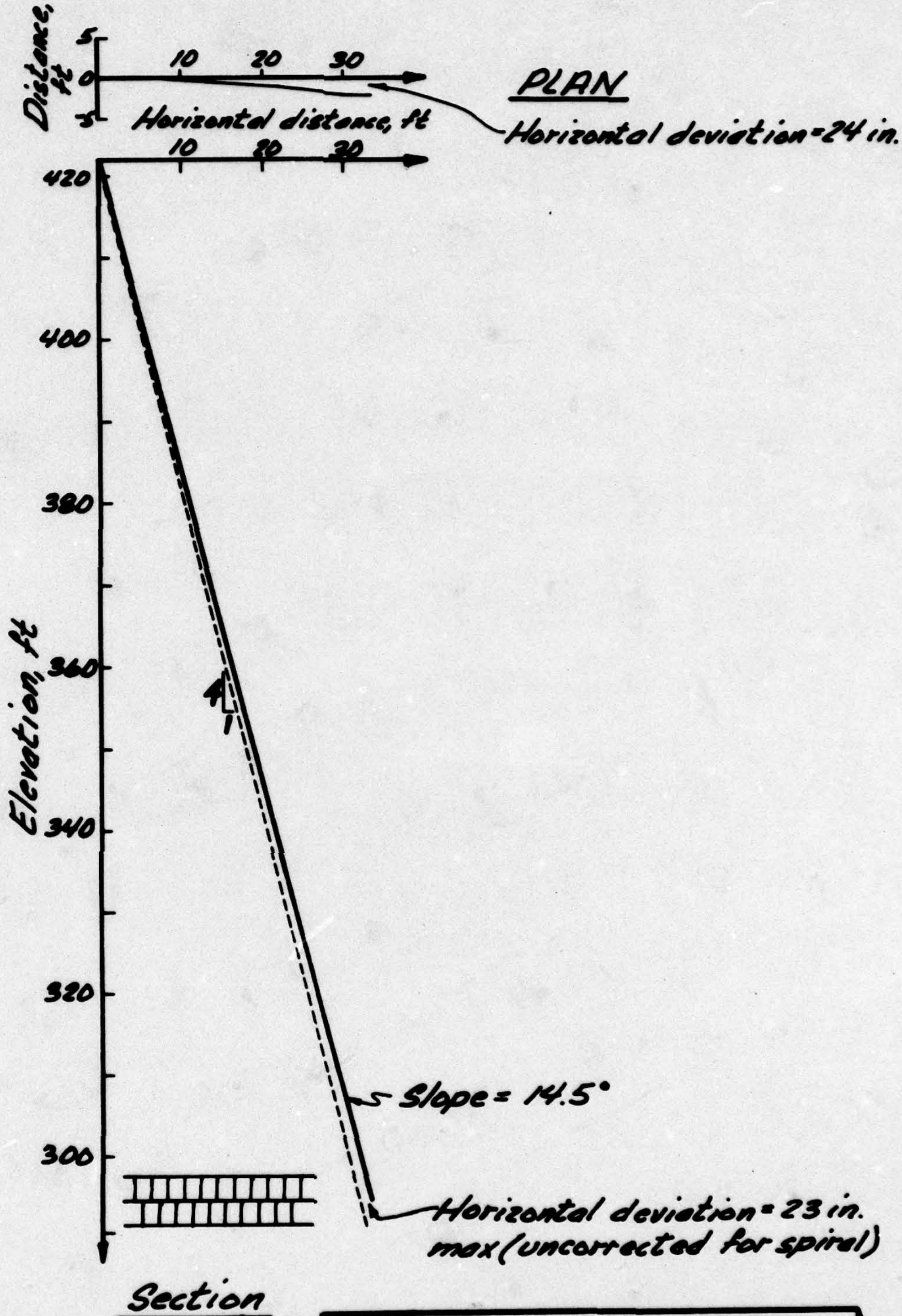
SUMMARY OF DATA RELATED TO SOIL BLOWS INTO CASING

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

77C626 Phase III

Table 6.1



Legend
- - - Theoretical alignment
—— As-built alignment

DRILLED-IN PILE TEST PROGRAM

AS-BUILT ALIGNMENT OF PILE DP-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-76-C-0006

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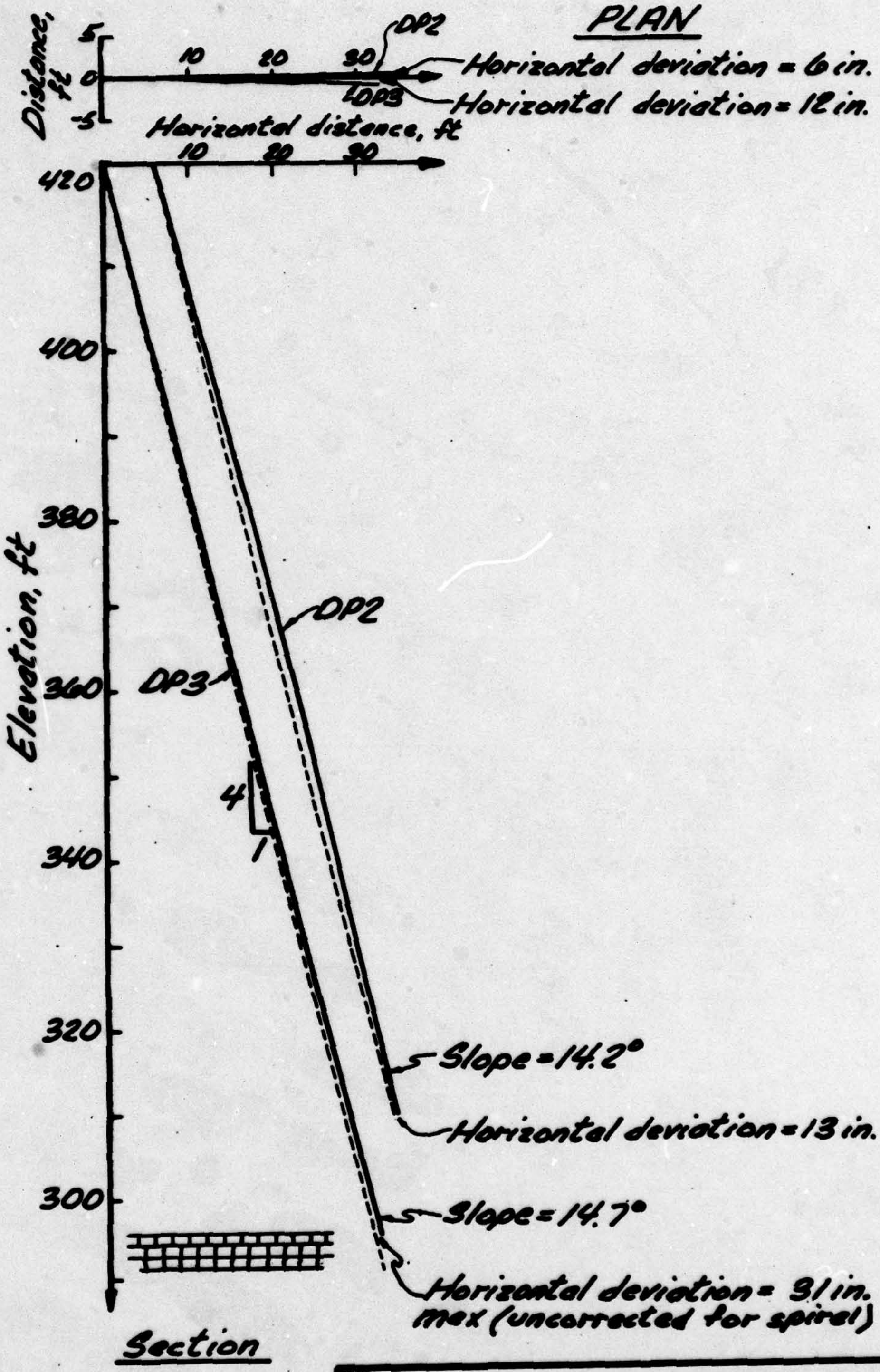
V7C025 Phase III

Fig. 6.1



Legend
- - - Theoretical alignment
——— As-built alignment

DRILLED-IN PILE TEST PROGRAM

AS-BUILT ALIGNMENT OF PILES DP-2 AND DP-3

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
[illegible]

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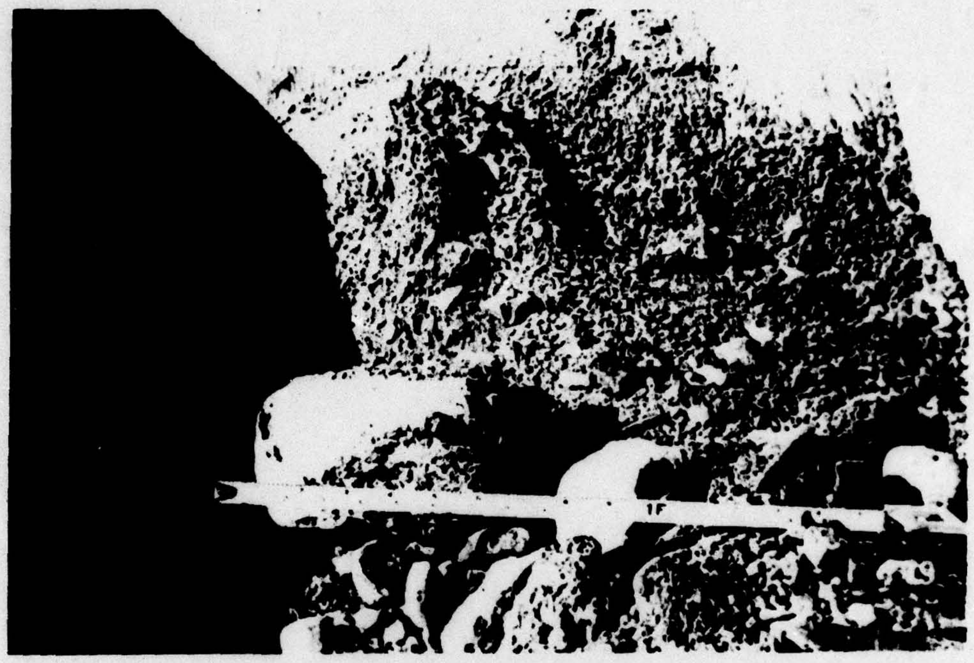
77C828 Phase II

Fig. 6.2



DRILLED-IN PILE TEST PROGRAM

EXCAVATED GRAVEL AND COBBLE-SIZED MATERIAL FROM PILE DP-3

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0000

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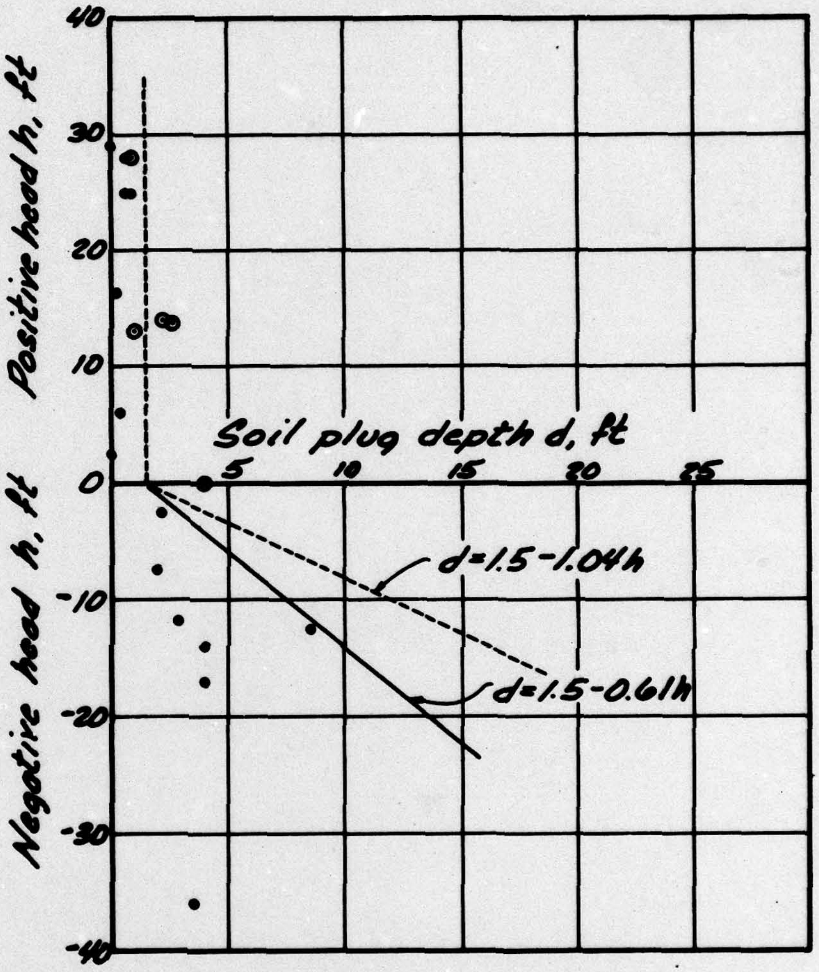
Fig.6.3



Note:
Saturated unit weight of soil
γ_{sat} = 122.4 lb/ft^3

Legend
- ⊙ Observed creep-in during prolonged shutdown
- • Observed blow-in.

DRILLED-IN PILE TEST PROGRAM

PLUG DEPTH VS HEAD DIFFERENTIAL AT BLOW-IN

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

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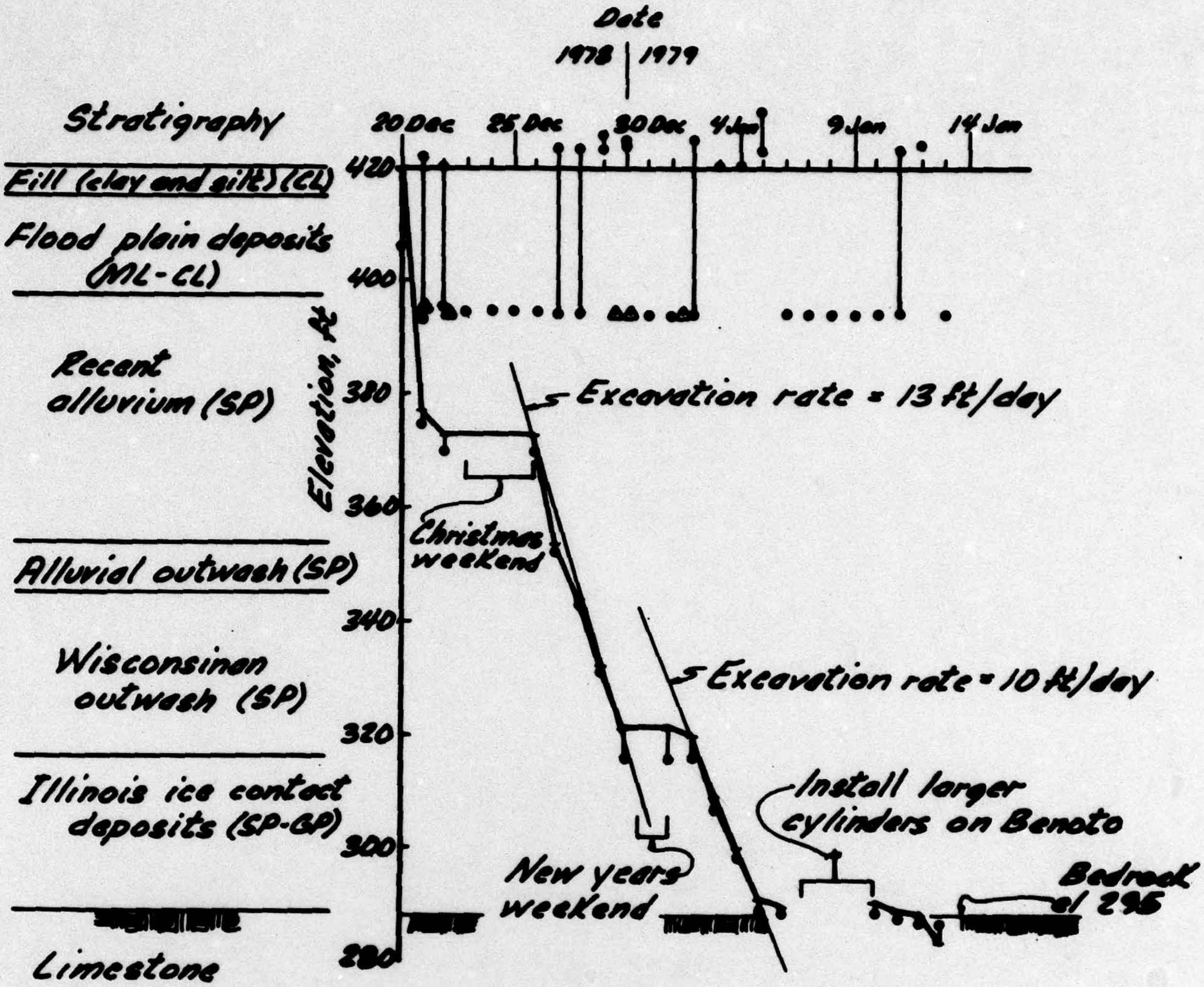
Y7C025 Phase III

Fig.6.4



Legend

- Elevation of excavated soil surface inside casing
- Elevation of casing tip
- Elevation of water in casing (maximum and minimum measured depths)
- Elevation of ground-water surface

DRILLED-IN PILE TEST PROGRAM

EXCAVATION PROGRESS FOR TEST PILE DP-2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-[illegible]

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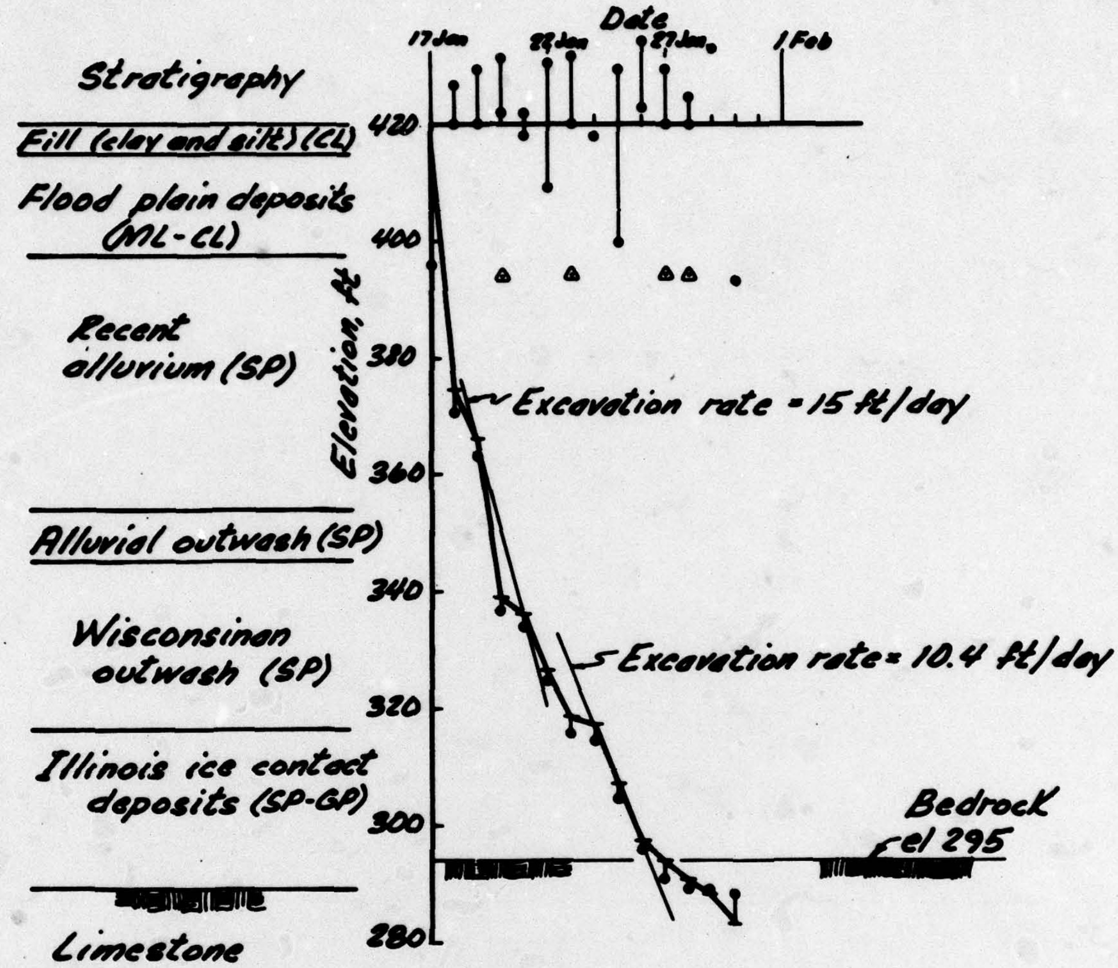
Y7C025 Phase II

Fig. 6.5



Legend

- Elevation of excavated soil surface inside casing
- Elevation of casing tip
- Elevation of water in casing (maximum and minimum measured depths)
- Elevation of ground-water surface

DRILLED-IN PILE TEST PROGRAM

EXCAVATION PROGRESS FOR TEST PILE DP-3

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

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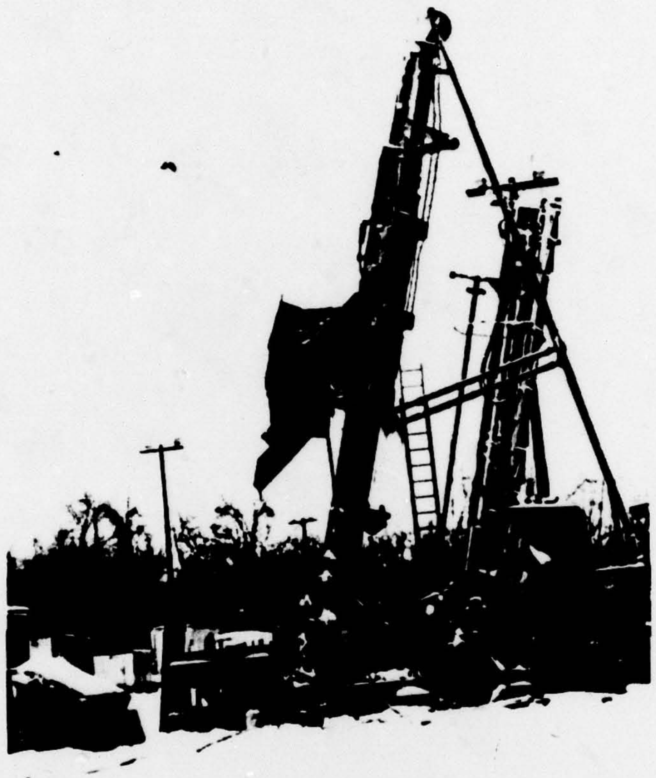
Fig.6.6

Y7C628 Phase III



DRILLED-IN PILE TEST PROGRAM

WINTERIZED BENOTO MACHINE

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

Y7C025 Phase IX

Fig.6.7

PHASE IV REPORT

VOLUME IV

RESULTS AND INTERPRETATION OF DRILLED-IN PILE TEST PROGRAM

SECTION 7

COST INFORMATION

7 COST INFORMATION

7.1 GENERAL

This section presents an analysis of estimated cost of installation of drilled-in piles by the method used in the test program. The costs presented are based on the assumption that construction will proceed at the rate attained at test pile DP3 utilizing the same type of equipment and crew. Unit costs for materials, equipment and labor are typical of the St Louis area in early 1979. The construction schedule used for this cost analysis is one 10-hr shift per day and 22 working days per month.

7.2 PRODUCTION RATE

For the purpose of estimating an installation cost per pile, it is assumed that the average rate of pile installation is 2.2 ft/hr for the first 50 ft; 1 ft/hr for the second 50 ft; and 0.5 ft/hr below a depth of 100 ft. Therefore, it would require approximately 130 hr or 13 days to install one pile, including rock socket (average production rate 8 ft/day). The total average time for completion of one drilled-in pile, including all activities, is as follows:

Activity	Time Shifts
Drilling (soil)	12.5
Drilling (rock)	0.5
Concreting	0.5
Moving to next pile	0.5
Total time per pile	14

7.3 EQUIPMENT AND LABOR

The following equipment was used for construction of the drilled-in piles in the test program. Listed with the equipment are monthly rental rates and corresponding shift rates.

Equipment	Rate Per Month $	Rate Per Shift $
Benoto Boring Machine, EDF55	10,000	455
Drill Rig, Driltech D-40K	16,000	727
Hydraulic Crane, 15-t	2,400	109
Welding Machine, 250-amp	580	26.50

The construction trades which participated in the drilled-in pile installation and their respective rates are as follows:

Labor	Rate Per Hour $	Rate Per Shift $
Operator (2 drums)	18.30	183
Oiler	16.00	160
Laborer	15.70	157
Welder	21.70	217

The above equipment and labor rates include 30 percent for contractor's overhead and profit.

7.4 MATERIALS

It is assumed that 20-in.-dia, 1.03-in.-thick steel casing will be installed. This casing weighs 206.33 lb/ft. The steel stub core weighs 57 lb/ft and is 9 ft long. The volume of concrete needed to fill the casing and the socket is approximately 9 yd^3. In summary, the cost of materials for one drilled-in pile is:

Material	Quantity	Unit Cost $	Total Cost $
Steel casing	26820 lb	0.60	16,092
Steel stub core	513 lb	0.60	308
Concrete	yd^3	50	450
TOTAL MATERIAL COST			16,850

7.5 COST ANALYSIS

For one 130-ft-long drilled-in pile at a batter of 4 (vert) to 1 hor), using production rates experienced and labor, equipment, and material requirements during the test program, the following costs are estimated.

7.5.1 Drilling Through Soil (12.5 shifts)

Item	Quantity	Rate Per Shift	Cost	
Equipment				
Benoto rig	1	455	5,688	
hydraulic crane	1	109	1,363	
welding machine	2	26.50	663	
	Total equipment cost per pile		7,714	
Labor				
operator	2	183	4,575	
oiler	1	160	2,000	
laborer	2	157	3,925	
welder	2	217	5,425	
	Total labor cost per pile		15,925	
Material				
casing (130 ft)			16,092	
TOTAL COST FOR DRILLING THROUGH SOIL				$ 39,731

7.5.2 Drilling Through Rock (0.5 shift)

Item	Quantity	Rate Per Shift	Cost	
Equipment				
hydraulic crane	1	109	55	
Drill rig	1	727	364	
	Total equipment cost per pile		419	
Labor				
operator	2	183	183	
oiler	1	160	80	
laborer	2	157	157	
	Total labor cost per pile		420	
Materials			None	
TOTAL COST FOR DRILLING THROUGH ROCK				$ 839

7.5.3 **Concreting (0.5 shift)**

	Item	Quantity	Rate Per Shift	Cost
Equipment				
hydraulic crane	1	109	55	
	Total equipment cost per pile		55	
Labor				
oiler	1	160	80	
laborer	3	157	236	
	Total labor cost per pile		316	
Materials				
steel stub core			308	
concrete			450	
	Total material cost per pile		758	
TOTAL COST FOR CONCRETING				$ 1,129

7.5.4 **Moving to Next Pile (0.5 shift)**

Item	Quantity	Rate Per Shift	Cost	
Equipment				
Benoto rig	1	455	228	
hydraulic crane	1	109	55	
	Total equipment cost per pile		283	
Labor				
operator	2	183	183	
oiler	1	160	80	
laborer	3	157	236	
	Total labor cost per pile		519	
Materials			None	
TOTAL COST FOR MOVING				$ 802

7.5.5 Cost Summary

Activity	Cost $
Drilling through soil	39,731
Drilling through rock	839
Concreting	1,129
Moving to next pile	802
GRAND TOTAL COST PER PILE	$42,501

This cost represents a unit cost of approximately $330/ft of drilled-in pile.

The costs presented above reflect the conditions experienced during the test program. Although actual production rates were modified to account for atypical delays inherent in the tests, the total cost per pile is still believed to represent an upperbound value. It is likely that the estimated total cost could be decreased by some unknown amount and be more representative of large scale production work.

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