A STUDY OF DRILLED SHAFTS CONSTRUCTED BY THE SLURRY DISPLACEMENT METHOD

BRIDGE DIVISION
TEXAS HIGHWAY DEPARTMENT

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A STUDY OF DRILLED SHAFTS CONSTRUCTED BY THE SLURRY DISPLACEMENT METHOD

by

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Conducted by

Bridge Division
Texas Highway Department

in cooperation with the
United States Department of Transportation
Federal Highway Administration

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PREFACE

This report presents a detailed description of all work done relative to the construction and test loading of three drilled shafts constructed in Houston, Texas, using a "slurry displacement" method of construction. It is hoped that the information contained herein will be useful to others who choose to use this method for drilled shaft construction.

This report is the result of the combined efforts of many people. The design and construction of the test shafts were under the general supervision of Mr. A. C. Kyser, former Engineer-Manager of the Houston Urban Office. The design work was supervised by Mr. W. V. Ward and construction was supervised by Mr. R. A. Vansickle and Mr. F. W. Geron. The soils investigations and load tests were under the supervision of Mr. G. P. Berthelot. Mr. Horace Hoy made significant contributions toward developing and implementing the design procedures which resulted from these tests.

Major contributions were made by Professor L. C. Reese and Mr. Fadlo Touma of the University of Texas Center for Highway Research. They were responsible for constructing and installing the instrumentation, collecting and analyzing the test data, and reporting the results to the Texas Highway

Department. Much of the material in this report was taken from that reported to the Texas Highway Department by Messrs. Reese and Touma and from field notes taken on the job by Mr. Touma.

Farmer Foundation Co. of Houston, Texas, was the contractor for this work and the fine cooperation of Mr. Glyen Farmer and his personnel is greatly appreciated.

ABSTRACT

A "slurry displacement" method for constructing drilled shafts in water bearing and/or caving soils without the use of casing was proposed by foundation drilling contractors for use in the Houston, Texas area. This is a method whereby the sides of the excavation are supported by a mud slurry which is subsequently displaced by the fluid concrete that forms the completed drilled shaft. Three instrumented full-size shafts were constructed in Houston to evaluate this method of construction. The shafts were then test loaded to determine the maximum load capacity and load transfer characteristics of each shaft. Upon completion of the load tests, the shafts were removed from the ground, inspected, and pull-out tests performed on two reinforcing bars.

Results of the tests and inspections performed indicate the following:

1. Drilled shafts can safely be constructed using the "slurry displacement" method of construction.
2. For the design of drilled shafts in the Houston area, it can safely be assumed that the shear strength developed is 0.7 times

the shear strength of the soil as determined from Triaxial and/or THD Cone Penetrometer Tests.

3. A significant portion of the load applied to a drilled shaft is transferred to the surrounding soil through skin friction: 88% for shaft G1, 95% for shaft G2, and 61% for shaft BB.

4. The frictional load capacity of drilled shafts constructed in the Houston area is independent of the construction method used.

CHAPTER I

INTRODUCTION

The use of drilled shafts as foundations for highway structures has experienced a rapid and unprecedented growth during the past two decades. Through several years experience, the Texas Highway Department has demonstrated that the use of drilled shafts offers a significant economic advantage over the use of driven piles in soils where holes can be drilled without the use of specialized drilling techniques. These soils, generally referred to as well-behaved soils, are mostly stiff clays, shales, and cemented sands.

Results of a comprehensive research study of the load transfer characteristics of drilled shafts, Research Study No. 3-5-65-89, revealed that further economic benefits could be realized through utilization of the ability of a drilled shaft to transfer load to the surrounding soil by skin friction as well as through point bearing. This resulted in two major benefits: (1) the belled footings required for point bearing design can be eliminated in many cases, and (2) drilled shafts may be used in areas where soil conditions are not suitable for drilling and therefore require the use of driven piles.

Early in 1971, the Texas Highway Department began investigating the feasibility of using straight drilled shafts in other soils, not so well-behaved, such as sands, silts, and soft clays. One promising method suggested by Houston area foundation contractors for constructing shafts in these materials was a "slurry displacement" method. This is a construction method whereby the sides of the excavation are supported by a mud slurry which is subsequently displaced by the fluid concrete that forms the completed drilled shaft. Similar methods have been used in the United States to construct cofferdams, retaining walls, and building foundations.

A series of load tests on full-size shafts constructed by the "slurry displacement" method was planned to evaluate the method. Specific objectives of the tests were as follows:

1. Verify the calculated load capacity for straight shafts constructed by this method.
2. Obtain information necessary for the preparation of construction specifications for this method.
3. Establish a relationship between the measured frictional load capacity of a shaft constructed

by this method and the frictional load capacity calculated from soils data.

The test shafts were to be loaded to failure and then extracted from the ground for inspection.

Methods for instrumenting drilled shafts were developed by the Center for Highway Research of the University of Texas as a part of Research Study No. 3-5-65-89. Through an Interagency Cooperation Contract the Center for Highway Research furnished personnel under the supervision of Professor L. C. Reese to fabricate and install instrumentation in the three test shafts, collect data during load tests, analyze the data, and report the results to the Texas Highway Department.

A contract was awarded Farmer Foundation Co. of Houston, Texas in July, 1971, for construction of the three test systems and extraction of the test shafts after all load tests were completed.

CHAPTER II

DESCRIPTION OF TEST SITES

Location

Three locations in Houston, Texas, were selected for the construction of the test shafts (Fig. 2.1). Two of these sites, G 1 and G 2, are located in the area of a proposed interchange at the intersection of Interstate Highways 45 and 610 (South Loop). The third site, BB, is located on the north bank of Brays Bayou at the proposed crossing of State Highway 288. These sites were chosen because soil conditions prohibited the use of drilled shafts designed to carry load in point bearing only. Successful completion of these tests would permit the use of straight drilled shafts rather than the more expensive driven piles. Furthermore, the caving conditions expected at these sites would provide a good test for the "slurry displacement" method of construction.

Soil Exploration

Three test borings were drilled very near the location of the test shafts. THD cone penetrometer data were taken from one hole; undisturbed samples for triaxial testing, supplemented by THD cone penetrometer data, were taken in one hole, and standard penetration tests were run in

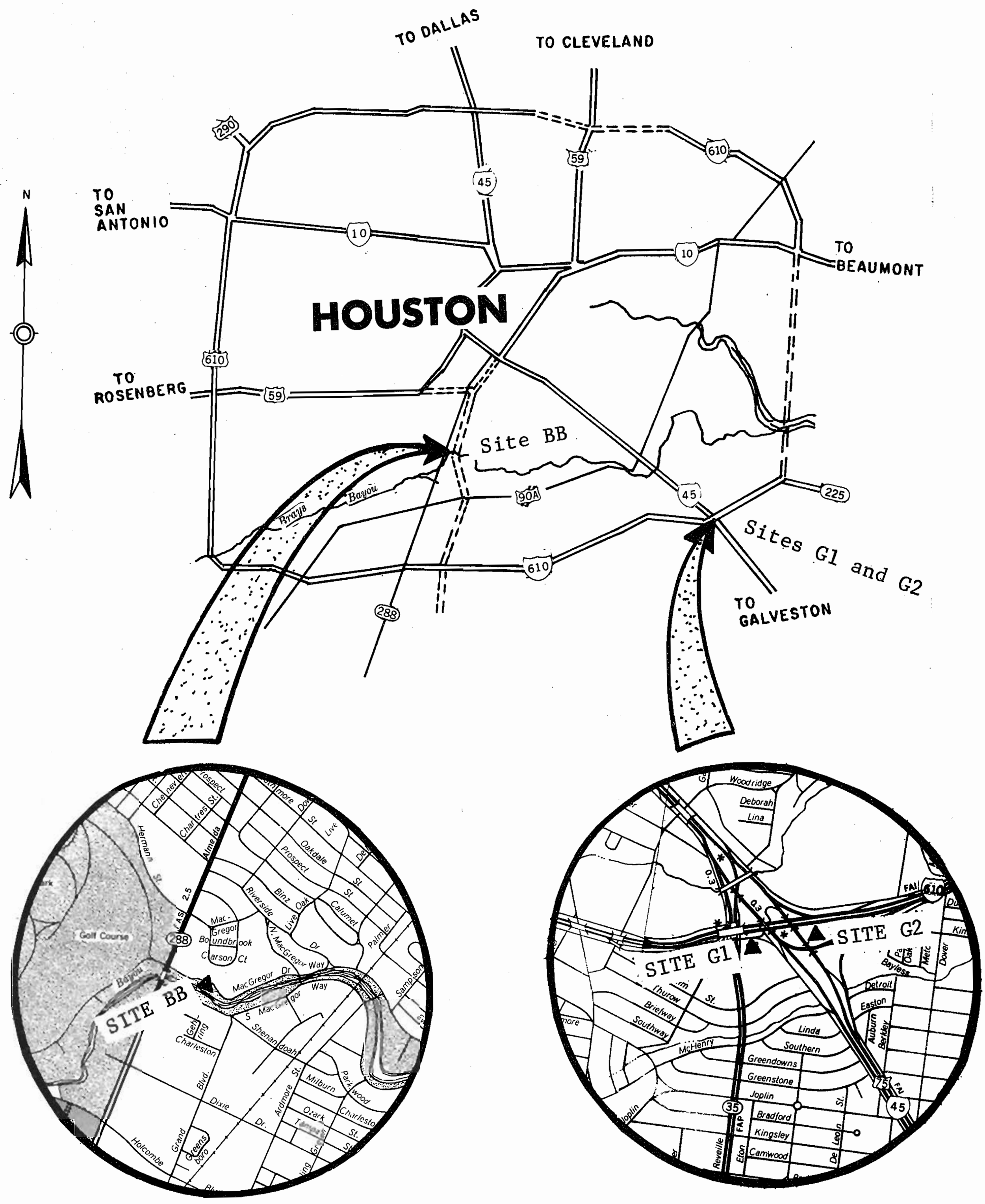


Figure 2.1. Location of Test Sites

the other hole for other work being done at the University of Texas Center for Highway Research. Appendix B contains the boring logs for the holes drilled for the THD cone penetrometer and triaxial tests, the two methods presently used by the Texas Highway Department.

Shear Strength Profiles

Profiles of shear strength were developed for each test site using soils data obtained from triaxial tests and/or THD cone penetration tests. These profiles are shown plotted in Figures 2.2 - 2.4.

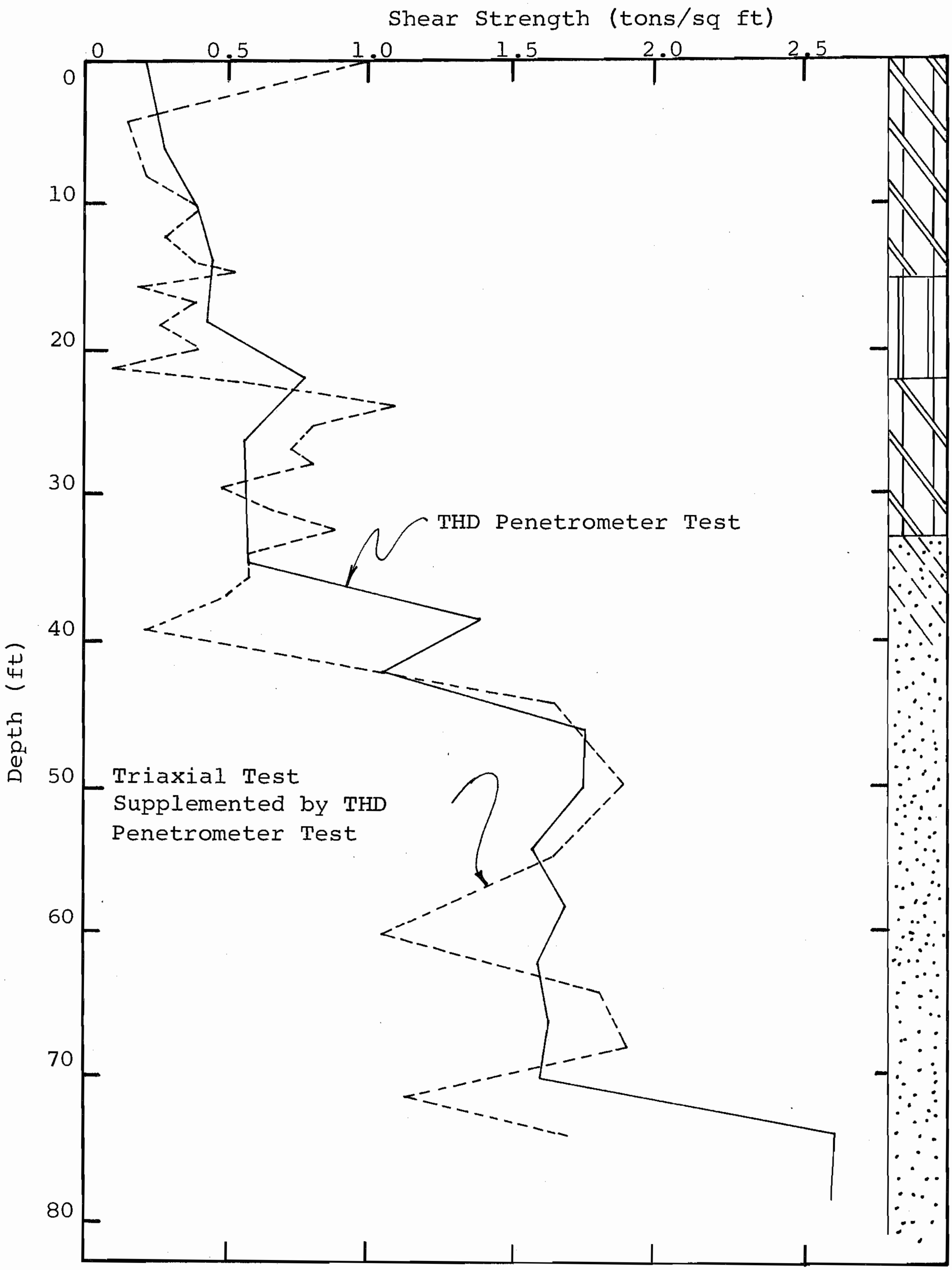


Figure 2.2. Shear Strength Profile for Site G 1

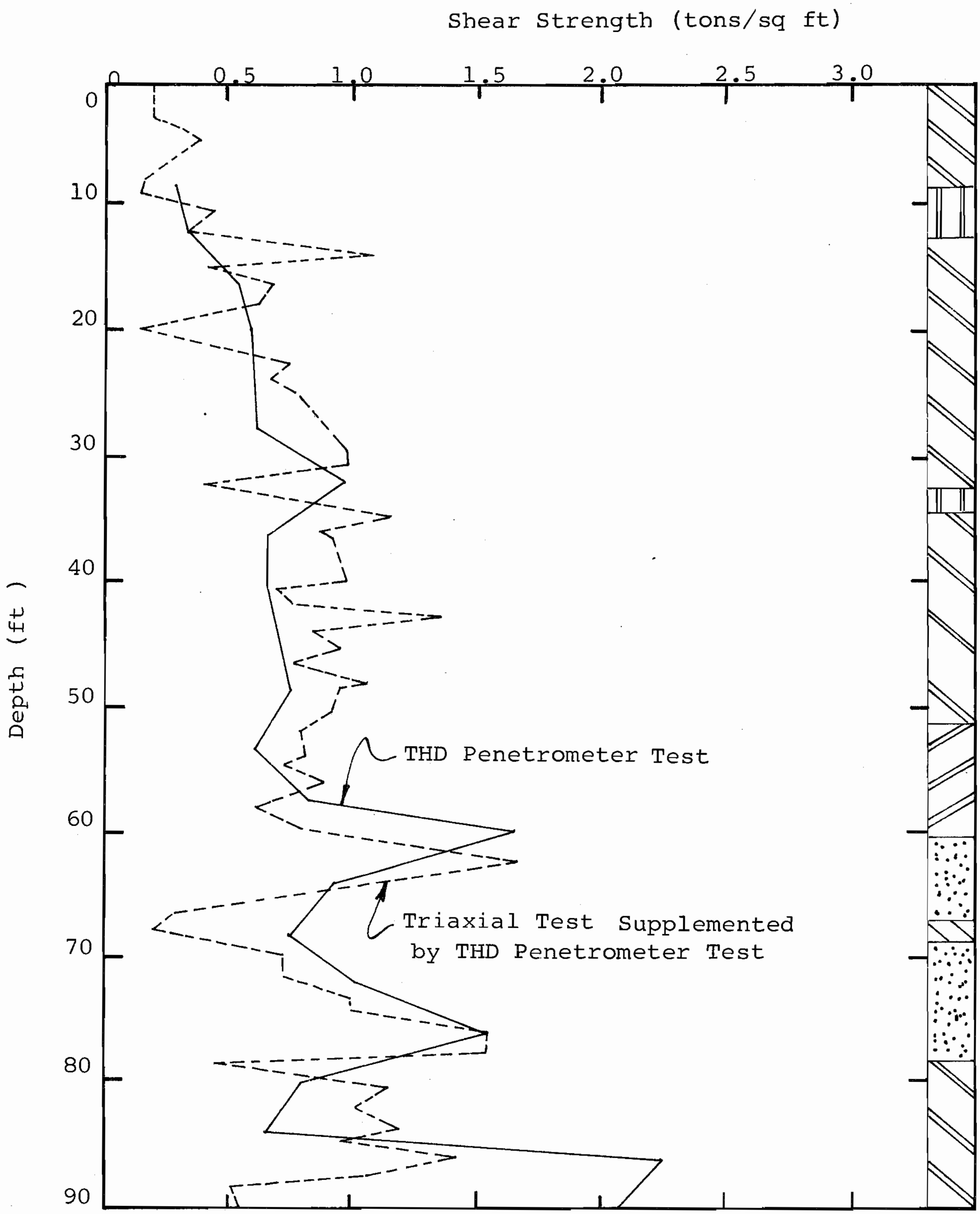


Figure 2.3. Shear Strength Profile for Site G 2

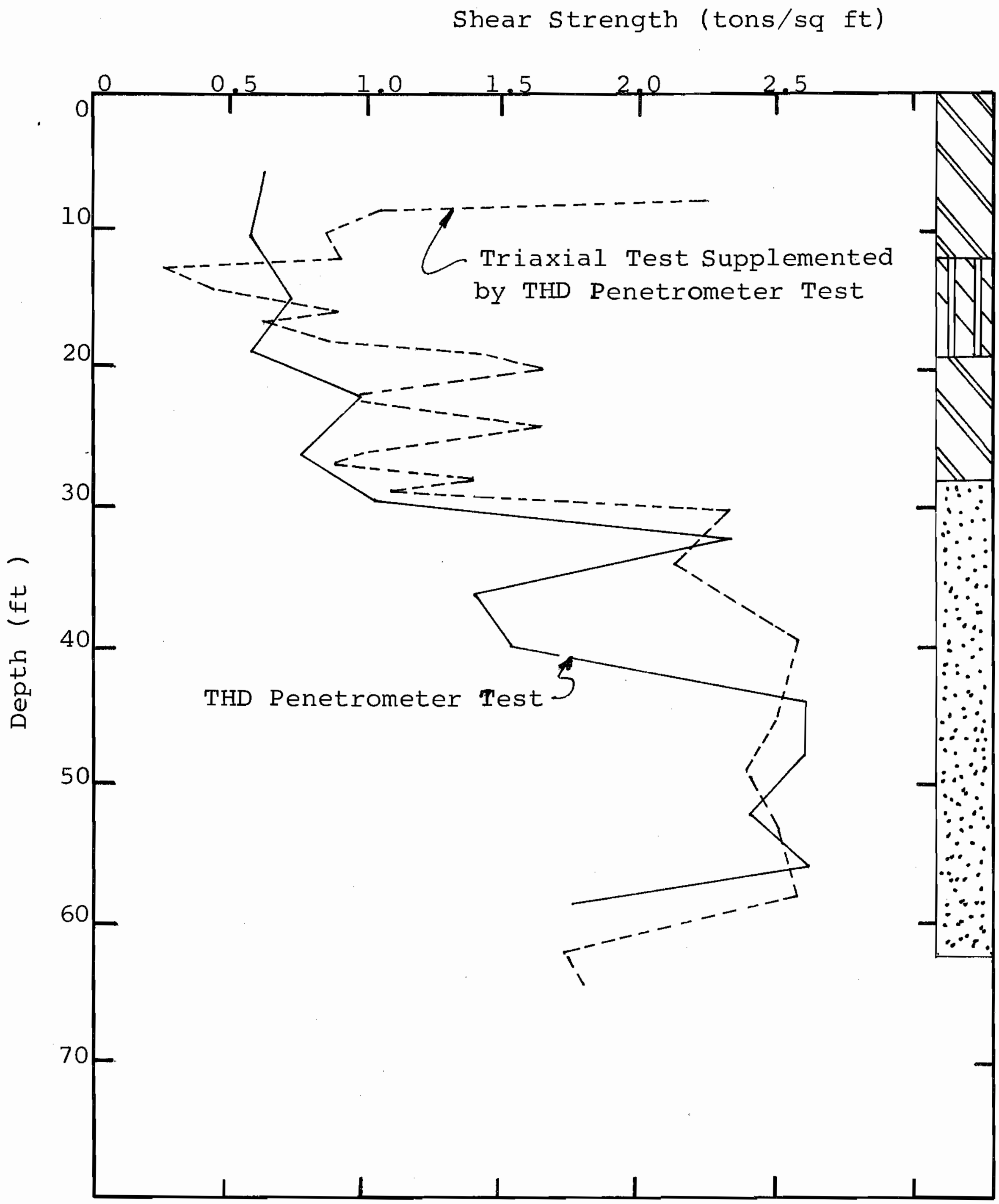


Figure 2.4. Shear Strength Profile for Site BB

CHAPTER III

TEST SYSTEMS

Test Shafts

Two shaft sizes were constructed for testing. The shafts at sites G 2 and BB were 2'-6" in diameter and the shaft at site G 1 had a diameter of 3'-0". The size and length of shafts to be test loaded are generally designed to fail at a load equal to, or slightly in excess of, two times the required service load on the shaft; however, the design of these shafts had to take into consideration the additional requirement of extraction of the shafts after testing. Data obtained from testing the two diameters selected would be used for scaling up to the required size should one or more of the test shafts fail to prove out the required design load. Details of the test shafts are shown in Figure 3.1.

Reaction System

The reaction system for these tests was made up of a reaction beam, two anchor posts, and two anchor shafts designed to become part of the permanent structures. Details of the anchor shafts are shown in Figure 3.1. The reaction beam, designed to be used for a variety of load

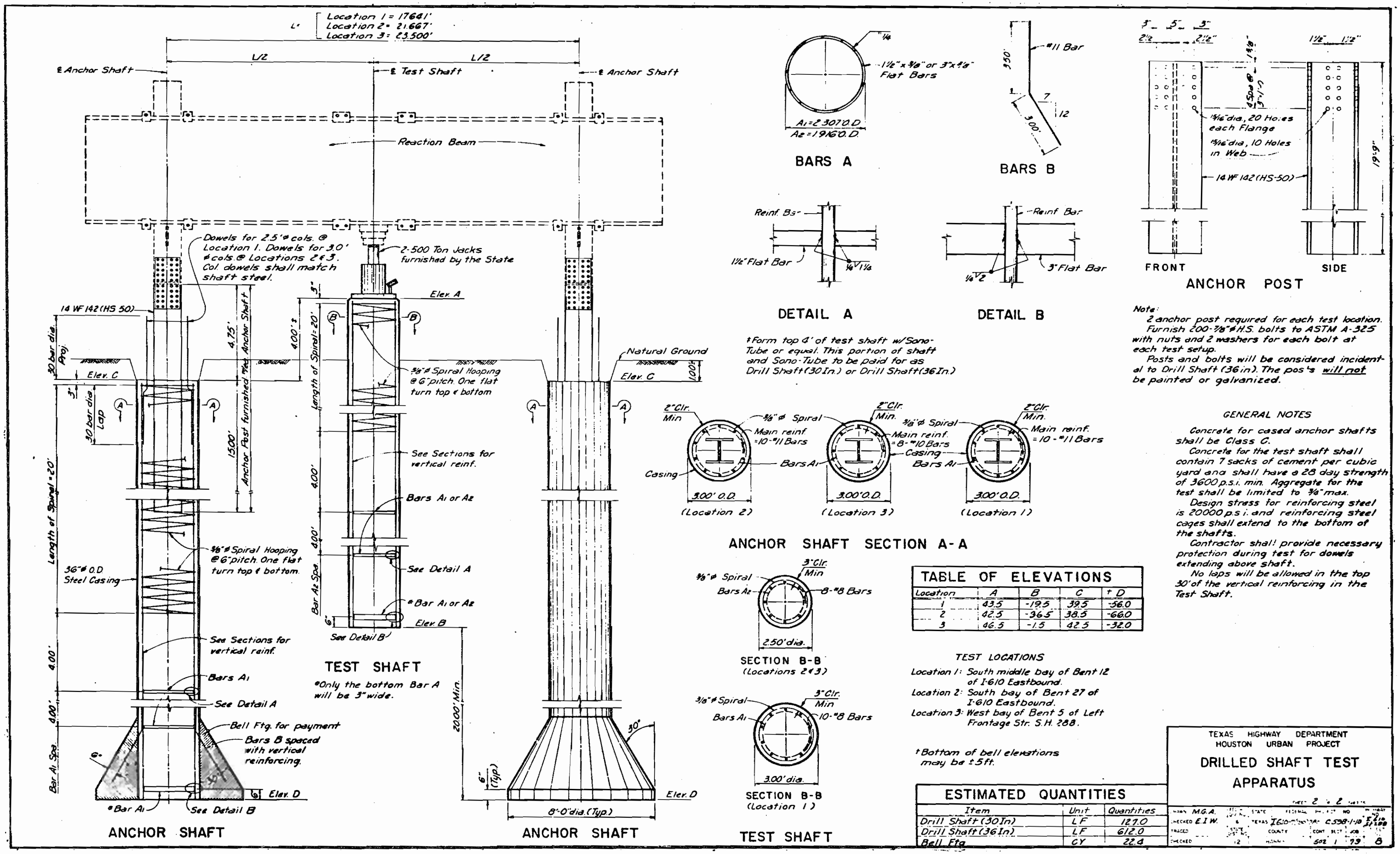


TABLE OF ELEVATIONS

Location	A	B	C	† D
1	43.5	-19.5	39.5	-56.0
2	42.5	-36.5	38.5	-66.0
3	46.5	-1.5	42.5	-32.0

ESTIMATED QUANTITIES

Item	Unit	Quantities
Drill Shaft (30 In)	LF	127.0
Drill Shaft (36 In)	LF	612.0
Bell Ftg.	CY	22.4

Figure 3.1. Details of Anchor Shafts and Test Shafts

tests, has a load capacity of 1000 tons and a span length which can be varied from 14 to 25 feet. Details of the reaction beam and anchor posts are shown in Figures 3.2 and 3.3.

Loading System

Load was applied to the test shafts with hydraulic rams jacked against the above reaction system. Two 400-ton-capacity double-acting Bayou Industries' jacks coupled in tandem with a common pressure supply were used. Pressure was applied to the jacks by an SC Hydraulic Engineering Corporation Model 10-600 hydraulic pump. This pump is air-operated and requires an air pressure of 90 psi. This loading system is pictured in Figure 3.4.

Instrumentation

Instrumentation was installed in the test shafts to determine the load transferred from the shaft to the surrounding soil. This instrumentation consisted of Mustran load cells which were developed by the University of Texas Center for Highway Research under Research Study No. 3-5-65-89. Load cells were located at several levels in the shafts with four cells in the top and bottom levels and two cells per level in each of the intermediate levels.

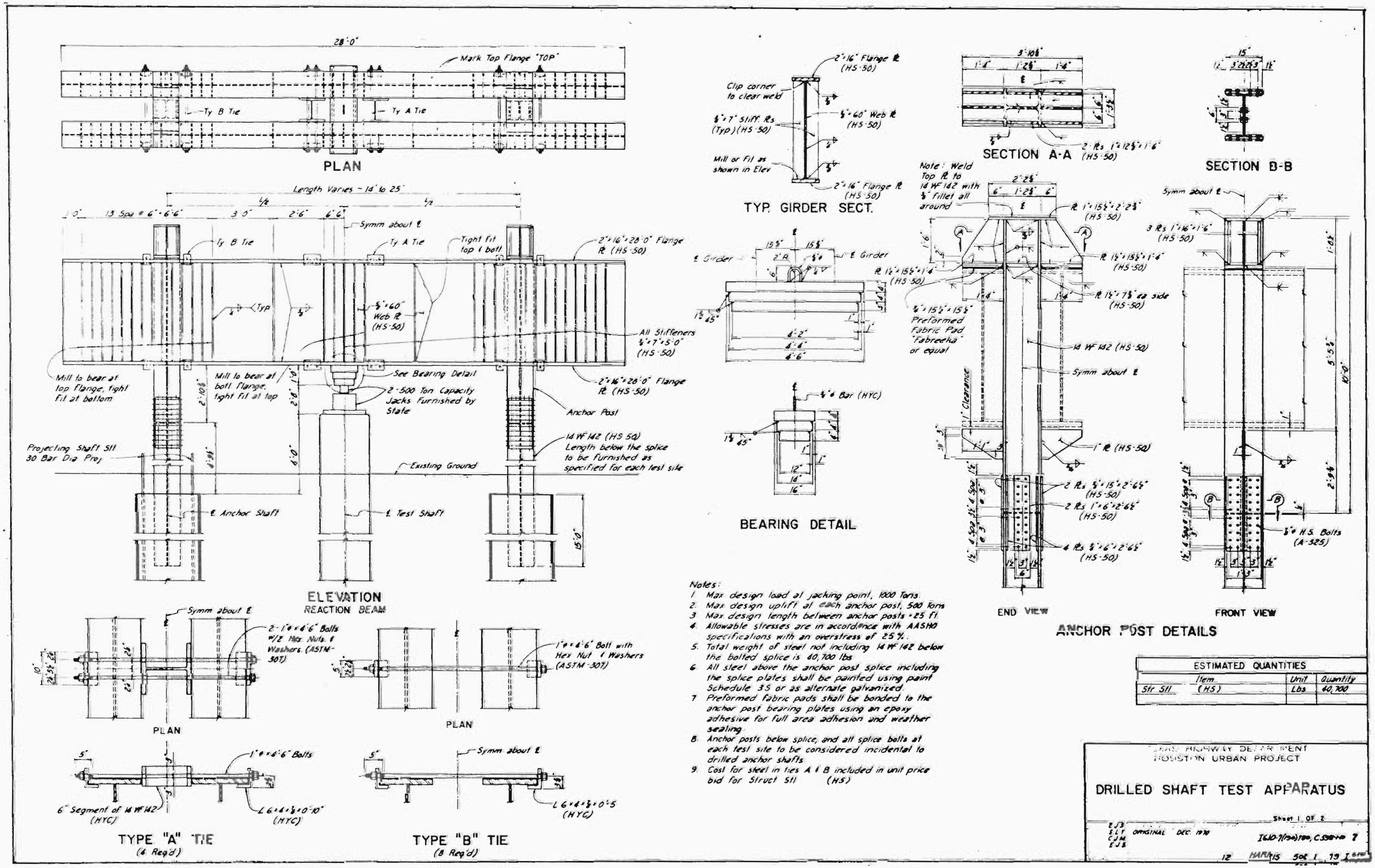


Figure 3.2. Details of Reaction Beam and Anchor Posts

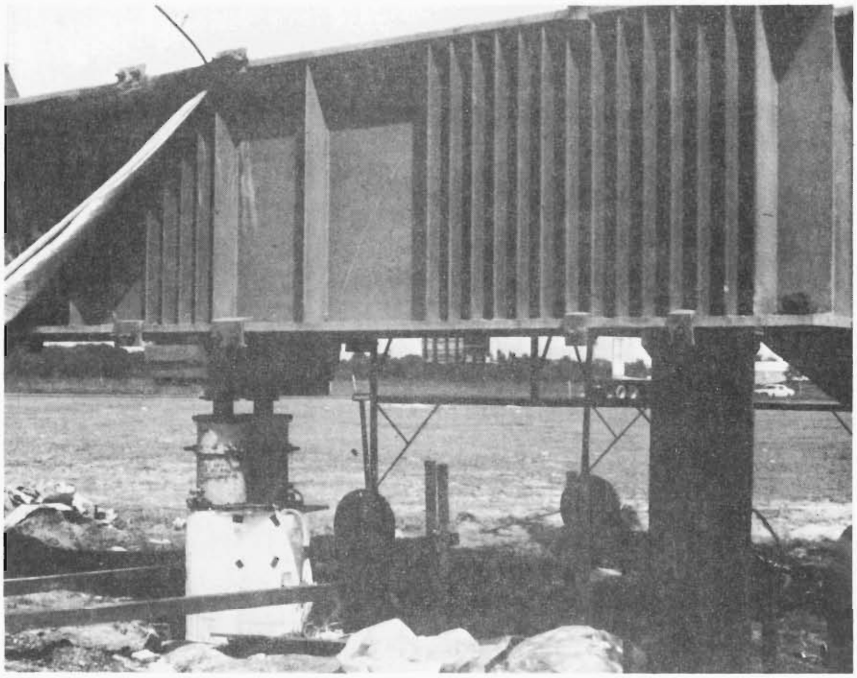


Figure 3.3. Reaction System

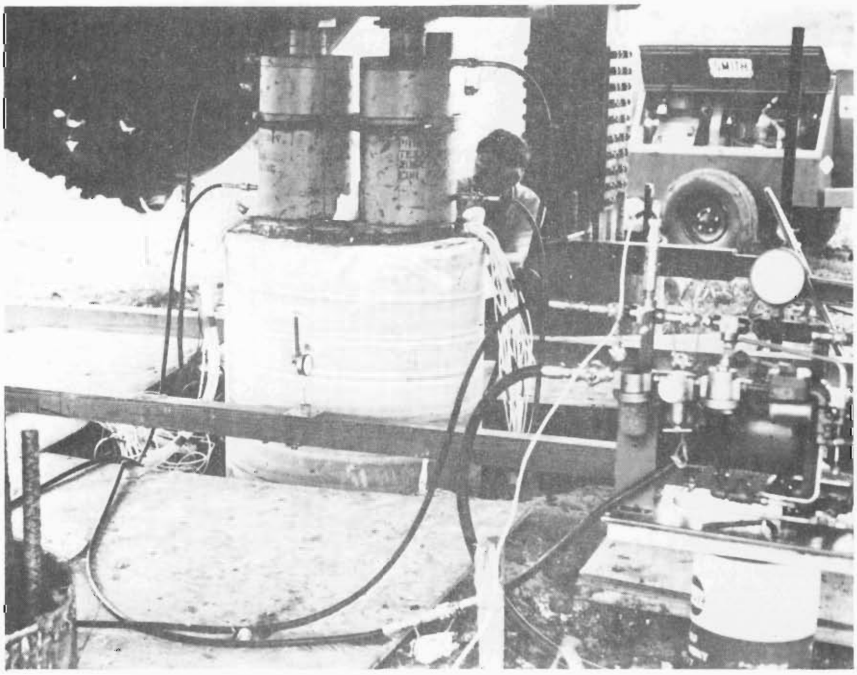


Figure 3.4. Loading System

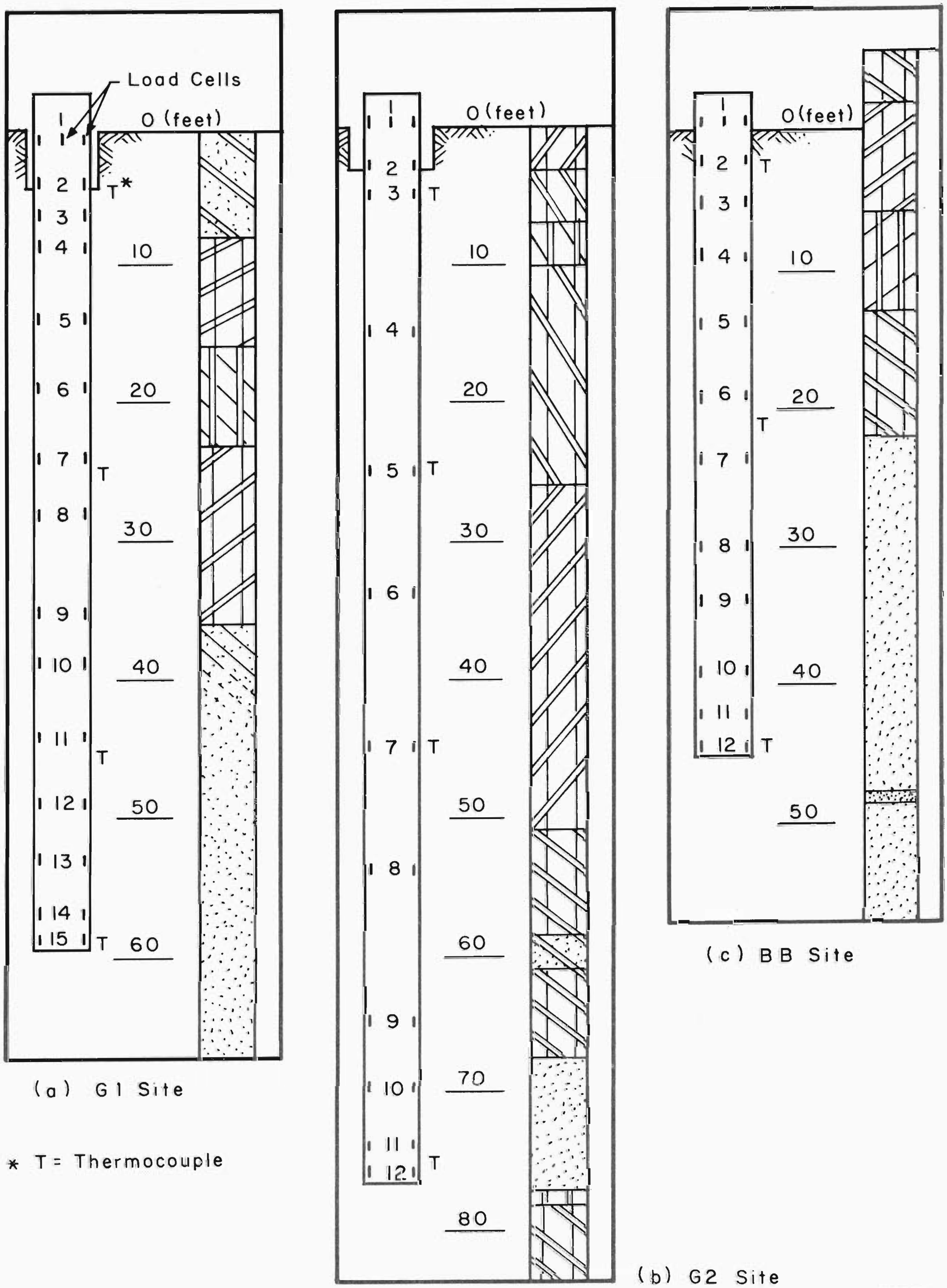


Figure 3.5. Soil Profiles and Location of Shaft Instrumentation

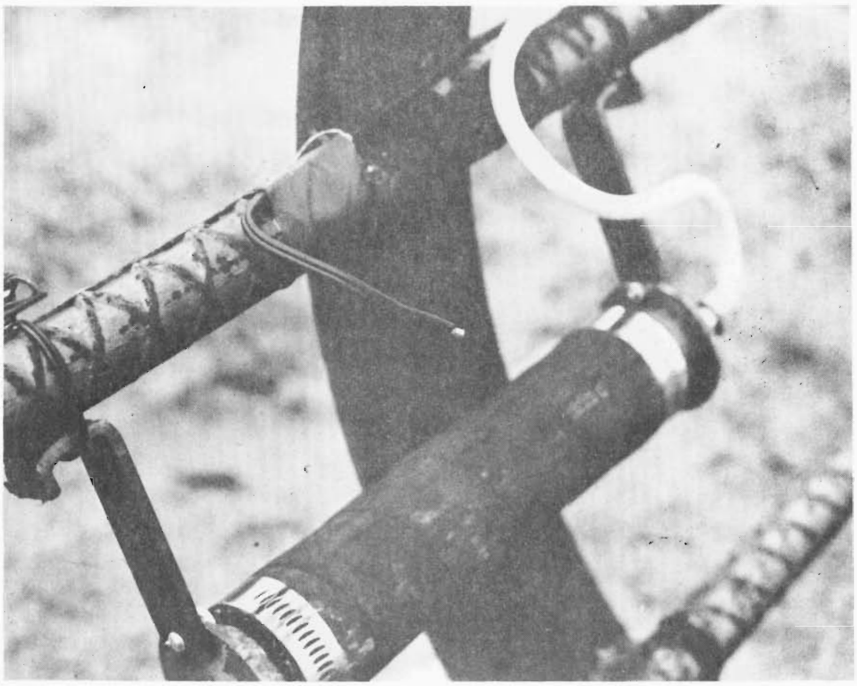


Figure 3.6. Mustran Cell Attached to Reinforcing Steel

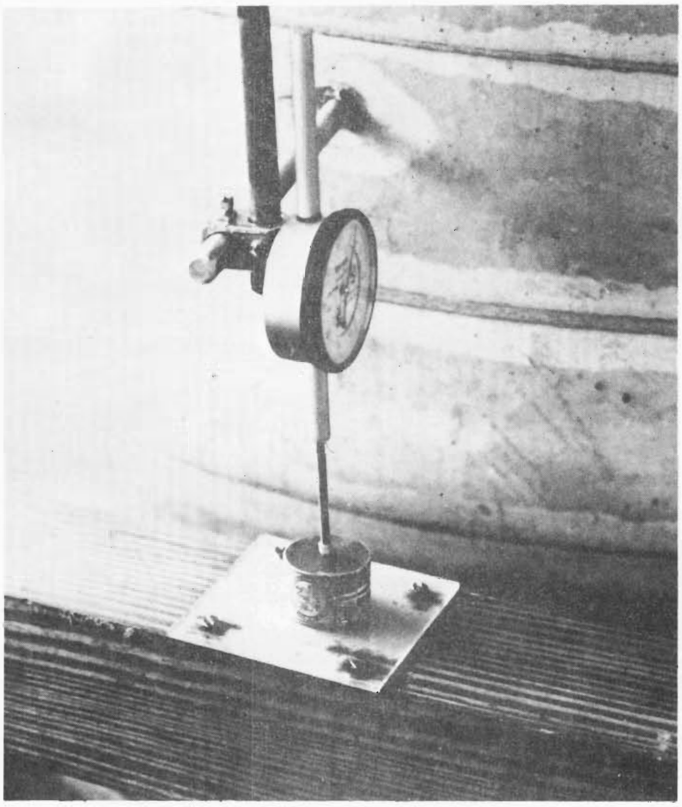


Figure 3.7. Settlement Gage

The location of the load cells was based on the following considerations:

1. To provide an accurate measure of the load reaching the tip of each shaft.
2. To provide a good calibration level at the top of each shaft.
3. To gain information about the load transfer characteristics of the various soil strata.

The location of the instrumentation levels, along with soil profiles, for each test shaft is shown in Figure 3.5. A typical Mustran cell installation is shown in Figure 3.6.

In addition to the Mustran load cells, dial gages were used to measure the settlement of the top of each test shaft. Two dial gages, mounted on the shaft and referenced to four-by-four wooden beams, were used at each test shaft. The gages were located on opposite sides of the shaft to detect nonuniform settlement should any occur. The method used for mounting these gages can be seen in Figure 3.7.

Data Acquisition

Data from the Mustran load cells were acquired digitally with a Honeywell Model 620 Data Logging System. This system, a portable modular unit shown in Figure 3.8, was housed in a van adjacent to the test shaft. Power was supplied by a 3 KV a.c. generator. When the ambient temperature exceeds 95° F. some means of cooling is required for proper operation of the data acquisition equipment. The ambient temperature during these tests was such that no cooling was required; however, air-conditioning has been required during some previous tests.

Figure 3.8. Honeywell Data Logging System

CHAPTER IV

CONSTRUCTION OF TEST SHAFTS

Equipment

Construction of the test shafts using the "slurry displacement" method required no specialized equipment, only that which is required for any drilled shaft construction. Major equipment used on this job included a crawler crane with drilling equipment mounted, water tank, and a holding tank for the mud slurry. A second crawler crane was generally available to handle casing, reinforcing steel, etc., and to serve as a back-up crane during concrete placement.

Site G 2

The test shaft at this site was 2'6" in diameter with its tip 75.1 feet below the ground surface. The contractor elected to construct this shaft first and construction began August 17, 1971.

The first significant ground water was encountered at an approximate depth of 56 feet. The soil above this level was a fairly stiff clay and it was relatively easy to drill the smooth round hole desired for a test shaft. Just prior to introducing the first slurry into the hole, a 39-foot length of surface casing, with an inside diameter of 30 inches, was set in place to prevent wallowing of the hole while drilling in the slurry.

The slurry used in this hole was mixed in the hole and consisted of water and Baroid Quick-Gel, described as a high yield Bentonite clay. Water was allowed to flow into the hole until the water level was about four feet above the bottom of the casing, making the water depth about 21 feet. Two sacks (50 pounds/sack) of Bentonite were then mixed into the water. When the drilling reached a depth of 64 feet another sack of Bentonite was added and the water level brought up to its original level where it was maintained until drilling reached a depth slightly less than plan depth. At this time the surface casing was removed and the hole was enlarged to a diameter of 30 inches. When drilling reached a depth of 75 feet two more sacks of Bentonite were added, making a total of five sacks, and the water level was brought up to 25 feet below the ground surface. Excavation continued for two more feet using a clean-out bucket in lieu of an auger (Figure 4.1). Use of the clean-out bucket permitted a detailed examination of the material on which the tip of the shaft would rest.

Approximately one and one-half hours after the removal of the surface casing the sides of the hole near the water

Figure 4.1. Clean-Out Bucket

level began caving. The water level was then raised to a level approximately 17 feet below the ground surface in an attempt to stop the caving. About one hour later, two and one-half hours after removal of casing, concrete arrived. Just prior to setting the reinforcement cage in place, and while making the final clean-out of the hole, substantial caving was discovered near the water level. Examination revealed that the area of caving was much too large for a test shaft, therefore this hole was abandoned and filled with concrete.

The location for this test shaft was moved to the adjacent bay in the same bent and the shaft installed August 19, 1971. It was believed that the previous failure resulted from leaving the upper portion of the hole open for too long a period of time. To preclude the possibility of another failure the hole was drilled in the dry until the water bearing stratum was reached and then completely filled with slurry. Drilling was then completed in the slurry and without the use of surface casing. The slurry used for drilling this hole was mixed one day before it was used and stored in an open-top tank. Twelve sacks of Baroid Quick-Gel were mixed with approximately 2900 gallons of water to form the slurry.

The total time required for construction of this shaft was approximately five hours. Three hours of this time were used for drilling the hole and about one and one-half hours were used in placing the concrete. The remaining one-half hour was used in placing and aligning a section of Sonotube used to form the portion of the shaft extending above ground.

Site G 1

The test shaft at this site was 3'-0" in diameter with its tip 58.7 feet below the ground surface. Construction of this shaft was completed September 3, 1971.

A water bearing sandy silt was encountered at an approximate depth of 17 feet. At this point, the hole was filled with slurry to a level three feet below the ground surface. Slurry was maintained at that level throughout the remainder of the drilling operation.

The slurry used for this shaft came from two sources: One was a mixture of Baroid Quick-Gel used the previous day in the drilling for an anchor shaft, and saved for re-use. Additional slurry was provided by mixing Macogel bentonite and water in a jet-cone mixer and pumping directly into the hole.

An auger was used for the final clean-out of this hole because a clean-out bucket of the correct size was not available. Use of an auger in lieu of a bucket has the disadvantage of cuttings tending to wash off during removal and falling back into the hole. This creates a weak foundation for the tip of the shaft and reduces the bearing capacity of the shaft. An indication that weak material had fallen into this shaft occurred when the tremie settled approximately six inches when the first bucket of concrete was dumped into it.

Installation of this test shaft was completed approximately four and one-half hours after drilling began.

Site BB

The test shaft at this site was the third and final one constructed. It was 2'-6" in diameter with the tip 44.3 feet below the ground surface and was installed September 21, 1971.

A water bearing sand was encountered at an approximate depth of 20 feet. A slurry was then mixed in the hole using four sacks (100 pounds/sack) of Macogel bentonite and water pumped from Brays Bayou. One more sack of bentonite was added when the hole was 40 feet deep.

Interpolation between known locations of sandstone on each side of the test shaft indicated that sandstone should be found at or near the planned tip elevation, however none was encountered at this elevation. The soil below the bottom of the hole was probed to locate the expected sandstone; first with a three-foot long chisel, and then with a 10-inch diameter auger. A layer of sandstone was located at an approximate depth of 47 feet below the ground surface.

Final clean-out of this hole was made approximately four hours after drilling had been completed. During this time an estimated two feet of sediments had settled in the bottom of the hole. Final tip elevation was 44.3 feet below the ground surface, or approximately 2.7 feet above the sandstone.

The depth of slurry in this hole decreased about ten feet during the four hours the hole was open. A similar loss in slurry depth occurred in an adjacent hole drilled for an anchor shaft the previous day and left open through the night. One probable cause for these losses is the steep gradient towards the bayou of the area in which these holes were drilled.

Approximately seven and one-half hours elapsed between the start of drilling and completion of concrete placement; however, concrete placement required only one and one-half hours. There was a delay of approximately four hours between completion of the drilling and beginning of the concrete placement.

Concrete Placement

A 10-inch diameter steel pipe tremie was used to place the concrete in the three test shafts. Steel guides were used to hold the tremie in the center of the reinforcing steel cage: one temporarily attached at the top of the cage and another permanently attached to the cage approximately 20 feet below the top (Figure 4.2). At site BB

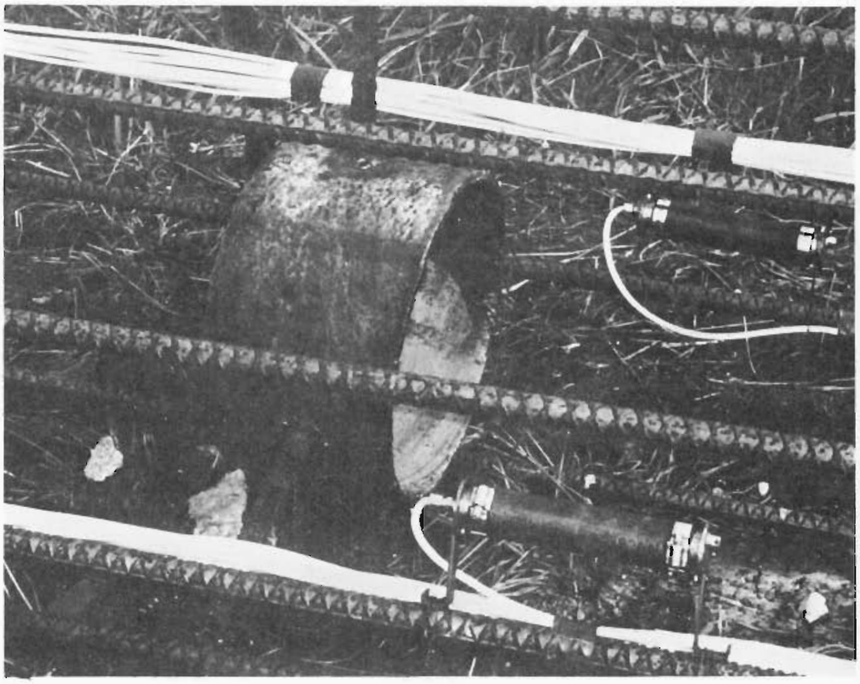


Figure 4.2. Tremie Guide

the short length of shaft did not require these guides.

When first inserted into the mud slurry the lower end of the tremie was sealed to prevent slurry entering it. This seal was temporary in nature and consisted of a plywood plug in the pipe with the end of the tremie covered with polyethylene sheeting held in place by rubber bands (Figure 4.3). This seal was broken by vigorously bouncing the tremie full of concrete.

At site BB some difficulty was experienced in getting the concrete flow initiated. The first attempt failed and the tremie had to be removed and emptied of concrete.

There were two possible reasons for this happening: (1) The tremie was not clean on the inside, and (2) it was not wetted before using. The flow of concrete in this shaft, as well as in the other two, could have been improved by using a tremie with a larger diameter.

Once the tremie seal has been broken and concrete begins to flow, the bottom of the tremie must remain submerged in the concrete; otherwise the slurry will contaminate the fresh concrete. The procedure used for placing concrete in these test shafts was to fill a one-cubic-yard dumping bucket with concrete from a Transit Mix truck, attach bucket to top of tremie, then slowly raise the tremie until a good flow of concrete was obtained while dumping concrete from the bucket into the tremie. After a bucket was emptied the tremie was lowered to the bottom of the hole and the above procedure repeated until good clean concrete was observed to flow from the excavation. Pictures of the concrete placing operation are shown in Figures 4.4 and 4.5.

(a) Wooden Plug

(b) Polyethylene Sheet

Figure 4.3. Installing Temporary Tremie Seal

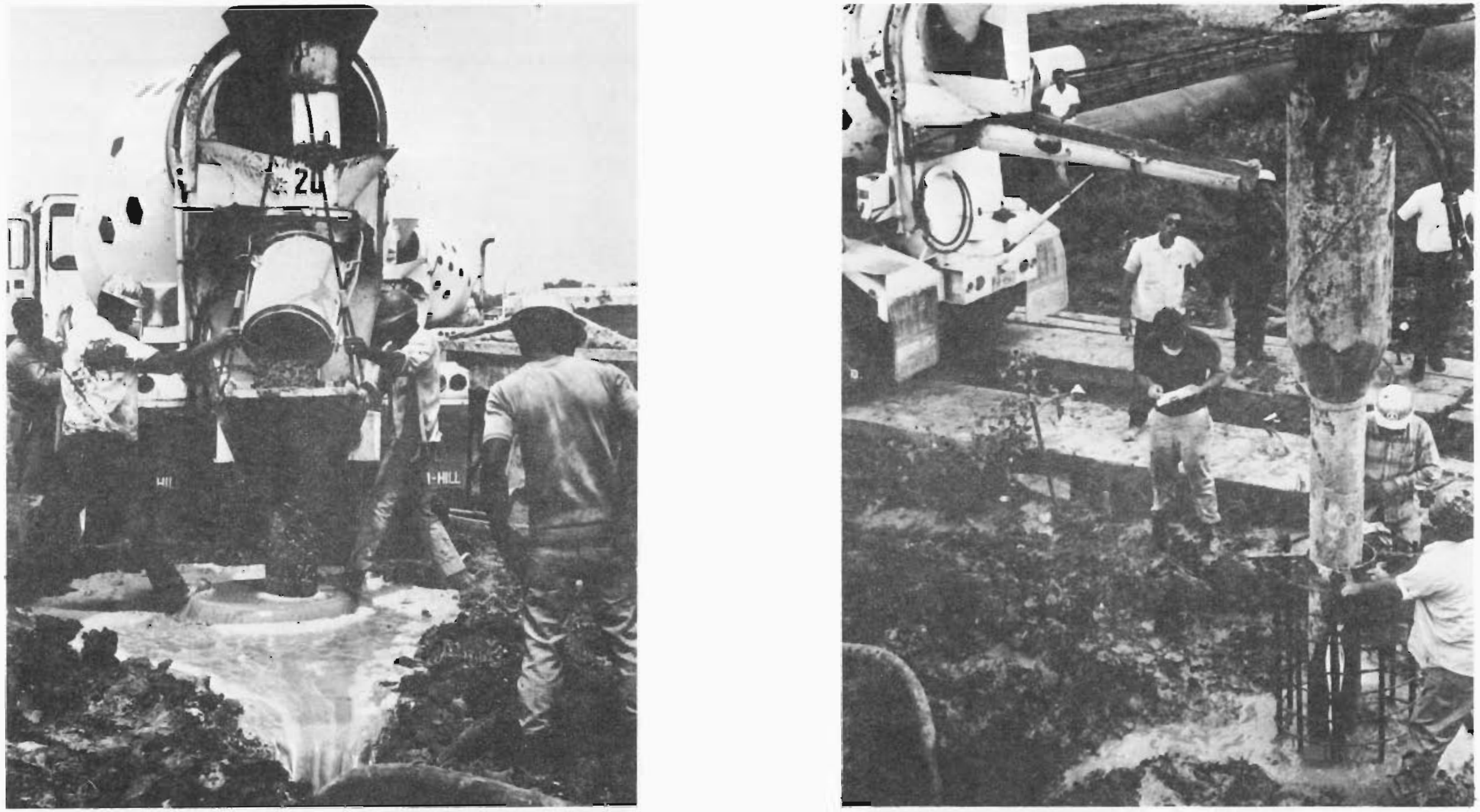
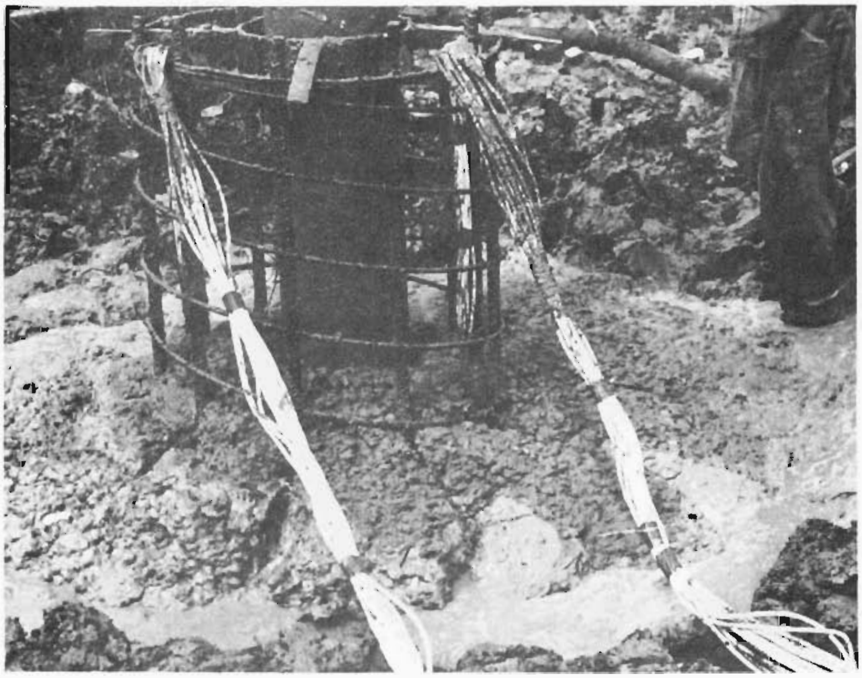


Figure 4.4. Concrete Placing Operation

(a) First Appearance of Concrete



(b) Good Flow of Concrete

Figure 4.5. Drilling Slurry Being Displaced by Concrete

CHAPTER V

LOAD TESTS

Load tests were performed on all three of the test shafts. Shaft BB was tested at an age of 16 days while G 1 and G 2 were tested at ages of 45 days and 52 days respectively. BB was tested at the earlier age to ascertain the effect, if any, of testing at an early age.

All but one of the tests were performed using the "quick-test" procedure described in the Texas Highway Department "Special Provisions to Specification, Item 405," dated July, 1965. The exception was the third and final test on shaft G 1 when a "cyclic" procedure was used. Dates of the load tests, maximum applied load, and other general information are shown in Table 5.1. Table 5.2 shows the amount of load, as a percent of maximum applied load, carried by frictional resistance.

Top of shaft load-settlement curves for Test Nos. 1 and 2 plus the tip load-settlement curve for Test No. 1 are shown in Figures 5.1 through 5.3. Load-distribution curves for the first load test performed on each shaft are shown in Figures 5.4 through 5.6. Two curves are shown in each figure; one is the distribution for the Plunging Failure Load and the other is for the Double Tangent Failure

Load. The Double Tangent Failure Load is the load indicated by the intersection of two lines drawn on a load-settlement curve; one tangent to the initial flat portion of the curve and the other tangent to the steep portion.

TABLE 5.1 DATES AND GENERAL INFORMATION ON LOAD TESTS

Shaft	Test	Date of Test	Load Incr.	Unloading Incr.	Maximum Load Applied
BB	1	10-7-71	25T	--*	750T
Cast 9-21-71	2	10-8-71	50T	75T	750T
Extracted 10-13-71	3	10-8-71	100T	300T	600T
G2	1	10-12-71	25T	50T	700T
Cast 8-19-71	2	10-12-71	50T	100T	680T
Extracted 10-26-71					
G1	1	10-19-71	25T	50T	480T
Cast 9-3-71	2	10-19-71	50T	100T	450T
Extracted 11-5-71	3 (Cyclic)	10-19-71	75T	one incr.	450T

*Hydraulic system broke and an unloading curve could not be obtained.

TABLE 5.2 PERCENT OF APPLIED LOAD CARRIED BY FRICTIONAL RESISTANCE

Site	Ultimate Plunging Load (Tons)	Frictional Load Transfer (Tons)	Frictional Load Transfer (%)
G1	480	425	91
G2	700	665	95
BB	750	460	88

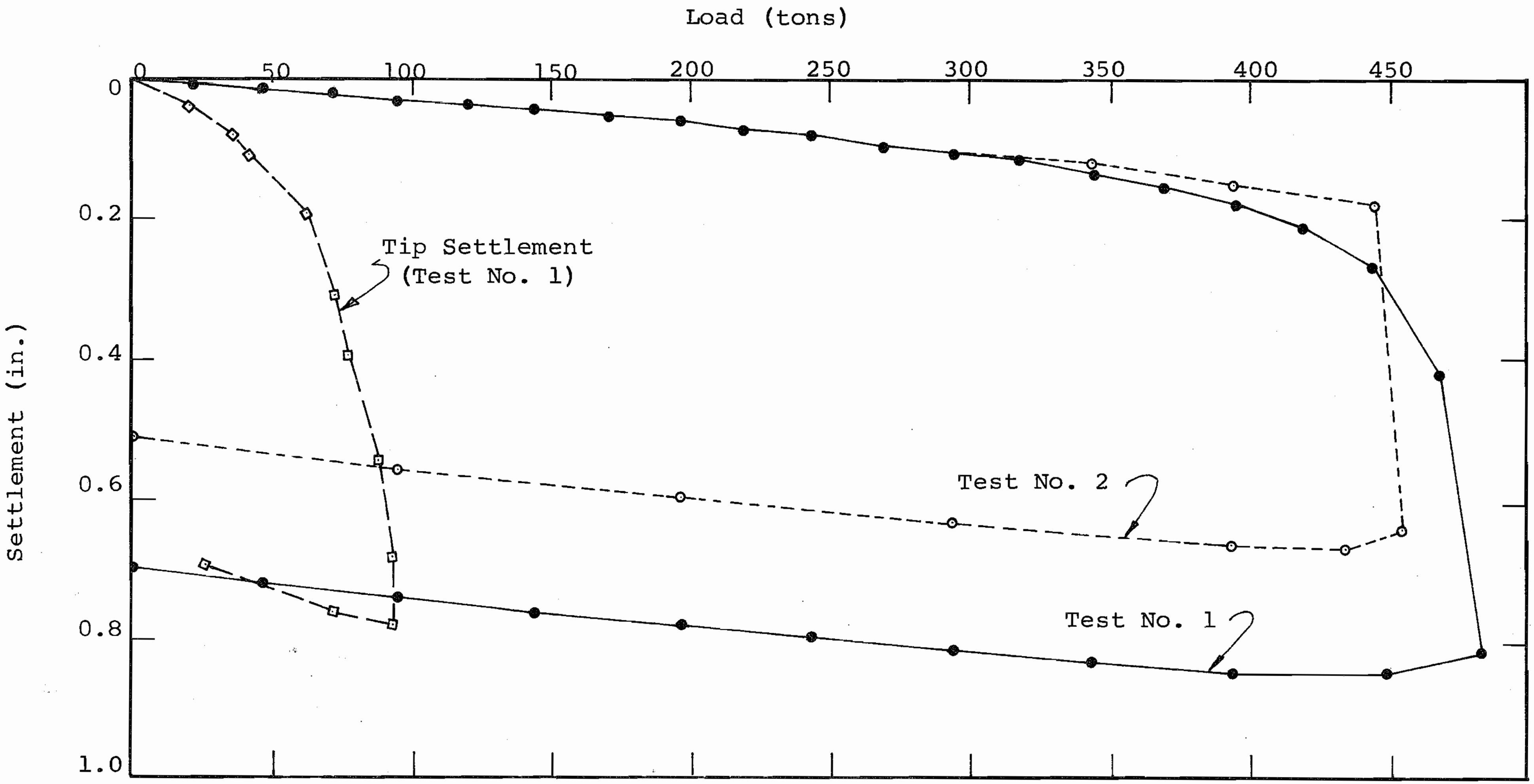


Figure 5.1. Load-Settlement Curves, Shaft G 1

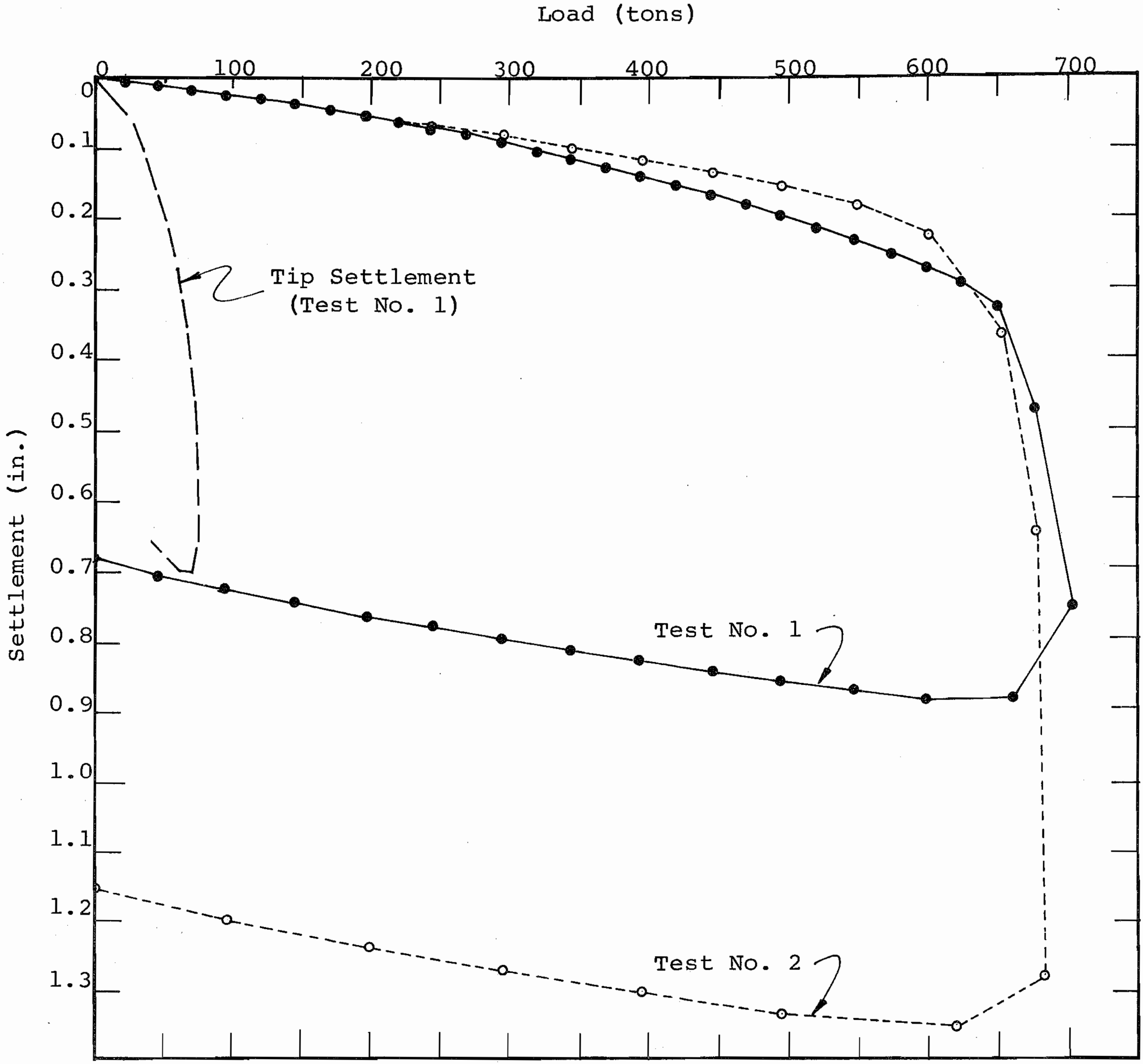


Figure 5.2. Load-Settlement Curves, Shaft G 2

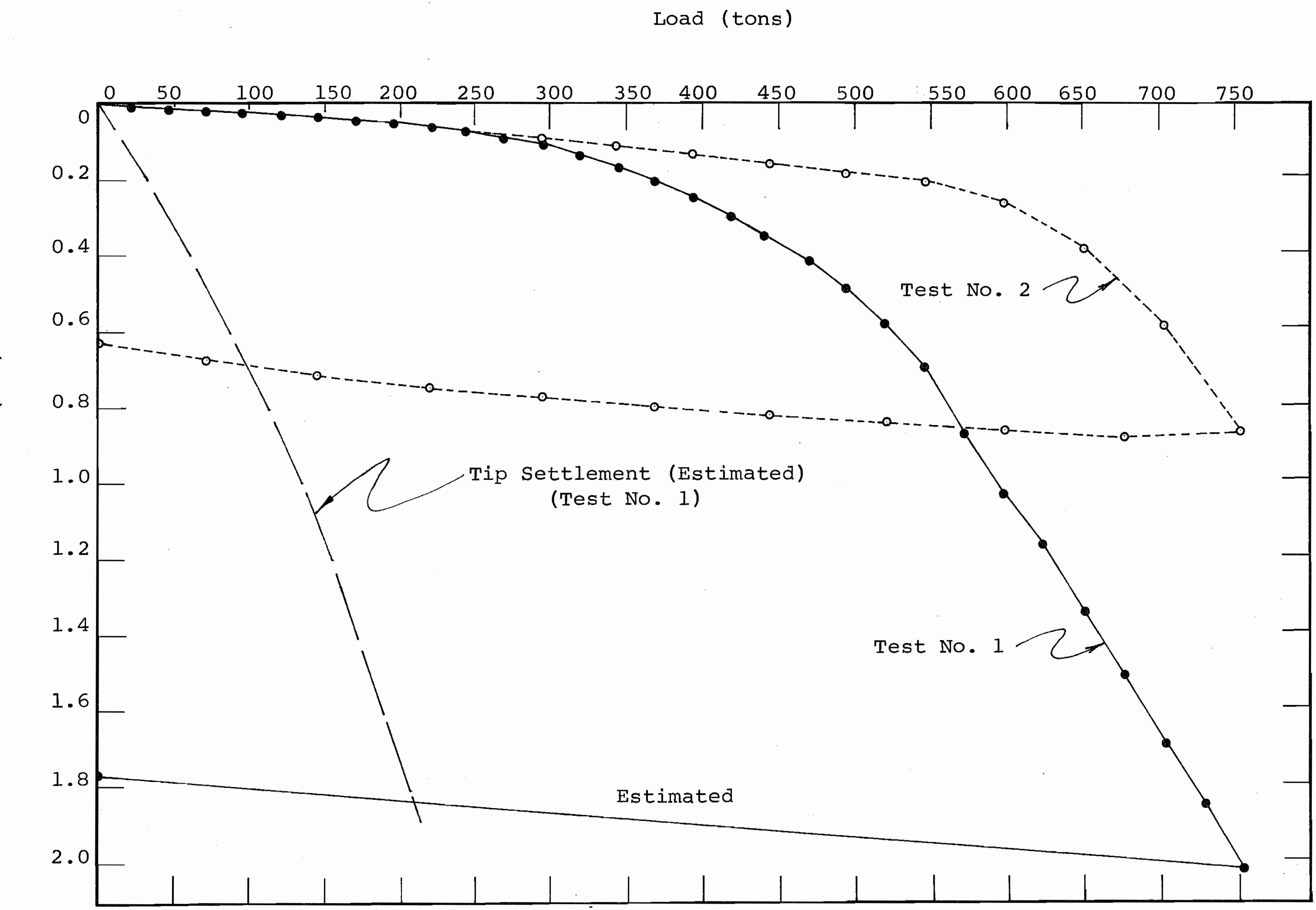


Figure 5.3. Load-Settlement Curves, Shaft BB

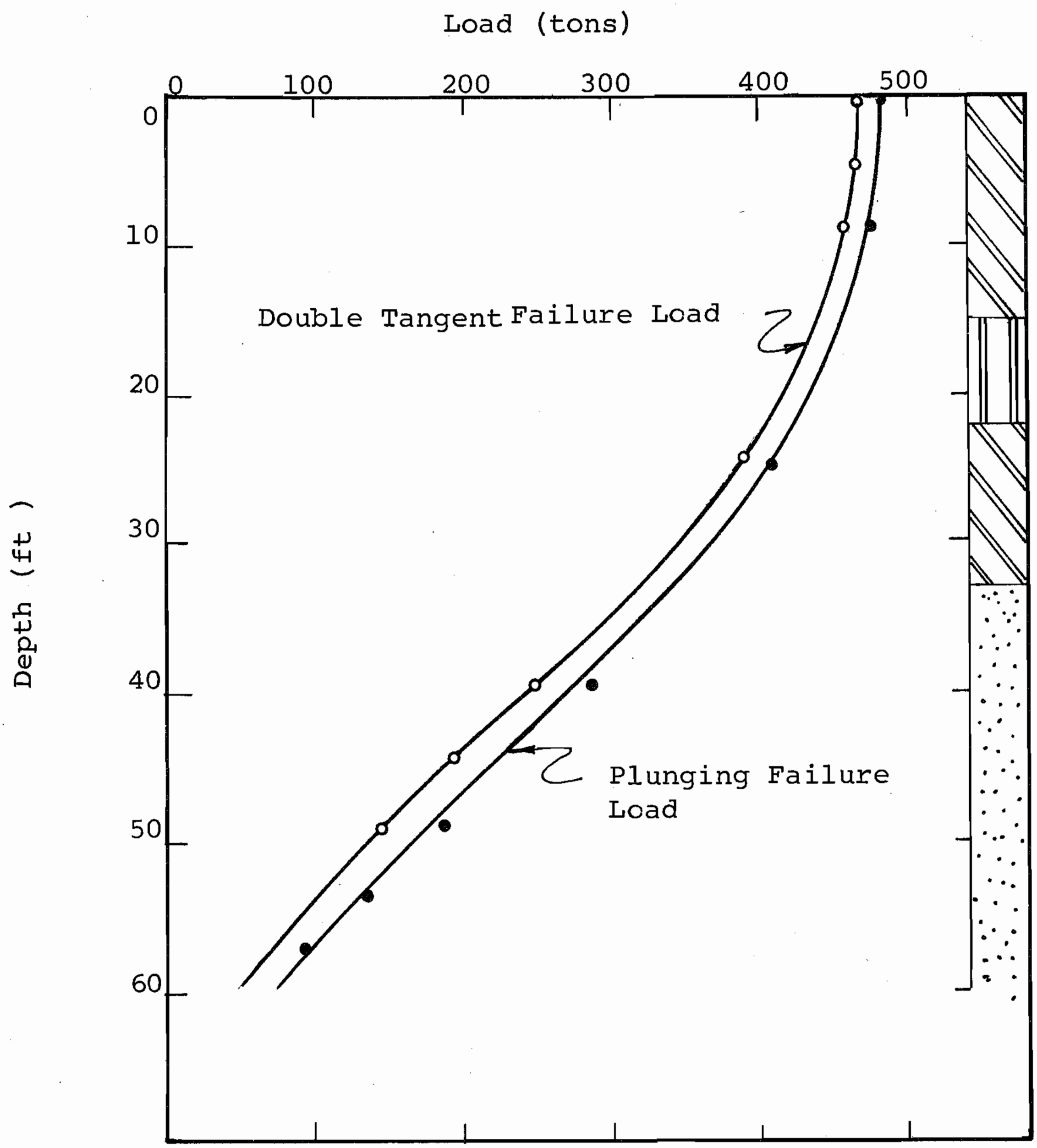


Figure 5.4. Load Distribution Curves, Shaft G 1, Test No. 1

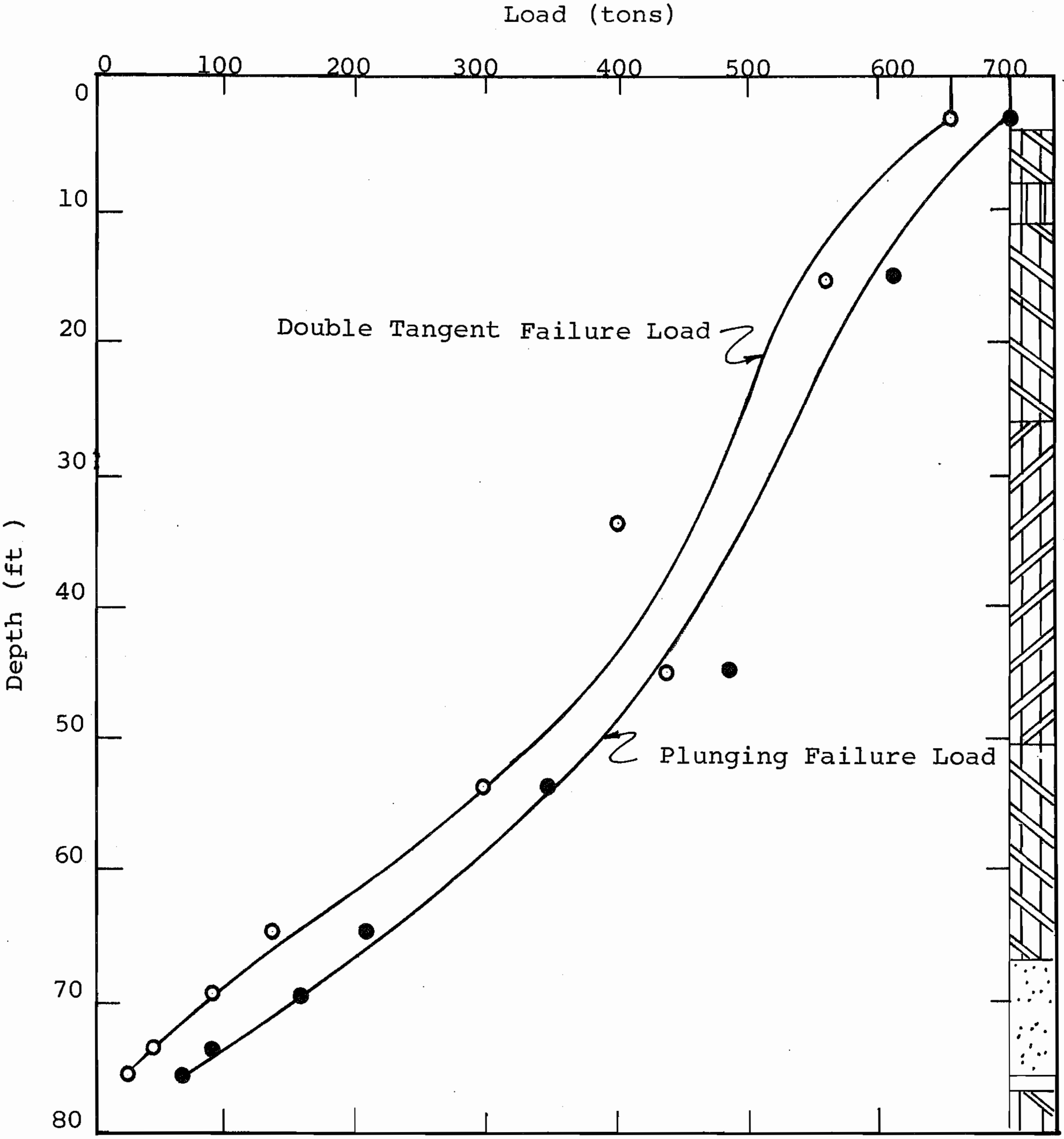


Figure 5.5. Load Distribution Curves, Shaft G 2, Test No. 1

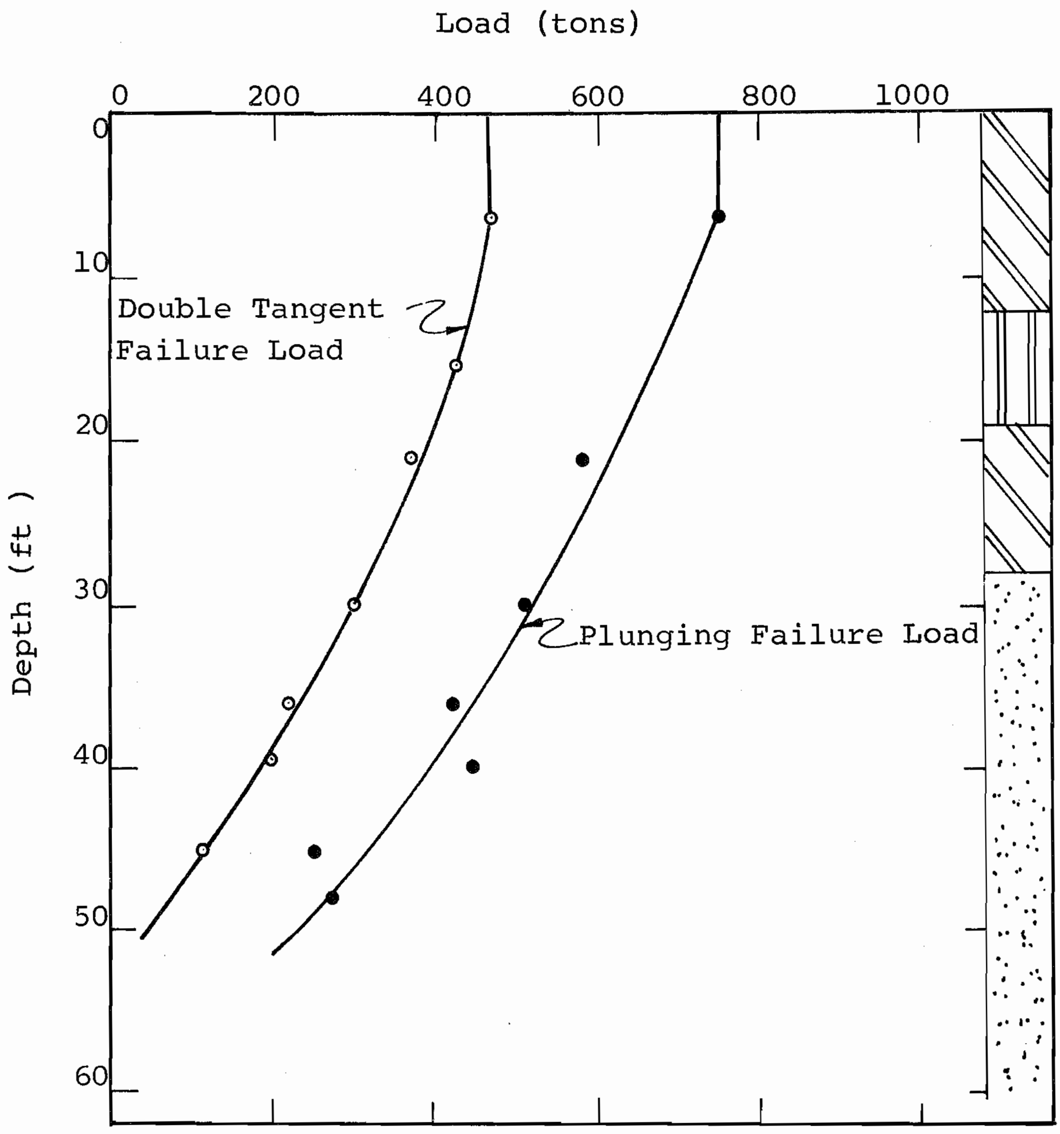


Figure 5.6. Load Distribution Curves, Shaft BB,
Test No. 1

CHAPTER VI

EXTRACTION AND INSPECTION

After load testing was completed, all test shafts were extracted and inspected. The procedure used for extraction was (1) drill an annular opening around the shaft for its full depth, (2) loosen the shaft from the soil using a specially constructed pulling device, and (3) lift shaft from hole with a crane (Figures 6.1 - 6.4). After removal of the shafts, the shaft diameter was measured at each instrumentation level and a careful examination of the concrete soil interface was made. Additionally, several gages were removed to inspect their seating in the concrete and the concrete inspected for slurry contamination. Results from the inspections are discussed below for each test shaft and photographs of the extracted shafts are shown in Figures 6.5-6.11 at the end of this chapter.

Shaft G 1

The shape of shaft G 1 was generally cylindrical and straight. Slight enlargements were found at the bottom of the section formed with the Sonotube and in the areas of sand strata. The small size of these enlargements indicated that no significant caving had occurred in the sand strata.

The tip of the shaft was irregular in shape with some reinforcing steel exposed. The wooden plug used as a temporary tremie seal was located to one side of the shaft and the polyethylene used for the same purpose had covered most of the tip and a portion of the sides near the tip.

A coating of discolored material which had the appearance of sand stabilized with cement and drilling slurry covered the entire length of the shaft. The thickness of this coating varied from ¼" to ½" in the portion of the shaft in the sand with a lesser thickness over the remainder of the shaft. Examination of the soil adherred to the sides of the extracted shaft indicated that failure had occurred in the surrounding soil and not in this coating.

Removal of concrete around some of the load cells provided an opportunity to inspect the interior of the shaft concrete. In the upper levels of the shaft evidence of entrapped sand and bentonite was found around the instrumentation cables and where the spiral steel and reinforcing bars were joined. The bottoms of some load cells also had trapped some mud. Some evidence of contamination, which appeared to be a stabilized mixture, was also found approximately six inches from the surface, however, this contamination did not appear to be widespread.

Figure 6.1. Drilling Around Shaft Prior to Extraction

Figure 6.2. Lifting Device in Place

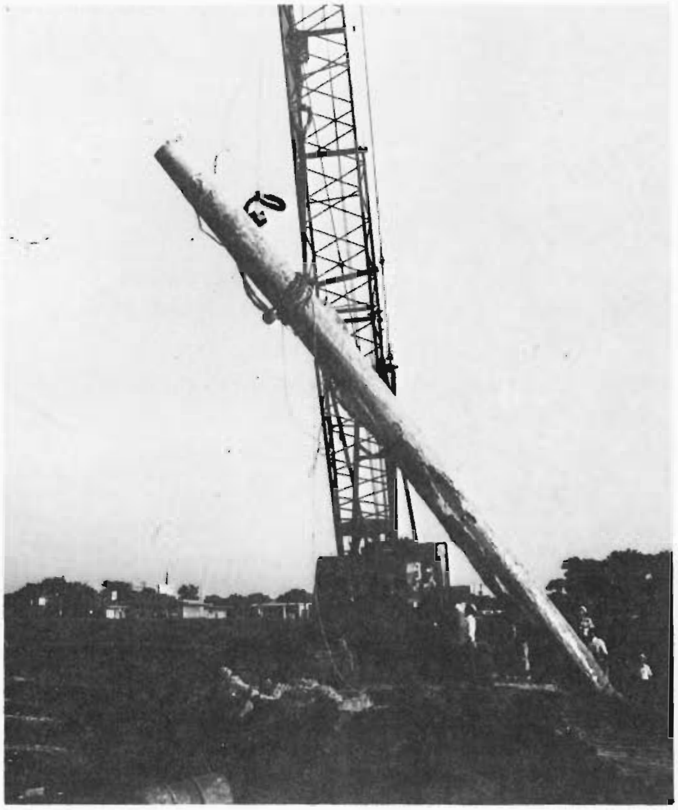


Figure 6.3. Removal of Shaft

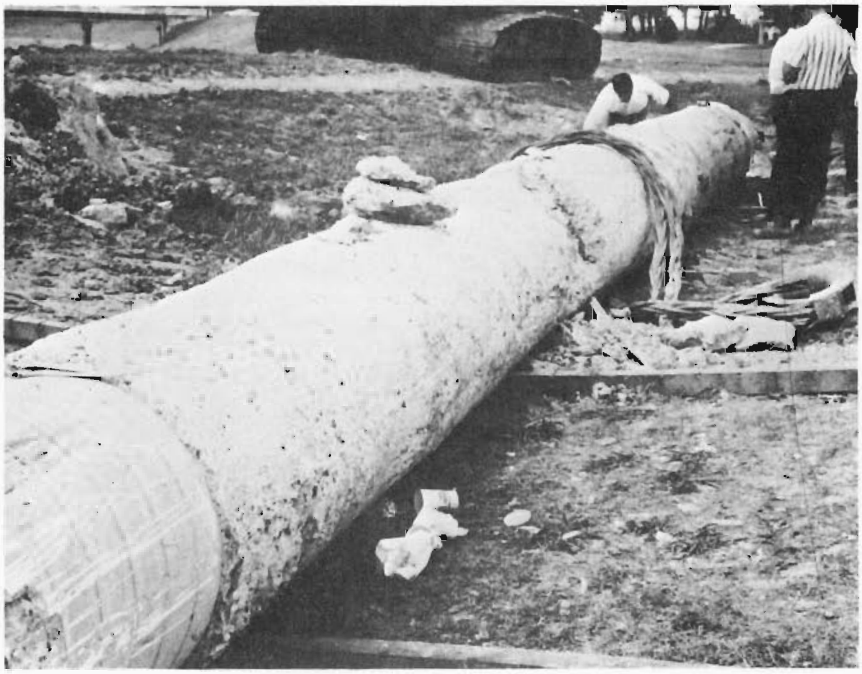


Figure 6.4. Inspection of Shaft

The texture of the concrete surface was somewhat roughened but there was no indication of any weak or bad concrete except at the tip where turbulent flow of the fluid concrete may have caused some mixing with sediments in the bottom of the hole. Soil attached to the tip indicated the tip had been founded in a relatively soft mixture of clays which had fallen into the hole during construction. The color of these clays matched that of the clay layers in the upper portions of the hole. A low tip resistance was measured during load testing further confirming the presence of soft material at the tip of this shaft.

The shape of the shaft tip suggested that the tremie might have been in an off-center position when the first fluid concrete was introduced into the hole.

Shaft G 2

Extraction of this shaft was considerably more difficult than the other two due to caving conditions in the upper soil layers. A large casing was required around the shaft and the shaft was broken several times during extraction.

This shaft was generally cylindrical in shape with slight enlargements at depths of 18 feet, 32 feet, and two feet above the tip. These enlargements, approximately two feet in length, increased the shaft diameter by no more than three inches, and corresponded to the depths of silt strata. The shaft appeared to be out of plumb in the lower 10-20 feet, however, no attempt was made to measure the alignment of shaft. The tip of this shaft was well formed with the tremie plug in the center of the shaft and there was evidence of a good concentric flow of fluid concrete with good scouring action at the bottom of the shaft (Figure 6.6).

There was no coating of sand or drilling mud on this shaft as on the other two test shafts. The concrete had been in contact with the clay soil and no evidence of significant entrapped mud was found.

Shaft BB

This shaft was straight with a relatively smooth surface and had a near constant diameter. Slight enlargements were found near the tip and at the bottom of the section formed with the Sonotube.

The upper sections of the shaft contained small pockets of sand-rich mud at the junction of the spiral and reinforcing steel, along instrumentation wires, and around the load cells. Except at the tip, no contamination was found in the lower portion of the shaft. No significant amount of entrapped mud was found between the concrete and adjacent soil.

A coating of slightly clayey sand covered the entire length of the shaft. This coating was 1/16" - 1/4" thick, depending on the roughness of the concrete surface, and failure apparently took place in this layer. The sand adjacent to the concrete appeared to be somewhat stabilized.

The bottom five feet of this shaft had layers of clay clinging to it but they were separated from the concrete by the clayey sand layer. The different colors of the clay indicated that it had come from the strata near the ground surface. It is likely that this clay had fallen into the hole during drilling and then pushed to the sides of the hole by the fluid concrete. A smooth flow of concrete at the bottom of this shaft had apparently been prevented by the temporary tremie seal just as in Shaft G 1. The tip of Shaft BB is pictured in Figure 6.7.

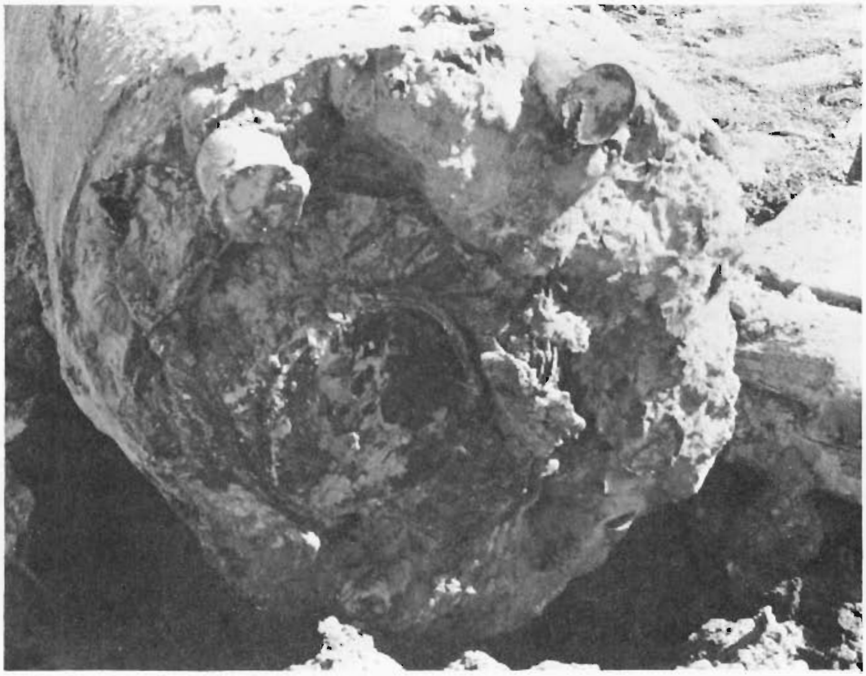


Figure 6.5. Tip of Shaft G1

Figure 6.6. Shaft G1

Figure 6.7. Tip of Shaft G2

Figure 6.8. Shaft G2

Figure 6.9. Close-up of Shaft G2

Figure 6.10. Tip of Shaft BB

Figure 6.11. Shaft BB

CHAPTER VII

BOND TESTS

A limited number of pull-out tests were performed to determine if the bond strength of the concrete was significantly affected by the "slurry displacement" method of placing concrete.

Two tests were performed using a section of shaft G 2 from a level approximately 65 feet below the ground surface. This section was 2'-6" in diameter and 5'-6" long. Concrete was removed and the No. 8 reinforcing bars cut to form two test specimens; one with a bar embedment length of 13 inches and one with an embedment length of 9 inches. Concrete cover was 2-5/8 inches on the 13-inch length and 2-1/4 inches on the 9-inch length.

The concrete over the 13-inch length did not fail by bond splitting. The direction of the applied load was not parallel to the axis of the bar and this caused a portion of the concrete cover to spall off. This failure occurred at an applied load of 55,500 pounds and a calculated bond stress of 1360 psi. The concrete over the 9-inch length failed by bond splitting at an applied load of 61,000 pounds and a calculated bond stress of 2160 psi.

Inspection of the concrete removed for the above tests did not reveal any significant slurry contamination in the concrete or around the reinforcing steel.

Additional pull-out tests were performed on 12 concrete specimens 12 inches in diameter, 14 inches long, and prepared as follows:

1. Three with No. 8 deformed bars coated with a mud slurry before casting in concrete.
2. Three with No. 8 deformed bars cast without a mud slurry coating.
3. Three with one-inch diameter smooth bars coated with a mud slurry before casting in concrete.
4. Three with one-inch diameter smooth bars cast without a mud slurry casting.

Two of the deformed bars cast with the slurry coating failed in a threaded area before concrete failure. These failed at loads of 54,000 and 56,000 pounds which represents an approximate bond stress of 1230 psi and 1275 psi. Using a different method for applying load, the third specimen with a slurry-coated deformed bar was loaded until the concrete failed by bond splitting. This occurred at a load of 61,500 pounds and a calculated bond stress of 1400 psi. The three specimens made with deformed bars without the slurry coating all failed by bond splitting. The average of the failure loads was 60,500 pounds with an average calculated bond stress of 1375 psi.

All of the specimens with smooth bars failed by slippage of the bars. The average of the failure loads for the specimens with slurry-coated bars was 5,800 pounds with an average calculated bond stress of 133 psi. The average of the failure loads for the specimens with un-coated bars was 10,700 pounds with an average calculated bond stress of 242 psi.

CHAPTER VIII

CONCLUSIONS

Load tests were conducted on three straight drilled shafts of constant diameter constructed using the "slurry displacement" method. The primary reasons for these load tests were: (1) to obtain a proven load capacity for design verification, (2) to evaluate the "slurry displacement" method of construction, and (3) to determine the load transfer characteristics of drilled shafts constructed by this method. Based on the results of these tests and the inspection of the extracted shafts, the following conclusions and recommendations are presented.

Conclusions

1. Straight drilled shafts designed to carry load by frictional resistance can safely be used in the areas tested. Results of the load tests indicated that a significant portion of the applied axial load was carried by frictional resistance.

2. Drilled shafts can safely be constructed in water bearing and caving soils using the "slurry displacement" method of construction. Con-

struction techniques have a significant influence on the condition of shafts constructed using this method and care should be exercised to prevent contamination of the concrete by the drilling slurry.

3. The values of load transfer developed in the soils surrounding the shafts constructed by the "slurry displacement" method were found to be comparable to those developed from previous research when shafts were constructed in the dry.

4. The tip resistance of shafts constructed by the "slurry displacement" method can be substantially reduced by loose materials falling into the hole and not being completely removed before placing concrete. This apparently occurred at shaft G1 where a low ultimate tip resistance was measured and material from upper levels of the hole were found clinging to the tip of the extracted shaft. This likely was the result of making the final clean-out of the hole with an auger rather than with a clean-out bucket. When an auger is used the soil cuttings have a tendency to wash off during withdrawl of the auger.

5. The "slurry displacement" method of placing concrete has no significant affect on the bond strength of the concrete when deformed bars are used and the load is applied axially. When smooth bars are used, the slurry will cause a reduction in bond strength. The average bond strength for the slurry coated smooth bars was 133 psi compared with 242 psi for the uncoated bars.

CHAPTER IX

IMPLEMENTATION

Based on the results from these tests and those conducted under Research Study No. 3-5-65-89, the following procedure has been adopted for utilization of frictional design for drilled shafts in the Houston area:

1. Use a Soil Reduction Factor* of 0.7 based on a shear strength obtained from THD Triaxial and/or THD cone penetrometer data.
2. Disregard the frictional capacity of the soil for the upper 10 feet of a single shaft.
3. Assume that frictional load transfer is independent of the shaft construction method used.

Evaluation of the "slurry displacement" method used to construct these test shafts resulted in approval of this method for construction of any straight shafts used in the I-45/I-610 (So. Loop) Interchange structures. The specifications governing this type of construction are included as Appendix C.

* See Appendix A for definition.

APPENDIX A

DETERMINATION OF SOIL REDUCTION FACTOR, S_R

RELATIONSHIP BETWEEN MEASURED AND CALCULATED LOAD TRANSFER

Previous research has shown that the load transferred from a drilled shaft to the surrounding soil is related to the shear strength of the soil. This relationship, even though sometimes greater than unity, is herein defined Soil Reduction Factor (S_R) and was determined using the following procedure:

1. Calculate for each soil stratum, a unit shear strength (tons/sq ft) based on Triaxial Test data and/or THD Cone Penetrometer data.
2. Multiply the unit shear strength from Step 1 by the stratum thickness to obtain a shear strength in <u>tons per foot of shaft perimeter</u>.
3. Make an accumulative total of the shear strengths from Step 2.
4. Calculate the potential ultimate load transfer for each soil type from the accumulative total shear strength in Step 3.
5. Prepare a load distribution curve (load vs. depth) for the "double-tangent" failure load obtained from load test data.

6. The ratio of measured load transfer from Step 5 to the calculated load transfer from Step 4 is the Soil Reduction Factor, S_R.

Values of measured unit load transfer, calculated total load transfer based on Triaxial Test Data and THD Cone Penetrometer Data, total load transfer measured during load tests, and S_R are shown tabulated in Table A.1. Values of S_R, based on data taken from Research Study 3-5-65-89 Reports, for a site approximately 2.5 miles east of Test Sites G1 and G2, are shown in Table A.2.

Table A.1. Frictional Load Transfer and S_R Values for Test Sites G1, G2, and BB

Site	Soil	Average Measured Unit Load Transfer (TSF)	Frictional Load Transfer Per Soil Type (Tons): Calculated Ult. THD Pen. Test	Calculated Ult. Triaxial Test	Measured	Soil Reduction Factor, S_R: THD Pen. Test	Triaxial Test
G-1	Clay	0.52	139	135	145	1.04	1.07
	Sand	1.05	400		280	0.70	--
G-2	Clay 1	0.81	230	272	320	1.39	1.17
	Clay 2	1.62	128	123	220	1.72	1.78
	Sand	1.79	74		125	1.69	--
BB	Clay	0.94	132	198	170	1.29	0.86
	Sand	1.53	378		290	0.77	--

Table A.2. Frictional Load Transfer and S_R Values for Load Tests at SH 225 Site Located Approximately 2.5 Miles East of Sites G1 and G2

Test	Soil	Frictional Load Transfer (Tons) Calculated THD Pen. Test	Frictional Load Transfer (Tons) Calculated Triaxial Test	Frictional Load Transfer (Tons) Measured	Soil Reduction Factor (S_R) THD Pen. Test	Soil Reduction Factor (S_R) Triaxial Test
S1 T1	Clay	88	110	97	1.10	0.87
S2 T1	Clay	65	87	86	1.32	0.99
S3 T1	Clay	88	110	121	1.37	1.08
S4 T1	Clay	237	259	191	0.81	0.74

APPENDIX B

BORING LOGS

LABORATORY LOG OF BORING #G-1-2
FOR
I-610 & I-45 INTERCHANGE

DATE May 21, 1971 TYPE THD Penetrometer LOCATION N 695,995 E 3,177,460

ELEVATION + 41.6

DESCRIPTION	BLOWS PER FOOT 1st 6"	2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT LBS/CU.FT.
Dk. gray tan silty clay	4	2			
Tan lt. gray w/calc. @ 8'	3	4			
Lt. gray tan w/o calc. @ 12'	7	8			
	7	9			
Tan lt. gray clayey silt w/calc. w/o calc. @ 20'	8	7			
	12	17			
Tan lt. gray silty clay Red lt. gray @ 26'	9	12			
	9	12			
	9	12			
Tan lt. gray silty sand 8" layer clay @ 40'	27	26			
	19	21			
	31	36			
Lt. gray tan @ 49'					
Tan lt. gray @ 52'	35	32			
	26	34			
	4½"	3¼'			
	6"	2¾'			
	4½"	3¾'			
	5"	3¾'			
	2¼"	1"			
	2½"	1½'			

POINT BEARING T.S.F. 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT 10 20 30 40 50

WATER

LABORATORY LOG OF BORING G-1-3 FOR

I-610 & I-45 INTERCHANGE

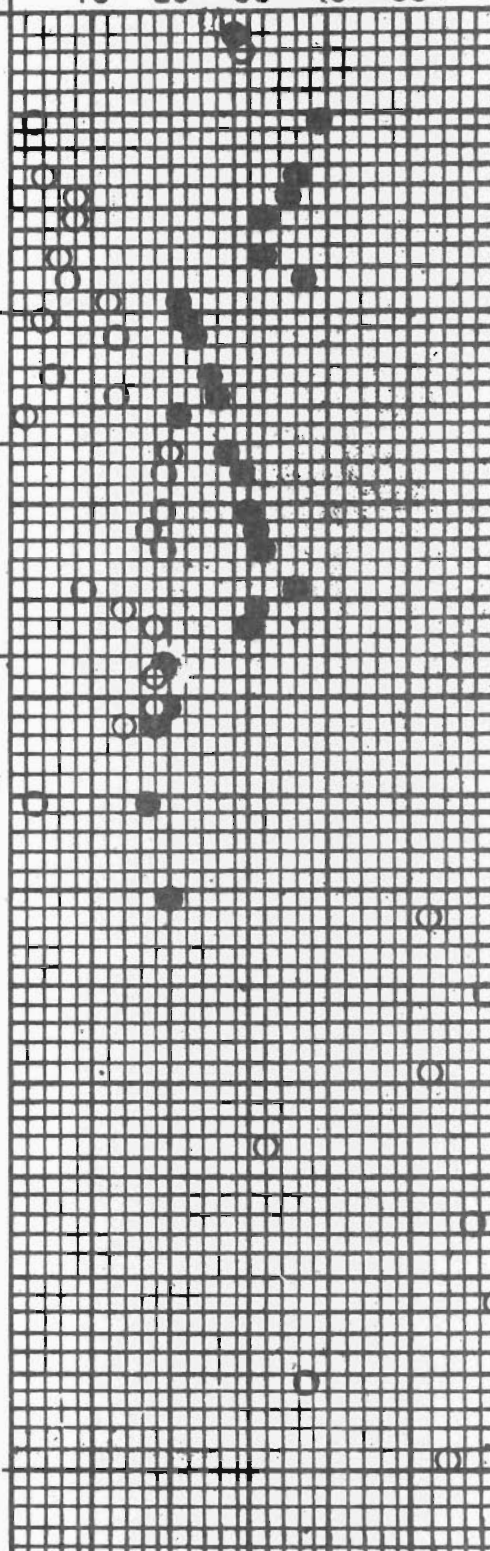
DATE May 25, 1971 TYPE 3" Shelby LOCATION N 695,975 E 3,177,440

ELEVATION + 42.0

DESCRIPTION	BLOWS PER FOOT 1st 6"	BLOWS PER FOOT 2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT. LBS/CU. FT.
Dk. gray tan silty clay					94
Tan lt. gray w/calc. @ 7'					86
					93
Tan lt. gray clayey Silt W/layers clay @ 17'					100
Red lt. gray silty clay Tan lt. gray @ 23'					96
					93
Tan lt. gray silty sand					110
	21	27			
					115
	23	26			
Lt. gray w/3" layer clay @ 46'	32	31			110
	32	40			
	32	31			
	19	20			
	39	31			
	4¼"	2¼"			
	22	23			
	5½"	2			

POINT BEARING T.S.F.: 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT: 10 20 30 40 50



WATER

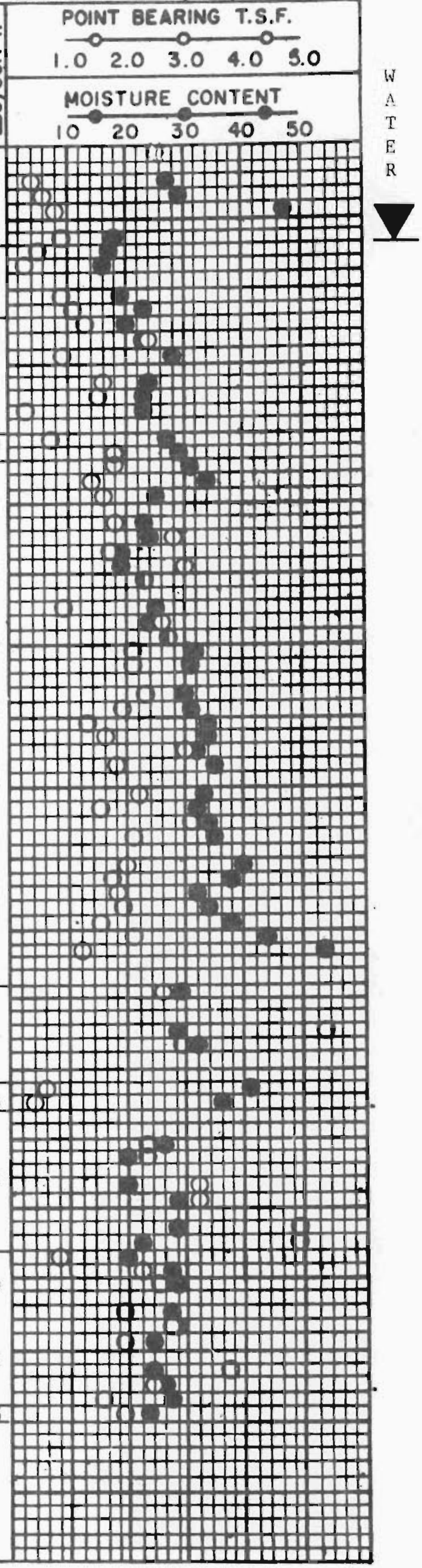
• Slickensided

LABORATORY LOG OF BORING G -2-2
FOR
I-610 & I-45 Interchange

DATE May 18, 1971 TYPE 3" Shelby LOCATION N 696,375 E 3,178,500

DEPTH FEET	DESCRIPTION ELEVATION + 39.6	BLOWS PER FOOT 1st 6"	2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT. LBS/CU.FT.
0	Dk. gray tan silty clay Tan lt. gray @ 4' W/silt pockets @ 6'					96
10	Tan lt. gray clayey silt w/layers of clay					106
	Tan lt. gray silty clay Lt. gray tan @ 14' Tan lt. gray @ 15' Red lt. gray @ 17'					104
20	Red lt. gray clayey silt w/clay layers					99
30	Tan lt. gray silty clay w/calc. W/o calc. @ 24' Red lt. gray @ 27' W/calc. @ 29' 3" layer silt @ 31' 2" layer silt @ 32'					96 108 98
40						95 94
50	Tan lt. gray @ 50' Dk. gray tan @ 52'					88 81
60	Gray tan silty sand 8" layer very sandy clay @ 62' 7" layer very sandy clay @ 63'					94
	Dk. gray tan silty clay					78
70	Gray tan silty sand 4" layer clay @ 76'					102
80	Red lt. gray silty clay 1" layer silt @ 85' 2" layer silt @ 86' 1" layer silt @ 88'					100 101
90	Red lt. gray clayey silt					95

● Slickensided



LABORATORY LOG OF BORING #G-2-3
FOR
I-610 & I-45 INTERCHANGE

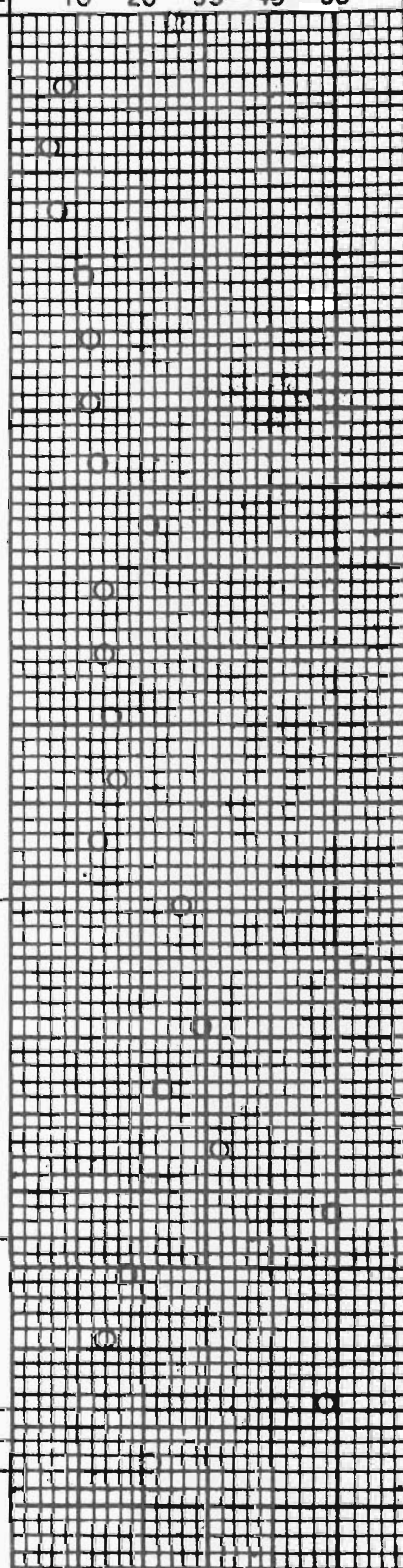
DATE May 19, 1971 TYPE THD Penetrometer LOCATION N 696,375 E 3,178,525

ELEVATION + 39.4

DEPTH FEET	DESCRIPTION	BLOWS PER FOOT 1st 6"	BLOWS PER FOOT 2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT. LBS/CU.FT.
0	Dk. gray tan silty clay					
	Tan lt. gray @ 4'	7	5			
	W/silt layers @ 7'	3	4			
10	W/calc. @ 10'					
	W/o silt layers @ 13'	5	6			
	Red lt. gray @ 18'	8	10			
20		11	10			
		10	11			
		10	12			
30	8" layer silt @ 32'	20	18			
		12	13			
40		12	12			
		13	13			
50	Lt. gray @ 50'	14	14			
	Dk. gray @ 54'	10	12			
	Dk. gray tan silty sand	16	15			
60		30	34			
		20	15			
70		14	14			
		20	19			
		3¼"	6"			
80	Red lt. gray silty clay	13	18			
		13	12			
		19	81 / 5¼"			
70	Red lt. gray clayey silt					
	Red lt. gray silty clay	19	18			

POINT BEARING T.S.F. 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT 10 20 30 40 50



WATER

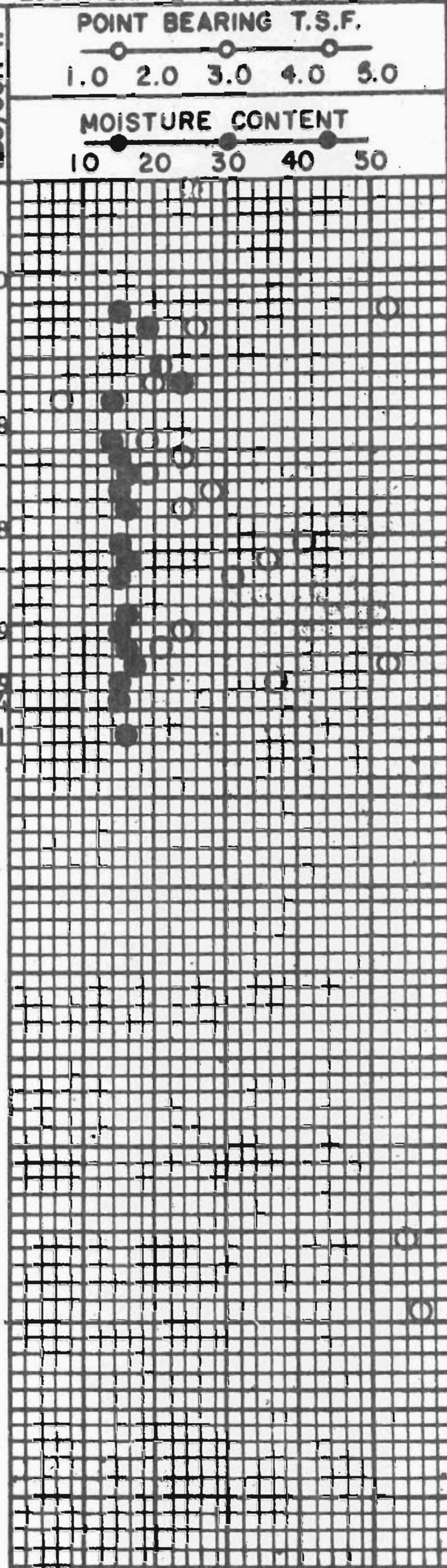
LABORATORY LOG OF BORING #B-B-2
FOR
STATE HIGHWAY 288

DATE May 27, 1971 TYPE 3" Shelby LOCATION N 700,965 E 3,149,205

ELEVATION + 50.0

DEPTH FEET	DESCRIPTION	BLOWS PER FOOT 1st 6"	BLOWS PER FOOT 2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT LBS/CU.FT.
0	Gray tan silty clay w/calc. Lt. gray tan @ 5'					110
10	• Tan lt. gray clayey silt w/calc.					118
	Tan lt. gray very silty clay w/calc. & sand pockets Lt. gray tan @ 18' W/o sand pockets @ 18½'					118
20	Lt. gray tan sandy clay w/calc. Very sandy @ 27'					119
	Tan lt. gray clayey sand					11[illegible] 114
30	Tan lt. gray silty sand					111
		32	49			
40		2"	½"			
		2"	3"			
		3½"	1¾"			
50	8" layer sand stone @ 53'	4½"	½"			
		1"	¾"			
60		5¼"	2¼"			
		5½"	1½"			
70						

• Slickensided



WATER

LABORATORY LOG OF BORING B-B-3
FOR
STATE HIGHWAY 288

DATE May 28, 1971 TYPE THD Penetrometer LOCATION N 700,960 E 3,149,215

ELEVATION + 50.1

DEPTH FEET	DESCRIPTION	BLOWS PER FOOT 1st 6"	BLOWS PER FOOT 2nd 6"	COHESION P.S.I.	FRICTION ANGLE	UNIT DRY WT. LBS./CU. FT.
0	Gray tan silty clay w/calc. Lt. gray tan @ 4'					
		13	14			
10		11	9			
	Tan lt. gray clayey silt	14	14			
	Lt. gray tan silty clay w/vertical silt seams	9	12			
20		16	21			
		9	19			
30	Lt. gray tan clayey sand					
	Lt. gray tan silty sand	41	49			
		24	29			
		48	52 / 5¾"			
40		2"	1"			
		2½"	1¼"			
50		3¼"	2"			
		1¾"	1¾"			
		3½"	3¾"			
60						
70						

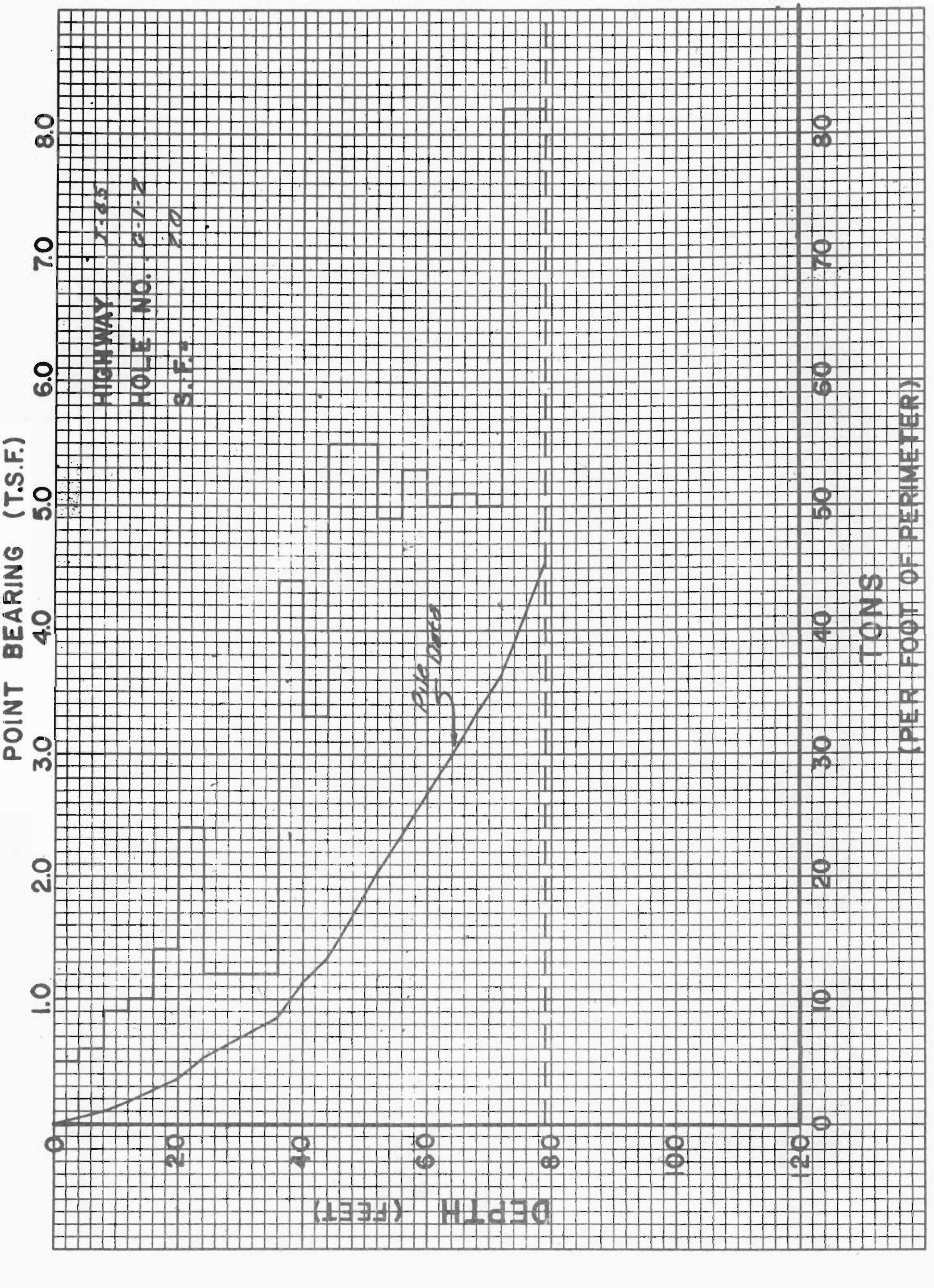
POINT BEARING T.S.F. 1.0 2.0 3.0 4.0 5.0

MOISTURE CONTENT 10 20 30 40 50

WATER

APPENDIX C

PLOTS OF ACCUMULATED SOIL SHEAR STRENGTHS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8.0
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HOLE NO.
S.F.=
TONS
(PER FOOT OF PERIMETER)
0
10
20
30
40
50
60
70
80
DEPTH (FEET)
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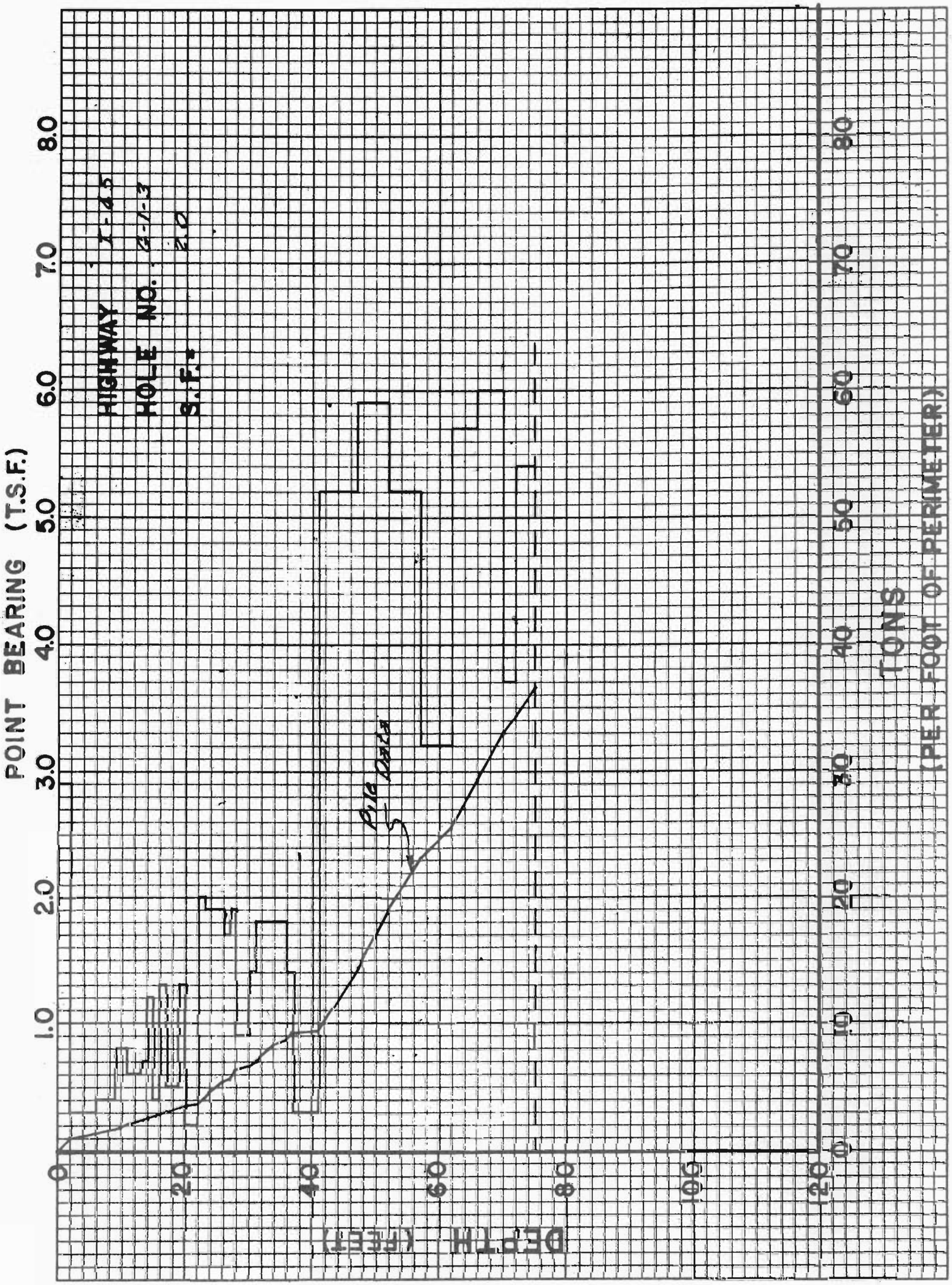
20
40
60
80
100
120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HOLE NO. G-1-3
S.F.= 2.0
Pile Data
DEPTH (FEET)
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80
100
120
TONS
(PER FOOT OF PERIMETER)
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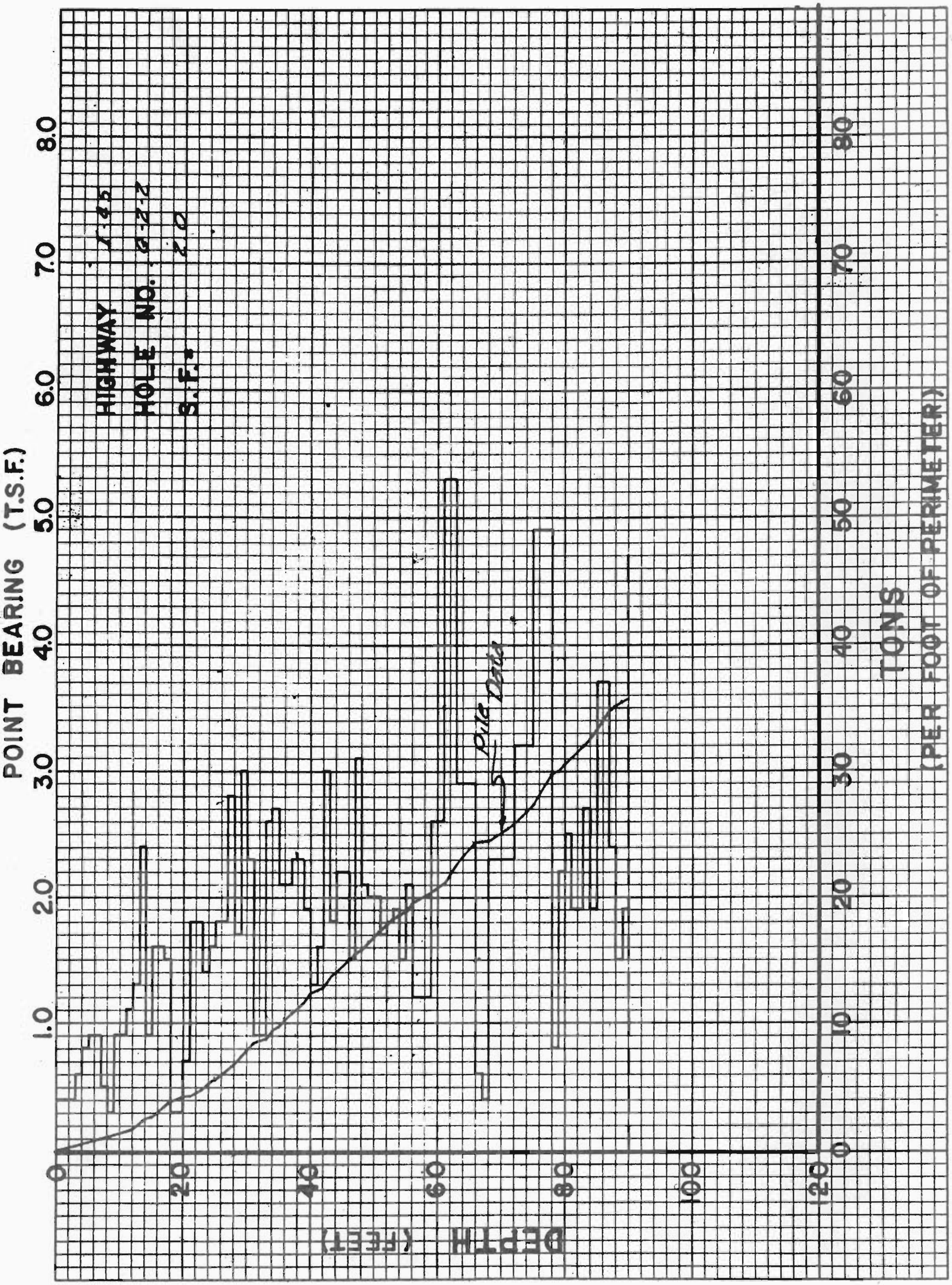
20
30
40
50
60
70
80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HOLE NO.
S.F. 2.0
Pile Data
TONS
(PER FOOT OF PERIMETER)
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DEPTH (FEET)
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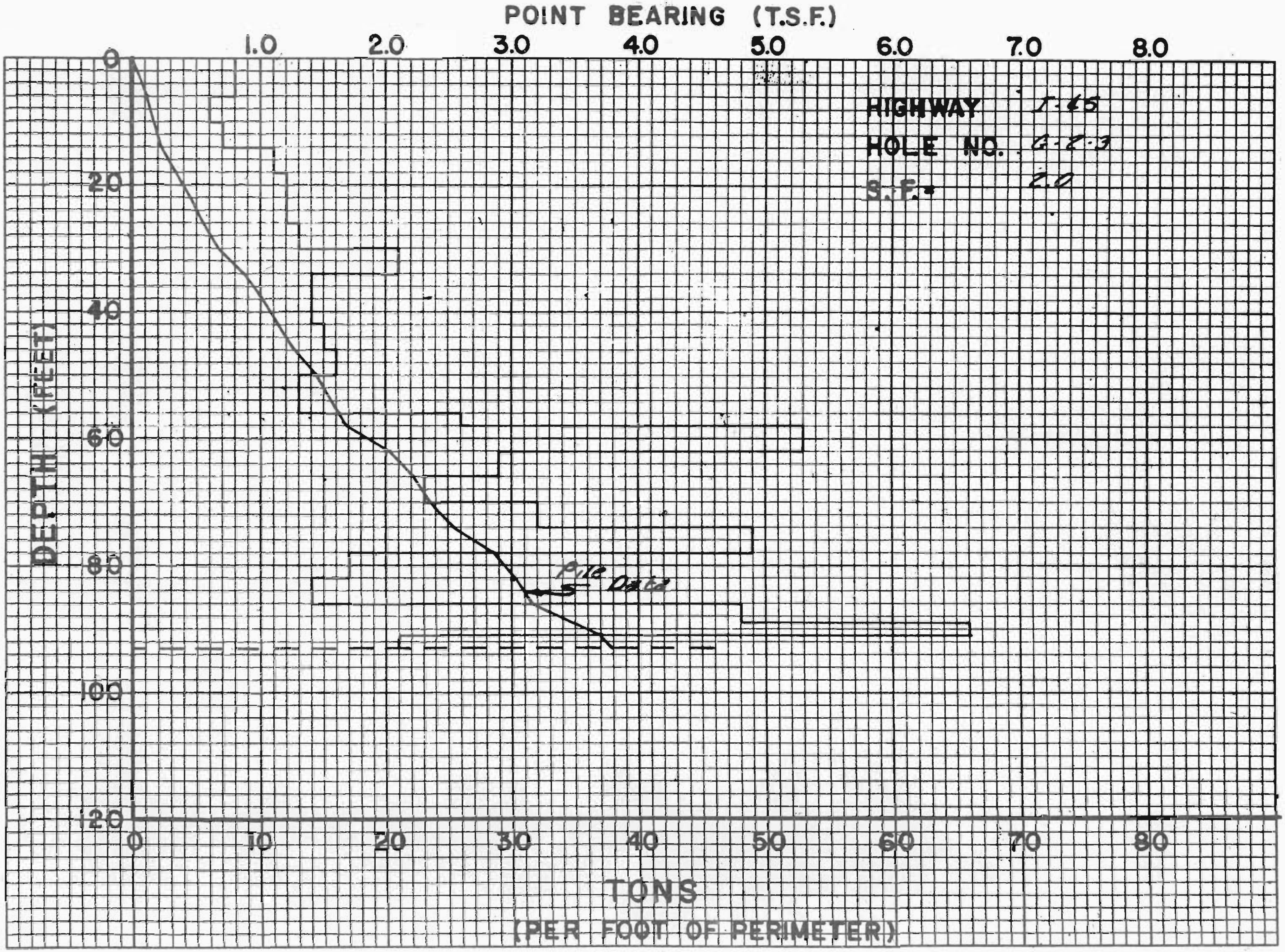
100
120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HOLE NO. G-2-3
S.F. 2.0
Pile Data
DEPTH (FEET)
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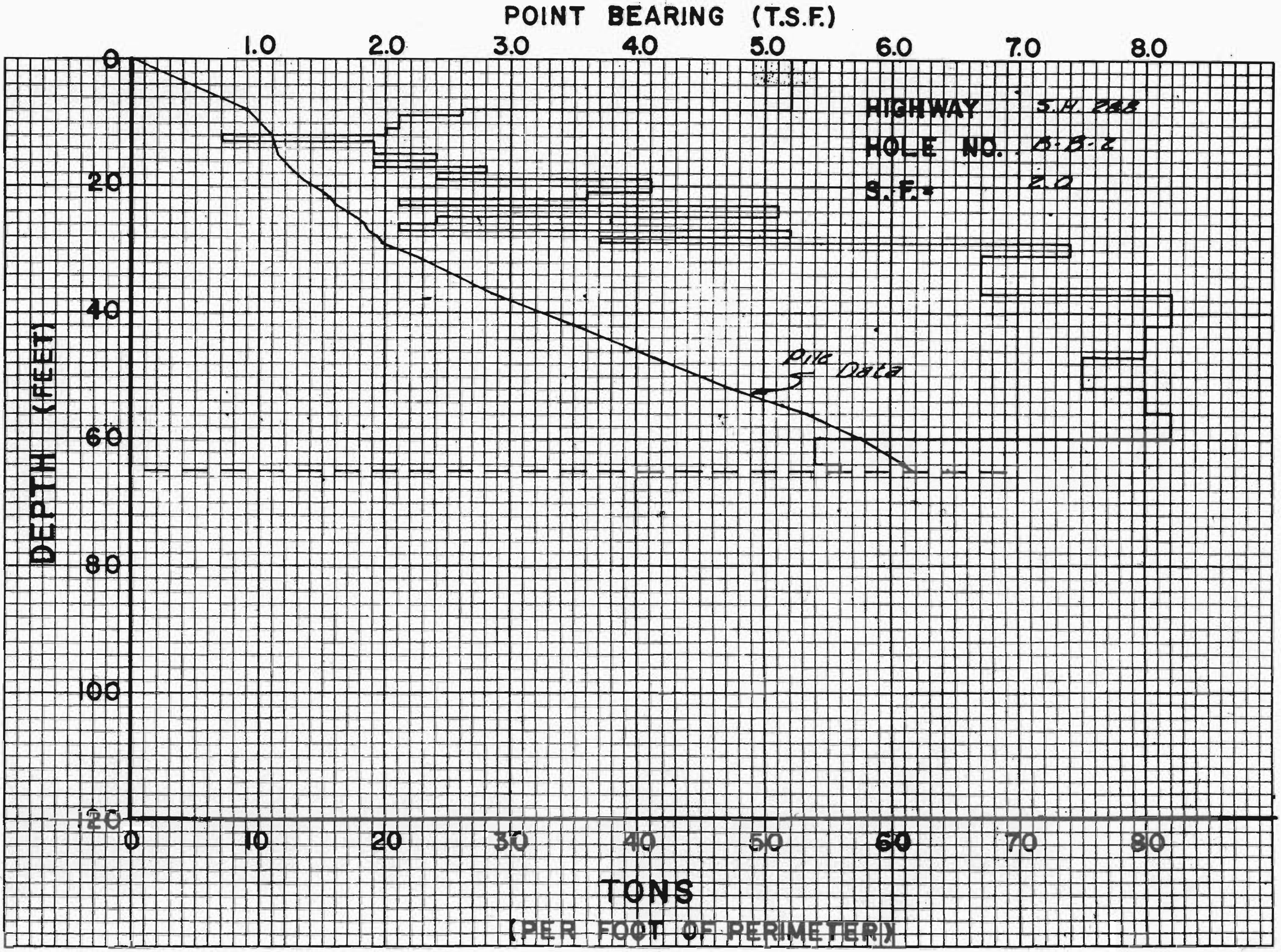
TONS
(PER FOOT OF PERIMETER)



POINT BEARING (T.S.F.)
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HIGHWAY S.H. 288
HOLE NO. B-B-2
S.F.= 2.0
Pile Data
DEPTH (FEET)
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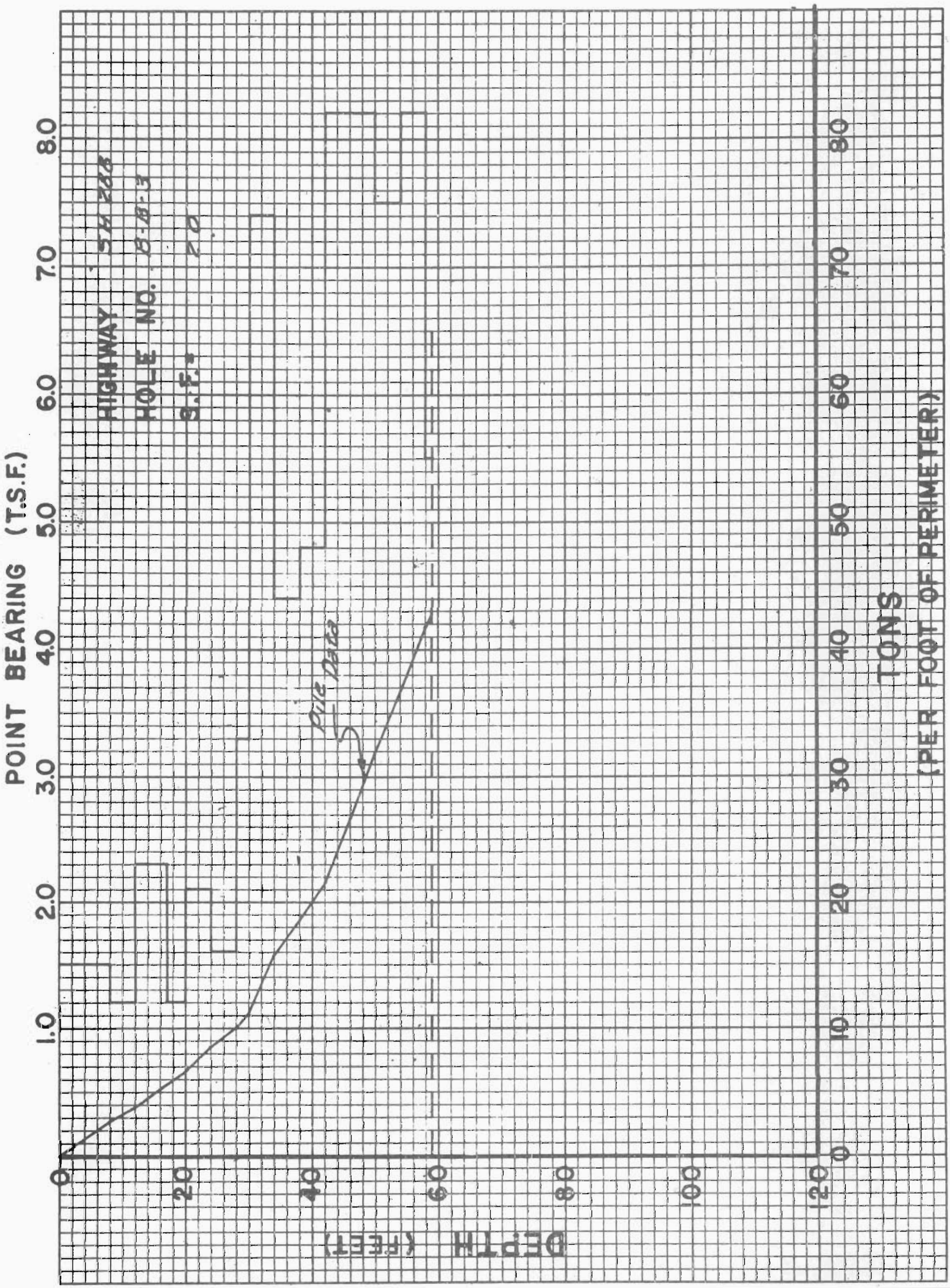
TONS
(PER FOOT OF PERIMETER)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HIGHWAY SH 288
HOLE NO. B-B-3
S.F.
Pile Data
DEPTH (FEET)
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20
40
60
80
100
120
TONS
(PER FOOT OF PERIMETER)
0
10
20
30
40
50
60
70
80

APPENDIX D

SPECIAL PROVISION TO ITEM 416

(416---002) OF TEXAS HIGHWAY

DEPARTMENT STANDARD SPECIFICATIONS, 1972

TEXAS HIGHWAY DEPARTMENT

SPECIAL PROVISION

TO

ITEM 416

DRILLED SHAFT FOUNDATIONS

For this project the Item, "Drilled Shaft Foundations" is hereby amended with respect to the clauses cited herein. No other clauses or requirements of this Item are waived or changed hereby.

Article 416.1. Description is supplemented by the following:

The "Slurry Displacement" method is defined as a construction procedure whereby the sides of the excavation are supported, all or in part, by a mud slurry and the slurry displaced by concrete, to form a continuous concrete shaft.

The "Slurry Displacement" method may be used, at the Contractor's option, to construct any drilled shaft not requiring bell footings. Regardless of the method of shaft construction, the length shown on the plans will be the minimum length placed and the length to be paid for, unless modified by design change.

One interior drilled shaft in Bent 11, Structure 37, will be instrumented with strain gages, mustran cells and other incidentals, installed on the reinforcing steel cage. The instrumentation cables for the gages will be brought out at the construction joint at the top of the shaft. Lead wires will be collected into a bundle at the top of the shaft in such a manner to not interfere with the placing of column forms. The instrumentation, installation and the completion of the instrumentation setup for long-term readings will be handled by State forces.

Article 416.2. Materials. This article is supplemented by the following:

Concrete for the "Slurry Displacement" method shall be Class "E", modified to contain a minimum of 7 sacks of cement per cubic yard, and a maximum grade 3 Coarse Aggregate. The slump shall be as required for use in a cased drilled shaft.

Article 416.3. <u>Construction Methods</u>. Subarticle (1) <u>Excavation</u>, is supplemented by the following when the "Slurry Displacement" method is used:

The shaft excavation, during and after drilling operations, shall be completely filled with slurry. When a surface casing is used, the slurry shall fill the excavation to at least 2 feet above the bottom of the casing. The casing shall not be extracted until after the concrete placing operations have been completed.

A good grade commercial bentonite of the type commonly used in the drilling of oil wells shall be mixed with water and excavation cuttings to produce a viscous slurry capable of supporting the sides of the excavation and to hold the excavation cuttings in suspension. A <u>minimum</u> of 30 pounds of bentonite per cubic yard of slurry will be required. If sufficient ground water is not available to produce the required slurry, additional water shall be supplied by the Contractor.

Just prior to placement of concrete, the drilling auger and/or other acceptable tools shall be passed down and up the excavation to free it of any large obstruction that may have fallen from its sides between the cessation of drilling operations and the placing of concrete.

If the mud slurry 'sets up' or forms a gel prior to concrete placement, the gelled slurry shall be agitated to liquification just prior to concrete placement and at other times when directed by the Engineer.

A sump pit adjacent to and connected with the shaft excavation, or a pump and portable container may be used to collect the slurry displaced by the concrete. The displaced slurry, if not contaminated, may be re-used in subsequent drilling operations.

Article 416.3. <u>Construction Methods</u>. Subarticle (2) <u>Reinforcing Steel</u>, is supplemented by the following:

The reinforcing cage may be positioned before or after concrete placement. However, should difficulty arise in submerging and positioning the reinforcing cage after concrete placement, the Engineer may direct that on subsequent drilled shafts the reinforcing cage be installed prior to the concrete placing operation. Suitable guides will be required at the lower end of the reinforcing cage to assist in centering it in the excavation.

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The entire cage of reinforcing steel for the instrumented shaft shall be the full shaft length and shall be completely assembled and available to the State two days prior to installation in the shaft to allow the placement of instrumentation. The Contractor shall use utmost care in handling the steel cage and placing concrete. The instrumented reinforcing cage shall be installed prior to the placing of concrete. Suitable guides will be installed on the reinforcing steel to keep the tremie from damaging the instrumentation.

Article 416.3. Construction Methods. Subarticle (3) Concrete, is supplemented by the following:

For the "Slurry Displacement" method, the concrete shall be placed by the tremie method in accordance with Article 420.14, "Placing Concrete in Water".

If it appears that the continuity of concreting has been compromised due to withdrawal of the submerged end of the tremie tube prior to completion of concrete placement, the tremie shall be removed, resealed at the bottom, forced well into the concrete already placed and recharged prior to progressing further, and the Contractor will be required to core the entire length of the completed shaft or otherwise prove that the shaft is free from inclusions or contamination unless this requirement is specifically waived in writing by the Engineer.

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