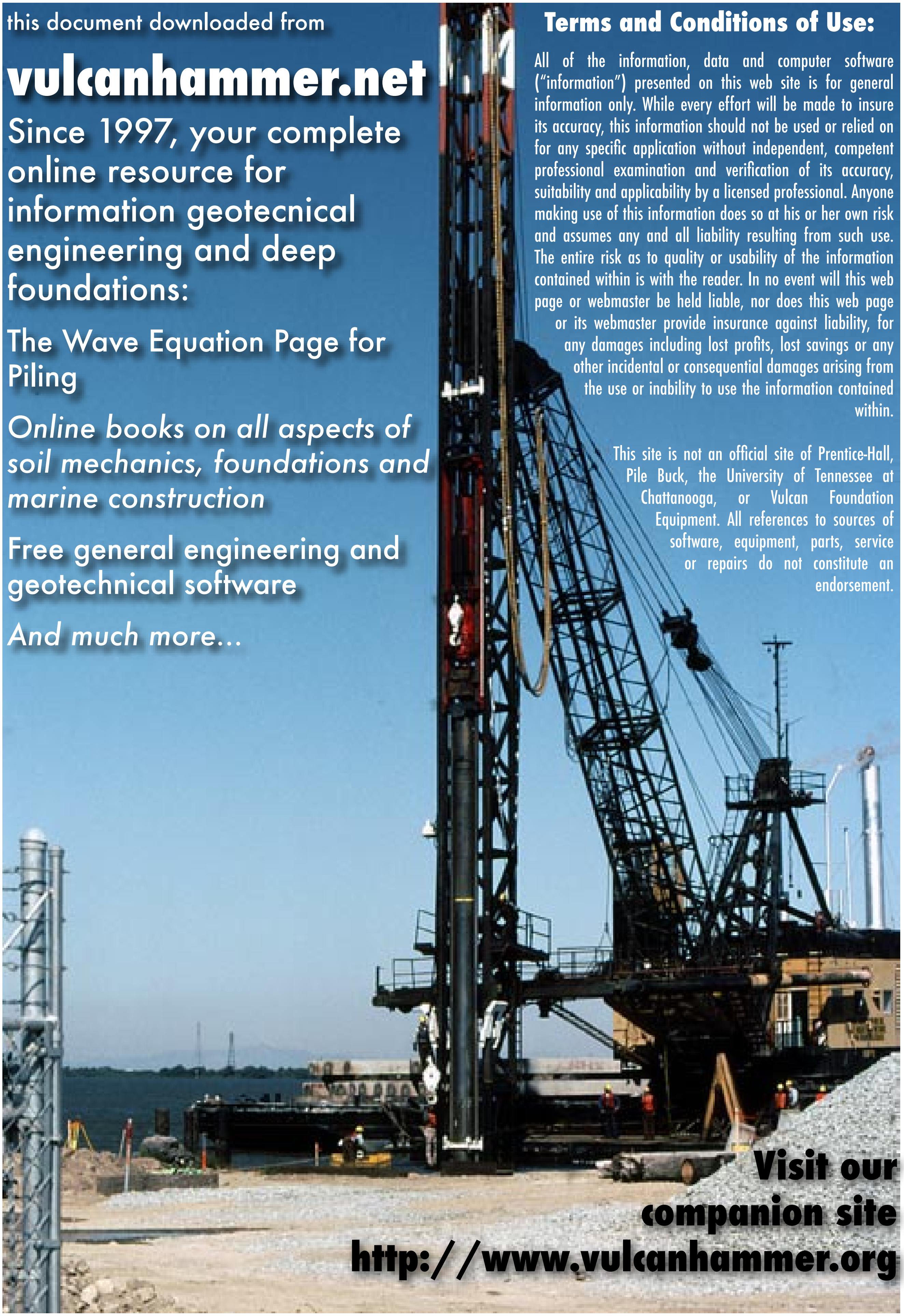
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The behavior of axially loaded drilled shafts which derive most of their resistance to compressive loads from clay-shales is studied. Four instrumented test shafts were loaded to failure using a new type of reaction system in which all the tension steel could be recovered after testing. Three shafts were tested in Montopolis near Austin, and one shaft was tested in Dallas. On the basis of detailed analyses of field data as well as laboratory and field evaluation of the shear strength of soils, the load transferred to the clay-shale has been correlated to the shear strength and in-situ dynamic penetration resistance of clay-shales. A design procedure, with indications of its limitations, has been suggested for computing axial capacity with drilled shafts in clay-shales.

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BEHAVIOR OF AXIALLY LOADED DRILLED SHAFTS IN CLAY-SHALES

by

Ravi P. Aurora
Lymon C. Reese

Research Report 176-4

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

conducted for

Texas
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PREFACE

This report is the fourth report on the findings of Research Project 3-5-72-176, "The Behavior of Drilled Shafts."

This report presents the results of an investigation of the behavior of axially loaded drilled shafts in clay-shales. The study is based on the results of load tests on four instrumented drilled shafts installed in clay-shales by different construction methods. Two shafts were built by the casing method, one by the slurry displacement method, and one by the dry method. This report suggests a procedure for designing axially loaded drilled shafts for use in clay-shales with due consideration of the construction method. The procedure is suggested on the basis of evaluation and analysis of field data acquired by testing instrumented drilled shafts to failure.

The authors would like to acknowledge the work of several dedicated individuals who contributed to this report. The field work was completed with the technical assistance of Messrs. Jim Anagnos, Harold Dalrymple, Gerardo W. Quiros, the late Terrance L. Bowman, and John E. Joerns; Messrs. Horace Hoy and Chet Safe of the Texas Highway Department helped during the planning and construction phases of the project. The Farmer Foundation Company of Houston, and Martin and Martin Foundation Drilling Contractors of Dallas, made financial contributions to the project and were helpful in other ways. Brown & Root of Houston made a contribution of the special a-daptor used in the reaction system. The cooperation and contributions of the above individuals and companies are gratefully acknowledged.

The authors would like also to gratefully acknowledge the support of the Federal Highway Administration.

ABSTRACT

The behavior of axially loaded drilled shafts which derive most of their resistance to compressive loads from clay-shales are studied. Four instrumented test shafts were loaded to failure using a new type of reaction system in which all the tension steel could be recovered after testing. Three shafts were tested in Montopolis near Austin, and one shaft was tested in Dallas. On the basis of detailed analyses of field data as well as laboratory and field evaluation of the shear strength of soils, the load transferred to the clay-shale has been correlated to the shear strength and in-situ dynamic penetration resistance of clay-shales. A design procedure, with indications of its limitations, has been suggested for computing axial capacity of drilled shafts in clay-shales.

KEY WORDS: drilled shafts, axially loaded, clay-shales, instrumented, testing, tension steel, design procedure.

SUMMARY

This study is concerned with the behavior of axially loaded drilled shafts in clay-shales. Four instrumented drilled shafts were installed in clay-shale using different methods of construction. Three shafts were tested at Montopolis, near Austin; they were built using the dry method, the casing method, and the slurry displacement method. One shaft was build in Dallas according to the casing method without use of any slurry. All the test shafts had total penetration of less than 30 ft and a penetration of about 5 ft in the clay-shale. A new reaction system was devised by which all the tension steel in the anchor shafts could be recovered after testing.

From this study it was found that:

(1) the dry method of construction gives the highest load transfer as well as the highest base resistance. The slurry displacement method and the casing method give about the same response under axial loading.

(2) The reaction system used in this test program worked very satisfactorily and was highly economical.

Based on the findings of this study a design procedure was suggested for shafts penetrating about 5 ft into clay-shale and having a total penetration of under 30 ft.

IMPLEMENTATION STATEMENT

This study presents a procedure for the design of axially loaded drilled shafts in clay-shales for various methods of construction. A new reaction system has also been developed during this study. The design procedure as well as the reaction system is recommended for immediate implementation to achieve economy in the design of axially loaded drilled shafts in clay-shales and in performing load tests on axially loaded members such as drilled shafts and piles.

The suggested design procedure is limited to shafts penetrating about 5 ft into clay-shale and having a total penetration of no more than 30 ft. It is, therefore, recommended that further field studies be carried out on instrumented shafts of different lengths. In order to ensure economy and safety, a procedure to estimate the in situ shear strength of clay-shales should be established.

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CHAPTER 1. INTRODUCTION

During the past few decades the acceptance of drilled shafts as a foundation element has rapidly increased because they have become very economical on the basis of cost per ton of load to be supported. This strong competitive position of drilled shafts is due to many reasons. Noteworthy among the reasons is the development of construction procedures to build the drilled shafts in conformity with the proposed design and specifications.

Drilled shafts are frequently used to support bridges, high-rise buildings, elevated expressways, waterfront structures, and machine foundations. They are even used with tie backs as earth retaining walls for deep excavations. In many cases, drilled shafts offer a cost-effective and safe solution to high-capacity foundation needs. Yet, more often than not, their design is based upon empirical, or semi-empirical, procedures. Such practices can lead to under-utilization of the load-carrying capacity of a drilled shaft. On the other hand, an unsafe design may also result from the same approach.

It is, therefore, essential to understand how drilled shafts interact with the supporting soil. In this regard, the past few years have seen a flurry of research by government agencies, drilled shaft contractors, consulting engineers, and research-oriented institutions. Most of the research has been directed towards establishing design criteria for various subsurface conditions.

The study described herein, sponsored by the Texas Highway Department and the Federal Highway Administration, is an attempt to fill the gap of information on the behavior of drilled shafts which derive a significant portion of their load-carrying capacity from end resistance and side-friction in shales. Shales are a predominant geologic formation in the Central and West Texas regions. They occur at relatively shallow depths in many parts of the state. Generally, they possess high shearing strength, and are widely used as the principal load-bearing stratum for heavy structures.

Purposes of This Study

The main purposes of this study are to

(1) Understand the mechanics of load transfer for drilled shafts in shales.

(2) Determine the shear strength characteristics of the shales encountered at two different test sites in Central Texas, namely, Montopolis near Austin, and Dallas.

(3) Study the effects of construction techniques on the load transfer response of shales.

(4) Suggest a rational approach for design of drilled shafts installed in shales.

(5) Identify additional areas of potentially useful research concerning drilled shafts in shales.

General Description of a Drilled Shaft

A drilled shaft is a cast-in-place concrete element, installed fully or partly below ground. It is usually cylindrical in shape throughout its length. Sometimes, it is built with an enlarged base which is monolithic with the cylindrical stem above. The drilled shaft may or may not need reinforcing steel, depending upon design and other considerations. It may be vertical or on batter. If it is properly designed, it can resist forces and moments from any direction. Its circular cross section provides equal section modulus about any axis normal to the length of the shaft. Most commonly, a drilled shaft is used to resist axial compression loads, although it is also used to resist axial tension, horizontal thrust, and overturning moments. Figure 1.1 shows a typical drilled shaft used to resist axial compression load.

The enlarged base of a drilled shaft is referred to as a bell bottom. A shaft with bell bottom is called a belled shaft or a belled-bottom shaft. A shaft without bell bottom is called a straight shaft. The terms drilled caisson, large diameter bored pile, and drilled pier are synonymous with drilled shaft. Other equivalent terms may also exist. In this study the term drilled shaft shall be used to signify a straight shaft.

Status of Information on Drilled Shafts in Shales

At the present time, published data on the behavior of drilled shafts in shales, are rather scarce. O'Neill and Reese (1970), in their state of the

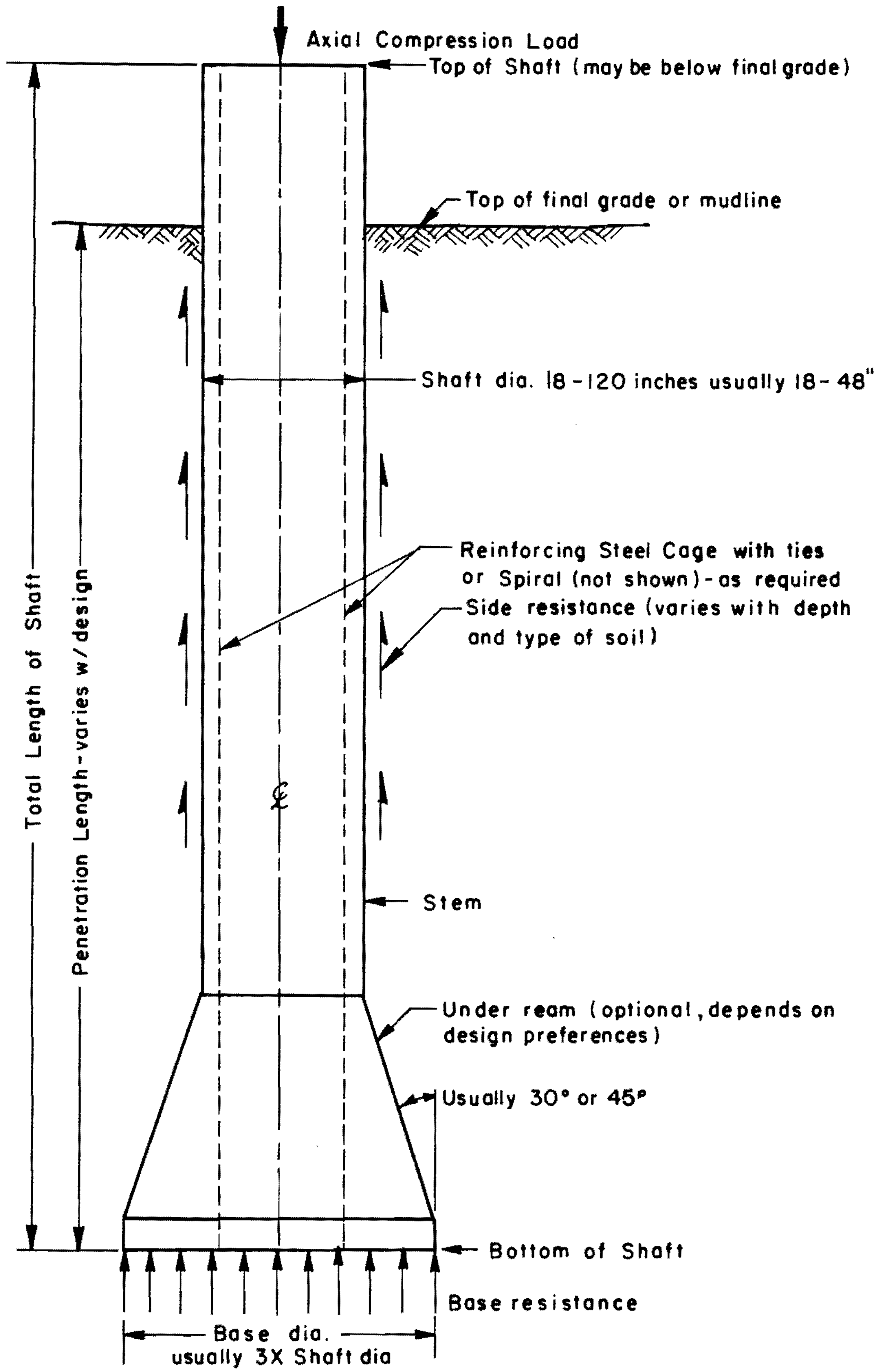


Fig 1.1. Typical drilled shaft to resist axial compression load.

art report, have tabulated research work done on drilled shafts from 1951 to 1969. Further information in regard to the load transfer characteristics of drilled shafts in shales were not found upon additional literature survey. Vijayvergiya, Hudson, and Reese (1969) reported load tests performed on a 30-in. diameter, 28 ft 6 in. long drilled shaft embedded about 10 ft into a clay-shale near San Antonio, Texas. The soil profile at the test site consisted of fat clay to 18 ft underlain by clay-shale. Due to sampling difficulties, the shear strength of the clay-shale was not determined. Although the Texas Highway Department cone penetration tests were performed in the field, erratic variation in dynamic resistance was noted throughout the shale stratum. However, results of load transfer versus movement at two elevations within the shale layer were reported, on the basis of data obtained from full-scale load tests on the instrumented shaft.

The general lack of information on drilled shafts in shales is, in part, due to certain difficulties peculiar to this type of research as discussed below.

Difficulties Involved With Studies on Drilled Shafts in Shales

It is desirable to obtain the following information in a study on the behavior of drilled shafts:

(1) plunging or failure load for the shaft,
(2) shear strength characteristics of the surrounding soil, and
(3) load transfer pattern along the sides and tip of the shaft.

Shales are relatively stronger than most soils encountered in nature. Consequently, the failure loads for drilled shafts in shales are rather high, and failure loads in excess of 500 tons for a single shaft, about 30-in. diameter, can be expected in many cases. Such high failure loads call for special jacks, reaction beams, and anchoring systems, which can be very expensive. Besides, the procurement, installation, and handling of the special equipment can cause delays. Therefore, some investigators have chosen to make small-scale studies in lieu of full-scale field tests on well-instrumented drilled shafts.

The evaluation of in-situ shear strength of soils, in general, is a difficult task. Completely satisfactory procedures to ascertain the shear strength of a soil in its undisturbed natural state are not available. In the case of stiff clays, shales, and clay-shales, the following major difficulties are encountered in accurately determining the in-situ shear strength of the soil:

(1) Undisturbed samples cannot be obtained, because the in-situ material has to be drilled to get any sample at all. This causes substantial disturbance.

(2) As soon as the sample is removed from its natural state, the release of confining pressures causes micro- and macro-fissures to form. Due to these fissures, the shear strength measured by laboratory tests may be significantly lower than the actual in-situ shear strength. The extent of strength variation due to fissures cannot be precisely determined.

(3) Progressive changes in soil properties, as well as further increase in the amount of fissures and cracks, occur during storage of samples.

The information on load transfer along the sides and tip of a drilled shaft is usually obtained by use of electrical measurement systems. Some difficulties are encountered with the use of electric systems employed in drilled shafts. Migration of water into electrical circuits, malfunction of strain gages, voids or cavities in concrete near the load-measuring cells, irregularities in the cross section of the shaft, and local crushing of concrete at some points in the shaft, are the chief causes of inaccurate load measurements.

The methods of minimizing possible errors due to the above mentioned difficulties will be discussed in later chapters.

CHAPTER 2. REVIEW OF PREVIOUS STUDIES

Types of Studies

Research done on axially loaded foundation elements can be divided into three main categories:

(1) theoretical analyses,

(2) laboratory model tests, and

(3) field tests.

The above categories of research are mutually complementary. Hence, individual research projects conducted in the past usually included more than one of the above categories of investigations.

Theoretical analyses are based upon mathematical treatment of an idealized model of the actual member and its surrounding soil. Idealization is done only to make the problem amenable to analysis. Thus, the results of such a study cannot be expected to predict the actual behavior of an axially loaded member. However, these results can be used to plan model tests as well as field tests. Moreover, theoretical studies lead to a better understanding of the important physical phenomena.

Laboratory model tests are done on geometrically similar but smaller versions of full-sized members. They are very economical compared to field tests, and can yield useful information, if properly planned and supervised. Their main limitation lies in their inability to simulate in situ geotechnical conditions, construction procedures, and effects of field conditions on the quality of the in-place concrete.

Field tests are run on geometrically smaller or actual size piles or shafts. Data obtained from these tests are most representative because the actual construction and site conditions are simulated throughout the test. However, they are difficult to manage and are always very expensive. Typically, one test on an instrumented drilled shaft can cost anywhere from 10 to 15 thousand dollars at the present time (1975). Besides, highly specialized field and office personnel are required for these tests. In view of the useful information they furnish, field tests are still considered highly cost effective by a vast majority of engineers and research workers.

Review of Theoretical Studies

Theoretical methods of analyzing axially loaded foundation elements can be broadly classified into three major groups:

(1) Elastic solutions using discretised pile elements.

(2) Numerical solutions using discretised pile and soil elements.

(3) Semi-theoretical methods based upon known or estimated relationships between pile resistance and pile movement at various depths along the pile-soil interface.

Generally, theoretical methods are applicable to piles as well as drilled shafts. Consequently, during the review of theoretical studies the terms pile and drilled shaft will be used interchangeably.

(1) Elastic Solutions Using Discretised Pile Elements: Mindlin (1936) derived equations for stresses and strains due to point loads acting beneath the surface of a semi-infinite elastic medium. His equations formed the basis of analytical studies by various investigators who followed the idea of discretising only the pile, not the soil, into elements as discussed further. The pile is divided into a number of discrete elements interconnected at nodal points. Pile displacements are obtained by considering stiffness of each pile element with respect to axial loading. Stresses and displacements within the soil mass are computed from Mindlin's equations assuming forces on pile elements concentrated at nodal points.

D'Appolonia and Romualdi (1963) used this approach to determine theoretical values of load transfer in end-bearing steel H-piles. They compared these values with actual measurements made on two instrumented piles. These two piles, 14BP89 and 14BP117, were driven through 40 feet of sand and gravel into weathered silty shale. Another study was done by Thurman and D'Appolonia (1965). The investigators of these two studies concluded, on the basis of theoretical computations and instrumented load tests, that methods employing theories of elasticity can be used to predict pile movement and load transfer. They pointed out that the accuracy of the computed results was greatly affected by three factors: the coefficient of lateral earth pressure, the modulus of elasticity of the soil, and the elastic-plastic tip movement.

Mindlin's equations were also used by Salas and Belzunce (1965) who reported an approach to solve for stresses along a loaded pile taking negative friction into account. They assumed that the soil medium could be represented by a Bossinesq half-space and that the pile was a line. They suggested the solution of their integral equation by electronic computer.

Poulos and Davis (1968) used Mindlin's equations to determine the settlement behavior of a single axially loaded incompressible cylindrical pile in an ideal elastic soil mass. They considered the pile as a number of uniformly loaded cylindrical elements together with a uniformly loaded circular base. Their analyses yielded solutions for the distribution of shear stresses along the pile and the displacements of various segments of the pile. They further extended the analyses to a single pile with an enlarged base and showed that, theoretically, an enlarged base had major significance only for relatively short piles. Mattes and Poulos (1969) published results of a similar study on single compressible piles.

Poulos and Mattes (1969) proposed analytical methods to study the behavior of axially loaded compressible single end-bearing piles in an elastic soil medium. Their results showed good agreement with field data reported by others. They concluded that the behavior of an end-bearing pile is largely influenced by the length to diameter ratio, stiffness of the pile relative to the surrounding soil and relative stiffness of the bearing stratum.

Mattes (1969) showed that for the settlement behavior of incompressible piles the consideration of radial compatibility was an unnecessary complication, involving a large amount of computer time compared with that required when radial compatibility is ignored. Butterfield and Bannerjee (1970), (1971) confirmed the observations made by Mattes (1969) with regard to the need for consideration of radial compatibility for compressible piles as well. Their study also included the use of elastic analysis to predict load-settlement behavior of straight and underreamed piles in groups.

Poulos and Davis (1974) presented useful elastic solutions to determine load transfer, pile and soil movement for a single pile or a pile group. Their plots consider cases of end-bearing piles and floating piles as well as compressible and incompressible piles. Their work represents the state of the art of applications of analytical methods based on Mindlin's equations and theory of elasticity.

O'Neill and Reese (1970) have presented an excellent summary of theoretical and semi-theoretical methods to predict the settlement of a drilled shaft.

(2) Numerical Solutions Using Discretised Pile and Soil Elements: During the past decade significant advances have been made in the field of electronic computations, and it is now possible to achieve at a nominal cost rapid solutions of problems involving enormous amounts of computations. These advances have led to the introduction of numerical methods using discretised elements in both the pile and the surrounding soil. Commonly, these methods are referred to as Finite Element Methods. The Finite Element Methods permit mathematical representation of important soil characteristics such as anisotropy, nonlinearity, nonhomogeneity, and time-dependent stress-strain behavior. These methods can be used satisfactorily only if the soil properties and geotechnical conditions can be determined accurately. Reference may be made to Zienkiewicz (1967) and Desai and Abel (1972) for presentations of the formulations using the Finite Element Methods.

Due to its recent development, the Finite Element Method has not been utilized significantly in the area of drilled shafts or piles. Ellison, D'Appolonia and Thiers (1971) made a comprehensive study on five instrumented drilled shafts ranging in diameter from 25 in. to 37 in. and in length from 30 ft 6 in. to 50 ft. They applied Finite Element Method analysis to predict butt movement, tip movement, load distribution, ultimate adhesion and ultimate tip load for each shaft. They compared the predicted values with observed values. Their conclusions indicate that the Finite Element Method can be used to predict accurately the load capacity and load-deformation behavior of drilled shafts.

Desai (1974) published further data comparing actual load-settlement and transfer measurements with values predicted by analysis using the Finite Element Method for three pipe piles, ranging in diameter from 16 in. to 20 in., approximately 55 ft long, installed in sandy soils at two locations in the Mississippi Valley Region. His study shows that, with proper simulation of soil properties, useful results can be obtained by the Finite Element Method.

It can be stated that, at the present time, the full potential of the Finite Element Method has not been realized due to lack of understanding of soil properties as well as inability to simulate relevant in-situ geotechnical conditions.

(3) Semi-Theoretical Methods: Coyle and Reese (1966) proposed a semi-theoretical approach to predict load-settlement curves for axially loaded members. The method uses load transfer data obtained from full-scale field tests or laboratory model tests on instrumented piles or drilled shafts. The data are presented in the form of curves showing α, the ratio of load transfer to the shear strength (quick) of soil, as a function of the movement of the pile. Such curves are given for different depths along the pile length. A method is presented to obtain tip load versus tip movement curves. Conceptually, the steps in the method are as follows:

(1) Divide the pile into sufficiently short segments.

(2) Assume a small pile tip movement.

(3) Determine tip load corresponding to the tip movement from the applicable tip load versus tip movement curve.

(4) Assume movement at midpoint of first pile segment (counting segment numbers from pile tip up) equal to movement of the tip. Find the load transfer, in side friction, from the load transfer curve applicable for the depth of the midpoint of segment. Linear interpolation between load transfer curves of adjacent depths is done if a load transfer curve at a particular depth is not available.

(5) Compute load acting in side friction on the segment.

(6) Compute load at top of segment by adding load found in (5) to the known load acting at the bottom of the segment.

(7) Determine load acting at midpoint of the segment, using a suitable variation of load along the length of the segment.

(8) Determine elastic compression at midpoint of the segment. For this purpose, assume a concentrated load, equal to the average of loads at the tip and midpoint of the segment, acting over half the segment length.

(9) Compute new movement of the midpoint of the segment by adding the quantities obtained in (4) and (8).

(10) Compute the difference between movements found in (9) and (4). Check if this difference is less than an acceptable specified value.

(11) Repeat steps (4) through (10) until the requirement for acceptably specified value mentioned in (10) is met. In every new iteration set movement in (4) equal to movement found in (9) in the preceding iteration.

(12) Set load at the tip of the next segment above equal to load found in (6).

(13) Consider the next segment above and assume the movement at the midpoint of this segment equal to the movement of the top of the segment next below. Repeat steps (4) through (11) for this segment.

(14) Repeat step (13) until the top segment of the pile has been considered and load and settlement on top of pile for the tip movement assumed in (2) have been determined.

(15) Repeat steps (2) through (14) until all specified pile tip movements have been considered.

The above approach is easily adaptable to computer analysis. The main appeal of this method lies in predicting a load-settlement curve based on available soil information at minimal cost. Load transfer curves and tip load versus tip movement curves are needed to use this technique. Seed and Reese (1957) first developed load transfer curves. Reference is made to O'Neill and Reese (1970) for a review of work done in the area of available information with regard to load transfer curves. This semi-theoretical method has found acceptance among many practicing engineers and contractors due to its simplicity and practicality. It is to be noted that in this approach the shear stresses at one point are assumed unaffected by stresses elsewhere around the pile. This limitation may require further refinements in isolated cases.

Review of Laboratory Model Tests

Published information on laboratory model tests simulating the construction and loading sequence of drilled shafts does not exist to date. Experiments on model piles with enlarged bases were conducted at the Building Research Station in England by Cooke and Whitaker (1961). One of the principal objectives of that study was to determine the proportions of the load carried by the sides and by the base. The models, which were of brass, had a shaft diameter of 0.75 in. The shaft length varied from 12.5 in. to 48.0 in. while the base diameter varied from 0.75 in. to 3.0 in. Load was measured at the top and base of the shaft and the tests were made by the constant rate of penetration method. This study showed that the resistance due to side friction was mobilized at very small penetration movements, about 0.5 percent of the shaft diameter, while penetrations of the order of 10 to 15 percent of the base diameter were required to mobilize the ultimate bearing capacity of the base. The experiments were conducted only for London clay. In all tests, the clay filling was done around the shaft after it had been placed in vertical position inside a container having a 6-in. thick layer of clay at its bottom.

Clisby and Mattose (1971) reported data on laboratory tests conducted on models to determine the relative bearing capacity of the single- and double-bell piles. The model piles consisted of 1-in. diameter steel shafts with a 2-1/2-in. diameter steel bell or bells. The soil medium consisted of oven-dried Yazoo clay with 20 percent petroleum jelly added to replace the moisture and make the clay workable. Double-bells were formed at different spacings along the vertical axis of the shaft. Their results showed that for axial spacing of bells in excess of twice the stem diameter the load carrying capacity stayed constant. This study was done in conjunction with tests on 7 drilled piles of 12-in. diameter with one or two bells which were 24 in. or 36 in. in diameter.

Review of Field Tests

Data on full-scale field tests have been collected only during the past two decades or so. Table 2.1 is a summary of relevant information published up to 1969 relating to full-scale load tests on drilled shafts. This table is from an earlier report by O'Neill and Reese (1970). Table 2.2 summarizes similar information published from 1970 to 1975.

It can be seen from the tables that little is known about the behavior of drilled shafts installed into shales. The study described herein is an attempt to furnish rational information regarding soil-structure interaction in shales with particular reference to drilled shafts subjected to axial loading.

TABLE 2.1. SUMMARY OF FULL-SCALE LOAD TEST RESULTS (UP TO 1969) (FROM O'NEILL AND REESE [1970])

INVESTIGATOR (S) & REFERENCE		Meyerhof & Murdock (1953)	Golder & Leonard (1954)	Harris (1951)	Green (1961)	DuBose (1955, 1956)
LOCATION		Southall and Barnet, London Area, England	Kensal Green, London Area, England	Plum Creek, Houston, Texas	Borehamwood, Hertfordshire, England	College Station, Texas
DATE OF TEST (S)		1950 - 1952	1950 - 1951	1951	1952 - 1955	1953 - 1955
TESTING METHOD		Quick. Shafts Loaded to Failure within Several Hours.	Cyclic with Load Maintained until Equilibrium Reached in Each Cycle. Tests Required 3 Days per Shaft.	Cyclic. Load Maintained Several Days Each Cycle.	Static, Long-term. Design Load Maintained 3 Years, 6 Months. Also Short-term M.L.	Maintained Load Method Most Tests, Required 4-5 Days. Also a Few Pullout and Quick Tests.
SOIL DESCRIPTION		London Clay. Preconsolidated, Fissured CH Material. No Ground Water. Shear Strength Varies from 0.5 tsf at 5' to 2.3 tsf at 25'-30'. Constant Below 30'.	London Clay. Typical. Shear Strength Varies from 0.4 tsf at Top of Clay to 1.9 tsf at 30' Depth.	Beaumont Clay. Firm Yellow-Blue Jointed Clay Interbedded with Waterbearing Silts. Strength not Explicitly Stated but about 1.0 tsf Indicated.	London Clay. Typical. Shear Strength about 1.1 tsf in Test Zone as Measured in Unconfined Compression Tests.	Layered Sandy CL and CH Material. Water-bearing Silt at 12'. Avg. Shear Strength of 0.5 tsf along Sides
NUMBER AND SIZE OF SHAFTS	STRAIGHT	11: 12"-14"φ, 20'-40' Depth.	3: 18"-24"φ, 23'-34.6' Depth.	1: 20"φ, 44' Depth.	8: 12"-14"φ, 10' Depth.	30: 6"-23"φ, 6'-21' Effective Depth.
	BELLED	0	0	0	0	5: 15"φ Stem, 3'φ Bell, 10'-19' Effective Depth
CONSTRUCTION PROCEDURE		Boreholes Drilled with Both Derrick-type Hand Augers and Lorry-mounted Power Augers. Dry Process. Required 1-5 Days to Install Each Shaft with Hand Augers. 45 Minutes - 4 Hours with Power Augers.	Presumably with Power Augers. Plate Loading Tests Conducted at Several Levels in Boreholes. Hence, Holes Presumably Open (but Cased) for Periods of Time Longer than Normal.	Power Auger with Casing. No Mud. Sides of Hole Wet. Considerable Sloughing. 4' Water in Bottom of Borehole. Installation Presumably Completed in One Day	Mechanical Auger to 6'. No Mud Used. Hand Auger to Finished Depth (10'). Three Shafts Cast in Wet Holes.	All 6" and 7" Shafts Hand Augered. Others Installed with Power Augers. Dry Process in Each Case.
INSTRUMENTED ? IF SO, HOW ?		No Side Resistance Deduced from Bearing Capacity Equation	No Side Resistance Deduced from Bearing Capacity Equation	No. Side Resistance Deduced from Bearing Capacity Equation	No.	Yes. Load Cells Used at Base of 6 Straight Shafts. SR-4 Gages Used on Re Bars of Several Shafts. Some Stability Problems with SR-4 Gages on Re Bars
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		0.21-0.42 at Southall. Water-Cement Ratio = 0.4. 0.4-0.6 at Barnet. Water-Cement Ratio = 0.2. (Average of UU Triaxial and Unconfined Tests)	0.64-0.74 as referred to Average Soil Strength 1.0 as Referred to Envelope of Minimum Soil Strengths (Triaxial Compression Tests)	0.6 at 1/4" Gross Settlement Computed by Harris. Probably High, but α Not Less Than 0.5 (Average of Triaxial Tests)	Not Measured, but 70%-100% of Short-term Value Indicated from Load Settlement Curves for Long Term Tests	1.0 (Average of UU THO Triaxial and Unconfined Tests.)
BEARING CAPACITY FACTOR N_c. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Measured in Shafts; but 9.4 Obtained for Plate Loading Tests in Boreholes. (Average of UU Triaxial and Unconfined Tests)	Not Measured in Shafts; but Factors Varying from 9.7 to 16.4 Measured in Plate Loading Tests in Lined Boreholes. (Average of Triaxial Tests Beneath Plates)	Not Measured.	Not Measured	12. Obtained from Shaft with Side Resistance Destroyed (Average of UU THO Triaxial and Unconfined Tests)
SETTLEMENT TO PRODUCE SIDE FAILURE		Not Given.	Not Given.	Not Given	Not Given.	0.02"-0.10" with Larger Values Occurring in Shafts with Larger Diameters
LOAD DISTRIBUTION INFORMATION		None.	None	None.	None.	Good Information Obtained in at Least One Test on Straight Shaft. Approximately Linear Reduction in Load with Depth
TIME EFFECT		No Increase in Resistance with Time up to 18 Months.	Not Given	Not Given	See Below	Not Given.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		1.) 2% to 7% Increase in Moisture Content in Soil within 2 inches of Shaft-Soil Interface at Southall. Higher Increases at Greater Depths. 2.) Also Tested Driven Piles at Barnet Site. Gave Higher Resistance Values by Factor of 2. Lower Values in Shafts Attributed to Softening of Soil by Migration of Water from Concrete.	These Tests Included in Skempton's 1959 Review, Skempton States Shear Strength Values Used by Investigators too Low. Kensal Green α Values Recomputed by Skempton as 0.58-0.60.		After 3 Years 6 Months, All Shafts Loaded to Approximately Twice Design Load. The Measured Settlement for this Loading Increment less than Corresponding Settlement in Short-term Shaft Tests. Indicates Prolonged Loading Not Detrimental to Load-Settlement Behavior.	1.) Good Agreement Between Quick and M.L. Tests for Small Diameter Shafts. Not as Good for Large Diameter Shafts 2.) Extracted Shafts Had 1/4"-1/2" Soil Adhering to Sides 3.) In Model Studies, no Moisture Content Increase in Soil Adjacent to Shafts Except at Very Low Initial Moisture Content. 4.) Pullout Capacity was 49%-79% of Compressive Capacity

(continued)

TABLE 2.1. (CONTINUED).

INVESTIGATOR (S) & REFERENCE	Mohan & Jain (1961)	Mohan & Chandra (1961)	Skempton (1959)	Woodward, Lundgren & Boitano (1961)	Burland, Butler & Dunican (1966)
LOCATION	Jabalpur, India	Poona, Bhopal, Ujjain and Jabalpur, India	Ten Sites in London Area, England	Lemoore, California	Moorfields, London Area, England
DATE OF TEST (S)	1957 – 1959	Late 1950's	1950 – 1959	Not Reported	Not Reported
TESTING METHOD	Short-term Pullout and Compression. Time Increments for Loading not Specified.	Short-term Pullout, Compression and Cyclic. Time Increments for Loading Not Specified	Varied	Short-term M.L.	Short-term M.L. to 1.5 Times Working Load Followed by C.R.P. to Failure
SOIL DESCRIPTION	Black Cotton Soil. Highly Expansive Clay. Shear Strength about 1–2 Tsf.	Black Cotton Soil Highly Expansive Clay. Shear Strength about 0.7 – 1.6 Tsf.	London Clay Typical. Shear Strength Varies from 0.4 tsf at Top of Clay to 2.5 tsf at 50' Depth.	Layered Stiff, Silty and Sandy Clay. Shear Strength about 1.0 tsf.	London Clay Typical Shear Strength about 1 tsf at Surface to 2.0 tsf at 50 Feet
NUMBER AND SIZE OF SHAFTS: STRAIGHT	11: 9"–12"ø, 6'–12' Depth.	45: 6"–12"ø, 5'–12' Depth.	34: 12"–36"ø, 6'–65' Penetration of London Clay	3 (in Clay), 16"ø, 36'–46' Depth	1: 36"ø, 31' Depth
NUMBER AND SIZE OF SHAFTS: BELLED	12: 9"–12"ø Stem, 21"–30" Bell, 6'–12' Depth	0	0	0	2: 36"ø Stem, 72"ø Bell, 21' and 31' Deep
CONSTRUCTION PROCEDURE	Used Hand Spiral Augers and Portable Hand Under-Reamers.	Not Specified.	Varied. Some by Hand and Some by Power Augering.	Mechanical Bucket Rig with Casing. No Mud Used. Shafts Concreted Immediately After Drilling Finished	Mechanically Bored. Straight Shaft Bored and Concreted Same Day. Belled Shafts Concreted Day After Boring
INSTRUMENTED ? IF SO, HOW ?	No. Side Resistance Determined from Pullout Tests and from Compression Tests on Shafts with False Bottoms.	No. Side Resistance Determined from Pullout Tests, Tests on Shafts with False Bottoms and Cyclic Tests Using Van Weele's Method of Separation of Base and Side Loads	No Side Resistance Deduced from Bearing Capacity Equation	No. All Shafts Had False Bottoms to Give Direct Reading of Side Capacity.	No Side Resistance Deduced from Bearing Capacity Equation
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)	0.3 (Average of Unconfined Compression Tests)	0.45 – 0.54 (Soil Tests Not Specified)	0.3 – 0.6, with 0.45 Average (Average of Triaxial Tests)	0.49 – 0.52 (Average of Unconfined Compression Tests)	0.6, Peak; 0.35, Residual for Large-displacement C.R.P. Tests (Average of UU Triaxial Tests)
BEARING CAPACITY FACTOR N_c (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)	6 – 9 for Straight Shafts. 3 – 4 for Belled Shafts. Calculated by Subtracting Shaft Area Times Average Shaft Resistance from Pullout Tests from Ultimate Load. (Average Unconfined Compression)	Not Given	Not Given.	Not Given	Not Given Results of Several Plate Bearing Tests Reported
SETTLEMENT TO PRODUCE SIDE FAILURE	Approximately 0.25".	Not Given	0.4" for two Shafts Constructed with Void Under Base.	Not Given	Approximately 0.25"
LOAD DISTRIBUTION INFORMATION	None.	None	None	None	None
TIME EFFECT	No Long-term Settlement on Shaft Loaded to 1/3 Ultimate Capacity for Two Years	Retesting after Lapse of One Year Showed No Increase in Frictional Resistance.	No Significant Change in Capacity on Retesting Several Shafts.	Not Given	Not Given.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.	Low N_c Values Calculated for Belled Shafts May Be Due to Reduced Side Resistance instead of Reduced Base Capacity.	1.) Friction Tests Between Soil and Dry Concrete Gave Coefficient of Friction of 0.75 at 1 Tsf Normal Pressure. 2.) Measured Radial Moisture Gradients Adjacent to Some Piles. 2–3 % Increase in Moisture Content in 2" Nearest Shaft. Greatest Increase Near Bottom.	1.) 1 Tsf Recommended as Limit for Side Resistance 2.) Low α Factors Associated with Sites where Construction was Slow and where Water Caused Deterioration of Sides of Borehole.	Driven Pipe Piles of Slightly Smaller Diameter Gave α Factors of about 0.6 in Same Soil. Other Shaft Tests Performed in Sandy Soil.	1.) Authors Conclude Residual Value of α Should be Used in Design of Belled Shafts. 2.) Authors also Report Results of Tests on 4 Large Piers by Darrington in which $\alpha > 0.5$ for 2 Straight Shafts and = 0.3 for 2 Belled Shafts

(continued)

TABLE 2.1. (CONTINUED).

INVESTIGATOR (S) & REFERENCE		Burland (1963)	Frishmann & Fleming (1962)	Fleming & Salter (1962)	Williams & Colman (1965)	Whitaker & Cooke (1966)
LOCATION		London, England	Blackfriars Road; St. Giles Circus; London, England	Cromwell Rd., London, England	Great St. Helens, London, England	Wembley, Middlesex, London, England
DATE OF TEST (S)		Not Reported	Not Reported	Not Reported	1962 – 1963	1962 – 1963
TESTING METHOD		Short-term M.L. and C.R.P.	Maintained Load.	Maintained Load	Maintained Load and Quick Tests.	Maintained Load to 70% Ultimate Load. C.R.P. to Failure
SOIL DESCRIPTION		London Clay. Shear Strength Profile Not Given.	London Clay. Blackfriars Site Silty. Avg. Shear Strength along Sides = 1.2 tsf (Blackfriars), 1.0 tsf (St. Giles). Avg. Shear Strength Beneath Base = 2.0 tsf (Blackfriars), 2.0 tsf (St. Giles).	London Clay. Shear Strength Varied from 0.9 tsf at Top of Clay to 2.4 tsf at 37 feet Penetration.	London Clay. Typical Strength Profile. Minimum Strength Envelope Varies from 1 to 2 Tsf.	London Clay Typical. Shear Strength Variation 0.7 tsf at Surface of Clay to 1.6 tsf at 60' Depth
NUMBER AND SIZE OF SHAFTS	STRAIGHT	2 : (Diaphragms) 4' x 1.8', 40' Depth	0	0	See Remarks	6: 2'-3' φ, 30.5'-50' Penetration of Clay.
	BELLED	0	2 : 2'-6" and 3'-0" φ Stems, 5'-0" and 8'-1" φ Bells, 50' Penetration of Clay.	1: 4' φ Stems, 10'-6" φ Bell, 35' Depth.	See Remarks	7 : 2'-3' φ Stems, 4'-6' Bells, 28'-48' Penetration of Clay
CONSTRUCTION PROCEDURE		Each Element Excavated with Grab. One Diaphragm installed Dry. One installed using Bentonite Slurry. Each element Took Two Days to Install. Both Cured Three Weeks Before Test.	Mechanically Bored. Blackfriars Concreted Immediately. Borehole Open Three Days at St. Giles. Dry Process.	Mechanical Auger.	Hand Excavated 6'. Open Shaft to 124' Depth. Hole Mostly Dry Except in One Zone.	Calweld Power Equipment, using Normal Good Practice. Bases of Belled Shafts Cleaned by Hand. Most Shafts Open One Day or Less
INSTRUMENTED ? IF SO, HOW ?		No.	Yes. St. Giles Shaft Instrumented with Flatjack at Top of Bell, Foil-type Strain-Gages on Vertical Steel, Mechanical Strain Wires. Flatjack and Strain Wires Performed Well.	No.	No.	Yes. Electrical Load Cells Placed at Top of Bells or Bottom of Straight Shafts.
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Given. Element Installed with Bentonite Slurry Had Nearly Identical Load Settlement Curve as Dry Element. Approximately Same α Factor Implied for Each.	Approximately 0.2 at Blackfriars and 0.33 at St. Giles. Low Value at Blackfriars Attributed to Higher Silt Content. (Soil Test Procedure Not Given)	Not Tested to Failure, but α Probably about 0.6 (Soil Test Procedure Not Given)	1.0 when Sides of Borehole are Dry (Minimum Envelope of UU Triaxial Tests)	0.44 - All Shafts. Little Variation with Depth or with Size of Base (Average of UU Triaxial Tests)
BEARING CAPACITY FACTOR N_c. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Given	Not Obtained at Blackfriars. St. Giles Shaft not Completely Failed.	Not Given.	9 for Bearing Tests on Blocks, Occurring at Settlement of 5% of Base Diameter in Quick Tests (Minimum Envelope of UU Triaxial Tests)	6.75 - All Shafts Mobilized at Settlement of 10%-20% of Base Diameter. (Average UU Triaxial) 9.0 (Minimum Envelope of UU Triaxial Tests)
SETTLEMENT TO PRODUCE SIDE FAILURE		Approximately 0.2" for Both Dry and Bentonite-coated Elements.	Not Given.	Not Given.	0.05"- 0.1" Where Soil Dry Behind Grout. 0.5" Where Soil Wet Behind Grout	0.15" for 2' φ Shafts to 0.30" for 3' φ Shafts. Little Difference Between Belled and Unbelled Shafts.
LOAD DISTRIBUTION INFORMATION		None.	Not Reported	None	None.	None
TIME EFFECT		Not Given.	Not Given.	Not Given.	Not Given.	Approximately 12% Increase in Side Resistance in One Year Based on Pull Tests of Anchor Shafts. Increase Proportional to Logarithm of Time.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		1.) Generally 3%-4% Increase in Moisture Content in Soil Near Both Elements. Extent of Moisture Migration was 2-3 inches. 2.) Using Skempton's Shear Strength Profile for London Clay and Bearing Capacity Equation, α was Approximately 0.6.	Strain Gages Experienced some Instability, but did Show Decreasing Load with Depth. Numerical Results Not Given.		1.) Jacked Between Base Blocks and Sections of Precast Concrete Shaft Liner Grouted to Soil to Obtain Frictional Resistance and Base Capacity at Several Levels. 2.) "Immediate Settlement" Achieved in 2 Minutes for the Liner.	1.) Side Resistance Developed Was Greater Than Fully Softened Shear Strength of Soil. 2.) Proposed Design Methods Presented.

(continued)

TABLE 2.1. (CONTINUED).

INVESTIGATOR (S) & REFERENCE		Deb & Chandra (1964)	Matich & Kozicki (1967)	Komornik & Wiseman (1967)	Van Doren, Hazard, Stallings & Schnacke (1967)
LOCATION		Jabalpur, Ujjain, Poona and Indore, India	Brookfield, Nova Scotia	North Tel Aviv, Israel	Wichita, Kansas
DATE OF TEST (S)		Not Reported	1964	Not Reported	1966
TESTING METHOD		Long-term ($2^1/_2$ years) Maintained Load and Short Term Maintained Load	Maintained Load Compression on Unrifled Shafts, M L Pull-out on Rifled Shafts	Cyclic	Maintained Load
SOIL DESCRIPTION		Black Cotton Soil. Highly Expansive Clay. Shear Strength about 1-2 tsf. Zone of Seasonal Moisture Change to 12' Depth	Dense Glacial Till (CL) Approximately 30' Thick Overlying Weathered Shale. Average Shear Strength; for Till, 1.0 tsf; for Shale, 2.5 tsf	Slightly Sandy Fat Clay Along Sides. Some Sand. Base Founded in Cemented Yellow Sand. Average Shear Strength of Clay 1.5 tsf	Silt, Sand and Silty Clay Overburden 25 Feet Thick Overlying Weathered-to-Intact Shale and Gypsum. Shear Strength of Shale Was 1-10 tsf. No Shear Strength Tests in Overburden
NUMBER AND SIZE OF SHAFTS	STRAIGHT	6: 9"-12" ϕ, 6'-12' Depth	4: 2 Smooth Sided, 52" ϕ, 40' Depth in Till and Shale. 2 Rifled, 24" ϕ, One 20' in Till, One 13'6" in Shale	1: 26" ϕ, 63' Depth	5: 30" ϕ, 44'-74' Depth
	BELLED	7: 8"-12" ϕ, 25"-35" Bells, 12' Depth, One Double Bell	0	0	0
CONSTRUCTION PROCEDURE		Method Not Specified. Presumably Portable Hand Auger	Mechanical Boring. Two Shafts Had Rifled Boreholes with a Spiral Groove 3" Deep with 18" Pitch	Benoto Method	Mechanical Auger. Borehole Through Overburden Processed with Bentonite Slurry, Cased and Bailed Out. Slurry Left in Holes Overnight. Drilling into Shale Accomplished in the Dry
INSTRUMENTED ? IF SO, HOW ?		No	No	No.	Yes. Hydraulic Pressure Cells and Borehole Extensometers at Different Levels
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Approximately 0.5 - All Shafts (Soil Tests Not Specified)	Unrifled: Tests Not Conducted to Failure, but α about 0.5 in Till Based on Shape of Load Settlement Curve. Rifled: Approximately 1.0 on Basis of 24" Shaft Diameter in Both Shale and Till (Average of UU Unconfined and Triaxial Tests)	Complete Failure Not Achieved, but at Least 500 psf Developed in Side Friction in Clay. Thus $\alpha > 0.17$ (Average of Vane Shear Tests)	Complete Failure Not Achieved, but Maximum of 2000 psf Side Friction Indicated in Overburden Material. Shear Failure Not Achieved in Shale
BEARING CAPACITY FACTOR N_c (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)		Not Given	Not Obtained in Tests	Not Obtained in Tests	Not Obtained in Tests
SETTLEMENT TO PRODUCE SIDE FAILURE		Not Given.	Complete Failure Not Quite Achieved in Pullout Tests in Rifled Shafts at 0.8"-1.0" Movement	About 0.1" Implied from Load Settlement Curve	Not Given
LOAD DISTRIBUTION INFORMATION		None	None	None	Load Decreased Somewhat Erratically with Depth
TIME EFFECT		Settlements Stabilized in 1-5 Months at 1.0-1.5 Times Working Load. Long-Term Shear Deformation of About 0.03 Inches at 0.5 Times Ultimate Load.	Not Given	Not Given	Not Given
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.		Comparison of Normal Belled Shafts with Belled Shafts with Side Friction Destroyed Indicated that Side Friction is Essentially Constant with Time at Loads of 0.3-0.6 of Ultimate in Long Term Tests.		Authors Discuss Shaft Installed in Sand at Nearby Site with Bentonite Slurry Technique. Test Results Indicated High Side Resistance.	1) No Evidence of Plane of Weakness Between Shaft and Soil in Overburden where Slurry Was Used. 2) Evidence Some "Balling" May Have Occurred at Interface Between Overburden and Shale

(continued)

TABLE 2.1. (CONTINUED).

INVESTIGATOR (S) & REFERENCE	U. S. Army Engineer District, Fort Worth, Texas (1968)	Reese & Hudson (1968)	Bhanot (1968)	Watt, Kurfurst, & Zeman (1969)	Reese, Hudson & Vijayvergiya (1969)
LOCATION	Lackland Air Force Base, San Antonio, Texas	Austin, Texas	Edmonton, Alberta	Saskatoon, Saskatchewan	San Antonio, Texas
DATE OF TEST (S)	1966-1967	1966-1967	1967	1967	1967-1968
TESTING METHOD	Short and Long Term Maintained Load.	Quick Tests.	Maintained Load. Load Cycled for Shafts in Till.	Equilibrium Method.	Quick Tests.
SOIL DESCRIPTION	13 Feet of CH and GC material Overlying Jointed Shale. Shale and Overburden Highly Expansive. Contain 75 % Montmorillonite. Average Shear Strength of Overburden 1 tsf; of Shale (In Test Zone) 5 tsf.	Fat Clay with Calcareous Material to 6' underlain by Lean Clay. Overconsolidated. Shear Strength Averaging 2.5 tsf. No Water.	Top 20', Stiff Silty Clay. Shear Strength = 1 tsf; Underlain by 5' of Silt. Glacial Till, Shear Strength = 5 tsf, Beneath Silt.	Highly Plastic Clay 0'-13' with Shear Strength = 0.9 tsf. Waterbearing Silty Clay 13'-18'. with Shear Strength = 0.57 tsf. Glacial Till 18'-28' with Shear Strength = 1.0 tsf.	Fat Clay to 18' Underlain by Clay-Shale. Considerable Gravel. Shear Strength Varied from 1 to 5 tsf where Samples Could be Recovered
NUMBER AND SIZE OF SHAFTS — STRAIGHT	3: 18"φ, 13' Depth, Base Resistance Destroyed	1: 24"φ, 12' Depth	4: 2 in Clay, 16"-20"φ, 14'-17' Embedment. 2 in Till, 20"-26"φ, 11'-22' Embedment.	5: 24"φ, 10.5'-26.9' Embedment. Voids Beneath Bases.	1: 30"φ, 26.5' Depth
NUMBER AND SIZE OF SHAFTS — BELLED	4: 16"-30"φ Stem, 36"-48" Bells, 30'-38' Depth	0	0	0	0
CONSTRUCTION PROCEDURE	Power Augers with Casing. Dry Process. After Initial Load Testing, Site was Ponded for One Year to Produce Soil Expansion.	Mechanical Auger. Installed in the Dry in Several Hours	Mechanical Auger. Installed in the Dry in 1-2 Hours.	Mechanical Auger. Drilled Below Water Table Without Use of Mud. Water in Longest Shaft. Installation Required 4 Hours Per Shaft	Power Auger. Dry Procedure. Installed in One Day
INSTRUMENTED ? IF SO, HOW ?	Yes. Bonded Strain Gages Along Reinforcing Steel. Carlson Stress Meters Installed Along Sides to Measure Lateral Pressure.	Yes. Concrete Embedment Gages, Strain Rods, Lateral Earth Pressure Cells at Several Levels	Yes. All Shafts Had Electronic Load Cells Near Base. Generally Reliable Results. Also Used Strain Rods	Not Reported	Yes. Concrete Embedment Gages, Strain Rods, Lateral Pressure Gages.
AVERAGE SHEAR STRENGTH REDUCTION FACTOR α AT ULTIMATE LOAD. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)	About 0.2 to 0.6 for Short Shafts in Overburden, on the Order of 0.3-0.5 in Shale. Based on Short Term Testing. (Average of UU Triaxial Tests)	0.55 (Average of Unconfined Compression Tests)	0.43 in Clay. 0.95 in Till (Average of Undrained Triaxial and Unconfined Tests)	0.5 in Clay 1.0 in Silty Clay and Till. (Average of Direct Shear and Torvane Tests)	Maximum of Approximately 900 Tons Transferred in Side Friction. α Not Computed Since Undisturbed Samples Could Not Be Recovered for Entire Length of Embedment
BEARING CAPACITY FACTOR N_c. (SOIL TEST TO WHICH REFERRED IN PARENTHESES.)	Ultimate Factor Not Clearly Defined.	9.2 (Average of Undisturbed Unconfined and Fully Softened Unconfined Tests)	Complete Bearing Capacity Equation of Meyerhof Recommended. N_{cp} = 15-25 Found in Till	None	Not Given
SETTLEMENT TO PRODUCE SIDE FAILURE	0.05"-0.2" in Short Shafts in Overburden. Not Clearly Defined for Shale.	Approximately 0.1"	0.12" in Clay. Varied in Till: 0.4"-1.5"	Approximately 0.15" for Shaft Embedded in Plastic Clay and Silty Clay. Not Clearly Defined for Others	Not Well-defined. Gradual Failure from 0.2" to 0.6" Settlement
LOAD DISTRIBUTION INFORMATION	Gages Fairly Stable Over Short Term. Showed Increased Load Transfer Near Bottom of All Shafts.	Load Distribution Approximately Linear. Only Four Levels of Axial Load Transducers	None	None.	Very Little Load Transfer in Top of Shaft; Highest Load Transfer (6-10 tsf) in Clay-Shale Near Base
TIME EFFECT	One Underreamed Shaft with Bell in Shale Under 120 Tons Sustained Load Showed About 40 Tons Tension in Shaft Above Bell One Year After Test Site Ponded	Not Given	Not Given	Not Given	Not Given.
MOISTURE CONTENT OF SOIL ADJACENT TO SHAFTS; SPECIAL FEATURES; OTHER REMARKS.	1.) Strain Gages Indicated Tension During Curing. 2.) Load Tests Conducted on Only Four Shafts 3.) Tensile Stresses on Order of Half of Maximum Shear Mobilized During Compression Testing Indicated in Shale. 4.) Lateral Pressure Cells Indicated Earth Pressure Coefficients Greater Than 1.0 in Shale One Year After Ponding Site	Tests Conducted Primarily to Evaluate Methods of Instrumentation	1.) Presents Results of Model Tests of Shafts in Silt. N_c About 6, α About 0.9. 2.) 24 Hour Settlement Readings Became "Notably Different" from Immediate Settlements at About 2/3 of Plunging Load.	Borehole Direct Shear Device Used to Obtain Shear Stress vs. Vertical Movement Curves for Various Normal Pressures against Walls of Borehole. From Results, Authors Deduced Maximum Pressure of 10 psi Acting Laterally During Actual Load Test.	Load Transfer and Load-settlement Correlation Made on Basis of Texas Highway Department Cone Penetrometer Tests

TABLE 2.2. SUMMARY OF FULL-SCALE LOAD TEST RESULTS (1970-1975)

Investigator(s) and Reference		O'Neill and Reese (1970)	Barker and Reese (1970)	Touma and Reese (1972)
Location		Houston	Houston	Houston, George West (US 59)
Date of Test(s)		1968-1969	1969-1970	1970-1971
Testing Method(s)		Quick	Quick, maintained, and cyclic	Quick and cyclic
Soil Description According to Unified Soil Classification System		0 - 29 ft CH 29 - 32 ft ML 32 - 42 ft CL 42 - 48 ft CH 48 - 51 ft CL 51 - 60 ft CH	0 - 0.75 ft concrete slab 0.75 - 14 ft SC 14 - 22 ft SM 22 - 34 ft ML 34 - 42 ft CL 42 - 70 ft CH & CL layers	15 - 30 ft of CL, ML, and CH soil layers overlying SP or S layer into which the shaft tipped
Number and Size of Shafts	Straight	3, 30 in. ϕ, 23 - 45 ft depth	1, 36 in. ϕ, 60 ft depth	3, 30 in. ϕ, 25.4 - 73.5 ft depth 1, 24 in. ϕ, 19.8 ft depth 1, 36 in. ϕ, 54.8 ft depth
	Belled	1, 30 in. ϕ, 23 ft depth	0	0
Construction Method		3 shafts by dry method 1 shaft by slurry method	Slurry method	2 shafts by dry method 3 shafts by slurry method

(continued)

TABLE 2.2. (Continued)

Investigator(s) and Reference		Beckwith and Bendenkop (1973) Report to Arizona Highway Department	Wooley and Reese (1974)	Engeling and Reese (1974)
Location		Phoenix and Tempe, Arizona	Houston	Bryan, Texas San Juan, Puerto Rico
Date of Test(s)		1972-1973	1973-1974	1973
Testing Method(s)		Maintained load	Sustained load (several months)	Quick
Soil Description According to Unified Soil Classification System		Layered, weakly to strongly lime cemented CL, CH, SC and GC layers	0 - 32 ft CH 32 - 70 ft SM	Bryan: 0 - 50 ft CH with CL layer from 15 - 21 ft and MH layer from 29 - 32 ft. Puerto Rico: 0 - 23 ft SC, 23 - 41 ft CL-SC, 41 - 48 ft S, 48 - 65 ft CH fissured.
Number and Size of Shafts	Straight	9: 30 - 36 in. ϕ, 15.6 - 35.6 ft depth	1: 30 in. ϕ, 62 ft depth	3: 30 - 36 in. ϕ, 42 - 62 ft depth
	Belled	18: toothed or belled shape 24 - 30 in. ϕ, 15.9 - 22.0 ft depth	0	0
Construction Method		Dry	Slurry method	Dry with casing Slurry method

(continued)

TABLE 2.2. (Continued)

Investigator(s) and Reference	O'Neill and Reese (1970)	Barker and Reese (1970)	Touma and Reese (1972)
Type of Instrumentation	Mustran cells and embedment gages	Mustran cells	Mustran cells
Average Shear Resistance Reduction Factor α at Ultimate Load	0.24 - 0.52	0.6 for clay 0.8 for sand and silt	0.3 - 0.9 for clay 0.6 - 1.8 for sand
Bearing Capacity Factor N_c	8.7 - 12.6	10	Not applicable - tip in sand
Type of Soil Test To Compute α and N_c	Quick Triaxial (UU)	Quick Triaxial (UU)	Quick Triaxial
Total Number of Load Tests	9	7	5 main tests and several cyclic tests

(continued)

TABLE 2.2. (Continued)

Investigator(s) and Reference	Beckwith and Bendenkop (1973) Report to Arizona Highway Department	Wooley and Reese (1974)	Engeling and Reese (1974)
Type of Instrumentation	Tell tales and LVDT's	Mustran cells and vibrating wire gages	Mustran cells
Average Shear Resistance Reduction Factor α at Ultimate Load	Not reported	0.18	0.59 - 0.70
Bearing Capacity Factor N_c	Not reported	Tip in sand - not applicable	9
Type of Soil Test To Compute α and N_c	Not applicable	Triaxial Quick	Triaxial Quick
Total Number of Load Tests	27	1	3

* At all sites the soils were weakly to strongly lime cemented.

CHAPTER 3. SOIL-STRUCTURE INTERACTION IN DRILLED SHAFTS

Mechanics of Soil-Structure Interaction

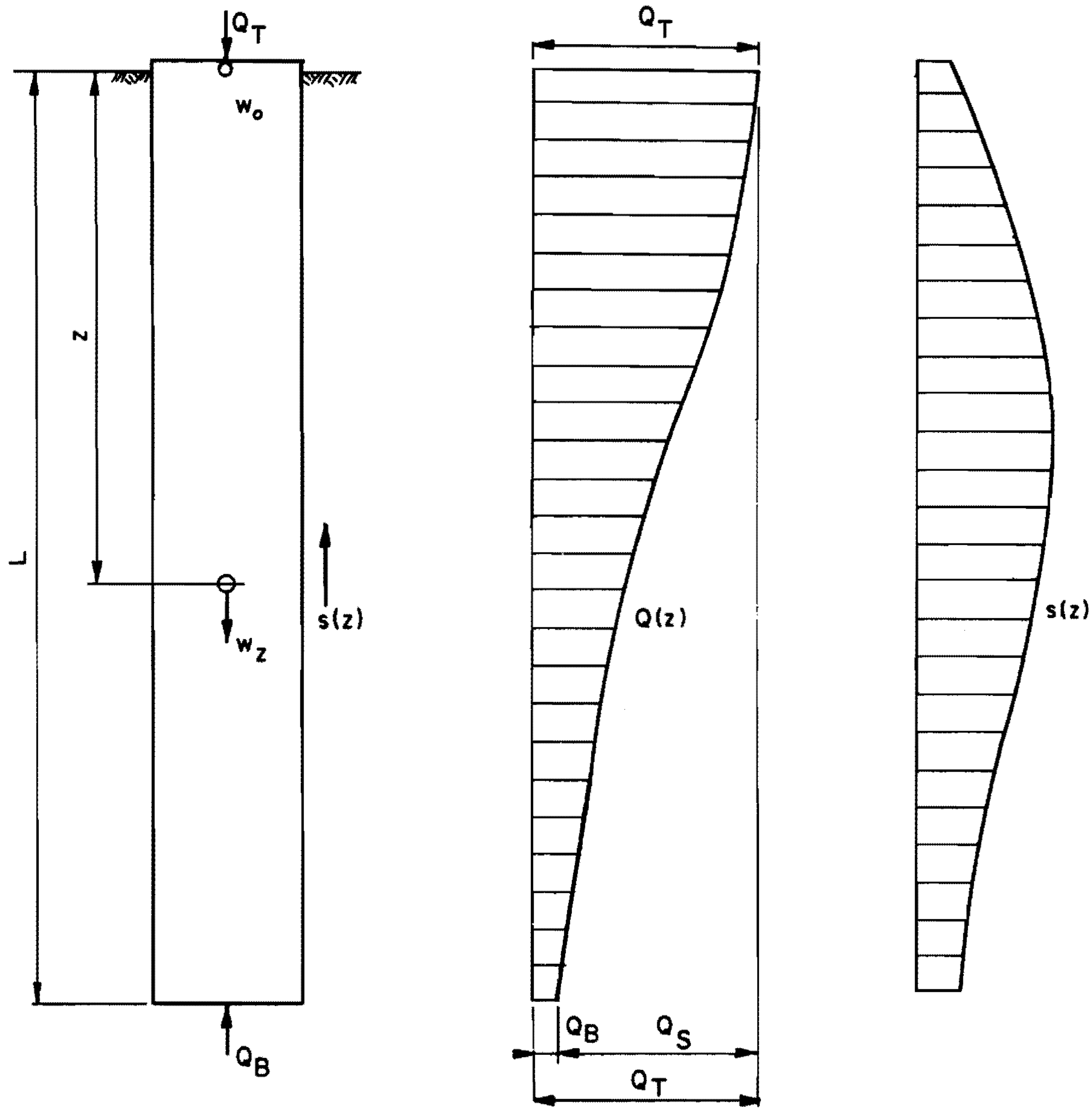
The mechanics of soil-structure interaction can be studied in two different ways. Vesic (1970) referred to these ways as "the transfer function" approach, and "the elastic solid" approach. In this chapter the former approach is referred to as "the distribution function" approach.

The distribution function approach was first proposed by Seed and Reese (1957) as illustrated in Fig 3.1. In this method the change in the magnitude of axial force with change in depth z along the pile is represented by a curve, called the load distribution curve. The curve shows that at the top of the pile, $z = 0$, the load in the pile equals Q_T , the applied load. At any other depth, z , the load in the pile equals the difference between the applied load, Q_T , and the side friction force acting on the circumference of the pile from the top of the pile to the depth z . Obviously, at the tip of the pile, $z = L$, the load on the pile equals Q_B , the axial force at the base of the pile. As shown in Fig 3.1(b), Q_s represents the total force of side friction acting on the shaft. Thus,

$$Q_T = Q_B + Q_s \tag{3.1}$$

If the function $Q(z)$ defining the load distribution curve is obtained along with the displacements at the top of the pile, a simple mathematical approach can be used to study the load transfer function $s(z)$, as explained below.

The movement of the pile wall at any depth z differs from that at depth $z + dz$ by an amount equal to the elastic compression of the length, dz , of the pile. Also, this elastic compression is equal to the



(a) Pile under axial load, Q_T ,

(b) Load distribution, Q(z).

(c) Load transfer, s(z).

Fig 3.1. Distribution function approach for a single pile.

unit strain of the pile, at depth z, multiplied by the length dz. This statement can be expressed mathematically as follows:

$$dw_z = \frac{Q_z}{AE} dz$$

or,

$$\frac{dw_z}{dz} = \frac{Q_z}{AE} \tag{3.2}$$

in which w_z is the movement of the pile wall at depth z, Q_z is the load acting on the pile at depth z, A is the cross sectional area of the pile, and E is the modulus of elasticity of the pile material.

Further, if the axial load acting on the pile at depth z, exceeds that at depth $z + dz$ by the amount dQ_z, then

$$dQ_z = - s_z \cdot C \cdot dz \tag{3.3}$$

where s_z represents the load transferred per unit circumferential area at depth z, and C is the circumference of the pile. Equation (3.3) can be rewritten as

$$s_z = - \frac{1}{C} \cdot \frac{dQ_z}{dz} \tag{3.4}$$

The above equation states that the magnitude of the load transferred per unit area from the pile to the soil, at depth z, is equal to the slope of the load distribution curve at that depth divided by the circumference of the pile.

Equation (3.4) also suggests that the load transfer information can be readily obtained once the load distribution curve is drawn. Equation (3.2) can be used to determine the displacement, w_z, at depth z, if the displacement, w_o, at the top of the pile, is known, because

$$w_o = w_z + \int_o^z \frac{Q_z}{AE}\, dz \qquad (3.5)$$

or

$$w_z = w_o - \int_o^z \frac{Q_z dz}{AE} \qquad (3.6)$$

With information about s_z and w_z on hand, it is possible to study the relationship between load transfer and pile movement at any depth along the pile. This relationship is usually referred to as the load transfer curve at a certain depth z. It can be expected that both the load distribution function $Q(z)$ and the load transfer curves will be affected by changes in the applied load Q_T.

If the load distribution curve and displacement at the top of the pile for a particular load Q_T are obtained, the soil-structure interaction at that load can be analyzed by using the distribution function approach. This approach has been often used during the last decade in an attempt to rationalize the design of drilled shafts. The approach is approximate only to the extent that it does not consider the effect of load transferred to the soil at one point upon the loads at other points within the soil mass.

The "elastic solid" approach is an attempt to minimize the above noted approximation by using the equations of Mindlin (1936) as described earlier in Chapter 2. The main drawback of this approach lies in idealizing the soil as a homogeneous, elastic, isotropic medium which can be defined by the two deformation characteristics, namely, modulus of deformation and Poisson's ratio. Generally such a simplification cannot be justified for real soils.

The distribution function approach is currently the best available method for gaining an understanding of the interaction between pile and soil. The approach makes use of field data and reveals the behavior of a drilled shaft under actual geotechnical conditions. The study reported here is based upon the distribution function approach.

Methods of Studying Load Distribution in Drilled Shafts

Information about load distribution in drilled shafts is usually obtained at discrete points spaced suitably along the length of shaft. The locations of these points are selected in such a way that anticipated changes in the load distribution pattern may be properly recorded. The changes may be due to changes in the geometry of the shaft or other in-situ conditions.

Axial loads in piles or drilled shafts can be measured by load cells, tell tales, vibrating wire cells, and electrical strain gages. For details of these and other types of load measuring devices, reference is made to Perry and Lissner (1962), Snow (1965), Barker and Reese (1969), Mansur and Hunter (1970), and O'Neill and Reese (1970). Important aspects of the particular electrical strain gages used in this study are discussed later in Chapter 5. General concepts of load measurements in axially loaded drilled shafts using electrical strain gages are given in the next few paragraphs.

In an elastic material, stress and, therefore, load is proportional to strain. From elementary knowledge of mechanics of elastic materials the following equation is obtained:

$$Q_z = A E \epsilon_z \qquad (3.7)$$

in which A , E , and Q_z have the same meaning as in Equation (3.2), and ϵ_z represents the strain in the shaft material at depth z . Equation (3.7) is true only if the material of the shaft is elastic, the applied vertical load Q_T is concentric with the axis of the shaft, the shaft is plumb, and any change in geometry of the shaft does not cause eccentricity between the centroid of the cross section and the line of application of the vertical load. In order to determine Q_z , all the quantities on the right hand side of Equation (3.7) must be known.

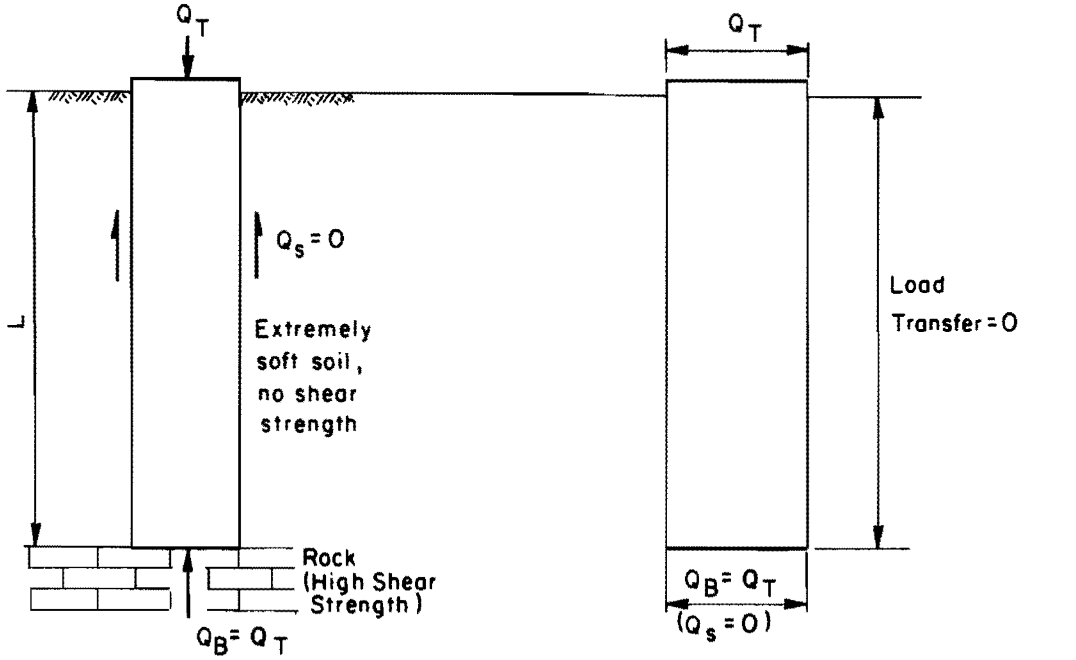
At present, it is assumed that both A and E are known, so that only ϵ_z , the strain at depth z , is to be determined. If a cylindrical slice of the shaft is imagined at depth z , having a thickness ℓ parallel to the axis of the shaft and positioned concentrically, then the strain on this slice is given by

$$\epsilon_z = \frac{\Delta \ell}{\ell} \tag{3.8}$$

in which $\Delta \ell$ is the axial compression of the slice itself, due to an average load Q_z. A Mustran cell described later in Chapter 5 is a device to measure ϵ_z using the principle of equation (3.8). By installing Mustran cells at suitable depths within the shaft, load distribution along the shaft can be established with reasonable accuracy. These basic concepts are utilized in the study of axially loaded drilled shafts.

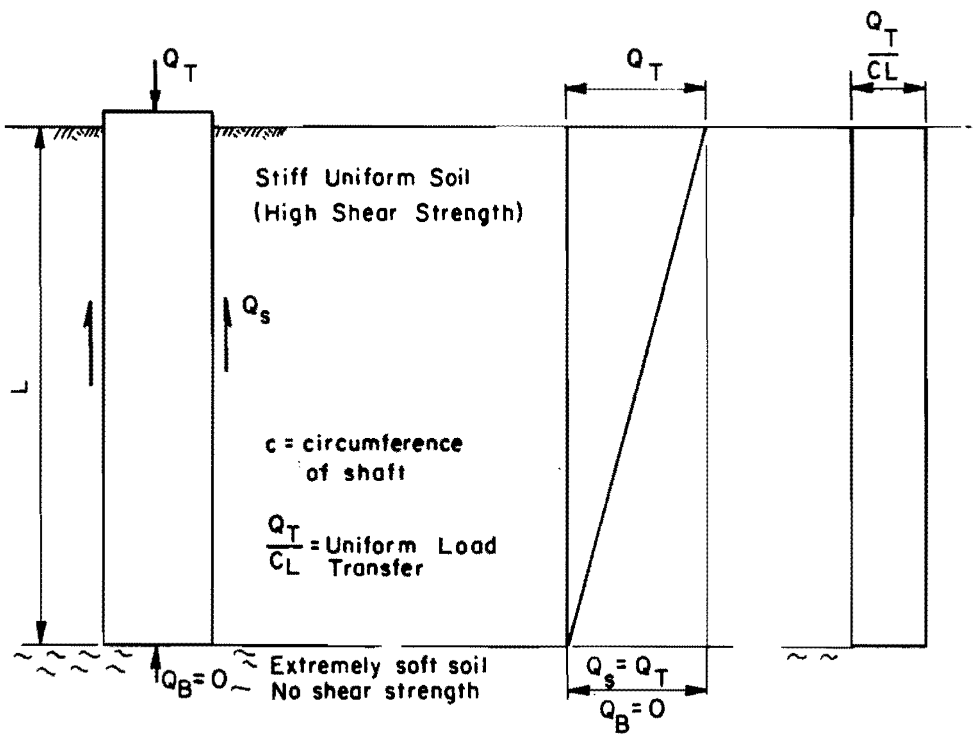
Factors Affecting Load Distribution and Total Capacity

Load distribution along the length of a drilled shaft depends, to a large extent, on the existing geotechnical conditions. The construction procedures as well as the shape and spacing of drilled shafts are greatly influenced by the environment into which the shaft is built. Those factors, in turn, affect load distribution. To get an idea of the influence of the subsurface conditions, it is helpful to consider the two extreme cases shown in Fig 3.2. In the case of a strong rock underlying a very weak soil stratum, the system is analogous to a spring of zero stiffness, representing side resistance, acting in parallel with a stiff spring, representing base resistance. Thus, any load Q_T applied at the top of the shaft is transferred entirely to its base. In the other case, a stiff spring, representing side resistance, is acting in parallel with a spring of zero stiffness, representing the base resistance. In this case all the load is taken up by side resistance. For simplicity, it has been assumed that the shaft is rigid, implying that all points within the shaft undergo the same movement relative to the surrounding soil: in a real system, the shaft material will be elastic, not rigid; and the shear strengths of soil layers will be neither as uniform nor abruptly contrasting as shown in the illustration. Therefore, the real load distribution curves will fall in between the two extremes shown in Fig 3.2(b) and (d). Thus, the actual shape of the curve is controlled, to a great extent, by the actual subsurface conditions which determine the relative stiffness of the in-situ soils.



(a) Axially loaded end-bearing shaft.

(b) Load distribution and load transfer for axially loaded end-bearing shaft.



(c) Axially loaded friction shaft.

(d) Load distribution and load transfer for axially loaded friction shaft.

Fig 3.2. Two extreme conditions showing zero and uniform load transfer due to different geotechnical conditions.

Geotechnical conditions also dictate the construction procedures which in turn affect the load distribution pattern. This will be further clarified in the next few paragraphs. Reference is made to Reese and Hudson (1968) for a formal presentation of the mechanics of drilled shafts.

The position of the groundwater table relative to the shaft often controls the method of installing the shaft. If the water table is high the drilled hole generally cannot stand open because high seepage gradients combined with relief of confining pressures cause the soil around the hole to collapse. This problem is more severe in loose, cohesionless soils which cannot stand vertical even for a short duration due to lack of cohesion and interparticle friction. Drilled holes in fissured or jointed clays can also collapse easily due to flow of water towards the hole.

The difficulties arising from high water table or soft soil conditions necessitate the use of the "casing method" or the "slurry method" as opposed to the "dry method" of construction. Each method of construction is described in detail in Chapter 6.

Highly alkaline or acidic soils tend to react with the shaft materials. This may not only alter the load distribution pattern, but may cause a considerable reduction in total capacity of the shaft, accompanied by structural damage. In such situations special types of cements, aggregates and reinforcing bars become necessary.

In expansive soils, an upward load is placed on the shaft by the swelling soil, thus changing the load transfer and concrete stresses significantly. A net tension load may result in spite of the compressive loads from the supported structure. Thus, the geometry as well as the structural details of the shaft have to be adjusted to counter the effects of swelling soils. Design values to estimate the effects of swelling on the load distribution for drilled shafts are not available. Mohan (1961), (1969), (1975) reported the use of single or multiple bells in the zone of insignificant moisture variation in black cotton soil areas in India. Newland (1968) reported foundation uplift up to 12 in. of a building in Adelaide, Australia, which was supported on a pier foundation in a swelling soil.

Farmer (1969) and Baker and Khan (1971) have cited some examples of drilled shafts where they believe the leaching out of cement was caused by ground water flow. Such a defect would inevitably impair the structural integrity of the shaft, thus causing failure of concrete before the soil fails.

Factors Affecting Load Transfer and Base Resistance

Load transfer denotes the resistance offered by soil, per unit area of soil-pile interface, when the shaft moves relative to the soil. The summation of the load transfer over the entire circumferential area of the shaft equals the side friction component, Q_s, of the axially loaded member. By its very nature, load transfer is related to the shear strength of the soil. Under ideal conditions, full shear strength should be mobilized as load transfer. Low values of load transfer do not necessarily imply low shear strength of the soil because load transfer may be affected by one or more of the following factors:

(1) construction defects,
(2) type of soil,
(3) depth of soil stratum, and
(4) type of soil underlying the base.

Presence of voids, slurry, or loose material between the shaft and the surrounding soil can significantly reduce load transfer even though the surrounding soil may be strong. Cavities within the shaft can cause unpredictable movements which will be reflected in the load transfer behavior as well. Poor quality of concrete can increase the compressibility of the shaft, thus requiring larger movement at the top to achieve maximum load transfer.

Shear strength of fissured clays and hard soils may be considerably reduced due to release of confining pressures. When a hole is drilled to install a shaft, the confining pressures on soil elements next to the shaft are either totally removed or appreciably reduced, thus causing reduction of shear strength and possibly load transfer capabilities of the soil.

Presence of soft layers under the base tends to have the effect of a soft spring. Thus, if the upper layers are stiff, there will be a greater load transfer above the base. This is shown by Poulos and Davis (1974) in their charts based on the analytical approach.

Factors affecting the base resistance are generally those which affect the bearing capacity of deep footings. They are discussed by O'Neill and Reese (1970) and in several text books such as one by Terzaghi and Peck (1968). Construction defects alone can supersede all factors which can affect the base resistance of a drilled shaft.

CHAPTER 4. GEOTECHNICAL INFORMATION

Site Selection, General Location, and Geology

Site selection for the tests near Austin centered around locating easily accessible areas with shale at shallow depths and with the least possibility of collapse of a drilled hole. These criteria were set to obtain maximum information on the behavior of drilled shafts in shales within available time and resources. A study of the Geological Highway Map of Texas (1963) revealed that the Taylor and Austin formations constitute shale outcrops near Austin. The drilling records of the Texas Highway Department confirmed that shale exists in Montopolis, about 6 miles southeast of Austin near the intersection of highways SH 71 and US 183, at depths ranging from 15 ft to 30 ft. One site for a load test in shale was selected by the Texas Highway Department. The site was at the construction site for the interchange of IH 35 - SH Loop 635 in Dallas. The Texas Highway Department furnished useful subsurface information at that construction site. Their drilling logs showed that shale deposits exist within a depth of 15 to 20 ft from ground surface.

With the general knowledge of subsurface information near the intersections of highways SH 71 and US 183, the task of site selection at Montopolis narrowed down to locating an area where the difficulty of drilling through the upper soil layers was insignificant, and the shale layer was at a shallow depth. Accessibility to the area was very good, being next to two highways. Three exploratory auger holes were made in the third week of August, 1974, in the general area of the intersection. At the first location, there were considerable drilling problems due to the presence of a loose, water-bearing sand and gravel layer within 10 ft from the ground surface. Moreover, shale was found at a depth of 43 ft, which was deeper than expected at that site. Another location was chosen about 1000 ft south of the first one. Here, the shale was located at a depth of 27 ft, the hole was dry and drilling through the upper soil layers was not difficult. About 225 ft west of the second location another exploratory auger hole was made. Here, the shale was at a depth of only 17 ft and the water-bearing sand and gravel layer, although present, did not cause drilling problems. The area around the third exploratory hole was considered most suitable for conducting research on three instrumented drilled shafts. This site is designated as the Montopolis Site. All load tests performed at this site are referred to as Montopolis Tests.

It was decided to conduct load tests on one instrumented drilled shaft at the construction site in Dallas, within the framework and schedule of the existing contract for construction of the interchange IH 35 - SH Loop 635. The personnel of the Center for Highway Research at The University of Texas at Austin were assigned the task of instrumenting one of the tests shafts, and recording and analyzing the data obtained from load tests. With the cooperation of the personnel of the Texas Highway Department as well as the foundation contractor, Martin and Martin, all the necessary arrangements were made to conduct tests at this site, which is designated as the Dallas Site in this study. All load tests performed at the Dallas Site are referred to as Dallas Tests.

Figure 4.1 shows the general location of the Montopolis and Dallas Sites with regard to the predominant geologic formations in Texas. Both these sites have shales from the same geologic formation known as the Austin formation. These shales are sedimentary deposits from the Gulf Series of the Cretaceous System, which is the most recent system of the Mesozoic Era. Generally, these shales are underlain by a limestone layer which is several hundred feet thick, according to the Geological Highway Map of Texas (1963). These two sites are about 200 miles apart.

Description of Borings at the Test Sites

Montopolis Site: Seven borings, up to 34 ft deep, were made by different techniques at the Montopolis Site. Their layout in relation to the test shafts (MT1, MT2, and MT3) and reaction shafts (MR1, MR2, and MR3) is indicated in Fig 4.2 and Table 4.1. Five of these borings, B1, B2, B3, SP1, and TP1, were taken by the personnel and equipment of the Texas Highway Department in accordance with their procedures. G1 was the original exploratory boring made during site selection on August 21, 1974, by the contractor, Farmer Foundation Company. G2 was a special boring made for in-hole tests

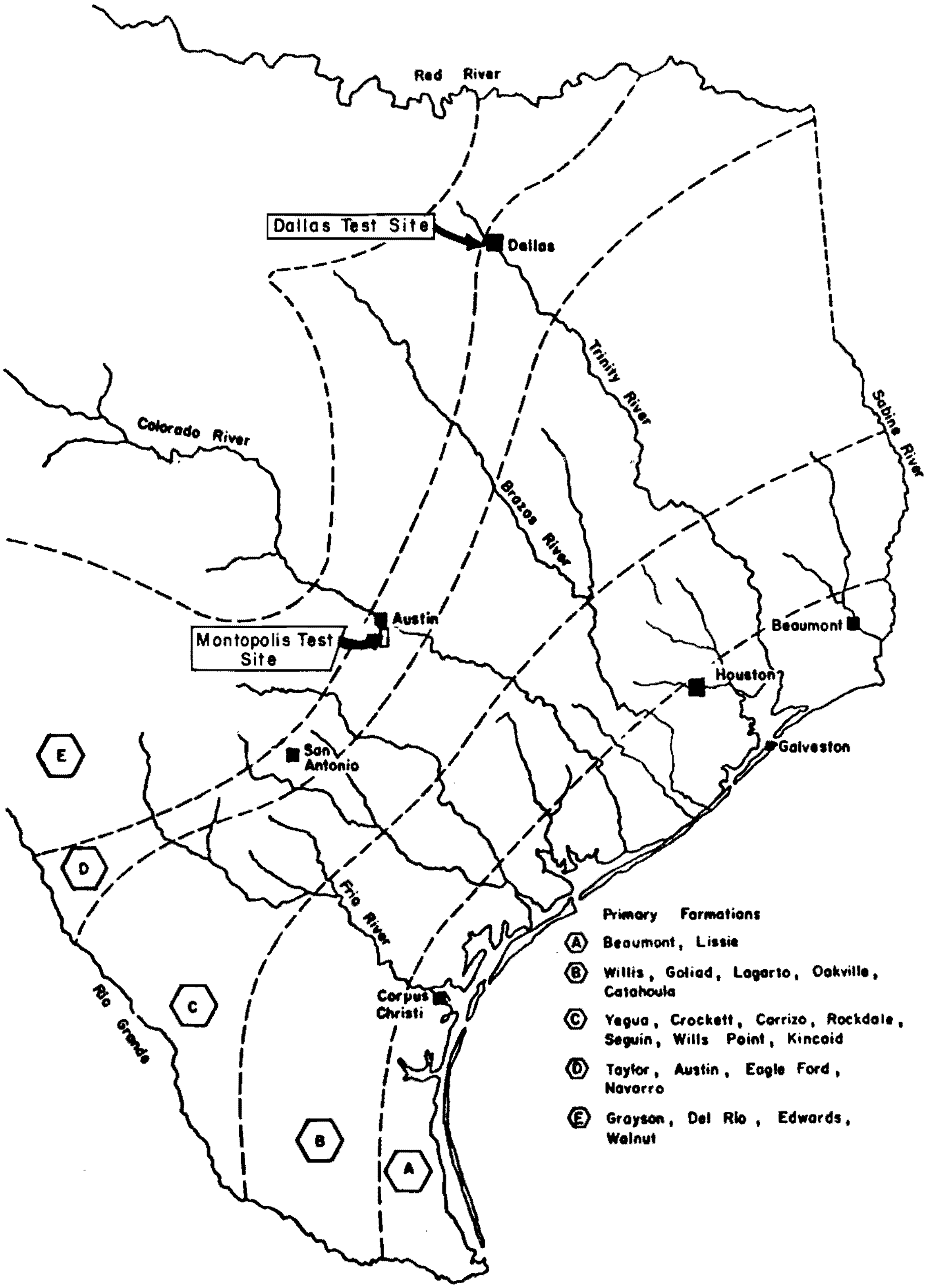
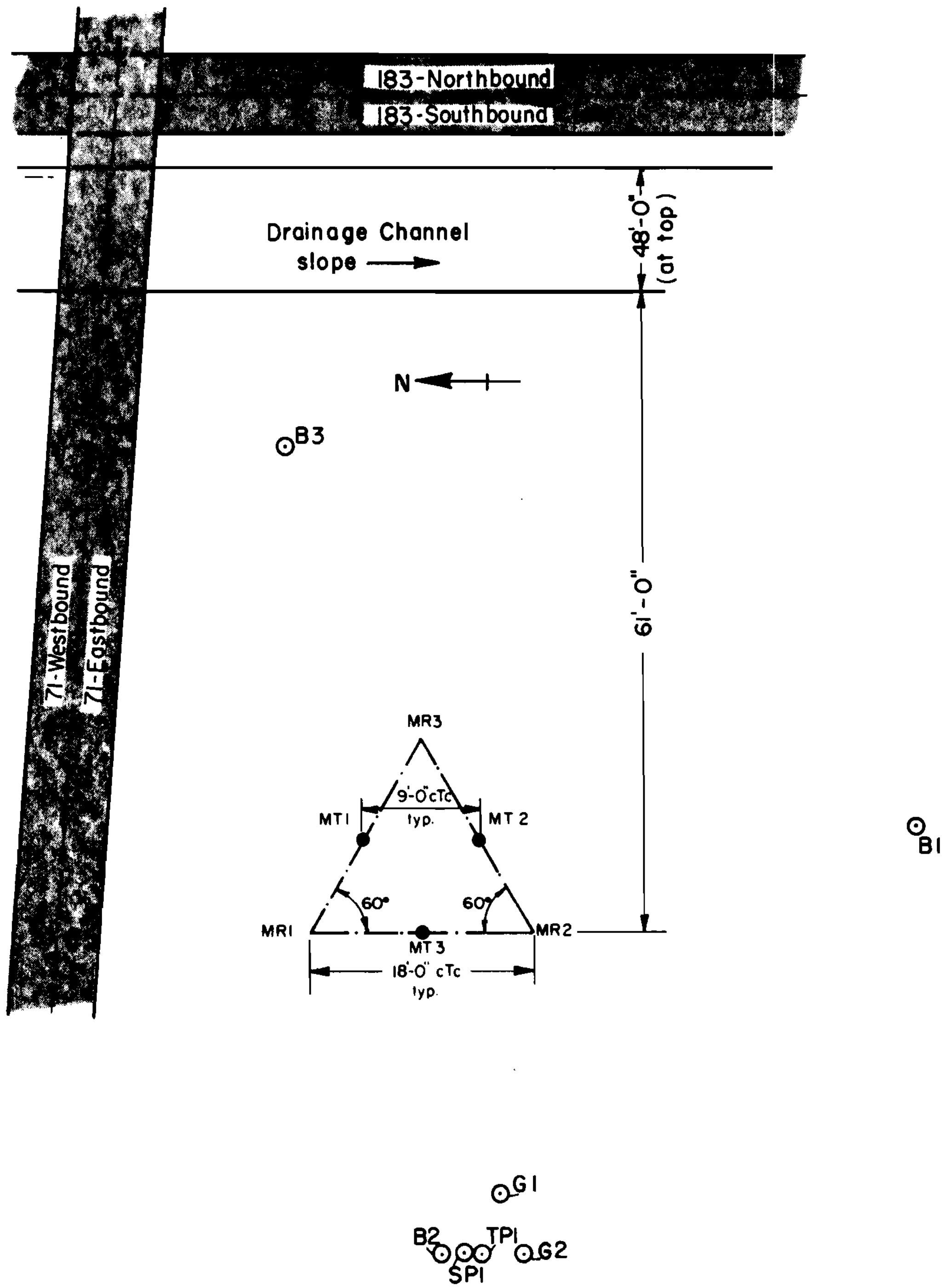


Fig 4.1. Locations of the two test sites (after Barker and Reese, 1970).



See Table 4.1 for Location of Drilled Shafts and Borings shown above.

Fig 4.2. Locations of drilled shafts and borings at the Montopolis Site.

TABLE 4.1. LOCATIONS OF DRILLED SHAFTS AND BORINGS AT THE MONTOPOLIS SITE (Reference Fig 4.1)

Symbol	Explanation	Location *
MT1	+ Test Shaft Casing Method	4.5 S 7.8 E
MT2	+ Test Shaft Slurry Method	13.5 S 7.8 E
MT3	+ Test Shaft Cased, End Bearing	9.0 S 0.0 E
MR1	Reaction Shaft	0.0 N 0.0 E
MR2	Reaction Shaft	18.0 S 0.0 E
MR3	Reaction Shaft	9.0 S 15.6 E
B1	Wet Barrel THD Boring	55.0 S 8.0 E
B2	Wet Barrel THD Boring	10.5 S 26.0 W
B3	Wet Barrel THD Boring	2.0 N 39.0 E
G1	Exploratory Auger Hole	16.0 S 21.0 W
G2	Auger Hole for In-Hole Tests	18.0 S 26.0 W
SP1	Boring with Standard Splitspoon	12.0 S 26.0 W
TP1	Boring with THD Cone	14.0 S 26.0 W

* With respect to MR1, distances in ft.

\+ Installation method of test shaft.

with the cooperation of the contractor. A brief description of the borings at the Montopolis Site follows.

Borings B1, B2, and B3 were made to obtain 3-in. diameter continuous undisturbed samples for shear strength and identification tests in the laboratory. The samples were taken between the ground surface and a few feet below the proposed tip elevation of the test shafts. These borings, as well as TP1 and SP1, were drilled from December 6, 1974, to December 12, 1974. The construction of all shafts at the Montopolis Site had been completed earlier, in October, 1974, under a separate contract. As a result, projecting shafts and excavated soil lying around them did not allow the borings to be located any closer to the test shafts than indicated on Fig 4.2. Boring B1, located about 42 ft south of MT1, was drilled with a 3-in. I.D. push barrel up to a 4-ft depth without any drilling mud. Thereafter, drilling mud was used along with a Modified Wet Barrel fitted with a Push Barrel, as described in the Texas Highway Department Foundation Exploration and Design Manual (1972). This boring could not be drilled beyond an 8-ft depth due to a suspicion that a drainage pipe passing through the area was in the line of drilling at this location.

Boring B2 was located away from the estimated centerline of the drainage pipe as close as possible to the test shafts. It was located approximately 26 ft west of MT3. Samples were recovered without the aid of drilling fluid to a depth of 6 ft. From 6 ft to the bottom of the hole at 34 ft, drilling mud was used. Drilling problems, caused by partial collapse of the hole, became significant from 8 ft to 14 ft in depth in spite of the drilling mud. A 5-1/2-in. I.D. casing was installed from the ground surface to a depth of 14 ft to overcome this problem which was due to water-bearing sand and gravel layers in that zone. No sample was recovered from a depth of 14 ft to 16 ft due to misjudgment of depth. From the 16-ft to the 34-ft depth a Modified Web Barrel fitted with a Push Barrel was used without any drilling problems. Work on this boring was done up to the 24-ft depth on December 6, 1974. The remaining portion of drilling was done on December 9, 1974.

Boring B3 was located on the east side of the test area, opposite B2. This was done to allow interpolation of the soil profile at each test shaft as accurately as possible. B3 was drilled from the ground surface to a depth of 22 ft on December 11, 1974; the remaining 12 ft were drilled on December 12, 1974. Drilling mud was used after taking samples up to a depth of 6 ft.

After a sample at the 16-ft depth had been taken, the upper soil layers began to fall into the drilled hole despite the use of drilling mud. The caving was probably due to a 4-ft thick, water-bearing sand and gravel layer found approximately 10 ft from ground surface. A 5-1/2-in. I.D. casing was installed to a depth of 16 ft to overcome this difficulty. Drilling proceeded from the 16-ft depth to the bottom of the hole without any difficulty.

Borings TP1 and SP1 were made to measure dynamic penetration resistance of the in-situ soils using the Texas Highway Department cone and the standard split-spoon, respectively. TP1 was located about 3 ft south of B2. A Wet Barrel, without the inner Push Barrel, was used to advance drilling and to get disturbed core samples, if possible. Drilling mud was used throughout the drilling process. Texas Highway Department cone penetration resistance was measured from depths of 2.5 ft to 30 ft at 2.5-ft intervals, and the last measurement was made at a depth of 33 ft. After each measurement of the THD cone penetration resistance, the hole was drilled to the next lower depth using a Wet Barrel and drilling mud. Samples thus obtained were highly disturbed and were used only for visual identification. The THD cone penetration measurements and recording were done in accordance with the current procedures of the Texas Highway Department. TP1 was drilled to a 22-ft depth on December 9, 1974. No work was done on December 10, 1974, due to inclement weather. Drilling to the bottom of the hole at a depth of 33 ft was completed on December 11, 1974. The dimensions of the THD cone are shown in Fig 4.3.

SP1 was located about 2 ft south of B2. The purpose of this hole was to get the Standard Penetration Resistance, using a split-spoon sampler and a 140-pound hammer falling 30 in. However, the drilling rig available at the site was geared only for the THD cone test in which a 170-pound hammer falling 24 in. is used. In order to utilize the available equipment as far as possible, the height of fall of the 170-pound hammer was adjusted to 24.7 in. so as to produce the same energy of fall as a 140-pound hammer with a fall of 30 in. Drilling mud was used throughout the drilling operation. Penetration-resistance readings were recorded at the ground surface, and from the depth of 5 ft to 12.5 ft at 2.5-ft intervals. The top of the shale layer was hit at about a 14-ft depth, and it was decided to end the hole at that depth. Samples obtained by the splitspoon were used only for visual identification. SP1 was drilled on December 12, 1974.

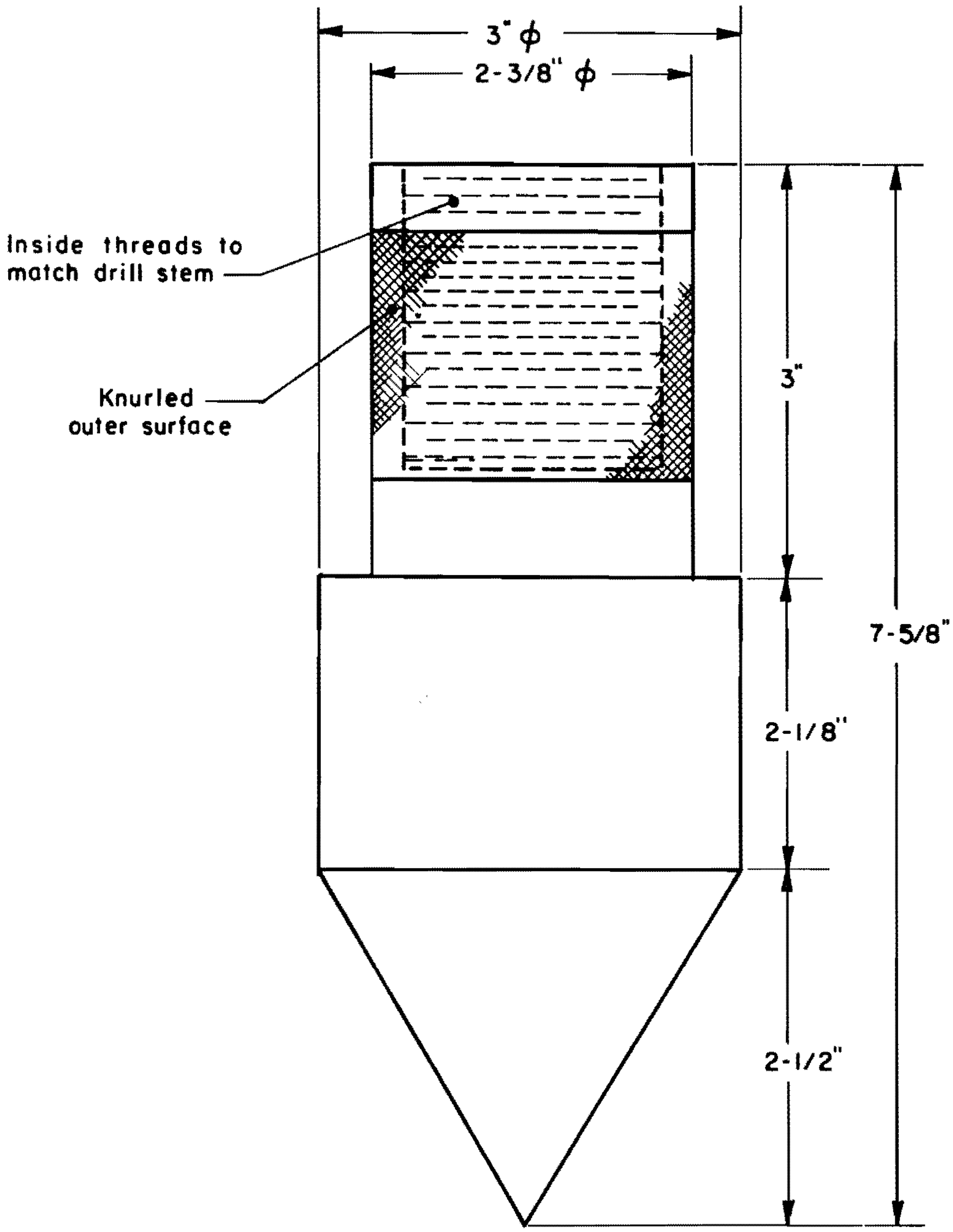


Fig 4.3. Dimensions of the Texas Highway Department Cone Penetrometer (courtesy Texas Highway Department).

The exploratory hole G1, situated about 6 ft south and 5 ft east of B2, was the first hole drilled at the Montopolis Site during the process of site selection. It was drilled with a 24-in. hollow stem auger using the rig normally employed for installing drilled shafts. The main purpose of this hole was to locate the top of the shale layer. To that extent, the information obtained from this hole was useful. This hole was drilled on August 21, 1974.

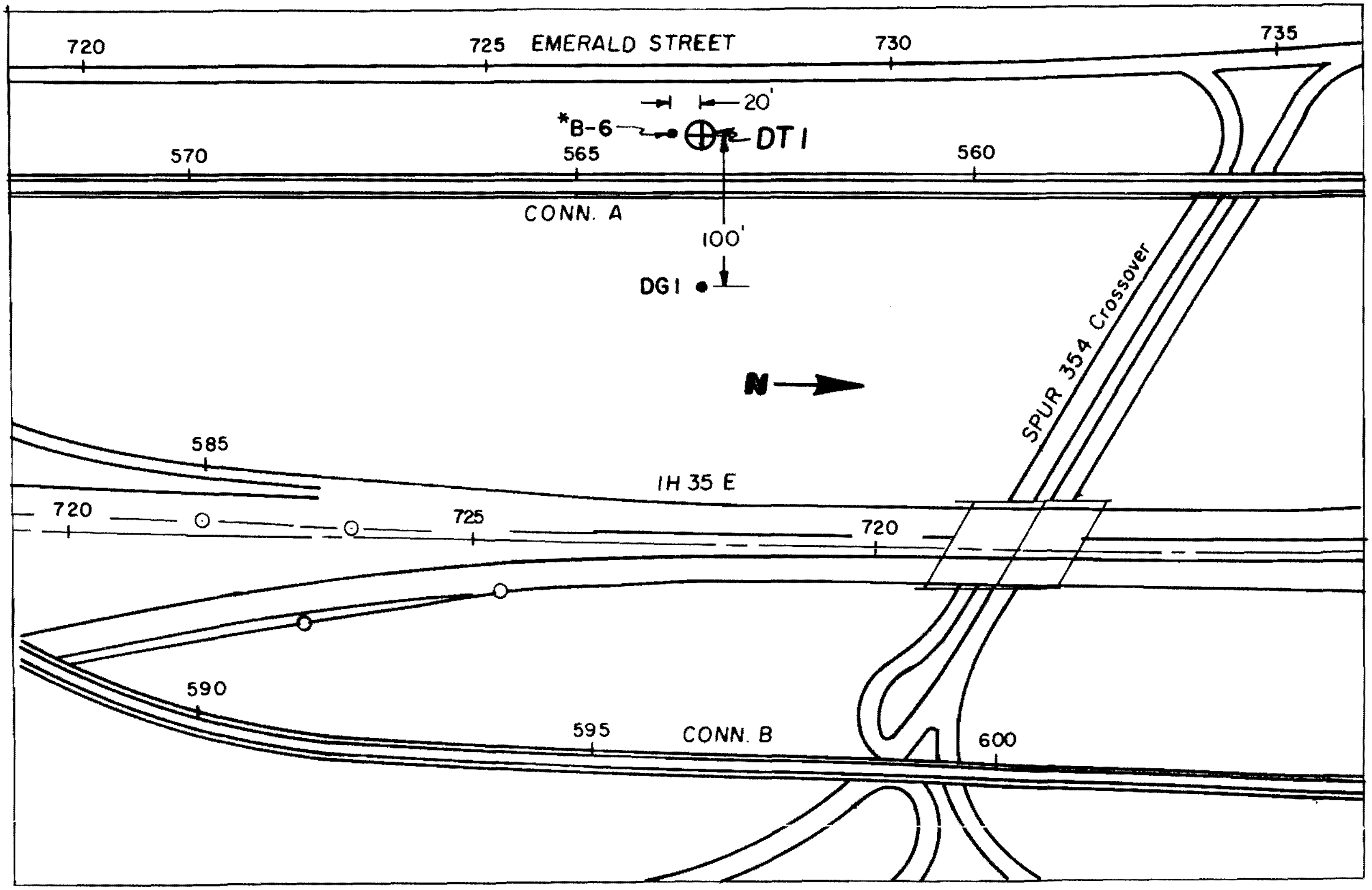
The second auger hole, G2, 48 in. in diameter, was drilled for the sole purpose of obtaining in-situ shear strength of the shale at the Montopolis Site. It was located about 8 ft south of B2. The drilling was done without any drilling mud. After advancing the hole to a depth of about 16 ft, where shale existed, a 48-in. diameter casing was installed to preclude problems due to caving of the soil and gravel layer above. Some tests were performed on the shale on December 16, 1974. The hole was advanced to a depth of 18 ft on December 17, 1974, and the remaining in-situ tests were performed at that depth. Details of in-hole tests are described later in this chapter.

Dallas Site: The Texas Highway Department furnished all the drilling logs prepared for the construction of IH 35E - SH Loop 635 Interchange. These logs provided information on soil types, strata thicknesses, number of blows for THD penetration tests using the THD cone, and method of coring. Limited resources precluded the possibility of making additional borings to procure undisturbed samples for laboratory tests. With the cooperation of the foundation contractor, Martin and Martin, a 54-in. diameter cased hole DG1, similar to the hole G2 at the Montopolis Site, was augered to a depth of about 25 ft below the ground surface. In-hole tests on the shale were conducted at this depth on August 15, 1975; these tests are described later in this chapter. The locations of the borings in relation to the test shaft and reaction shafts are shown in Fig 4.4.

The drilling logs are presented in Appendix A.

Methods Used to Procure and Preserve Soil Samples

Undisturbed samples were obtained only at the Montopolis Site. The samples were 3 in. in diameter and were recovered by a Modified Wet Barrel fitted with a Push Barrel. Briefly, the sampler consists of two barrels,



*Taken by Texas Hwy. Dept.. Figs 4.10 and 5.9 show information obtained from this boring.

Fig 4.4. Locations of test shaft DT1 and boring B-6 at the Dallas Site.

inner and outer. The outer barrel is rigidly connected to the drill rod and rotates with the rod, whereas the inner barrel, a thin-walled tube, is connected to the outer barrel with a swivel arrangement so that it does not rotate with the outer barrel. Therefore, the torsion applied to the outer barrel is not transmitted to the core, which remains within the inner barrel due to friction. The inner barrel stays in a slightly retracted position inside the outer barrel. The outer barrel may have face-hardened saw teeth or a diamond core bit at the drilling end. In this study an outer barrel with hardened saw teeth was used because of knowledge of its successful application in the Montopolis area during the past few years. Figure 4.5 shows a Modified Wet Barrel sampler in use.

Generally, the rotating sampler was forced down to about 24 in. or so with a steady push. Then it was rotated about two turns to shear off the core at its bottom. Thereafter it was withdrawn, and the inner push barrel, with the core in it, was removed from the outer barrel as shown in Fig 4.6. The push barrel was mounted on a hydraulic jack whose piston pushed the core within the push barrel while the push barrel itself was being held tightly in position. This procedure is shown in Fig 4.7 and Fig 4.8. The extracted core was immediately double-wrapped in heavy-duty aluminum foil, properly labeled, and sealed by dipping it several times in a bucket of molten wax. The sealed samples were properly arranged as shown in Fig 4.9 and they were stored in a humidity and temperature-controlled room as soon as possible.

Disturbed samples from the Montopolis Site were stored along with the undisturbed samples. The disturbed samples were double-wrapped in heavy-duty aluminum foil, but were not sealed. The disturbed samples recovered at the Dallas Site were stored, classified, and analyzed in accordance with the procedures of the Texas Highway Department as described in their Manual referred to earlier.

Field Tests Run to Estimate Shear Strength of Soils

Dynamic penetration resistance tests and static tests using a Dutch cone and a 2.5-in. diameter plate were run to estimate the in-situ shear strength of soils. Static tests and THD cone resistance tests were done at both sites, while the Standard Penetration Test (with a modified hammer) was done only at the Montopolis Site.

Fig 4.5. Modified wet barrel sampler in use.

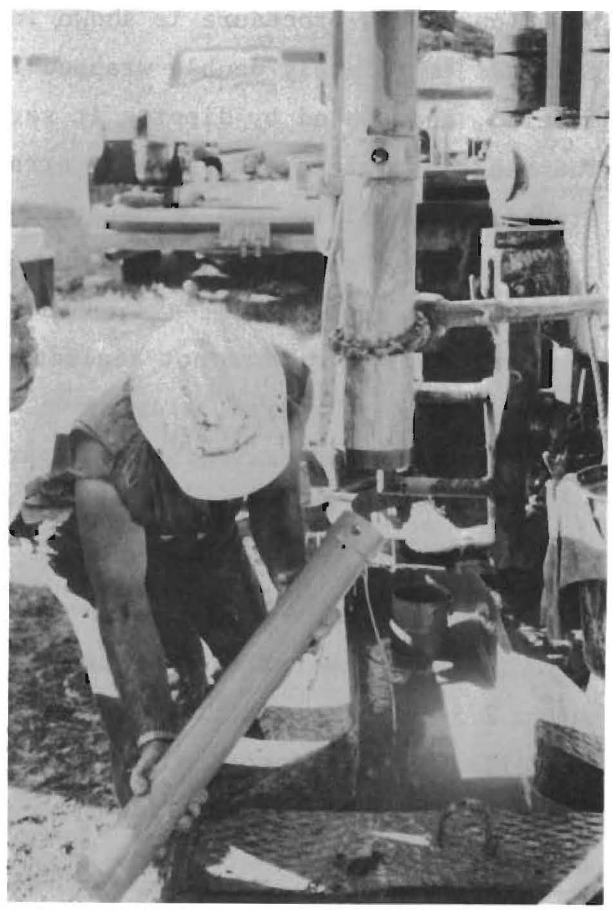


Fig 4.6. Inner push barrel removed from outer barrel.

Fig 4.7. Soil sample partially pushed out of the push barrel.

Fig 4.8. Soil sample almost completely pushed out of the push barrel. (Notice cracks forming on sample just after extraction.)

Fig 4.9. Soil samples sealed at site.

The THD cone test is a dynamic penetration resistance test used by the Texas Highway Department. This test is intended to determine the relative density or consistency and thereby the load-carrying capacity of in-situ soils. The test consists of counting the number of blows required to drive a "standard" cone a distance of 6 in. into the soil. Two consecutive counts of the number of blows per 6-in. penetration are taken at each depth. If the material is very hard, the penetration of the cone, in inches, is measured for each of two consecutive sets of 50 blows. The cone has the shape and dimensions indicated in Fig 4.3. A 170-pound hammer is used with a fall of 24 in. The fall of the hammer is regulated by an automatic trip mechanism. Several charts prepared by the Texas Highway Department correlate the cone penetration resistance to the allowable point-bearing load and friction load per unit area for designing axially loaded drilled shafts. Figure 4.10 shows the variation of N_{THD} and N_{SPT} with depth at the Montopolis and Dallas Sites. These N_{THD}- and N_{SPT}-values are taken from relevant borings presented in Appendix A.

The Standard Penetration Test is another dynamic penetration resistance test commonly used to approximate the density or consistency of in-situ soils. In this test a standard splitspoon is driven with a 140-pound hammer falling 30 in. The number of blows required to drive the spoon 6 in. into the soil are recorded for three consecutive 6-in. increments. A disturbed sample is also recovered in the spoon. The total number of blows required to penetrate the last two 6-in. increments is expressed as N_{SPT}. Over the past few years, it has been used extensively by practicing engineers to get a general idea of the soil strength. The chart and tables commonly used for this purpose are reported by Peck, Hanson, and Thornburn (1974). The variation of N_{SPT} with depth is shown for the Montopolis Site in Fig 4.10. The N_{SPT} values are based on the boring SP1 included in Appendix A.

Six static Dutch cone tests and five static plate load tests were run on the shale at the Montopolis Site. The main purpose of these tests was to determine the in-situ shear strength of shale, thus minimizing the effect of macro- and micro-fissures formed due to release of confining pressure.

Some planning, fabrication, and laboratory work was necessary prior to performing in-hole tests at each site. It was decided to apply downward load on the Dutch cone or plate by means of a hydraulic jack butting against the

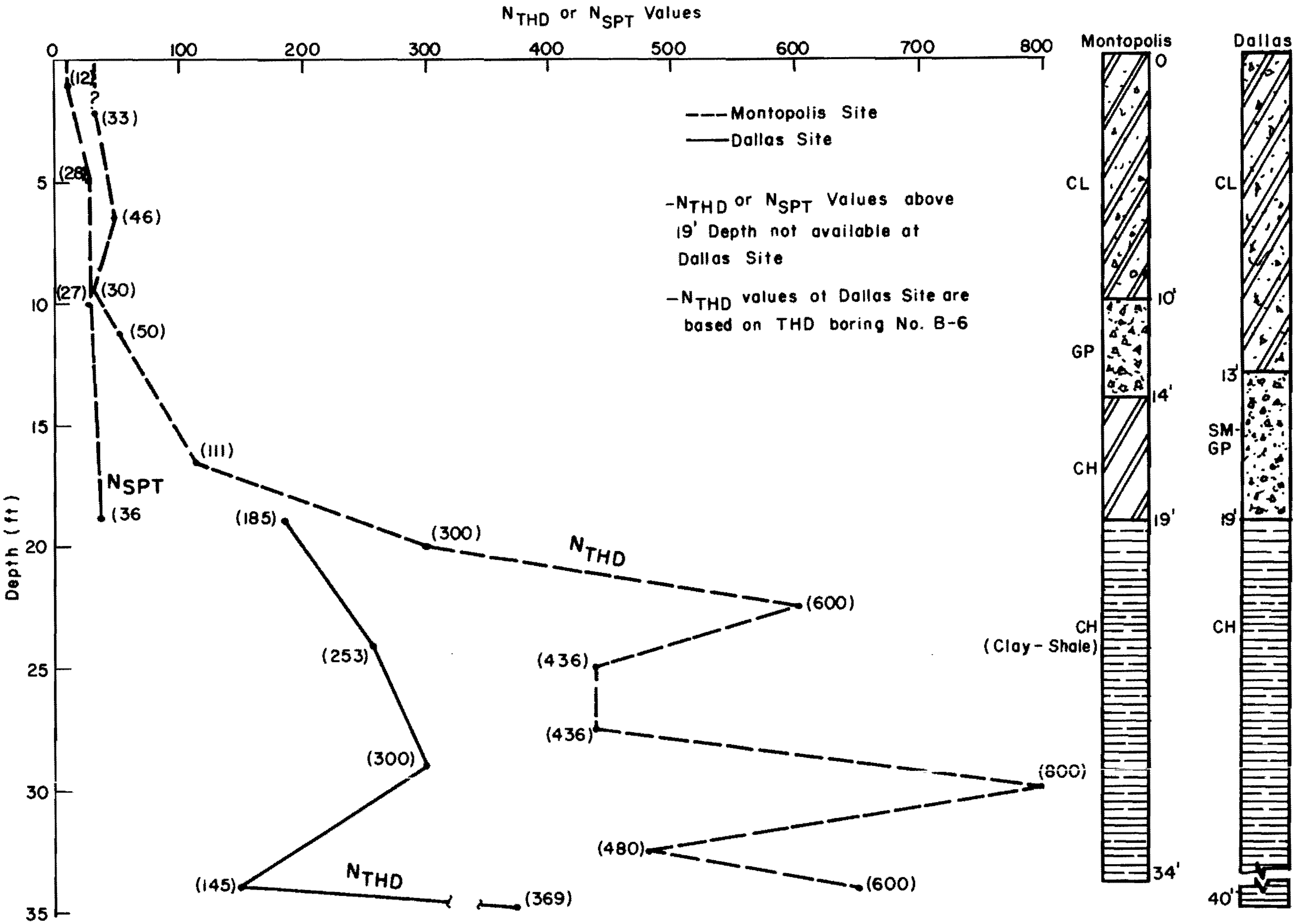


Fig 4.10. Variation of N_{THD}- and N_{SPT}-values at test sites.

kelly of a heavy drill rig used for installing the drilled shaft. It was stated by the rig owners that the kelly would sustain an upward thrust of up to 15 tons. In order to measure the vertical load accurately, a calibrated pressure transducer was attached to the side of the jack. A 12-in. long steel rod, having a cone of the dimensions of a Dutch cone, was specially fabricated to be attached to the bottom of the hydraulic jack which had a threaded recess in it to receive the threaded top of the steel rod. The details of the steel rod are shown in Fig 4.11. Figure 4.12 shows the assembly ready for use. A similar threaded road, attachable to the bottom of the jack, was fabricated for conducting plate load tests. The details of the rod for the plate load tests are shown in Fig 4.13.

At the Montopolis Site, a 48-in. diameter hole was drilled to a depth of 16 ft below the ground surface to expose the underlying shale. The hole was augered without slurry and was quickly cased with a heavy steel pipe to avoid any caving of the upper water-bearing sand and gravel layer encountered during drilling. The bottom was hand-cleaned by a workman who was lowered with the help of a line operated from the drill rig. The cone was jacked against the exposed shale and readings of the pressure transducer were noted with a strain indicator for every 1/4-in. penetration. As the cone penetrated the soil, two important points were noted. First, it was almost impossible to keep the steel rod plumb because the noticeable horizontal planes of weakness in the shale caused chunks of shale to break loose. These chunks were usually about 2 in. wide and were less than 1/2 in. thick. This caused the cone to "slide" sideways: lateral displacement of the cone from its original position in excess of 2 in. was noticed, in some cases, at the end of the tests. Second, radial cracks, with vertical sides, appeared as the cone penetrated into the shale.

Static Dutch cone tests were also performed on the shale at Dallas on August 15, 1975, using the same procedures as described earlier. During these tests the two points regarding the behavior of shale during the penetration of the cone, mentioned in the preceding paragraph, were also noted.

The plate load tests were conducted in the same manner as the static Dutch cone tests. The noticeable difference in this case was that the width of the chunks of shale that broke loose during penetration of the plate were much greater than those in the cone tests. They were as wide as 6 to 8 in. Lateral displacements occurred in these tests also.

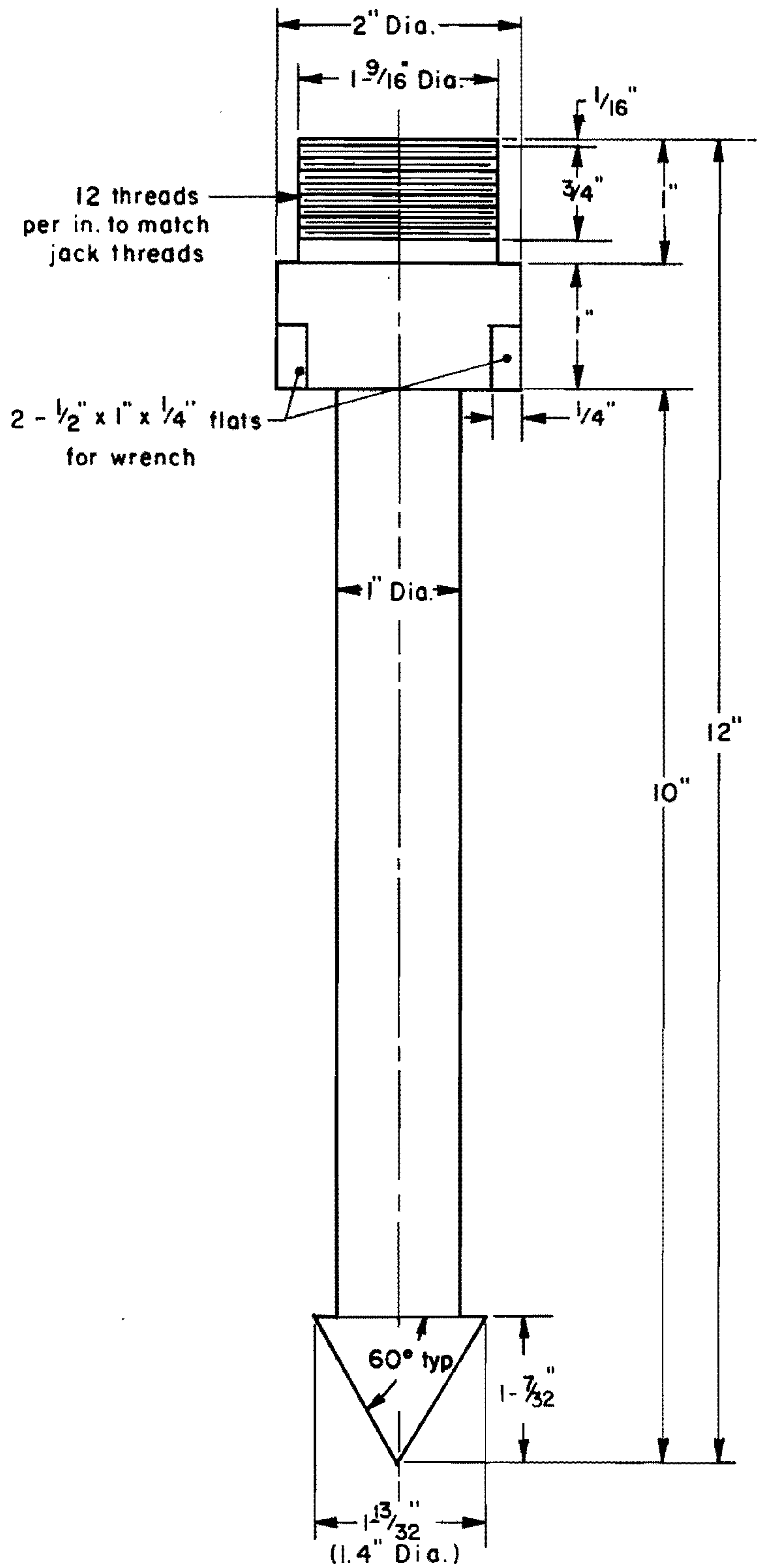


Fig 4.11. Dutch cone device used for in-hole tests.

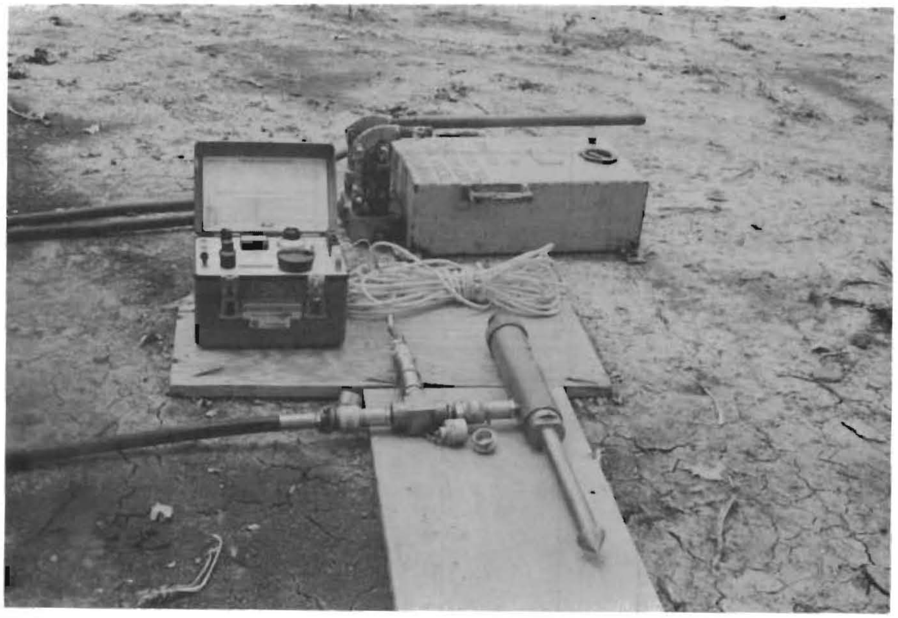


Fig 4.12. Dutch cone device ready for use at site.

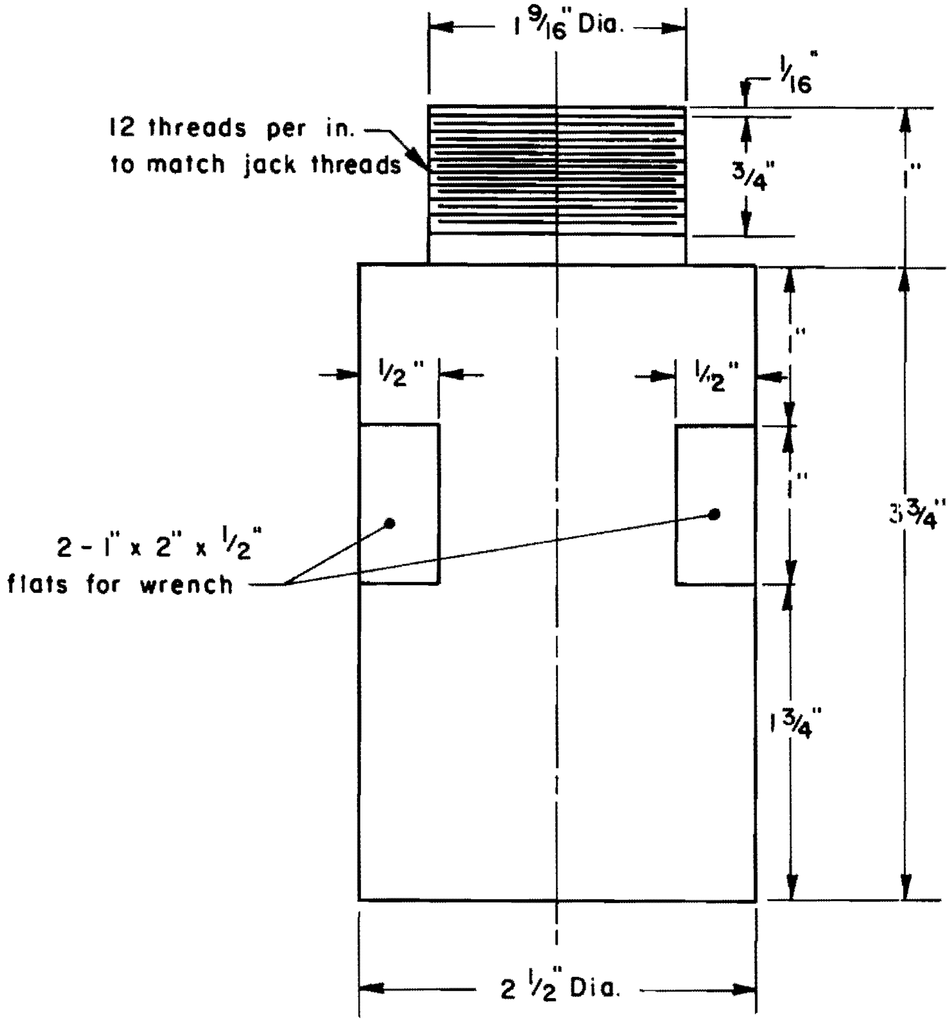


Fig 4.13. Plate load device used for in-hole tests.

The results of the static Dutch cone tests and plate load tests are included in Appendix B.

Laboratory Tests Run to Estimate Shear Strength of Soils

Undisturbed soil samples from the Montopolis Site were tested in the Geotechnical Engineering Laboratories of The University of Texas at Austin. The samples remained in a temperature and humidity-controlled room for about 6 months before testing.

Q-tests, also called "quick tests" or "unconsolidated undrained tests," were run on 3-in. diameter samples about 6 in. long. Triaxial cells were employed that used a self-compensating mercury control with a pressure range from 0 to 120 psi. Details of this type of apparatus are described by Bishop and Henkel (1964). Figure 4.14 shows a picture of the system ready for use. The rate of strain for all these undrained tests was 2 percent per minute. Various techniques, including the use of a band saw and wire saw, were tried for trimming the samples; the most satisfactory results came from the use of a fine-toothed household knife. Most of the samples were tested at a confining pressure of 10 psi, which was close to the vertical overburden pressure. Confining pressures of 20, 60, and 100 psi were also used to study the effect of variation of confining pressure on the shape of the Q-envelope. Whenever possible two samples from the same core were tested at two different confining pressures. It was found that confining pressure in excess of 10 psi had little or no effect on the shape of the Q-envelope. An increase of about 10 percent in the value of $\sigma_1 - \sigma_3$ at failure was observed in a few cases when the confining pressure was increased from 10 to 60 psi. In another case a sample tested at a confining pressure of 60 psi showed a considerably lower value of $\sigma_1 - \sigma_3$ at failure in comparison with the one tested at a confining pressure of 10 psi, although both the samples were obtained from the same core. It was concluded that the Q-envelope for this shale was essentially horizontal. The stress-strain curves obtained during triaxial tests are shown in Figs 4.15 to 4.18. A few important details were noticed during testing of the clay and shale specimens as indicated next.

In many samples, moisture was visible when the aluminum foil was unwrapped. In all cases where this happened, about a 1/8-in. to 1/4-in. thick circumferential band on the outer skin of the specimen was found to be

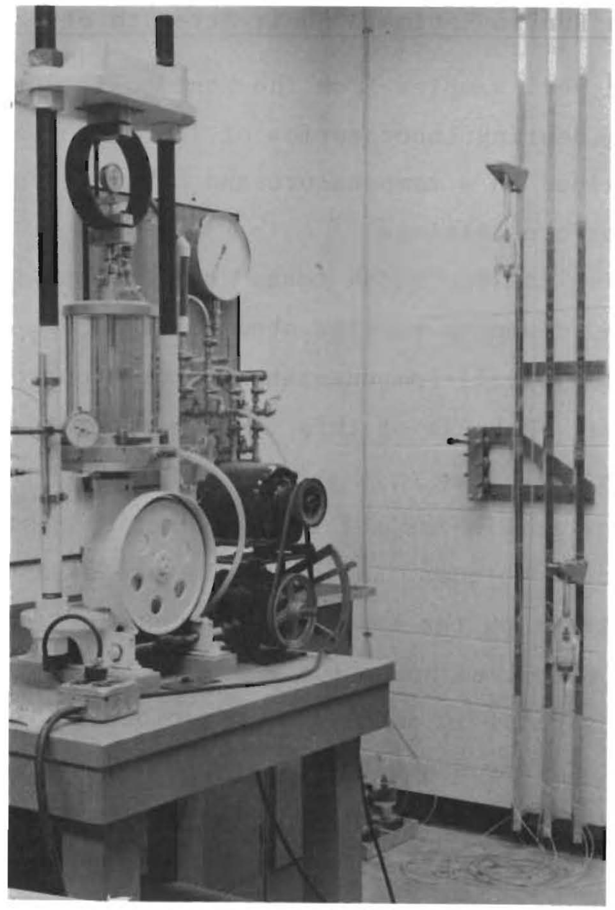


Fig 4.14. Triaxial test set-up ready for use.

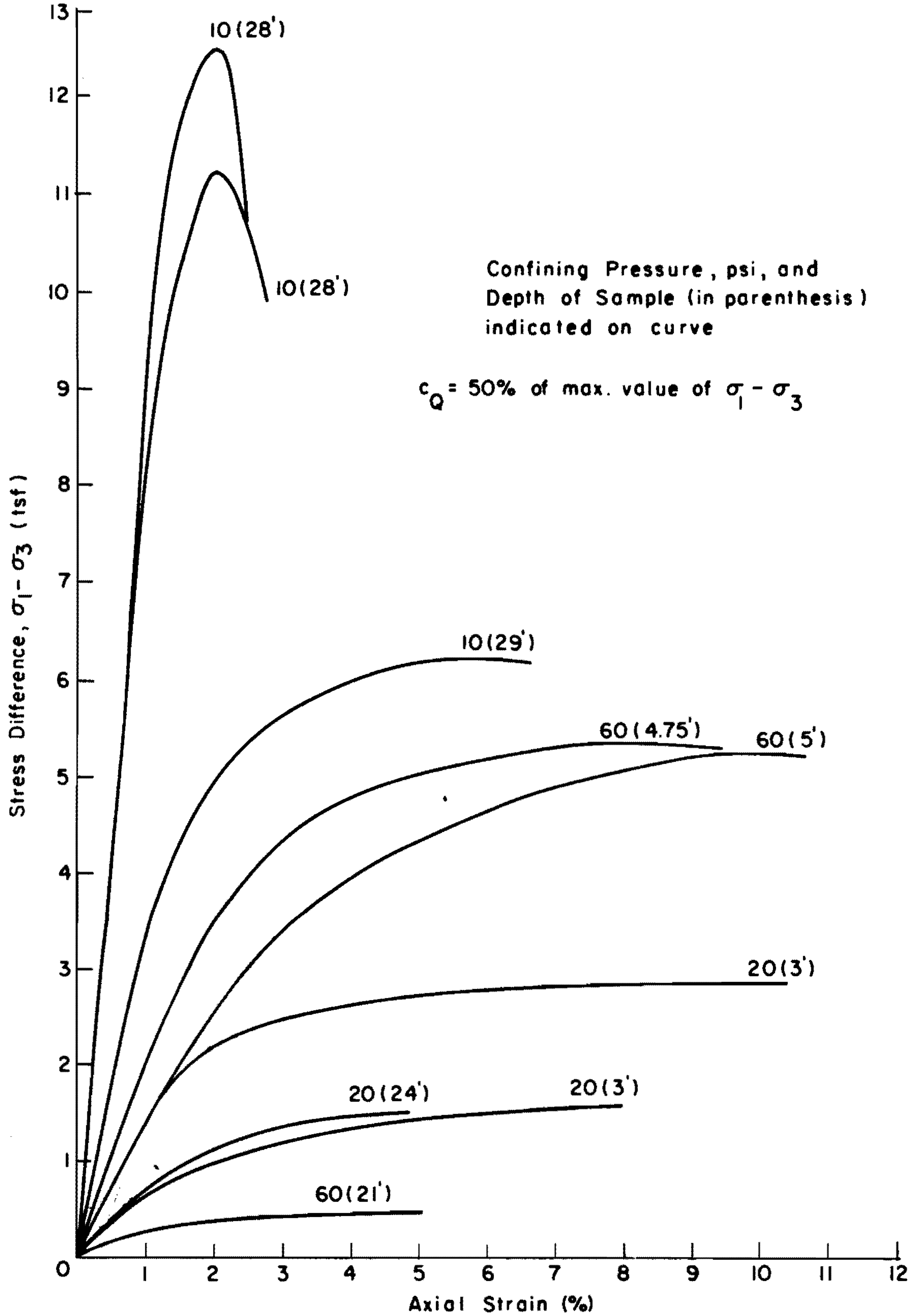


Fig 4.15. Stress-strain curves from triaxial tests.

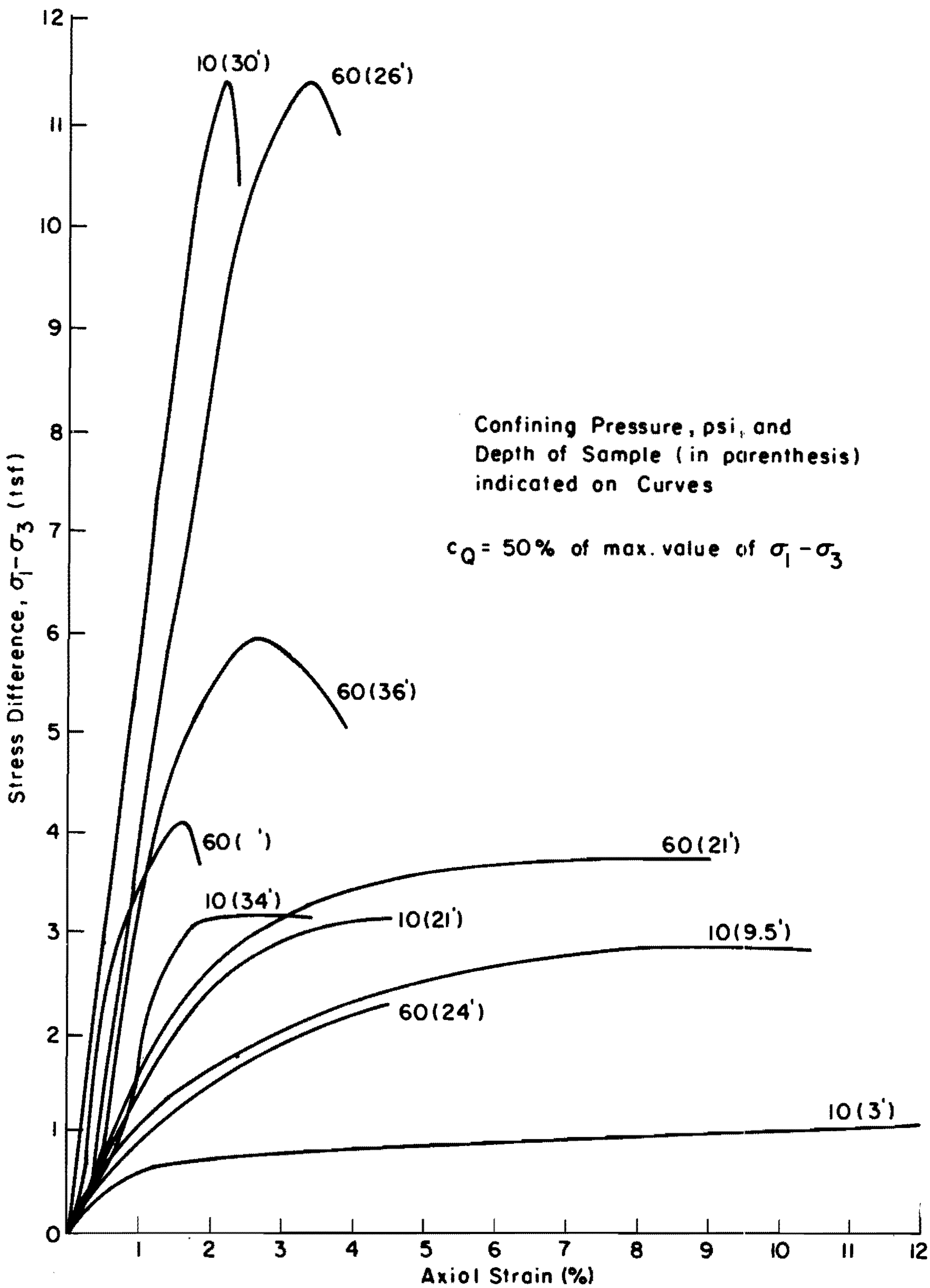


Fig 4.16. Stress-strain curves from triaxial tests.

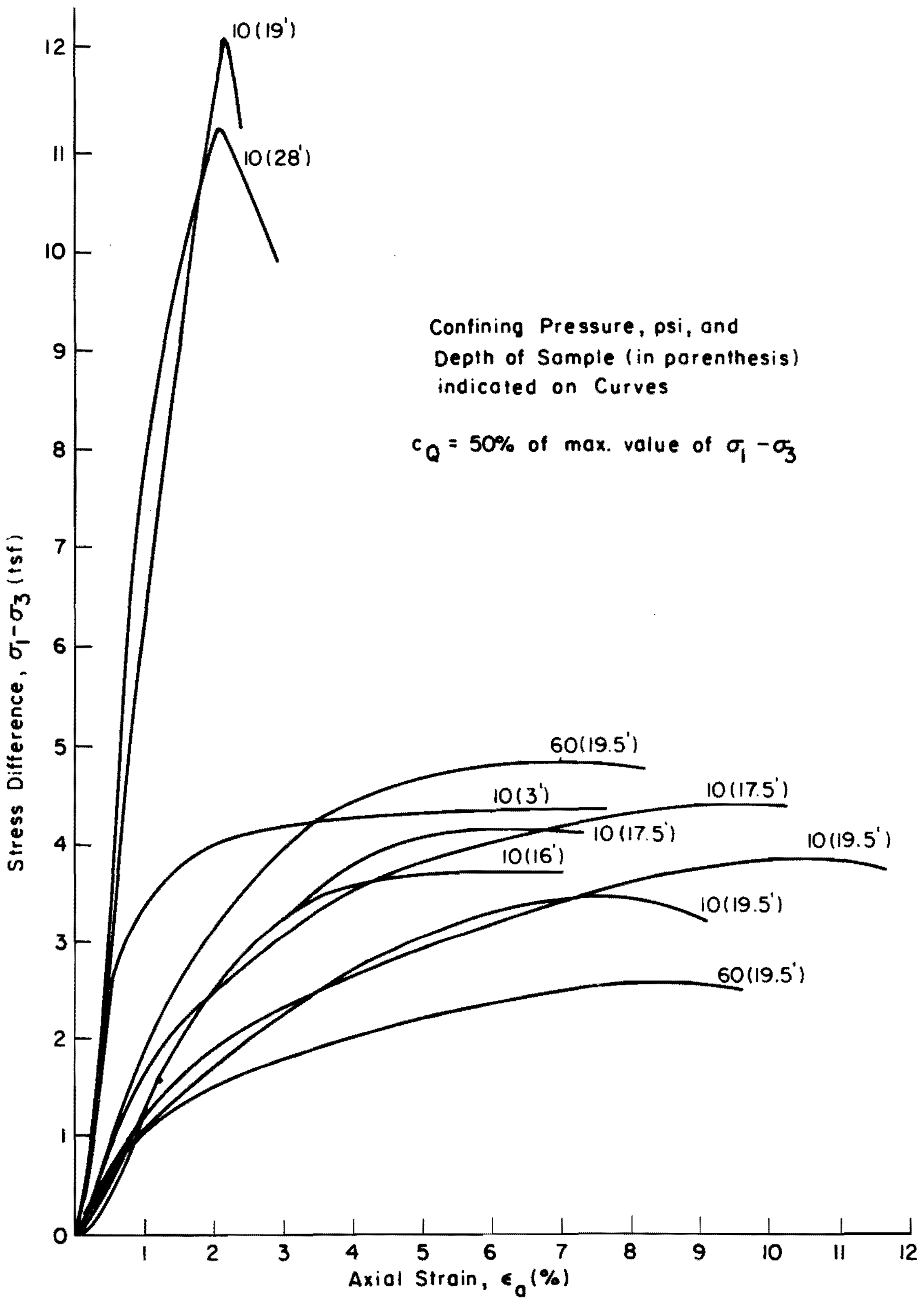


Fig 4.17. Stress-strain curves from triaxial tests.

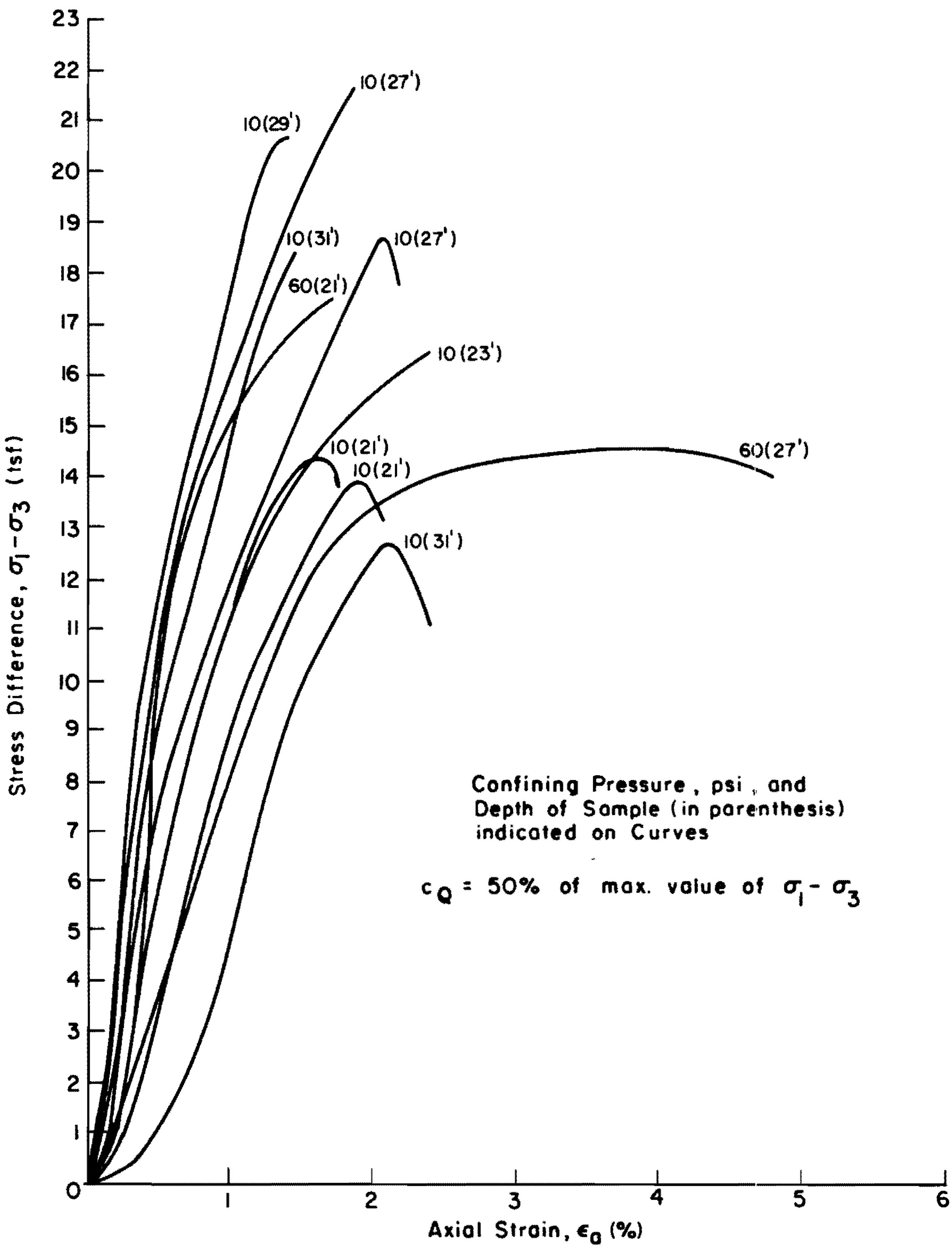


Fig 4.18. Stress-strain curves from triaxial tests.

slightly darker than the inner part of the core. Besides, this band was also judged to be softer than the inner part of the core during trimming. Generally, it was difficult to get two 6-in. long specimens from one core, although the core itself was 18 in. to 24 in. long. This happened because most shale specimens fissured along the planes of weakness, almost normal to the axis of drilling, either immediately after the core was extruded from the inner barrel at the site or perhaps during storage. The shale specimen that gave the highest shear strength value was noticeably free from any moisture seepage effects. The planes of weakness, whenever noticed, had a shiny surface that was slightly concoidal. In some specimens very fine grains of free limestone were seen sticking to the concoidal surface. All shale samples on which acid was dropped, showed effervescence.

Classification Methods Used for Clays and Shales

All clay samples were classified in accordance with the Unified Soil Classification System as shown on the plasticity chart in Fig 4.19. The soil was suitably crushed and sieved before conducting liquid limit and plastic limit tests.

As of now, there is no standard method of classifying shales. Wide difference of opinion exists with regard to the meaning and interpretation of various terms that are commonly used to describe different types of shales. Mead (1936) separated shales into two groups, which he called "soil-like shales" and "rock-like shales." Underwood (1967) made a study of data reported by others on the properties of shales. He suggested a procedure for engineering evaluation of shales, but he pointed out that a completely satisfactory engineering classification for shales was difficult to develop due to imprecise data and unstandardized testing methods. Morgenstern and Eigenbrod (1974) proposed a method of classifying argillaceous soils and rocks based on the idea that unconfined samples of soil will disintegrate when exposed in an unconfined manner to water, while unconfined samples of rock will not. They proposed two tests to explore systematically the spectrum of behavior from "clays" to "mudstones." The first test consists of determining the undrained shear strength of the undisturbed material. The second test, called the "Standard Compression Softening Test," determines the loss of shear strength after immersing the unconfined undisturbed specimen in water. If the shear strength is less than 250 psi and loss of shear strength in the compression softening test is greater than 60 percent, the material is classified as "clay." Further, if t_{50}, the softening time for 50 percent loss of original shear strength,

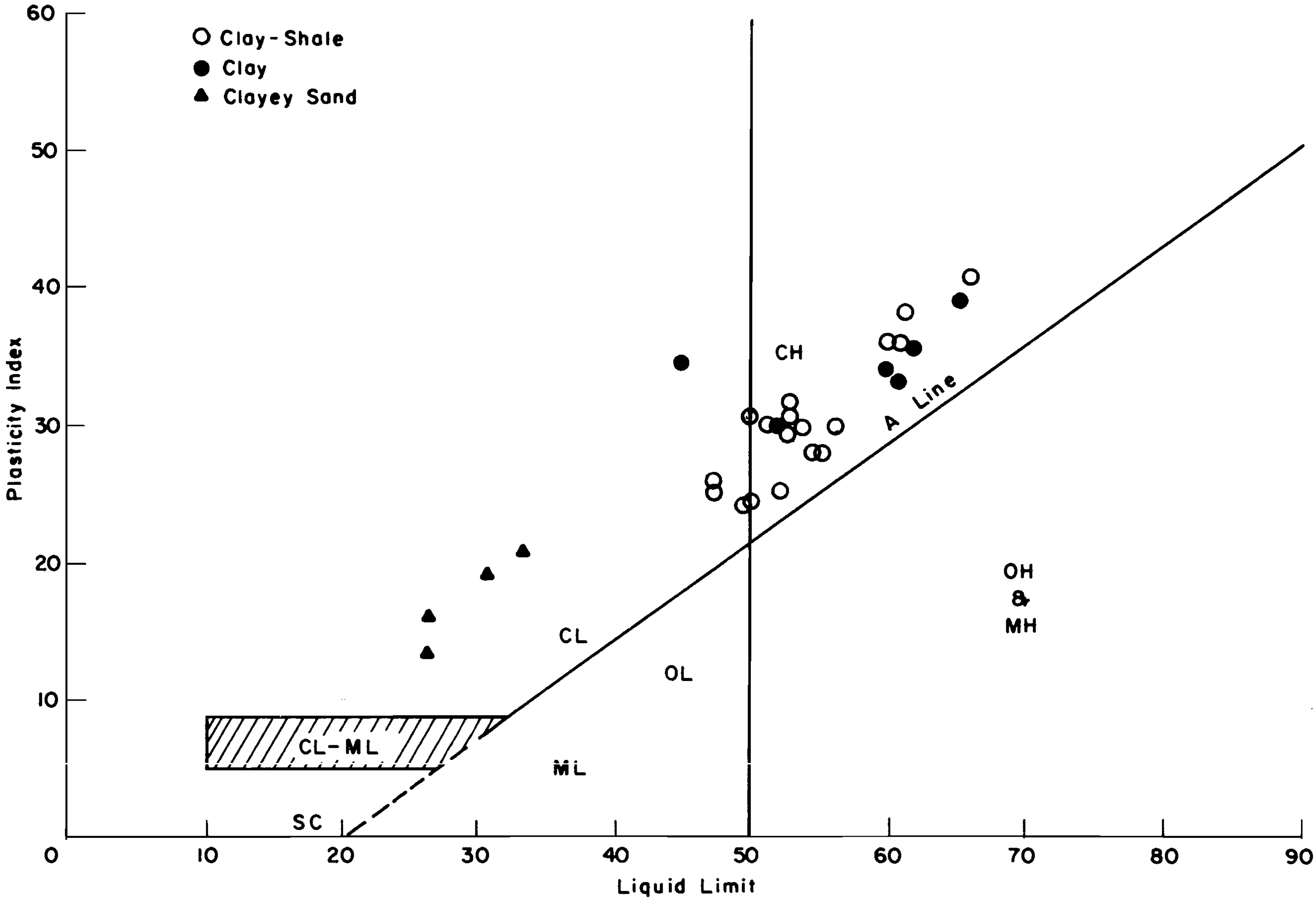


Fig 4.19. Classification of soil samples on plasticity chart.

is 1 hour or less, the clay is classified as "medium to soft," whereas the clay is classified as "stiff" for t_{50} less than 1 day, and as "hard clay" or "clay-shale" if t_{50} is greater than 1 day. Materials with shear strengths greater than 250 psi and loss of shear strength, in compression softening tests, of less than 40 percent, are classified as "mudstones." They are further classified as "claystones" and "siltstones."

In this study, the approach suggested by Morgenstern and Eigenbrod was used to classify the shale at the Montopolis Site. Only two tests were made due to limited numbers of samples available. These tests indicated that the shale at the Montopolis Site is a "clay-shale." Classification tests were not done for the shale at the Dallas Site; however, it is believed that that shale is also a "clay-shale."

Limitations of Shear Strength Information Obtained

The shear strength data obtained for this study by field and laboratory tests have a few limitations.

Probably, laboratory tests indicated shear strengths less than the in-situ shear strengths, mainly because of the possible softening due to the free moisture that was noticed when the samples were unwrapped. The extent to which this reduction in shear strength occurred is not known. A comparison with the Dutch cone and plate load tests indicates that the shear strength may have been reduced by as much as 40 percent during the storage period of about 6 months.

There are several limitations of the dynamic penetration resistance tests. The most important is the fact that reliable correlations between dynamic penetration-resistance values and the shear strength of clays and shales does not exist. Some initiative has been taken in this direction by the Texas Transportation Institute, who recently sponsored a research study for cohesive soils. Results of the study were reported by Hamoudi, Coyle, and Bartoskewitz (1974). Similar studies for shales are not known. Therefore, in this study the results of penetration tests have been supplemented with the results of triaxial tests and in-hole tests to establish correlations between shear strength and load transfer, as will be explained in later chapters. It may be pointed out that triaxial tests and in-hole tests are also subject to certain limitations, although they are believed to approximate in-situ shear strength of soil more accurately than many other tests.

CHAPTER 5. INSTRUMENTATION

Basic Ideas Used for Instrumentation

It was pointed out in Chapter 3 that by using Equations (3.4), (3.5), and (3.6) load transfer information can be obtained at any depth along the shaft. Measurements of the following quantities are necessary to use the equations:

(1) Load applied at the top of the shaft, Q_T ;

(2) Movement at the top of the shaft, w_o ;

(3) Slope of the load distribution curve at depth z , $\frac{dQ_z}{dz}$;

(4) Effective cross sectional area of the shaft concrete, A ; and

(5) Modulus of elasticity of concrete, E .

The first three quantities mentioned above can be measured by two interdependent instrumentation systems. The first system is designed to measure loads and movements at the top of the shaft. The second system is set up to measure loads at selected locations. Load distribution curves are obtained by statistical methods to "best fit" the load data obtained at discrete points. Then, the slope of the load distribution curve at any depth is calculated mathematically.

Usually the diameter of the drilled hole is assumed 1 in. larger than the nominal diameter of the auger used to drill the hole. This assumption was made in the present study to estimate the effective cross sectional area of the shaft. The area of vertical steel was transformed into equivalent concrete area by multiplying the area of vertical steel by the ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete.

The modulus of elasticity of concrete, E , was determined indirectly from the compressive strength of concrete by using the following equation suggested by the American Concrete Institute (1971):

$$E \text{ (psi)} = 57{,}000 \sqrt{f_c'} \tag{5.1}$$

in which f_c' is the compressive strength, in psi, of 12-in. long and 6-in. diameter concrete cylinders cured under water for 28 days. The quantity f_c' was determined in a laboratory by testing concrete cylinders using the same concrete as used for building the test shafts. The value of E determined according to the above approach assumes linear stress-strain relationship at all levels of stress. At stresses below $0.5\ f_c'$, this assumption does not introduce any significant error. For all the shafts tested in this study, the maximum stress in concrete did not exceed $0.5\ f_c'$.

After getting A and E, the following two types of measurements were necessary to obtain the remaining quantities:

(1) Measurement of loads and movements at the top of the shaft; and
(2) Measurement of load at selected locations within the shaft.

The details of instrumentation used for these measurements are given in the remaining parts of this chapter.

Measurements of Loads and Movements at the Top of the Shaft

Axial loads were applied to the top of the shaft by means of two hydraulic jacks supplied by the Texas Highway Department. Each jack was capable of applying a load of up to 500 tons. The schematic arrangement of using these jacks is shown in Fig 5.1. Pressure was measured in the hydraulic oil lines, running between the air-driven hydraulic pump and the hydraulic jack, and was converted into applied load by use of a calibration chart supplied by the manufacturer of the hydraulic jacks. Two Bourdon pressure gages were used to measure the pressure in the hydraulic oil lines. These gages had pressure measurement ranges of 0 - 5,000 psi and 0 - 20,000 psi. In the field, the hydraulic pressure needed to apply a particular load through the jacks was determined from the manufacturer's chart. In addition to the Bourdon gages mentioned above, an electrical pressure transducer was attached to the hydraulic oil line for more accurate determination of the applied load. The pressure transducer was calibrated in the laboratory. It allowed measurements of applied loads to a sensitivity of about 0.1 ton. A trailer-mounted air compressor was used to supply compressed air for driving the hydraulic pump. This compressor, powered by gasoline, was capable of delivering 100 cubic feet of air at 120 psi, and was adequate to drive the

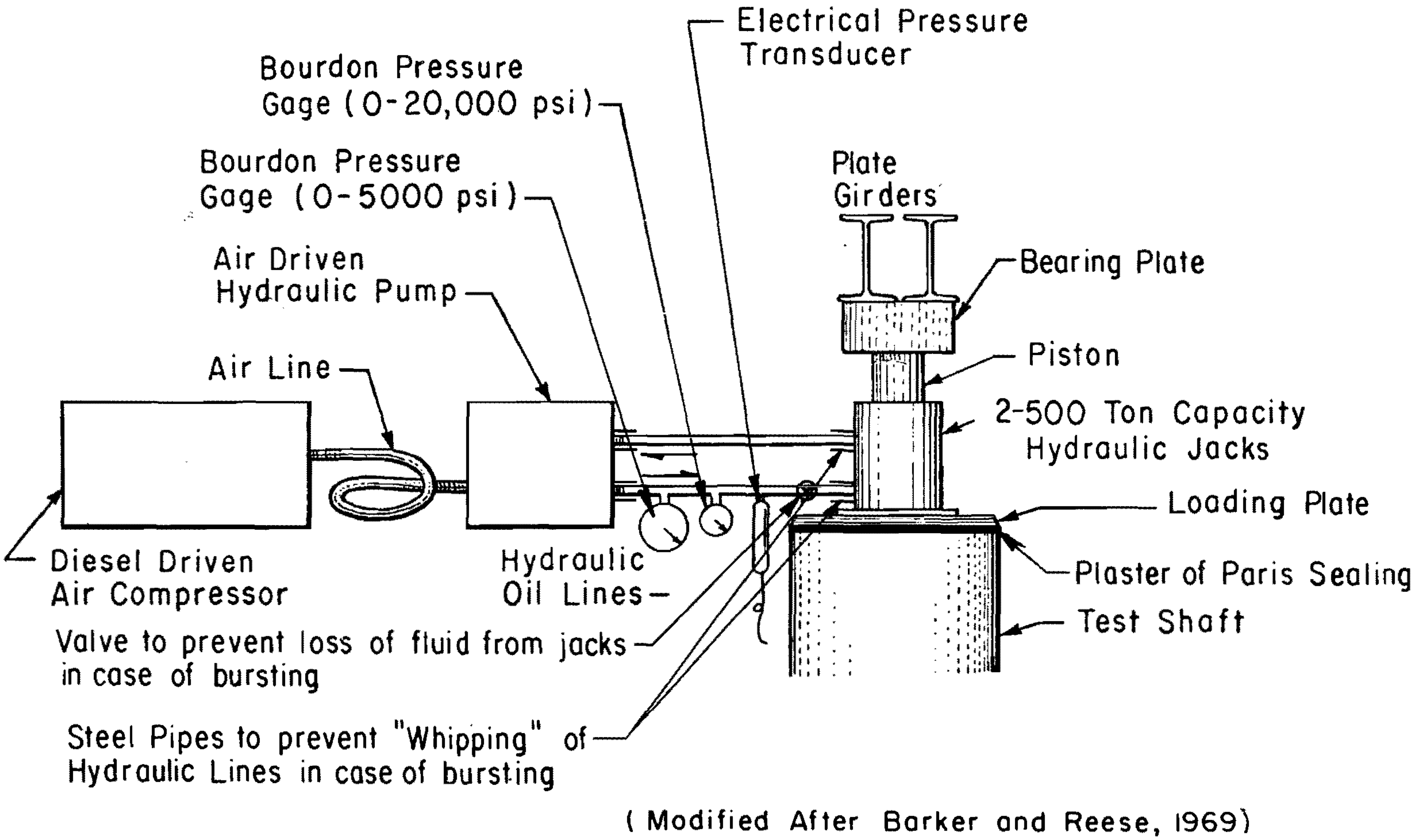


Fig 5.1. Schematic diagram of loading system.

pump at 90 psi. The system described above worked satisfactorily to measure and apply vertical loads at the top of a test shaft.

The vertical movements, near the top of a test shaft, were measured by dial indicators mounted on a stationary reference frame made of timber beams arranged suitably around the shaft without touching it. Aluminum angles were glued to the shaft by means of an epoxy resin. They were positioned at equal spacing along the circumference of the shaft at a distance of 18 in. vertically below the top of the shaft. The projecting leg of each angle supported the movable stem of a dial indicator. The reference frame itself was supported laterally and vertically at some points which were at least 1C ft away from the vertical axis of the test shaft. Each dial gage could measure a movement as small as 0.001 in. and had a travel of 2 in. Any dial gage was reset when the measured settlement appeared to approach the full extent of its travel. Three dial gages were used at the Montopolis Site, and two were used at the Dallas Site. All dial gages worked satisfactorily throughout their use.

Measurements of Loads at Selected Locations Within the Shaft

Different types of instrumentation systems capable of measuring loads in drilled shafts have been discussed in some detail by Barker and Reese (1969) and O'Neill and Reese (1970). During the past five years the Mustran cell system, developed and fabricated at The University of Texas at Austin, has been successfully used for measurements of axial loads in drilled shafts. The term Mustran is an abbreviation for "Multiplying Strain Transducer." The statement regarding successful use of a Mustran cell system is based upon the experience reported by O'Neill and Reese (1970), Touma and Reese (1972), Engeling and Reese (1974), and Wooley and Reese (1974). These reporters carried out research on axially loaded drilled shafts at The University of Texas at Austin under the sponsorship of the Center for Highway Research. It was found convenient to use the Mustran cell system for this study as well, since a better load measurement system for drilled shafts has not been reported so far. The Mustran cell system itself has undergone a few minor modifications aimed at getting better response since its inception in 1969. The theory of its use, as reported by Barker and Reese (1969), remains unchanged by modifications.

The components of a typical Mustran cell, used at the Montopolis and Dallas Sites, are shown in Fig 5.2. A mild steel bar, tightly screwed at each end to a flanged cap, provides the surface on which the strain gages are mounted. The flanges of the caps help to bond the Mustran cell to concrete. The butting surfaces of the steel bar and the cell caps are machined to ensure good bearing. A rubber hose, which slips tightly over the cell caps, is clamped to the cell caps at both ends. This arrangement does not allow the concrete to come in contact with the strain gages, and makes it possible to keep the strain gage in a dry environment, as explained below. It is well known that the electrical resistance of any conductor changes erratically in the presence of moisture. Since the measurements of the electrical resistance of the strain gages are the key to measurement of the vertical loads, the strain gages must remain dry. Therefore, in addition to the hose, dry nitrogen under pressure is admitted into the space between the steel rod, the hose, and the cell caps. The lead cable of each Mustran cell is brought to a manifold where the four wires within the lead cable are connected to a properly labeled socket. The socket is provided to later plug in the leads from the data logging system which registers strain measurements during the test. One manifold is used for one test shaft. Each manifold houses two socket boards and each socket board contains several sockets. The manifold is made air tight and nitrogen is admitted into it through an inlet. Thus the manifold acts as the central unit to distribute pressurized nitrogen to each Mustran cell through the lead cable. The picture of a Mustran cell with its lead cable and a manifold ready to be assembled are shown in Figs 5.3 and 5.4 respectively. The shcematic arrangement of pressurizing Mustran cells through the manifold is shown in Fig 5.5. The Mustran cells, connected to the manifold, are placed in a box and transported to the site.

The Mustran cells were clamped to the main reinforcement with their axes parallel to the axis of the test shaft. One leg of a steel angle, made from a 1/2-in. wide and 1/8-in. thick steel strip, is connected to the flange of each cell cap. The outstanding leg of each angle is attached to the main reinforcing steel by means of a hose clamp. This arrangement is shown in Fig 5.6. At any location, two or four Mustran cells were set symmetrically on a circle concentric with the circumference of the shaft. Theoretically, the effects of symmetrical bending on axial strain readings can be cancelled by averaging the readings of two cells placed symmetrically opposite about the

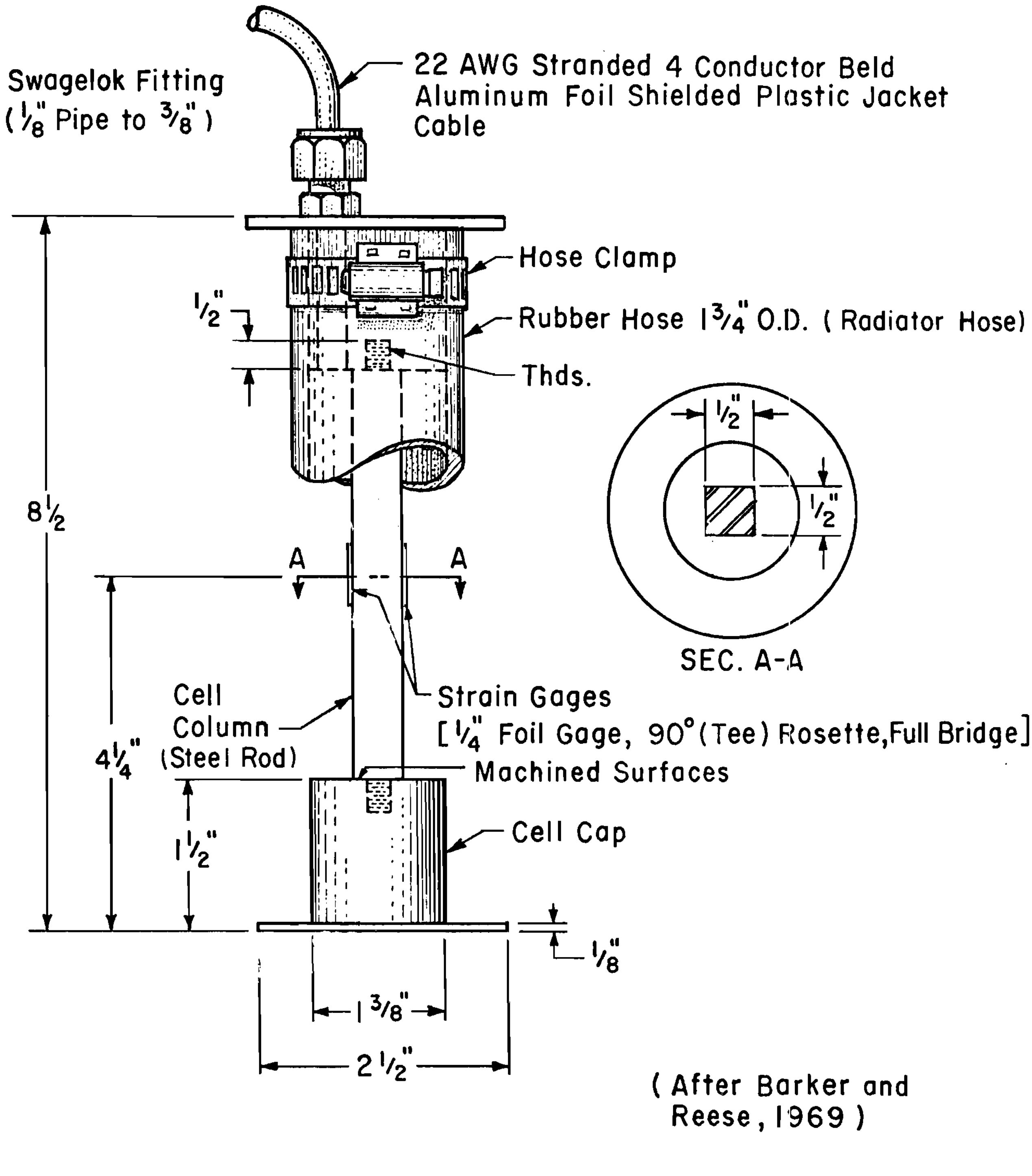


Fig 5.2. Components of a Mustran cell.

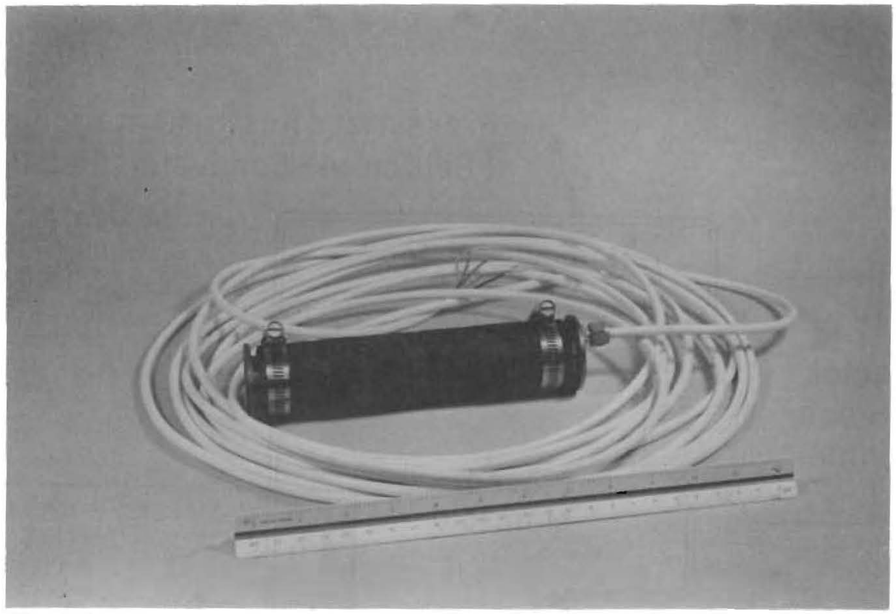


Fig 5.3. An assembled Mustran cell with its lead cable.

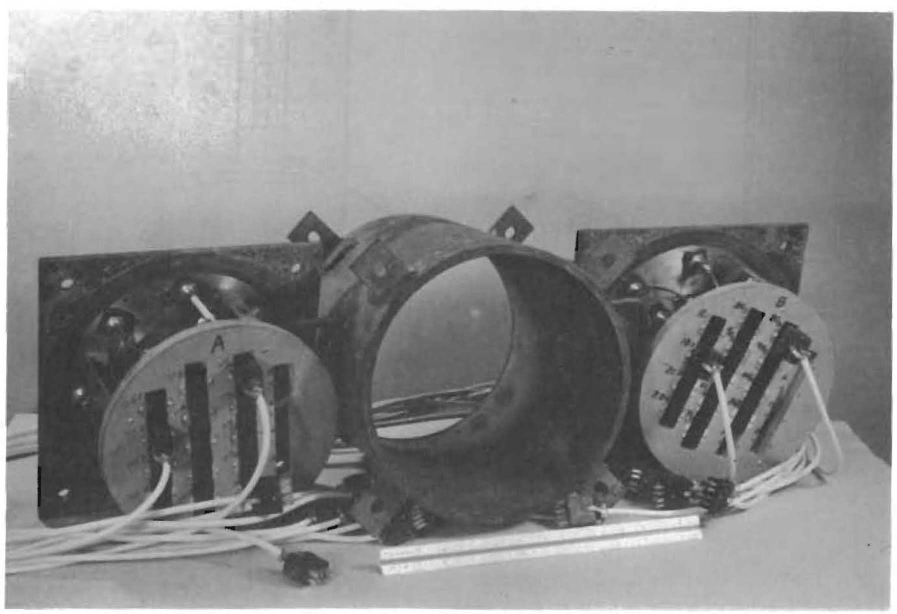


Fig 5.4. Picture of an unassembled manifold.

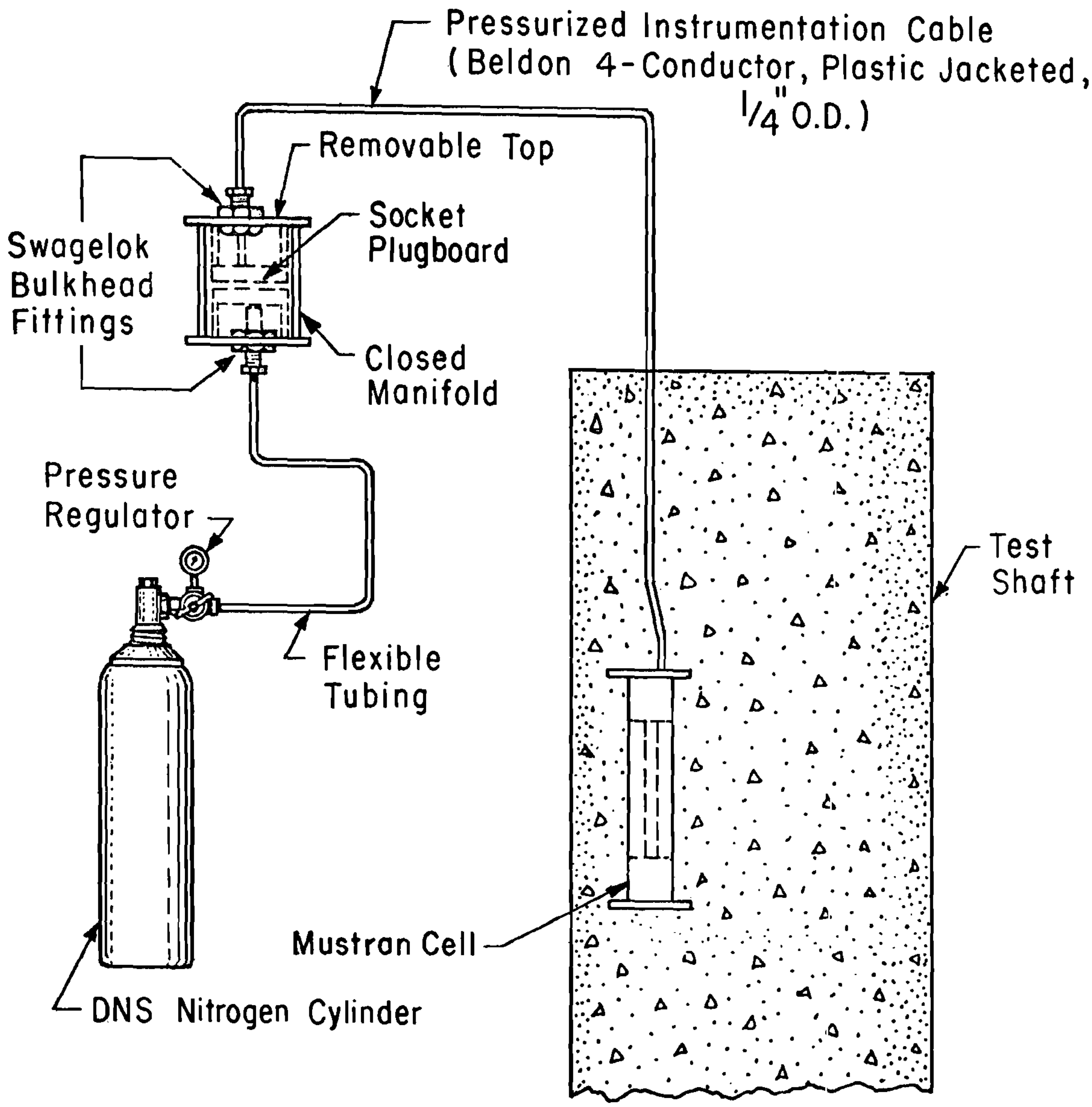


Fig 5.5. Schematic arrangement of pressurizing a Mustran cell (after Barker and Reese, 1970).

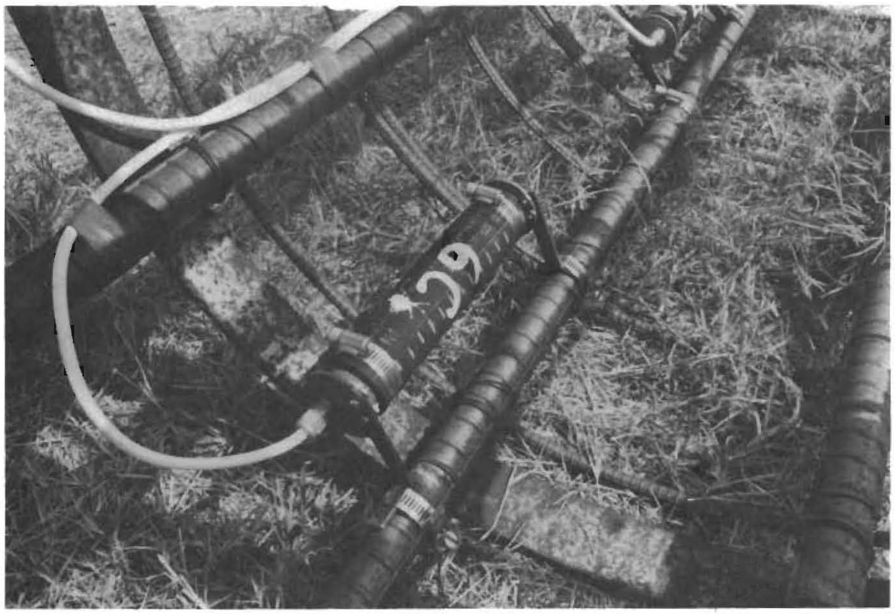


Fig 5.6. Arrangement of attaching a Mustran cell to the main reinforcement.

axis of bending. Therefore, adjacent cells were spaced 180 or 90 degrees along the circle, depending upon whether two or four cells were used at that cell location.

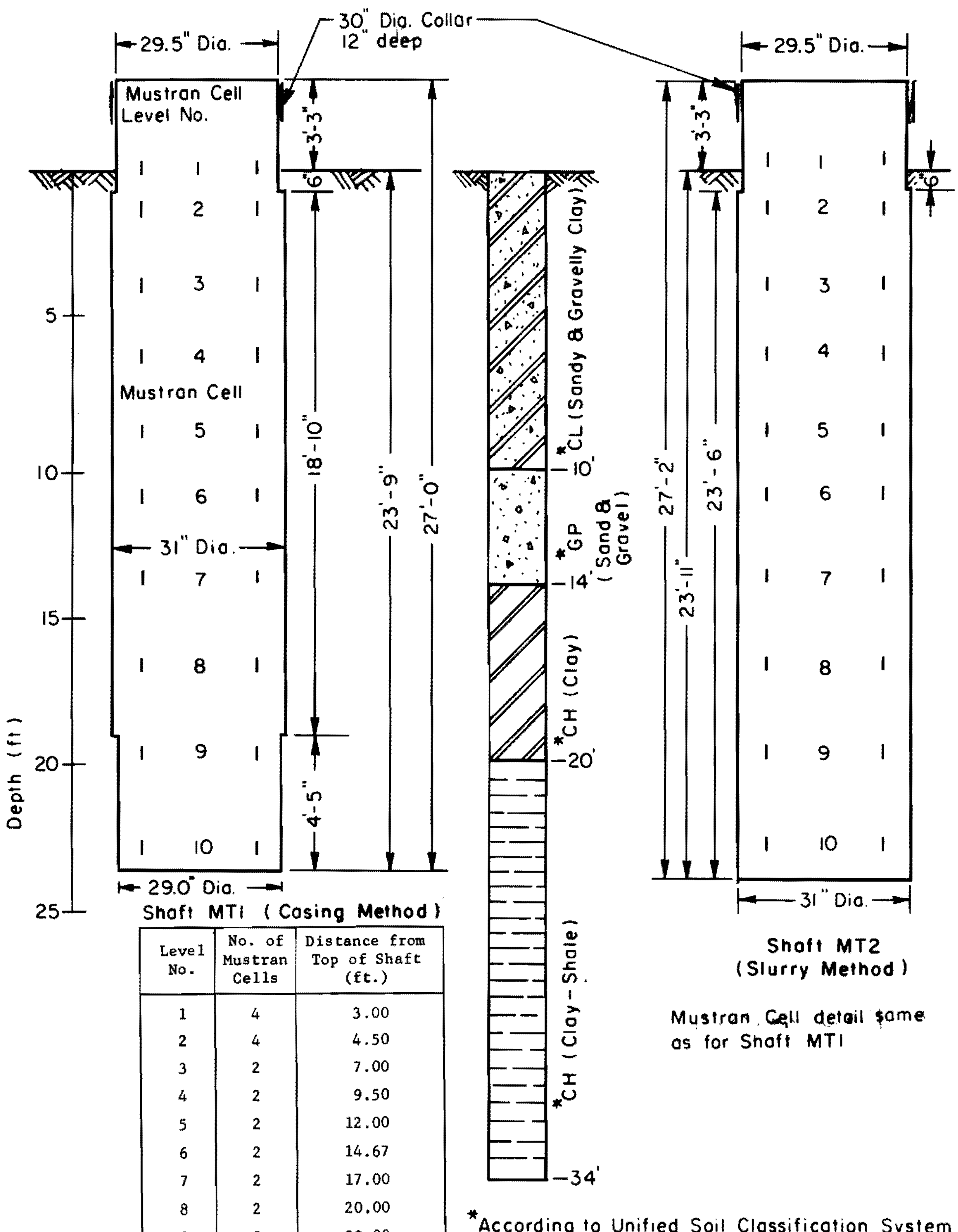
The steel cage was laid horizontally in the field. Mustran cells were mounted at predetermined locations, proceeding from the bottom of the cage. All Mustran cells and their sockets in the manifold were labeled prior to their shipment to the site. This helped in the identification and positioning of each Mustran cell in the field. The lead cables of the Mustran cells, cut to predetermined lengths, corresponding to each side of the manifold, were tied together near the top of the cage, thus providing two easy-to-handle clusters of lead cables, each leading to a socket board of the manifold. The manifold was tightly closed and securely attached to the cage, ready to be installed in its final position.

The Mustran cell system, described in the preceding few paragraphs, provided the instrumentation to obtain strain readings at discrete locations within a test shaft. With the knowledge of the geometry of the test shaft and its material properties, the strain readings were converted into loads by analytical procedures mentioned earlier and described later in detail in Chapter 8. The strain readings were recorded by a data logging system described next. The relative positions of soil layers and the Mustran cells are shown in Figs 5.7 to 5.9 for all the test shafts.

Data Logging System Used for Tests

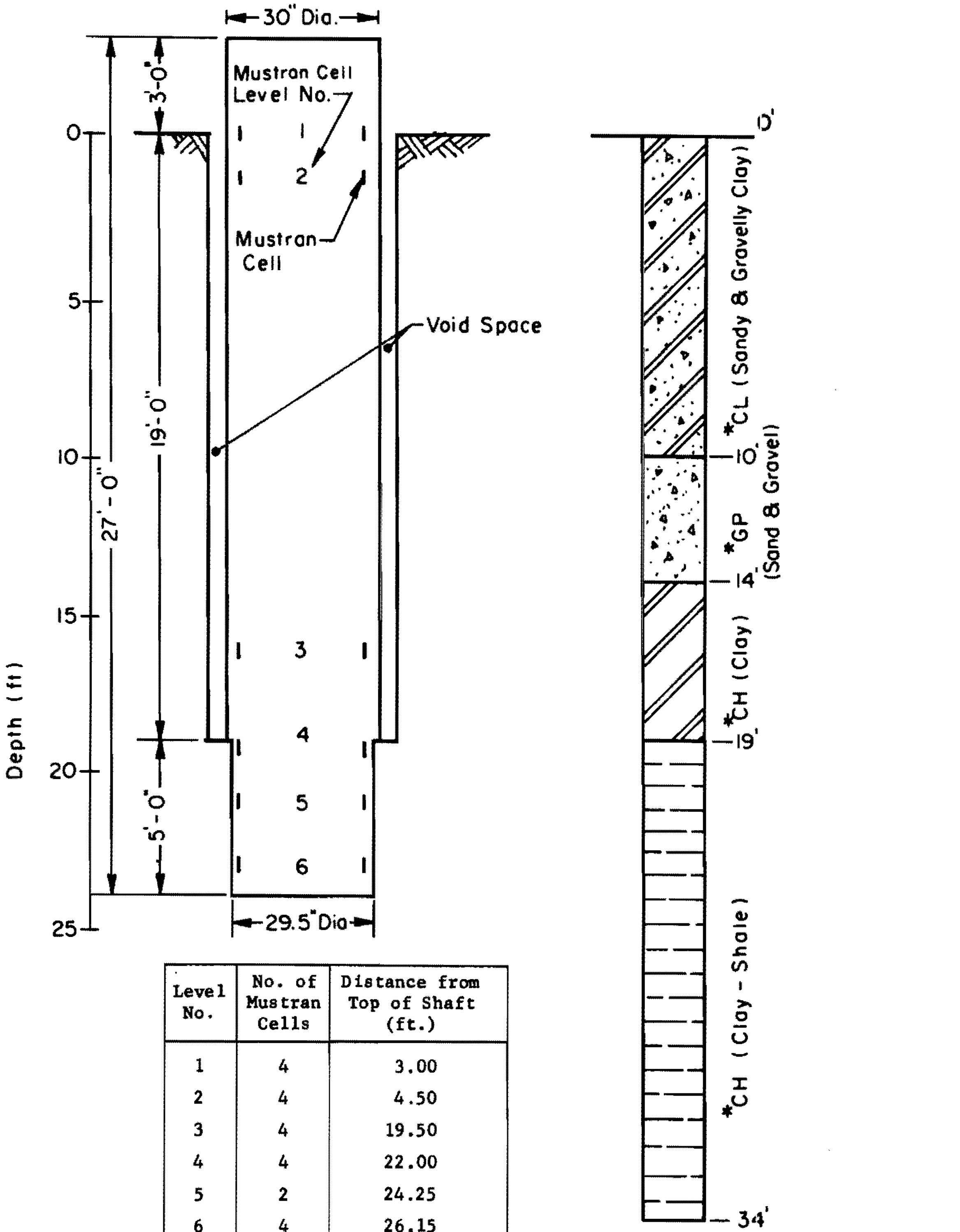
The data logging system used for tests on drilled shafts records the voltage output from the Wheatstone bridge formed by the strain gages of a Mustran cell. The output is measured in microvolts. In order to convert this information into load in the pile, the computer program DARES was employed. This program is described in detail by Barker and Reese (1969), and its application is discussed in Chapter 8.

The data logging system used for this study is shown schematically in Fig 5.10. A gasoline-powered 3-kilowatt portable AC generator served as the power source at the site. The Honeywell Data Logging System requires a fluctuation-free, 110-volt AC supply for accurate readings. To achieve this, the 110 volts of AC power were first converted into a 12-volt DC supply through a battery charger, and then reconverted into a stable 110-volt AC supply through an inverter. Two 12-volt batteries were connected in parallel with the battery charger and the inverter to serve as a standby power supply



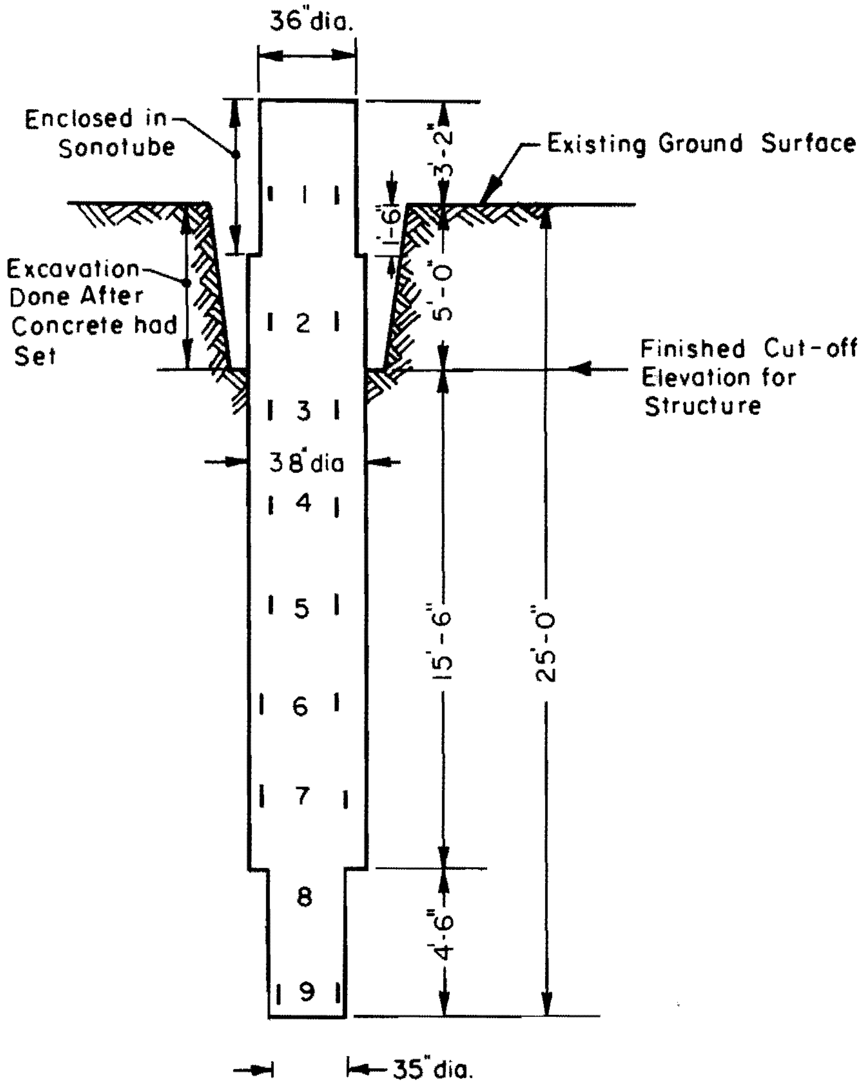
Level No.	No. of Mustran Cells	Distance from Top of Shaft (ft.)
1	4	3.00
2	4	4.50
3	2	7.00
4	2	9.50
5	2	12.00
6	2	14.67
7	2	17.00
8	2	20.00
9	2	23.00
10	4	26.50

Fig 5.7. Soil profile and instrumentation at shafts MT1 and MT2 built by Casing Method and Slurry Method.

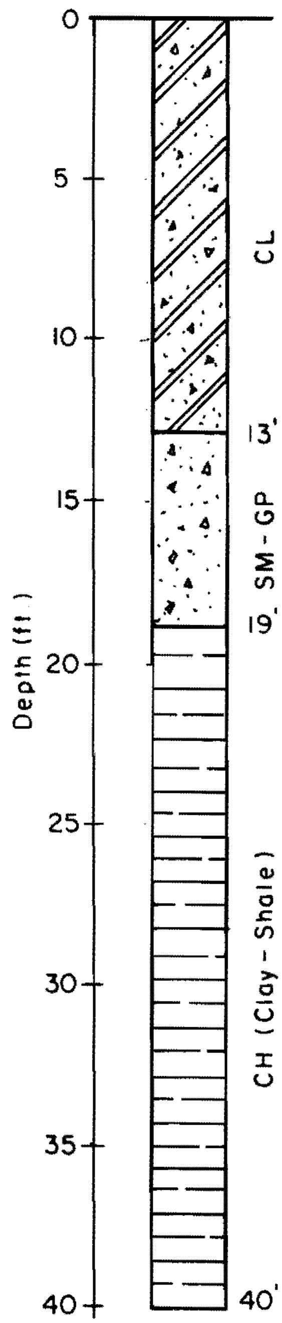


Level No.	No. of Mustran Cells	Distance from Top of Shaft (ft.)
1	4	3.00
2	4	4.50
3	4	19.50
4	4	22.00
5	2	24.25
6	4	26.15

Fig 5.8. Soil profile and instrumentation at shaft MT3 built by Dry Method.



Level No.	No. of Mustran Cells	Distance from Top of Shaft (ft.)
1	4	3.0
2	4	7.0
3	2	9.5
4	2	12.5
5	2	15.5
6	2	18.5
7	2	21.5
8	2	24.5
9	4	27.5



Soil Information above is based on Boring No. B-6 taken by THD

Fig 5.9. Soil profile and instrumentation at shaft DT1 built by Casing Method.

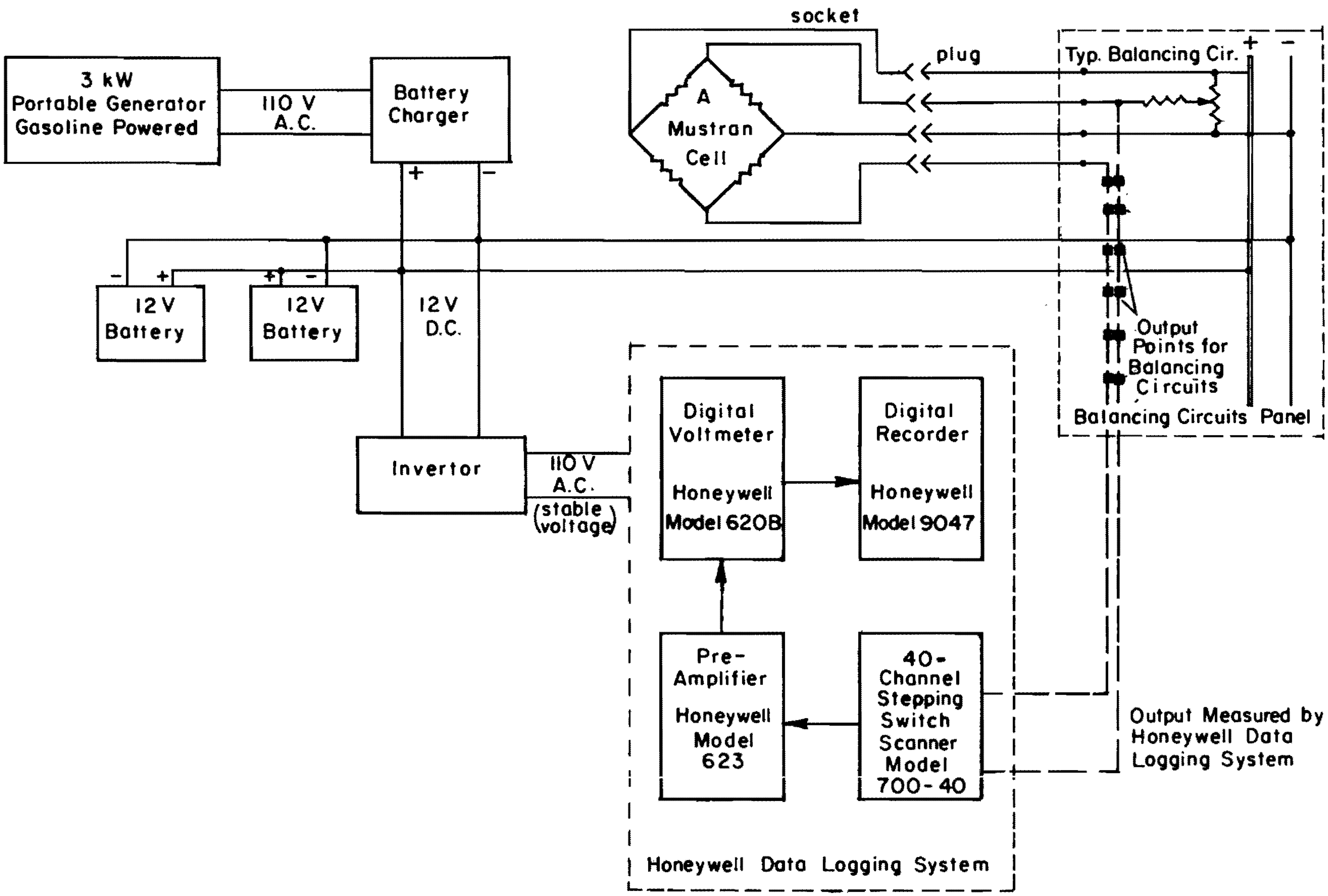


Fig 5.10. Schematic arrangement of the data-logging system.

source in case of a breakdown of the generator in the course of data acquisition. The balancing circuit panel is powered by a stable 12-volt DC power supply. The output of a Mustran cell is measured at the output points of the corresponding balancing circuit. The panel had 40 balancing circuits built into it to allow measurements of an equal number of Mustran cells. In none of the tests was this number exceeded.

The 40-channel stepping switch scanner (Honeywell Model 700-40) serves to connect the output points of one balancing circuit at a time. The signal received by the scanner is then amplified, on a pre-amplifier (Honeywell Model 623), to be displayed on a digital voltmeter (Honeywell Model 620B). This displayed reading can be printed on a paper tape by means of a digital recorder (Honeywell Model 9047). This arrangement eliminates manual procedures of taking strain readings by means of a conventional strain-indicator. The data-logging system used at the site could record the output of 26 Mustran cells in less than 1-1/2 minutes, thus saving time during actual load tests. A picture of the data logging system is shown in Fig 5.11.

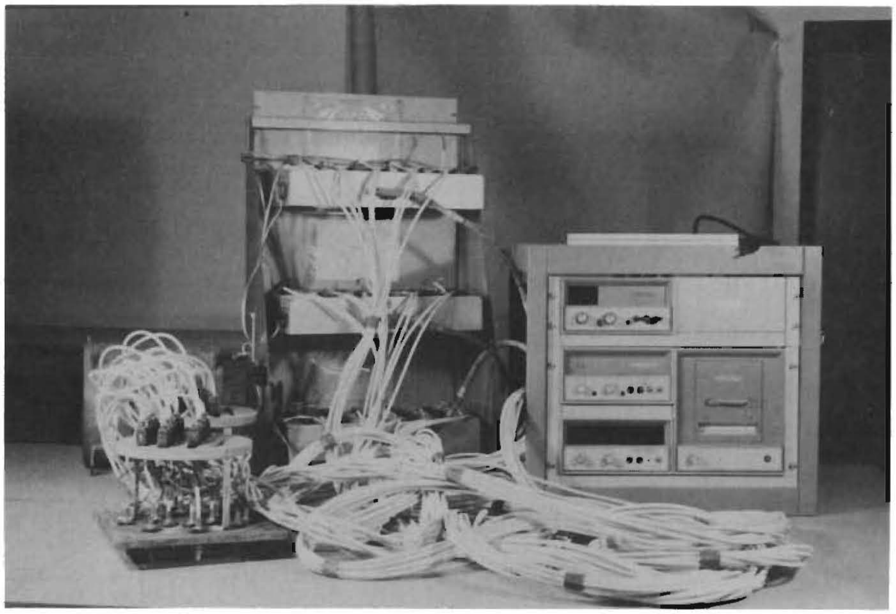


Fig 5.11. A picture of the data-logging system.

CHAPTER 6. CONSTRUCTION DETAILS

Preparatory Work for Construction

Considerations for safety and economy governed the details of the test shafts as well as the reaction system. Each test shaft was designed in accordance with current methods to carry safely the load required to fail the soil. The ultimate capacities of the soil were computed roughly from the criteria for clays suggested by Engeling and Reese, recognizing that the soils were clay-shales rather than clays. The main idea was to get the lower and upper bound values of ultimate soil capacities and then, after allowing some margin for error in judgment, arrive at a conservative figure for the ultimate load capacity for a test shaft. This approach had to be taken due to lack of information on the behavior of drilled shafts in shales.

Based on preliminary design considerations, it was decided to use 30-in. and 36-in. diameter shafts with suitable vertical and lateral reinforcement. Stone aggregate concrete with a 28-day compressive strength of about 3500 psi was considered adequate for structural strength. Rough calculations based on preliminary soil information and arbitrary design criteria, as mentioned earlier, showed that the ultimate soil capacity for any of the test shafts would not exceed 1000 tons, the maximum load allowed for the reaction beams and for the pair of hydraulic jacks supplied by the Texas Highway Department. The anchor shafts were taken deeper than the test shaft and underreamed to provide necessary resistance against uplift. The design of the anchor shafts was based on preliminary information and a conservative approach. At the Montopolis Site, all the test shafts were of the same total length to allow ease in the design and construction of test shafts, and the interpretation of test results.

Mustran cells were placed where strata changes were anticipated, and were also placed at two levels between the ground surface and the top of the shaft to act as an "in-shaft" load calibration system, as outlined in Chapter 8.

One set of Mustran cells was near the base of the test shaft. At least one additional set of Mustran cells was placed a few feet above the base but within the shale layer. The number of Mustran cells at any location was either two or four. Maximum cells were used at locations which were thought to be important from the standpoint of getting load transfer information.

The need for achieving economy led to the innovation of a reaction system in which the entire tensile steel could be fully recovered after its use. The available reaction beams were utilized by transferring the uplift load to the base of the anchor shaft by means of continuously threaded DYWIDAG bars connected at their tops to an adapter and at their bottoms to a circular plate embedded in the concrete of the underream. The details of this system are discussed later in this chapter.

Construction Details of Test Shafts

Three test shafts were constructed at the Montopolis Site on October 10 and 11, 1974. One test shaft, located at the Dallas Site, was installed on January 30, 1975. The test shafts at the Montopolis Site are designated as MT1, MT2, and MT3. The test shaft at the Dallas Site is referred to as DT1. The layout of test shafts was indicated in Figs 4.2 and 4.4. The test shafts at the Montopolis Site were located, in plan, at the vertices of an equilateral triangle in order to built the least number of anchor shafts using two anchor shafts per test.

The three steel cages, fabricated in a local shop, were delivered at the Montopolis Site on October 7, 1974. A meeting was held at the site on October 8, 1974, between representatives of the contractor, the Texas Highway Department, and the Center for Highway Research. The construction procedures for the three test shafts were discussed at this meeting. MT1 was chosen to be built by the casing method, while MT2 was selected for installation by the slurry displacement method. It was decided that MT3 would be free-standing above the shale layer by isolating it from the surrounding soil with a steel casing. In order to provide suitable arrangements for pouring concrete into this shaft, it was decided to use a Sonotube form in order to leave an annular air space between the steel casing and the form, above the shale layer. It was agreed that the dry method would be used to install MT3 below the top of the shale. Thus, three different construction methods, namely, the casing method, the slurry displacement method, and the dry method, were agreed upon for constructing the test shafts at the Montopolis Site. This decision was made to study the effects of construction methods on the load transfer characteristics of shales.

Mustran cells were installed on the three steel cages at the construction site on October 9, 1974, and on the morning of October 10, 1974. Throughout this period, the weather remained clear and the temperature ranged between about 65 degrees Fahrenheit and 75 degrees Fahrenheit. Initial readings of all Mustran cells were recorded by using a manually operated conventional strain indicator. This was done to locate those cells which showed abrupt changes in readings before or after embedment in concrete. On October 9, 1974, stakes were driven into the ground to locate the center of each test shaft.

MT1 was constructed on October 10, 1974, using the construction procedure known as the casing method. Drilling was started with the use of a 30-in. diameter auger. Drilling was done to about 8 ft from the ground surface without the aid of drilling fluid. Thereafter slurry was used to advance the hole. The level of slurry inside the hole was maintained at about 2 ft from the ground surface, until the hole was advanced to the top of the shale layer. The lumps of soil brought out by the auger after drilling were continuously examined visually so as to mark strata changes. Drilling was temporarily stopped at about the top of the shale layer. At this stage, the 30-in. diameter auger was disconnected from the kelly, and a 30-in. I.D. steel casing was "screwed" into the hole by turning the kelly against a removable rectangular tube fitted diametrically across the top of the casing. By comparing the depth of drilling to the depth of penetration of the casing, it was ascertained that the bottom of the casing was inside the shale layer. Measurements indicated that the bottom of the casing was 19 ft 4 in. below the ground surface. A cleaning bucket was then fitted to the kelly and it was used to bail out the slurry and clear the hole inside the steel casing up to the bottom of the steel casing. There was enough natural light to enable a visual inspection of the bottom of the hole while standing at the ground surface. The bottom of the hole did not show any sign of water seepage.

A 28-in. diameter auger was fitted to the kelly and lowered into the steel casing to reach the bottom of the hole at 19 ft 4 in. below the ground surface. This auger was used to drill an approximately 29-in. diameter hole from the depth of 19 ft 4 in. to 23 ft 9 in. A man was then lowered with a common bucket into the hole to clean the bottom manually and to see if any seepage was visible on the sides and bottom of the 29-in. diameter uncased hole. He reported that there was no visible seepage inside the hole.

The steel cage for MT1, ready for installation, was gently lifted from its top side by means of the cable of a crane. While its bottom was still resting on the ground, its manifold was securely tied to the main reinforcement. This process is shown in Fig 6.1. Thereafter, the cage was fully lifted into the air, as shown in Fig 6.2, and lowered into the final position. Its projection above the ground surface was checked to ascertain that the intended projection of the shaft would be possible after concreting. No error was detected. The manifold was detached from the cage and placed some distance away from the hole. Nitrogen pressure was maintained to avoid any seepage of moisture into any Mustran cell during the concreting operation, which is described next.

A truck carrying ready-mixed concrete arrived at the site about 3:00 p.m. The slump of concrete was checked and found to be between 6 in. and 7 in. This was judged as acceptable. A 10-in. diameter, 28-ft long steel tube with rectangular openings spaced at about 10 ft throughout its length and with open ends, was lifted by means of a crane into a vertical position and was lowered centrally into the hole. It was supported by the bottom of the shale at its base and it was kept vertical by holding it with the cable of a crane. A steel chute leading from the concrete truck was lined up with the rectangular opening in the tube and concrete was allowed to drop from the chute into the tube so as to travel to the bottom of the shaft. The concreting operation is shown in progress in Fig 6.3. The steel tube was lifted up as the level of concrete rose inside the cased shaft. When the top of the concrete was judged to be about 1 ft from the ground surface, the steel casing was pulled out. This procedure was a departure from the usual casing method of construction as described by O'Neill and Reese (1970), and Touma and Reese (1972). In the usual casing method, the casing is pulled two to three feet at a time and the top of the wet concrete inside the casing is kept several feet above the bottom of the casing. In the case of MT1, the entire casing was pulled out of the ground in one continuous operation when the top of the wet concrete was close to the ground surface. As opposed to the usual casing method, no slurry

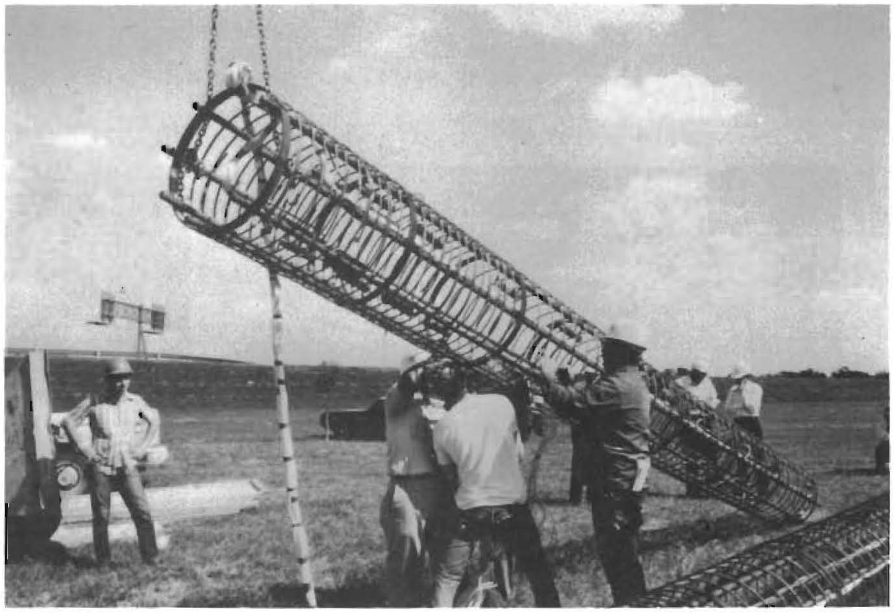


Fig 6.1. Manifold being tied to the steel cage for shaft MT1.

Fig 6.2. Steel cage fully lifted, ready to be lowered into the predrilled hole.

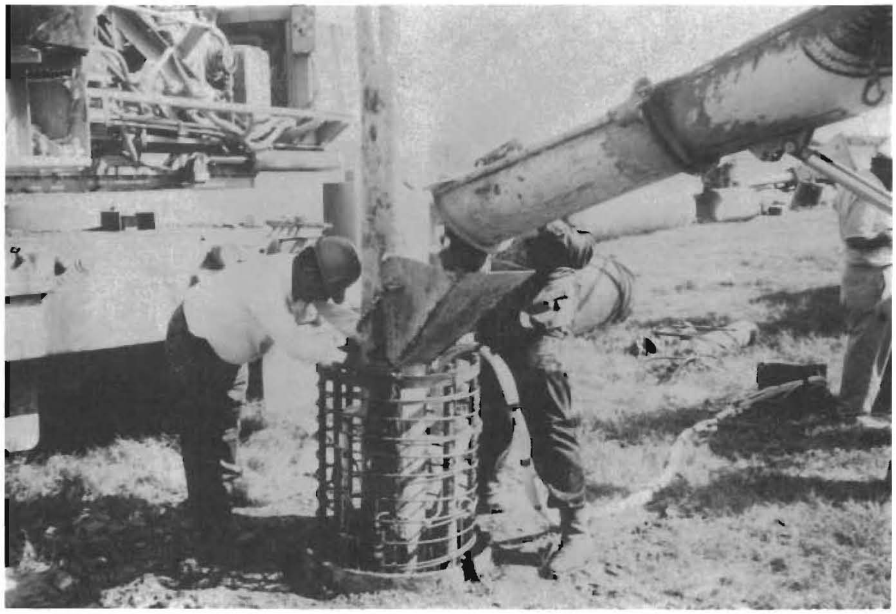


Fig 6.3. Ready-mixed concrete being poured from a delivery truck with the aid of a steel chute.

was seen as the casing was pulled out, which should not happen when the casing method is used. Concreting was continued inside the uncased hole till the top of the wet concrete reached within 6 in. below the ground surface.

At this stage a 30-in. O.D. steel pipe about 4 ft long was placed concentric to the steel cage, and concrete was poured and tamped into it till the top of the concrete reached 3 ft 3 in. above ground surface. After about an hour, that portion of the 30-in. O.D. steel tube which projected above the concrete surface was cut off with the aid of gas flame. The purpose of this steel tube was to provide lateral confinement to the wet concrete during construction, and later to prevent spalling of concrete near the top of the shaft during load tests. Three concrete cylinders, taken during the concreting operation, were transported to a humidity-controlled room at the end of the day for curing and testing at a later date. Next day, leaving the top one foot of the 30-in. O.D. steel tube in place, the rest of the tube was cut off with gas flame, avoiding sparks on the lead cables connected to the manifold. The geometry of the shaft as built was shown in Fig 5.7 along with the best estimate of the soil profile at the shaft location, and the locations of Mustran cells.

MT2 was built on October 11, 1974, using the slurry displacement method. Drilling was done with a 30-in. diameter auger and the hole was drilled dry to a depth of about 6 ft; slurry was used for subsequent drilling. The hole was drilled to a depth of 23 ft 11 in. from the ground surface with the 30-in. diameter auger. The rig operator pointed out that, in his judgment, some sand and gravel pieces were falling to the bottom of the hole as the auger hit some loose pieces of gravel while traveling in and out of the hole during drilling. He tried to clear the gravel pieces from the bottom as much as possible, but he suspected that some loose gravel and sand had still been left at the bottom. However, visual identification of materials being recovered from the bottom did not show a significant amount of gravel or sand. Drilling was stopped at a depth of 23 ft 11 in. from the ground surface. The steel cage and its manifold, which were lying a few feet away from the drilled hole, were picked up and placed in the hole with the help of a crane. The details of this step were the same as followed for MT1. The manifold was detached from the steel cage and placed a few feet away on the ground. Nitrogen supply was maintained throughout the construction of this shaft.

Concreting of MT2 was done with the help of a tremie which was lifted and positioned coaxially inside the steel cage by means of a crane. The tremie was fitted with a steel hopper at its top to admit wet concrete. The bottom of the pipe had a hinged flap which acted as a valve to prevent the slurry from entering the pipe. The tremie, with its bottom almost touching the base of the drilled hole, was held in position with a crane. The ready-mixed concrete delivery truck arrived at the site at about 11:30 a.m. A slump test was done and the concrete slump was found to be 6-1/2 in., which was considered acceptable.

Concrete was poured into the tremie through the hopper, with the bottom of the tremie still closed. When concrete reached the top of the tremie, that is, about 3 ft above the level of slurry in the drilled shaft, the concrete inside the tremie was allowed to flow to the base of the shaft by lifting the tremie a small distance with the help of a crane, thus allowing the flap at the base of the tremie to open out towards the base. The top of the concrete was always kept high enough with respect to the top of the slurry level so as to ensure that a positive displacement of the slurry by concrete was taking place. As the tremie was lifted, the chute from the concrete delivery truck to the tremie was directed towards the closest slot in the wall of the tremie. The slots were spaced at about 10 ft along the length of the tremie. When concrete was about 6 in. below the ground surface, a 30-in. I.D. and 4-ft long steel tube was placed coaxially with the steel cage. This steel tube, near the top of the shaft, was filled with concrete and was later burnt off to leave a 12-in. long sleeve at the top of the shaft in the same manner as was done for MT1. The shape of the test shaft MT2 as built was shown in Fig 5.7 along with the best estimate of the soil profile and the location of Mustran cells in the test shaft. The concreting of this shaft took about 50 minutes. Readings of all Mustran cells were taken as done for MT1.

The test shaft MT3 was built on October 11, 1974. The hole was drilled to a depth of 19 ft from the ground surface with a 36-in. diameter auger without the use of slurry. Shale was encountered at about 19 ft from the ground surface. There was no caving problem during drilling. However, some loose pieces of gravel did drop down occasionally into the hole while drilling was being done. When the top of the shale layer was reached, the auger was withdrawn from the hole and was detached from the kelly. A 36-in. I.D. steel casing was "screwed" into the hole by turning and pushing the kelly against the casing using a suitable attachment at the top of the casing. The casing was tightly screwed into the shale about 2 in., as determined from field measurements. A cleaning bucket was then used to clean the bottom of the hole. A 28-in. diameter auger was then used to advance the hole from a depth of 19 ft to 24 ft. A 30-in. I.D. and 25-ft long Sonotube (a commercial brand of circular cardboard form) was then lowered into the hole till its bottom hit the shale layer at a depth of 19 ft from the ground surface. The Sonotube was pushed by hand at its top to seat it into the shale as much as possible. Approximate measurements indicated that the Sonotube penetrated the shale by about 4 in. After placing the Sonotube in position, a man, carrying a bucket, was lowered to the bottom of the hole to clean it manually. He did not notice any seepage at the bottom of the hole. The positioning of the steel cage and concreting were done in exactly the same way as for MT1. The slump of concrete used was 6 in., and three concrete cylinders were taken during actual pouring operations. At the end of concreting, the top end of the Sonotube was cut even with the top of the shaft. Mustran cell readings were recorded before the end of the day. The construction of this shaft concluded the installation of all shafts at the Montopolis Site. The shape of the test shaft MT3 as built was shown in Fig 5.8 along with the best estimate of the soil profile and the location of Mustran cells in the test shaft.

The test shaft DT1 was built at the Dallas Site on January 30, 1975, by using the casing method. Slurry was not used at any time during the construction of this shaft. The hole was advanced to the top of the shale layer by means of an auger, and then a 36-1/2-in. I.D. steel tube was used as the casing and was "screwed" a distance of 6 in. into the shale layer to ensure an adequate "seal." An auger with a nominal diameter of 34 in. was used to obtain a 35-in. diameter dry hole for the bottom 4-ft-6-in. length of the test shaft. Concreting was done using the same technique as described for the shaft MT1. After the casing was pulled out, additional concrete was poured into the hole until the level of concrete was about 1 ft below the existing ground surface. After the concrete had set, the remaining length of the shaft was built using a sonotube as indicated in Fig 5.9 in order to comply with the construction plans. The concrete used has a slump of 6-1/2 in. and its 28-day compressive strength is estimated to be at least 3000 psi.

Readings of all Mustran cells were taken with the help of a strain indicator and the cells having erratic readings were promptly identified to closely observe their behavior during load tests.

Details of Reaction System

Important details of the reaction system are shown in Figs 6.4 to 6.6. The reaction to the applied compression load is furnished equally by two anchor shafts, each located at the same distance from the test shaft. The test shaft and the two anchor shafts are aligned to eliminate eccentric loads. Two parallel plate girders span the anchor shafts. The bottom flanges of the plate girders run about 2 ft above the hydraulic jacks which are symmetrically seated on the test shaft. A 1-ft thick bearing plate is used to distribute the applied load to the bottom flanges of the plate girders. Suitable filler plates are used between the pistons of the jacks and the bearing plate in order to maximize the travel of pistons. An anchor post is suspended from the top flanges of the plate girders above each anchor shaft. Each anchor post is connected at its bottom to an adapter against which 12 DYWIDAG bars, spaced 30 degrees on a 24-in. diameter circle, react in tension. The DYWIDAG bar is a commercial brand of high strength steel bar with coarse threading over its full length. The DYWIDAG bars run to the bottom of the anchor shaft where they are connected to an annular base plate. Each DYWIDAG bar is enclosed in a plastic tube to eliminate its contact with the concrete of the anchor shaft. When a load is applied to the test shaft the pistons of the hydraulic jacks react against the bearing plate, which applies an uplift force to the plate girders. The uplift force is finally transmitted to the bottom of each anchor shaft through the DYWIDAG bars.

The reaction system conceptually described above represents a new and economical reaction system for testing drilled shafts. Prior to this innovation, the uplift force was transmitted to an anchor shaft by a heavy wide flange beam or steel bars properly bonded to the concrete of the anchor shaft. At the end of the test the wide flange beam or steel bars had to be left embedded in the anchor shaft and the portion projecting above the anchor shaft had to be cut off to clear the site of all protruding objects. Thus a considerable amount of steel and labor previously were underutilized. With the use of the new reaction system, the entire quantity of tension steel is

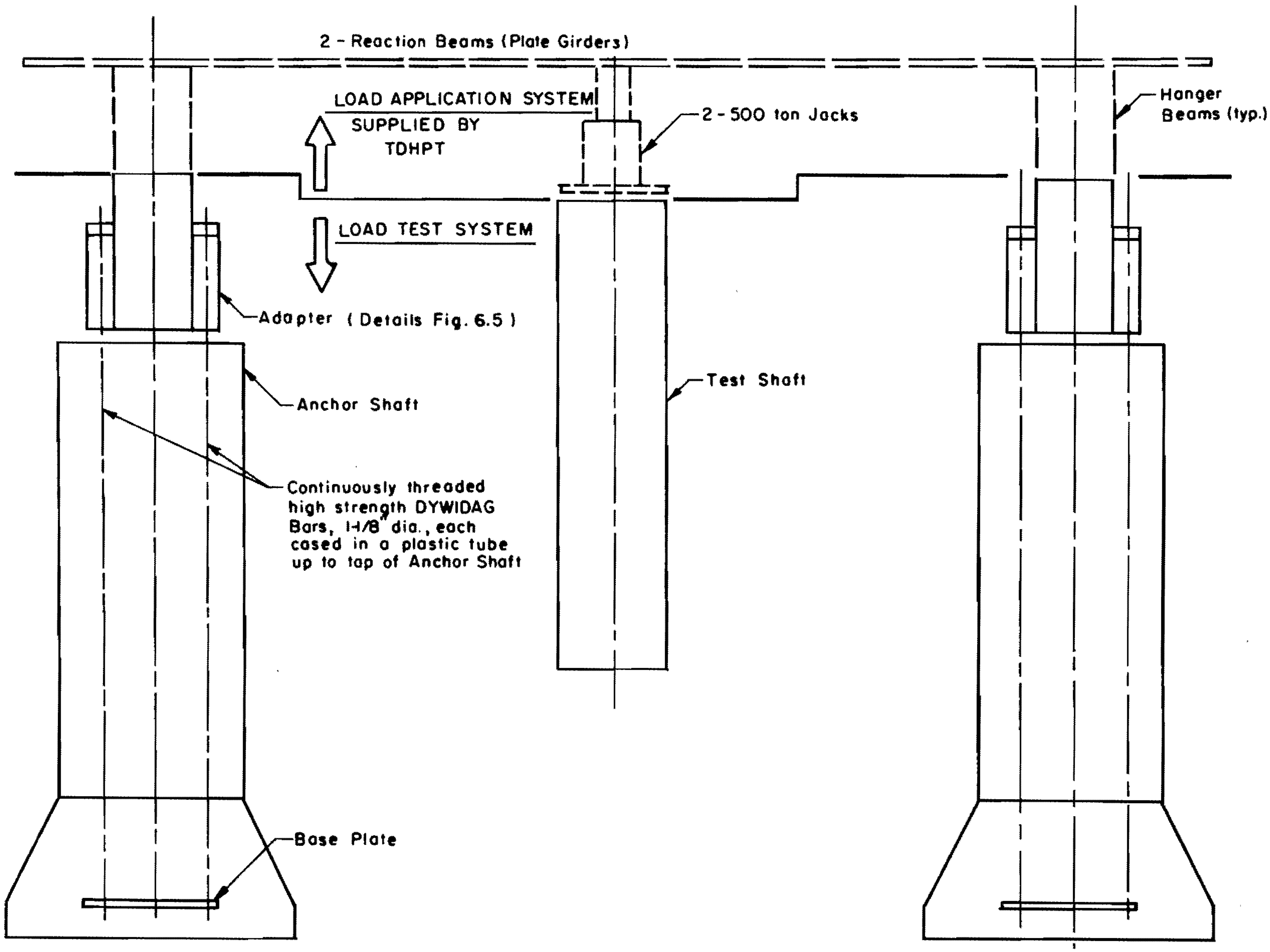
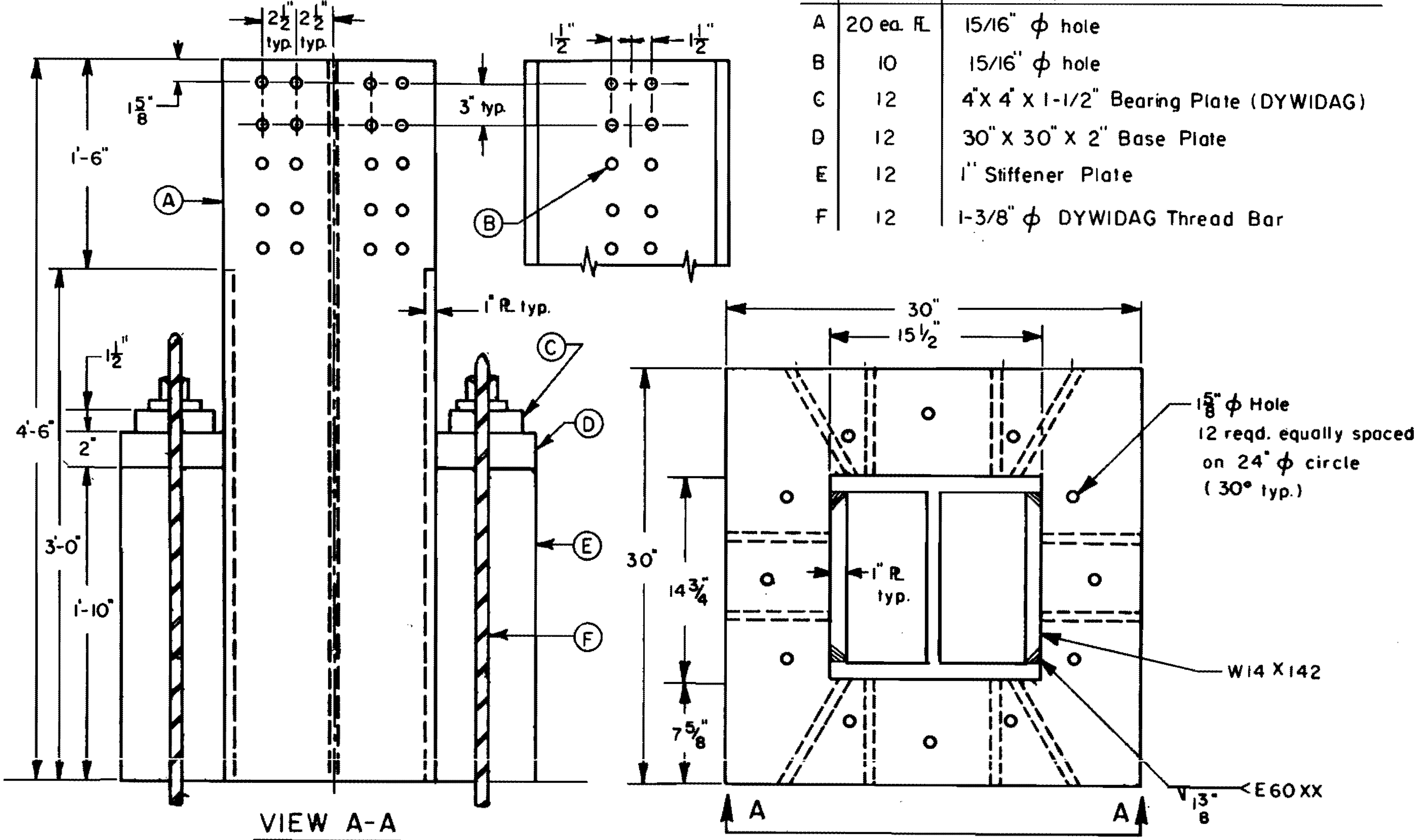


Fig 6.4. Schematic arrangement of the reaction system.

Sym	No. Reqd.	Description
A	20 ea. PL	15/16" ϕ hole
B	10	15/16" ϕ hole
C	12	4" X 4" X 1-1/2" Bearing Plate (DYWIDAG)
D	12	30" X 30" X 2" Base Plate
E	12	1" Stiffener Plate
F	12	1-3/8" ϕ DYWIDAG Thread Bar



Note: All holes in W14 X 142 are to match the holes in the Cover Plates coming down from the W14 X 142 hanger beam (not shown) to form a suitable system of transferring upward pull on the hanger beam to the adopter

Fig 6.5. Details of adapter.

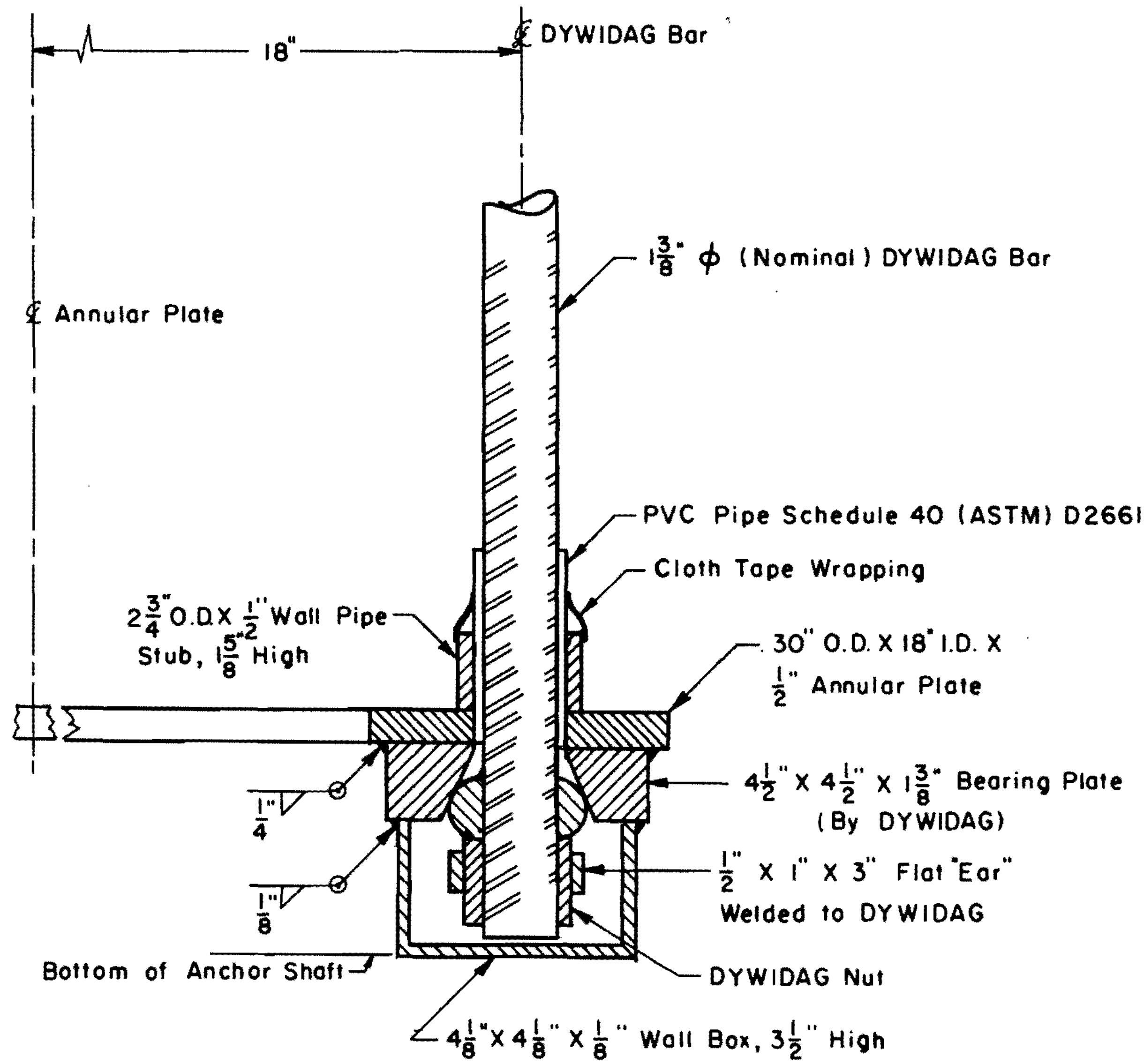


Fig 6.6. Details of base plate.

recovered and can be used many times. The base plate and the plastic tubes have to be left embedded in concrete, but the overall economy of the system remains highly attractive. It is hoped that this system will prove useful to contractors and engineers to conduct load tests on axially loaded foundation elements.

Three anchor shafts were installed at the Montopolis Site. Their centers were located at the corners of an equilateral triangle, each side of which was 18 ft long. The centers of the test shafts were located at the midpoints of the sides. Each anchor shaft at the Montopolis Site had a total length of 40 ft and had a 45-degree bell. The hole was drilled by a 36-in. diameter auger without the use of slurry. The top of each anchor shaft was about 6 in. below the ground surface.

The procedures for construction of anchor shafts were almost identical at the Montopolis and Dallas Sites. Therefore, in the remaining paragraphs of this chapter, the construction details of the anchor shafts at the Montopolis Site only are presented. However, it should be noted that one anchor shaft at the Dallas Site was actually one of the permanent shafts designed to support the superstructure. Therefore, its design was modified to permit its use as an anchor shaft during load tests.

Proper quantities of 1-3/8-in. nominal diameter DYWIDAG bars and fittings arrived at the Montopolis Site on October 7, 1974. Pipe stubs and anchor plates were welded on opposite faces of the annular base plate. The two sides of an annular base plate are shown in Fig 6.7. A few plain annular plates, without any anchor plate or pipe stub, are also shown in the figure. The top ends of the DYWIDAG bars were slipped about 6 in. through the holes of the plain annular plates and the DYWIDAG nuts were then screwed on, as shown in Fig 6.8. The bottom ends of the DYWIDAG bars, enclosed in plastic tubes, were similarly slipped through the holes of the annular base plates and DYWIDAG nuts with "ears" welded to them were screwed on as shown in Fig 6.9.

A square steel box, as shown in Fig 6.10, was then welded to the anchor plate. The entire assembly was then lifted vertically by a crane hooked to the plain annular plate. Molten wax was then poured into the steel stub around the DYWIDAG bar as shown in Fig 6.11. The plastic tube was then pushed tightly into the pipe stub and cloth tape was wrapped around it. This is shown in Fig 6.12. The entire assembly was then moved to the drilled hole which was only a few feet away from the location of assembly. As the system

Fig 6.7. Two sides of the annular base plate used in the anchor shafts.

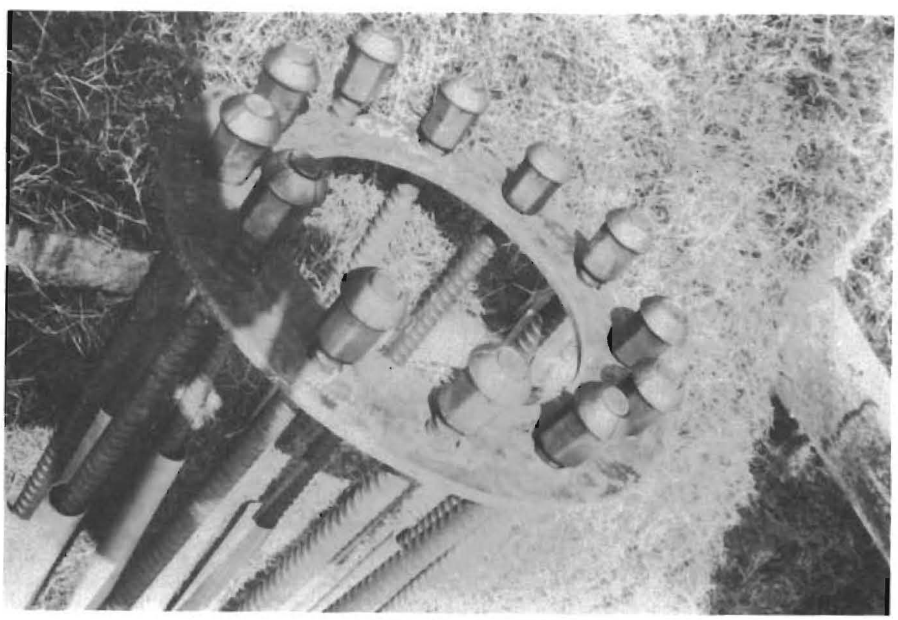


Fig 6.8. Plain annular plate used at top of DYWIDAG bars to facilitate lifting of bars together.

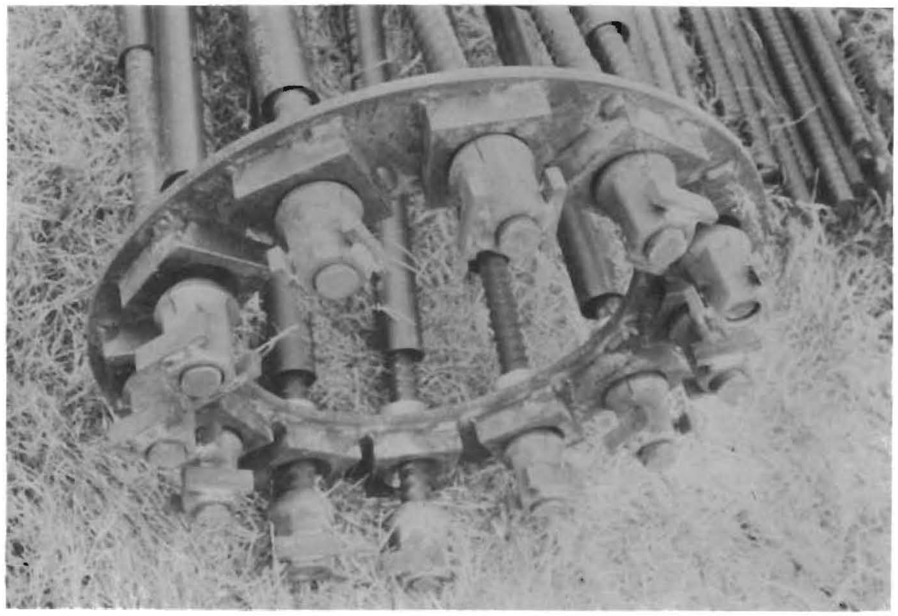


Fig 6.9. DYWIDAG nuts with "ears" at bottom ends of DYWIDAG bars.

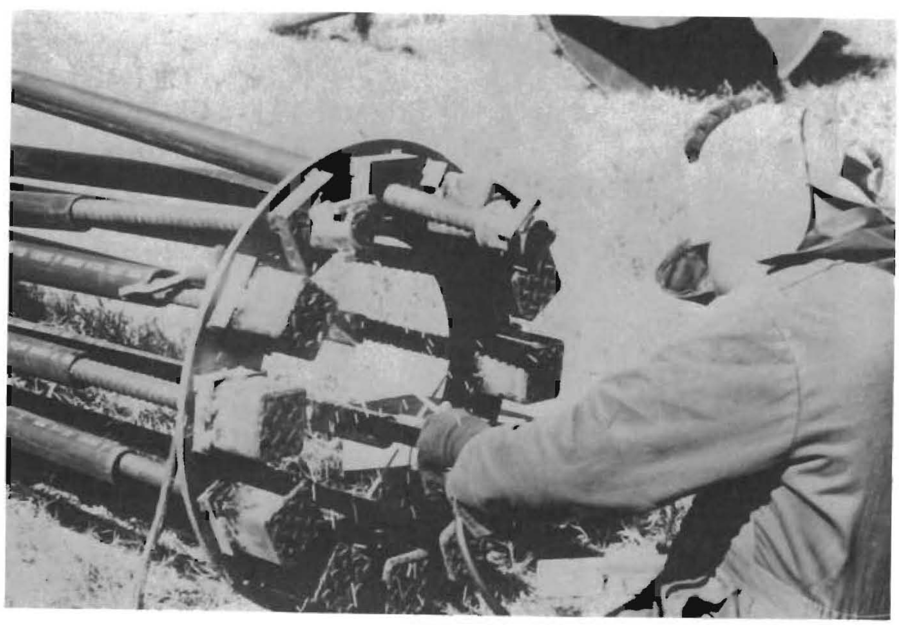


Fig 6.10. Square steel boxes being welded to the anchor plates which are already welded to the annual base plate.

Fig 6.11. Molten wax being poured into the steel stub around the DYWIDAG bars.

Fig 6.12. Plastic tube pushed into the steel stub and cloth tape wrapped around it.

of DYWIDAG bars, enclosed in plastic pipes, was lowered into the hole, 1-in. wide and 3/8-in. thick steel rings were tightly attached around the circle of plastic pipes at about 10-ft vertical spacing. This was done to provide support to the plastic pipes against lateral movements induced by the tendency of the wet concrete to move from the center towards the circumference of the shaft.

Inadvertently, concrete cylinders were not taken and slump tests also were not performed. It is estimated that the slump was between 6 and 7 in. It is believed that the compressive strength of the concrete met the specification of 3500 psi. After concreting was over, each DYWIDAG bar was turned with a wrench to check that the voids between the plastic pipe and the DYWIDAG bar had not been filled with cement paste or concrete. All DYWIDAG bars could be turned easily by hand or by a wrench applied at the top of a bar.

At each anchor shaft, four DYWIDAG nuts, having the same configuration as the ones at the bottom, were screwed down from the top of four equally spaced DYWIDAG bars to a predetermined elevation above the ground surface. The adapter was then lowered, passing each DYWIDAG bar through the hole in the base plate of the adapter, until the adapter rested on the four DYWIDAG nuts. During the course of work it was found necessary to tape the four DYWIDAG nuts to the DYWIDAG bars so that they would not turn due to the shaking of the bars while the adapter was being lowered into position. After the adapter was placed in position all the DYWIDAG nuts were screwed down from the top of the DYWIDAG bars until they butted against the bearing plates on the adapter. The plate girders and the anchor posts were placed in position and each anchor post was connected to an adapter. The plate girders were held by a crane until both the anchor posts had been connected to the adapters and until the pistons of the hydraulic jacks had applied a nominal uplift force to the anchor shafts.

CHAPTER 7. PARTICULARS OF FIELD TESTS

General Information

Field tests were run on four drilled shafts, three at the Montopolis Site and one at the Dallas Site. The Montopolis tests were conducted between November 14, 1974, and January 9, 1975. The Dallas tests were performed on February 12, 1975, and March 21, 1975. Construction of the test shafts at the Montopolis and Dallas Sites was completed on October 11, 1974, and January 30, 1975, respectively. During the period intervening between the completion of construction and the commencement of field tests, the Mustran cells were kept pressurized with dry nitrogen to keep them free of moisture. For three days following the construction of a drilled shaft, resistance measurements were noted for each Mustran cell in the drilled shaft to identify cells with abrupt resistance changes, which could have resulted from seepage of moisture into the cell.

During a routine visit to the Montopolis Site on November 1, 1974, water was noticed inside the 1-1/2-in. wide annular space of the shaft MT3. Measurements were taken immediately to determine the height of the annular column of water. It was found that the water column was about 16 in. high above the bottom of the 36-in. diameter hole that was augered during construction for installing the steel casing. The water surface was at a depth of 18 ft from the ground surface. Several attempts were made to remove the water, but those attempts were not successful due to non-availability of electric power at the site and the narrow working space. The depth to the top of the water surface was noted on the days of tests on MT3. This information is furnished in the description of tests on this shaft. The effects of the water on the behavior of the test shaft are discussed later.

Details of Field Tests

Coordination among the persons working on the load test was achieved by dividing the team as described next. The team consisted of one plotter, one pump operator, one or two dial readers, and one data logger. The plotter performed the duties of a dial reader in addition to his other duties. The plotter had a calculator and graph paper to compute and plot settlement data. The pump operator had a calibration chart to enable him to apply the appropriate hydraulic pressure corresponding to a particular load. Settlement readings were taken by the dial readers 30 seconds and two minutes after a particular load was applied. The data logger operated the Honeywell data logging system. The working procedure described below was closely followed.

The plotter called out the desired load to the pump operator. The pump operator then actuated the pump to develop the necessary hydraulic pressure corresponding to the desired load. Upon reaching the necessary hydraulic pressure, the pump operator signalled back to the plotter indicating that the desired load was attained. The plotter waited for 30 seconds after receiving the pump operator's signal, and the plotter then called out to the data logger and the dial readers to take the 30-second readings. The plotter himself took the reading of a dial gage assigned to him. The dial readers called out the dial gage readings to the plotter and he quickly computed and plotted the average settlement corresponding to the load acting on the shaft. The plotter waited for the data logger to signal the completion of recording of the 30-second Mustran cell readings on the paper tape. Upon receiving the signal from the data logger, marking the end of 30-second readings, the plotter waited until two minutes had elapsed since the application of the particular load. The plotter then called out to the data logger and the dial readers to take the 2-minute readings. The 2-minute readings were taken and plotted in the same manner as the 30-second readings. When the data logger signalled to the plotter marking the end of the 2-minute readings, the plotter called out to the pump operator the next desired load based upon his judgment by looking at the trend of the load settlement curve already plotted. The load increments were decreased as the test shaft approached plunging failure. During the unloading phase, 30-second and 2-minute readings were taken in the same way as described above. When all the load on the test shaft had been removed, 5-minute, 10-minute, and 15-minute readings were taken in addition to the 30-second and 2-minute readings.

The field tests at the Montopolis Site were first conducted on shaft MT3. These tests were followed by tests on shaft MT1 and finally on MT2. The next few paragraphs describe the field tests on MT3, MT1, and MT2, in that order.

The field tests on DT1, the only test shaft at the Dallas Site, are described after the accounts of the Montopolis tests.

Shaft MT3 had been built to derive its resistance to axially applied loads by interaction with shale only. The bottom 5 ft of this shaft were in contact with the shale. The remaining portion of this shaft was in contact with air as shown earlier in Fig 5.8. It was pointed out earlier in this chapter that water had accumulated in the annular space around this shaft. On the test day, November 14, 1974, all the equipment, instruments, and personnel arrived at the test site by 9:00 a.m. The work on setting up the load application system, consisting of the air compressor, the hydraulic pump, and the hydraulic jacks, was finished while the data logging system was being set up. Beginning at about 10:30 a.m., readings of the Mustran cells were recorded on paper tape every 15 to 30 minutes without applying any load with the hydraulic jacks. This was done to note the drift in the output readings of the Mustran cells. There was evidence of drift in the readings of some of the Mustran cells, and it was found desirable to record readings in multiples of 10^{-5} volts instead of microvolts by suppressing the last digit of the displayed readings. The probable cause of the drift was thought to be the presence of some moisture either in the socket board of the manifold or inside the Mustran cells. The possible error caused by recording the Mustran cell output in multiples of 10^{-5} volts instead of microvolts was considered to be insignificant in comparison to the magnitude of applied loads. The dial gages, to measure settlements near the top of the shaft, were set up on position by noon and the load application began at 12:30 in the afternoon.

The 30-second and 2-minute readings were taken at applied loads of 50, 100, 150, 200, 225, 250, 275, 300, 320, 340, 360, 380, 400, 420, 440, and 460 tons. At 1:30 p.m., when the applied load was 460 tons, the nut connecting the high pressure hose to the hydraulic jack cracked visibly and began leaking. Realizing that the hydraulic pressure at the nut was of the order of about 10,000 psi, and was therefore a potential hazard, the pressure at the pump was carefully and quickly reduced to zero to prevent any sudden bursting at the nut. Thus the shaft was completely unloaded after the applied load had reached 460 tons at a settlement of about 0.9 in. When the shaft was completely unloaded, the 15-minute reading indicated a settlement of about 0.6 in. at the top of the shaft. The nut was replaced and testing was resumed at 2:40 p.m. During this phase, 30-second and 2-minute readings were taken at 50-ton intervals up to 400 tons. At the applied load of 400 tons the settlement was about 0.4 in. against 0.6 in. during the first loading sequence. The applied load was increased from 400 to 440 tons, and thereafter the applied load was increased in 20-ton increments. When the applied load reached 520 tons, the settlement near the top of the shaft was noted as about 1.0 in.

At this stage, it was considered desirable to reset the dial gages so that the settlement readings could proceed uninterrupted at higher loads because the shaft was expected to settle at a faster rate, causing the dial gages to run out of travel at an awkward time if resetting of the dial gages was not done. As the dial gages were being reset, at 3:45 p.m., the high pressure hose suddenly burst at its junction with the replaced nut. The nearly horizontal jet of oil erupting out of the hose with a high velocity, barely missed the face of one person, and hit a vertical board of plywood 15 ft away at a horizontal angle of about 30 degrees with the vertical board. Two layers of the ply on the board were ripped off. The jet was deflected from the plywood board towards an automobile parked about 30 ft away from the board. The automobile was covered with a spray of oil as the travel of the jet ended. The entire incident happened in such a short time-frame that the hydraulic pump was shut only after the jet of oil had lost its entire velocity. Due to the presence of the DYWIDAG nuts below the base plate of the adapter at each anchor shaft, the plate girders remained stable in vertical position despite the sudden release of the applied load of 520 tons. The rest of the day was spent in loading the equipment back into the vehicles in which it had been brought to the site in the morning. It was decided to get the necessary repairs done and to design a fail-safe hydraulic system for future tests. Because the shaft MT3 had not plunged at an applied load of 520 tons, it was decided to retest the shaft when the hydraulic system became operative.

The Texas Highway Department, who own the hydraulic pump and the jacks, decided to repair the connection at their workshop in the Austin area. To guard against the "whip" effect of the hose in case of sudden bursting, it was decided to enclose each end of the hose in a securely fixed steel pipe at the hydraulic pump and at the jack. A valve was introduced at the jack which ensured that the oil inside the jack would remain in without loss of pressure even if the hose burst at its junction near the jack. The new arrangement, as used, was shown schematically in Fig 5.1.

On January 6, 1975, another attempt was made to load shaft MT3 to failure. All equipment and personnel arrived at the site at about 9:00 a.m. Upon opening the manifold for the test shaft, moisture was seen on the socket boards. These socket boards were dried with a jet of dry nitrogen gas for more than two hours until the readings of Mustran cells, as read on the data logging system, appeared reasonably stable. However, about three or four Mustran cells continued to behave erratically. It was thought probable that the problem existed at the strain gages of these cells rather than at the socket board. The field test on the shaft was started at about 1:00 p.m.

The depth to the water surface in the annular space of the shaft was measured and found to be 10 ft 6 in. from the ground surface. Thus the water level had risen approximately 6 ft 6 in. between November 14, 1974 and January 6, 1975. The load was applied to the shaft in 50-ton and 25-ton increments from 0 to 250 tons and from 250 tons to 500 tons, respectively. After increasing the load from 500 tons to 520 tons, 10-ton load increments were applied up to 570 tons. The shaft plunged at 570 tons at a settlement of 1.9 in. near the top. The shaft is considered to have plunged when settlement continues without increase in load. Throughout the test, the 30-second and 2-minute readings were taken following the working procedure described earlier in this chapter. Similar readings were taken during the unloading phase at applied loads of 500, 400, 300, 200, 100, 50, and 0 tons. At the 0-ton applied load, 5-minute, 10-minute, and 15-minute readings were also taken in addition to the 30-second and 2-minute readings. The system worked without any sign of distress during this test. The load-settlement curves for the test shaft MT3 are shown in Fig 7.1.

On January 7, 1975, the reaction system was set at the new position to test the shaft MT1 which had been built by the casing method. The aluminum angles and the reference beams were also set into position on the same day so that testing could begin early on the next day.

Testing of the shaft MT1 started at 10:00 a.m. on January 8, 1975, using the same equipment and working procedures as described earlier in this chapter. Applied load was increased from 0 to 200 tons in 50-ton increments. From 200 tons to 500 tons the applied load was increased in 25-ton increments. Thereafter, the load was increased from 500 tons to 520, 530, and 540 tons.

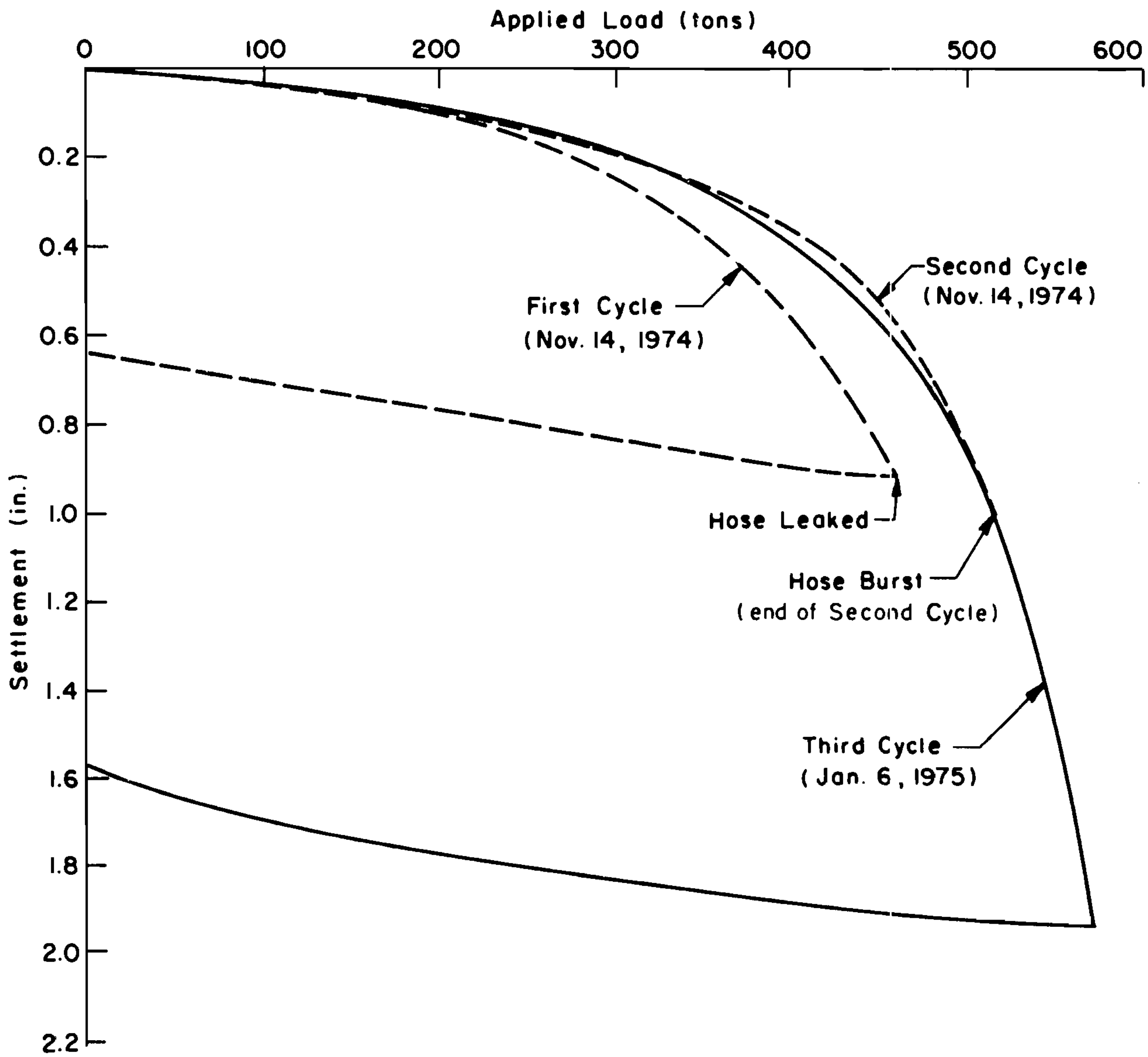


Fig 7.1. Load settlement curves for shaft MT3 built by Dry Method.

When the load reached 540 tons, it was realized that the hydraulic pump needed a supply of oil in its tank. The hydraulic line from the pump to the jacks was shut off by means of a valve and the pump was stopped. An adequate quantity of oil was filled into the tank of the pump. This process took about 10 minutes, during which time the applied load fell off from 540 tons to 515 tons. After starting the pump again, the applied load was first increased to 525 tons and then to 540 tons, taking the 30 second and 2-minute readings at both 525 tons and 540 tons. From 540 tons to 600 tons the load was increased in 10-ton increments. At 600 tons the settlement near the top of the shaft had reached about 3 in. and the piston of the hydraulic jacks had moved up almost 7-1/4 in., which was believed to be the maximum travel of the piston. At this stage unloading was begun. Appropriate readings were taken at 500, 400, 300, 200, 100, 50, and 0 tons. At 0 ton, 5-minute, 10-minute, and 15-minute readings were taken in addition to the 30-second and 2-minute readings taken at all discrete applied loads. During the process of data acquisition, a few Mustran cells showed clear evidence of malfunction or excessive strains, particularly those near the tip. However, the testing continued uninterrupted. The entire test was finished by 1:30 p.m. Load-settlement curves for the shaft MT1 are shown in Fig 7.2.

In the afternoon of January 8, 1975, the reaction system was moved to the new position for testing the shaft MT2 which had been constructed by the slurry method. In the same afternoon, the reference timber beams and the aluminum angles were also set in position.

Field tests on the shaft MT2 began at about 10:00 a.m. on January 9, 1975. This shaft was loaded in 50-ton increments from 0 tons to 200 tons. The load was increased from 200 tons to 500 tons in 25-ton increments. Thereafter, 20-ton increments were applied up to 600 tons. The shaft plunged at 610 tons at a settlement of about 2.7 in. Readings were taken during the unloading phase at applied loads of 500, 400, 300, 200, 100, and 0 tons. At all applied loads the 30-second and 2-minute readings were taken as described earlier in this chapter. At the 0-ton load, 5-minute, 10-minute, and 15-minute readings were taken in addition to the 30-second and 2-minute readings.

During the loading phase of the tests on MT2, a few Mustran cells located close to the base behaved very erratically. However, the same Mustran cells began showing an apparently acceptable pattern of behavior during the unloading phase. Since the shaft had already been tested to failure, it was

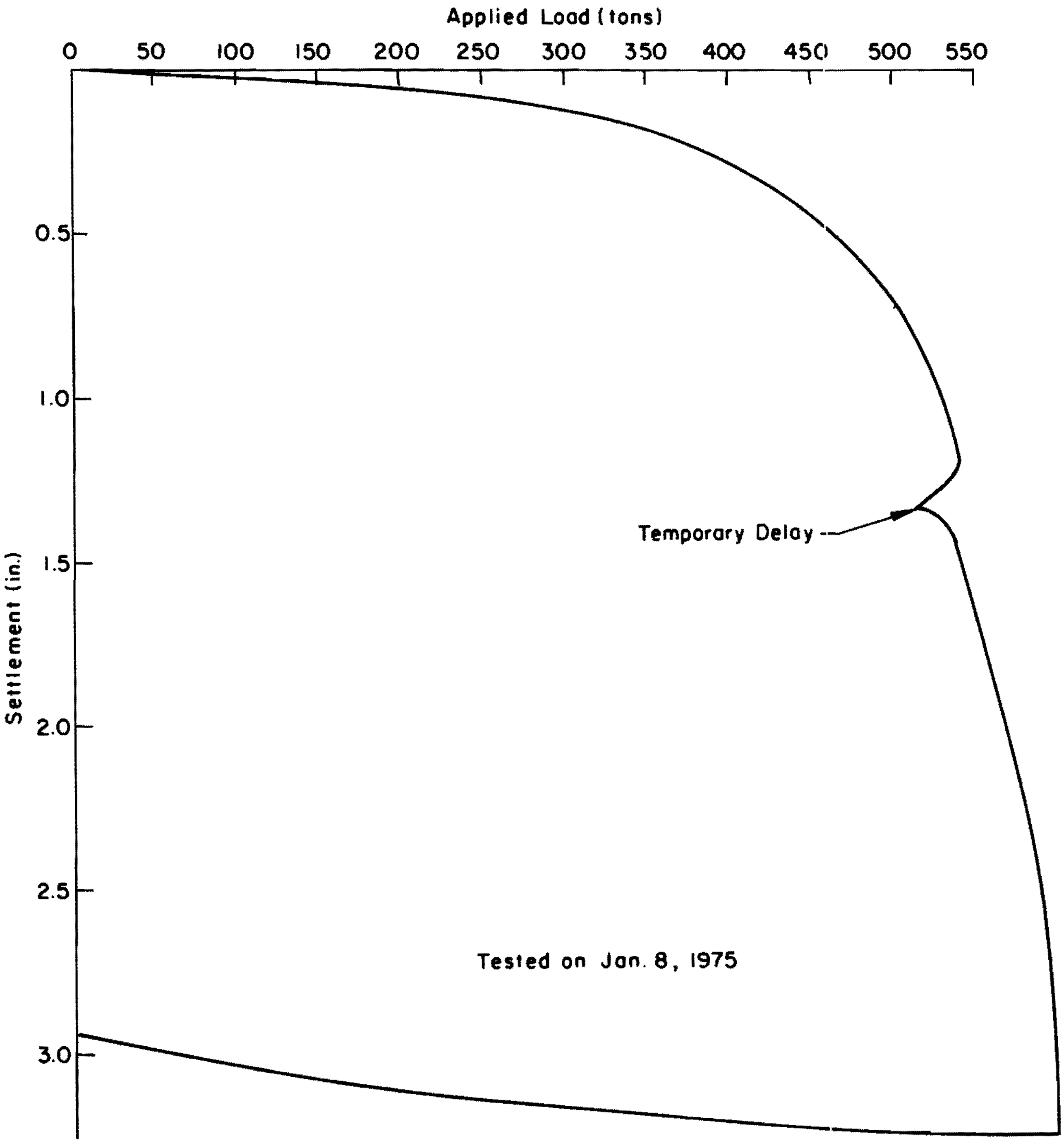


Fig 7.2. Load settlement curve for shaft MT1 built by Casing Method.

decided to watch the behavior of these Mustran cells upon reloading and then unloading. The load was increased on the shaft in 50-ton increments up to 550 tons and all readings were taken as usual. The Mustran cells under watch again started to show erratic behavior as the load increased. However, as the load was being increased from 550 tons to 575 tons, the high pressure hydraulic hose burst at its junction with the hydraulic jacks. This incident marked the end of the tests at the Montopolis Site. At about 2:00 p.m., all the equipment brought to the site was loaded back into the vehicles and returned to the normal place of its storage. The load-settlement curves for the test shaft MT2 are shown in Fig 7.3.

The first load test on the shaft DT1 at the Dallas Site was performed on February 12, 1975. The shaft had to be unloaded at an applied load of 75 tons in order to check into the erratic behavior of some Mustran cells. Reloading of the shaft was continued up to 375 tons in 25-ton increments. At the applied load of 375 tons the reaction beam began to tilt due to the "hinge-like" behavior of some couplers used to extend the DYWIDAG bars. The load was decreased from 375 tons to 300 tons, and thereafter it was decreased to 0 ton in 100-ton steps. The shaft had a settlement of about 1.0 in. at the applied load of 375 tons. The shaft was tested again on March 21, 1975, after correcting the above noted difficulty regarding the couplers, and the shaft plunged at an applied load of 450 tons at a settlement of 1.9 in. with respect to its last fully unloaded position. The readings during these tests at the Dallas Site were taken in accordance with the procedures followed at the Montopolis Site. The load-settlement curves for the test shaft DT1 are shown in Fig 7.4.

Comments on Field Tests

The experience with the hydraulic system during the tests at the Montopolis Site indicates the need to improve its safety and dependability. General discussions with contractors in the drilled shaft and piled-foundations industry indicate that flexible hoses capable of withstanding several cycles of 0 - 20,000 psi pressure range are not available. Generally, the current practice allows only three or four cycles of loading and unloading on a high pressure hose. Experience at Montopolis seems to confirm the current practice. The main difficulty lies in the fact that manufactured hose cannot

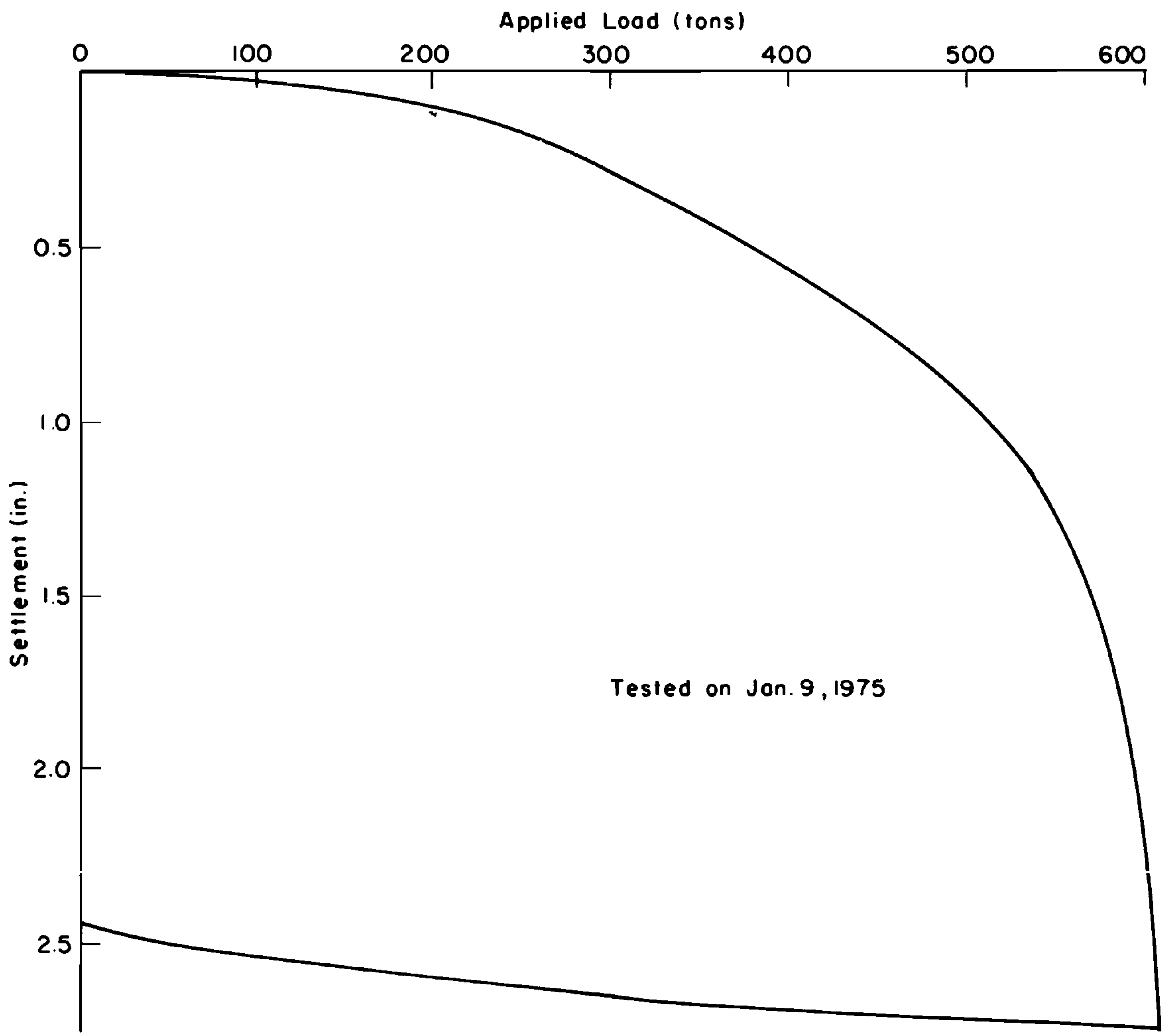


Fig 7.3. Load settlement curves for shaft MT2 built by Slurry Method.

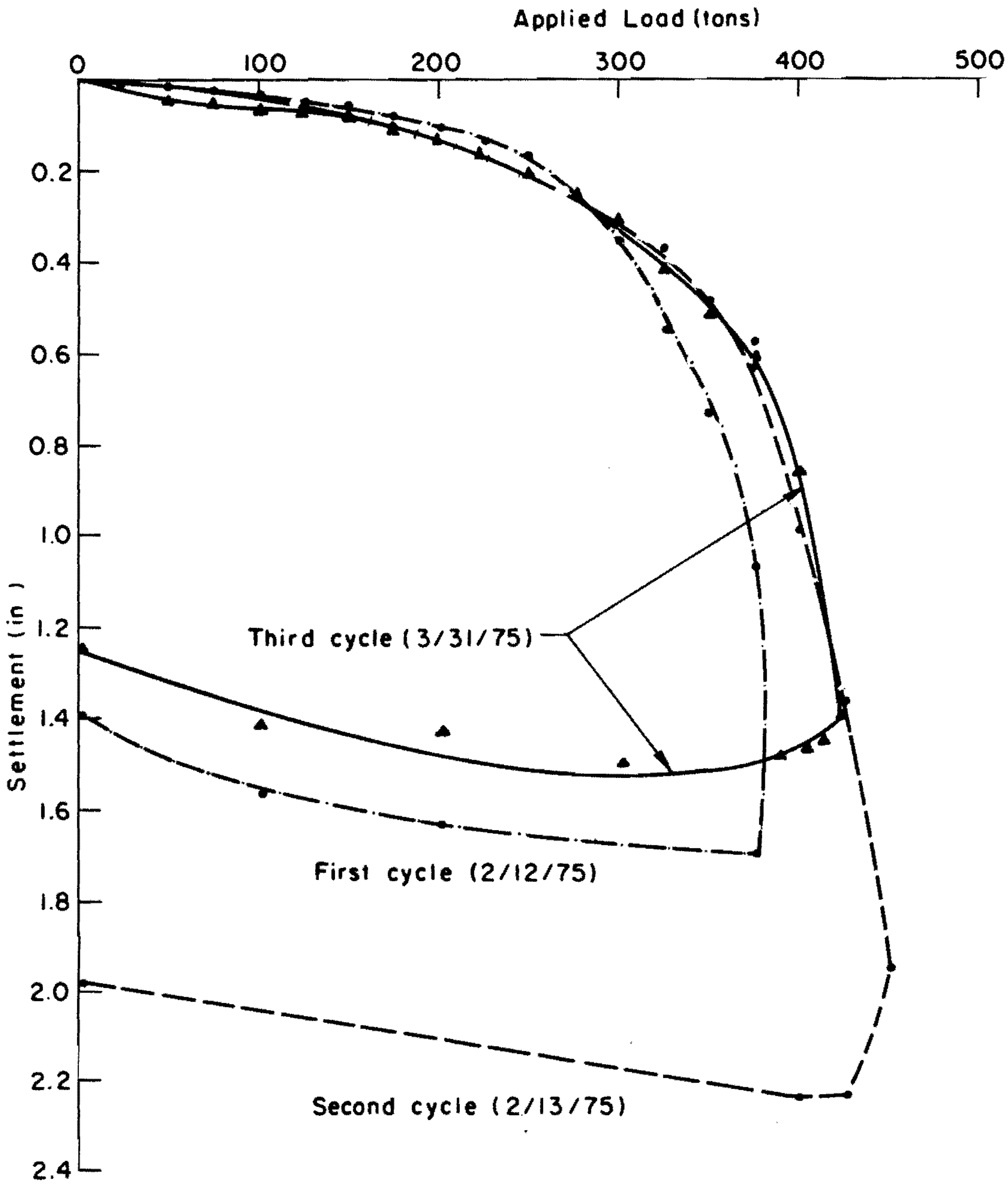


Fig 7.4. Load-settlement curves for the shaft DT1 built by Casing Method.

be adequately tested at sustained high pressures due to lack of suitable testing facilities. It is a common practice to apply the test pressure to a hose in the form of an impulse and then use the hose for pressures not exceeding 25 percent of the maximum impulse pressure. Some people working in the area of high pressure hydraulic hoses believe that the hose will generally fail slowly rather than abruptly, giving enough time to an operator to de-energize the system. Experience at the Montopolis Site indicates that the hose can fail abruptly near the end.

A few Mustran cells in each shaft gave problems and their readings appeared to be of little or no value. It is felt that there were perhaps two possible causes of this behavior: very high stress concentrations near the tip or damage to the strain gages in the Mustran cells. At the present time, a completely reliable method of isolating the cause of the erratic behavior of Mustran cells is not known.

The reaction system designed for the tests worked satisfactorily. All the tension steel consisting of the DYWIDAG bars was recovered and reused after each test, thus proving its potential to achieve overall economy in the cost of testing axially loaded foundation elements.

CHAPTER 8. ANALYSIS AND DISCUSSION OF TEST DATA

Approach Used for Analysis of Field Data

It was pointed out in the preceding chapter that a few Mustran cells behaved erratically in each test. It was, therefore, essential to evaluate and modify the Mustran cell readings prior to the actual use of the readings for analysis and interpretation. The procedure used for this purpose is explained below.

The Mustran cell readings were arranged in a tabular form to see the pattern in which the readings of the Mustran cells varied along the depth of shaft at different applied loads. Necessary corrections were made in the readings to account for the variation of shaft diameter with depth. In some cases it was observed that a Mustran cell at a particular level gave widely different readings with respect to other Mustran cells at the same level at all applied loads. The readings of such Mustran cells were discarded. For each level of Mustran cells, curves were plotted to show the variation of the readings of each Mustran cell at the level due to variation in the load applied at the top of the shaft. For convenience, all the curves of the readings of Mustran cells at a particular level were plotted on the same sheet. These curves were then smoothed to discount obviously erratic readings. Finally, an average curve for the specific level was drawn on the basis of the smoothed curves. The average curves for each level were then checked to ensure that for any specific applied load the strain at a particular level was not greater than the strain at a lower level, after making necessary adjustments for the difference in the shaft diameter at the particular level with respect to the shaft diameter at the top of the shaft.

The modified Mustran cell readings of each test shaft were analyzed using the computer program DARES developed by Barker and Reese (1970) at The University of Texas at Austin. The theoretical bases of the program may be studied in the above noted reference. A brief description of the important details of the program are furnished next.

The readings of the Mustran cells near the top of the shaft are used to convert the readings of the Mustran cells at all other levels into units of load such as pounds. In view of the fact that the Mustran cells near the top of the shaft are used to calibrate the readings of other cells, these Mustran cells are referred to as the calibration cells and the level of their location is known as the calibration level. The Mustran cells near the top of the shaft are used as calibration cells for two reasons. First, the applied load at the calibration level is known since there is no load transfer above the ground surface. Second, the dimensions of the shaft above the ground surface are known quite accurately by direct measurements. For a particular applied load, the load at any Mustran cell level is obtained by computing the product of three quantities. These quantities are: the applied load; the ratio of the average reading of Mustran cells at the level to the average reading of the calibration cells; and the ratio of the square of the shaft diameter at the level to the square of the shaft diameter at the calibration level. In order to improve the accuracy of analysis, the program first obtains the best fitting polynomial curve which relates the average readings of the calibration cells to the known applied load. The coefficients of the calibration curve are then used to convert the readings of the Mustran cells at all levels to compute loads after making necessary adjustment for variation in the shaft diameter.

After the loads at all Mustran cell levels are computed for an applied load, the best fitting polynomial curve, representing the load distribution curve for the applied load, is then obtained by the method of least squares. The load distribution curve is then used to compute load transfer information using the principles outlined in Chapter 3.

The program DARES has the additional capability of plotting load-settlement, load distribution, load transfer, and calibration curves after analyzing the Mustran cell readings according to the approach outlined above. The results obtained by the use of this program are discussed in the next few paragraphs. During the forthcoming discussion it will be helpful to note the positions of the soil layers with respect to each test shaft and the positions of the Mustran cells in each test shaft. This information was presented earlier in Figs 5.7 to 5.9.

Load Distribution Curves

The load distribution curves for the shafts DT1 and MT3 were obtained by utilizing all the capabilities of the program DARES. However, in the cases of shafts MT1 and MT2, the program could not be utilized beyond the stage of obtaining loads at all Mustran cell levels for each applied load. This limitation became apparent when preliminary examination of the data revealed that it would be more accurate to fit a load distribution curve over the depth of each soil layer instead of a single polynomial curve, furnished by the program DARES, for the entire penetration length of the shaft. Sudden contrasts in the physical properties of adjacent soil layers indicated the need for fitting individual load distribution curves over the depth of each soil layer.

In view of the above, the load distribution curves for the shafts MT1 and MT2 were fitted by inspection of the load points plotted by the program DARES along the depth of the shaft, with due regard to the vertical extent of each soil layer. It was assumed that a straight line load distribution curve for each soil layer would not cause a significant error in estimating the average load transfer characteristics of the soil within a soil layer. The principles outlined in Chapter 3 were used to compute the load transfer versus pile movement curves for the shafts MT1 and MT2.

The load distribution curves for all the test shafts are presented in Figs 8.1 to 8.4. The dashed segments of the load distribution curves represent those portions of the curves for which an estimate of the Mustran cell readings could not be made.

Load Transfer Curves

Figures 8.5 to 8.7 represent the load transfer curves based on the load distribution curves presented earlier.

A study of the load transfer curves pertaining to the shafts MT1 and MT2 indicated several inconsistencies. For example, in Fig 8.5, the upper clay layer in the shaft MT1 appears to have a higher load transfer capability than the lower clay layer, even for large movements, although the lower clay has about twice the average shear strength of the upper clay. Moreover, the load transfer for the sand and gravel layer is rather high since the $\bar{p} \tan \bar{\phi}$ value for this soil is less than the load transfer indicated for the sand and gravel layer. Similar discrepancies exist in the load transfer results of

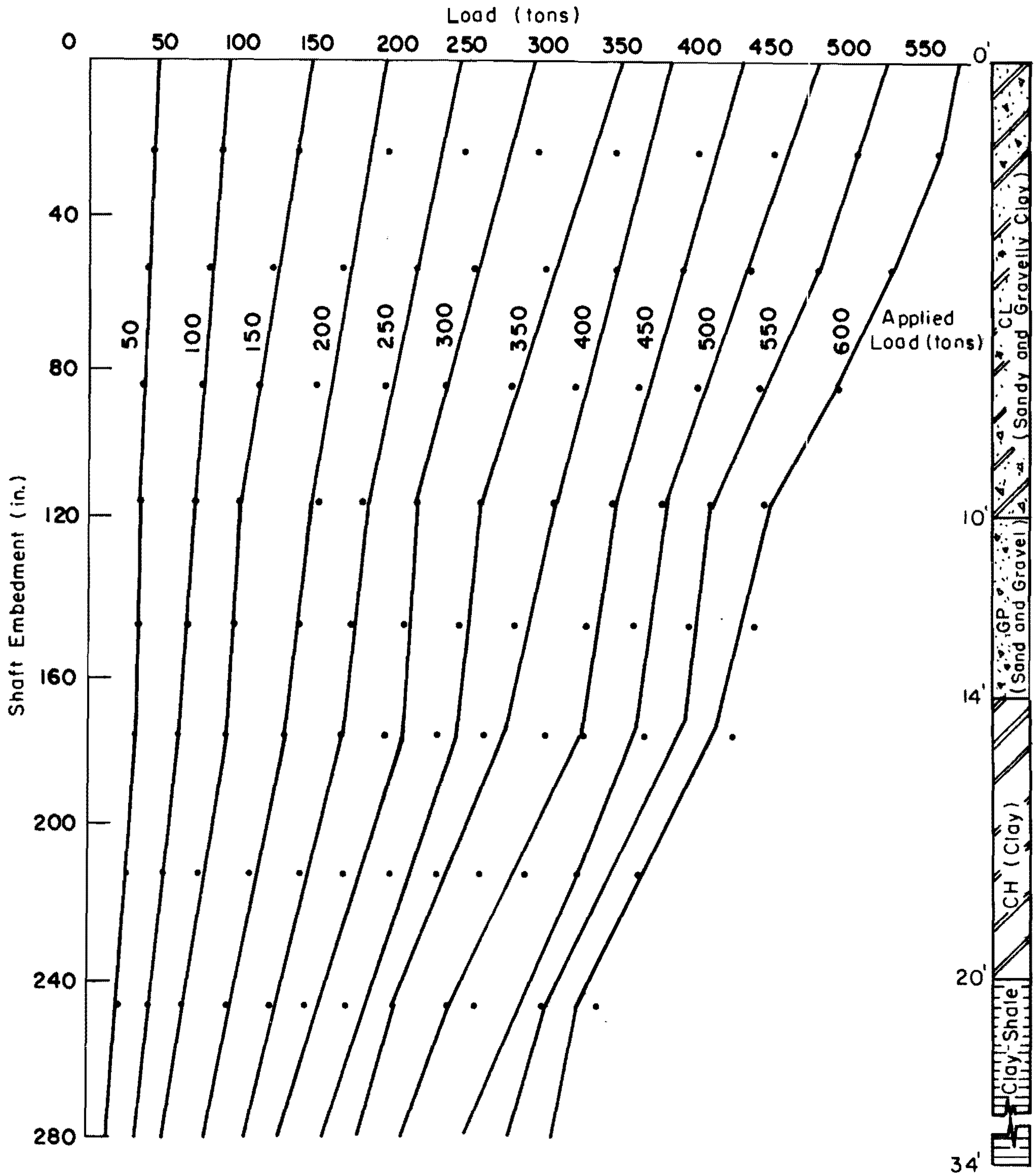


Fig 8.1. Load distribution curves for shaft MT1 built by Casing Method.

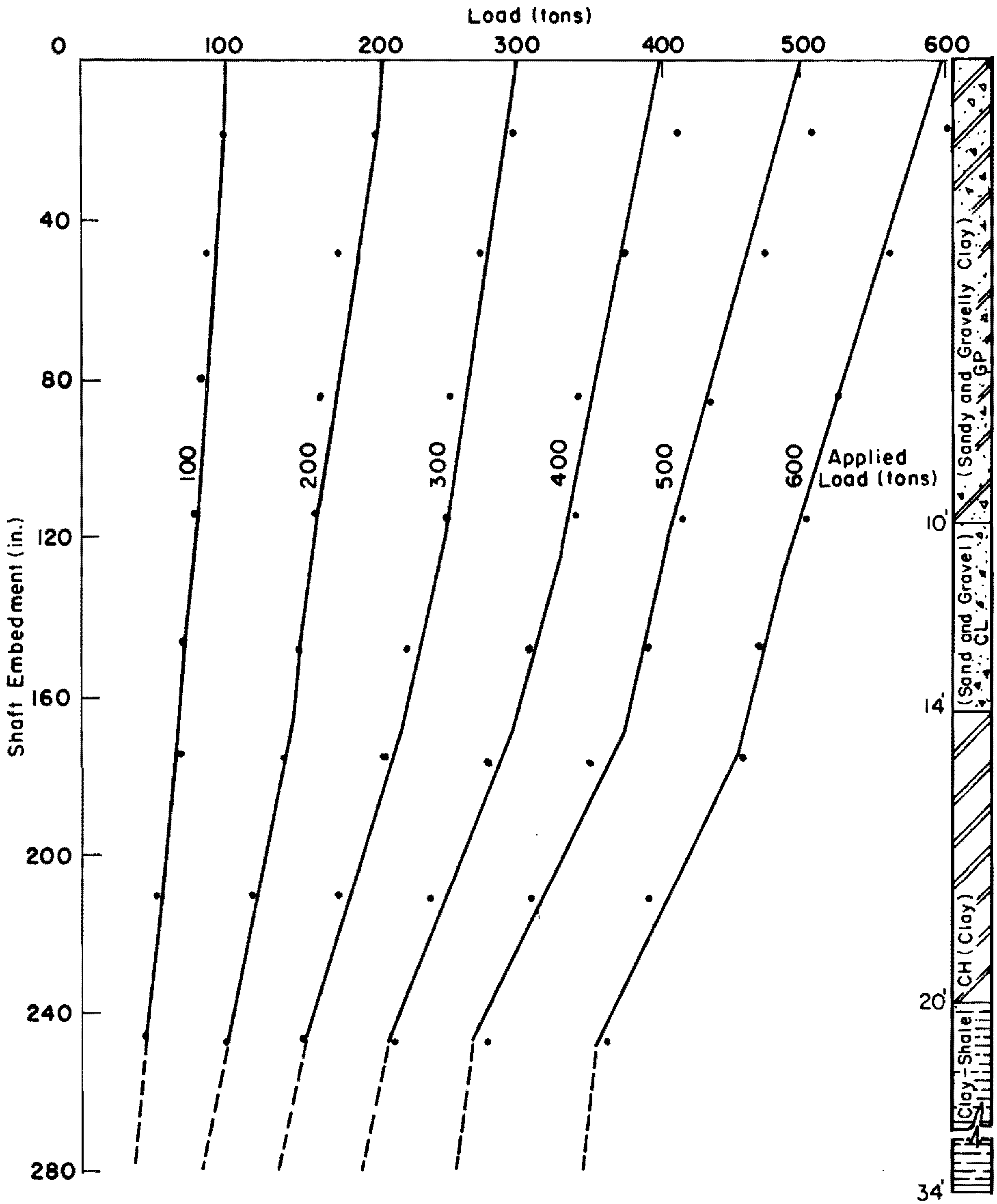


Fig 8.2. Load distribution curves for shaft MT2 built by Slurry Method.

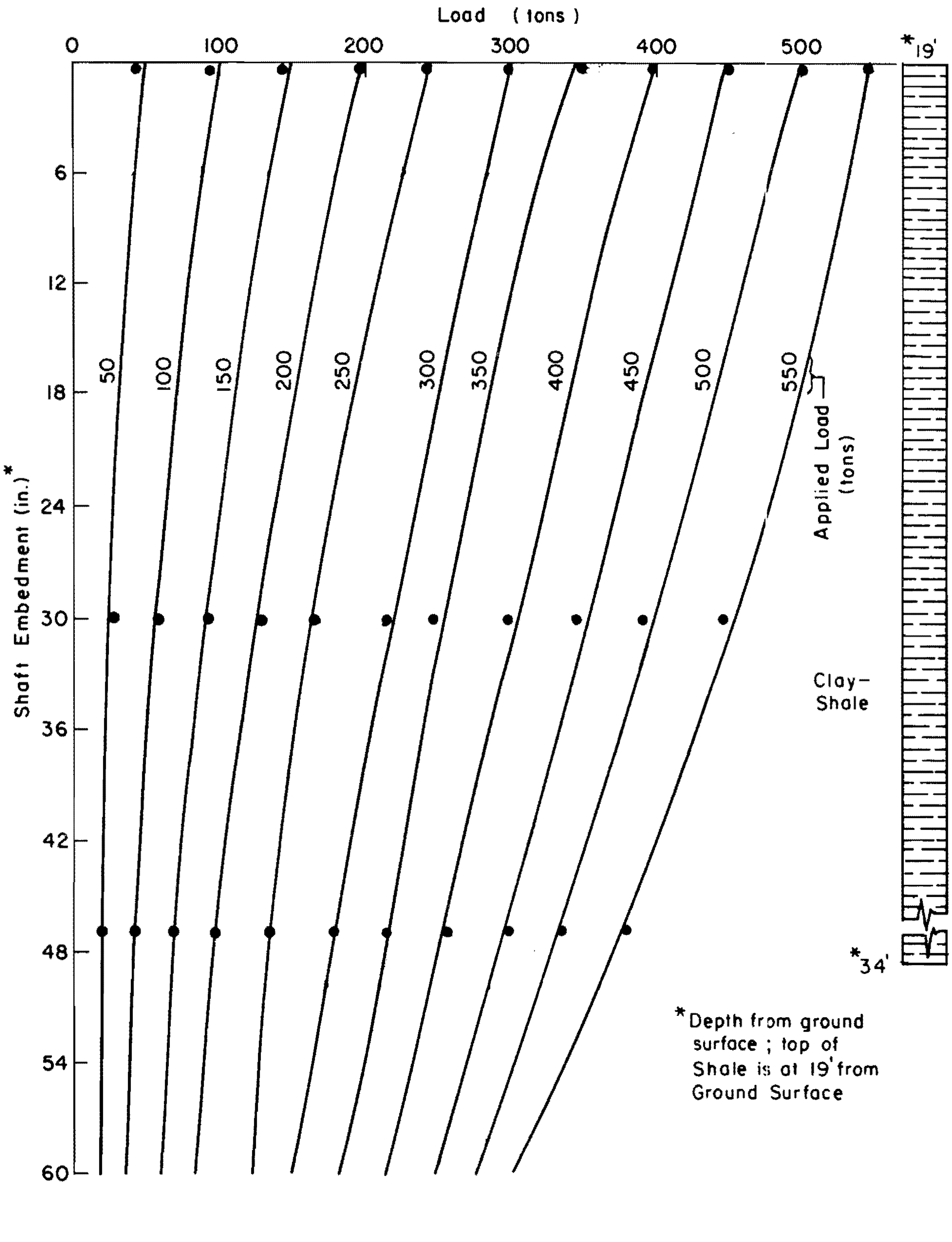


Fig 8.3. Load distribution curves for shaft MT3 built by Dry Method.

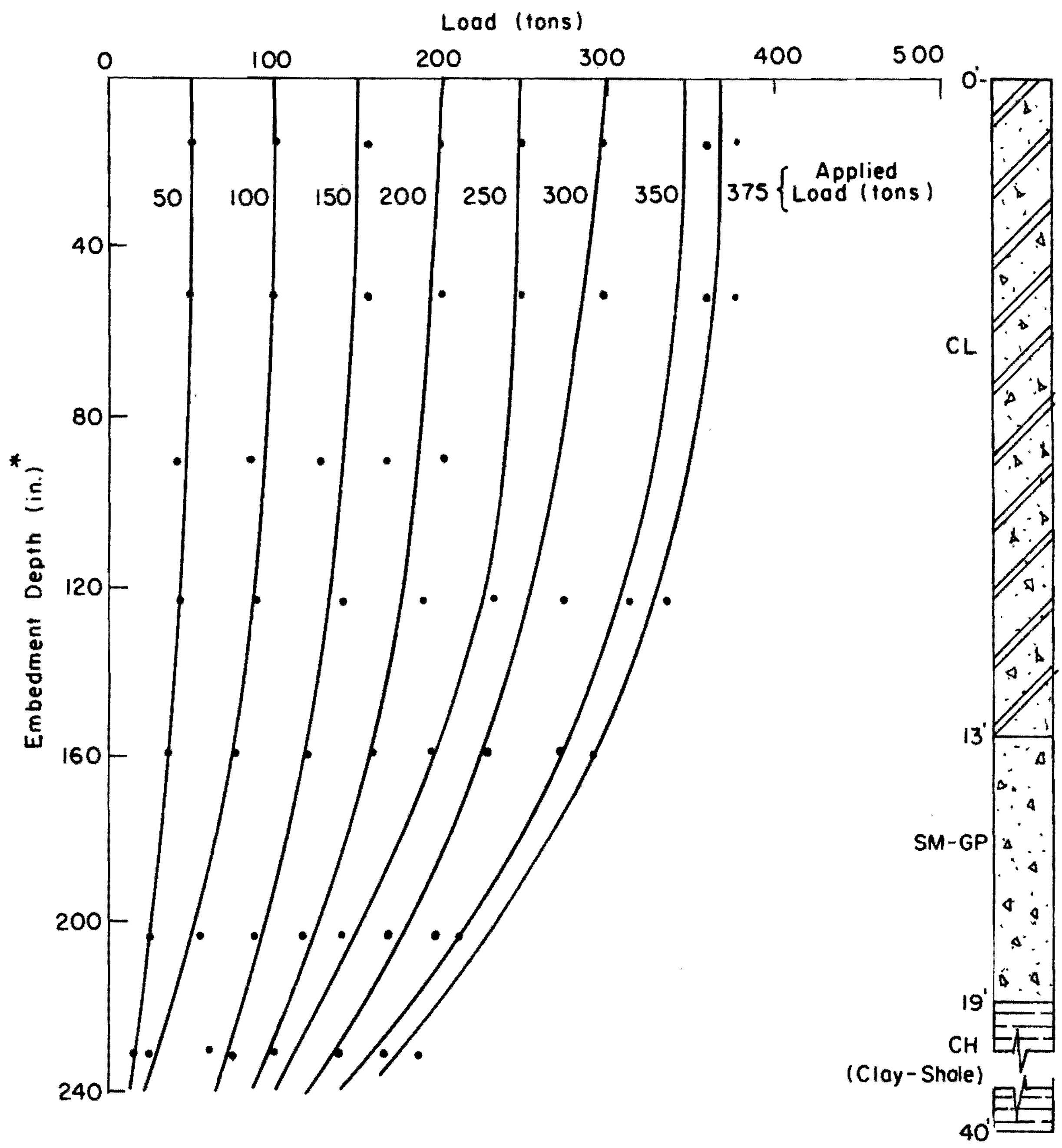


Fig 8.4. Load-distribution curves for shaft DT1 built by Casing Method.

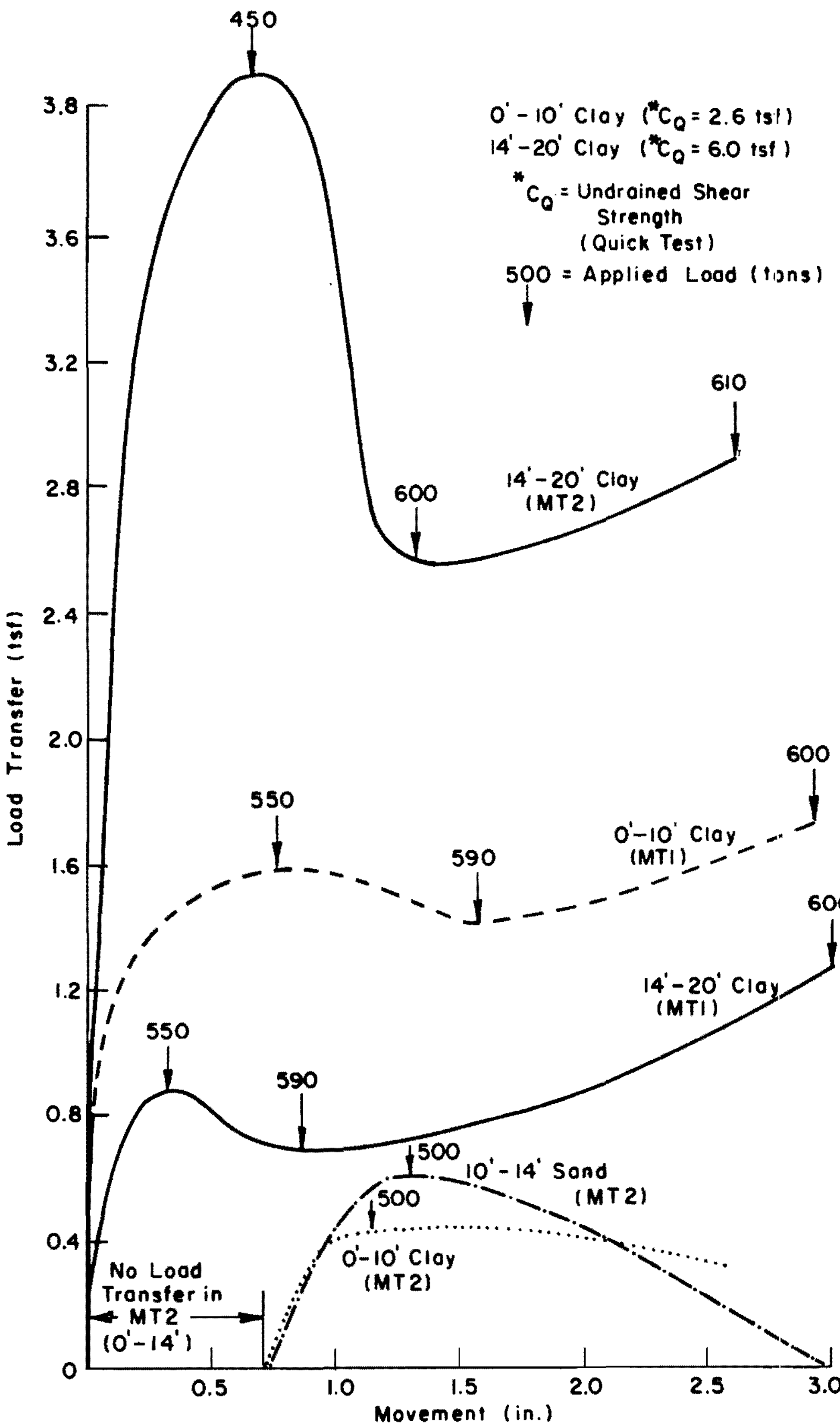


Fig 8.5. Load transfer curves for shafts MT1 and MT2 built by Casing Method and Slurry Method.

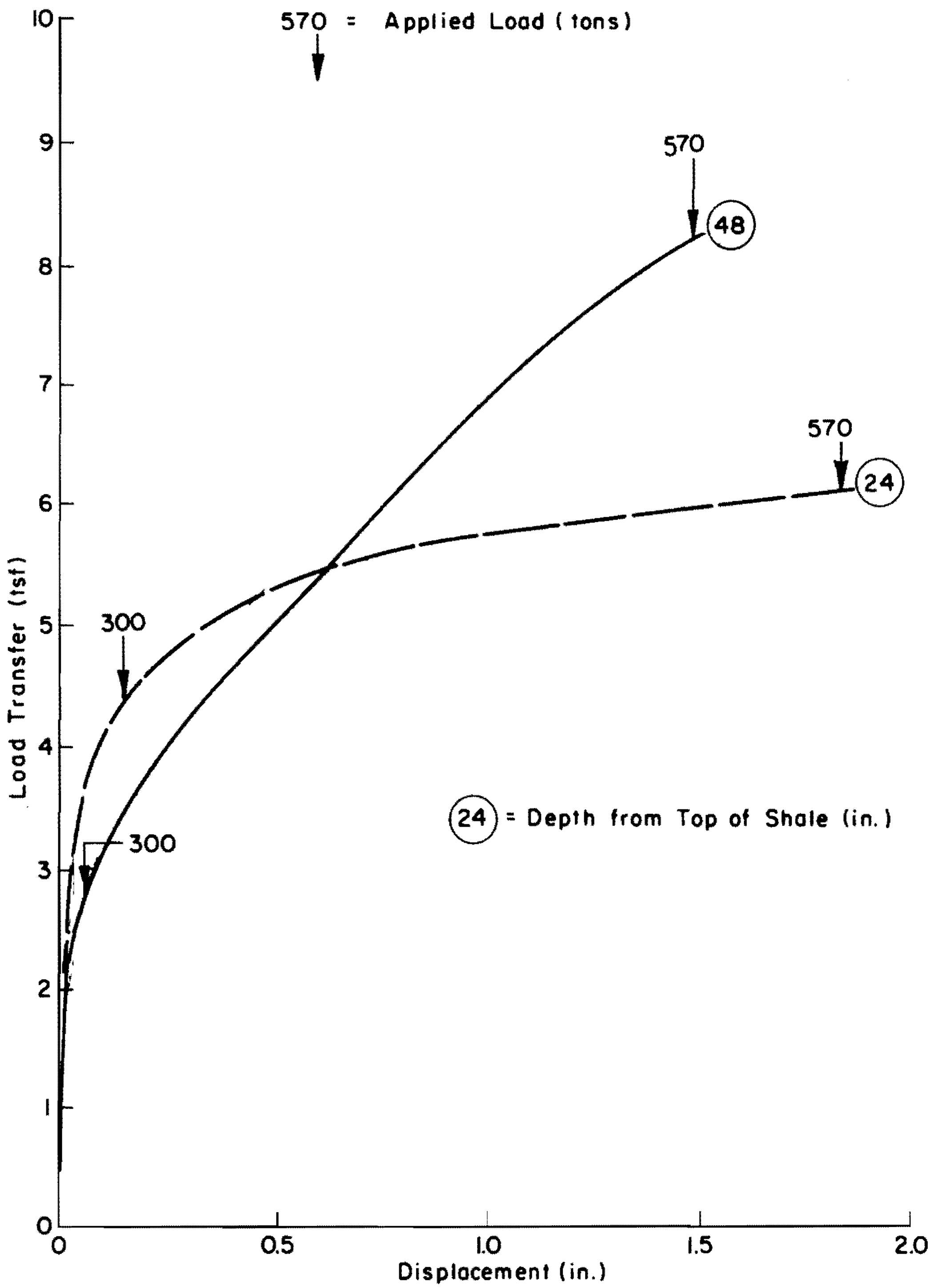


Fig 8.6. Load transfer curves for shaft MT3 built by Dry Method.

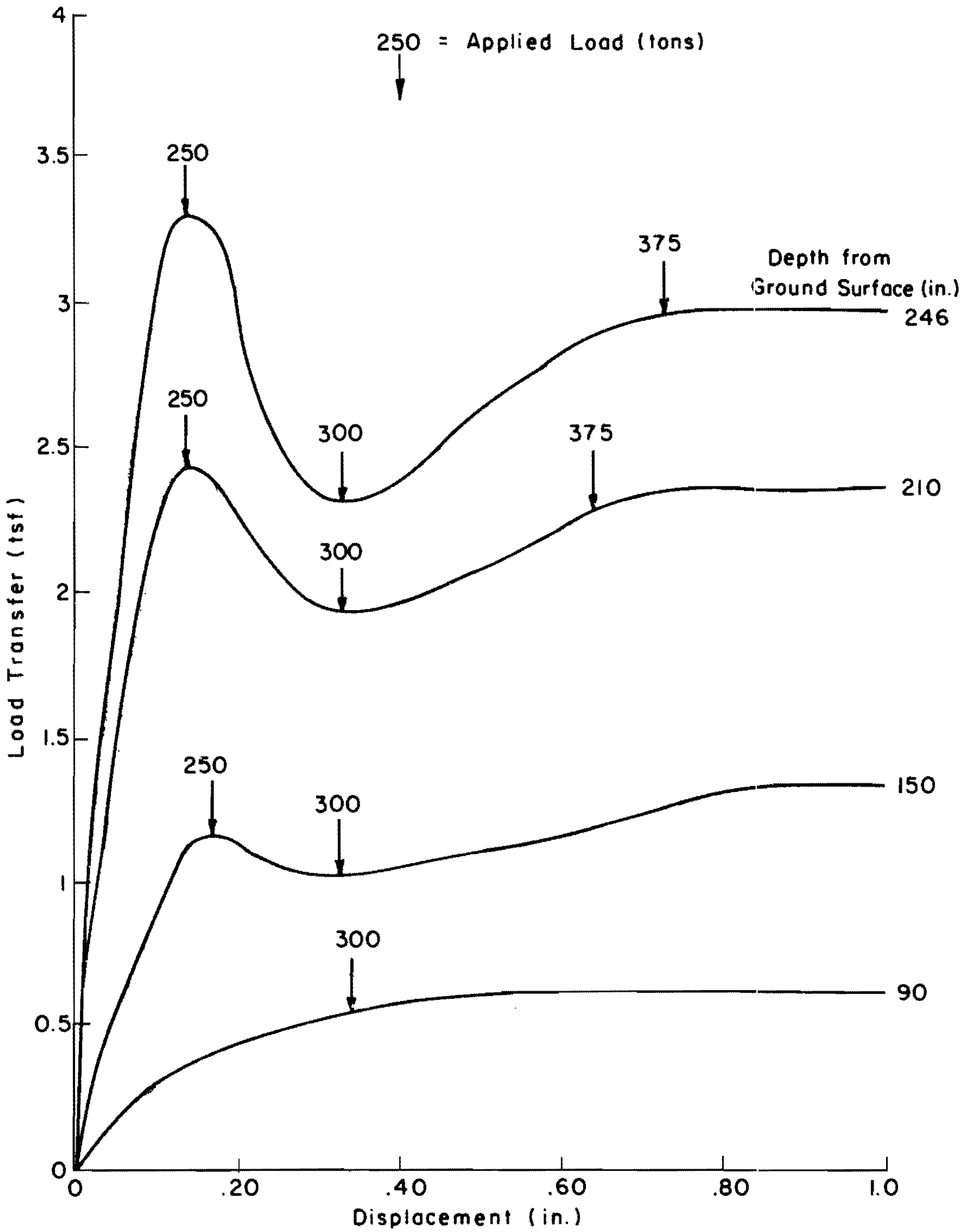


Fig 8.7. Load-transfer curves for shaft DT1 built by Casing Method.

the shaft MT2 as well. The load transfer curves for the shafts MT1 and MT2 are, therefore, ignored insofar as the behavior of the soil layers above the shale layers are concerned.

The load transfer curves for the shafts MT3 and DT1 indicate that within the shale layer the load transfer increased with depth. It is also seen that maximum load transfer was achieved at displacements of about 1/4 in. These trends are in general agreement with the results reported by Vijayvergiya, Hudson, and Reese (1969). It should be noted that in all instrumented drilled shafts studied to date the penetration of the shaft into the shale layer was less than 5 ft. It is likely that for large penetrations of drilled shafts into a shale layer the trends of load transfer may vary with changes in the depth of penetration into the shale layer as observed for clays and sands by O'Neill and Reese (1970) and Touma and Reese (1972), respectively.

Base Resistance Curves

The base resistance curves, also known as tip-load versus tip-movement curves, are shown in Fig 8.8 for all the test shafts. These curves were obtained with the help of the program DARES. During the discussions and descriptions that follow, the term ultimate base resistance will be used to indicate the pressure q at which the movement of the tip equals 5 percent of the tip diameter. It is apparent from the tip-load versus tip-movement curves that the ultimate base resistance, q, of the shafts at the Montopolis Site varied from about 53 tsf, for the shaft MT2, to about 63 tsf, for the shaft MT3. The value of q for the shaft DT1 was about 25 tsf. It is also seen from the shapes of these curves that the shale acted as a stiffer material for the shafts MT1 and MT3 in comparison to the shaft MT2, although all these three shafts were located very close to each other. It may be noted that both the shafts MT1 and MT3 were installed into the shale by pouring concrete in a precleaned dry hole, whereas the shaft MT2 was installed into the shale by the slurry displacement method. In the shaft DT1 the shale, although having a relatively low value of q, appears to act as stiff soil. The shaft DT1 was installed into the shale layer in the same manner as MT1. In all cases, at least 50 percent of the ultimate base resistance was mobilized at a tip movement of 1/2 in.

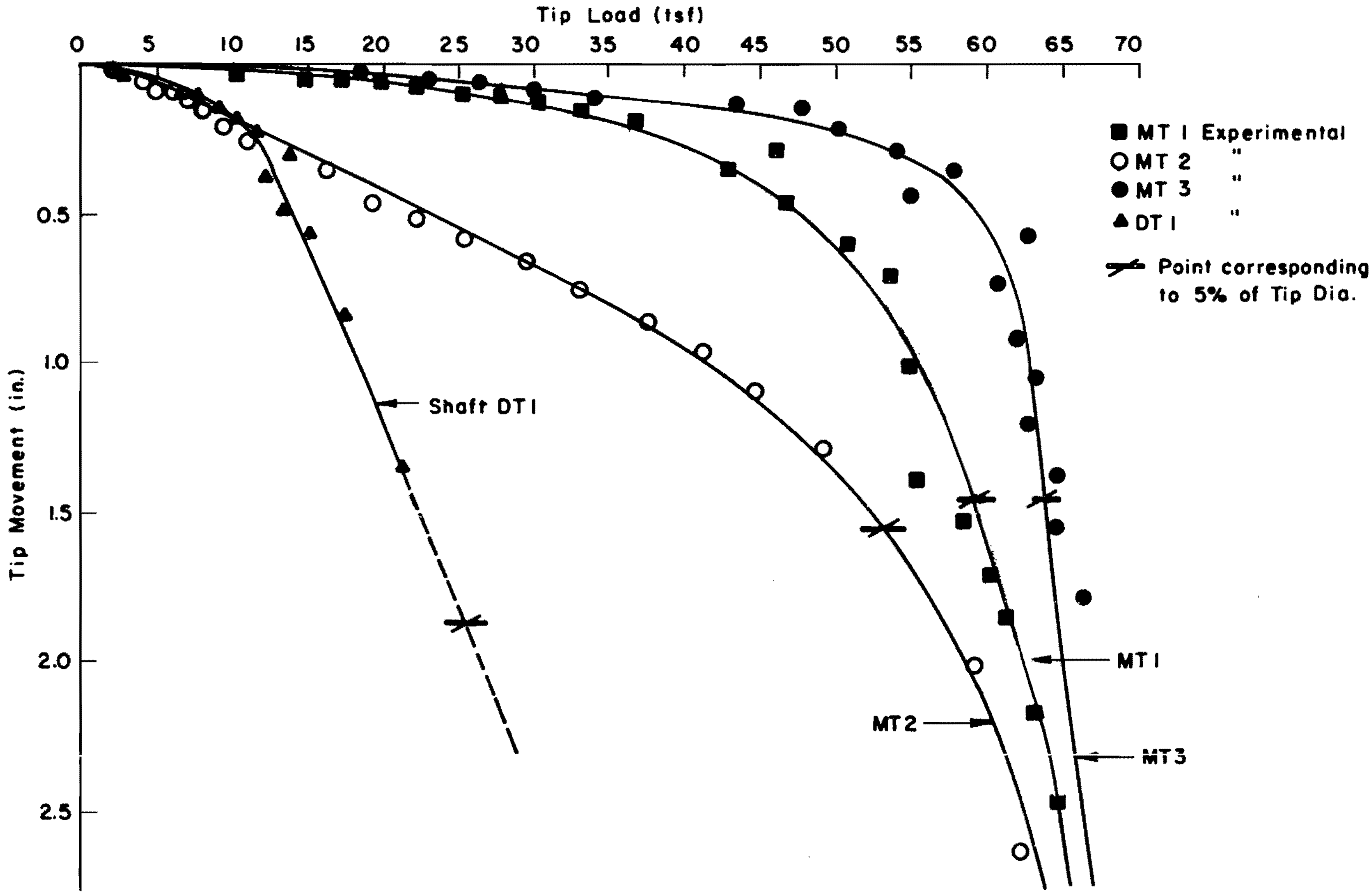


Fig 8.8. Tip-load versus tip-movement curves for all test shafts.

Summary of Results and Their Discussion

In order to clearly understand the phenomenon of soil-structure interaction for the test shafts, the following information will be examined closely:

(1) load-settlement curves observed in the field;
(2) triaxial test, THD cone resistance, and Dutch cone resistance data;
(3) base resistance data; and
(4) load transfer data.

Complete load-settlement curves were presented earlier in Chapter 7. These curves are reproduced in Fig 8.9 excluding the details of the unloading and reloading phases. These curves indicate that the failure loads for the shafts MT1, MT2, MT3, and DT1 were 600, 610, 570, and 450 tons, respectively. The term failure load is used here to indicate the load at which the shaft could not support additional load with additional movement into the ground. In physical terms, at the failure load, additional pressure could not be built into the hydraulic lines by the pump although the shaft kept moving downwards. Sometimes, the failure load, as defined above, is also referred to as the plunging load. The load-settlement curves show that at least 50 percent of the failure load was resisted at a settlement of about 3/8 in., and at least 75 percent of the failure load was resisted at a settlement of 3/4 in. These figures indicate that the clay-shale was generally acting as a stiff brittle material.

The triaxial test, THD cone penetration resistance, the standard penetration resistance, and the Dutch cone penetration resistance data were presented in Chapter 5. These data were evaluated to estimate the variation of the shear strength of soil along the length of the test shafts. Triaxial test data could not be obtained for the shaft DT1. Fig 8.10 shows the various estimates of shear strength along the length of the tests shafts at the Montopolis Site. The shear strength values were estimated from the THD cone penetration resistance and the standard penetration resistance data using the following correlations suggested by Engeling and Reese (1974):

$$c_Q = \frac{N_{SPT}}{15} \tag{8.1}$$

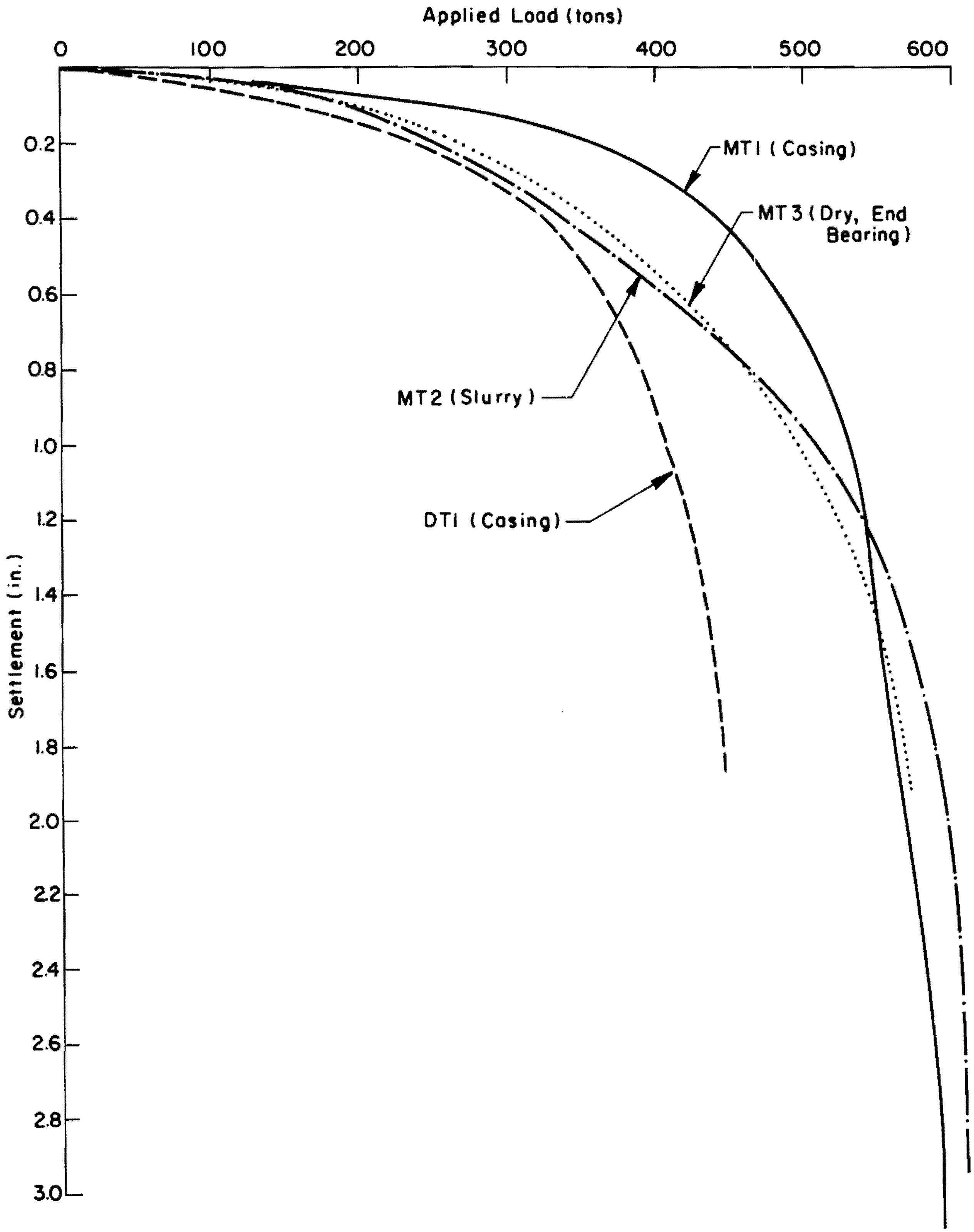


Fig 8.9. Load-settlement curves for all test shafts.

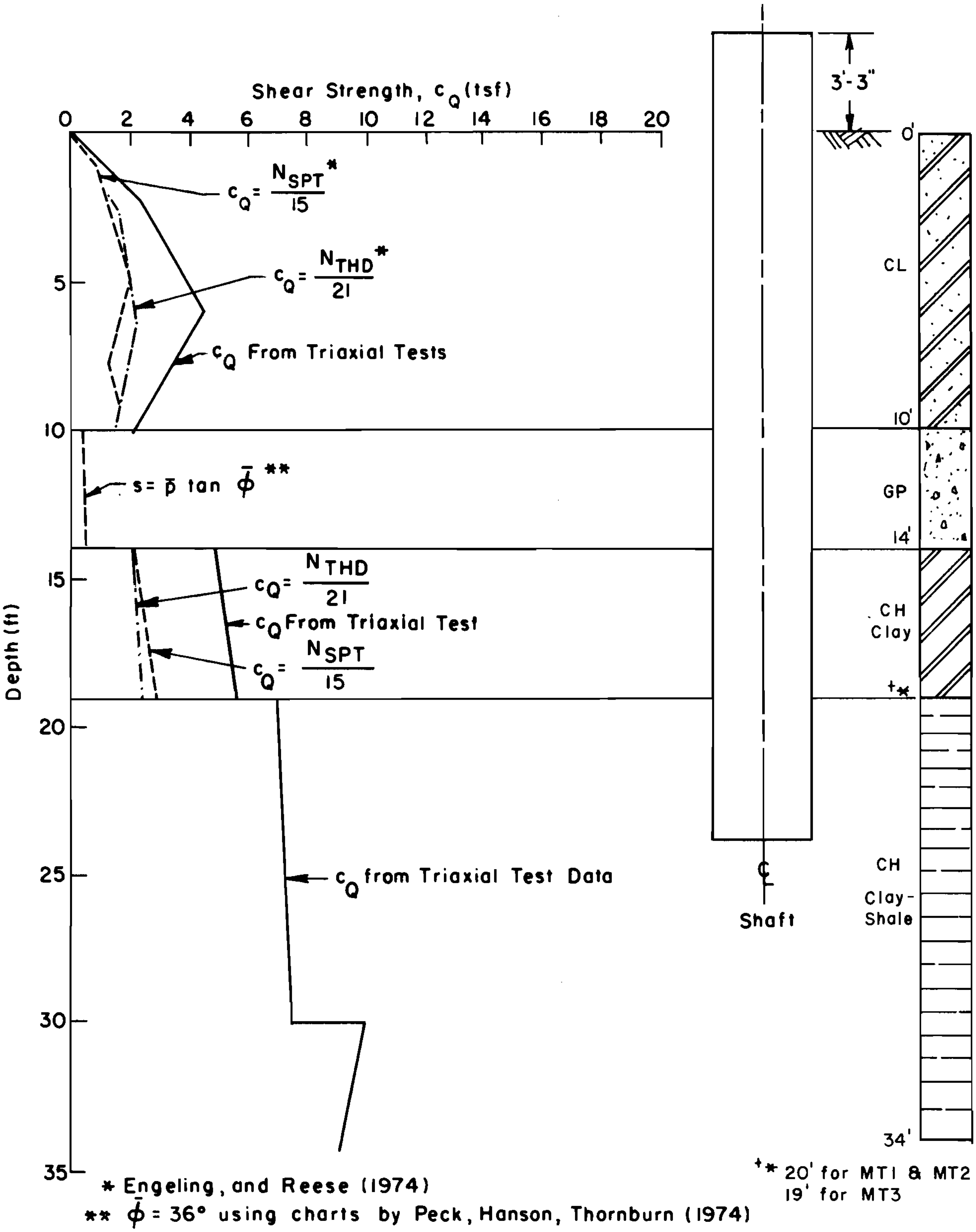


Fig 8.10. Shear strength profile at the Montopolis Site from field and laboratory data.

$$c_Q = \frac{N_{THD}}{21} \tag{8.2}$$

in which the terms c_Q , N_{SPT} , and N_{THD} are defined as follows:

c_Q = Unconsolidated undrained (quick) shear strength of soil (tsf);

N_{SPT} = Total number of blows required for the second and third 6-in. penetrations during three consecutive 6-in. penetrations of a standard splitspoon driven with a 140-pound hammer falling 30 in. for each blow, in accordance with the ASTM Standard D1586-67. When the standard splitspoon cannot penetrate in the manner described above, N_{SPT} is defined arbitrarily as follows: $N_{SPT} = \frac{12}{s} \times 100$ where s = spoon penetration in inches due to 100 consecutive blows.

N_{THD} = Number of blows required to penetrate the standard THD cone a distance of 12 in. into the soil in accordance with the procedure described in the Foundation Exploration and Design Manual (1972) of the Texas Highway Department. When the cone cannot penetrate 12 in. into the soil in 100 consecutive blows, N_{THD} is defined arbitrarily as follows: $N_{THD} = \frac{12}{s} \times 100$ where s = cone penetration in inches due to 100 consecutive blows.

Fig 8.11 shows the best estimate of shear strength profile at the Montopolis Site. Since both triaxial test and Dutch cone resistance data were available for the Montopolis Site, it was possible to determine $N_{c\ ,\ cone}$, the bearing

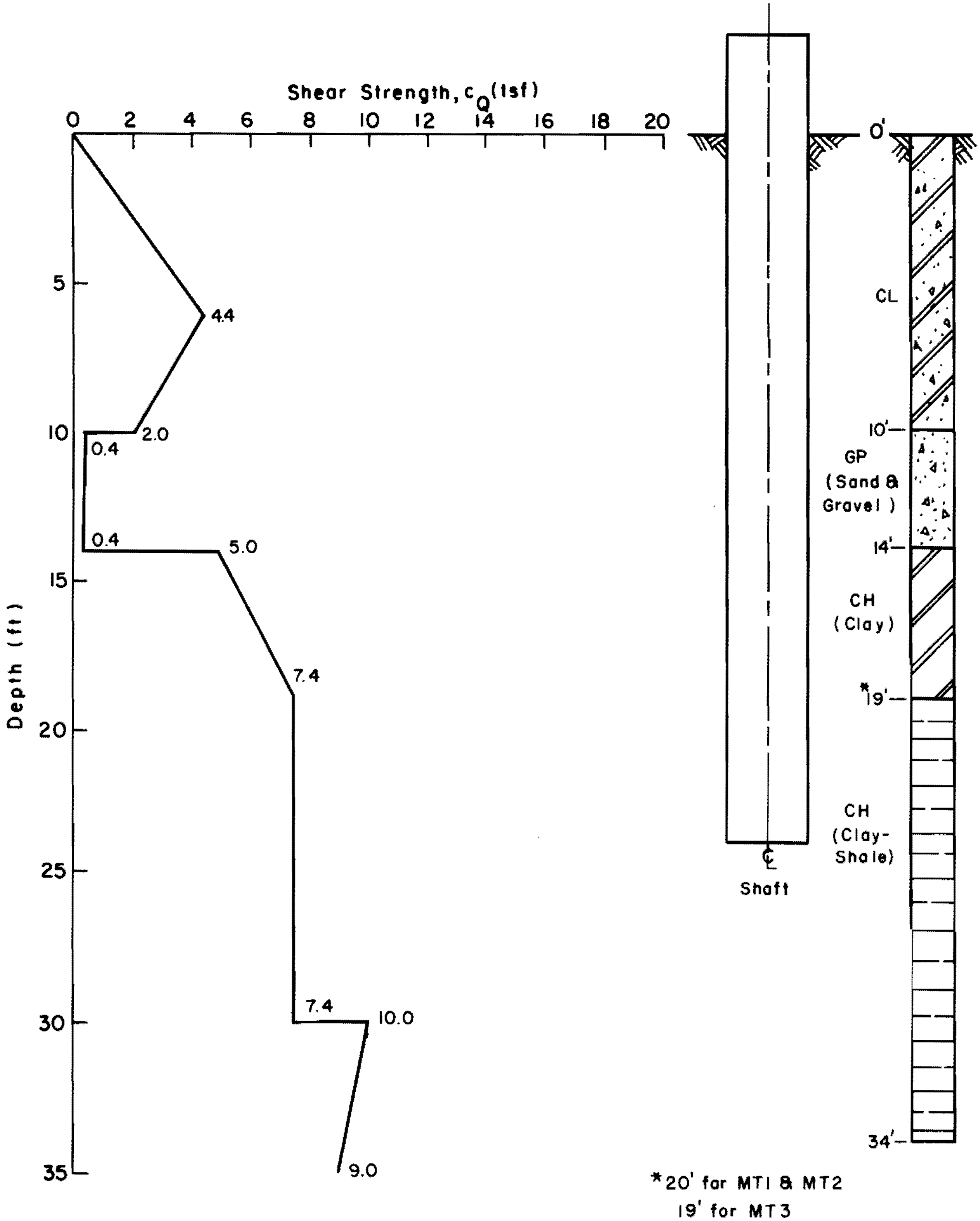


Fig 8.11. Estimate of shear strength profile at the Montopolis Site.

capacity factor for the cone, using the following approach suggested by Schmertmann (1975) for clay soils:

$$N_{c\,,\,cone} = \frac{q_c - p_o}{c_Q} \tag{8.3}$$

in which q_c is the unit bearing capacity on cone in a quick cone penetration test, p_o is the overburden pressure at the depth at which the cone penetration test has been performed, and c_Q is the shear strength of the soil in a quick triaxial test. In equation (8.3) above the parameters q_c , p_o , and c_Q have identical units and $N_{c\,,\,cone}$ is a dimensionless parameter. Since the Dutch cone tests were made at the surface of the exposed shale, p_o was taken as equal to zero in equation (8.3). The value of $N_{c\,,\,cone}$ computed thus was found to be about 19. The following equation was then used to estimate c_Q at the Dallas Site, assuming that $N_{c\,,\,cone}$ at both the sites was identical:

$$c_Q = \frac{q_c}{19} \tag{8.4}$$

Computations to determine $N_{c\,,\,cone}$ are shown in Table 8.1. The values of c_Q estimated for the Dallas Site are shown in Table 8.2. A possible explanation for the relatively high value of $N_{c\,,\,cone}$ in comparison to the usual bearing capacity factor N_c equal to 9 will be offered later in this chapter after presenting the N_c values obtained for the test shafts.

The base resistance data obtained by analysis of Mustran cell readings can now be correlated to the shear strength of the shale. Table 8.3 is an attempt to estimate the bearing capacity factor N_c and the relationship between N_{THD} and the ultimate tip resistance for all the test shafts.

TABLE 8.1. $N_{c\text{ , cone}}$ VALUES FROM DUTCH CONE AND TRIAXIAL TEST DATA AT MONTOPOLIS

Depth Below Top of Shale (ft)	Dutch Cone Test Results				
	P_{max} (lb)	A_c (sq.in.)	q_c (tsf)	c_Q (tsf) By Triaxial Tests	$N_{c\text{ , cone}} = \frac{q_c}{c_Q}$
3	3200	1.54	149.7	7.4	20.2
	3150	1.54	147.4	7.4	19.9
	3290	1.54	153.9	7.4	20.8
5	3420	1.54	160.0	8.0	20.0
	2800	1.54	131.0	8.0	16.4
	2872	1.54	134.3	8.0	16.8

Average value of $N_{c\text{ , cone}}$ = 19.0.

P_{max} = Maximum force applied to the cone during the test to cause its full penetration into the soil (lb)

A_c = Area of the base of cone (sq.in.)

$$q_c = = \frac{P_{max}}{A_c} \times \frac{144}{2000} \text{ (tsf)}$$

TABLE 8.2. ESTIMATE OF c_Q FROM DUTCH CONE TESTS AT DALLAS

(Depth below top of shale = 5.0 ft)

P_{max} (lb)	A_c (sq.in.)	q_c (tsf)	$N_{c\text{ , cone}}$	$c_Q = \frac{q_c}{N_{c\text{ , cone}}}$ (tsf)
1350	1.54	63.1	19.0	3.3
1260	1.54	58.9	19.0	3.1
1300	1.54	60.8	19.0	3.2

Average value of c_Q at the Dallas Site = 3.2 tsf.

TABLE 8.3. ESTIMATES OF N_c, AND FACTORS CORRELATING q TO N_{THD} AND c_Q

Shaft	q (tsf)	c_Q (tsf)	$N_c = \frac{q}{c_Q}$	N_{THD}	$R = \frac{N_{THD}}{q}$	$\frac{N_{THD}}{c_Q} = S$
MT1	59.5	7.4	8.0	550	9.2	74.3
MT2	53.5	7.4	7.2	550	10.3	74.3
MT3	64.0	7.4	8.6	550	8.6	74.3
DT1	25.5	3.2	8.5	250	9.8	78.1

Average value of N_c = 8.1

Average value of R = 9.5

Average value of S = 75.3

As pointed out earlier in this chapter, the load transfer results for the shafts MT3 and DT1 could be obtained fairly accurately on the basis of the Mustran cell readings. However, in the case of the shafts MT1 and MT2, the Mustran cell readings were generally questionable. It was, therefore, necessary to employ some indirect means to estimate the load transfer characteristics of the shale in these two shafts. Three different studies were made in this regard, as explained below.

Study No. 1 consisted of the following steps:

(1) Assume that the portion of the shaft MT1 or MT2 embedded into the shale layer has the same load transfer characteristics as the shaft MT3.

(2) Using the unit values of base resistance and load transfer in shale obtained for the shaft MT3, compute the tip load versus tip movement values for the shafts MT1 and MT2, assuming that they both terminated at the top of the shale.

(3) Use the load transfer data obtained for the shafts MT1 and MT2, for the soil layers above the clay shale.

(4) Use the load transfer results of step 3 and tip load versus tip movement results of step 3 for the shafts to compute the load-settlement curve for each shaft by using the approach suggested by Coyle and Reese (1966) as outlined in Chapter 2. Use the computer program PX4C3 developed at The University of Texas at Austin for these computations.

Study No. 2 consisted of the same approach as used for Study No. 1 with the exception that the load transfer data for the shaft were used by studying the values of α , the ratio of maximum load transfer to c_Q , determined in several earlier studies on drilled shafts conducted at The University of Texas at Austin by the Center for Highway Research (89 and 176 Series Reports). The values of c_Q as noted earlier were used for this study. In accordance with the procedure suggested by Engeling and Reese (1974), it was assumed that there was no load transfer in the first five feet of penetration of the shaft. Table 8.4 gives a comparison of the experimental failure loads with the failure loads computed by the above studies. The comparisons between computed and experimental load-settlement curves for MT1 and MT2 are shown are shown for Study No. 1 and Study No. 2 in Figs 8.12 and 8.13, respectively.

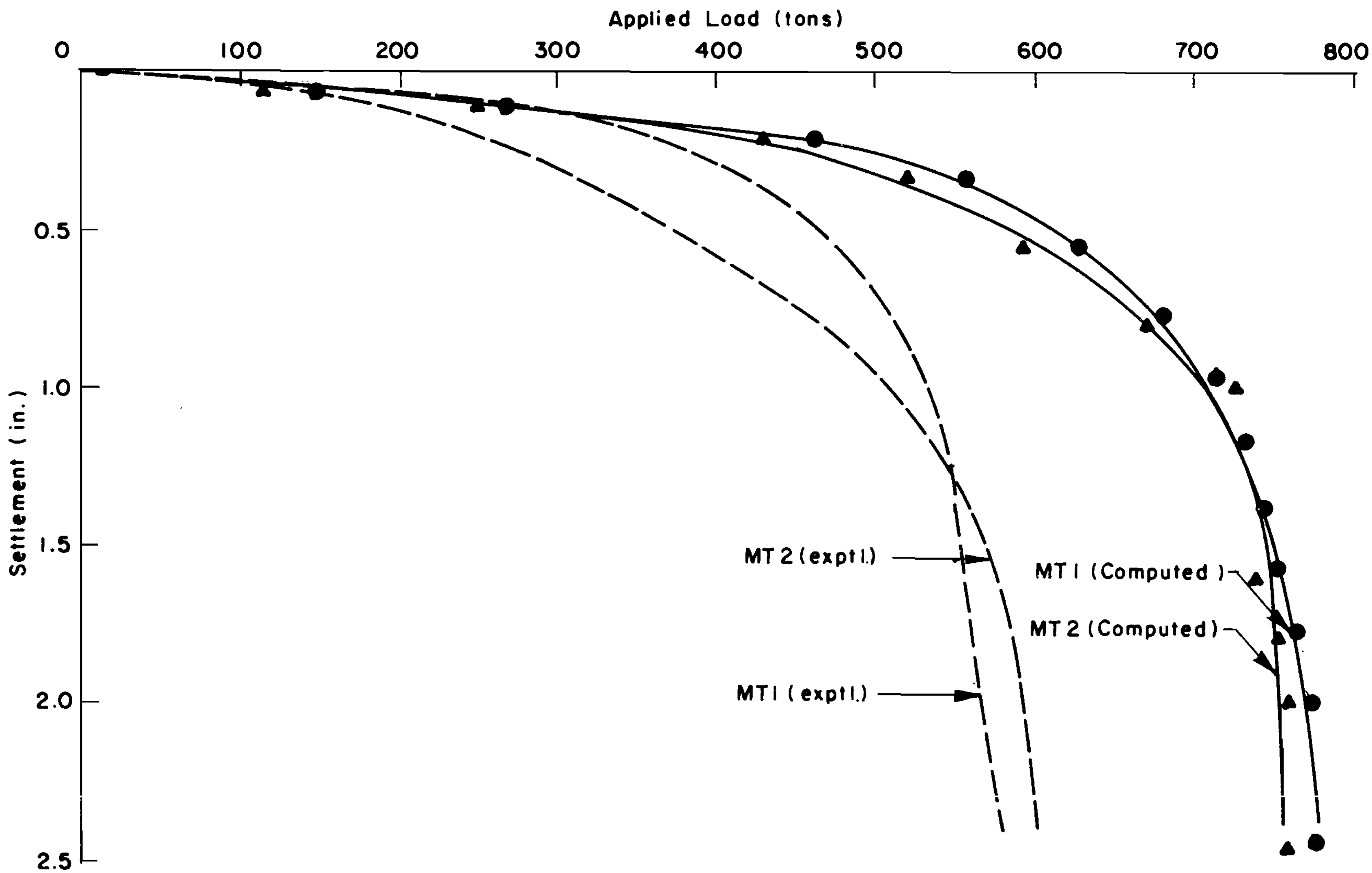


Fig 8.12. Comparison of experimental and computed load-settlement curves: study no. 1.

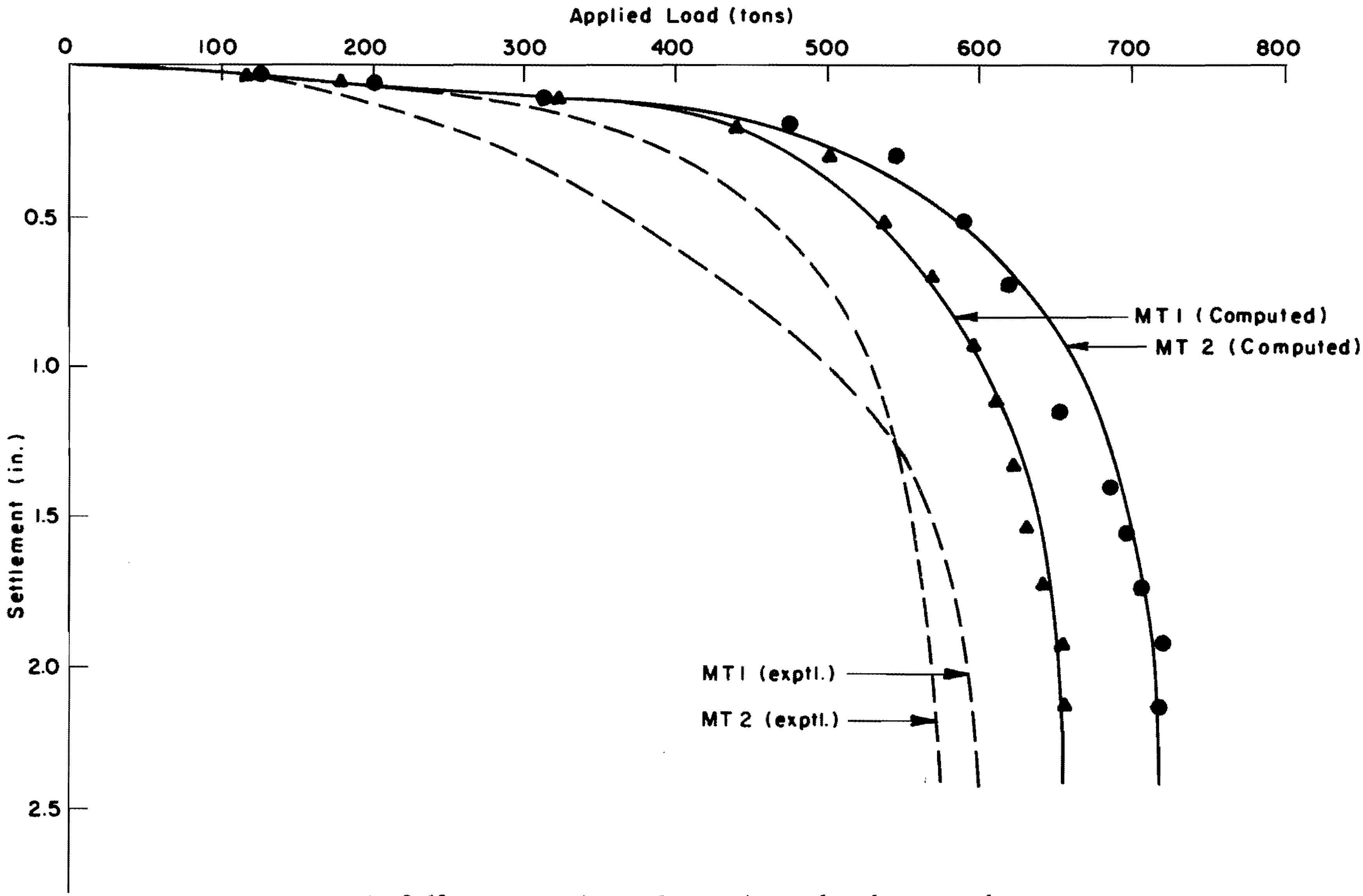


Fig 8.13. Comparison of experimental and computed load-settlement curves: study no. 2.

TABLE 8.4. COMPARISON OF EXPERIMENTAL AND COMPUTED FAILURE LOAD FOR SHAFTS MT1 AND MT2 USING INDIRECT METHODS

Shaft	Q_{ult} (tons)	Study No. 1		Study No. 2	
		$Q_{ult,1}$ (tons)	R_1	$Q_{ult,2}$ (tons)	R_2
MT1	600	700	1.30	655	1.09
MT2	610	760	1.25	720	1.18

Q_{ult} = Failure load determined by field test (tons)

$Q_{ult,1}$ = Failure load determined by Study No. 1 (tons)

$Q_{ult,2}$ = Failure load determined by Study No. 2 (tons)

R_1 = Ratio of $Q_{ult,1}$ to Q_{ult}

R_2 = Ratio of $Q_{ult,2}$ to Q_{ult}

From the relatively large values of R_1 and R_2 as determined above, it became apparent that the assumption made in step 1 for the above two studies was not applicable and that in both the shafts MT1 and MT2 the load transfer was relatively less than in MT3. It was, therefore, decided to compute load transfer in the shale for the shafts MT1 and MT2 in accordance with the steps for Study No. 3, outlined below:

(1) Use the Reese and Engeling (1974) load transfer criteria and determine the load transferred to the layers above the shale.

(2) Subtract the load transferred to the layers above the shale, Q , from the experimental failure load, Q_{ult}. Designate the net quantity as Q_{top}.

(3) Subtract the tip load Q_B at failure from Q_{top}. Designate the net quantity as Q_{side}.

(4) Load transferred to the shale is then computed as

$$\tau_{max} = \frac{Q_{side}}{2\pi D \ell}$$

in which

D = diameter of the portion of shaft embedded in shale (ft),

ℓ = length of the portion of shaft embedded in shale, and

τ_{max} = maximum load transfer in shale (tsf).

Table 8.5 shows the τ_{max} and α values computed for the shafts MT1 and MT2 using the approach outlined above.

Table 8.6 summarizes the relationships noted between load transfer and undrained shear strength for the shale on the basis of the four load tests.

TABLE 8.5. ESTIMATE OF τ_{max} AND α VALUES FOR SHAFTS MT1 AND MT2 USING THE INDIRECT APPROACH

Shaft	Q_{ult} (tons)	Q (tons)	Q_{top} (tons)	Q_B (tons)	Q_{side} (tons)	D (ft)	ℓ (ft)	τ_{max} (tsf)	c_Q (tsf)	α
MT1	600	83	517	273	244	2.42	3.75	4.28	7.4	0.58
MT2	610	89	521	278	243	2.58	3.92	3.82	7.4	0.52

TABLE 8.6. LOAD TRANSFER DATA FOR SHALE ON THE BASIS OF LOAD TESTS

Shaft	c_Q (tsf)	τ_{max} (tsf)	$\alpha = \frac{c_Q}{\tau_{max}}$	Installation Method
MT1	7.4	4.28	0.58	Casing, dry in shale
MT2	7.4	3.82	0.52	Slurry displacement
MT3	7.4	7.2	0.97	Dry
DT1	3.2	2.9	0.91	Casing, dry in shale

Note: c_Q is the shear strength obtained by a quick test in a triaxial testing machine.

It can be seen from Table 8.6 above that the load transfer characteristics of shale vary significantly. The shafts MT1 and DT1 were both installed by the casing method, and even the shaft MT3 was installed into the shale in a pre-cleaned dry hole as was done for the shafts MT1 and DT1. The value of α for the shaft MT1 is rather low in comparison to the α values found for DT1 and MT3. It may be recalled that the construction of shaft MT1 was not strictly in accordance with the casing method as was pointed out in Chapter 5. The casing of the shaft MT1 was pulled suddenly and continuously in single operation, and no slurry was noticed near the top of the shaft after the casing was pulled out completely. No slurry was used to construct DT1. It is difficult to relate this departure from the construction procedure to the low value of α, but it is desirable that future investigations be directed towards evaluation of the effects of construction procedures on the values of τ_{max} or α.

The α value for the slurry shaft is the lowest of all other α values. Engeling and Reese (1974) had suggested an α value of 0.5 in clay for those shafts in which there is a possibility of entrapment of the drilling mud between the sides of the shaft and the natural soil. A value of 0.5 for α appears reasonable for drilled shafts under 30 ft long installed into clay-shales whose shear strength as determined by quick triaxial tests is under 10 tsf. This statement is based upon very limited information obtained in this study. Under ideal conditions and when the method of installation is similar to the dry method, the value of α may be slightly increased, perhaps to 0.75 only for the type of drilled shafts and clay-shales mentioned above.

The base resistance data presented in Table 8.3 indicate that the shaft MT2 constructed by the slurry displacement method had the lowest ultimate base resistance value. For such shafts a conservative value of 7.0 for N_c appears reasonable. N_c may be increased to 8.0 if the dry method of construction is used.

For a circular footing resting on the surface of an elastic half space, the value of N_c is about 7.4. In the case of the Dutch cone tests, $N_{c\text{, cone}}$ was found to be about 19. According to Schmertmann (1975), the $N_{c\text{, cone}}$ values have been reported to range from 5 to 70. It is believed that in the case of the clay-shale the size effects were responsible for yielding a high value of N_c for the cone in comparison with the N_c for the test shafts. Bishop and Little (1967) reported similar experience for the fissured London clay. They concluded that small size samples or footings considerably overestimated the undrained shear strength of fissured London clay, not only at shallow depths where fissures were predominant but even at large depths where the fissures could not be visually detected.

Limitations of Test Results

The data on the behavior of drilled shafts in shales are very limited at the present time. The results of this study would have been more conclusive had the instrumentation for the shaft MT1 and the shaft MT2 behaved normally. Lack of triaxial test data for the shaft DT1 made it necessary to use an indirect approach to estimate the undrained shear strength of the clay-shale at the Dallas Site. In view of these limitations a conservative approach has been adopted for suggesting the design parameters to predict the axial capacity of drilled shafts in clay-shales. The suggested design values based on this study are included in the next chapter.

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

Introductory Remarks

The conclusions made in this study are based upon limited data on instrumented drilled shafts in clay-shales loaded axially to failure. Such data were not available prior to this study. It is hoped that future investigators will add to the knowledge gained through this research effort.

Some uncertainties were imposed because of limitations in determining the in situ shear strength of clay-shale. Further, in some instances the pattern of load distribution above the base could not be determined due to erratic behaviour of Mustran cells. These experimental difficulties necessitate recourse to a conservative approach in selection of design parameters.

However, the values of the ultimate base resistance obtained through field measurements are judged to be within 10 percent of actual values for all test shafts. A significant finding in this study is that the clay-shale at Montopolis provided an ultimate base resistance value ranging from 53.5 to 64.0 tsf; the clay-shale at Dallas provided an ultimate base resistance of 25.5 tsf. These values are considerably higher than the values adopted by many agencies for design of drilled shafts. It is hoped that these data will provide useful guidelines to engineers and contractors. It should be noted that the clay-shale, which could be easily drilled with augers had high base resistance values. Recognition of this fact may be helpful in considering the use of drilled shafts as an economical alternate in many foundation designs.

General Conclusions

An understanding of the behavior of axially loaded drilled shafts in clay-shales can be achieved through proper combination of field and laboratory tests. Design inferences can be drawn only through adequate information about the in-situ shear strength profile and the load transfer characteristics of the supporting soils media. It is possible to estimate in-situ shear strength by means of in-hole tests in the field and triaxial tests in the laboratory. Load transfer characteristics can be determined by performing field tests on properly instrumented drilled shafts. The results of this study have led to the following general conclusions:

(1) The load-carrying capacity of axially loaded drilled shafts can be significantly affected by construction methods.

(2) The dry method of construction yields the highest load transfer in clay-shales.

(3) The shear strength of clay-shales of the same geologic formation can be significantly different at different locations. Therefore, the axial capacity of drilled shafts in clay-shales can be widely different at different locations even though identical construction methods are used.

(4) The peak load transfer in clay shales is mobilized at movements of 1/4 in. or less.

(5) The reaction system used in this study worked very satisfactorily and can be used to cut down the cost of load tests on drilled shafts or piles.

Design Conclusions

The following design conclusions have been reached on the basis of this study.

(1) An α value of 0.5 can be used for drilled shafts in clay-shales if the slurry displacement method or the casing method of construction is employed. This α is correlated to the shear strength of the clay-shale obtained by a quick triaxial test in which the confining pressure is almost equal to the overburden pressure. The α value can be increased to about 0.75 if the shaft is constructed by the dry method. These α values apply to shafts whose penetration into the clay-shale is about 5 ft and whose total length is under 30 ft.

(2) From a design standpoint, the bearing capacity factor, N_c , is about 7.0 for shafts built by the slurry displacement method. Its value may be increased to 8.0 for shafts built by the casing method or dry method. These values of N_c are probably conservative due to limited data obtained in this study.

(3) Unit base resistance, q , may be obtained from dynamic penetration

resistance data using the following equation:

$$q(\text{tsf}) = \frac{N_{THD}}{10} \tag{9.1}$$

where q and N_{THD} are as defined in the preceding chapters. The above correlation must be used with great caution since it is based on very meager data.

(4) The shear strength of the clay-shale may be roughly estimated from the following equation:

$$c_Q(\text{tsf}) = \frac{N_{THD}}{75} \tag{9.2}$$

The above equation has the same limitations as equation (9.1). A similar correlation reported by Hamoudi, Coyle and Bartoskewitz (1974) suggests the following equation for CH soils with secondary structure:

$$c_Q(\text{tsf}) = \frac{N_{THD}}{55} \tag{9.3}$$

On the basis of the conclusions outlined in the preceding paragraphs of this chapter, a procedure to estimate the axial capacity of drilled shafts in clay-shales is suggested next.

Suggested Design Procedure

The following tentative design procedure is suggested to estimate the axial capacity of drilled shafts in clay-shales.

(1) Estimate the base resistance, Q_B (tons) , using the following equation:

$$Q_B = q\,A_b \tag{9.4}$$

where Q_B is the base resistance in tons, q is the ultimate base resistance in tons per sq.ft., and A_b is the base area in sq.ft. The value of q can be determined by using the following equation:

$$q = c_Q N_c \tag{9.5}$$

where c_Q is the shear strength, in tsf, of the clay-shale obtained in a quick triaxial test in which the confining pressure is almost equal to the estimated effective vertical overburden pressure. If only dynamic penetration resistance data are available, a rough approximation of the value of c_Q may be made using equation (9.2).

The following values of N_c are suggested:

N_c = 7.0 if the shaft is constructed by the slurry displacement method

= 8.0 if the shaft is constructed by the casing method or the dry method

(2) Estimate the side resistance, Q_s (tons), using the following equation:

$$Q_s = \alpha c_Q A_s \tag{9.6}$$

where α is the shear strength reduction factor (dimensionless), c_Q is the same as defined above, and A_s is the circumferencial area, in sq.ft., of the shaft in contact with the clay-shale. The suggested values of α are as indicated below:

α = 0.5 for shafts constructed by the slurry displacement or the casing method

= 0.75 for shafts constructed by the dry method

(3) Estimate the total ultimate axial capacity, Q_T (tons) , of the shaft using the following equation:

$$Q_T = Q_B + Q_s \tag{9.7}$$

(4) The total allowable load, Q_A (tons) , may be obtained from the following equation:

$$Q_A \text{ (tons)} = \frac{Q_B}{2} + Q_s \tag{9.8}$$

Equation 9.8 takes into account the relatively small movements required to mobilize full side resistance and 50 percent of the ultimate base resistance. Suitable adjustments may be made to Equation 9.8 to accommodate special criteria of allowable movements. Table 9.1 compares the values of ultimate axial capacity in clay-shale determined by field experiments and the suggested design procedure outlined above.

TABLE 9.1. COMPARATIVE ULTIMATE AXIAL CAPACITIES IN CLAY-SHALES: EXPERIMENTAL AND COMPUTED

Shaft No.	Construction Method	Experimental Axial Capacity (tons)	Computed Axial Capacity (tons)
MT1	Casing	517	396
MT2	Slurry	521	404
MT3	Dry	570	485
DT1	Casing	290	237

It may be seen from Table 9.1 that the Suggested Design Procedure is somewhat conservative.

Limitations of the Suggested Design Procedure

The following limitations exist in the tentative design procedure suggested above.

(1) It is based upon limited information obtained by field tests on only four test shafts whose embedment lengths were under 30 ft and which penetrated about 5 ft into clay-shale. The data obtained from these tests are not conclusive at the present time.

(2) The design procedure can be applied to single axially loaded drilled shafts whose vertical axes are spaced at least 3 shaft-diameters apart.

(3) The recommendations made in this study are based upon experience with clay-shales whose shear strength in a quick triaxial test is less than 10 tsf.

Recommendations for Future Studies

Drilled shafts installed into clay-shales offer an economical and sound foundation alternate for several types of structures. They can support heavy loads at a low cost per ton of load to be supported. With this study, the initial step has been taken to understand their behavior under axial loads. It is recommended that future studies should be conducted as indicated below.

(1) The effect of construction methods on the load transfer characteristics of clay-shales should be studied in detail and appropriate design values should be arrived at based upon additional data obtained from tests on instrumented shafts of different lengths in clay-shales of a wide range of shear strength.

(2) A practical procedure should be established to estimate the in-situ shear strength of clay-shales.

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APPENDIX A

DRILLING LOGS

Texas Highway Department
Form 554

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

Sheet 1 of 1

County Travis
Highway No. SH 71 & US 183
Control
Project No. 3-5-72-176

Structure Test Shafts: Montopolis Site
Hole No. B-1
Station See Boring Location Plan
Loc. from Centerline Rt. Lt.

District No. 14
Date 12-6-1974
Grd. Elev.
Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	Samples†	Log	THD Pen. Test No. of Blows 1st 6"	THD Pen. Test No. of Blows 2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Description of Material and Remarks
	0											
						1			12	38	22	Dark gray sandy clay 0 - 2 ft (24"*)
						2			14	40	18	Dark gray sandy clay 2 - 4 ft (24"*)
	5											
						3			16	38	12	Tan sandy clay 4 - 6 ft (24"*)
						4			17	39	11	Tan sandy clay 6 - 7 ft (12"*)
												Hole stopped at 7 ft depth due to suspected
												obstruction of an outfall sewer pipe.
	10											
												*Recovery

Driller Louis Gourley
Logger Ravi P. Aurora
Title

†Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

29-850 F291 2-69 10M

Texas Highway Department
Form 554

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

Sheet 1 of 1

County Travis
Highway No. SH 71 & US 183
Control
Project No. 3-5-72-176

Structure Test Shafts: Montopolis Site
Hole No. B-2
Station See Boring Location Plan
Loc. from Centerline Rt. Lt.

District No. 14
Date 12-6-1974 and 12-9-1974
Grd. Elev.
Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	† Samples	Log	THD PEN. TEST No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	DESCRIPTION OF MATERIAL AND REMARKS
	0											5-1/2-in. I.D. casing embedded 14 ft after drilling with mud to 14 ft depth.
						1			11	40	20	Dark gray sandy clay 0 - 2.0 ft (18"*)
						2			13	40	15	Dark gray sandy clay 2.0 - 4.0 ft (18"*)
	5					3			15	32	10	Tan sandy clay with gravel 4.0 - 6.0 ft
						4			16	45	20	Tan sandy clay into gravel 6.0 - 8.0 ft
	10											
												8.0 - 14.0 ft with caving problems while drilling.
		X				5			25	60	27	Tan clay with sand & gravel 14.0 - 16.0 ft
	15											
												Dark gray shale 16.0 - 18.0 ft No sample recovered as inner barrel rotated
						6			24	52	26	Dark gray shale 18.0 - 20.0 ft (23" *)
	20											
						7			26	50	25	Dark gray shale 20.0 - 22.0 ft (22" *)
						8			27	50	25	Dark gray shale 22.0 - 24.0 ft (24" *)
	25					9			25	50	26	Dark gray shale 24.0 - 26.0 ft (17-1/2"*)
						10			24	60	26	Dark gray shale 26.0 - 28.0 ft (21-1/2"*)
						11			22	52	22	Dark gray shale 28.0 - 30.0 ft (17-1/2"*)
	30											
						12			20	53	24	Dark gray shale 30.0 - 32.0 ft (24" *)
						13			21	56	26	Dark gray shale 32.0 - 34.0 ft (18" *)
												End of drilled hole at 34 ft depth

*Recovery

Driller Louis Gourley Logger Ravi P. Aurora Title

†Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

29-350 F291 2-69 10M

Texas Highway Department
Form 554

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

Sheet 1 of 1

County Travis
Highway No. SH 71 & US 183
Control
Project No. 3-5-72-176

Structure Test Shafts: Montopolis Site
Hole No. B-3
Station See Boring Location Plan
Loc. from Centerline Rt. Lt.

District No. 14
Date 12-11-1974 and 12-12-1974
Grd. Elev.
Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	† Samples	Log	THD PEN. TEST No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	DESCRIPTION OF MATERIAL AND REMARKS
	0											5-1/2 in. I.D. casing embedded 15 ft after drilling
												with mud to 16 ft depth.
						1			12	35	19	Lt. Br. sandy clay
						2	CL		14	45	35	Tan and gray clay with sand & gravel 2 - 4 ft
	5											
						3	CL		26	31	19	Tan and gray clay 4 - 5.5 ft (10 in.*)
						4						Tan and gray clay 5.5 - 7.5 ft (14.5 in.*)
	10					5	CL		14	27	14	Tan sandy clay 7.5 - 9.5 ft (16.5 in.*)
						6	CL		16	26	17	Tan sandy & gravelly clay 9.5 - 10.5 ft
						7			15	-	-	Tan clayey gravel & sand 10.5 - 11.2 ft
						8			15	-	-	Tan clayey gravel & sand 11.2 - 12.3 ft
						9						Top: tan " " " " 12.3 - 14.0 ft
	15						CH		26	62	36	Bottom: tan sandy clay 14.0 - 14.5 ft
						10	CH		23	65	39	Tan clay slickenzided 14.5 - 16.0 ft
						11	CH		21	61	33	Tan clay slickenzided 16.0 - 17.5 ft
	20					12	CH		24	66	41	Tan clay slickenzided 17.5 - 19.5 ft
						13	CH		22	61	38	Tan clay slickenzided 20.0 - 22.0 ft
						14	CH		23	60	35	Gr. clay with shale 22.0 - 24.0 ft
	25					15						Dark gray shale 24.0 - 24.5 ft
						16	CH		20	53	32	Dark gray shale 24.5 - 26.0 ft
						17						Dark gray shale 26.0 - 28.0 ft
	30					18	CH		20	54	30	Dark gray shale 28.0 - 30.0 ft
						19	CH		21	54	29	Dark gray shale 30.0 - 31.5 ft
						20	CH		21	55	27	Dark gray shale 32.0 - 34.0 ft
												End of drilled hole at 34 ft depth

Driller Louis Gourley Logger Ravi P. Aurora Title

†Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

29-850 F291 2-69 10M

Texas Highway Department
Form 584

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

Sheet 1 of 1

County Travis
Highway No. SH 71 & US 183
Control
Project No. 3-5-72-176

Structure Test Shafts: Montopolis Site
Hole No. SP-1
Station See Boring Location Plan
Loc. from Centerline Rt. Lt.

District No. 14
Date 12-12-1974
Grd. Elev.
Grd. Water Elev.

Elev. (Ft.)	Depth (Ft.)	Samples†	Log	Standard Penetration Test 1st 6"	2nd 6"	3rd 6"	N_{SPT} Value	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	DESCRIPTION OF MATERIAL AND REMARKS
	0											
				3	5	7	12					Dark gray sandy clay 0 - 1.5 ft
	5											
				9	13	15	28					Tan sandy clay 5 - 6.5 ft
				10	8	11	19					Tan sandy and gravelly clay 7.5 - 9 ft
	10											
				11	13	14	27					Tan sand and gravel 10 - 11.5 ft
				6	13	23	36+					Top 9" stiff gray clay 12.5 - 13.25 ft
	15											Bottom 9" dark gray shale 13.25 - 14.0 ft
												End of hole at 14 ft depth
	20											
	25											

Driller Louis Gourley Logger Ravi P. Aurora Title

†Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

29-850 F291 2-69 10M

Texas Highway Department
Form 554

DRILLING REPORT

(For use with Undisturbed Sampling & Testing)

Sheet 1 of 1

County: Travis
Highway No.: SH 71 & US 183
Control:
Project No.: 3-5-72-176

Structure: Test Shafts: Montopolis Site
Hole No.: TP-1
Station: See Boring Location Plan
Loc. from Centerline: Rt. ______ Lt. ______

District No.: 14
Date: 12-9-1974 and 12-11-1974
Grd. Elev.:
Grd. Water Elev.:

Elev. (Ft.)	Depth (Ft.)	† Samples	Log	THD PEN. TEST No. of Blows 1st 6"	2nd 6"	Sample Number	Lat. Pressure & Ult. Stress (psi)	Wet Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	DESCRIPTION OF MATERIAL AND REMARKS
	0		*									*No cores taken. Visual identification only.
												Dark gray sandy clay 0 - 2.5 ft
				16	17	1						
	5											Dark gray sandy clay 2.5 - 5 ft
				26	20	2						
				18	12	3						Dark gray sandy clay 5 - 7.5 ft
	10			26	24	4						Tan sandy and gravelly clay 7.5 - 10 ft
				50(5-1/2")	50(5-1/4")	5						Tan clayey sand & gravel 10 - 12.5 ft
	15											
				50(2-1/2")	50(1-1/2")	6						Dark gray shale
				50(1-1/2")	50 (1/2")	7						Dark gray shale 15 - 17.5 ft
	20			50 (2")	50 (3/4")	8						Dark gray shale 17.5 - 20 ft 12- 9-74
				50 (1/2")	50 (3/4")	9						Dark gray shale 20 - 22.5 ft 12-11-74
	25			50 (1")	50 (1/2")	10						Dark gray shale 22.5 - 25 ft
				50 (2")	50 (1/2")	11						Dark gray shale 25 - 27.5 ft
	30			50(1-3/4")	50 (1/4")	12						Dark gray shale 27.5 - 30 ft
				50(2-1/4")	50 (1/2")	13						Dark gray shale 30 - 33 ft End of hole at 33.2 ft
	35			**Tests done from 2.5 ft - 30 ft at 2.5-ft intervals. Last test at 33 ft depth.								

Driller: Louis Gourley
Logger: Ravi P. Aurora
Title:

†Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

29-850 F291 2-69 10M

APPENDIX B

RESULTS OF IN-HOLE TESTS

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/16/1974 Test Static Dutch Cone No. 1

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 16 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
*	28	56	2.6	*Contact	2.00	890	1780	82.7	
** = 0.00	425	850	39.5	Cone Full in **	2.25	1010	2020	93.8	
0.25	655	1310	60.9		2.50	1065	2130	98.9	
0.50	778	1556	72.3		2.75	1240	2480	115.2	
0.75	672	1344	62.4		3.00	1302	2604	121.0	
1.00	860	1720	79.9		3.25	1318	2636	122.4	
1.25	872	1744	81.0		3.50	1370	2740	127.3	
1.50	942	1884	87.5		3.75	1600	3200	148.6	See Note 3
1.75	993	1986	92.3		4.00	1955	3910	181.6	

Notes:

1. It was not possible to penetrate the shale in a truly vertical direction. The steel rod tended to slide sideways as small chunks of shale came off with increased penetration of the cone.
2. The ram of the jack was probably at the end of travel at a penetration of 4 in. below the top of the shale. Last reading should be ignored.
3. P_{max} = 3200 lb; q_c = ultimate cone bearing pressure = 148.6 tsf.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/16/1974 Test Static Dutch Cone No. 2

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 16 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
*	18	36	1.7	*Contact	2.00	922	1844	85.7	
** = 0.00	448	896	41.6	**Cone Full in	2.25	990	1980	92.0	
0.25	584	1168	54.3		2.50	1150	2300	106.8	
0.50	742	1484	68.9		2.75	1280	2560	118.9	
0.75	682	1364	63.4		3.00	1342	2684	124.7	
1.00	857	1714	79.6		3.25	1420	2840	131.9	
1.25	620	1240	57.6		3.50	1532	3064	142.3	
1.50	790	1580	73.4		3.75	1575	3150	146.3	See Note 3
1.75	906	1812	84.2		4.00	2832 ?	-	-	See Note 2

Notes:
1. Same as Note 1 for Static Dutch Cone Test No. 1.
2. Same as Note 2 for Static Dutch Cone Test No. 2.
3. P_{max} = 3150 lb; q_c = ultimate cone bearing pressure = 146.3 tsf.

*Tip of cone just touching shale.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/16/1974 Test Static Dutch Cone No. 3

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 16 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
*	12	24	1.1	*Contact	2.00	1220	2440	113.3	
** = 0.00	462	924	42.9	**Cone Full in	2.25	1180	2360	109.6	
0.25	590	1180	54.8		2.50	1210	2420	112.4	
0.50	530	1060	49.2		2.75	1320	2640	122.6	
0.75	737	1474	68.5		3.00	1452	2904	134.9	
1.00	660	1320	61.3		3.25	1625	3250	151.0	
1.25	910	1820	84.5		3.50	1645	3290	152.8	See Note 2
1.50	958	1916	89.0		3.75	1550	3100	144.0	
1.75	1106	2212	102.8						

Notes: 1. Same as Note 1 for Static Dutch Cone Test No. 1.

2. P_{max} = 3290 lb; q_c = ultimate cone bearing pressure = 152.8 tsf.

*Tip of cone just touching shale.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Static Dutch Cone No. 4

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 18 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	510	1020	47.4	Cone Full in **	3.25	1210	2420	112.4	
0.50	580	1160	53.9		3.50	1323	2646	122.9	
1.00	800	1600	74.3		3.75	1416	2832	131.6	
1.50	784	1568	72.8		4.00	1510	3020	140.3	
2.00	916	1832	85.1		4.25	1568	3136	145.7	
2.25	966	1932	89.7		4.50	1620	3240	150.5	
2.50	972	1944	90.3		4.75	1710	3420	158.9	See Note 3
2.75	1055	2110	98.0		- End of	Ram Travel -			
3.00	1132	2264	105.2						

Notes:
1. Same as Note 1 for Static Dutch Cone Test No. 1.
2. More than 2 in. eccentricity at end of test.
3. P_{max} = 3420 lb; q_c = ultimate cone bearing pressure = 158.9 tsf.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Static Dutch Cone No. 5

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 18 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	428	856	39.8	**Cone Full in	2.25	1284	2568	119.3	
0.25	628	1256	58.3		2.50	1305	2610	121.3	
0.50	626	1252	58.2		2.75	1372	2744	127.5	
0.75	565	1130	52.5		3.00	1318	2636	122.5	
1.00	784	1568	72.8		3.25	1300	2600	120.8	
1.25	904	1808	84.0		3.50	1400	2800	130.1	See Note 4
1.50	955	1910	88.7						
1.75	1014	2028	94.2						
2.00	1126	2252	104.6						

Notes:
1. Same as Note 1 for Static Dutch Cone Test No. 1.
2. Eccentricity at end of test was more than 2 in.
3. Ram leaked in this test. Results of this test may be questionable.
4. P_{max} = 2800 lb; q_c = ultimate cone bearing pressure = 130.1 tsf.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Static Dutch Cone No. 6

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 18 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	490	980	45.5	**Cone Full in	2.25	1174	2348	109.1	
0.25	616	1232	57.2		2.50	770	1540	71.5	See Note 3
0.50	730	1460	67.8		2.75	830	1660	77.1	
0.75	854	1708	79.3		3.00	1106	2212	102.8	
1.00	768	1536	71.3		3.25	1194	2388	110.9	
1.25	886	1772	82.3		3.50	1275	2550	118.5	
1.50	998	1996	92.7		3.75	1436	2872	133.4	See Note 4
1.75	992	1984	92.2		- End of	Ram Travel -			
2.00	1022	2044	95.0						

Notes:
1. Same as Note 1 for Static Dutch Cone Test No. 1.
2. Eccentricity was in excess of 1 in. at end of test.
3. Soil broke loose at penetration of 2-1/2 in.
4. P_{max} = 2872 lb; q_c = ultimate cone bearing pressure = 133.4 tsf.

IN-HOLE TEST RESULTS

Location Dallas Project 3-5-72-176 Date 8/15/1975 Test Static Dutch Cone No. 7

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 1.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = *** ft. I.D. of Steel Casing = 54 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	480	480	22.3	**Cone Full in					
0.25	950	950	44.1						
0.50	1110	1110	51.6						
0.75	1270	1270	59.0						
1.00	1150	1150	53.4						
1.25	1120	1120	52.0						
1.50	1270	1270	59.0						
1.75	1350	1350	62.7	See Note 3					

Notes:
1. Top of exposed shale 5 ft below top of shale layer. ***
2. Same as Note 1 for Static Dutch Cone Test No. 1.
3. P_{max} = 1350 lb; q_c = ultimate cone bearing pressure = 62.7 tsf.

IN-HOLE TEST RESULTS

Location Dallas Project 3-5-72-176 Date 8/15/1975 Test Static Dutch Cone No. 8

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 1.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = *** ft. I.D. of Steel Casing = ______ in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	570	570	26.5	**Cone Full in	2.25	1260	1260	58.5	See Note 3
0.25	845	845	39.3		2.50	1180	1180		
0.50	550	550	25.5						
0.75	790	790	36.7						
1.00	860	860	39.9						
1.25	1030	1030	47.8						
1.50	1125	1125	52.3						
1.75	1230	1230	57.1						
2.00	1260	1260	58.5						

Notes:
1. Top of exposed shale 5 ft below top of shale layer. ***
2. Same as Note 1 for Static Dutch Cone Test No. 1.
3. P_{max} = 1260 lb; q_c = ultimate cone bearing pressure = 58.5 tsf.

IN-HOLE TEST RESULTS

Location Dallas Project 3-5-72-176 Date 8/15/1975 Test Static Dutch Cone No. 9

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 1.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = *** ft. I.D. of Steel Casing = 54 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	760	760	35.3	**Cone Full in	2.25	1300	1300	60.4	See Note 3
0.25	805	805	37.4						
0.50	710	710	33.0						
0.75	840	840	39.0						
1.00	950	950	44.1						
1.25	1140	1140	53.0						
1.50	980	980	45.5						
1.75	960	960	44.6						
2.00	1170	1170	54.3						

Notes:
1. Top of exposed shale 5 ft below top of shale layer. ***
2. Same as Note 1 for Static Dutch Cone Test No. 1.
3. P_{max} = 1300 lb; q_c = ultimate cone bearing pressure = 60.4 tsf.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/16/1974 Test Plate Load Test No. 1

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 16 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	44	88		**Contact					
0.25	332	664							
0.50	688	1376							
0.75	1025	2050							
1.00	1300	2600							
1.25	1505	3010							
1.50	1715	3430							
1.70	2108	4216							
2.00	2170	4340		See notes below					

Notes:
1. There was excessive horizontal sliding of plate at soil-plate interface.
2. Chunks of shale about 0.50 - 0.75 in. thick and 8 to 10 in. long in radial sectors came off as plate penetrated the shale. Radial cracks in shale always started from the edge of plate and extended outwards towards edge of the test hole. Eccentricities exceeded 2 in. at the end of test.
3. Due to conditions in notes 1 and 2 above it is not possible to determine plunging conditions.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Plate Load Test No. 2

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 16 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	0	0	0	**Contact					
0.25	594	1188	17.4						
0.50	905	1810	26.5						
0.75	1277	2554	37.5						
1.00	1570	3140	46.0						
1.25	1820	3640	53.4						
1.50	2252	4504	66.0						
1.75	2532	5064	74.3	See Note 1					
2.00	2373	4746	69.6						

Notes: 1. All notes for Plate Load Test No. 1 apply.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Plate Load Test No. 3

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 18 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
* = 0.00	18	36	0.5						
0.25	908	1816	26.6						
0.50	2182	4314	64.0	See Note 1					
0.75	590	1180	17.3						

Notes: 1. All notes for Plate Load Test No. 1 apply.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Plate Load Test No. 4

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 18 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	12	24	0.4						
0.25	1623	3246	47.6						
0.50	2110	4220	61.9						
0.75	2276	4552	66.8						
1.00	2460	4920	72.1	See Note 1					

Notes: 1. All notes for Plate Load Test No. 1 apply.

IN-HOLE TEST RESULTS

Location Montopolis Project 3-5-72-176 Date 12/17/1974 Test Plate Load Test No. 5

Strain Indicator Constant for Pressure Transducer: 1 microinch/inch = 2.0 lb. By R. Aurora & J. Anagnos

Depth from Existing Ground Surface to Top of Exposed Shale = 18 ft. I.D. of Steel Casing = 48 in.

Cone Size: 1.4 in. dia (A = 10 sq.cm. = 1.55 sq.in.) Plate Size: 2.5 in. dia (A = 4.91 sq.in.)

Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks	Penetration Below Top of Shale (in.)	Strain Indicator Reading microinch/inch	Load P (lb.)	Bearing Pressure $\frac{P}{A} \times \frac{144}{2000}$ (tsf)	Remarks
** = 0.00	38	76	1.1	**Contact					
0.25	1314	2628	38.5						
0.50	1970	3940	57.8						
0.75	1904	3808	55.8						
1.00	1380	2760	40.5	See Note 1					

Notes: 1. All notes for Plate Load Test No. 1 apply.