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16. Abstract

This study is concerned with the analysis of the behavior of three full-scale instrumented drilled shafts under axial loading. Two of the shafts were in soil profiles containing sand and clay while the third was entirely in clay.

The data from the load tests were correlated with soil properties and the correlations were compared with results from previous research. Based on the test results and a reevaluation of previous tests, some improvements in the design criteria for drilled shafts in clay have been recommended.

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BEHAVIOR OF THREE INSTRUMENTED DRILLED SHAFTS UNDER SHORT TERM AXIAL LOADING

by

Donald E. Engeling
Lymon C. Reese

Research Report Number 176-3

The Behavior of Drilled Shafts
Research Project 3-5-72-176

conducted for

The Texas Highway Department

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

by the

CENTER FOR HIGHWAY RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

May 1974

PREFACE

The funds for this study were provided by the Texas Highway Department through the Center for Highway Research. Test 1 was conducted in Bryan, Texas, by a cooperative effort of the Texas Highway Department and the Center for Highway Research. Tests 2 and 3 were conducted in Puerto Rico as a combined effort of The University of Texas at Austin personnel, personnel of Farmer Foundation Company of Houston, Texas, and Puerto Rican engineers. The data from Tests 2 and 3 were made available to the Center for Highway Research and are included because of their value to the research being conducted.

The authors' appreciation is extended to Dr. Ken Stokoe for reviewing the manuscript and making helpful suggestions. Special thanks are also due to a number of people who contributed to this report. The field work was completed with the assistance of Mr. Harold Dalrymple, Mr. James Anagnos, Mr. Fred Koch, Mr. John Wooley, Mr. John Havelka, and Mr. Jerry Havelka. The planning and execution of the work were done with the cooperation of Messrs. H. D. Butler, Horace Hoy, and Guy R. Ward. Mr. Glyen Farmer deserves special recognition for his invaluable aid in the conduct of the research. Finally, the patience and excellent work of Mrs. Barb Johnston, who typed and assisted in the preparation of this manuscript, are greatly appreciated.

Don Engeling

Lymon C. Reese

May 1974

LIST OF REPORTS

Report No. 176-1, "The Behavior of Axially Loaded Drilled Shafts in Sand," by Fadlo T. Touma and Lymon C. Reese, presents the results of an investigation of the behavior of drilled shafts in sand.

Report No. 176-2, "The Behavior of an Axially Loaded Drilled Shaft Under Sustained Loading," by John A. Wooley and Lymon C. Reese, presents the results of an investigation of a drilled shaft under sustained loading.

Report No. 176-3, "Behavior of Three Instrumented Drilled Shafts Under Short Term Axial Loading," by Donald E. Engeling and Lymon C. Reese, is concerned with the analysis of the behavior of drilled shafts under axial loading.

ABSTRACT

This study is concerned with the analysis of the behavior of three full-scale instrumented drilled shafts under axial loading. Two of the shafts were in soil profiles containing sand and clay while the third was entirely in clay.

The data from the load tests were correlated with soil properties and the correlations were compared with results from previous research. Based on the test results and a reevaluation of previous tests, some improvements in the design criteria for drilled shafts in clay have been recommended.

SUMMARY

This study presents detailed information on three load tests of instrumented drilled shafts. Test 1 was near Bryan, Texas, and Tests 2 and 3 were at San Juan, Puerto Rico. Information is presented in the report on the test system that was employed at each of the sites and on the test procedures which were employed. Also presented are the results for each of the tests in the form of a load-settlement curve and load distribution curves.

Comprehensive soil studies were performed at each of the sites. Some difficulties were encountered in evaluating soil parameters, particularly at the Puerto Rico sites. Soil at Puerto Rico was hard and brittle and was particularly difficult to trim; therefore, the results of laboratory tests of "undisturbed" samples are questionable.

The data from each of these tests were analyzed and the results indicate the portion of the load at each site which is carried in skin friction and the portion which is carried in point resistance. These data are correlated with soil properties in order to develop design information.

The last chapter of the report includes a summary of design criteria for drilled shafts in clay and for drilled shafts in sand. Much of the information in the summary is from previous reports; however, on the basis of results from these three additional tests, some significant changes are proposed in the design criteria for drilled shafts in clay.

IMPLEMENTATION STATEMENT

This study presents information relevant to the design of drilled shafts. The results of three load tests of instrumented drilled shafts are presented, adding considerably to the information on the manner in which these types of foundations behave under axial load. The results from the load tests were analyzed, correlations were made with soil conditions at the sites, and recommendations are made for the design of drilled shafts.

The design recommendations are based on a relatively small number of tests and consequently are somewhat conservative; however, the design recommendations do allow significant savings to be made because the designs can be made considering both point resistance and side resistance.

Further field studies are needed to continue to improve design recommendations. The studies reported herein allow use of a higher unit load transfer than previously and additional improvements can be made in a number of other features of the design recommendations if there are available data from other load tests on instrumented drilled shafts.

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CHAPTER I

INTRODUCTION

GENERAL

A drilled shaft is a deep foundation constructed by drilling a hole and casting concrete directly against the soil. For axial loads, the shaft provides vertical support by both end bearing and side friction. The base of the shaft may be belled to increase the capacity of the shaft. Drilled shafts have a wide variety of uses in the construction of highways and buildings and the recent trend has been toward increased use, even in unstable soils.

CONSTRUCTION PROCEDURES

Drilled shafts are constructed by several methods (O'Neill and Reese, 1970). Perhaps the most common procedure is the so-called "dry end" method where a hole is excavated by drilling, the reinforcing steel is inserted, and concrete is then placed. The "dry end" method permits rapid construction and is employed in the absence of water-bearing sands and when drilling can be completed quickly enough to circumvent caving in clay and silt soils.

Another procedure is the so-called "casing" method. This method is used when dry hole drilling is not possible and involves the use of a drilling mud during advancement of the hole. The drilling mud is placed in the hole when water-bearing or caving materials are encountered and is left in the hole during drilling. When a bearing stratum is reached below the caving materials, a casing is placed in the hole and is sealed at its tip. The drilling mud is bailed from the interior of the casing, and a slightly smaller drill is used to advance the hole to its final depth. At this point the reinforcing steel is inserted and concrete is then placed as before. However, under these conditions it is necessary to have a considerable head of concrete in the casing when it is pulled to prevent the drilling mud which has been trapped behind the casing from flowing into the hole, causing a weak zone in the shaft.

A third method of construction has recently been developed for caving soils in which drilling mud is carried to the full depth of the excavation. Reinforcing steel is placed directly in the drilling mud and concreting is accomplished by use of a closed tremie which is kept at the base of the shaft during the concreting procedure. The concrete displaces the drilling mud and concreting is stopped when good quality concrete appears at the ground surface. A series of load tests were performed in Houston, Texas using the slurry displacement method and the results were quite favorable (Touma and Reese, 1972).

SCOPE

In 1965 the Center for Highway Research of The University of Texas at Austin began a study to investigate the behavior of drilled shafts under axial loading. To date, thirteen instrumented drilled shafts have been installed and tested at various locations. Eleven of these tests, including a long-term test now in progress, were in the state of

Texas and the remaining two tests were in Puerto Rico. Fig. 1.1 shows the location of the test sites in Texas. Test sites I through IX have been the subject of previous research and have been reported by O'Neill and Reese (1970), Barker and Reese (1970), and Touma and Reese (1972). Test site XI and the two tests in Puerto Rico constitute the subject of this report.

In this report the load test at site XI will be referred to as Test 1 and the load tests in Puerto Rico will be referred to as Test 2 and Test 3.

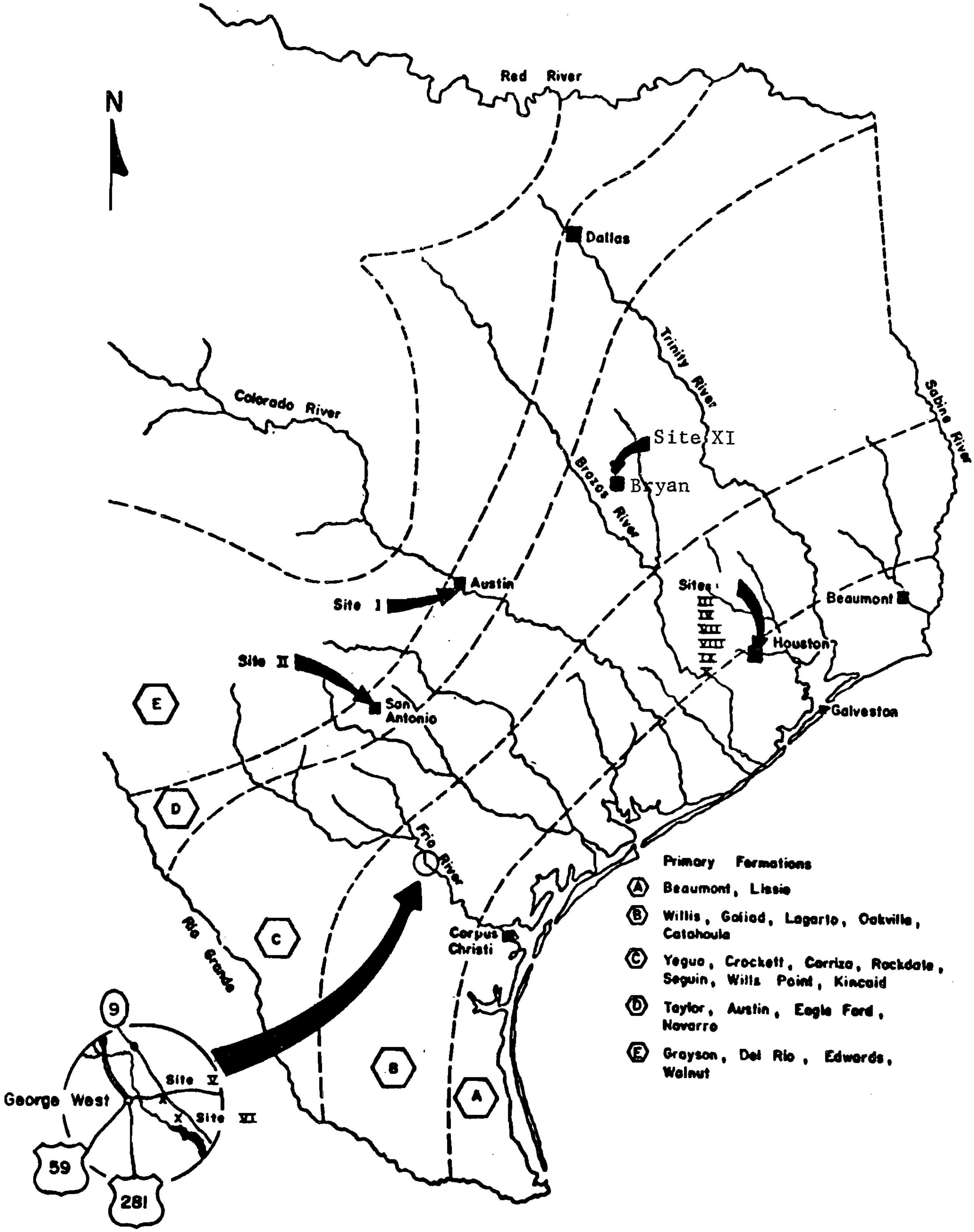


Fig. 1.1 Location of the Different Test Sites (Adapted after Barker and Reese, 1970)

CHAPTER II

SITE CONDITIONS

TEST 1 - BRYAN, TEXAS

Site Location

Test 1 was located approximately ten miles west of Bryan, Texas adjacent to State Highway 21. The load test was performed in conjunction with the construction of a new bridge over the Little Brazos River. The total length of the structure is to be 1,120 feet with span lengths up to 70 feet. In the structure there will be 17 individual bents, each supported by an average of three columns. The completed structure will require in excess of 2,000 linear feet of drilled shafts with diameters ranging from 18 to 36 inches. Average column loads will be on the order of 150 tons.

The site selected for the test shaft was in Bent 5 near the west end of the bridge. The test was designed so that the reaction shafts could be used as column foundations in the completed structure.

Soil Profile

The soil profile was determined from three borings located near the test shaft. The first two borings, designated as Research Borings 1 and 2, were located about 39 feet south and about 24 feet north of the test shaft, respectively, and were sampled and logged by personnel of the

Texas Highway Department. The third boring, designated as Research Boring 3, was logged by personnel from The University of Texas and was located about 25 feet west of the test shaft.

The test borings were drilled with a truck-mounted rig, capable of boring and sampling to depths in excess of 100 feet. Undisturbed samples were obtained using thin-walled sample tubes which were 24 inches long and 3.0 inches in diameter. The tubes were prepared and used according to standard procedures of the Texas Highway Department (Texas Highway Department, 1964). From a sample tube three samples 6 inches long and 3.0 inches in diameter were obtained. The samples were extruded at the site, identified, and sealed with parafix wax in a cardboard container. Each sample was stored at 20°C in a moist room until testing could be performed. In addition to undisturbed samples, the soil was evaluated by use of the cone penetrometer, a dynamic test developed by the Texas Highway Department. The test and the evaluation of shear strength from the data are described by Touma and Reese (1972). The results of the penetrometer test at Research Boring 2 are shown in Fig. 2.1.

The soil profile shown in Fig. 2.2 was constructed by using data from all three borings. As shown in the figure, the profile is almost entirely clay of medium to high plasticity with the exception of a three-foot thick, water-bearing silt layer at a depth of 29 feet.

Laboratory Tests

Laboratory tests were conducted by personnel from both the Texas Highway Department and The University of Texas. Samples from Research

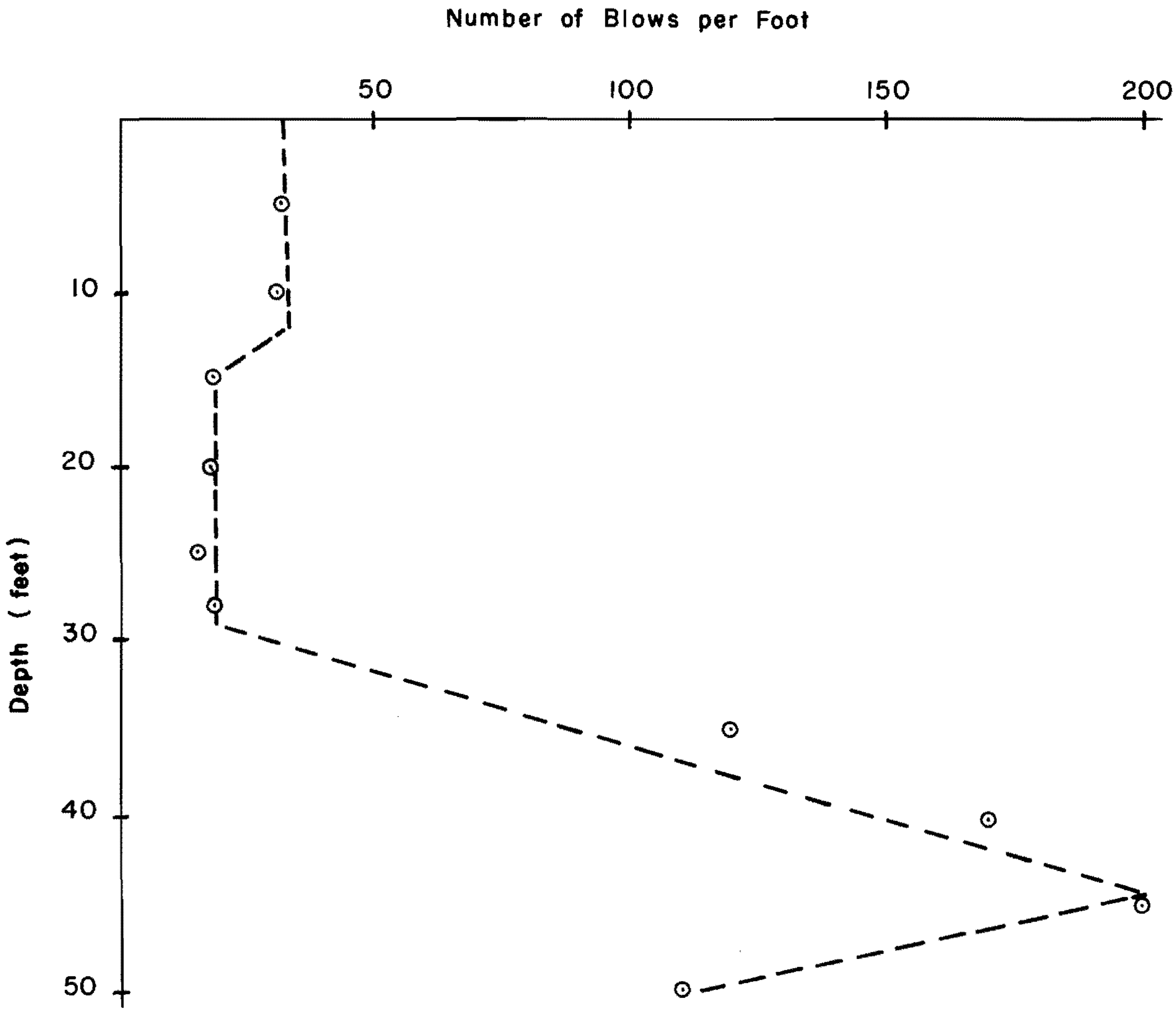


Fig. 2.1 Profile of THD Cone Penetrometer - Test 1

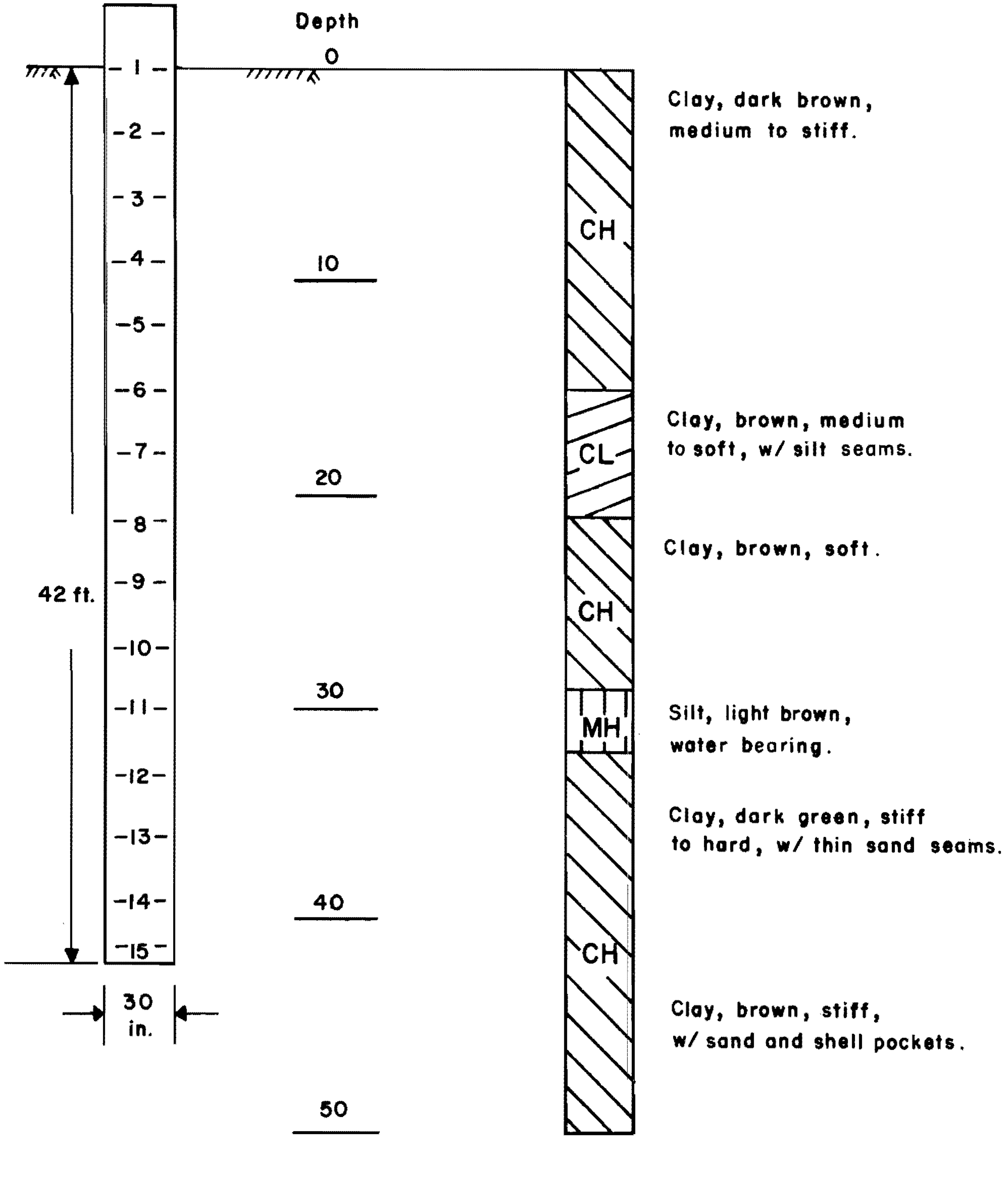


Fig. 2.2 Soil Profile and Shaft Instrumentation - Test 1

Boring 2 were tested by personnel of the Texas Highway Department using the THD-triaxial test procedure, while samples from Research Boring 3 were tested by The University of Texas personnel. The tests conducted on the latter samples included Atterberg limits, water content, and unconsolidated-undrained triaxial tests.

Atterberg Limits. Determination of water content and Atterberg limits were performed in accordance with standard laboratory procedures. The test results are shown in Fig. 2.3 along with the soil classification according to the Unified System.

Triaxial Tests. Personnel of the Texas Highway Department tested samples from Research Boring 2 using the THD triaxial device. The THD cell and testing procedure is described in detail in the Texas Highway Department Manual of Testing Procedures. The samples tested were three inches in diameter and six inches long and were loaded at a constant rate of strain. Results of the tests are tabulated in Table 2.1 and are shown plotted in Fig. 2.4.

Samples from Boring 3 were tested by The University of Texas personnel using a controlled-rate-of-strain procedure. The samples were tested under unconsolidated-undrained conditions at a confining pressure equal to the total overburden pressure. Whenever possible the samples were trimmed to 1.4 inches in diameter by 3.0 inches in length. Such trimming was possible for the samples above 30 feet but below 30 feet the samples were fissured and contained calarceous deposits and shell pockets which prevented extensive trimming. These samples were tested at the ex-

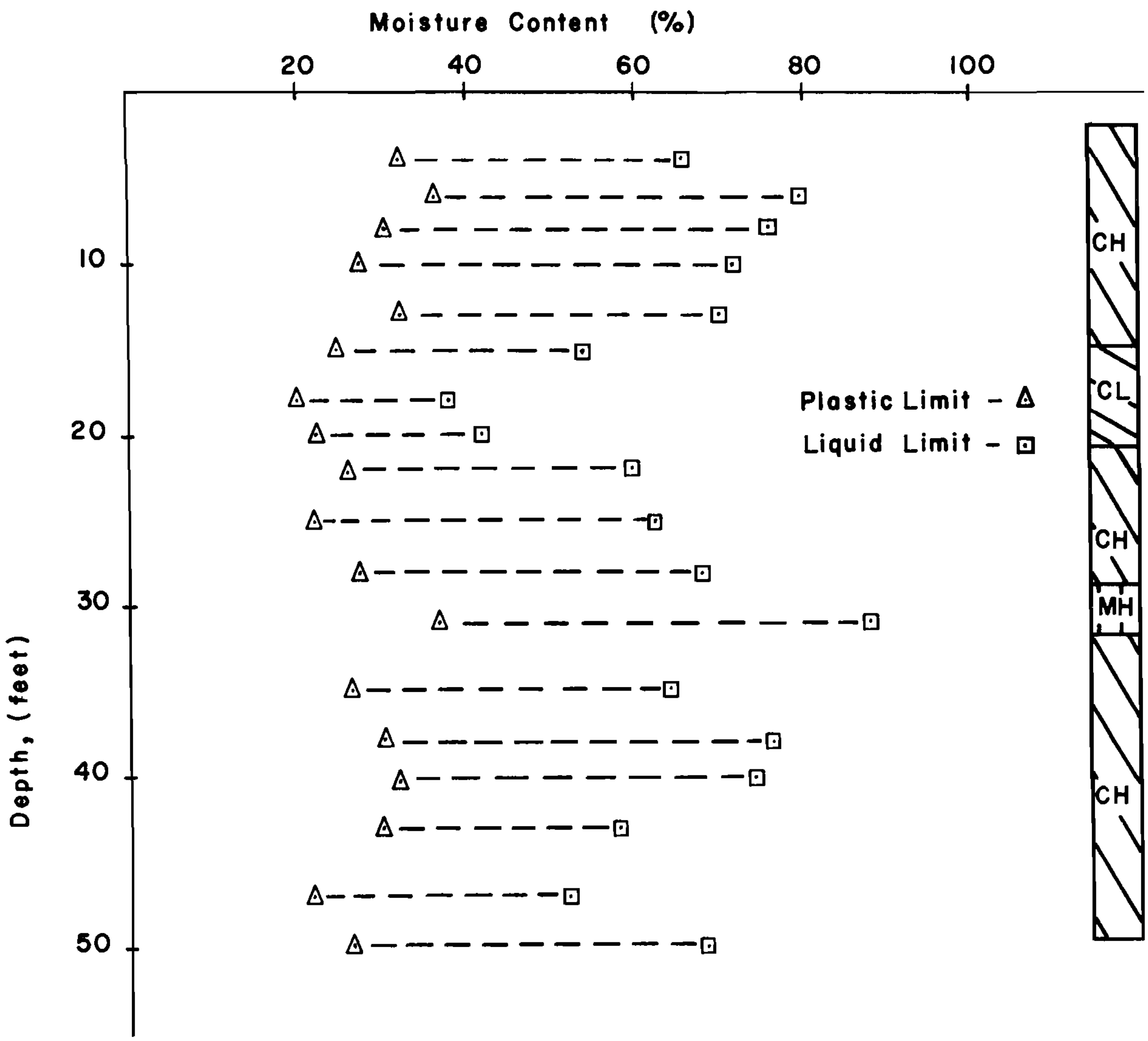


Fig. 2.3 Atterberg Limits for Test 1

TABLE 2.1

RESULTS OF THD TRIAXIAL TESTS

Depth (feet)	Angle of Internal Friction (Degrees)	Cohesion (Psi)	Overburden Pressure (Psi)	Shear Strength (Psi)
0- 5	No Test Data	-	-	-
5- 7	26.5°	29	5.5	30.5
7- 9	0°	20	7.5	20.0
9-12	43.0°	15	9.7	19.6
12-14	30.5°	13	11.9	16.1
14-16	7.0°	21.5	13.9	22.4
16-18	7.0°	22.5	15.2	23.4
18-20	0°	15	17.1	15.0
20-22	25.0°	7	18.8	11.4
22-24	16.0°	10	20.1	12.9
24-26	22.5°	7.5	21.8	11.9
26-28	19.0°	3.8	23.0	7.6
28-32	23.0°	3.5	24.5	8.5
32-42	0°	48	31.0	48.0
42-44	27.0°	25.5	36.8	34.7
44-46	23.0°	26	39.4	34.4
46-48	32.5°	13.5	40.3	26.1
48-50	20.0°	31.8	42.4	38.9

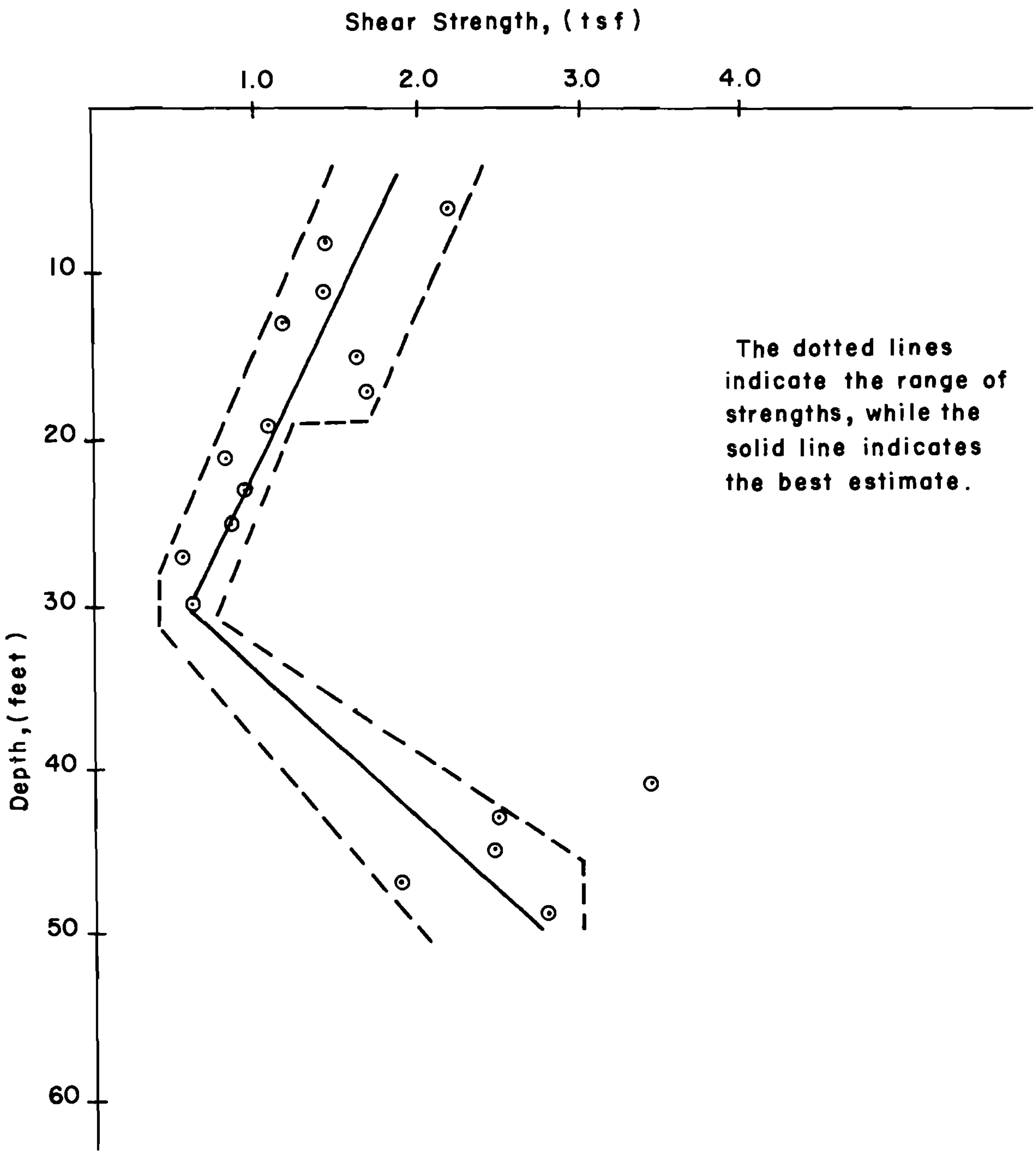


Fig. 2.4 Profile of Peak Shear Strength from THD Triaxial Tests - Test 1

truded diameter of 3.0 inches and were trimmed on the ends to lengths of about six inches.

The results of the U. T. triaxial tests are tabulated in Table 2.2 and are shown plotted in Fig. 2.5. The results are somewhat erratic due mainly to the presence of slickensides, calcarceous nodules, and silt seams. The tabulated results were obtained from the triaxial stress-strain curves.

Shear Strength Profile. Strength profiles obtained from the various methods are shown in Fig. 2.6 for purposes of comparison. For the dynamic penetrometer tests, shear strength values were obtained by use of correlation curves published by the Texas Highway Department. It is apparent that the magnitude of the shear strength is dependent on the type of test used. The same trend in strength is found in all of the tests, but the magnitudes vary greatly for the different tests. The comparison of the results from the U. T. triaxial and the THD triaxial tests shows that the U. T. triaxial tests give higher values of shear strength. This comparison is consistant with the findings of O'Neill and Reese (1970) and Barker and Reese (1970).

Figure 2.6 demonstrates the difficulties involved in obtaining a shear strength profile. Determination of the strength profile is very important in the analysis and design of drilled shafts. The analysis of the shaft in Chapter V will be based on both the U. T. triaxial and the THD triaxial test results. The penetrometer results will not be used in the analysis due to the limited number of tests which were performed.

TABLE 2.2

RESULTS OF U. T. TRIAXIAL TESTS

Depth (feet)	Confining Pressure (Psi)	Maximum Deviater Stress (Psi)	Strain at Maximum Stress (in/in)	Shear Strength $\frac{\sigma_1 - \sigma_2}{2}$ (Psi)
3- 4	3.2	32.5	.046	16.4
5- 6	4.9	64.9	.020	32.5
6- 7	5.8	95.6	.028	47.8
7- 8	6.7	66.8	.020	33.4
9-10	9.0	67.3	.013	33.6
10-11	9.0	93.6	.034	46.8
11-12	10.4	70.5	.006	35.2
12-13	11.7	53.9	.020	26.9
14-15	11.7	75.1	.037	37.5
17-19	16.7	74.1	.026	37.1
19-21	18.5	53.9	.046	26.9
22-23	20.3	32.6	.057	16.3
23-24	22.1	22.4	.020	11.2
25-26	23.9	23.9	.037	11.9
27-28	23.9	26.9	.031	13.5
28-29	25.7	19.7	.020	9.8
30-32	28.0	25.4	.017	12.7
32-33	29.3	68.9	.042	34.5
35-36	32.9	34.1	.029	17.0
38-39	34.7	131.5	.014	65.7
41-43	38.8	81.7	.014	40.8
44-45	41.0	94.7	.015	47.3
46-47	41.0	56.5	.011	28.3
48-50	43.3	120.7	.018	60.4

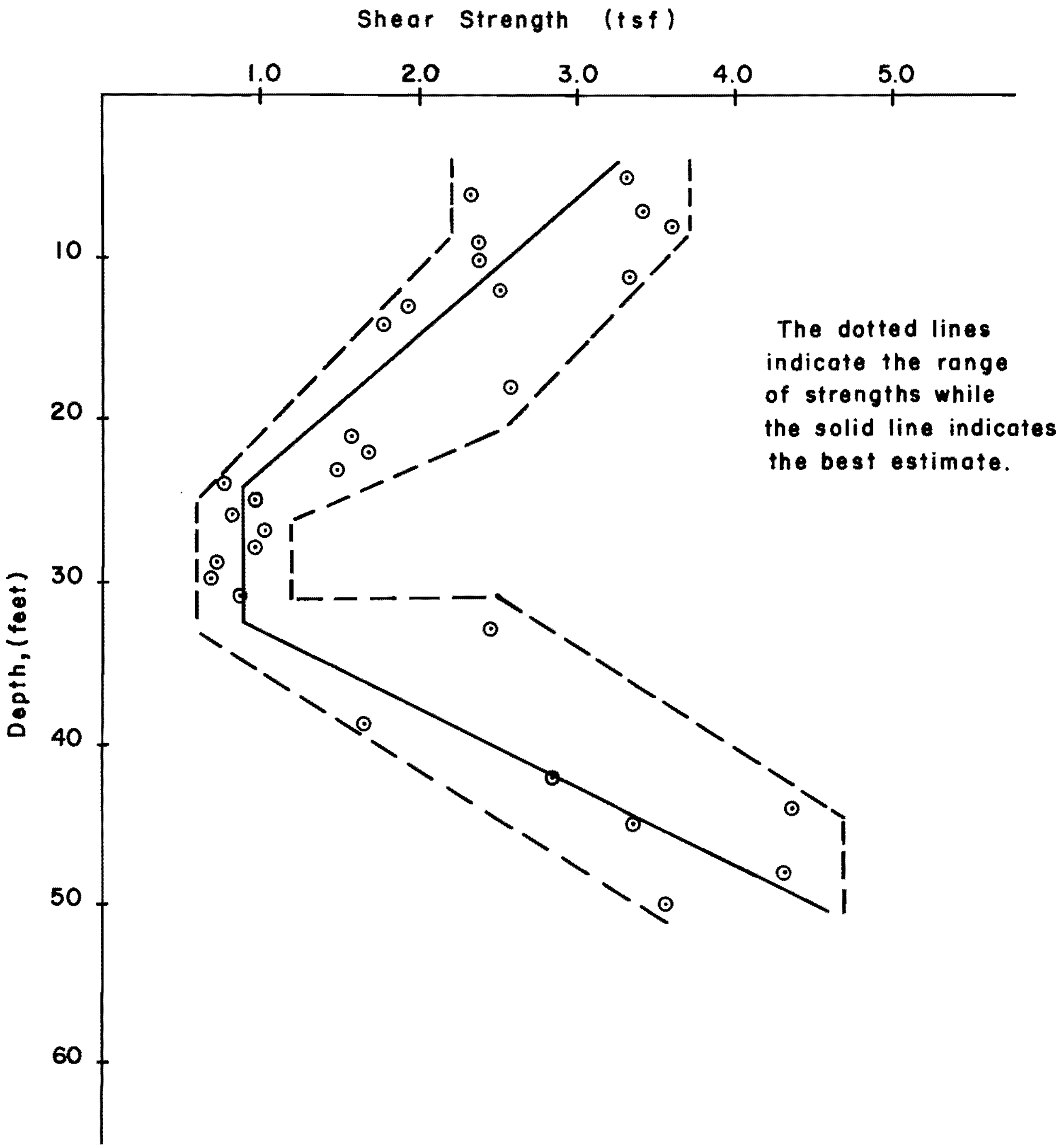


Fig. 2.5 Profile of Peak Shear Strength from U. T. Triaxial Tests - Test 1

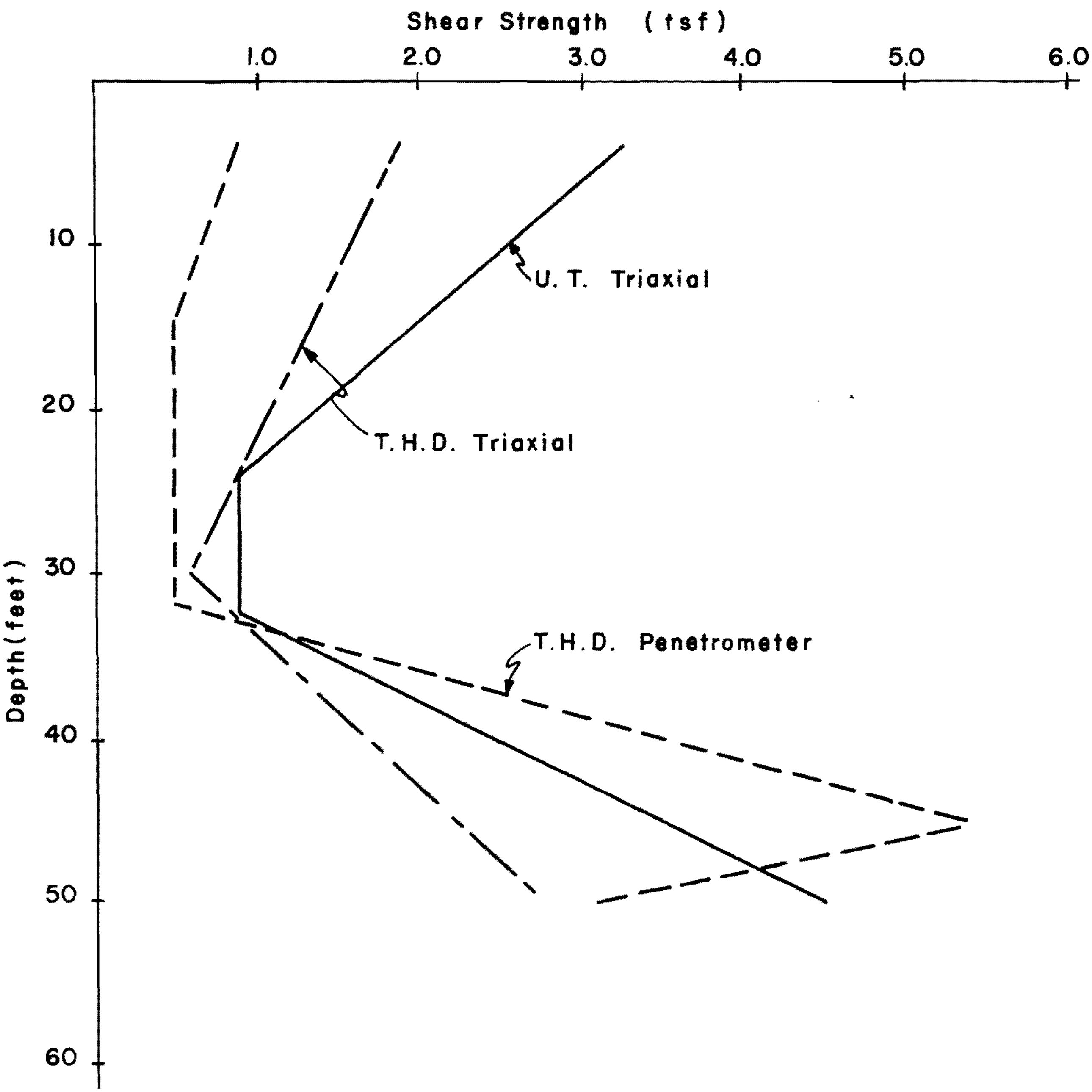


Fig. 2.6 Comparison of Shear Strength Profiles from Various Methods - Test 1

TEST 2 - SAN JUAN, PUERTO RICO

Site Location

Test 2 was located several miles from the downtown area of San Juan, Puerto Rico in the suburb of Hato Rey. This site is the location of a construction project designated as the Parque de las Fuentes Project. The load test was performed in conjunction with the design of several multistory structures at the project. The structures are to be supported by 36-inch diameter drilled shafts with column loads of up to 450 tons.

The test shaft was located adjacent to a structure designated as Tower 1. Both the test shaft and the two reaction shafts which were used to load the test shaft were to be used as supports for a wall of the structure.

Soil Profile

Information on soil properties adjacent to Test Shaft 2 was obtained from tube samples and from standard penetration tests. Samples were obtained by Puerto Rico Testing Services to a depth of 34 feet from Boring 110 which was located approximately 10 feet west of the test shaft. The samples were taken with a liner-type sampler and gave the appearance of being badly disturbed.

Puerto Rico Testing Services also performed standard penetration tests to a depth of 81 feet at the same boring. The results of the standard penetration test are plotted versus depth in Fig. 2.7. The test

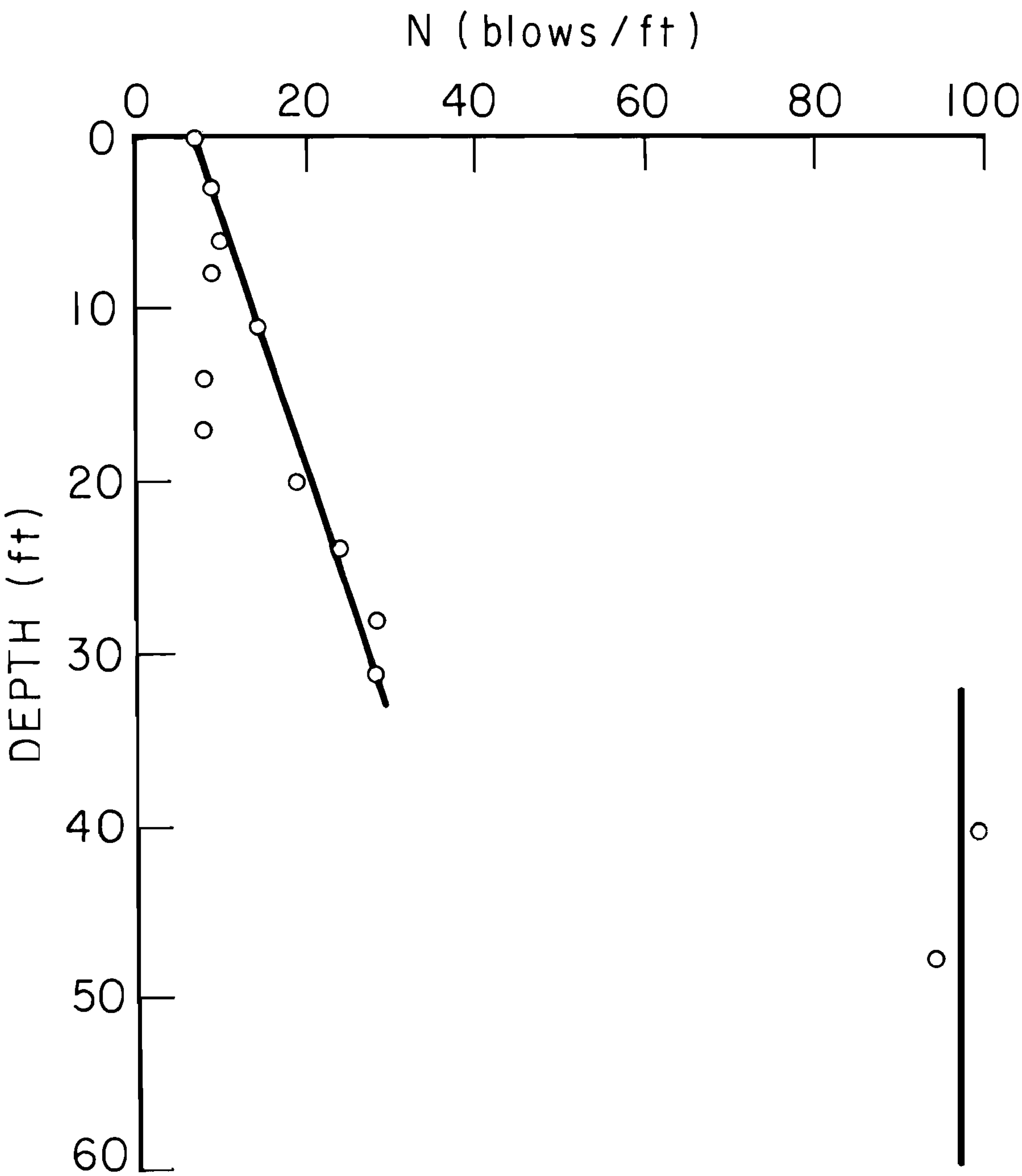


Fig. 2.7 Standard Penetration Test Results for Test 2

results show that the consistency of the soil increases greatly below 30 feet.

Personnel of Farmer Foundation Company took three-inch diameter, thin-walled tube samples from Boring E1 located 25 feet north of the test shaft. These samples were obtained at five-foot intervals from depths of 30 to 60 feet. All of the samples appeared to be essentially undisturbed up to the time they were removed from the tubes.

The soil profile shown in Fig. 2.8 was constructed from the two borings. As shown in the figure, the profile consists of a five-foot thick layer of clay underlain by alternating sand and clay layers of varying thicknesses. The top layer of clay was removed before construction of the test shaft and therefore has no bearing on the test.

Laboratory Tests

Laboratory tests were conducted at The University of Texas under the supervision of Dr. R. E. Olson. The tests which were performed included unconsolidated-undrained triaxial, unconfined compression, Torvane, and direct-shear.

Boring 110. Samples from Boring 110 were badly disturbed and were very soft. Because of the condition of these samples, no attempt was made to perform refined strength tests. A total of eleven Torvane tests and one unconfined compression test were performed on the samples. The test results are summarized in Table 2.3.

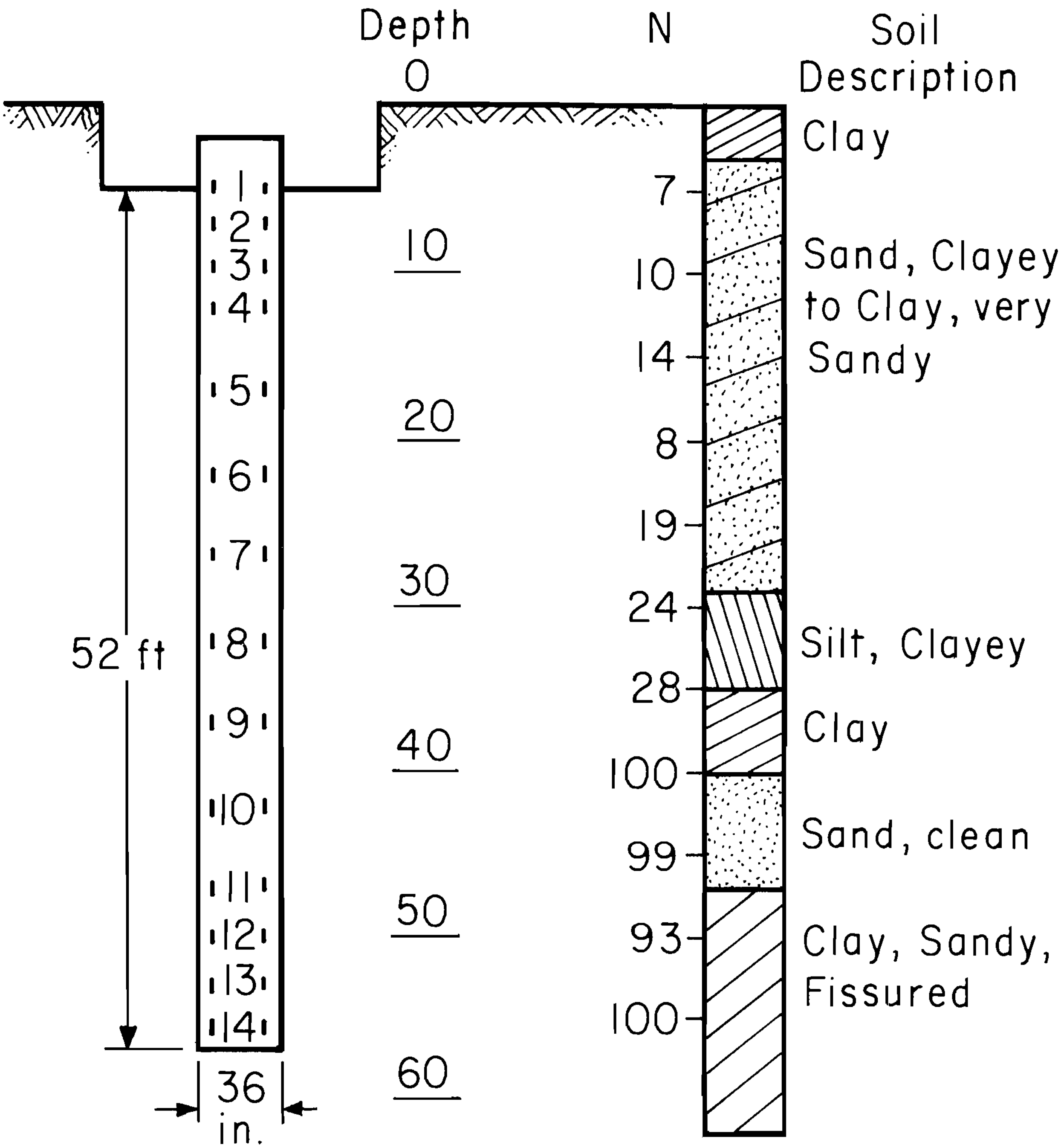


Fig. 2.8 Soil Profile and Shaft Instrumentation for Test 2

TABLE 2.3

SUMMARY OF SOIL DATA FROM BORING 110

Depth Range feet	Shearing Strength psf	Testing Method	Soil Description
1 - 3	600 800	Torvane Unconfined Compression	CLAY, sandy, tan and gray, mottled, pockets of organic matter, roots
3 - 5	700	Torvane	SAND, clayey, brown and tan mottled, some organic matter, to CLAY, very sandy, brown and tan mottled, some organic matter, Torvane on clay (w=35%)
6 - 8	600	Torvane	SAND, brown, dark brown spots, to 1/4", to SAND, clayey, brown and gray, on clayey sand L_w = 50%, I_w=16%, Torvane on clayey sand
9 - 11	100 300	Torvane Torvane	SAND, clean to clayey, gray and brown spots
16 - 18	100	Torvane	SAND, clayey, brown and tan mottled, w=34%
21 - 23	100 300	Torvane Torvane	SAND, clayey brown, to SAND, clayey, brown and gray mottled with roots, w=24%
27 - 29	400	Torvane	SAND, clean to clayey, to SAND, clayey, brown and gray mottled, roots w=23%
32 - 34	800 500	Torvane Torvane	SILT, clayey, pockets of sand to 1/4", tan and gray mottled, to SAND, clayey, brown and gray mottled with roots, w=33%

Boring E1. Samples from Boring E1 were hard, brittle, and strongly fissured. When extruded from the sample tubes, the stress relief allowed fissures to open and the samples underwent major distortions. It was found that trimming the sides of the samples was impossible because they would fall apart.

When the testing of samples from Boring E1 began, it was assumed that the shafts would be loaded under undrained conditions in the field. Consequently, unconsolidated-undrained triaxial tests were planned for all clays. Initial tests indicated that strengths might be dependent on confining pressure, so the decision was made to run stage-type triaxial compression tests when possible. In the stage-type test, the speciman is loaded just to the point of failure under the initial confining pressure. Then the confining pressure is increased and the speciman is again loaded to failure. When samples of sandy soil were encountered, it was concluded that drained strengths were needed. Since fully drained triaxial tests would be time consuming, it was decided to perform the drained tests in the direct shear apparatus. Results of laboratory tests on Boring E1 are summarized in Tables 2.4 and 2.5.

Shear Strength Profile. The strengths for samples from Borings 110 and E1 are shown in Fig. 2.9. For the sand layer from five to 30 feet, the Torvane strengths seem largely irrelevant. The strength line that is shown for the stratum from five to 30 feet was defined by assuming an effective angle of internal friction of 30 degrees and by using the effective overburden pressure. For the clay layer from 30 to 38 feet, an average strength of 2300 psf was used. For the sand layer from 38 to 48 feet,

TABLE 2.4

BORING LOG FOR BORING E1

Sample Number	Depth Range feet	Liquid Limit %	Plastic Limit %	Soil Description
1	30 - 31	--	--	CLAY, very sandy to SAND, very clayey, brown, finely fissured
	31 - 32	45	24	CLAY, silty, light greenish tan, strongly fissured, brittle, could not be trimmed
2	35 - 36	--	--	SILT, slightly cemented, mottled tan and gray, fissured, brittle
	36 - 37	43	36	CLAY, gray and tan mottled, slightly fissured, piece of dolomite 1 1/2" in diameter, angular
3	40 - 41	--	--	SAND, clean, medium, brown, fissured, with 3/4" diameter pockets of pink sand, to SAND, slightly cohesive, fine, borwn to light gray
4	45 - 46	--	--	SAND, clean, slightly cemented, brown, coarse
5	50 - 51	--	--	CLAY, silty, some fine sand, tan and gray mottled, strongly fissured, brittle
6	55 - 56	--	--	CLAY, slightly sandy, brown, ranges from slightly fissured to very strongly fissured
7	60 - 61	--	--	CLAY, slightly sand, tan and gray mottled, fissured but fissures did not open up upon extrusion as in above samples

TABLE 2.5

SUMMARY OF TEST DATA FROM BORING E1

Sample Number	Test	Drainage	Stresses, psf Normal	Stresses, psf Shear	Strain at Failure %	Water Content %
1	T	Q	5090	1330	10	20
	DS	S	4320	3500	--	31
2	T*	Q	4860	2230	7	38
			8090	2630	9	
3	T*	Q	8680	4980	2	20
			2250	1380	4	
4	T	Q	8820	4040	7	21
	DS*	S	1440	1800	--	--
			5760	5320		
			14400	10240		
5	T	Q	6980	4600	6	17
6	T*	Q	5970	4480	6	--
			6910	4810		
			7860	5140		
			8900	5600		
	DS*	S	2880	3160	--	--
			7920	5370		
7	T*	Q	7000	5600	$4\frac{1}{2}$	22
			14800	7380	5	
			26000	8600	9	

T Triaxial Compression Test
DS Direct Shear Test
* Stage Test
Q Unconsolidated-Undrained
S Drained

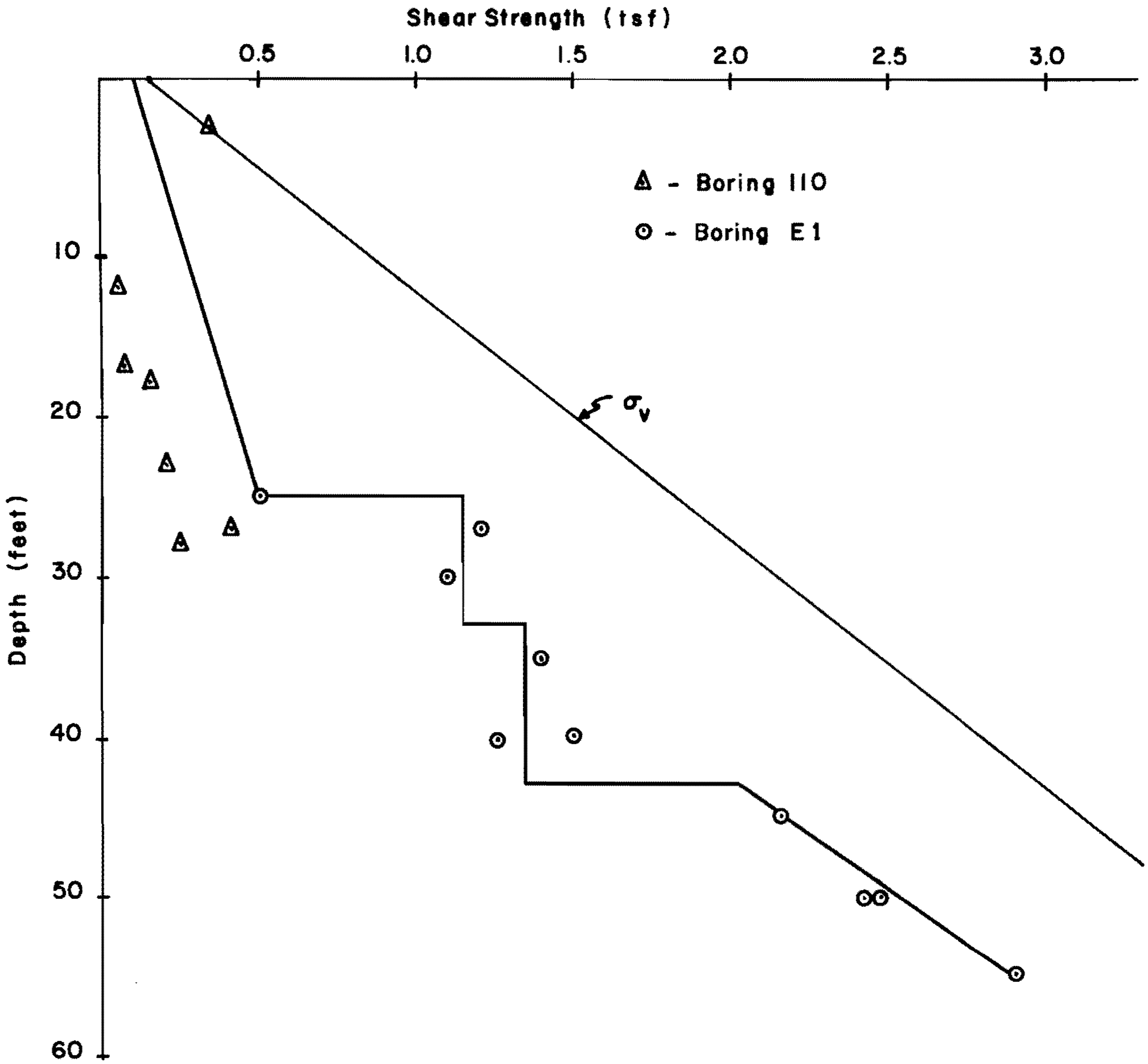


Fig. 2.9 Shear Strength Profile for Test 2

an average strength of 2700 psf was selected. For the underlying clay layer, the test results seem to define the sloping line shown in the figure.

TEST 3 - SAN JUAN, PUERTO RICO

Site Location

Test 3 was located at the same project as Test 2. The two tests were only about 300 feet apart but the soil conditions at the two sites were vastly different. Test Shaft 3 was located adjacent to a structure designated as Tower 2. Both the test shaft and the reaction shafts were to be incorporated into the structure.

Soil Profile

Information on soil properties adjacent to Test Shaft 3 was also obtained from tube samples and from standard penetration tests. Samples were obtained by Puerto Rico Testing Services to a depth of 25 feet from Boring 101. Boring 101 was located approximately 10 feet north of the test shaft. The samples were taken with a liner-type sampler and appeared to be badly disturbed.

Puerto Rico Testing Services performed standard penetration tests to a depth of 80 feet at the same boring. The blow counts are plotted versus depth in Fig. 2.10. The test results show a gradual increase in strength to a depth of 75 feet. Below 75 feet the strength increases significantly with most of the blow counts exceeding 100 blows/foot.

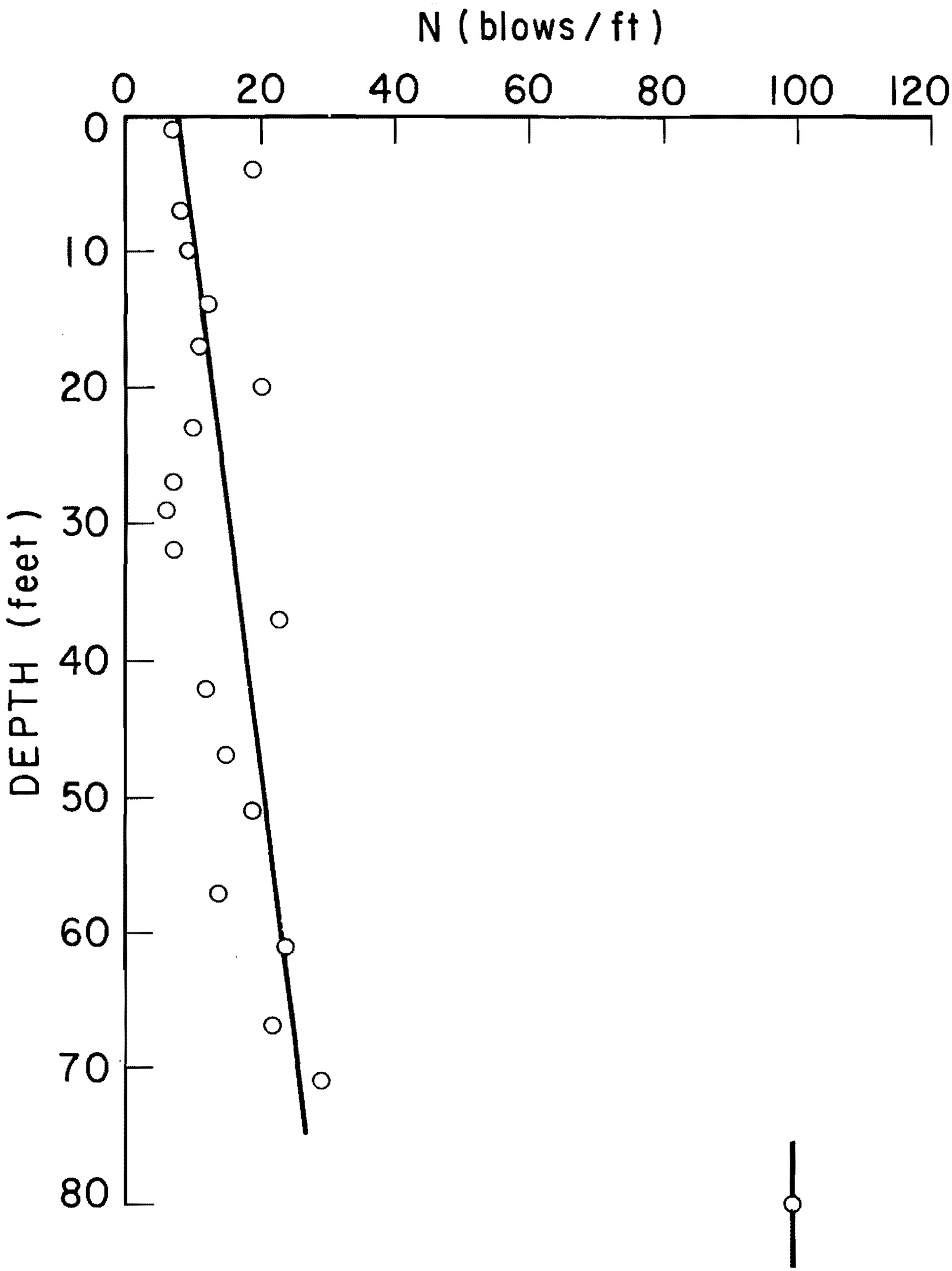


Fig. 2.10 Standard Penetration Test Results for Test 3

Personnel of Farmer Foundation Company took three-inch diameter, thin-walled tube samples from boring E2 located 15 feet north of the test shaft. These samples were taken every five feet from a depth of 30 feet to a depth of 75 feet. No samples were taken below 75 feet because the equipment available was incapable of obtaining samples below this depth. The samples which were recovered appeared to be essentially undisturbed up to the time they were removed from the tubes.

The soil profile shown in Fig. 2.11 was obtained from the two borings. The profile consists of a three-foot thick clay layer underlain by sand to a depth of 20 feet. Below 20 feet there is a 25-foot thick zone composed of silt and clay, and below this stratum there is sand to 75 feet. From field observations during drilling, it is apparent that the sand layer continues to the final shaft depth of 91 feet. The top four feet of the profile was removed prior to construction of the test shaft.

Laboratory Tests

The laboratory tests of Test 3 were similar to those conducted for Test 2 and were also conducted under the supervision of Dr. R. E. Olson.

Boring 101. Samples from Boring 101 were badly disturbed and were very soft. No refined strength tests were performed on the soil from this boring. A total of six Torvane tests and one unconfined compression test were performed on the samples. Test results are summarized in Table 2.6.

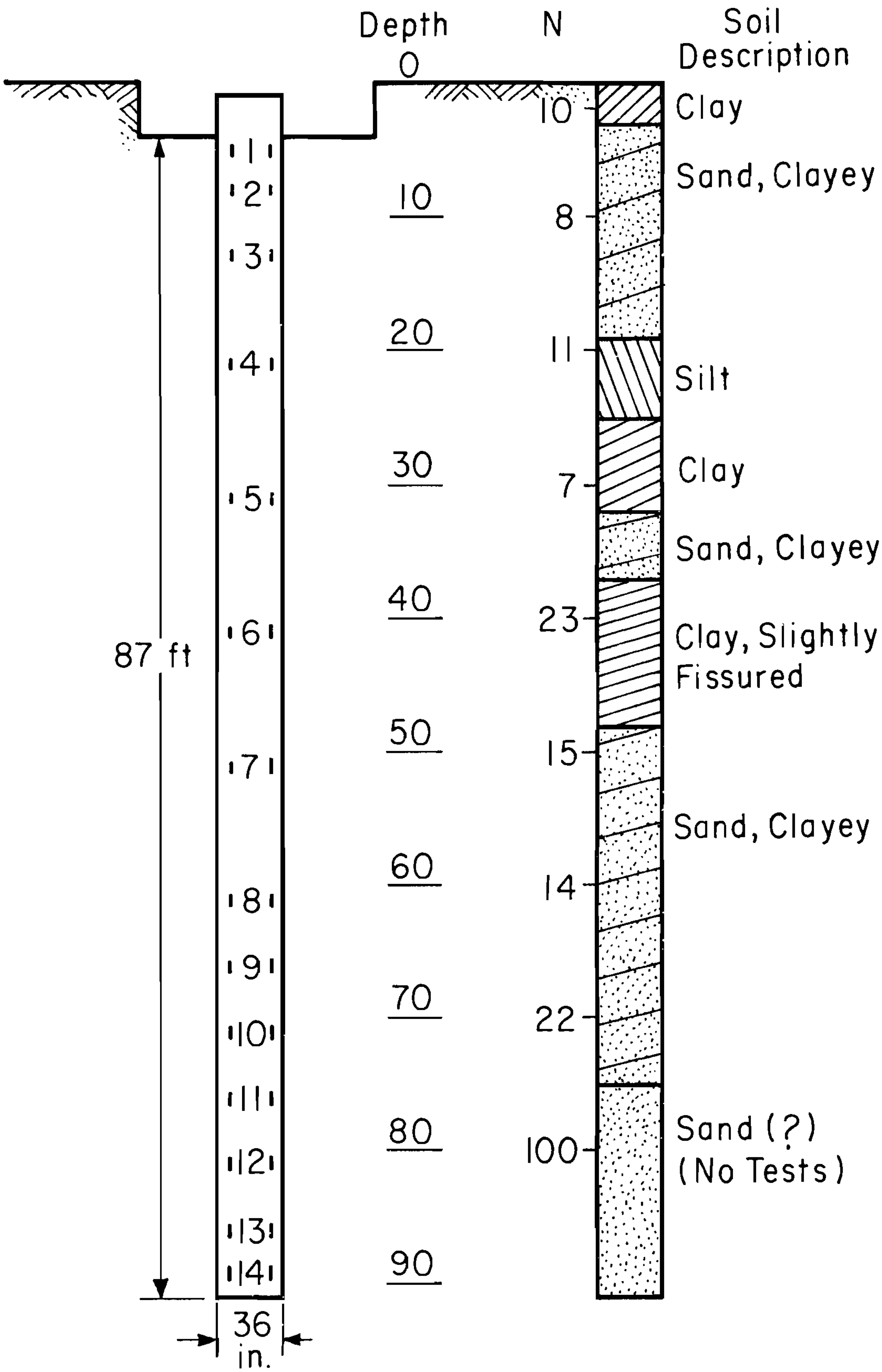


Fig. 2.11 Soil Profile and Shaft Instrumentation for Test 3

TABLE 2.6

SUMMARY OF SOIL DATA FROM BORING 101

Depth Range feet	Shearing Strength psf	Testing Method	Soil Description
1 - 3	800	Torvane	CLAY, sandy, brown, with tan and gray mottling, roots
6 - 8	500	Torvane	SAND, brown with tan and gray mottling, to SAND, clayey brown and gray, some gravel
	1020	Unconfined Compression	
11 - 13	750	Torvane	SAND, clayey, brown, tan and gray mottling, w=19%
13 - 15	600	Torvane	SAND, clayey, brown, tan and gray mottling, black spots up to 1/4" in diameter
17 - 19	100	Torvane	SAND, slightly clayey, brown to dark brown, tan and gray mottling
23 - 25	300	Torvane	SILT, slightly clayey, gray and tan mottling, black spots to 1/8" in diameter

Boring E2. Samples from Boring E2 were either intact or only slightly fissured. They were somewhat softer than those obtained from Boring E1, but still could not be trimmed except on the ends.

The triaxial tests, performed on the clay samples, failed at such high strains that stage tests could not be performed. Instead efforts were made to test two samples from each tube at different confining pressures. The direct shear tests on the clayey sand were performed by consolidating the samples fully and then shearing them to failure in about five minutes. This procedure probably allowed drained conditions to be approximated. Results of laboratory tests on Boring E2 are summarized in Tables 2.7 and 2.8.

Shear Strength Profile. A strength profile is shown for samples from Borings 101 and E2 in Fig. 2.12. Above 30 feet the strength profile is the same as that for Test 2. Below 30 feet the strengths shown were generally determined by entering the undrained-triaxial envelopes at the total overburden pressure or the direct-shear envelopes at the effective overburden pressure. The strengths of the samples at 55 feet were ignored in plotting the strength line. The last sample taken was at 75 feet because the soil became so hard that further samples could not be obtained. The measured shear strength at this depth was in excess of 9000 psf. Shear strengths below 80 feet probably exceed 10,000 psf.

TABLE 2.7

BORING LOG FOR BORING E2

Sample Number	Depth ft.	Soil Description
1	30	CLAY, mottled brown and gray, pockets of brown sand, clay very sticky
2	35	SAND, clayey, brown
3	40	CLAY, mottled brown and tan, slightly fissured
4	45	CLAY, brown, some small pockets of sand, a few fissures
5	50	SAND, clayey, brown, one seam of clay 1/4" thick
6	55	SAND, clayey, brown, some pockets of clay
7	60	SAND, clayey, brown, some pockets of clay
8	65	SAND, clayey, brown, some small pockets of clay
9	70	SAND, clayey, brown, one clay seam, some small pockets of clay
10	75	SAND, slightly clayey, cemented, dense, brown

TABLE 2.8

SUMMARY OF TEST DATA FROM BORING E2

Sample Number	Test	Drainage	Stresses, psf Normal	Stresses, psf shear	Strain at Failure,%	Water Content %
1	T*	Q	3,540	1,380	7	34
			15,960	1,560	11	
	DS*	S	1,440	1,500	-	--
			4,320	2,800		
			14,400	6,940		20
2	T	Q	6,420	1,380	20	24
3	T*	Q	7,270	1,510	14	36
			19,170	1,890	23	
	DS*	S	1,440	1,500	--	
			5,760	2,700		
			14,400	4,200		32
4	T	Q	9,840	3,360	16	31
	DS*	S	1,440	1,120		
			6,480	3,350		
			14,400	4,420		
5	T	Q	9,940	2,740	10	20
	T	Q	19,090	1,810	10	27
6	T	Q	8,740	820	20	29
	T	Q	18,180	900	16	30
7	T	Q	13,000	4,360	10	20
	T	Q	20,790	3,510	8	24
8	T	Q	12,000	2,630	10	23
	T	Q	22,560	5,280	8	25
9	T*	Q	12,620	2,540	3	28
			19,830	2,550	3	
10	T	Q	20,160	9,360	12	18

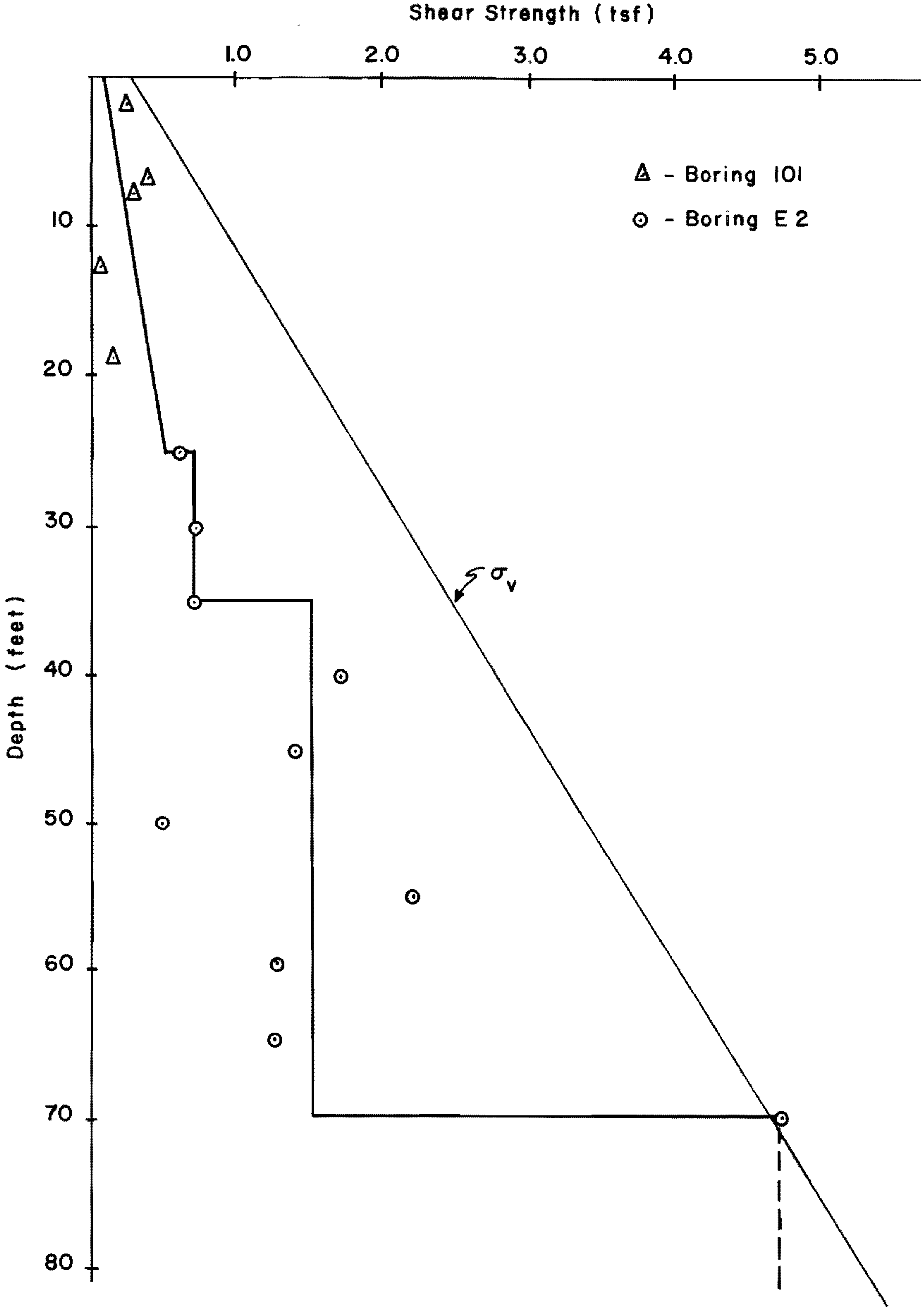


Fig. 2.12 Shear Strength Profile for Test 3

CHAPTER III

TEST SYSTEM

TEST SHAFTS

As stated earlier, tests were conducted on three instrumented drilled shafts. One of the test shaft was constructed using a variation of the casing method and the other two shafts were constructed using the slurry displacement method.

Test Shaft 1

Test Shaft 1 was a 45-foot long drilled shaft with a penetration of 42 feet beneath the ground surface. The reinforcing cage for the shaft consisted of eight, No. 9 deformed bars extending the full depth of the shaft and a spiral extending from the top of the shaft to a depth of 20 feet. The spiral was constructed of smooth No. 3 bars and had a six-inch pitch. Below 20 feet the cage was held together with circular bands constructed of flat bars 1.5 inches wide.

Instrumentation was installed on the reinforcing cage on the morning of 20 March 1973. The instruments were read to check proper operation and were then pressured with nitrogen to prevent moisture penetration.

Drilling was begun on Test Shaft 1 at 7:30 a.m. on 21 March. The hole was advanced rapidly to a depth of 33 feet where a water-bearing silt seam was encountered. Drilling continued to a depth of 38 feet at which time a 30 inch diameter casing was set to prevent caving. No drilling fluid was required, as the hole would stand open long enough to permit drilling in the dry. After installation of the casing, the hole was advanced to the desired 42-foot depth. Drilling was completed at 8:15 a.m.

The instrumented reinforcing cage was set and positioned properly in the hole and at 8:45 a.m. concreting was begun. A 10-inch diameter steel pipe was employed as a tremie for concreting. The tremie had openings cut in the sides at intervals along its length which allowed the concrete to be poured directly from the ready-mix trucks. The hole was filled to the top of the casing and the casing was removed. After the casing was completely removed a section of Sonotube was pushed into the hole a distance of approximately two feet, leaving three feet of the form above ground. The Sonotube was then filled with concrete and the top of the shaft was leveled. Concreting was completed at 9:30 a.m.

<u>Test Shaft 2</u>

Test Shaft 2 was a 55-foot long drilled shaft with a penetration of 52 feet, beneath the ground surface. The reinforcing cage for the shaft consisted of 14, No. 11 deformed bars extending the full depth of the shaft and a smooth spiral extending from the top of the shaft to a depth of 20 feet. The spiral was made from No. 3 bars. Below 20 feet the cage was held together with circular bands constructed of No. 4 deformed bars spaced approximately every five feet.

Instrumentation was installed on the reinforcing cage on the morning of 26 June 1973. Shortly after installation of the gages, a rain storm hit the site causing all work to be stopped. The job site was under several inches of water and it was not possible to continue work for some time. During this time the gages were pressurized with nitrogen and left in the weather.

Drilling was begun on Test Shaft 2 on 28 June at 7:00 a.m. At a depth of approximately 16 feet a water-bearing sand seam was encountered and the hole was filled with water to prevent caving. Drilling proceeded rapidly to a depth of 30 feet where stiffer material was encountered. The hole was completed at 11:30 a.m. and was ready for concrete at 1:00 p.m.

Two of the concrete trucks arrived at 1:45 p.m. but the third truck was late in arriving. It was decided to wait on the third truck before beginning the pour in order to assure that enough concrete would be available. The third truck finally arrived at 2:15 p.m. and pouring of the concrete was begun. The concrete was placed quite rapidly and the whole pour was completed in 45 minutes. Due to the delay in concrete arrival, the concrete had begun to set before completion of the pour. Some difficulty was encountered in pulling the tremie pipe and the concrete that reached the surface was quite hard and very hot. There was concern that the hot concrete might damage the instrumentation by melting the lead wire sheaths and thus destroying the water proofing. Later checks on the instrumentation showed that the system had not been adversely affected by the heat.

The two reaction shafts for Test 1 were poured on 29 June 1973 and 2 July 1973. No difficulty was encountered in the construction of either shaft except that the stiffness of the soil made it difficult to bell.

Test Shaft 3

Test Shaft 3 was a 90-foot long drilled shaft with a penetration of 87 feet below the ground surface. The reinforcing cage was similar to the cage used for Test Shaft 2 except for the addition of wide bands at 10 foot intervals below a depth of 30 feet. These bands were approximately three inches wide and were cut from 32 inch casing.

Instrumentation was installed on the reinforcing cage on the morning of 6 July 1973. Construction of the test shaft was not begun until 11 July. During the five day period between instrumentation and construction the gages were pressurized with nitrogen to prevent moisture from entering the gages.

Drilling of Test Shaft 3 was begun at 7:00 a.m. on 11 July. Water was added to the hole at a depth of 10 feet to prevent caving. Drilling continued with little difficulty to about 70 feet where stiffer material was encountered. The hole was completed at 1:00 p.m. and was ready for concreting soon afterward.

After completion of the hole it was discovered that no concrete was available that day. Therefore the hole was allowed to stand open until the next morning. Concrete arrived at the site at 10:00 a.m. and the pour was completed in approximately one hour. Due to the unavailability of concrete, the soil was exposed to free water for a period of approximately 24 hours.

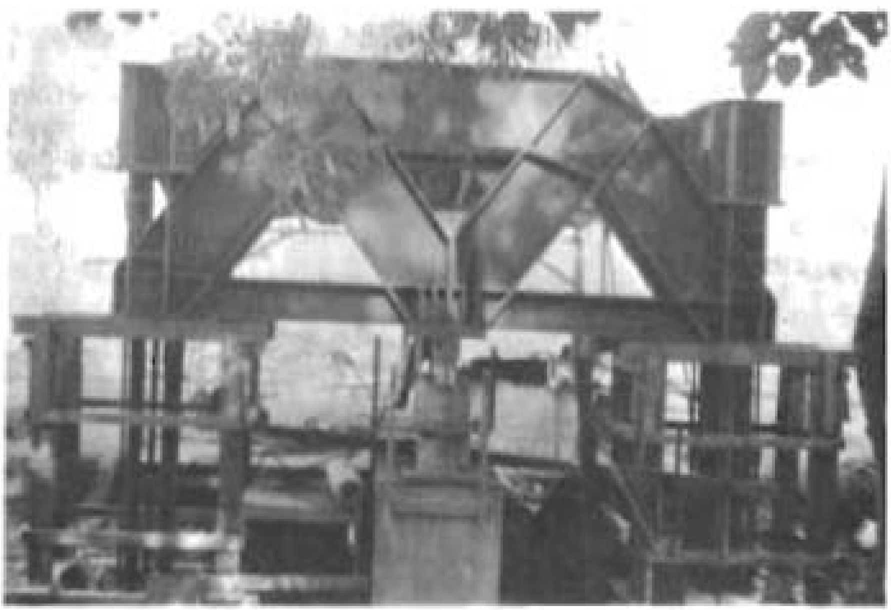
REACTION SYSTEM

The reaction system for Test 1 was the same as that described in detail by O'Neill and Reese (1970). A newly built reaction beam was used which allowed easier erection and also the loading box design was improved. The reaction system used for Tests 2 and 3 was similar in design except that a truss was used for the reaction beam. The truss was designed to withstand a load of 1000 tons as was the built-up wide flange section used in Test 1. The two reaction beams are shown in Fig. 3.1. The reaction shafts were belled in all cases to provide additional uplift capacity. The spacing of the anchor shafts for Test 1 was eight times the diameter of the test shaft while for Tests 2 and 3 the spacing was five times the test shaft diameter. It would have been desirable to have a wider spacing for the later tests, but it is believed that the anchor shafts had a minimal influence on the behavior of the test shafts.

INSTRUMENTATION

The settlement of Test Shaft 1 was measured with dial indicators mounted on diametrically opposite positions on the top of the shaft. Each dial indicator had a two-inch travel and a resolution of 0.001 inches. They were mounted on reference beams which were 20 feet in length and were supported at the ends by stakes driven in the ground. At Tests 2 and 3

a. Reaction Beam - Test 1



b. Reaction Beam - Tests 2 and 3

Fig 3.1. Reaction System

only one dial indicator was employed; it was located in the center of the shaft. As before, the dial indicator had a two-inch travel and a resolution of 0.001 inches and was mounted on a twenty foot reference beam. In addition, at Tests 2 and 3, surveying instruments were employed as a secondary device for measuring the settlement of the shafts during testing.

The measurement of load as a function of depth along the drilled shafts was accomplished by use of Mustran cells. These devices are described elsewhere (Barker and Reese, 1969) and have been used in previous investigations for studying the behavior of drilled shafts under axial loading (O'Neill and Reese, 1970; Barker and Reese, 1970; and Touma and Reese, 1972). The cell consists of a short bar, one-half inch by one-half inch in cross section, on which electrical resistance strain gages are fixed. End caps are mounted on each end of the bar and a protective covering is fitted to the end caps. Lead wires are brought to the surface for reading the electrical resistance strain gages. Water proofing is accomplished by pressurizing the sheath on the lead wires with nitrogen gas.

READOUT SYSTEMS

Two systems were used for reading the Mustran cells. A Budd Model P-350 Portable Strain Indicator was used for Tests 2 and 3, and a Honeywell Model 620 Data Logging System was used for Test 1.

The Honeywell System employed in Test 1 was the same system described by Barker and Reese (1969). This system makes use of a 40-channel stepping switch scanner with a scanning rate of approximately one reading per second. In addition, the system has an automatic print out which allows readings to be taken very rapidly. The system worked satisfactorily for Test 1.

For Tests 2 and 3 which employed the Budd strain indicator, three switch and balance units were utilized to provide a means of initially balancing the gages to zero and of switching rapidly from one gage to another during testing. A complete set of reading using this system could be taken in approximately ten minutes.

LOADING SYSTEM

The loading system shown schematically in Fig. 3.2 is the same as that used in previous tests conducted by The University of Texas personnel. The load was applied with hydraulic rams which were jacked against the reaction system. At Test 1, two 400-ton rams were used, and at Tests 2 and 3, two 600-ton rams were used. Special precautions were taken during installation of the rams to minimize any eccentricity in the applied load. The top of the shafts were carefully leveled with a quick setting capping compound before placing a one-inch thick steel plate and the rams. In addition the rams were equipped with swivel heads to reduce the effects of eccentricity.

Hydraulic pressure was applied to the rams by a variety of types of pumps. For Test 1 an SC Hydraulic Engineering Corporation Model 10-600 air driven pump was used. An air pressure of 90 psi enabled the pump to pressurize the hydraulic fluid to 20,000 psi. The pump permitted load increments to be applied in a few seconds and precise regulation of the load

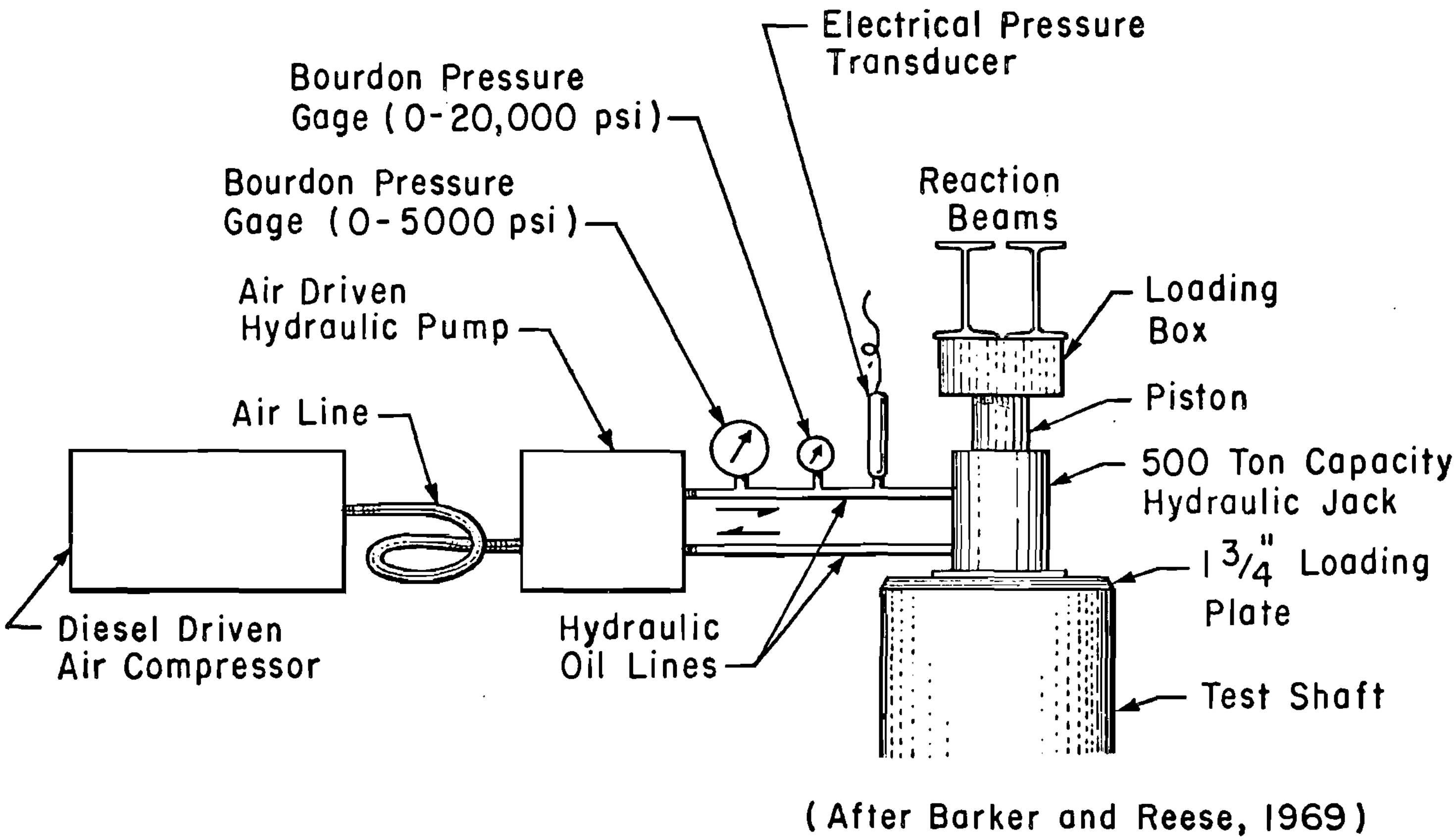


Fig. 3.2 Schematic Diagram of Loading System

was possible. An electric pump was used for Tests 2 and 3, with which pressures up to 8,000 psi could be generated, but beyond which the pump would not function properly. At that magnitude of pressure the lines were switched to a hand pump and the loading was continued. The electric pump allowed rapid application of the load increments but precise control was difficult.

The magnitude of the load applied to the top of the shafts was measured by metering the pressure in the hydraulic lines. The applied load was computed by making use of a curve, supplied by the manufacturer of the rams, in which load is shown as a function of hydraulic pressure. Pressure metering for Test 1 was achieved with a BLH GP 20,000 psi capacity electrical pressure transducer which was read with the Honeywell Logging System. Values of applied load could be resolved to the nearest fifth of a ton with this system. Pressure metering for Tests 2 and 3 was achieved by the use of Bourdon gages. The resolution of these gages is of the order of two tons.

JACK PRESSURE ERRORS

The accuracy of measuring the applied load by measurement of hydraulic pressure has been questioned by some investigators. It has been reported that friction in the piston may cause considerable error especially under eccentric loading. Due to the care taken in leveling the loading plates and the rams, no significant eccentric loads are thought to have occurred.

To study the effects of eccentricity and to determine the repeatability of the load-pressure curve, a series of twenty-four laboratory tests were performed. The testing procedure consisted of jacking one of the 400-ton hydraulic rams against a testing machine. The testing machine yielded a direct readout of the applied load and a pressure transducer was used to measure the hydraulic pressure. In this manner a series of load-pressure curves were obtained.

Eccentric loadings were obtained by placing a steel plate between the loading machine and the ram. The plate used was three inches wide and six inches long and by moving the plate with respect to the ram it was possible to obtain the desired eccentricities. The first series of loadings were concentric ($e = 0$) and the last series had an eccentricity of one inch. Results of the eccentric load tests are plotted in Fig. 3.3. Only the data from the concentric load and the eccentric load of one inch is plotted. All of the data from all the tests plotted on essentially the same line. Regrettably, the capacity of the testing machine was limited to 400 kips and therefore loading was stopped at that point. At the maximum load the calibration factor, k, in kips per division ranged between 0.165 and 0.167 for the six series of loadings with eccentricity between 0.0 and 1.0 inches. This range of values of k would lead to an error of less than five kips for a 400 kip load. Thus the maximum error was just over one per cent.

The unloading curves exhibited a small hysteresis which is assumed to be due entirely to piston friction. In all cases the unloading curve fell below the loading curve so that the calculated load based on a

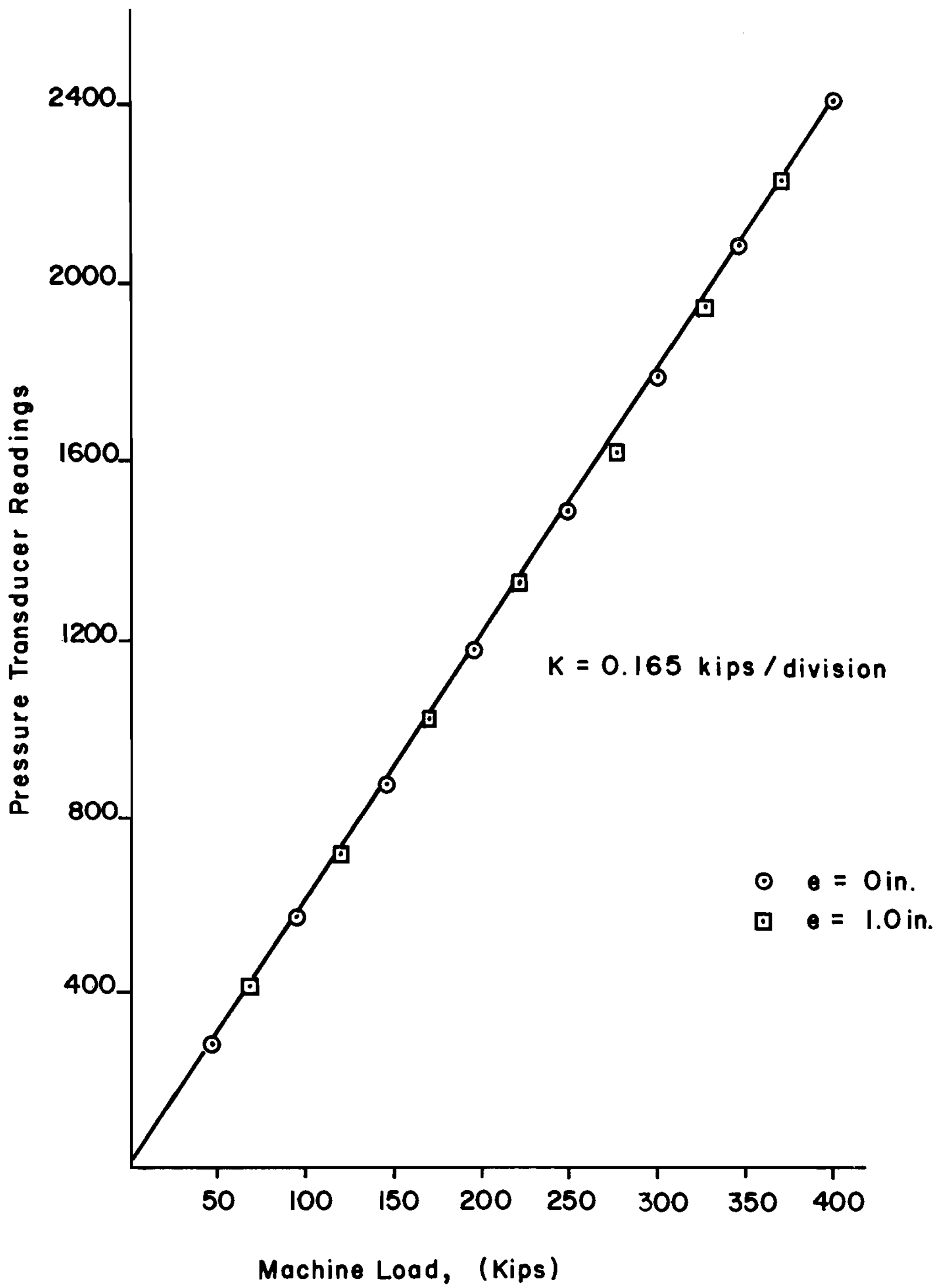


Fig. 3.3 Pressure Vs. Load for a Hydraulic Ram

loading curve will underestimate the actual load. For the series of decreasing load tests, k ranged between 0.169 and 0.171 kips per division. In general, maximum errors for individual tests were below two per cent with the largest error being three per cent.

For the types of tests being conducted, the errors introduced by measurement of hydraulic pressures are tolerably small. Based on the results of the laboratory tests it is felt that additional instrumentation to measure the applied load is not essential.

CHAPTER IV

LOAD TESTS

TEST PROCEDURES

Two methods were used for applying the load to the top of the test shafts. The first method used at Test 1 was the Texas Highway Department "quick load" procedure. The "quick load" test is performed by adding prescribed increments of load in prescribed increments of time. At Test 1 the load increment was chosen as 50 tons up to a load of 350 tons at which time the increment was lowered to 25 tons. The load increments were applied at 2 1/2-minute intervals and two sets of readings were taken for each incremental application of load.

The loading procedure used for Tests 2 and 3 was similar to the "quick load" procedure. During the first portion of the test, the loads were applied in 50 ton increments with the load being maintained for a long enough time to allow the instrumentation to be read. Usually the loads were maintained for a period of about ten minutes. In addition, after attaining a load which was two or more times the design load for the shaft, the load was maintained for a period of twenty-four hours. Maintenance of the load of large magnitude for twenty-four hours was done to comply with building-code specifications.

Experiments performed using the "quick load" procedure show that the results in most instances agree closely with those from other testing techniques (Fuller and Hoy, 1970). The magnitude of the failure load obtained from the "quick load" procedure agrees closely with the more universally accepted "maintained load" testing procedure (ASTM, 1970). Also, settlement values from these different techniques correspond closely up to about one-third the ultimate load for straight shafts.

TEST 1

The load test of Test Shaft 1 was performed on 29 March 1973, eight days after construction of the shaft. On the morning of the test a series of zero readings were taken to check the instrumentation and the readout system. A total of 15 sets of readings were taken between 9:30 a.m. and 10:45 a.m. The readout system was operating properly and the readings indicated that all the gages were stable.

The load test was begun at 10:45 a.m. A seating load of approximately five tons was applied to the top of the shaft and the instrumentation was read. The test was continued in 50-ton increments up to 350 tons and then in 25-ton increments to the plunging load of 425 tons. Loading was continued until a settlement of about 1.5 inches was obtained.

The time required to load the shaft was approximately one hour. After attaining the ultimate capacity, the shaft was unloaded in 100-ton increments. Unloading began at 11:45 a.m. and took approximately 15 minutes. After complete removal of the load, a series of zero reading were taken as a further check on the gages.

The load-settlement curve for the top of the shaft is shown in Fig. 4.1. As indicated in the figure, the maximum settlement was 1.4

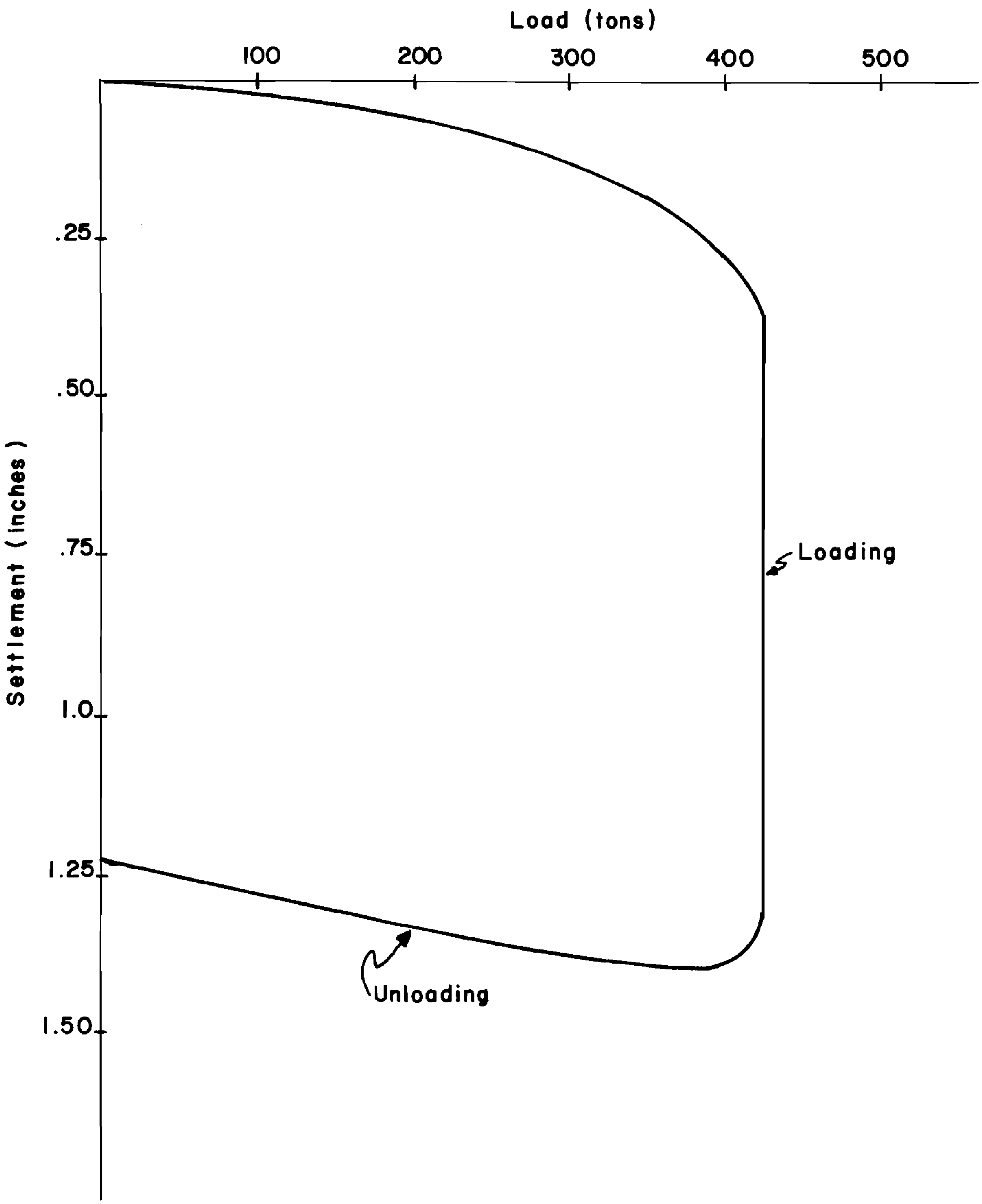


Fig. 4.1 Load Settlement Curve - Test 1

inches. Upon unloading, the shaft rebounded and the permanent settlement was of the order of 1.2 inches.

TEST 1 - RELOAD

Approximately one month after completion of the first loading, the test shaft was reloaded. The purpose of the retest was to verify the results of the first test and also to study the effects of reloading on the load-distribution and load-settlement curves.

The load test was performed on 27 April 1973. A series of zero readings were taken before the test was begun to check gage stability. Loading was begun at 12:30 p.m. and the shaft was loaded in 25-ton increments up to 350 tons at which time the increment was decreased to 10 tons. At a load of 370 tons the test was stopped due to the breakdown of the air compressor being used to operate the hydraulic pump. After a fifteen minute delay the loading was continued to the maximum load of 420 tons.

The time required to load the shaft was approximately one hour and fifteen minutes. After reaching the maximum load, the shaft was unloaded in 100-ton increments. Unloading took approximately fifteen minutes. After complete removal of the load, a series of zero readings were taken.

The load-settlement curve for the top of the shaft is shown in Fig. 4.2. As indicated by the figure, the maximum settlement was 1.7 inches and the permanent settlement was 1.5 inches.

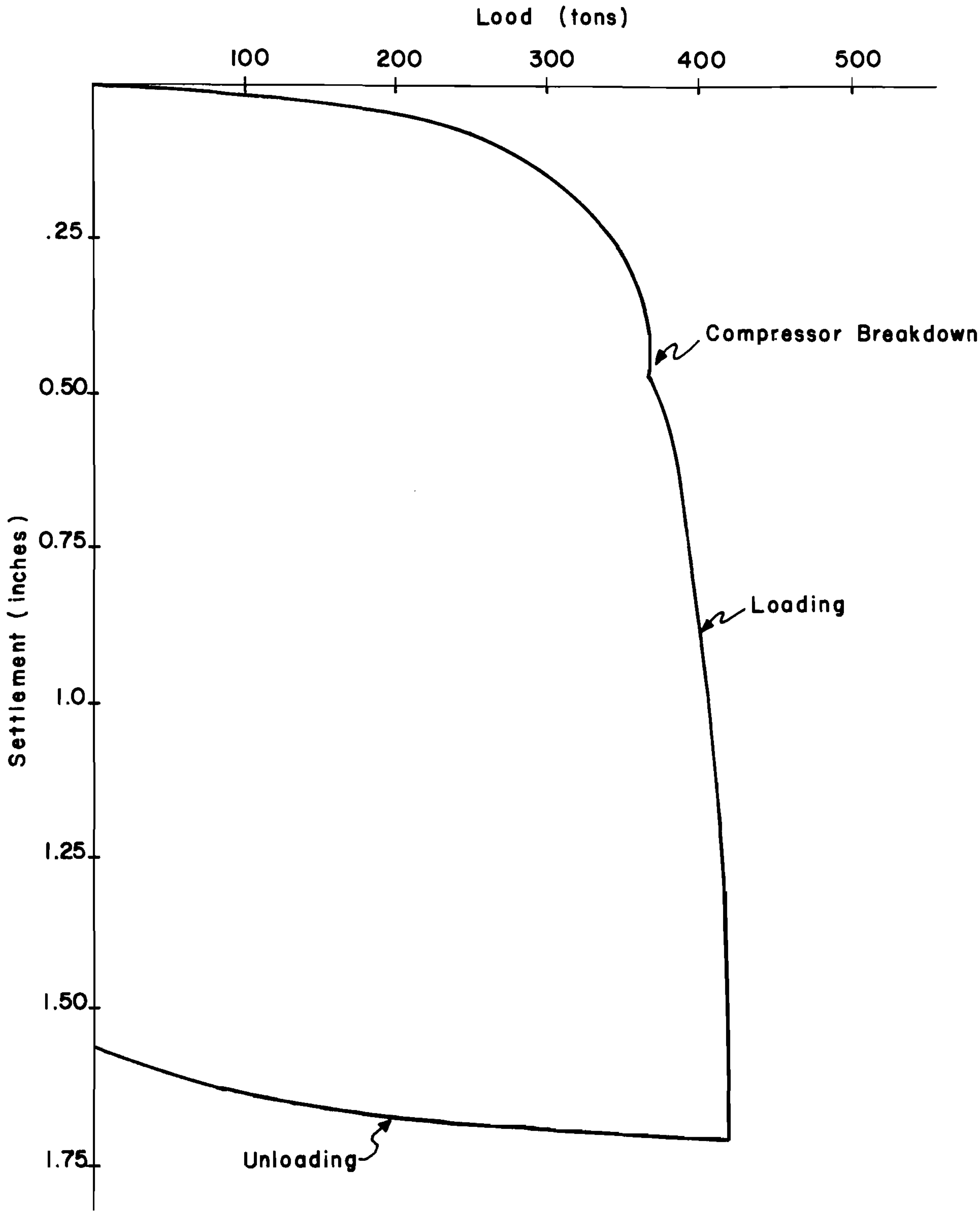


Fig. 4.2 Load Settlement Curve - Test 1, Reload

TEST 2

The load test of Test Shaft 2 was performed on 23 July 1973 approximately three weeks after installation of the shaft. On the preceding day the instrumentation was checked for proper operation and, in addition, a series of zero readings were taken the morning of the test to check gage stability. The system was found to be in good condition and only two of the 36 Mustran cells appeared to be damaged.

The load test was begun at 9:30 a.m. It had been previously decided to load the shaft in 50-ton increments and to hold each increment long enough for the gages to be read. The loading proceeded in this fashion until a load of 250 tons was reached at which time it was noticed that one of the load beams had moved and was not properly in line. It was feared that if loading continued the beam might be damaged so it was decided to unload the shaft and correct the problem.

At 12:00, after reallignment of the load beams, reloading was begun. Reloading proceeded in the same manner as before and the load was carried up to 750 tons. At this load the electric pump would not maintain the required pressure and it was necessary to switch to a hand pump. With the hand pump the load was carried to 900 tons at which time the seals in the pump began to leak. It was decided that no additional load should be added at this point because of the possibility of a complete failure of the pumping system.

The time required to load the shaft was approximately three hours. After reaching the maximum load of 900 tons, the instrumentation was read at ten-minute intervals for a period of one hour. For the remainder of the twenty-four-hour loading period the instruments were read every hour.

The shaft was unloaded at 3:00 p.m. on 24 July 1973. The load was removed in 100-ton increments and the instrumentation was read after each increment was removed. Unloading took approximately one hour and a series of zero readings were taken after complete removal of the load.

The load-settlement curve for the top of the shaft is shown in Fig. 4.3. As indicated in the figure, the maximum settlement at the end of the twenty-four-hour sustained loading was about 0.4 inches. Upon unloading, the shaft rebounded and the permanent set was of the order of 0.1 inches.

TEST 3

The load test of Test Shaft 3 was conducted on 13 August 1973, approximately five weeks after installation of the shaft. On the preceeding day the readout system was connected, and the gages were checked for proper operation. A series of zero readings were taken to check gage stability and sensitivity tests were performed to check gage accuracy. A total of nine of the gages were found to be unusable either due to drift or low sensitivity. Most of the damaged gages were near the bottom of the shaft and only two of the four gages at the lowest level were functioning properly. Despite the large number of unusable gages, it is believed that the remaining gages provided sufficient information to allow analysis of the test shaft behavior.

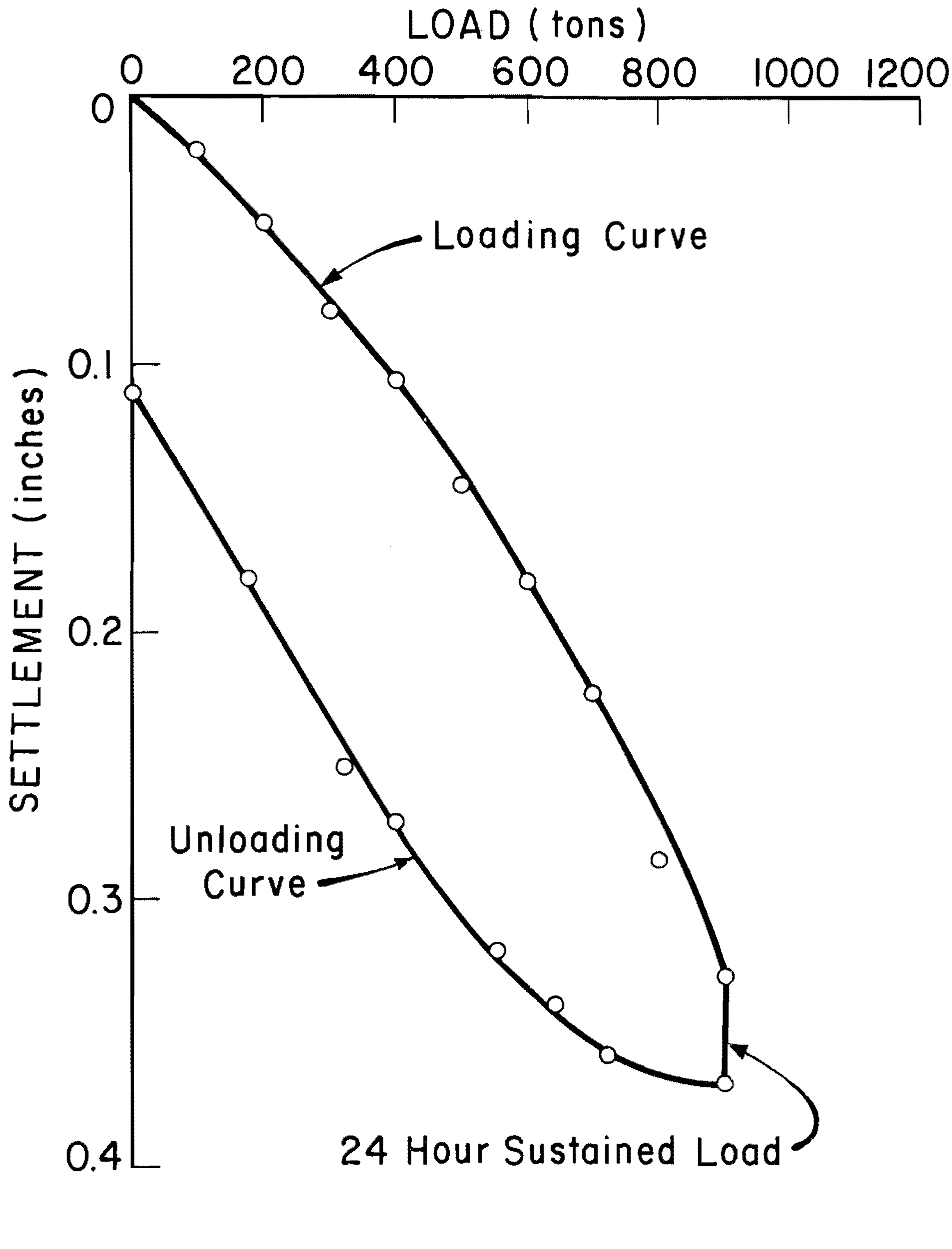


Fig 4.3 Load Settlement Curve for Top of Test Shaft 2

The load test was begun at 11:30 a.m. As in the previous test, the shaft was loaded in 50-ton increments with each load being held long enough to read the instrumentation. Loading preceeded in this fashion up to a load of 700 tons at which time the electric pump would no longer maintain the required pressure and it was necessary to switch to a hand pump. With the hand pump the load was carried to 825 tons at which time the piston on the pump broke. The test was stopped at this magnitude of load due to the failure of the pump.

The time required to load the shaft was approximately two hours. After reaching the maximum load of 825 tons, the instrumentation was read every 10 minutes for a period of one hour. For the remainder of the twenty-four-hour loading period the instruments were read every hour.

The shaft was unloaded at 1:30 p.m. on 14 August 1973. The load was removed in 100-ton increments with the gages being read after each increment was removed. Unloading took approximately 45 minutes and a series of zero readings were taken after complete removal of the load.

The load-settlement curve for the top of the shaft is shown in Fig. 4.4. As indicated by the figure, the maximum settlement at the end of the twenty-four-hour sustained loading was 0.77 inches. Upon loading, the shaft rebounded and the permanent settlement was 0.37 inches.

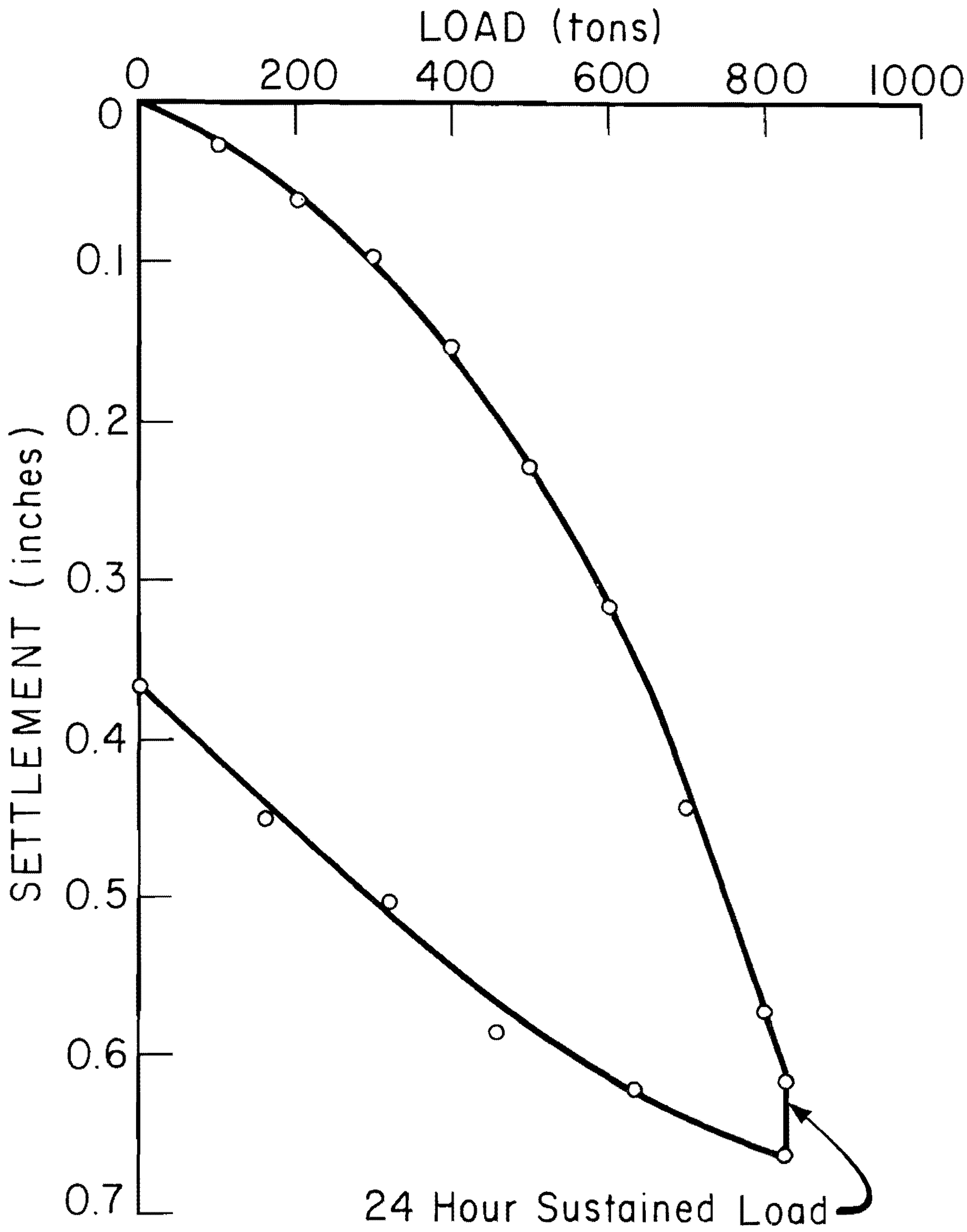


Fig. 4.4 Load Settlement Curve for Top of Test Shaft 3

CHAPTER V

ANALYSIS OF DATA

GAGE BEHAVIOR

The first step in the reduction of the data was to determine if defective gages were present. This was accomplished by obtaining plots of the readings from the Mustran cells versus the hydraulic pressure in the loading system. The slopes of these gage-response curves are functions of the shaft properties and load transfer characteristics of the shaft. It is generally possible to detect defective gages by a careful study of the gage-response curves. Observations concerning these curves are discussed in detail by Touma and Reese (1972) and are briefly summarized below:

1. A well behaved gage should either return to the origin or reflect a locked-in compressive stress upon unloading. Gages that indicate locked-in tension after unloading are discarded in the analysis.
2. The slope of the gage-response curve should increase with the applied load until it becomes constant after the soil above the gage has failed. A decrease in slope indicates that the cell must be moving into a void.
3. Gages indicating a significantly higher response than the rest of the gages are discarded as they generally reflect a reduced section or poorly compacted concrete.

In addition to detecting faulty gages, the gage-response curves can also give an indication of diameter changes in the shaft. Such changes can exist where caving occurs in sand or silt seams. In the analysis, the data were corrected to take care of changes in the cross section of the shaft when these changes were known. For Test 1 the diameter of the shaft was enlarged slightly at the top and was somewhat smaller below the tip of the casing. Visual inspection of the borehole indicated that the diameter was essentially constant along the remainder of the shaft.

Due to the use of drilling fluid in Tests 2 and 3, no visual inspection was possible. The gage-response curves indicated that both shafts were enlarged in the top 10 feet and this was taken into account in the analysis.

REDUCTION OF LOAD TEST DATA

After elimination of defective gages as discussed above, the data from reliable gages were reduced using the computer program DARES prepared by Barker and Reese (1970). This program uses the top level of gages as a calibration level and produces a plot of gage output versus applied load. This calibration curve is used to determine the load in the shaft at all gage levels along the shaft. The loads thus obtained are plotted versus depth and a best-fit polynomial is fitted to the data. This curve is known as the load distribution curve and is the basis for computing the data for load transfer curves. The slope of the load distribution polynomial between two points represents the load being transferred from the shaft to the soil at that depth. Thus, a curve showing load transfer versus depth can be developed by differentiation of the load distribution polynomial.

In addition to computing the above quantities, the program also computes shaft movements at selected increments along the shaft. The field load-settlement curve is employed, along with the load distribution curves, in computing the shaft movements at points along the shaft. The shaft movement at any elevation is determined by subtracting the elastic compression of the shaft, from the top to the point in question, from the measured shaft settlement. The elastic compression is computed by integrating the load distribution polynomial and using the elastic properties of the concrete. In this manner a load-settlement curve for the tip of the shaft can be established.

Program DARES also has the capability of considering variations in gage sensitivity and variations in the shaft diameter. The program has been supplemented with routines to plot the load-settlement, load distribution and load transfer curves.

RESULTS OF LOAD TESTS

General

The results of the four load tests are presented in the following pages. The results are presented in graphical form with only a brief discussion of the plots. A more complete discussion of the results will be presented in Chapter VI.

The results are presented separately for each shaft in the following order:

1. Load distribution curves. These curves present a plot of the load versus depth. It was found in all tests that a fourth-order polynomial provided the most reasonable representation of the load distribution. Some visual adjustment of the curves was required at the top and bottom of the shafts for Tests 2 and 3.
2. Load transfer curves. These curves were obtained at selected depth along the shaft by plotting the shear stress developed at a point versus the displacement of that point with respect to its original position.
3. Load transfer versus depth curves. These curves present the load being transferred at various depths along the shaft for selected values of applied load. On the same plot, the shear strength profile is plotted for comparison.

<u>Test 1</u>

The load distribution curves are plotted in Fig. 5.1. The curves indicate that little load is being transferred to the soil between 25 and 35 feet. Above and below this zone the load transfer is somewhat larger. At the maximum 425 ton load, approximately 170 tons is being carried by the tip. The two curves for the applied load of 425 tons indicate the load distribution before and after the shaft began to plunge.

Plots of load transfer versus shaft movement and load transfer versus depth are shown in Figs. 5.2 and 5.3 respectively. The load-

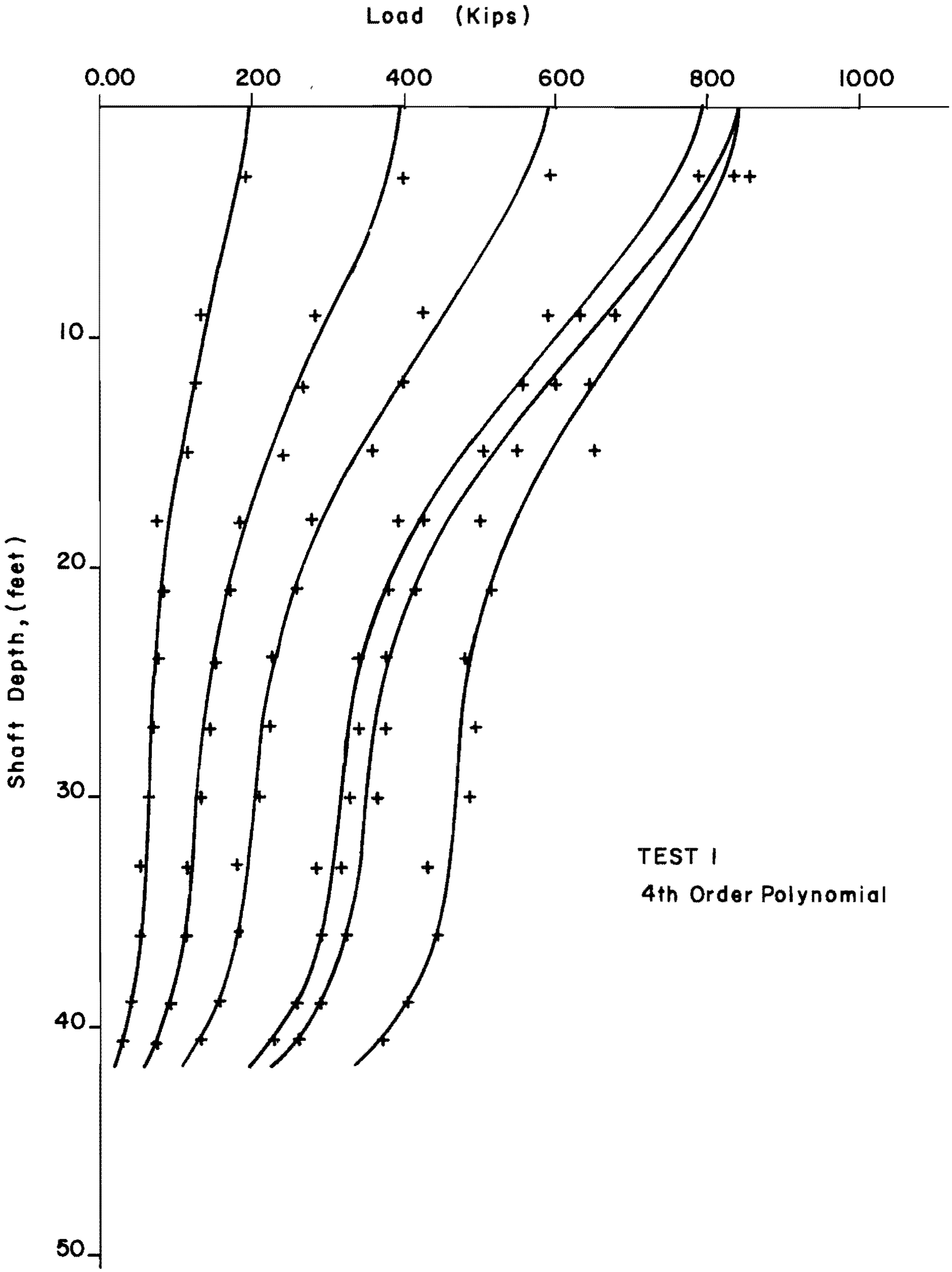


Fig. 5.1 Load Distribution Curves - Test 1

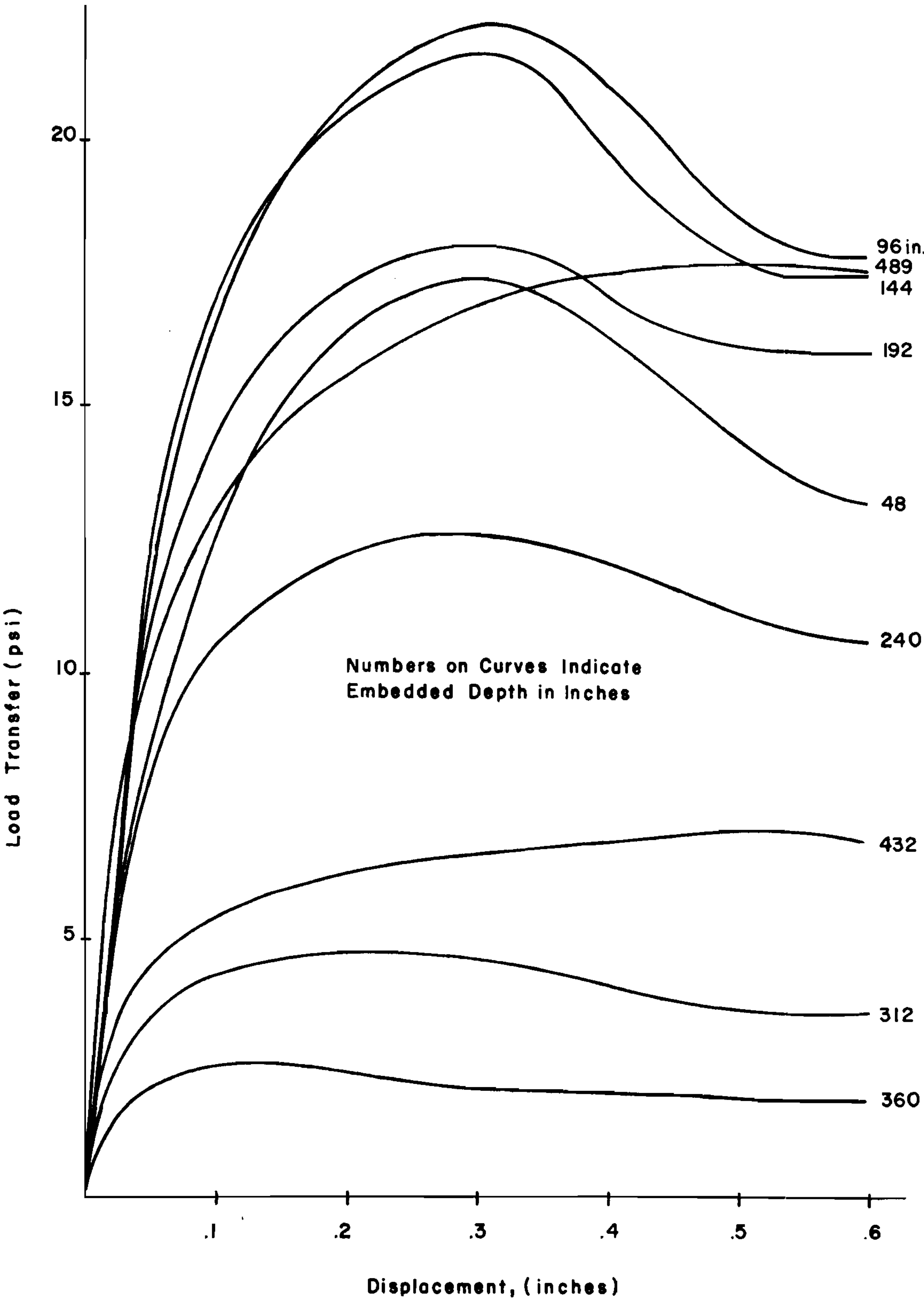


Fig. 5.2 Load Transfer Curves - Test 1

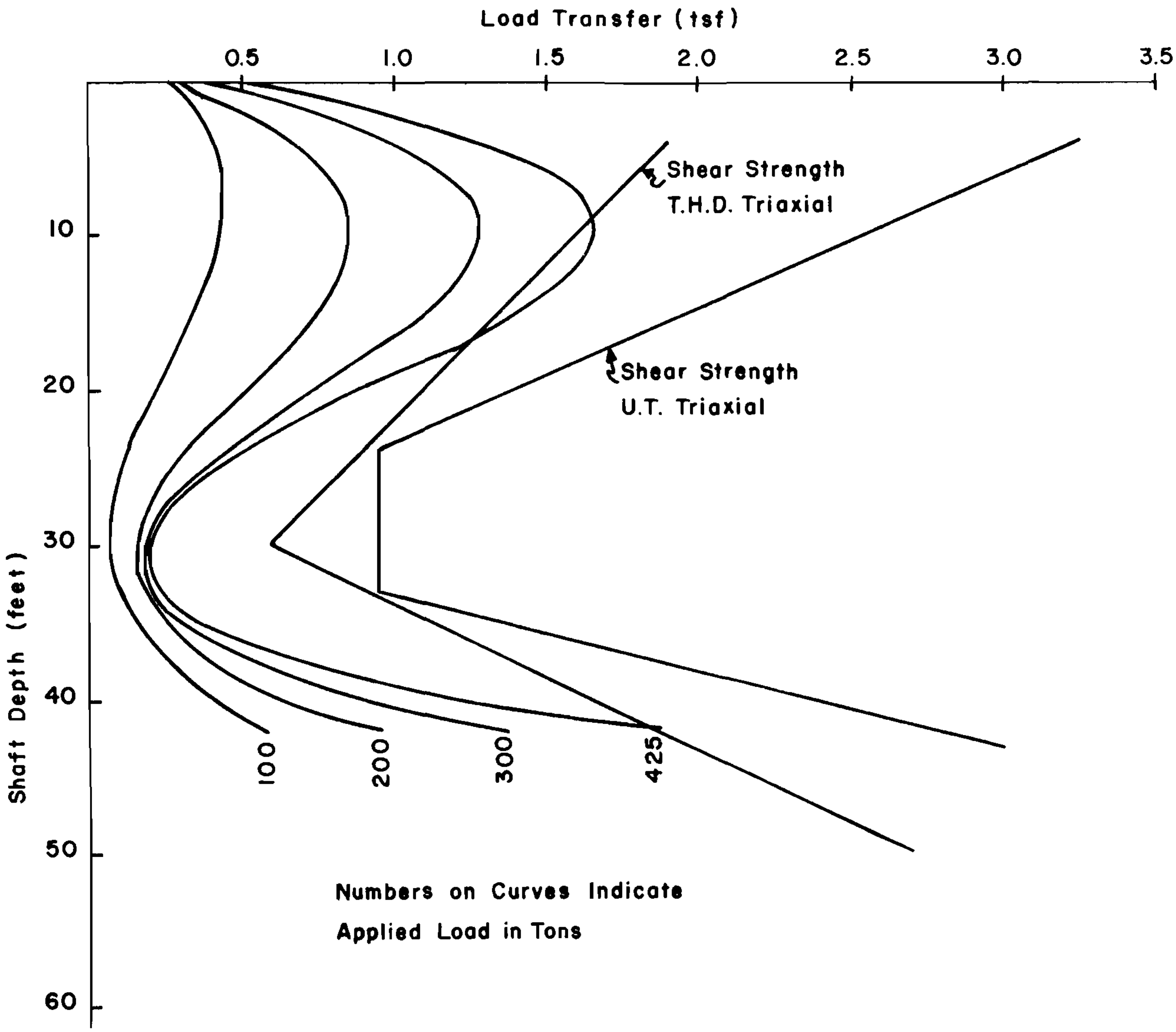


Fig. 5.3 Load Transfer Vs. Depth - Test 1

transfer versus movement plots indicate that the maximum load transfer has been developed at all depths.

Test 1 - Reload

The load distribution curves are plotted in Fig. 5.4. The curves are similar to those for Test 1 except that less load is being transferred in the top 20 feet and more load is being transferred between 25 and 35 feet. The maximum tip load was 200 tons when the shaft began to plunge.

Plots of load transfer versus shaft movement and versus depth are shown in Figs. 5.5 and 5.6.

Test 2

The load distribution curves are shown in Fig. 5.7. It is evident from the curves that the majority of the applied load is carried in side shear. At the maximum 900-ton load only approximately 60 tons is carried by the base. It is also evident that the majority of the load transfer occurs between 20 and 45 feet. The low load transfer in the top 20 feet is due to the relatively weak soil in this zone, while the low load transfer below 45 feet is due to end effects.

Plots of load transfer versus shaft movement and peak load transfer versus depth are shown in Figs. 5.8 and 5.9 respectively. The plot of load transfer versus movement indicates that below a depth of about 25 feet, the peak load transfer has not been developed and that additional capacity is available in side shear.

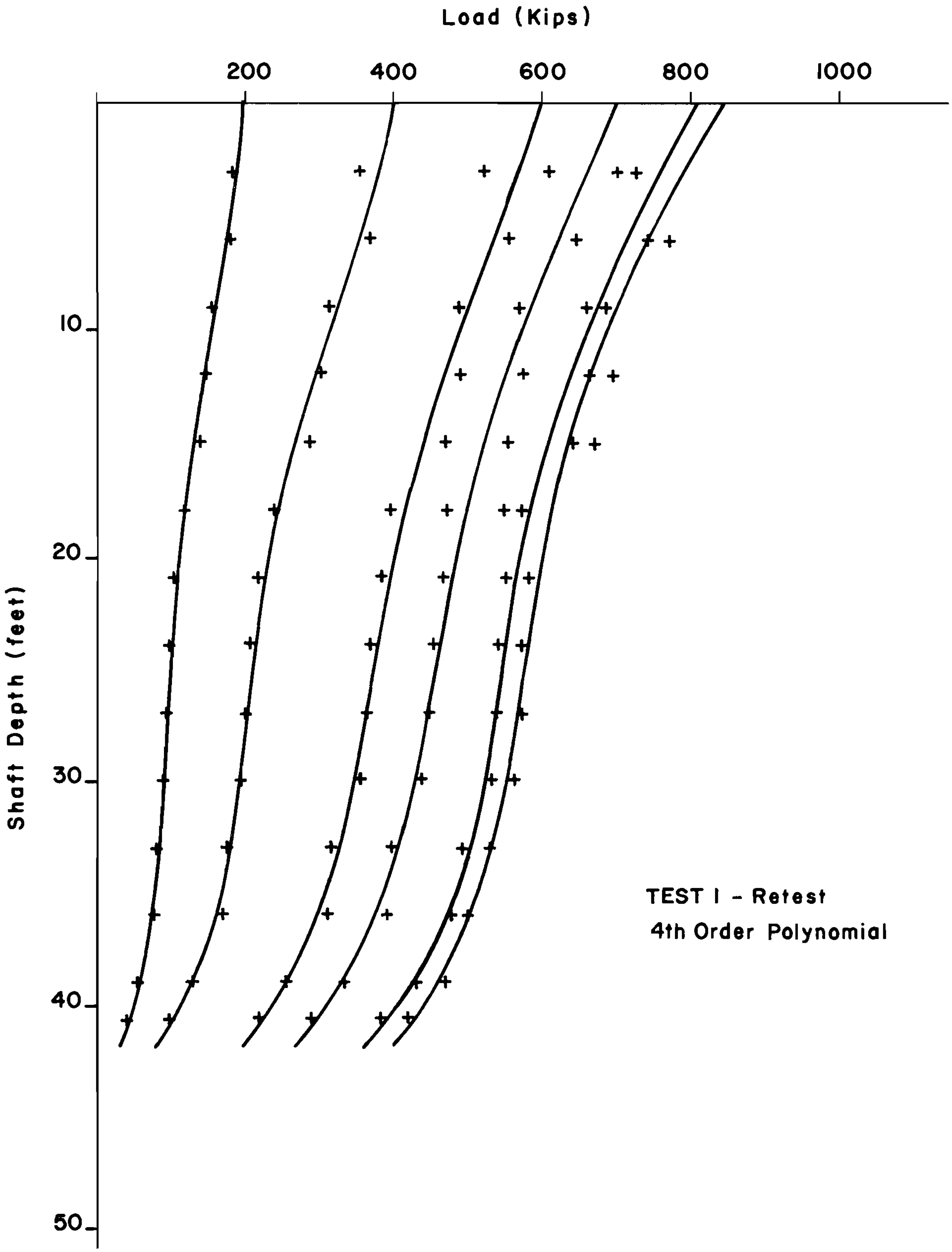


Fig. 5.4 Load Distribution Curves - Test 1, Reloaded

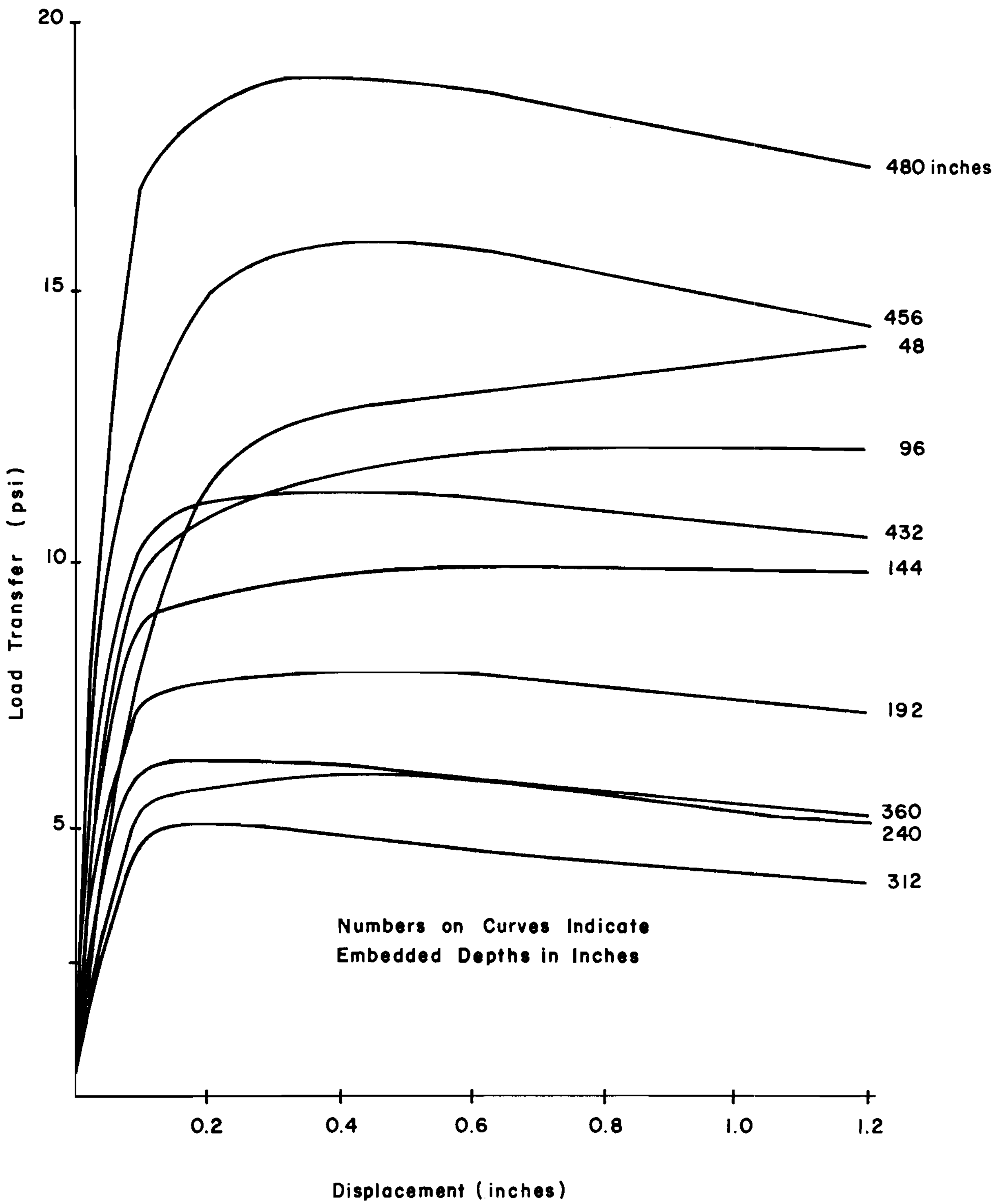


Fig. 5.5 Load Transfer Curves - Test 1, Reload

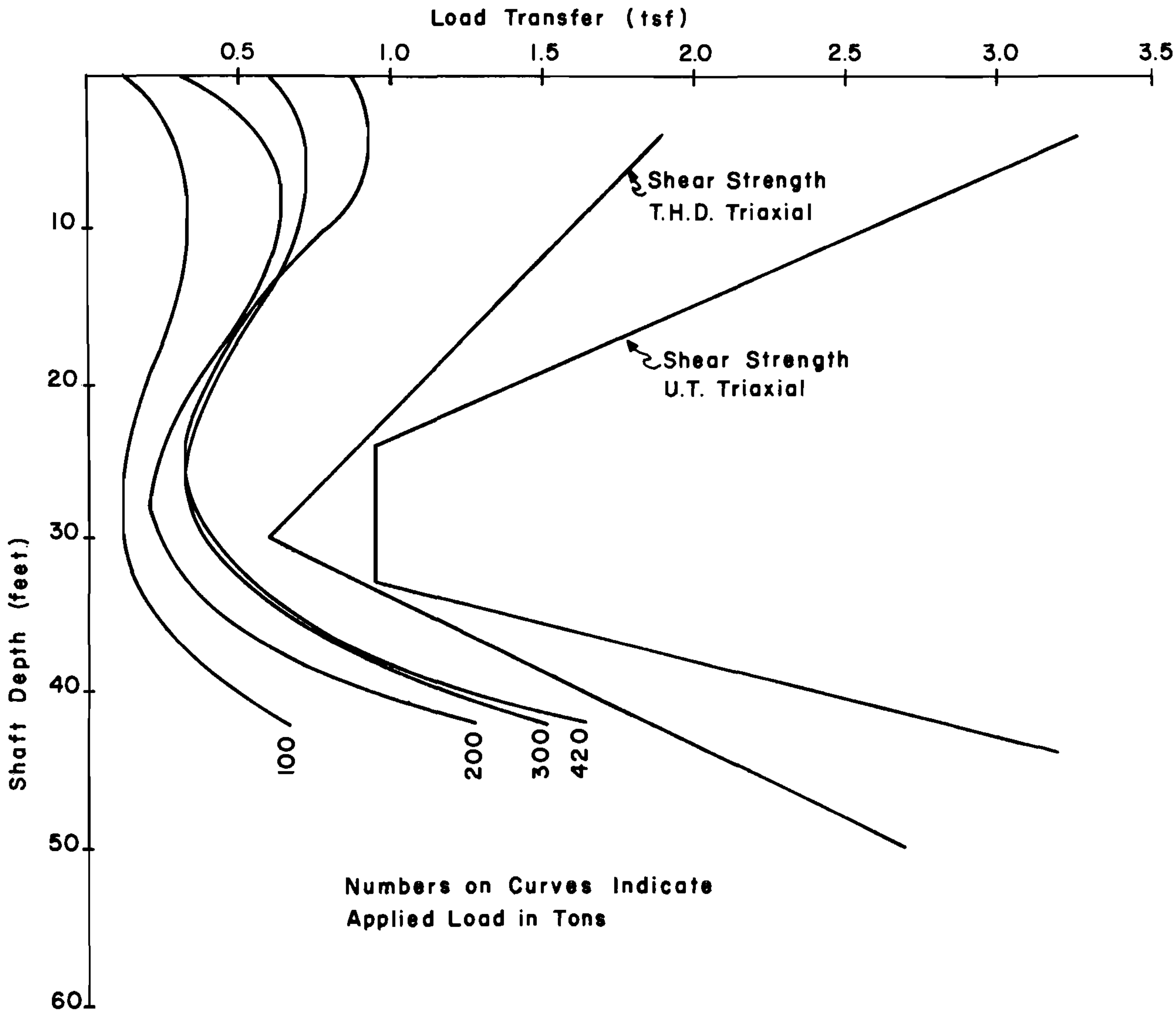


Fig. 5.6 Load Transfer Vs. Depth - Test 1, Reload

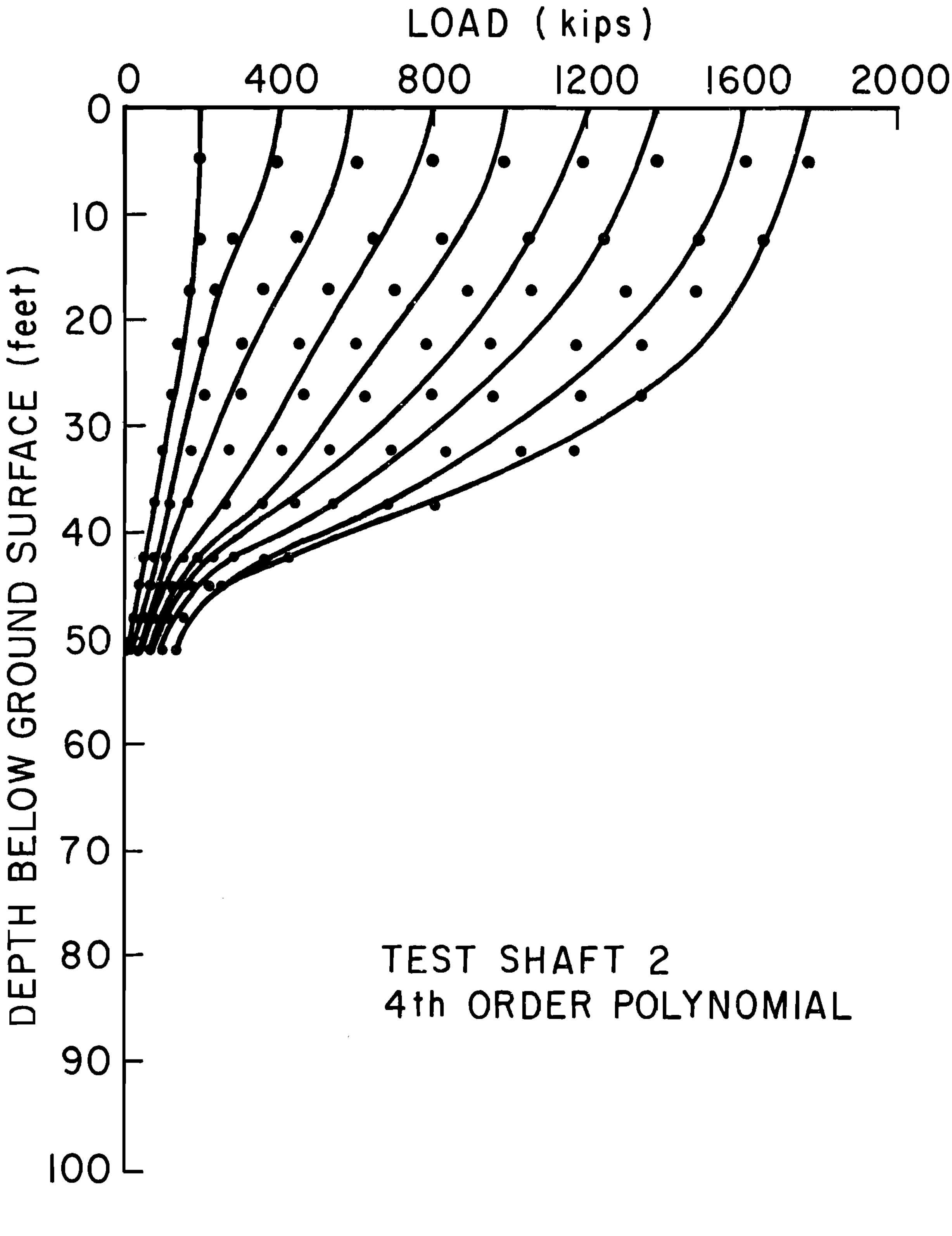


Fig. 5.7 Load Distribution Curves for Test Shaft 2

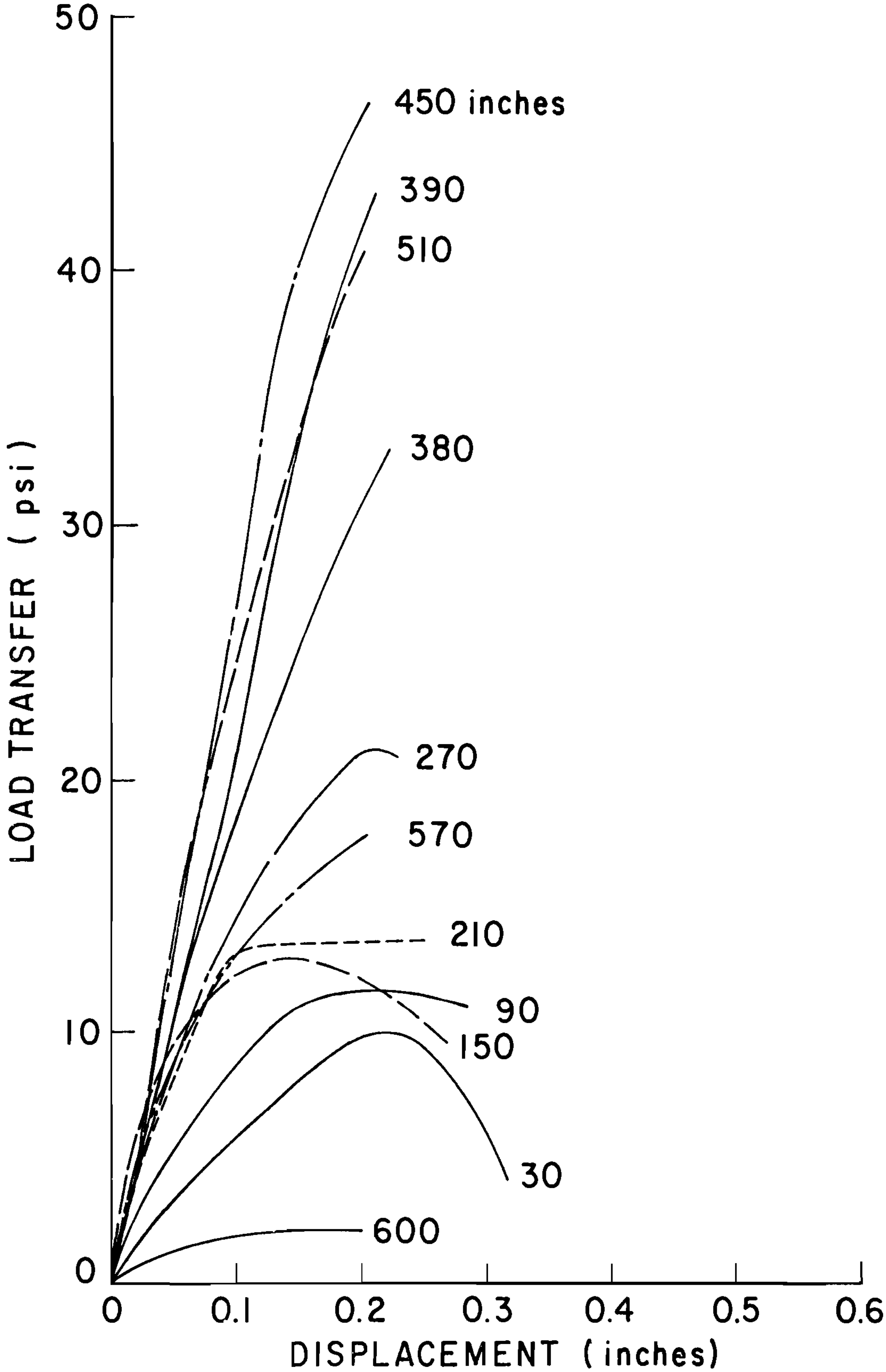


Fig. 5.8 Load Transfer Curves for Test Shaft 2

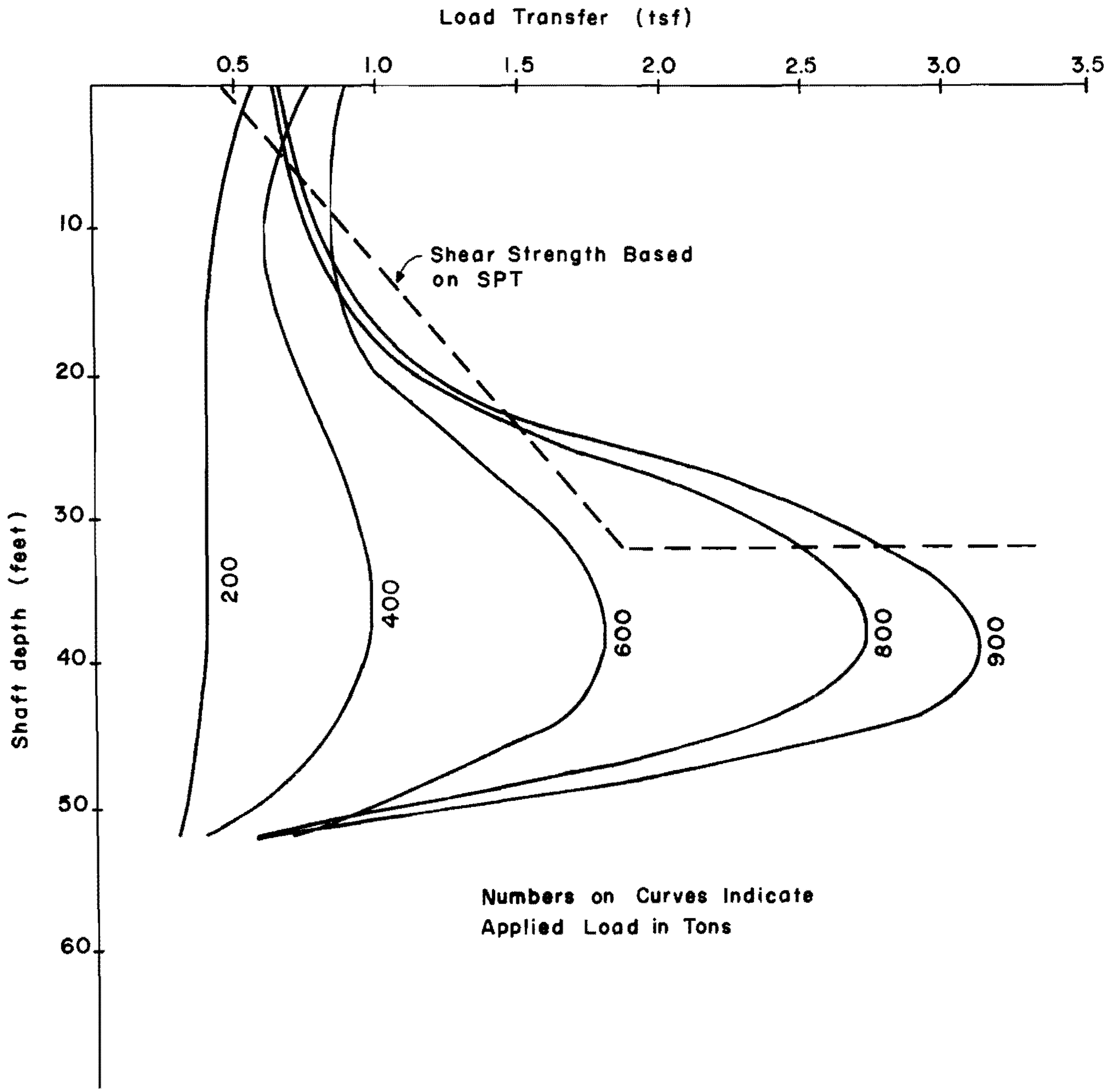


Fig. 5.9 Load Transfer Vs. Depth for Test Shaft 2

Test 3

The load distribution curves are shown in Fig. 5.10. It is evident from the curves that a significant portion of the load is being carried in side shear. At the maximum load of 825 tons, approximately 230 tons is being carried by the base. Thus, the base load is approximately 28% of the total load as compared to 7% for Test Shaft 2.

Plots of load transfer versus shaft movement and peak load transfer versus depth are shown in Figs. 5.11 and 5.12 respectively. The plots indicate that the maximum load transfer has been developed along most of the length of the shaft.

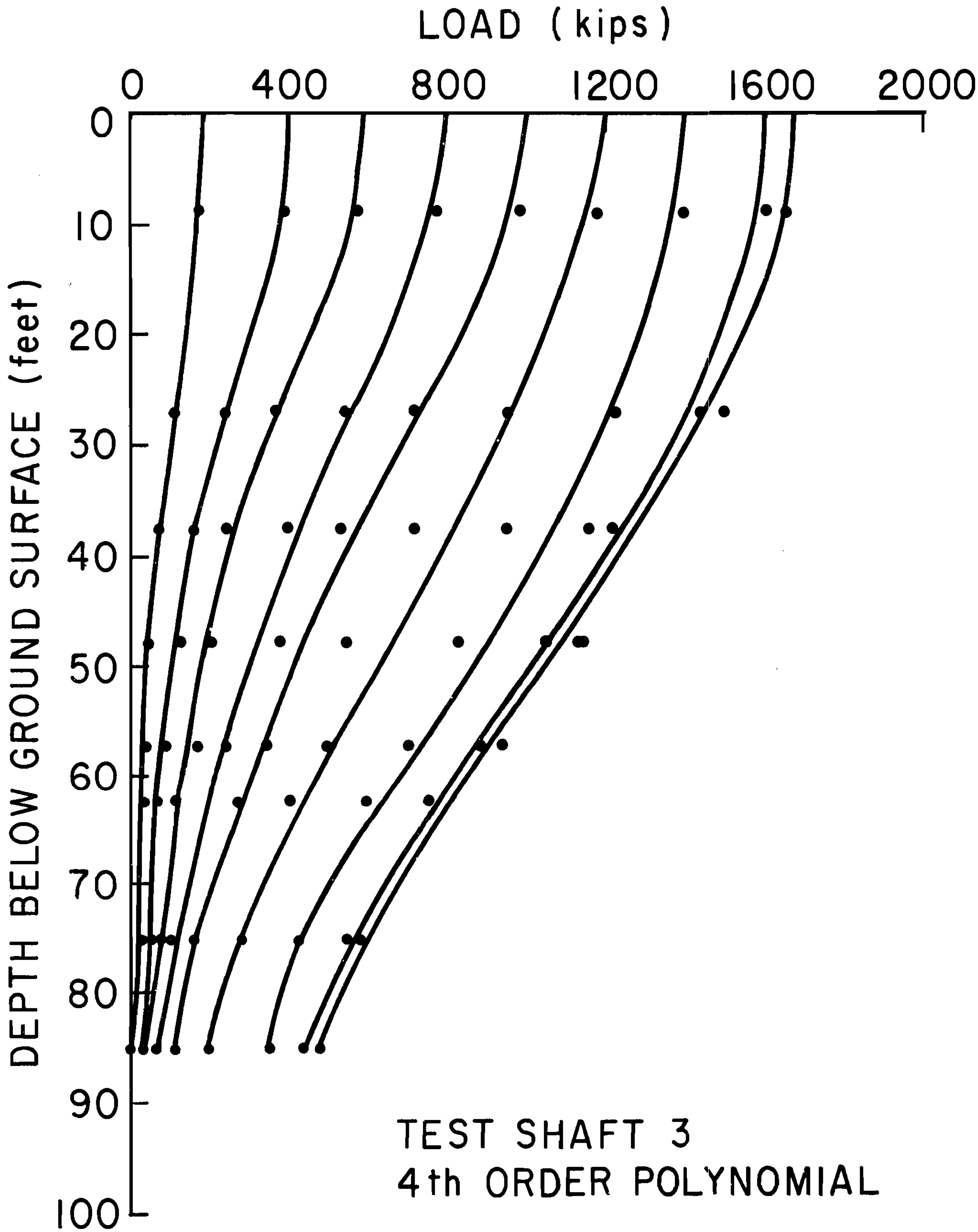


Fig. 5.10 Load Distribution Curves for Test Shaft 3

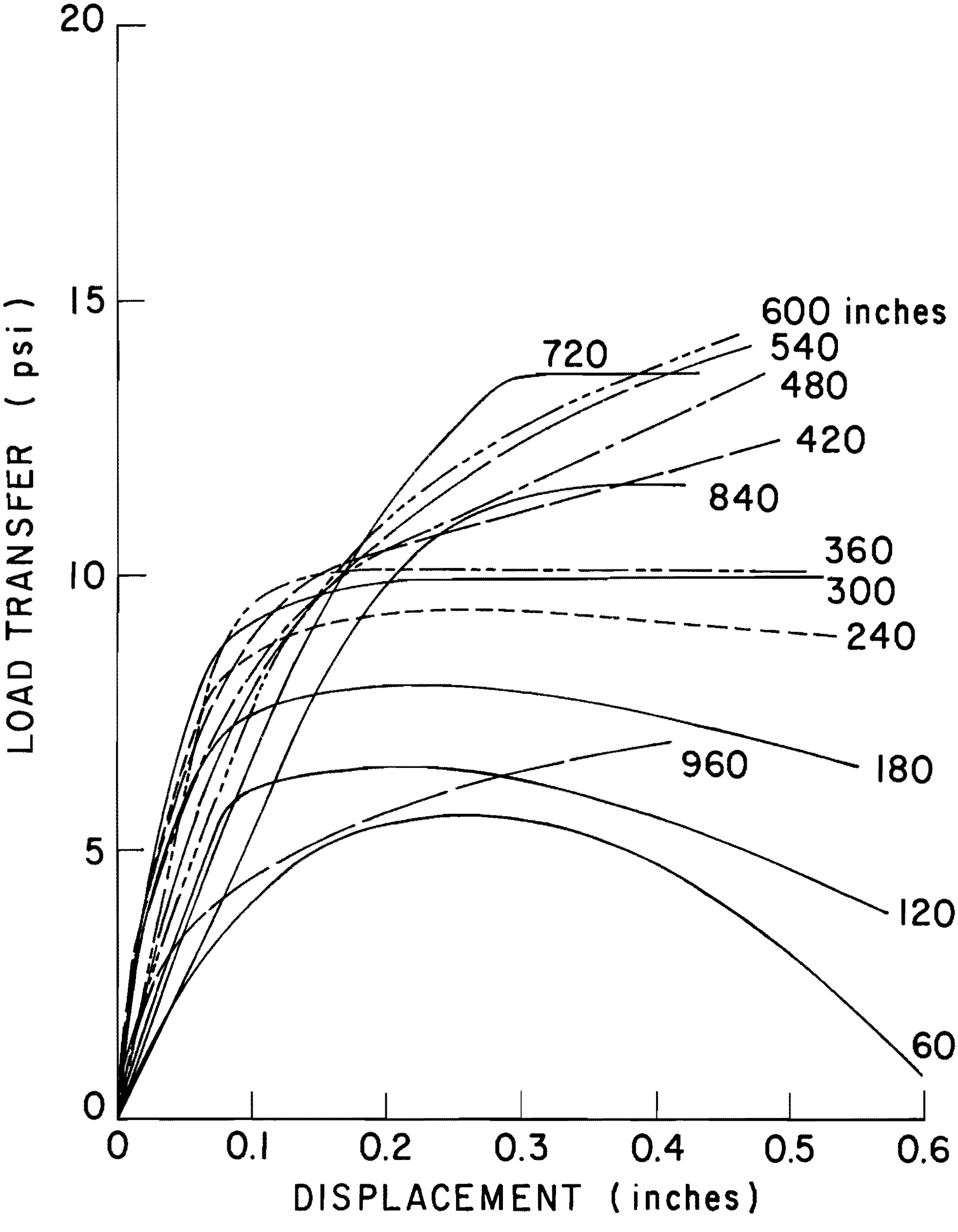


Fig. 5.11 Load Transfer Curves for Test Shaft 3

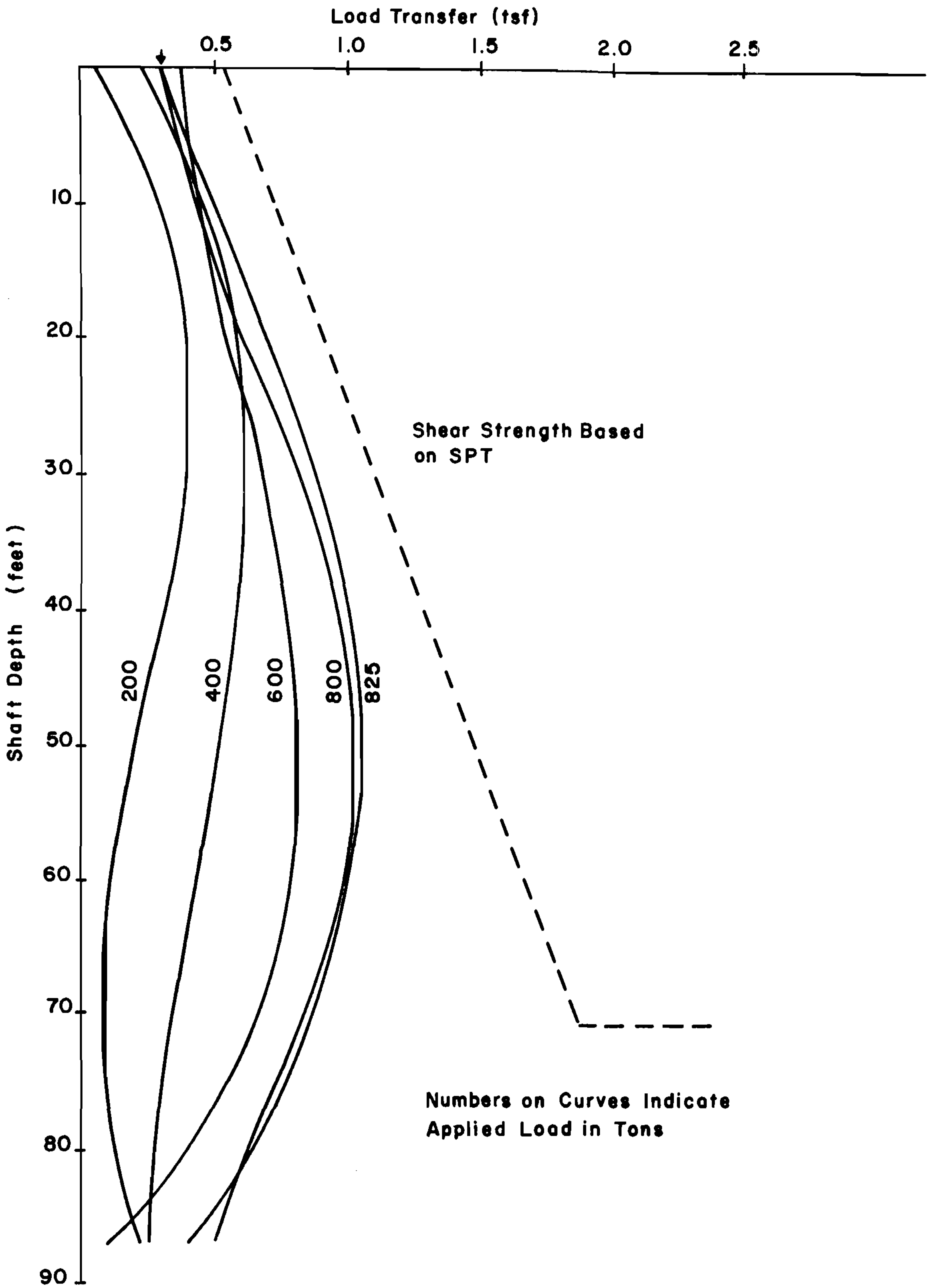


Fig. 5.12 Load Transfer Vs. Depth for Test Shaft 3

CHAPTER VI

DISCUSSION OF RESULTS

In this chapter studies will be reported that are aimed at correlating the behavior of the three test shafts with soil properties adjacent to the shafts. Each individual test was analyzed with respect to tip behavior and side behavior and an overall discussion of the significance of the results is presented.

TEST 1

For the purposes of this study, the ultimate capacity of each test shaft is defined as the load at plunging failure. The ultimate capacity of Test Shaft 1 was 425 tons for the first loading and 420 tons for the reloading.

Tip Resistance

The tip load corresponding to each of the applied loads was obtained from the load distribution curves (Fig. 5.1). Figure 6.1 presents plots of tip load versus tip settlement for the two tests conducted on Test Shaft 1. Also shown in the figure is the predicted load settlement curve based on a procedure proposed by Skempton (1951) for piles in clay. The procedure provides a correlation between the load-settlement curve and the stress-strain curve for the undrained triaxial compression test. The correlation can be expressed by the simple equations:

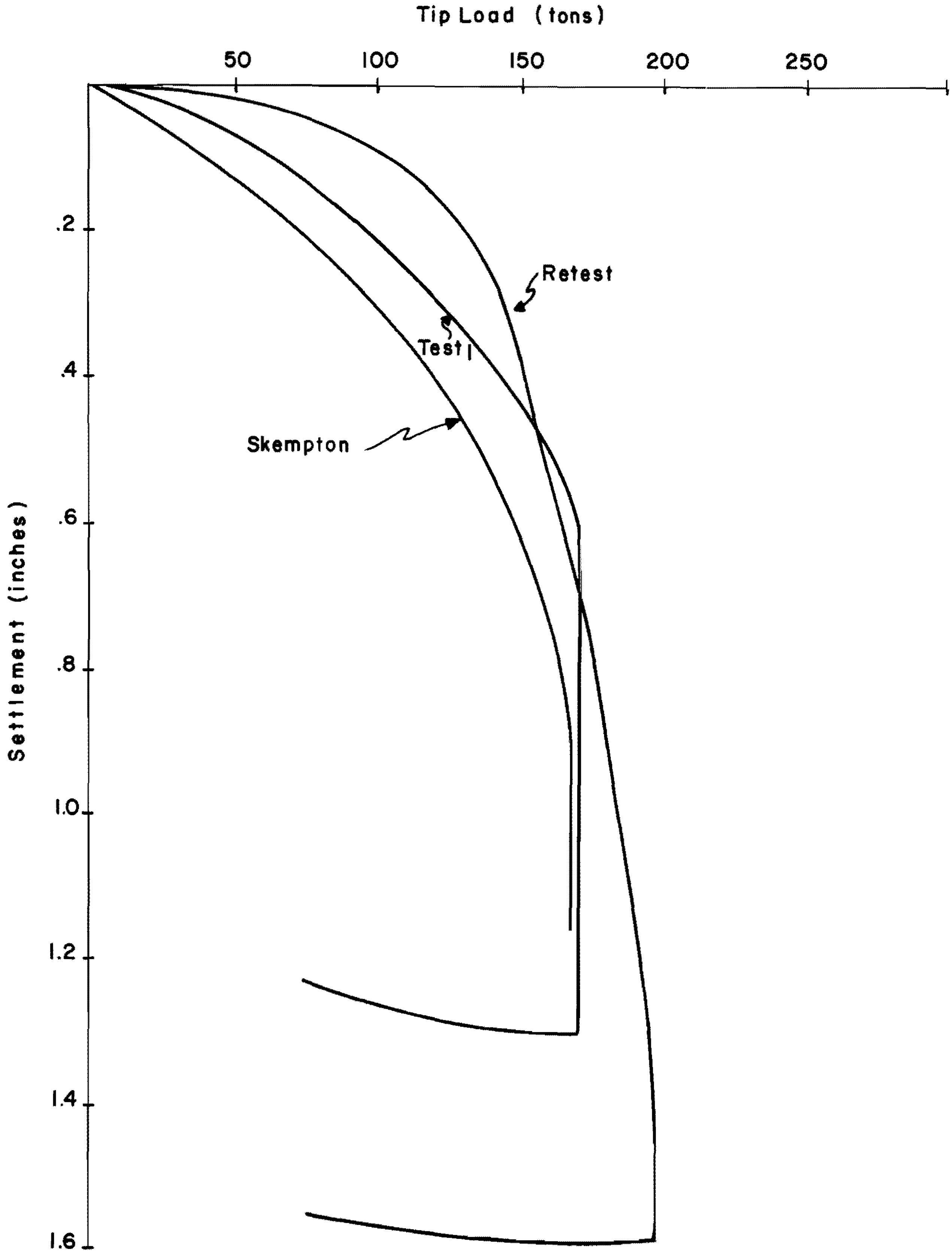


Fig. 6.1 Tip Load - Settlement Curves for Test Shaft 1

$$z = 2\varepsilon B \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.1)$$

$$q_f = \frac{1}{2}(\sigma_1 - \sigma_3)_f N_c \quad \ldots\ldots\ldots\ldots \quad (6.2)$$

$$q = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} q_f \quad \ldots\ldots\ldots\ldots \quad (6.3)$$

where

z = settlement of foundation in inches,

B = diameter of a circular foundation in inches,

ε = dimensionless strain from undrained triaxial test,

N_c = bearing capacity factor,

q_f = ultimate tip capacity in pounds per square inch,

$(\sigma_1 - \sigma_3)_f$ = principal stress difference at failure for undrained triaxial test in pounds per square inch.

The Skempton curve shown was computed using a stress-strain curve of a sample from a depth of 45 feet and a value of N_c of 10. This value of N_c is in general agreement with the findings of Skempton (1951) and is also close to values found by Barker and Reese (1970) and O'Neill and Reese (1970). The computed value of N_c based on the known bearing capacity and the U. T. triaxial undrained shear strength at one base diameter below the tip are 10.2 and 11.8 for the first and second tests respectively. The range of values reported by O'Neill and Reese (1970) for four test shafts in Houston, Texas, was from 8.7 to 12.6. The results indicate that the commonly accepted value of N_c of 9 is probably somewhat conservative but is probably appropriate for design purposes. If the shear strength based on the THD triaxial test is used, the computed values of N_c are 15.7 and 18.3 for the two tests. The high values of N_c probably indicate that the THD triaxial test underestimates the actual shear strength, as discussed previously.

The correlation between observed and predicted load-settlement relationships for the tip of Test 1 is quite good. The observed behavior is somewhat stiffer than the predicted behavior. This is possibly due to the fact that the laboratory samples underwent some swell when removed from the sample tubes. Then, upon loading in the triaxial cell the samples recompressed, thus yielding higher strains than would have occurred had no swelling been allowed.

The load-settlement curve for the tip for the retest of Test Shaft 1 is considerably stiffer in the early part of the curve. This increased stiffness is possibly because the soil at the tip had been loaded previously and failed to rebound fully upon removal of the load.

Side Resistance

The usual procedure for correlating side resistance with soil properties is to define a shear strength reduction factor. This factor is the ratio of the developed load transfer to the undrained shear strength and can be expressed as:

$$t = \alpha s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.4)$$

where

t = load transfer (tons per square foot)

s = undrained shear strength (tons per square foot)

α = shear strength reduction factor.

Peak values of α versus depth are shown in Figs. 6.2 and 6.3 for the two tests. These peak values occurred at downward displacements of the order of 0.2 to 0.4 inches. Actual values of displacement necessary to develop the maximum load transfer can be obtained from the load transfer versus movement curves presented in Chapter V.

Values of α_{max} based on U. T. triaxial test results range from 0.2 to 0.7 for the first test and 0.3 to 0.6 for the second test. The value of α_{max} varies considerably with depth and no distinct soil zones are indicated.

It is possible that a significant portion of the variation in α_{max} is due to general inaccuracies in the data. The values of shear strength and load transfer used for computing values of α_{max} are not known quantities but are subject to individual interpretation. Changes in the magnitudes of either of the two quantities can significantly alter the value of α_{max}. The low value of α_{max} at a depth of 30 feet in Fig. 6.2 is

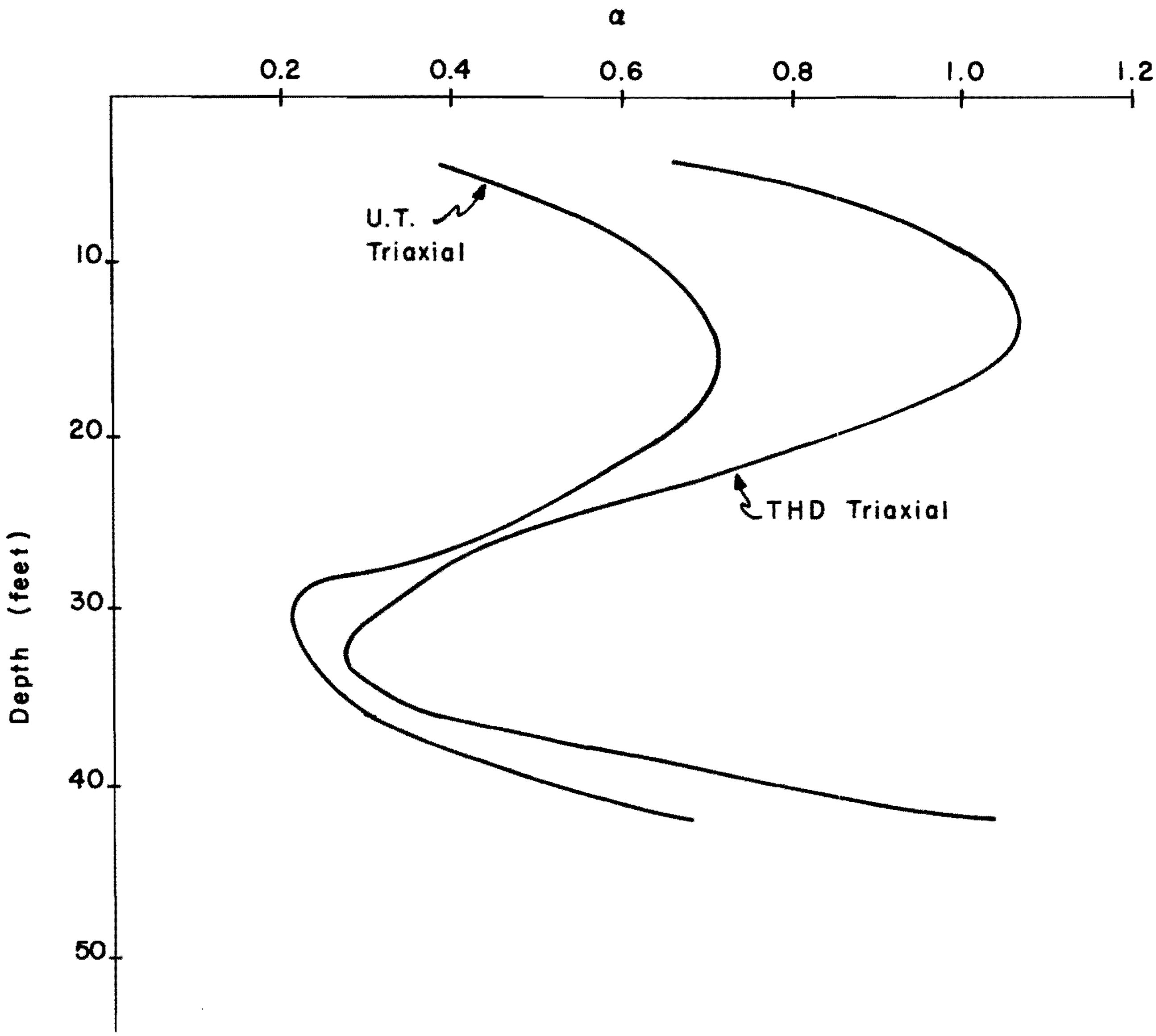


Fig. 6.2 Peak Value of α Vs. Depth for Test 1

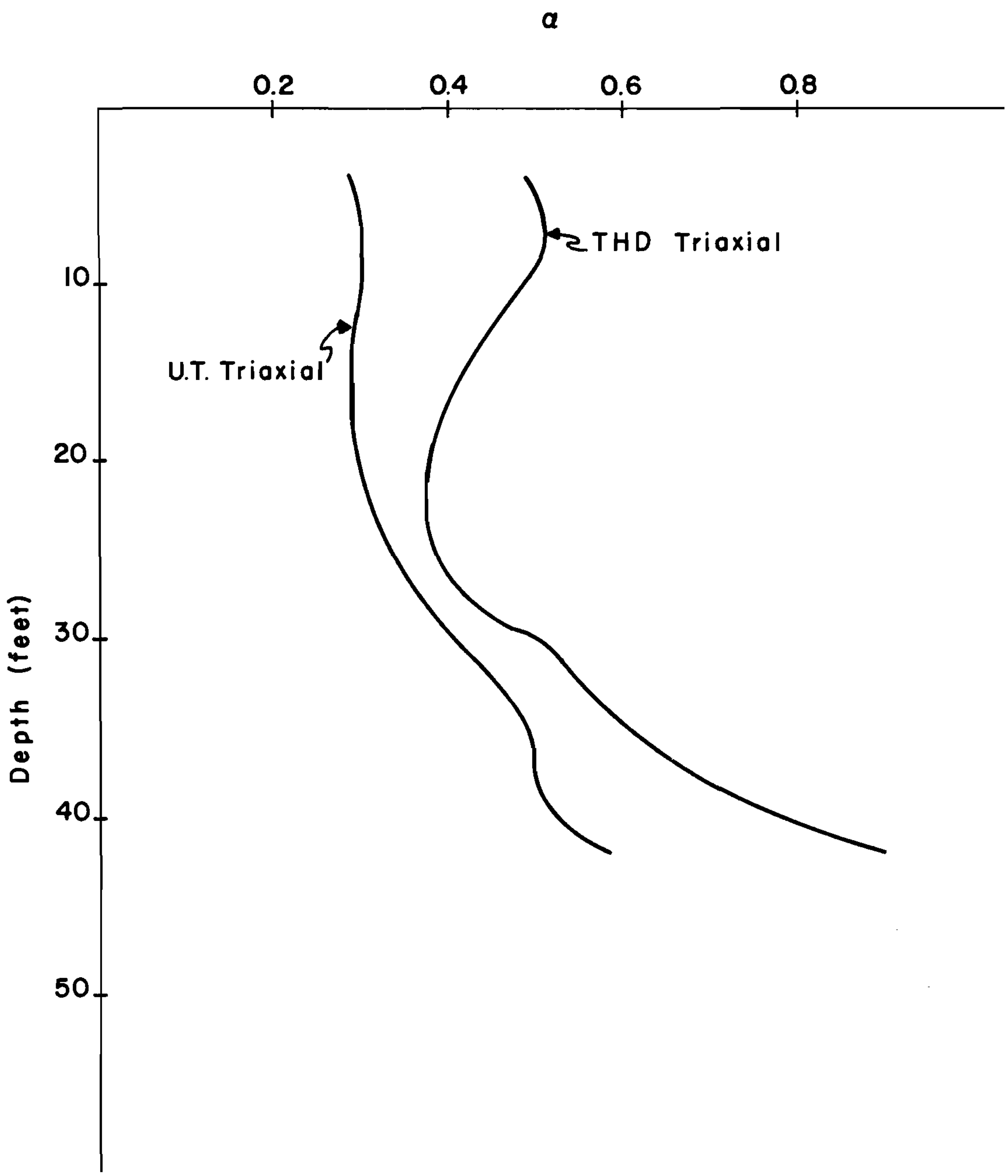


Fig. 6.3 Peak Value of α Vs. Depth for Test 1, Reload

possibly due to the presence of free water in the silt seam at that depth. As indicated by previous studies, the value of α_{max} decreases near the ground surface. However, no decrease in α_{max} was noted near the tip of the shaft until plunging occurred. At movements in excess of 1.0 inches the load transfer in the bottom five feet decreased somewhat.

Because no distinct soil zones were present, it is convenient to define an average shear strength reduction factor. This factor α_{avg} is the average α_{max} for the full depth of the shaft. The computed values of α_{avg} for the first test are 0.74 based on TDH triaxial tests and 0.52 based on U. T. triaxial tests. For the retest, computed values of α_{avg} are 0.56 and 0.40 respectively. The values of α_{avg} based on U. T. triaxial tests are in general agreement with the findings of previous research.

TEST 2

Due to the lack of capacity of the loading system, Test Shaft 2 was not loaded to plunging failure. The inability to plunge the shaft decreases somewhat the value of the data obtained in that maximum values of side and tip capacity can only be estimated. The maximum load applied to the shaft was 900 tons of which only 60 tons was transferred to the base.

Tip Resistance

The instrumentation allowed determination of the tip load, however, due to the small amount of load transferred to the base only the initial portion of the load-settlement curve for the tip was obtained.

Figure 6.4 presents the curve for tip load versus tip settlement for Test 2 and also presents the predicted curve based on Skempton's procedure. The Skempton curve was computed using a stress-strain curve of a sample from a depth of 60 feet and a value of N_c of 9. The predicted load-settlement curve indicates an ultimate tip capacity of 275 tons at 6.5 inches of movement. It seems unlikely that such a large settlement would be required to develop the tip capacity in a stiff clay soil. More than likely the laboratory samples yielded strains that are considerably higher than would occur had truly undisturbed samples been tested. It also appears from the limited data that the actual capacity would be greater than the predicted capacity.

Due to the difficulties in testing the laboratory samples it is possible that the standard penetration test results give a better indication of the actual tip capacity of the test shaft. The penetration test results are converted to undrained shear strength by use of the equation (Touma and Reese, 1972):

$$\text{in clays:} \quad s = \frac{N}{15} \quad \ldots\ldots\ldots\ldots\ldots \quad (6.5)$$

where

s = shear strength (tons per square foot)

N = number of blows per foot.

The value of shear strength at the tip from the above equation is 6.7 tons per square foot. Based on this value of shear strength and N_c of 9, the

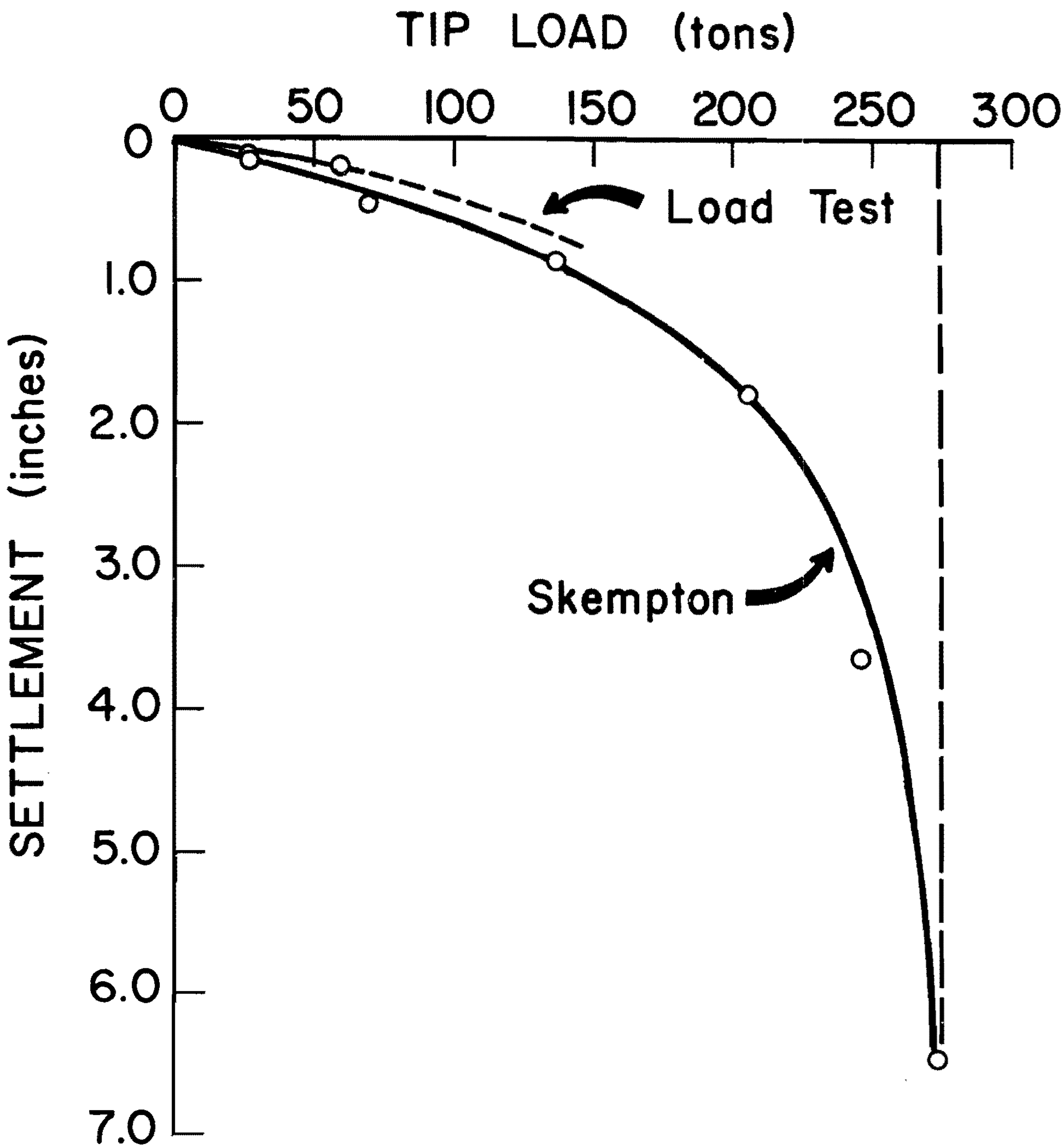


Fig. 6.4 Tip Load - Settlement Curve for Test 2

predicted tip capacity is 424 tons. This value is significantly higher than that predicted from laboratory tests. It is impossible to say for certain which value is more correct, but it is believed that the ultimate tip capacity would be of the order of 350 to 400 tons.

Side Resistance

The soil profile at the site consists of both sand and clay layers. The procedure for correlation of load transfer with soil properties in clay soils has been explained earlier. In sand deposits the procedure is somewhat different. A correlation factor α is used with a definition as shown below:

$$\text{in sand:} \quad t = \alpha \int_0^H \bar{p} \tan \bar{\phi} \quad \ldots\ldots\ldots\ldots \quad (6.6)$$

where

t = load transfer (tons per square foot)

$\bar{p}$ = effective overburden pressure (tons per square foot)

$\bar{\phi}$ = effective angle of shear resistance

Touma and Reese (1972) found that the average side resistance developed on the periphery of a drilled shaft in sand is related to the quantity $\bar{p} \tan \bar{\phi}$. The value of $\bar{\phi}$ is obtained from dynamic penetrometer blow counts using charts relating the relative density of the sand to the friction angle.

In this report the side resistance will be analyzed by two different procedures. The first procedure will use the laboratory shear strengths with no distinction being made between sand and clay zones. The second procedure will use the standard penetration test blow counts with the shear strength for sand and clay being obtained separately. The shear strength of clay is determined by dividing the blow count by 15, and the shear strength of sand is related to the quantity $\bar{p} \tan \bar{\phi}$ as discussed above. Only the layer of clean sand at a depth of 40 feet will be analyzed as a sand. The upper sand layer is very clayey and it is thought that it will behave more like a clay than a sand.

Peak values of α based on laboratory shear strengths are plotted versus depth in Fig. 6.5. As shown in the figure the values of α_{max} range from 5.0 at the ground surface to 0.3 near the tip. The fact that α_{max} exceeds a value of 1.0 for almost the entire length of the shaft indicates that either the instrumentation yielded incorrect values of load transfer of that the measured shear strengths are incorrect. Based upon observations made during construction and testing related to the test shaft, it is believed that most of the error is in the measured values of shear strength.

Peak values of α based on penetrometer blow counts are plotted versus depth in Fig. 6.5. The α values based on the penetration test are significantly lower than those based on laboratory strengths but are still higher than would be expected.

As for Test 1 it is useful to calculate an α_{avg} for the test shaft. Values of α_{avg} based on laboratory shear strength and shear

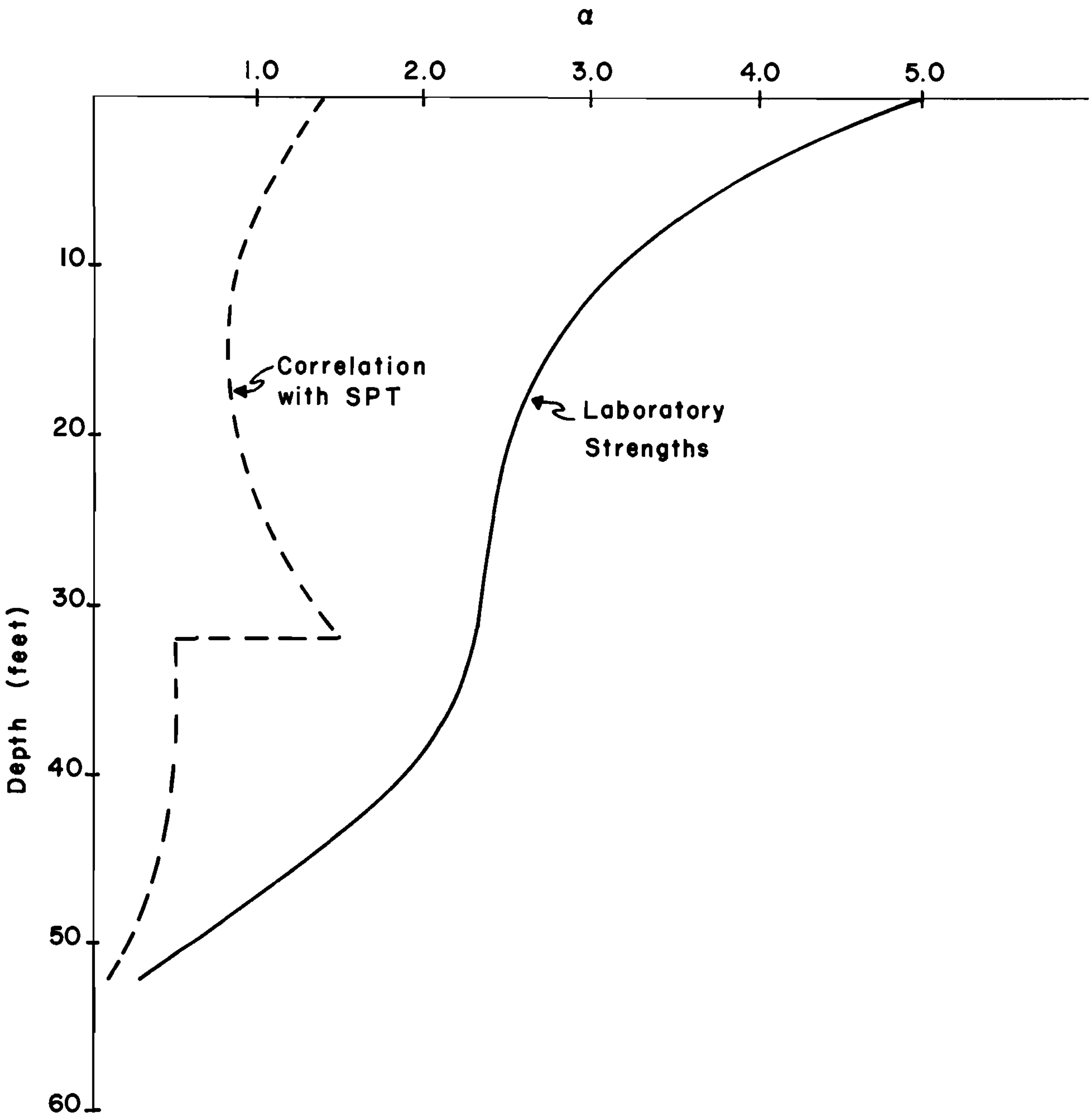


Fig. 6.5 Peak Values of α Vs. Depth for Test 2

strengths determined from correlation with penetrometer blow counts are 2.40 and 0.79 respectively. The values of α_{avg} are considerably higher than expected but the value based on penetrometer results is not unreasonable.

TEST 3

Due to failure of the pumping system, Test Shaft 3 was not loaded to plunging failure. The results indicate that most of the side shear capacity was developed and that any additional load would be carried to the tip. The maximum applied load was 825 tons of which 240 tons was carried by the tip.

Tip Resistance

The load-settlement curve for the tip of Test Shaft 3 is plotted in Fig. 6.6. The test results indicate a settlement of 0.4 inches at a load of 240 tons. The tip of Test Shaft 3 is in very dense sand. Touma and Reese (1972) suggest a method for obtaining the failure tip resistance Q_t (lb) in sand at one inch of downward movement. The method can be stated in equation form as

$$Q_t = \frac{A}{0.6B} q_t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.7)$$

where

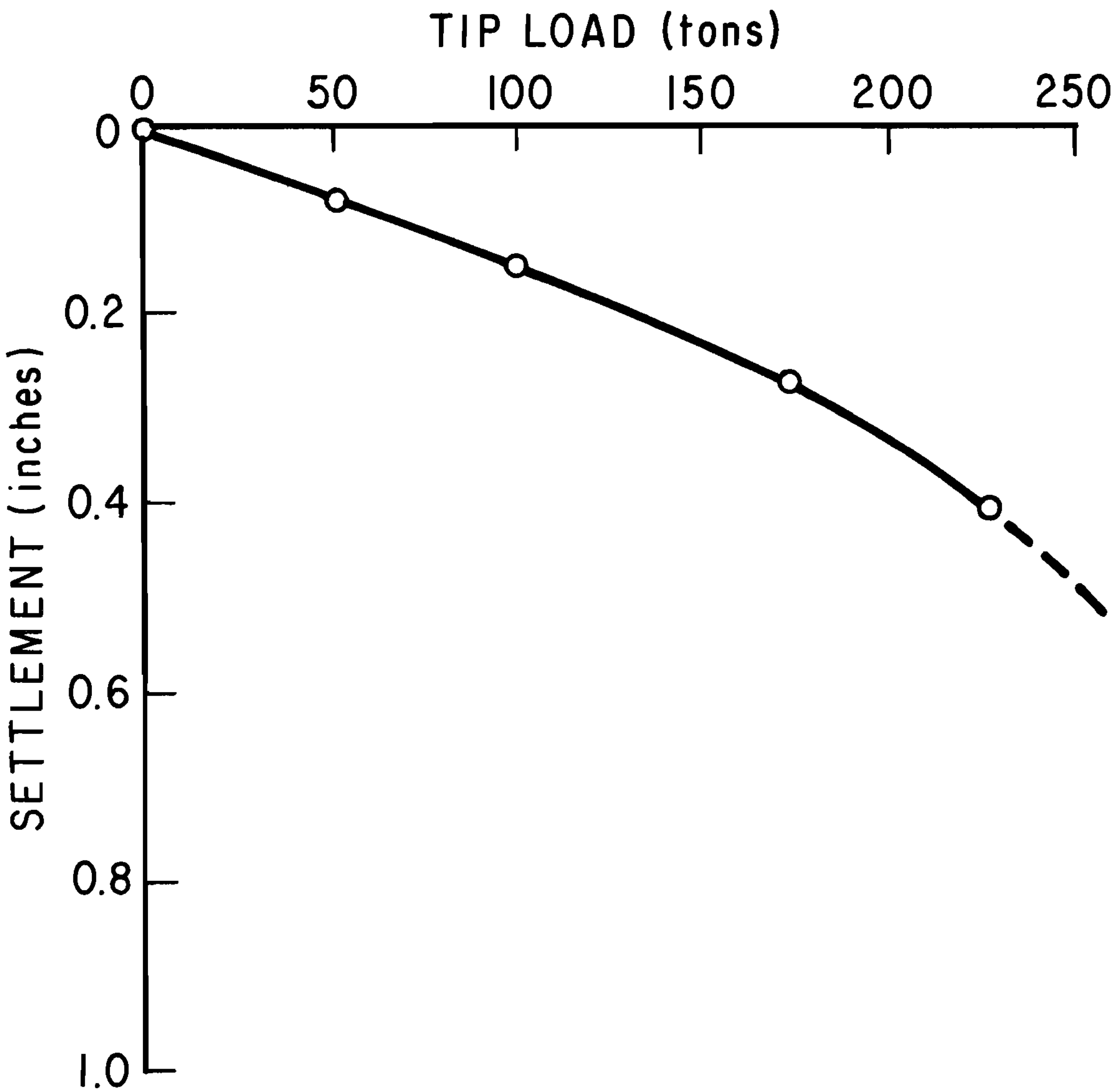


Fig. 6.6 Tip Load - Settlement Curve for Test 3

A = area of shaft, ft^2,

B = diameter of shaft, ft,

q_t = tip resistance at five percent movement. q_t is taken equal to 0, 32000, and 80000 pounds per square foot. For loose, medium, and dense sands respectively.

Based on this equation and assuming a dense sand, the tip capacity at one inch of movement is 157 tons. This value is significantly lower than the measured value as might be expected because the sand at the tip was very dense. There is little data in the literature on the tip resistance of deep foundations in very dense sands. Vesic (1970) summarizes the results of laboratory model tests on buried piles and the results of load tests in sand reported in the literature. His summary indicates that ultimate tip pressures vary from 10 tons per square foot in sands of medium density to over 100 tons per square foot in very dense sands. Assuming a tip pressure of 100 tons per square foot the ultimate tip capacity of Test 3 would be 706 tons. It is not possible to accurately predict the tip capacity, but the small value of settlement at a tip load of 240 tons indicates that the ultimate tip capacity would be at least 500 tons.

Side Resistance

The soil profile at Test 3 consisted of layers of clay, silt, and clayey sand. For purposes of this study, with regard to side resistance, all of the layers will be treated as clay. The side resistance will be analyzed using laboratory strengths and standard penetration test results. The standard penetration test results are converted to shear strength in tons per square foot by dividing the blow count by 15.

Values of α_{max} based on the two methods of obtaining shear strength are plotted versus depth in Fig. 6.7. The laboratory tests gave values of α_{max} ranging from 3.0 at the top to 0.2 at the tip. It is of interest to note that the high values of α_{max} occur in the tip 40 feet where the laboratory samples were badly disturbed. Below about 35 feet, α_{max} is less than 1.0 and results from the laboratory tests compare quite well with penetrometer results. The penetrometer results give α_{max} values that range from 0.8 in the center of the shaft to 0.1 at the tip. Both methods of defining shear strength show a definite decrease in α_{max} in the bottom 15 feet where the strength increases significantly.

Values of α_{avg} based on penetrometer blow counts and laboratory strength tests are 0.58 and 1.16 respectively. The lower of the two values agrees quite well with current design recommendations for drilled shafts in clay. It appears that had better samples been obtained in the top 30 feet that the laboratory results would also have given a good correlation with design criteria.

SUMMARY OF RESULTS

Tip Resistance

Of the three test shafts described, only one was loaded to plunging failure. Therefore, the tip capacity of the other two shafts

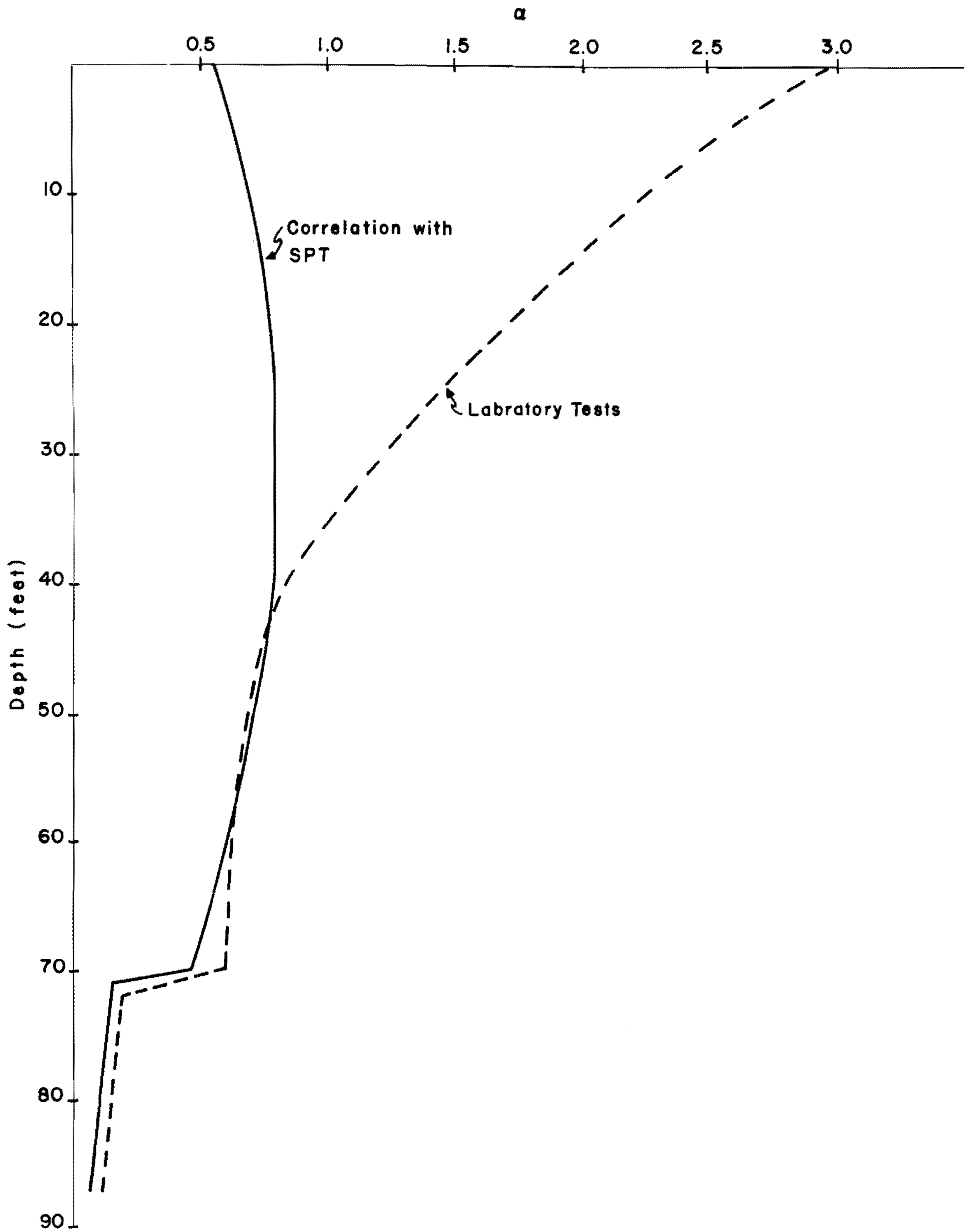


Fig. 6.7 Peak Values of α Vs. Depth for Test 3

can only be estimated. The tips of two of the shafts were founded in clay while the tip of the third was in a very dense sand.

The one shaft that was plunged was in a stiff clay soil. The analysis of the tip behavior based on controlled rate of strain triaxial tests yielded a value of N_c of 10.2. This value is well within the range of reported values from previous research. Analysis of the remaining two shafts indicated that the laboratory strengths yielded tip capacities which were too low. Therefore, the standard penetration test results were used to predict the tip capacity. For Test 2 which was tipping in hard clay, it appears that a value of N_c between 9 and 10 would yield the best estimate of the tip capacity. The results of the two shafts in clay indicate that present design procedures are adequate for predicting tip behavior in clay.

Attempts to predict accurately the tip capacity of Test Shaft 3, which was tipping in very dense sand, yielded tip capacities which were too low. It is believed that the present design criteria do not apply to sands of such high densities. It would be desirable to modify the criteria to take into account very dense sands but such modifications cannot be justified based on one test that was not carried to plunging failure.

The value of tip capacity to be used in design will depend on the settlement that can be tolerated. The magnitude of settlement required to develop the tip capacity at Test 1 was of the order of 0.6 inches while at Tests 2 and 3 it appears that larger settlements would be required. Because the amount of settlement that can be tolerated depends on the type of structure, methods will be presented in Chapter VII regarding tip settlement and will suggest limiting the settlement to tolerable amounts.

Side Resistance

The soil profiles at the three sites contained silt, clay, and sand. For the most part the sand deposits were very clayey and were therefore treated as clay when converting from penetrometer results to shear strength. Because of this, no discussion of side resistance in sand is presented.

The results of the three tests indicated that α_{max} varies considerably along the length of the shaft, and it was decided to determine an α_{avg} value for each shaft. For the two loadings of Test Shaft 1, the α_{avg} values based on controlled-rate-of-strain triaxial tests were 0.52 and 0.40 for the first and second tests respectively. For Tests 2 and 3 the α_{avg} values based on shear strength obtained empirically from results of the standard penetration test were 0.79 and 0.58 respectively. In most cases the peak values of α occurred at movements between 0.2 and 0.4 inches.

CHAPTER VII

DESIGN INFERENCES AND CONCLUSIONS

DESIGN INFERENCES

Design procedures for drilled shafts in various types of soils have been previously established based on full-scale load testing of instrumented drilled shafts. The current criteria for clay soils are presented by O'Neill and Reese (1970) and the criteria for sands are presented by Touma and Reese (1972). Those criteria are purposely somewhat conservative to insure that shafts designed with the criteria will have an adequate factor of safety against plunging failure. As more instrumented shafts are tested and more data become available, the criteria can be reevaluated to determine if the design parameters can be altered to provide greater design economy while maintaining appropriate safety.

Based on additional data which have become available it is felt that a revision of the design criteria for clay soils is desirable. No revision of the criteria for sand is suggested because very little additional data have been obtained.

Design Criteria for Drilled Shafts in Clay

The following sections discuss the design criteria for side resistance and tip resistance in clay formations, principally using the outline and equations employed by previous authors (O'Neill and Reese, 1970).

The criteria for tip resistance have not been altered but are included for completeness. Other material suggested by previous authors has also been included for completeness.

The computation of the ultimate axial load capacity of a drilled shaft in clay may be made by use of Eq. 7.1.

$$(Q_T)_{ult} = (Q_s)_{ult} + (Q_B)_{ult} \quad \ldots\ldots\ldots \quad (7.1)$$

where

$(Q_T)_{ult}$ = the ultimate axial load capacity of the shaft,

$(Q_s)_{ult}$ = the ultimate capacity of the sides,

$(Q_B)_{ult}$ = the ultimate capacity of the base.

<u>Side Resistance</u>. The ultimate side capacity in clay soils is calculated by use of Eq. 7.2.

$$Q_s\ (ult) = \alpha_{avg}\ s\ A_s \quad \ldots\ldots\ldots\ldots\ldots \quad (7.2)$$

where

α_{avg} = the appropriate shear strength reduction factor discussed later,

s = the shear strength of the soil in tons per square foot determined from laboratory tests or penetrometer tests as discussed below,

A_s = the peripheral area of the stem in square feet.

If results of unconsolidated-undrained triaxial tests conducted at the *in situ* overburden pressure are available, the value of s to be used in Eq. 7.2 is the shear strength determined from the tests. If other methods are used for determining the shear strength, the value of s to be used in Eq. 7.2 must be determined by correlation with triaxial test results. Correlations for the standard penetration test, the THD cone penetrometer, and the THD triaxial test used by the Texas Highway Department are shown by the following equations:

$$\text{SPT:} \qquad s = \frac{N}{15} \quad \ldots\ldots\ldots\ldots \quad (7.3)$$

$$\text{THD Penetrometer:} \qquad s = \frac{N}{21} \quad \ldots\ldots\ldots\ldots \quad (7.4)$$

$$\text{THD triaxial:} \qquad s = 1.7\, s_{THD} \quad \ldots\ldots \quad (7.5)$$

where

s = the shear strength to be used in Eq. 7.2 in tons per square foot,

N = the average number of blows per foot for the standard penetration test and the THD cone penetrometer, respectively,

s_{THD} = the shear strength determined from the THD triaxial testing procedure in tons per square foot.

Equation 7.3 was suggested by Touma and Reese (1972) for clays of medium to high plasticity. Equations 7.4 and 7.5 were determined from the statistical correlations shown in Figs. 7.1 and 7.2, respectively.

The selection of a value for the shear strength reduction factor, α_{avg}, to be used in Eq. 7.2 is based upon results of 12 load tests as mentioned earlier. The values of α_{avg} for clays presented earlier in this report and those reported by other authors were computed for the full depth of the shaft with no exclusion of the top or bottom five feet of the shaft as recommended by the current design criteria. Most of the test shafts indicate a definite decrease in load transfer in the top five feet so it is felt that the top five feet should be excluded. The matter of whether or not to exclude the bottom five feet when computing side resistance is not clear cut. About 50% of the shafts indicate a reduction in load transfer in this zone while the other 50% indicate no reduction. At this time it is recommended that the bottom five feet be excluded and that the value of α_{avg} be adjusted accordingly.

Computed values of α_{avg} neglecting the top and bottom five feet are tabulated in Table 7.1 for the 12 test shafts. The steps involved in the computation of the α_{avg} values are outlined below:

1. Obtain the measured side resistance in clay from the load distribution curves for each shaft.
2. Obtain the average shear strength of the clay along the shaft according to the methods described previously.
3. Compute the side resistance using Eq. 7.2 and assuming an α_{avg} of 1.0 and the value of shear strength determined in step 2.

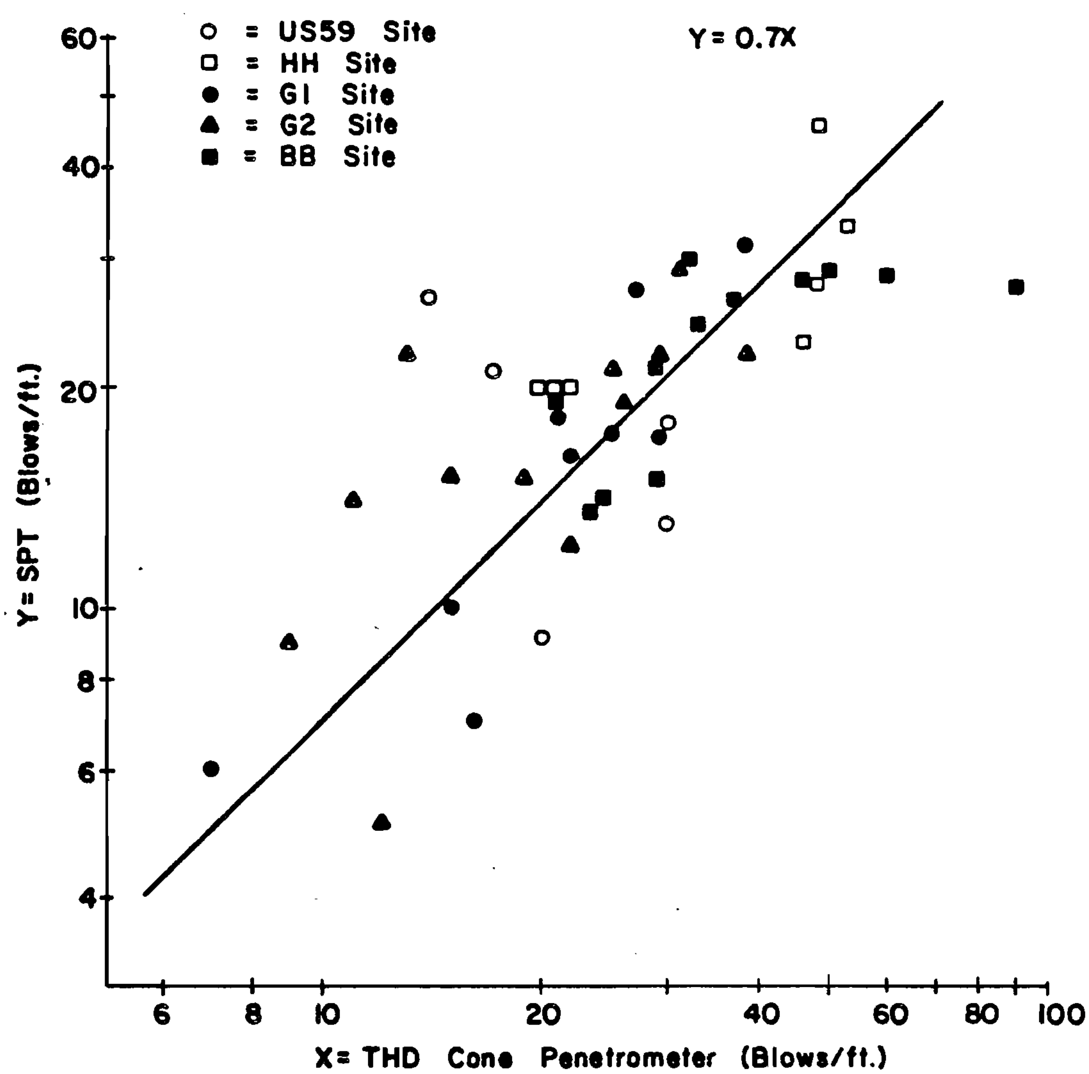


Fig. 7.1 Correlation Between SPT and THD Cone Penetrometer in Clay

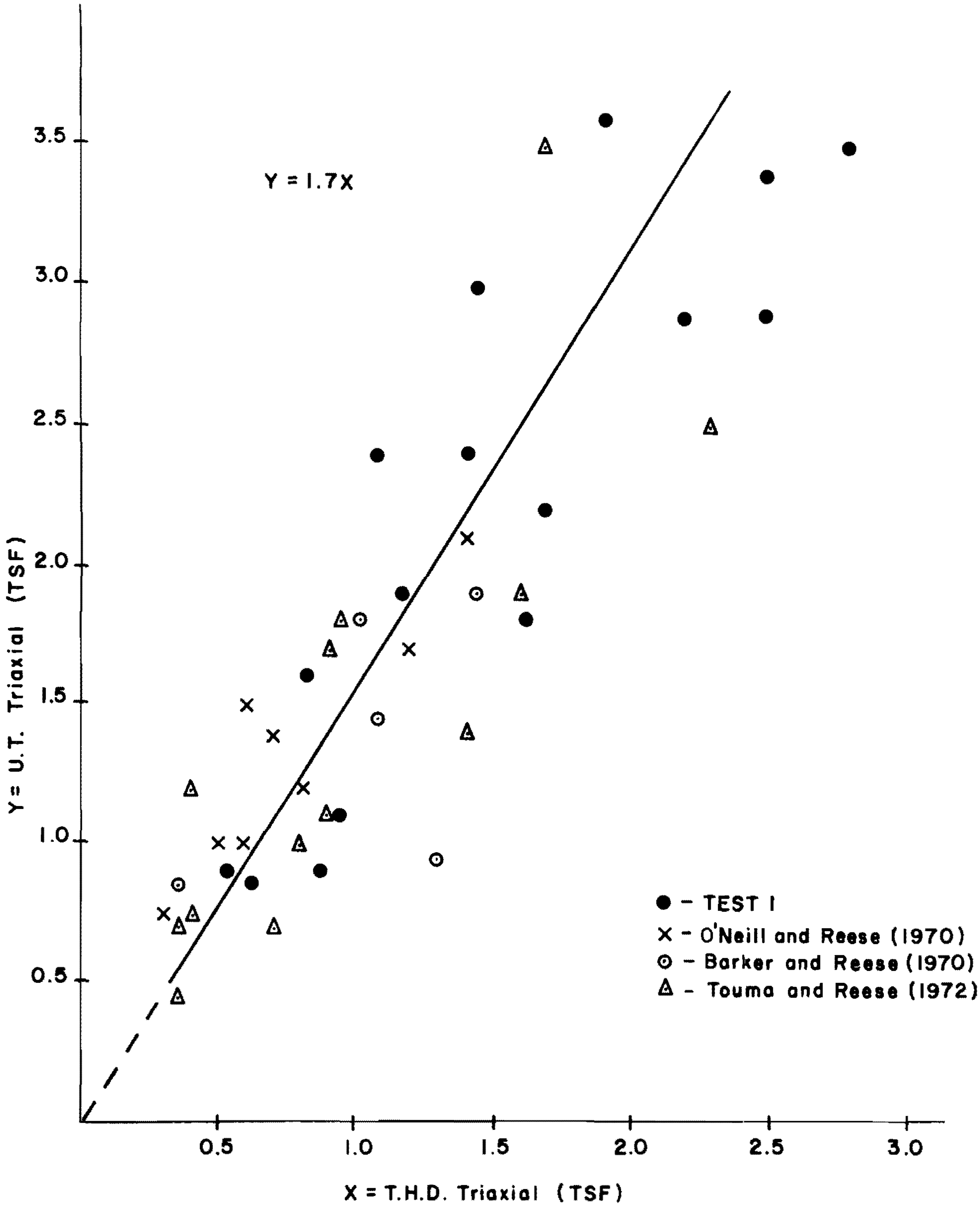


Fig. 7.2 Correlation Between U. T. Triaxial and THD Triaxial in Clay

TABLE 7.1

VALUES OF α_{avg} BASED ON LOAD TESTS

	Shaft	Diameter (inches)	Depth in Clay (feet)	Measured Side Resistance (tons)	Average Shear Strength (tsf)	Computed Side Resistance (tons)	α_{avg}
	Test 1	30	42	240	1.6	400	0.60
	Test 2	36	44*	740	3.3	1050	0.70
	Test 3	36	72*	480	1.3	820	0.59
Touma and Reese	US 59	30	13	50	1.4	88	0.91
	HH	24	12	90	1.7	75	1.21
	G1	36	27	130	0.9	185	0.70
	G2	30	55	360	1.5	590	0.61
	BB	30	23	220	2.7	390	0.58
O'Neill and Reese	S1	30	23	90	1.3	130	0.68
	S2	30	23	90	1.3	130	0.68
	S3	30	24	120	1.3	140	0.83
	S4	30	45	190	1.3	360	0.53

* Includes layers of clayey sand

4. Compute α_{avg} by dividing the measured side resistance by the computed side resistance.

The computed values of α_{avg} range from 0.53 to 1.21 and the average is 0.72. It appears that a value of 0.6 would be reasonable for design.

Tip Resistance. The ultimate tip capacity in clay soils is calculated using Eq. 7.6 or 7.7 depending on the type of test data available.

From laboratory tests:

$$(Q_B)_{ult} = N_c \, c \, A_B \quad \ldots\ldots\ldots\ldots\ldots \quad (7.6)$$

From penetrometer results:

$$(Q_B)_{ult} = \frac{N}{p} A_B \quad \ldots\ldots\ldots\ldots\ldots \quad (7.7)$$

where

N_c = a bearing capacity factor given in Table 7.2,

c = the average undrained cohesion of the soil in tons per square foot, for a depth of two base diameters beneath the base ("shear strength" may be substituted for cohesion for soils having undrained angle of internal friction of 10 degrees or less)

A_B = the area of the base in square feet,

TABLE 7.2

DESIGN PARAMETERS FOR DRILLED SHAFTS IN CLAY

	Parameter	Design Category A.1	A.2	B.1	B.2	C	D
Side Resistance in Clay	α_{avg}	0.6	0.3[A]	0.3	0.15[C]	0	0
	Limit on Side Shear (tsf)	2.0	0.5[B]	0.5	0.3[D]	0	0
Tip Resistance in Clay	N	9	9	9	9	9	9
	p (SPT)	1.6	1.6	1.6	1.6	1.6	1.6
	p (THD penetrometer)	2.8	2.8	2.8	2.8	2.8	2.8

A May be increased to Category A.1 value for segments of shaft drilled dry

B Limiting side shear = 2.0 tsf for segments of shaft drilled dry

C May be increased to Category B.1 value for segments of shaft drilled dry

D Limiting side shear = 0.5 tsf for segments of shaft drilled dry

p = the appropriate base capacity factor given in Table 7.2,

N = the average number of penetrometer blows per foot for a distance of two base diameters beneath the base.

Design Categories. There are several categories of drilled shafts from the standpoint of design, as shown below. The categories are dependent on the method of construction, the geometry of the shaft, and the nature of the soil at the site, and are identical to those recommended previously (O'Neill and Reese, 1970).

Category A: Straight-sided shafts in either homogeneous or layered soil with no soil of exceptional stiffness below the base.

Category A.1: Shafts in Category A installed dry or by the slurry displacement method.

Category A.2: Shafts in Category A installed with drilling mud along some portion of the hole such that the entrapment of drilling mud between the sides of the shaft and the natural soil is possible.

Category B: Belled shafts in either homogeneous or layered clays with no soil of exceptional stiffness below the base.

Category B.1: Shafts in Category B installed dry or by the slurry displacement method.

Category B.2: Shafts in Category B installed with drilling mud along some portion of the hole such that the entrapment of drilling mud between the sides of the shaft and the natural soil is possible.

Category C: Straight-sided shafts with base resting on soil significantly stiffer than the soil around the stem.

Category D: Belled shafts with base resting on soils significantly stiffer than the soil around the stem.

Design parameters for drilled shafts in clay are shown in Table 7.2. The parameters are shown for the various design categories as listed above.

A design can be made by making computations as shown in Eqs. 7.1 through 7.7, except that the side resistance over portions of the drilled shafts shall not be counted as explained.

1. The top five feet of a drilled shaft in clay shall be excluded from consideration in calculating axial load.
2. The periphery of a bell shall be excluded in calculating the axial capacity of a drilled shaft in clay.
3. The bottom five feet of a straight shaft and the bottom five feet of the stem of a shaft above the bell shall not be considered in calculating the axial capacity.

These recommendations are illustrated in Fig. 7.3.

The final point in the design recommendations concerns a check to insure that settlement is not excessive. This check is based on the

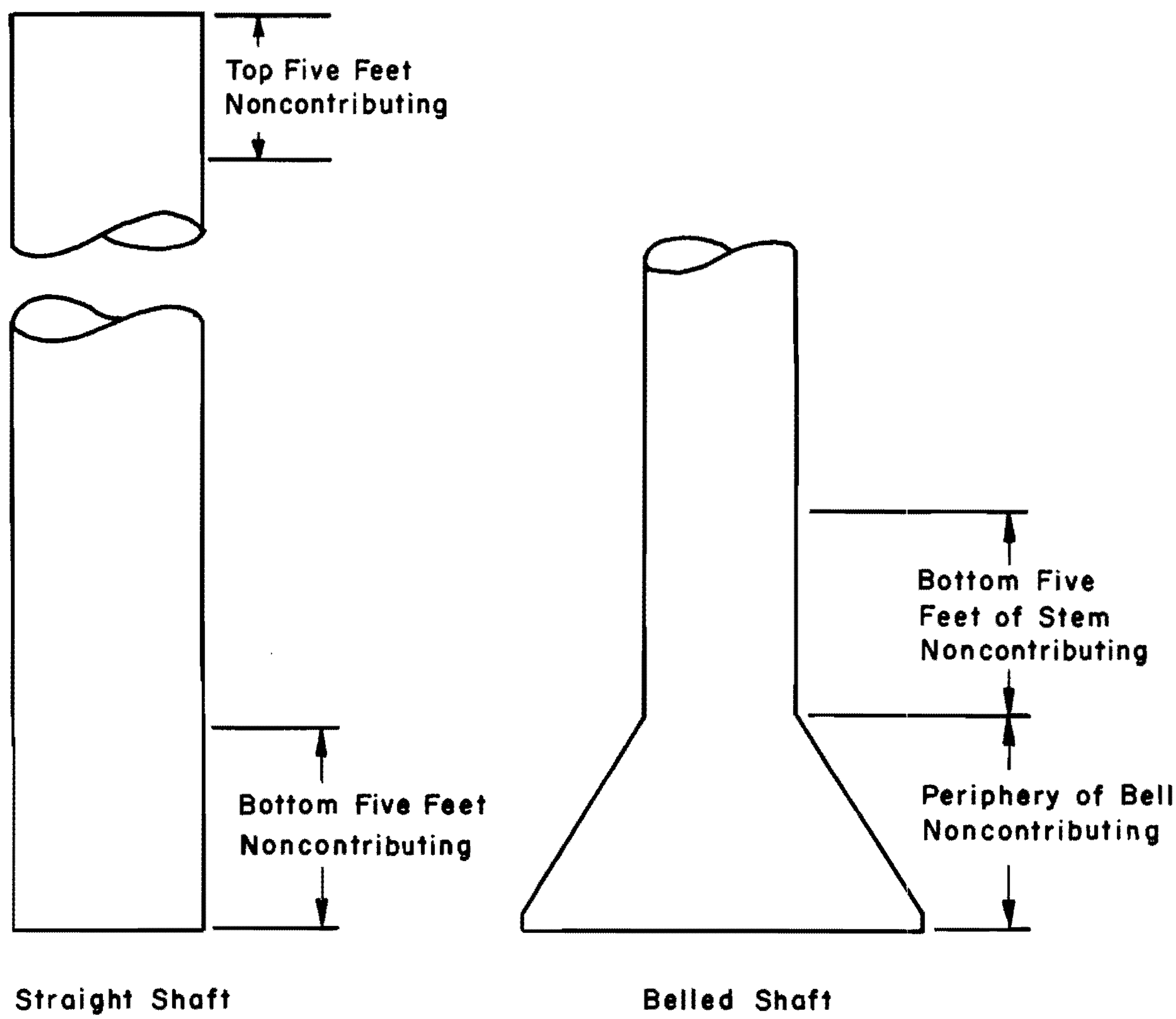


Fig. 7.3 Design Recommendations for Drilled Shafts in Clay

concept that only a small amount of downward movement is necessary to develop full load transfer in side resistance and that the downward movement necessary to develop the full resistance of the base of the shaft is a function of the shaft diameter.

Equation 7.8 must be satisfied in order for immediate settlement to be tolerable.

$$(Q_T)_{Design} = (Q_S)_{ult} + \frac{(Q_B)_{ult}}{3} \quad . \; . \; . \; . \; . \; . \; . \quad (7.8)$$

The equation insures the immediate settlement for most cases will be less than one inch. Only for relatively short, belled shafts will Eq. 7.8 govern the design (O'Neill and Reese, 1970).

Design Criteria for Drilled Shafts in Sand

The following sections discuss the design criteria for side resistance and tip resistance in sand formations. The criteria presented were developed by Touma and Reese (1972) and have not been revised. This presentation is that presented by those authors and is presented for completeness.

The axial load capacity of a drilled shaft at failure may be computed by use of Eq. 7.9.

$$(Q_T)_f = (Q_S)_f + (Q_B)_f \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (7.9)$$

where

$(Q_T)_f$ = the axial load capacity of the shaft at failure,

$(Q_S)_f$ = the capacity of the sides at failure,

$(Q_B)_f$ = the capacity of the base at failure.

The side and base capacities are calculated independently from results of laboratory tests on representative soil samples or from subsurface penetrometer soundings.

Concepts which were employed in developing the design recommendations for drilled shafts in sand are as follows:

1. Experiments reveal that the ultimate resistance of the sides of the shaft is developed at a downward movement of about 0.25 to 0.50 inches.
2. Large downward movements are necessary for mobilizing the base resistance. (For a 30-inch diameter shaft, experiments show that most of the base resistance is developed by a downward movement of 1.5 inches.)
3. "Failure" is assumed to occur at a downward movement of one inch.
4. A factor of safety of two is normally recommended for computing working load.

The base resistance for a drilled shaft in sand may be computed by use of Eq. 7.10.

$$(Q_B)_f = \frac{A}{K} q_t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7.10)$$

where

A = cross-sectional area of base of shaft, ft^2,

q_t = base resistance at a downward movement of 5% of diameter (see Table 7.3), lb/ft^2,

K = a reduction factor (see below)

The base resistance q_t is tiven in Table 7.3 in terms of the relative density of the sand at the site. Equation 7.10 applies to drilled shafts with diameters of two feet or larger.

The reduction factor is to account for the fact that the allowable stress on the base should be limited by deflection and may be understood by reference to Fig. 7.4. The figure shows the initial portion of a hypothetical unit bearing stress versus settlement curve for the base of a drilled shaft. If it is assumed that the upper portion of the curve can be approximated by a straight line, the following equation results from similar triangles.

$$\frac{q_t}{q_t/K} = \frac{0.05B}{0.083}$$

$$K = 0.6B \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7.11)$$

The capacity of the sides of a drilled shaft in sand at failure may be computed by Eq. 7.12.

TABLE 7.3

BEARING STRESS FOR DRILLED SHAFTS IN SAND FOR SETTLEMENT OF 5 PER CENT OF BASE DIAMETER

Relative density of sand	q_t, lb/ft^2
Loose	0
Medium	32,000
Dense	80,000

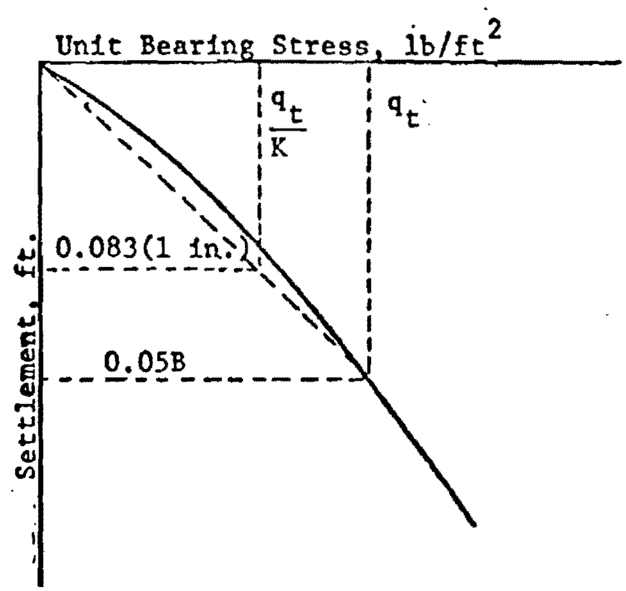


Fig. 7.4 Development of Reduction Factor for Tip Resistance

$$(Q_S)_f = \alpha_{avg}\ C \int_0^H \bar{p} \tan \bar{\phi}\ dz \quad . \ . \ . \ . \ . \ . \ . \ . \ . \quad (7.12)$$

where

α_{avg} = a factor to allow correlation with experimental results (see below)

C = circumference of shaft, ft,

H = total depth of embedment in sand, ft,

$\bar{p}$ = effective overburden pressure, lb/ft^2

$\bar{\phi}$ = effective friction angle, degrees,

dz = differential element of length, ft.

The results of the load tests indicated a maximum α_{avg} of about 0.7 which can safely be used for the design of shafts of a penetration in sand not exceeding 25 feet. There are indications that the value of α_{avg} may decrease with greater penetration in sand and, therefore, smaller values must be used in the design of deeper shafts. Pending future research, it is suggested that for shafts penetrating more than 25 feet but less than 40 feet in sand an α_{avg} value of 0.6 shall be used and that an α_{avg} value of 0.5 shall be used for shafts of greater penetration.

Where an effective angle of internal friction is not known, correlations from the results of the standard penetration test can probably be used if appropriate precautions are taken.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the results of the three instrumented load tests described in this report, several conclusions have been drawn. These conclusions have been discussed in previous sections and are enumerated below.

1. The Mustran system of instrumentation is adequate for the measurement of axial loads in drilled shafts. The system is reliable and stable and is able to resist considerable abuse.
2. The loading procedure used by the Texas Highway Department is well suited to the type of research being done.
3. Plunging of the shafts is essential in order to obtain the most information from the load tests. Had the shafts in Puerto Rico been plunged the information obtained would have been more useful.
4. It appears that the greatest uncertainty in the design of drilled shafts is the determination of the *in situ* soil properties. Different testing methods yield results that vary considerably. The variation is due to disturbance and differences in testing techniques.
5. The design criteria presented by O'Neill and Reese (1970) for drilled shafts in Beaumont clay are conservative. Based upon the three load tests described in this report and upon a reevaluation of previous tests it is recommended that the value of α_{avg} for clays be increased from 0.5 to 0.6 and that

the limiting side shear be increased from 0.9 tons per square foot to 2.0 tons per square foot.

6. The bearing capacity of the tip of drilled shafts in clay can be estimated reliably using usual bearing capacity theories. It appears that a value of N_c of 9 or 10 gives good correlation with test results.

The following recommendations are made concerning future research on drilled shaft behavior.

1. Additional short-term load tests are needed in a variety of soil formations in order to verify the design procedures and to allow the procedures to be used confidently at any location. Test shafts must be instrumented and tests must be conducted to plunging failure in order to advance the state of the art.
2. Better techniques are needed for the determination of _in situ_ soil properties. A device that minimizes disturbance and that gives repeatable results would be desirable. If an improved method is developed it would be desirable to reanalyze previous sites based on better strength data.
3. Development of a device for remote measurement of borehole diameter would be desirable. Such a device would eliminate errors in data interpretation due to variations in shaft diameter. It would be necessary for the device to operate in dry holes as well as holes drilled with a slurry.

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