THE DETERMINATION OF SOIL PROPERTIES *IN SITU*

by

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Research Report Number 89-7

Soil Properties as Related to Load-Transfer
Characteristics of Drilled Shafts

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.

PREFACE

This report is the seventh in a series of reports from Research Project 3-5-65-89 of the Cooperative Highway Research Program. It describes and discusses three devices used for determining soil properties *in situ*: the Menard Pressuremeter, the Texas Highway Department cone penetrometer, and The University of Texas *in situ* device.

This report is the product of the combined efforts of many people. Specific thanks for technical contributions are due James N. Anagnos, Michael W. O'Neill, John W. Chuang, and Vasant N. Vijayvergiya. The authors wish to thank Lymon C. Reese, the project supervisor, for his support and helpful advice. Preparation and editing of the manuscript were supervised by Art Frakes and staff.

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LIST OF REPORTS

Report No. 89-1, "Field Testing of Drilled Shafts to Develop Design Methods," by Lymon C. Reese and W. Ronald Hudson, describes the overall approach to the design of drilled shafts based on a series of field and laboratory investigations.

Report No. 89-2, "Measurements of Lateral Earth Pressure in Drilled Shafts," by Lymon C. Reese, J. Crozier Brown, and H. H. Dalrymple, describes the development and evaluation of pressure gages to measure lateral-earth pressures on the drilled shaft.

Report No. 89-3, "Studies of Shearing Resistance Between Cement Mortar and Soil," by John W. Chuang and Lymon C. Reese, describes the overall approach to the design of drilled shafts based on field and laboratory investigations.

Report No. 89-4, "The Nuclear Method of Soil-Moisture Determination at Depth," by Clarence J. Ehlers, Lymon C. Reese, and James N. Anagnos, describes the use of nuclear equipment for measuring the variations of moisture content at the drilled shaft test sites.

Report No. 89-5, "Load Distribution for a Drilled Shaft in Clay Shale," by Vasant N. Vijayvergiya, W. Ronald Hudson, and Lymon C. Reese, describes the development of instrumentation capable of measuring axial load distribution along a drilled shaft, the development, with the aid of full-scale load testing, of a technique of analysis of observed data, and the correlation of observed data with the Texas Highway Department cone penetration test.

Report No. 89-6, "Instrumentation for Measurement of Axial Load In Drilled Shafts," by Walter R. Barker and Lymon C. Reese, describes the development and performance of various instrumentation systems used to measure the axial load distribution in field tests of full-scale drilled shafts.

Report No. 89-7, "The Determination of Soil Properties *In Situ*," by David B. Campbell and W. Ronald Hudson, describes the use of the Menard Pressuremeter, the Texas Highway Department cone penetrometer, and The University of Texas *in situ* device in estimating soil properties *in situ* and estimating load transfer values obtained from drilled shaft tests.

ABSTRACT

The determination of the mechanical properties of soils as they exist in nature, free from the disturbances due to sampling and laboratory handling, is a useful and often necessary step toward proper foundation design. This report discusses three devices used in the field for determining these properties by testing the soil "in place." They are referred to as *in situ* devices. The three discussed here are the Menard Pressuremeter, the Texas Highway Department cone penetrometer, and The University of Texas *in situ* device.

The description and evaluation of each device includes equipment, testing procedure, analysis of data, and limitations. Observed data from a full-scale drilled shaft test are correlated in several ways to show the effectiveness of each device in predicting load transfer characteristics of the soil. Correlations with other methods of measuring soil properties are also included.

Recommendations are made as to further use of each of the three *in situ* devices.

KEY WORDS: foundation engineering, drilled shafts, soil testing, clay soils, shear strength, frictional resistance.

SUMMARY

The determination of soil properties is an essential step to proper foundation design. It is useful and often necessary to determine the numerical values of these properties as they exist in nature through the use of *in situ* soil testing.

In this study three methods of *in situ* soil testing are examined. They are

(1) the Menard Pressuremeter,

(2) the Texas Highway Department (THD) cone penetration test, and

(3) The University of Texas (UT) *in situ* device.

The Menard Pressuremeter was not available for field testing, but was studied by review of prior research. Results obtained from the THD penetrometer are well correlated with maximum load transfer values obtained from tests of instrumented drilled shafts. Shearing resistances obtained from field testing of the UT *in situ* device do not correlate well with conventional laboratory test results, with penetration resistance measured by the THD cone penetrometer or with load transfer data.

IMPLEMENTATION STATEMENT

This study of three in situ devices was undertaken with the hope of finding a method of measuring shear strengths in the field. These are particularly needed for soils which do not readily lend themselves to undisturbed sampling methods. Such a measurement tool would have been very useful in the design of the drilled shaft foundations.

However, as noted in the recommendations, the Menard Pressuremeter has not been available to The University of Texas personnel for correlation studies and The University of Texas in situ device in its present form does not yield reliable results. Therefore, all future effort to measure in situ shear strengths will be through correlations with the THD cone penetrometer which is currently being used by the Texas Highway Department.

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NOMENCLATURE

Symbol	Typical Units	Definition
A	ft^2	Total area of both shear plates
E	kg/cm^2	Young's Modulus of the soil
F	tons	Maximum shearing force
K_B	--	Coefficient dependent on the stiffness E/P_L of the soil and the soil structure
P_f	kg/cm^2	Creep pressure
P_L	kg/cm^2	Limit pressure
P_o	kg/cm^2	Lateral pressure at rest at the depth of the pressuremeter test
q_u	kg/cm^2	Unconfined compression strength
N	blows/ft	Penetration resistance, Standard Penetration Test
R	$tons/ft^2$	Shearing resistance measured by UT *in situ* device
s	in.	Movement of drilled shaft at any depth
S_o	kg/cm^2	Shear strength measured by Menard Pressuremeter

s_o	in.	Maximum settlement of drilled shaft
T	$tons/ft^2$	Load transfer at any depth

CHAPTER 1

INTRODUCTION

The application of the principles of soil mechanics in order to achieve proper foundation design has become increasingly important in recent years. In addition to the attention given to theory, more and more emphasis is being placed on improving sampling and testing techniques. As a result, numerous devices have been developed for taking undisturbed soil samples.

Such samples are desirable because soils as found in the field are rarely homogeneous; their properties can vary in both the horizontal and vertical directions. These properties may be radically altered by boring, sampling, testing, and other handling in the laboratory. As a result of such alterations, the mechanical properties as determined in the laboratory may often differ appreciably from the natural properties of the soil.

Furthermore, certain soils as found in nature cannot be sampled satisfactorily or prepared properly for laboratory testing. These include soils with a considerable secondary structure, such as fissures, joints, slickensides, and concretions, and those soils which contain rocks and shells of appreciable size. It is therefore often desirable to test the soil as it is found in nature without sampling, and *in situ* soil testing apparatus becomes important in determining the soil properties.

There are several devices in present use for determining properties of soil *in situ*. These include a number of devices designed for use in boreholes of various sizes, vane shear apparatus, and a variety of penetrometers. One such instrument has been developed by Iowa State University for determining the load-bearing capacity, cohesion, and the angle of internal friction of a test soil. This device consists of two diametrically opposed, grooved expansion plates which are expanded by automotive brake cylinders. At the field site the device is lowered into a test hole. The plates are expanded against the walls of the hole and then pulled upward, thus shearing the soil in contact with the plates. The normal force and lifting force are measured separately. The result of the test is a plot of the relationship between shear strength and normal force. This device has been found satisfactory for testing homogeneous soft and medium stiff cohesive material but not for testing mixtures of gravel and clay.

The vane shear device is widely used for testing soft and medium stiff cohesive soil. It consists of a four-bladed vane fastened to the bottom of a vertical rod. This assembly can be pushed into the soil without any appreciable disturbance of the material. The shear strength is computed from the applied torque required to turn the assembly.

A direct shear test apparatus has been developed by the Comision Federal de Electricidad in Mexico for the purpose of investigating the shear strength of a soil _in situ_ (Ref. 13). The device consists of a flat circular steel disc onto which blades have been welded. The total shearing area is approximately 0.5 m^2. After the blades have been forced into the soil by a normal force, a torsion couple is transmitted to the disc by means of two steel cables operated by horizontal hydraulic cylinders.

This device tests shear strength of the soil in the horizontal direction only, whereas shear strength in the vertical direction is usually desired.

A variety of penetrometers is used for in situ determination of the consistency of cohesive materials or the relative density of cohesionless materials. Further discussion of penetrometers is continued in later chapters.

The purpose of this report is to discuss and evaluate three available in situ devices: the Menard Pressuremeter, the Texas Highway Department cone penetrometer, and The University of Texas in situ device. For each device the study describes the equipment and testing procedure involved, the analysis of data obtained, and the limitations of its use. Correlations between the devices, with conventional laboratory test results, and with load transfer data measured from field tests of drilled shafts are developed and analyzed. Conclusions are drawn from these correlations, and recommendations are made concerning the desirability of using each device.

CHAPTER 2

MENARD PRESSUREMETER

Operational Concept

One of the instruments recently developed for *in situ* measurement of soil properties is the Menard Pressuremeter. The pressuremeter system, developed by Terrametrics (Ref. 16), utilizes the concept of stressing the walls of a borehole and determining the accompanying deformation of the surrounding material. An expandable probe is lowered into the borehole and used to apply a selected radial pressure to the surrounding soil. At each test, the pressure and accompanying volumetric change of the probe are measured by a pressure gauge and volumeter at the ground surface.

Analysis and interpretation of the pressure-volume data from the test provide values for several of the primary mechanical properties of the material. Of most importance among these are Young's Modulus, the bearing capacity, the creep pressure, and the shear strength of the material.

Equipment

The Menard Pressuremeter (Fig. 2.1) consists of two main components: the probe, which goes down the borehole; and the pressure-volumeter, which is a surface instrument. These two components are connected by plastic tubes through which water and gas pressure are applied.

The probe is a cylindrical metal assembly with rubber membranes attached in such a manner as to form three independent cells. Carbon

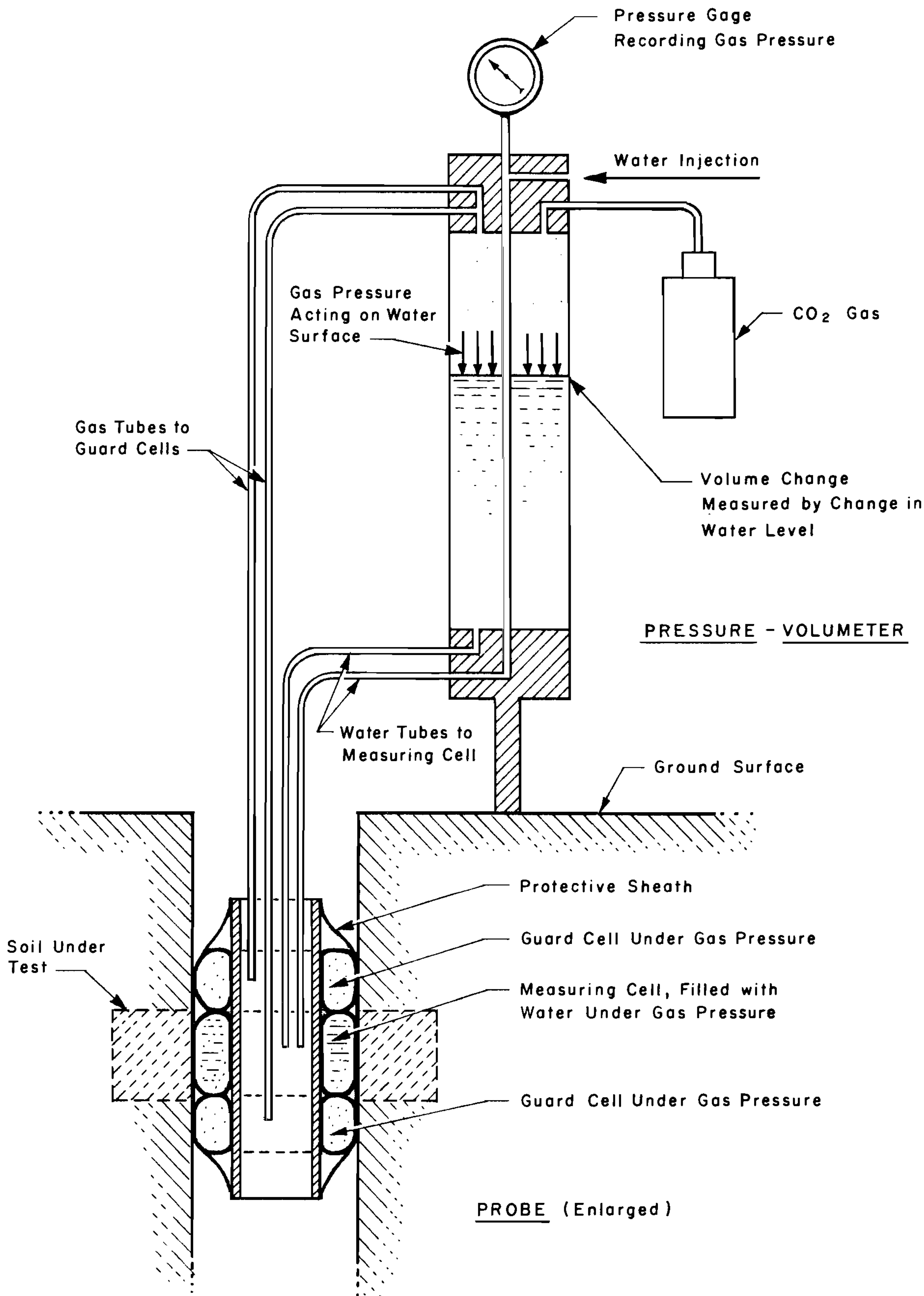

Fig. 2.1. Menard Pressuremeter

dioxide gas pressure is applied to the three cells from the surface and inflates them to approximately twice their original diameter (Ref. 8). Water is present in the central measuring cell and the volume of this cell is measured by the lowering of the water level in the volumeter at the surface. The upper and lower cells are known as guard cells, and they expand under a gas pressure equivalent to that which is applied to the central measuring cell. The purpose of these cells is to minimize the effects of end restraint on the measuring cell. Probes are currently available for holes with diameters of 1-1/2, 2, 2-3/8, and 3 inches.

The volumeter is equipped so that a monitored gas pressure can be used to force water into the measuring cell. In addition, a measured gas pressure is applied to the guard cells.

The plastic tube connecting the volumeter to the measuring cell is enclosed inside a plastic tube going to the guard cells. The purpose of this arrangement is to minimize any expansion of the tube going to the measuring cell. Any expansion of this tube would result in inaccurate estimates of the amount of water being forced into the measuring cell.

Collection of Data

If laboratory soil testing is to accompany the use of the Menard Pressuremeter, samples should be obtained from the borehole used for the pressuremeter. It is desirable to make the pressuremeter tests as soon as possible after the removal of the samples. Readings should be taken at the approximate center of the increment of the hole from which the sample was taken.

At each desired depth, data are obtained by applying increments of pressure to the water in the measuring cell and recording the corresponding volume changes of the probe. Approximately eight different pressures are used at each depth. Changes in volume are recorded at 15, 30, and 60 seconds after the application of each pressure. Pressuremeter tests are taken at each depth of interest in the soil strata.

Analysis of Menard Pressuremeter Data

The data obtained from the pressuremeter tests are plotted on a graph of pressure versus volume and ideally result in curves similar to those shown in Fig. 2.2 (Ref. 10). The P-V curve is a plot of the cell pressure versus the 60-second readings on the volumeter. The initial sloping portion of the curve results from restoration of the earth pressure to the undisturbed condition prior to the removal of the soil from the hole. Following this initial portion, the curve becomes nearly linear. It is from this nearly linear phase that an approximation of Young's Modulus of the material can be obtained. The end of this pseudo-elastic portion of the curve marks the beginning of the yield condition at the wall of the borehole. With further increase in pressure the volume change increases rapidly, and the curve asymptotically approaches a vertical line as the ultimate failure pressure of the soil is approached. The pressure reached is referred to as the limit pressure, P_L . It gives an indication of the bearing capacity of the soil.

There are, however, several factors which must be considered in determining a final limit pressure. First, the resistance of the rubber membrane of the probe must be taken into account. This is accomplished by developing a pressure-volume curve for the probe alone, without insertion in the ground, from which the pressure required for the expansion of the probe

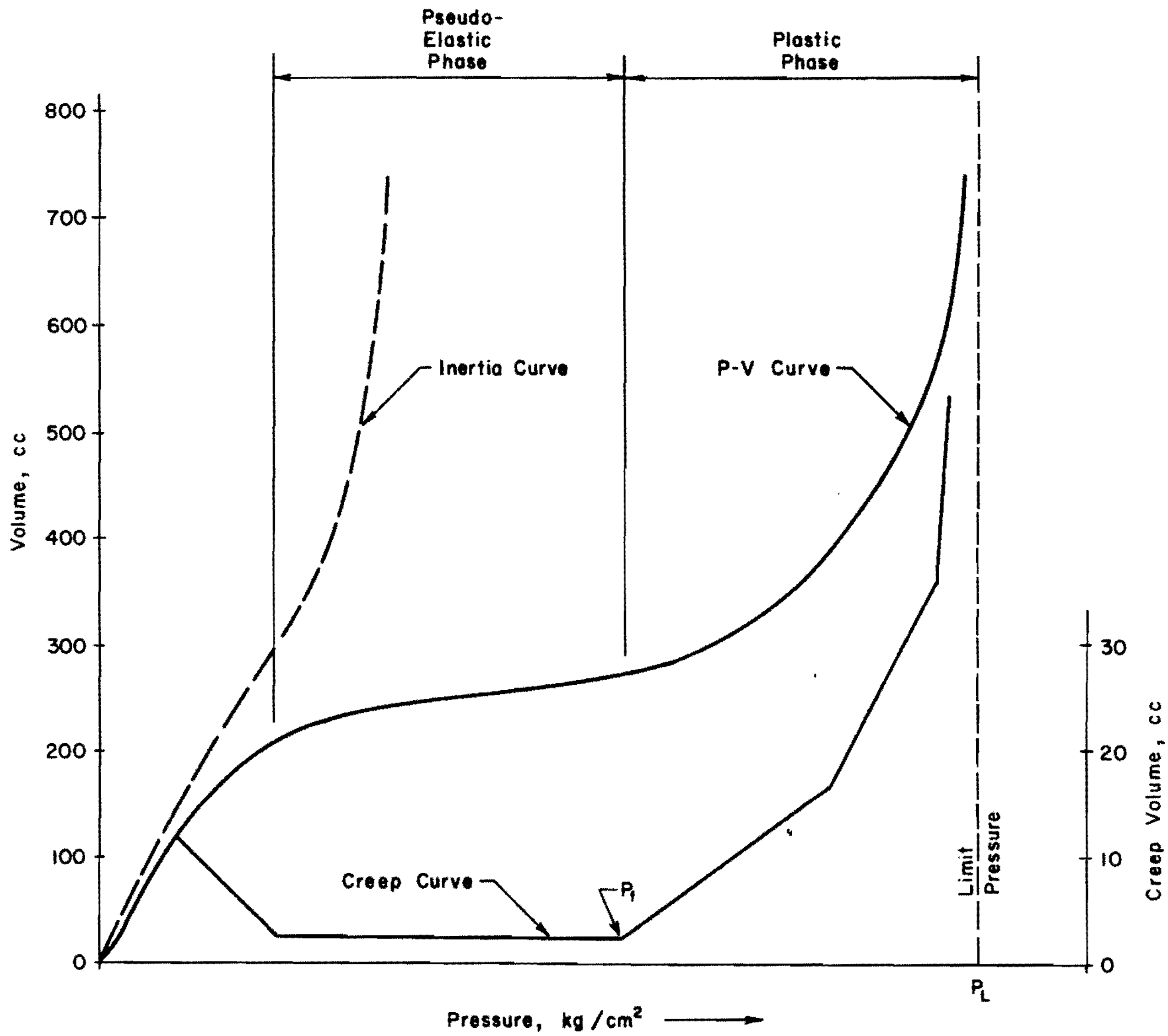

Fig. 2.2. Curves Derived from Menard Pressuremeter Data

may be obtained for any volume and applied as a correction to the pressure observed during the field test. This curve is known as the inertia curve and is shown in Fig. 2.2 as a dashed line. Second, an effect similar to the hydrostatic effect of the column of water in the measuring cell, which becomes important at depths beyond 30 feet, must be introduced into the guard cells (Ref. 8). This is done by using a second pressure-volumter in the surface assembly. A third correction to be considered is that for the expansion of the plastic water tube; however, this may be neglected if the tube is constructed in a particular way as described in the section on equipment.

During each applied pressure increment, the soil being tested will usually deform with time. This aspect of the nature of the soil is illustrated by the creep curve in Fig. 2.2, a plot of the volume change between the 30-second and 60-second readings. The pressure at which this curve makes a definite upward break is designated the creep pressure, P_f , and is usually close to the end of the pseudo-elastic phase of the pressure-volume curve.

The shear strength of the soil in question may be calculated from the data obtained from the pressuremeter test. The formula for shear strength as postulated by Menard is

$$S_o = \frac{P_L - P_o}{2K_B} \tag{2.1}$$

where

P_L = the limit pressure as obtained from the test,

P_o = the lateral pressure at rest at the depth of the test,

K_B = a coefficient dependent on the stiffness E/P_L of the soil and the soil structure.

Pressuremeter Correlation

Use of the Menard Pressuremeter at the Houston drilled shaft test site and correlation of the data with laboratory test results for that location are considered to be desirable, but a pressuremeter has not been available for this study.

A pressuremeter correlation study on cohesive soils has been made by C. M. Higgins, Soils Research Engineer for the Louisiana Department of Highways (Ref. 10). The study involved comparing shear strength results obtained by the Menard Pressuremeter with those obtained by conventional test methods. The conventional tests used for comparative purposes were the unconfined compression test, the vane shear test, and the consolidated undrained triaxial test.

Pressuremeter results from a test site located at Plaquemine, Louisiana, are shown with unconfined compression test results in Fig. 2.3 and with vane shear test results in Fig. 2.4. The correlation between the pressuremeter results and the results of the two conventional methods of testing is fairly good. In general the pressuremeter results fall between the unconfined test results and the vane shear test results. The soil at this location ranged from a heavy clay to a silty clay loam (Ref. 10).

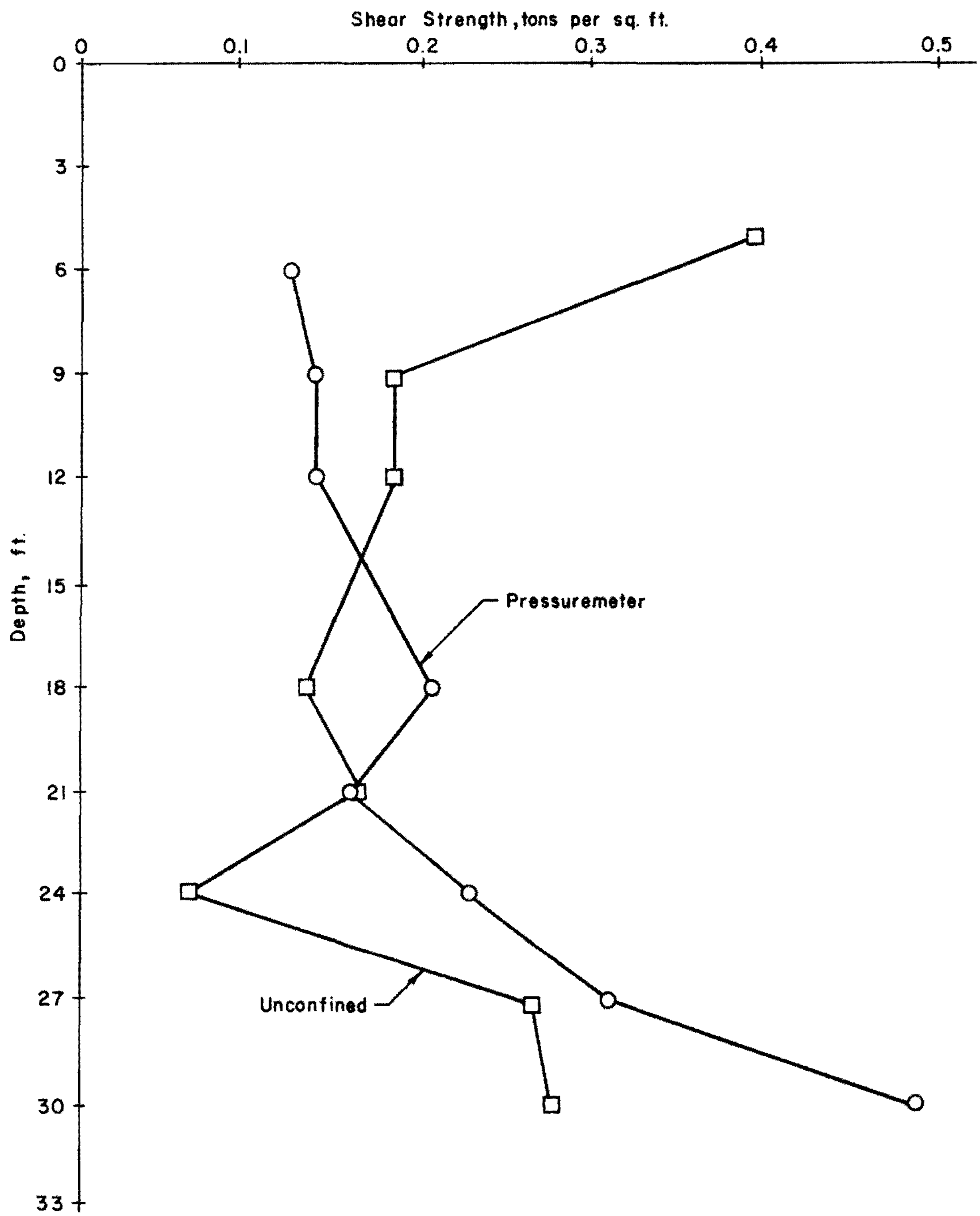

Fig. 2.3. Comparison of Shear Strengths Obtained by Menard Pressuremeter and Unconfined Compression Tests, Plaquemine Test Site (After Higgins)

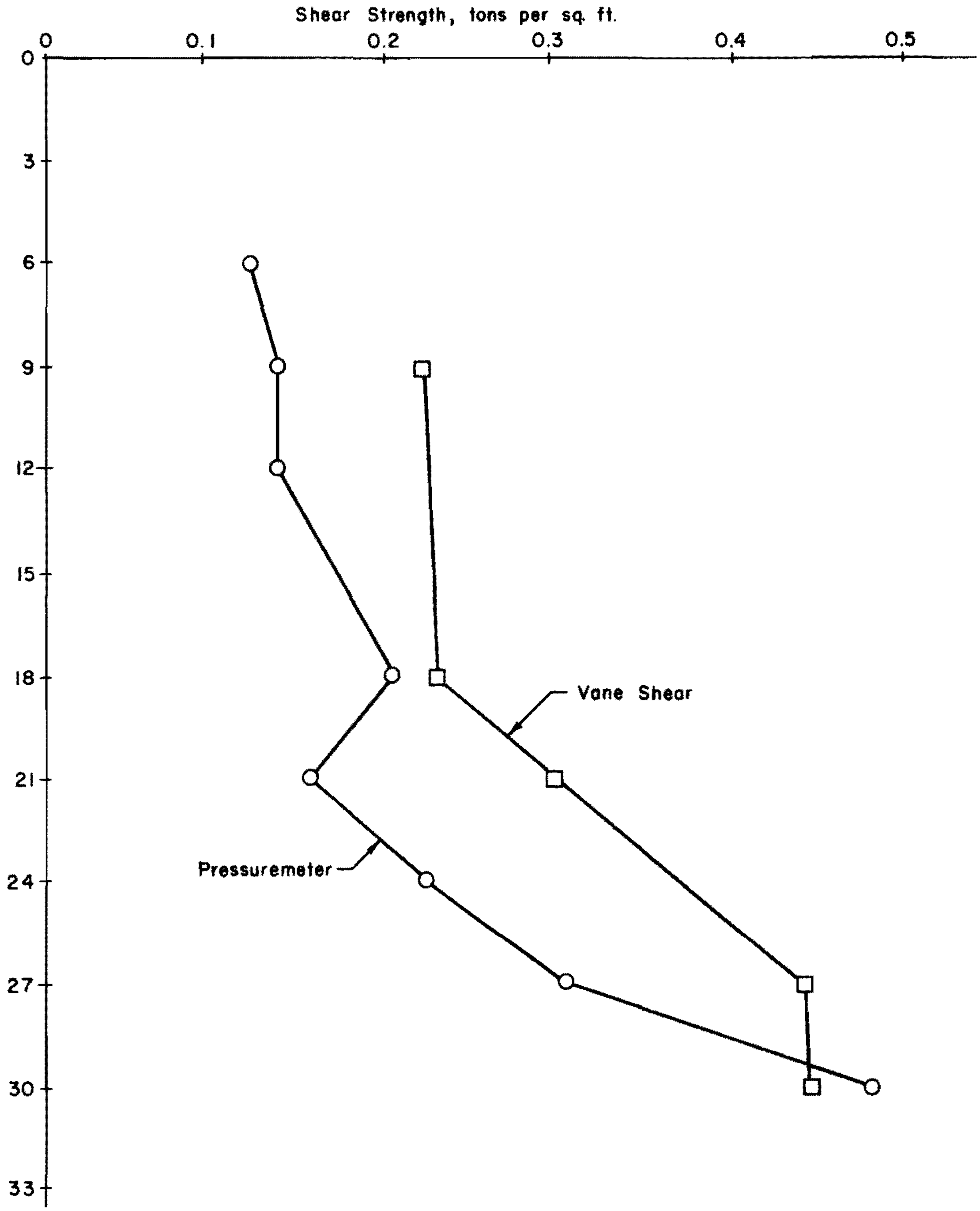

Fig. 2.4. Comparison of Shear Strengths Obtained by Menard Pressuremeter and Vane Shear Tests, Plaquemine Test Site (After Higgins)

Pressuremeter results for Lake Charles, Louisiana, were compared with shear strengths obtained from unconfined compressions tests (Fig. 2.5.) and consolidated undrained triaxial tests (Fig. 2.6). The agreement between these pressuremeter test results and the conventional test results at this location is not as close as between those at the Plaquemine site. The pressuremeter-derived strengths are somewhat greater than both the unconfined test values and the triaxial values. The soil was a stiff clay with numerous calcareous inclusions (Ref. 10).

This variation in the results has been attributed mainly to the nature of the soil encountered at the Lake Charles site. When soils are tested by either unconfined compression or triaxial testing, the sample normally fails at the weakest point within the core which was at the calcareous inclusions in the Lake Charles soil. The pressuremeter, however, tests an area of soil about 18 inches in height, and the strengths obtained are therefore more of an average of the material strengths within this zone. It is believed that this average was not influenced appreciably by the inclusions, since they are small when compared to the size of the area being tested. Soil disturbance caused by sampling, handling, and testing in the laboratory also caused the laboratory strengths to be lower than those measured by the pressuremeter. It is therefore probable that the strengths measured by the pressuremeter are more representative than the lab tests of the strength of the material at this location.

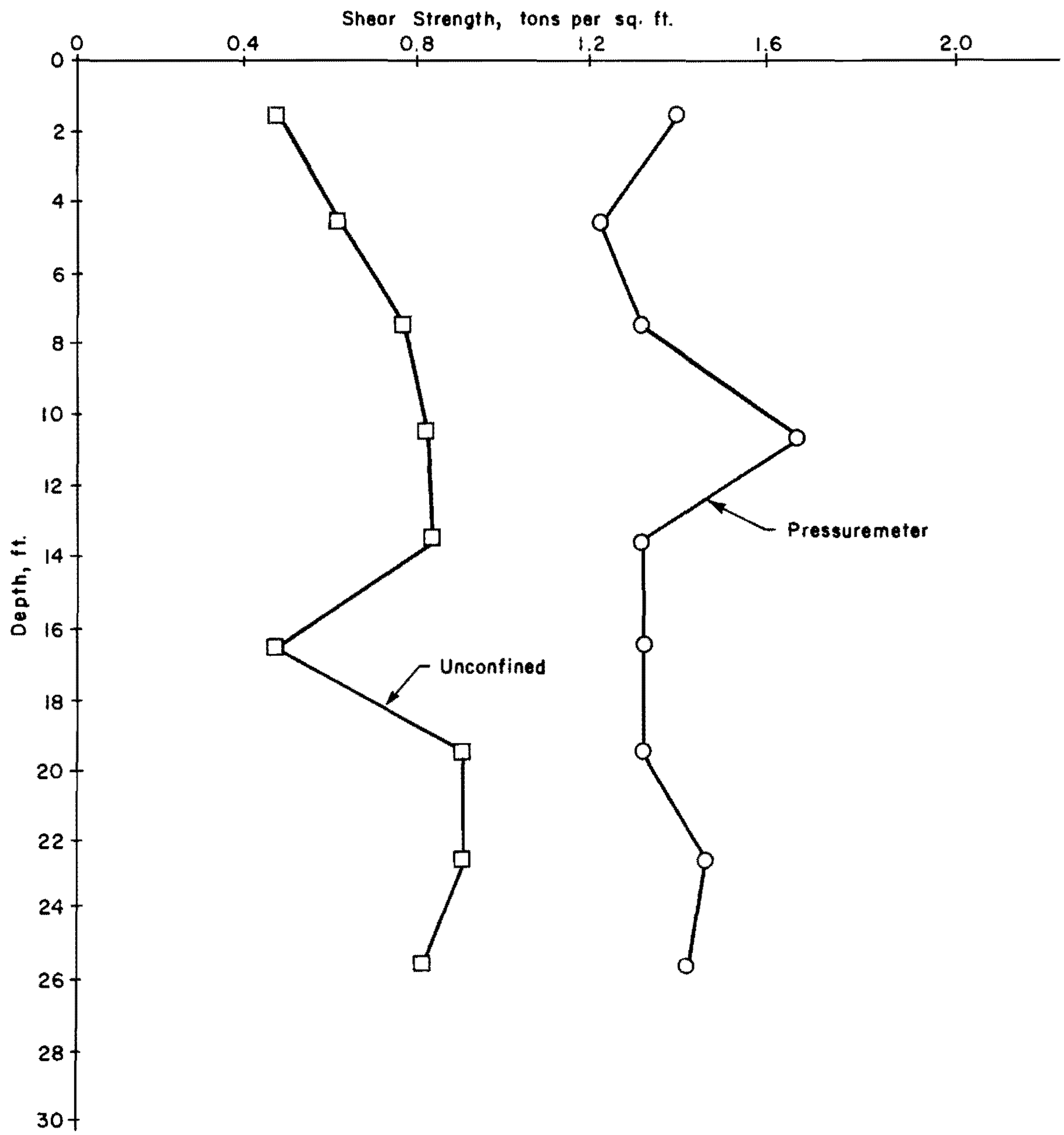

Fig. 2.5. Comparison of Shear Strengths Obtained by Menard Pressuremeter and Unconfined Compression Tests, Lake Charles Test Site (After Higgins)

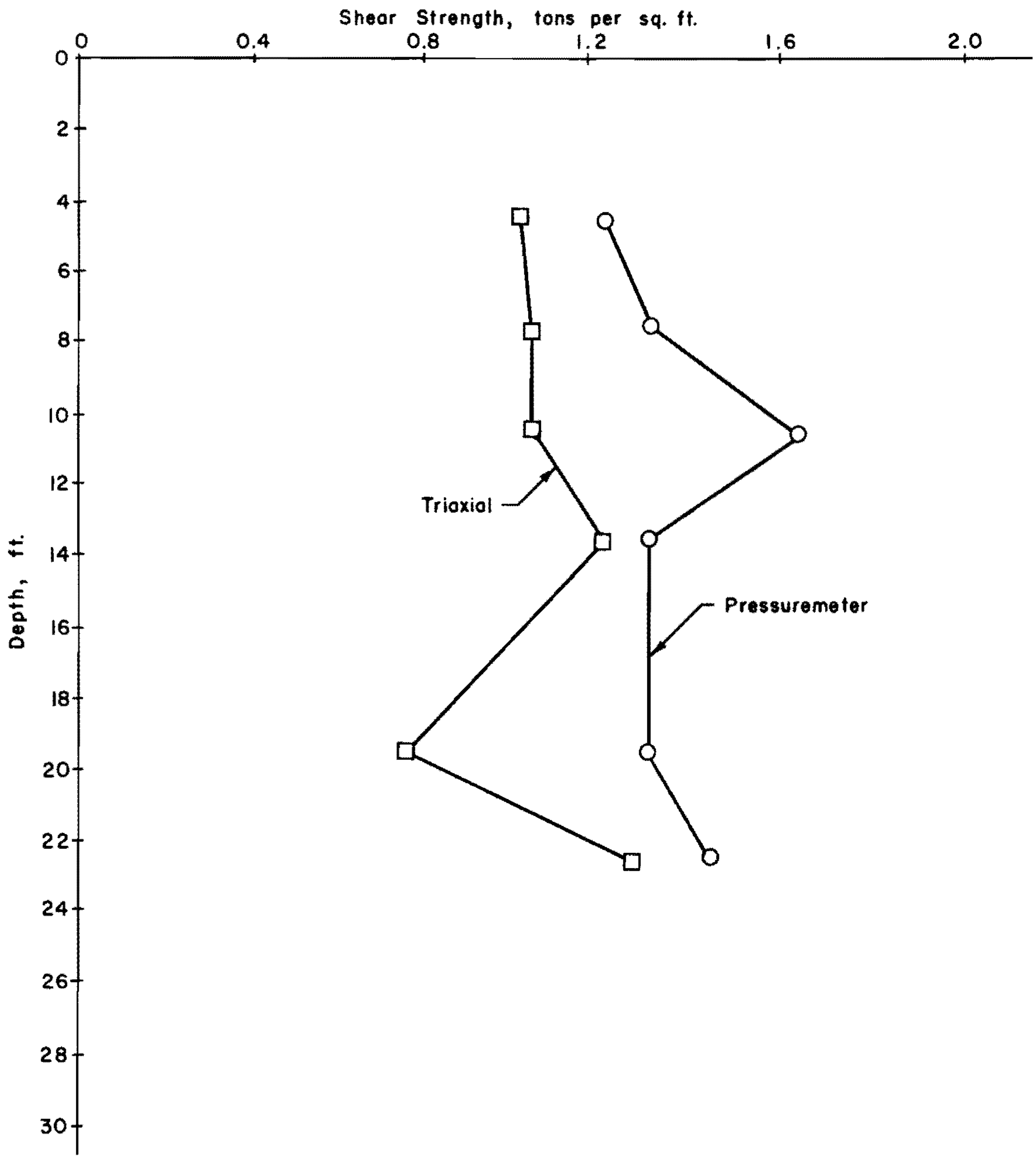

Fig. 2.6. Comparison of Shear Strengths Obtained by Menard Pressuremeter and Triaxial Tests, Lake Charles Test Site (After Higgins)

Higgins (Ref. 10) draws the following conclusions from the results of his study:

(1) For very soft cohesive soils, the agreement between shear strengths measured by the pressuremeter and shear strengths measured by conventional methods was good.

(2) For cohesive soils of medium strength, not greatly influenced by cracking of the soil near the ground surface, the pressuremeter shear strengths and those obtained by conventional methods are very close at shallow depths. The pressuremeter strengths do become considerably greater than the conventional shear strengths as the depth increases.

(3) For medium strength cohesive soils with numerous inclusions or cracks, the pressuremeter strengths are greater at all depths than those strengths derived by conventional methods. It is probable, however, that the pressuremeter strengths are the more representative of the true strength of the soil.

Further correlations involving Menard Pressuremeter data have not been found in the literature. Mori and Tajima (Ref. 18) present a discussion of the pressurementer in which they cite some of Menard's work (Ref. 15) on estimating the allowable bearing capacity and settlement of deep foundations. De Mello (Ref. 6) cites two authors, Goulet and Karst (Refs. 9 and 12), who also report using the pressuremeter for foundation applications. Calhoon (Ref. 4) discusses the pressuremeter and its ability to estimate the shear strength and stress-strain characteristics of soils, but presents no correlation between results obtained with the Menard system and soil properties measured by conventional laboratory tests.

Limitations

Several difficulties have been encountered in the use of the Menard Pressuremeter (Ref. 10). Some of these difficulties are:

(1) The quantity of water available in the volumeter may not be sufficient to reach the actual limit pressure. When this is true, the limit pressure can only be estimated.

(2) In very soft soils the inertia of the probe may approach the strength of the soil, making it necessary to use the probe without the outer rubber sheath and to use a softer rubber membrane for the measuring cell.

(3) The probe is extremely sensitive to hole size. If the hole is too large, there is not enough water in the volumeter to allow a sufficient pressure to be applied to the probe to estimate the limit pressure. If the hole is too small, the probe must be forced into the hole, thus disturbing the surrounding soil and causing misleading readings.

(4) If the probe is to be used in a material containing angular or sharp aggregates, it must be covered with a heavy plastic tape to prevent the rubber sheath from bursting.

(5) Use of the pressuremeter at any depth should immediately follow the extraction of the soil from that depth. If further coring is continued the hole diameter at the original desired level of testing may be enlarged sufficiently to invalidate the information obtained.

(6) Pressuremeter tests in sand are difficult due to caving in of the hole. Erroneous results are obtained unless some means is used to prevent the caving in.

(7) The operations necessary for effective use of the pressuremeter are time consuming.

CHAPTER 3

PENETRATION TESTS

The simplest and most widely used method of determining _in situ_ soil properties is the penetration test. The testing procedure consists of measuring the resistance offered by the soil to the advancement into the ground of a penetrometer. A penetrometer is often a rod with an enlarged end which is either driven or pushed into the ground, but those discussed here are specifically designed for their intended uses. The measured resistance, which usually gives some indication of the strength of clays and of the relative density of cohesionless deposits, is developed as a result of both point resistance and side friction, with the amount of each depending on the soil properties.

Many varieties of penetration tests have been designed, each especially adapted to certain kinds of material. They can, however, be separated into two general groups, static and dynamic.

Static Penetration Tests

If the penetrometer is forced into the soil at a steady rate under a fixed static load, the procedure is referred to as a static penetration test. The pressure required to advance the penetrometer at this constant rate is measured and is known as the penetration resistance.

A static penetration test very popular in Europe is the Dutch cone penetration test (Refs. 1, 5, 6, 17, 20, 24, 28, and 34). The Dutch penetrometer, as shown in Fig. 3.1, is a 60° cone with a diameter of 1.4 inches. The cone, attached to the end of a rod which is inside a casing to prevent development of side resistance, is either driven into the ground to the desired depth of testing or positioned in an open hole at this depth. The cone is forced at a constant velocity, usually 0.4 inches per second, into the soil being tested, and the pressure required is the penetration resistance.

Use of the cone penetration test in soft clays, silts, and peats is quite successful (Ref. 20), and it has been used to investigate the relative density of sands. The measurements from the static penetration test provide considerable detail for a bearing capacity profile of the soils encountered at the test site. Each test represents a form of static bearing capacity test which, with suitable theory, can be used to extrapolate bearing capacity values for larger foundations (Ref. 23). From test results in clays, the undrained shear strength can be computed with bearing capacity theory. In sands, the relative density and/or the angle of internal friction can be estimated from the magnitude of the cone bearing value (Ref. 23).

An extensive literature search has yielded very little concerning correlation studies performed on the static cone test. De Mello (Ref. 6) briefly cites several authors (Refs. 2, 3, 21, 30, 31, and 33) who have made some correlations between penetration resistance and shear strengths measured by vane shear, unconfined compression, and undrained triaxial testing in clays. Meyerhof (Ref. 17) correlates static cone resistance with penetration resistance measured by the Standard Penetration Test, which is discussed later, and with the relative density of sands, as shown in Table 3.1.

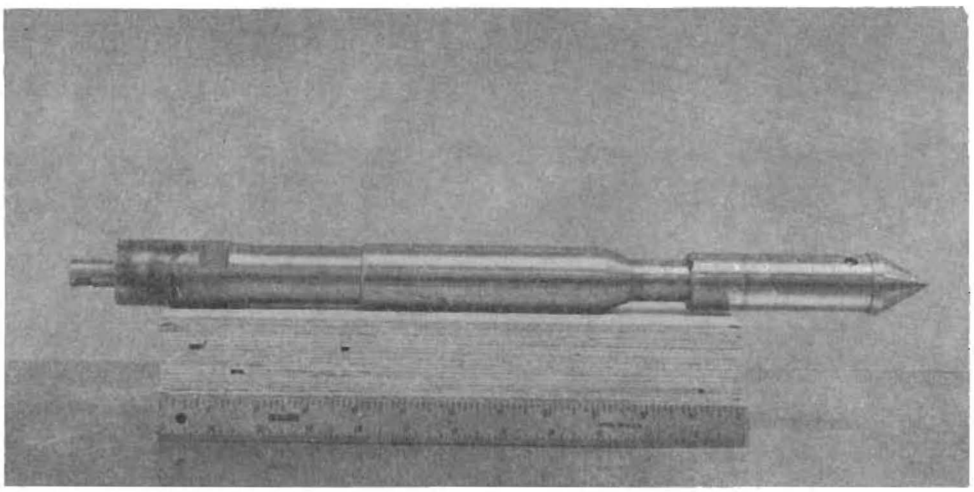

Fig. 3.1. Dutch Cone Penetrometer

TABLE 3.1. CORRELATION BETWEEN STANDARD AND CONE PENETRATION TESTS (Ref. 17)

Relative Density	Standard Penetration Resistance, blows/ft	Static Cone Resistance, tsf
Very loose	Less than 4	Less than 20
Loose	4 - 10	20 - 40
Medium	10 - 30	40 - 120
Dense	30 - 50	120 - 200
Very dense	Over 50	Over 200

Dynamic Penetration Tests

A penetration test is referred to as dynamic when the penetrometer is driven into the soil, rather than being forced at a steady rate. The penetration resistance in this test is the number of blows, N , required to produce a penetration of one foot.

The test of this type most widely used in this country is known as the Standard Penetration Test (Refs. 17, 20, 25, 27, 28, and 34). It utilizes a split-spoon sampler as the penetrometer (Fig. 3.2). It is attached to drill rods and driven into the soil by a 140-pound hammer which falls through a distance of 30 inches.

The Standard Penetration Test has the advantage of also obtaining representative disturbed soil specimens. Correlation of N , the number of blows per foot, and the relative density of sands is considered quite reliable. However, the correlation of N values with the consistency of

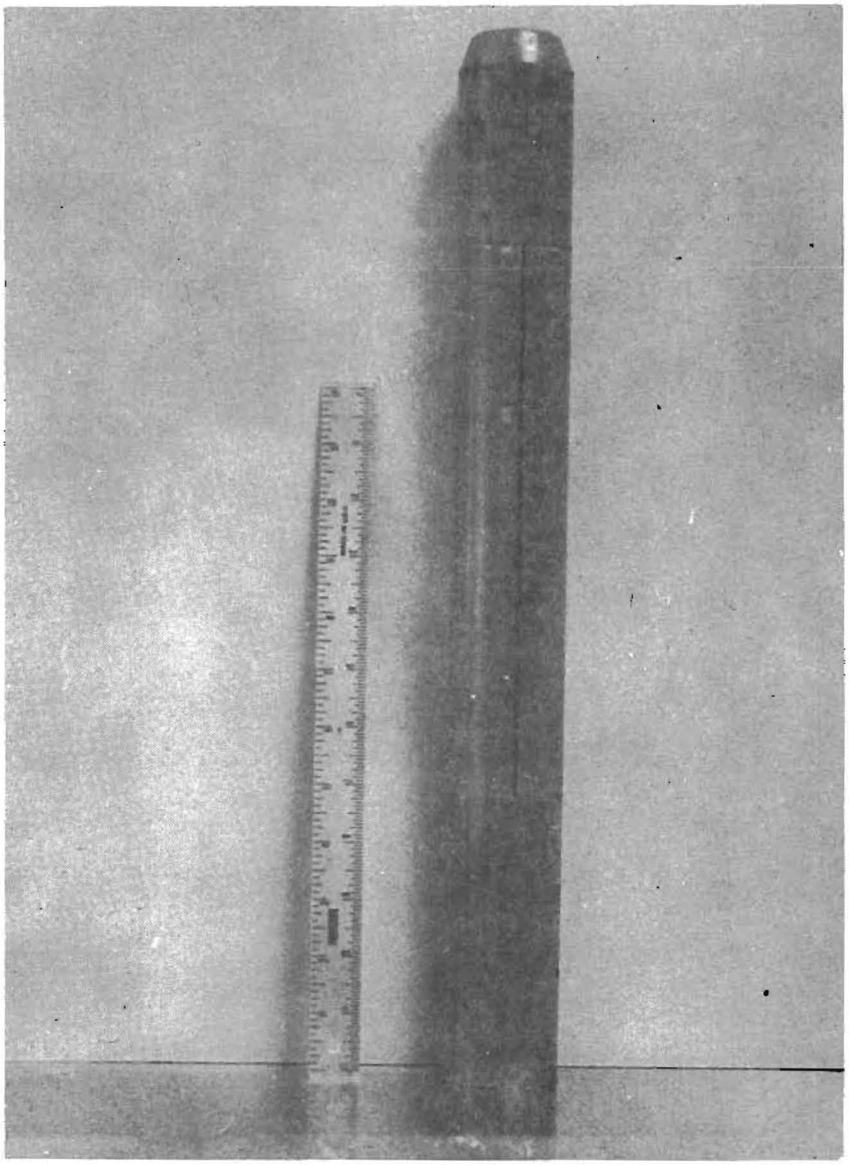

Fig. 3.2. Split-spoon Sampler Used in Standard Penetration Test

clays is regarded as only a crude approximation. These correlations are given in Table 3.2.

TABLE 3.2. PENETRATION RESISTANCE AND SOIL PROPERTIES BASED ON THE STANDARD PENETRATION TEST (Ref. 20)

Sands		Clays	
N , blows/ft.	Relative Density	N , blows/ft.	Consistency
0 - 4	Very loose	Below 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium	4 - 8	Medium
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

Several authors have made correlations between N values and unconfined compression test results, q_u , in clays, as shown in Fig. 3.3. Trow (Ref. 31) correlates data from tests of eight different clays in a study restricted to lean clays to avoid problems with sensitive clays. Sowers (Ref. 26) correlates N and q_u values for highly plastic clays, clays of medium plasticity, and clays of very low plasticity. Peck and Reed (Ref. 19) plot hundreds of data on Chicago clays and suggest a conservative boundary of $q_u = N/6$, for use in estimating the allowable bearing pressure of footings, although their data approximates $q_u = N/4$. De Mello, et al, (Ref. 7) obtain a statistical

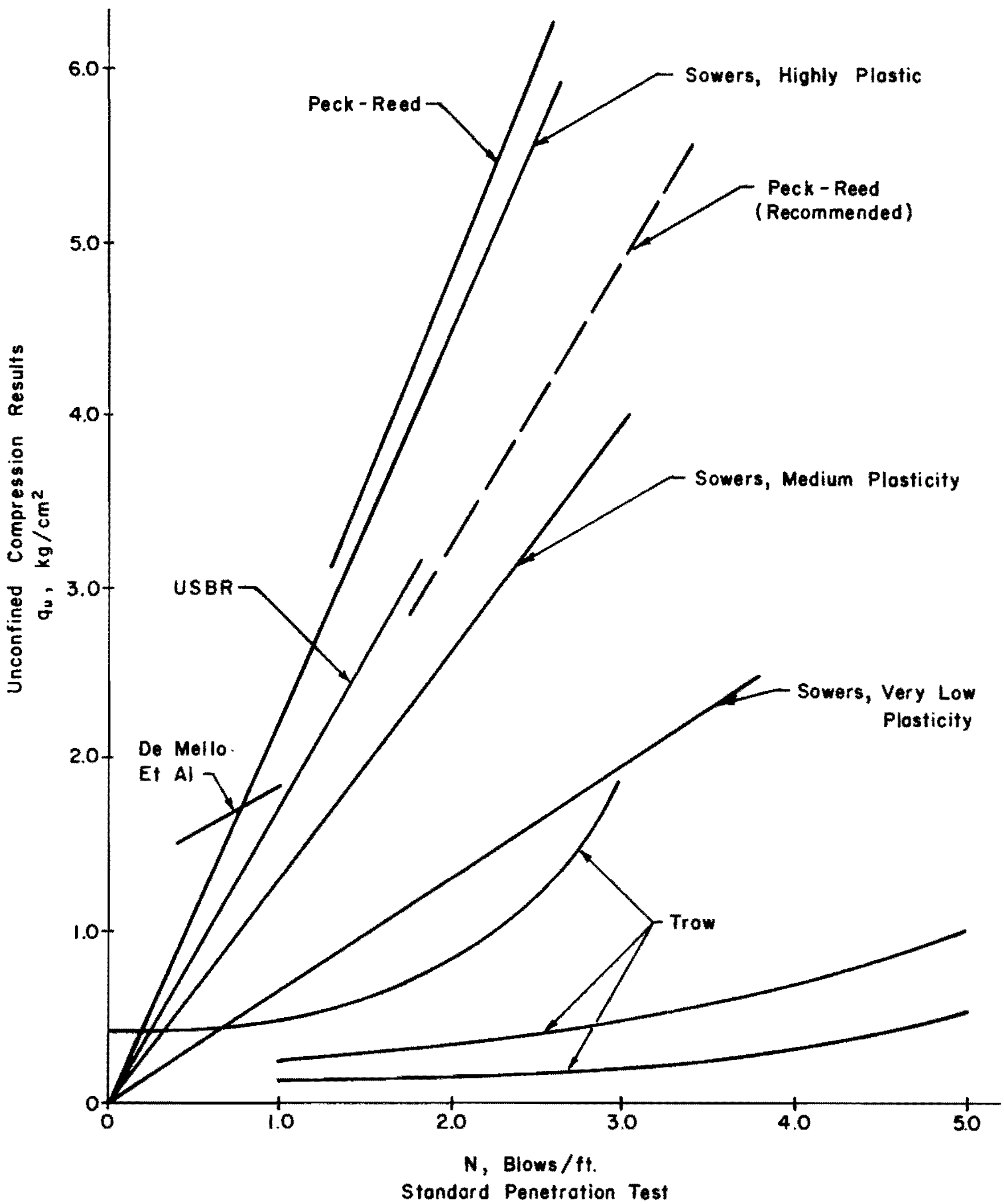

Fig. 3.3. Attempted Correlations Between N and q_u (kg/cm^2) for Clays (After De Mello)

relationship $q_u = 0.061\ N + 1.3\ (kg/cm^2)$ for an unsaturated silty clay. Another curve shown in Fig. 3.3 represents the results of a correlation study of N and q_u values made by the United States Bureau of Reclamation (Ref. 6). Terzaghi and Peck (Ref. 28) suggest the approximate relationship between N values and unconfined compressive strengths shown in Table 3.3.

TABLE 3.3. RELATION OF NUMBER OF BLOWS N ON SAMPLING SPOON AND UNCONFINED COMPRESSIVE STRENGTH (Ref. 28)

N (blows/ft.)	Below 2	2-4	4-8	8-15	15-30	Over 30
q_u (tsf)	Below 0.25	0.25-0.50	0.50-1.00	1.00-2.00	2.00-4.00	Over 4.00

As seen from the data in Fig. 3.3 and Table 3.3, the scatter of the predicted values of q_u for a given N value is quite large. Therefore, compression tests should always be made on the samples which are recovered from the split-spoon sampler used in the Standard Penetration Test.

Peck, Hanson, and Thornburn (Ref. 20) give an empirical relationship between the penetration resistance and the relative density and angle of internal friction of sands, as shown in Fig. 3.4. No further useful correlations involving Standard Penetration Test results have been found in the literature.

THD Penetration Test

Since this study is of direct interest to the Texas Highway Department, all penetration test results noted in the following chapters were

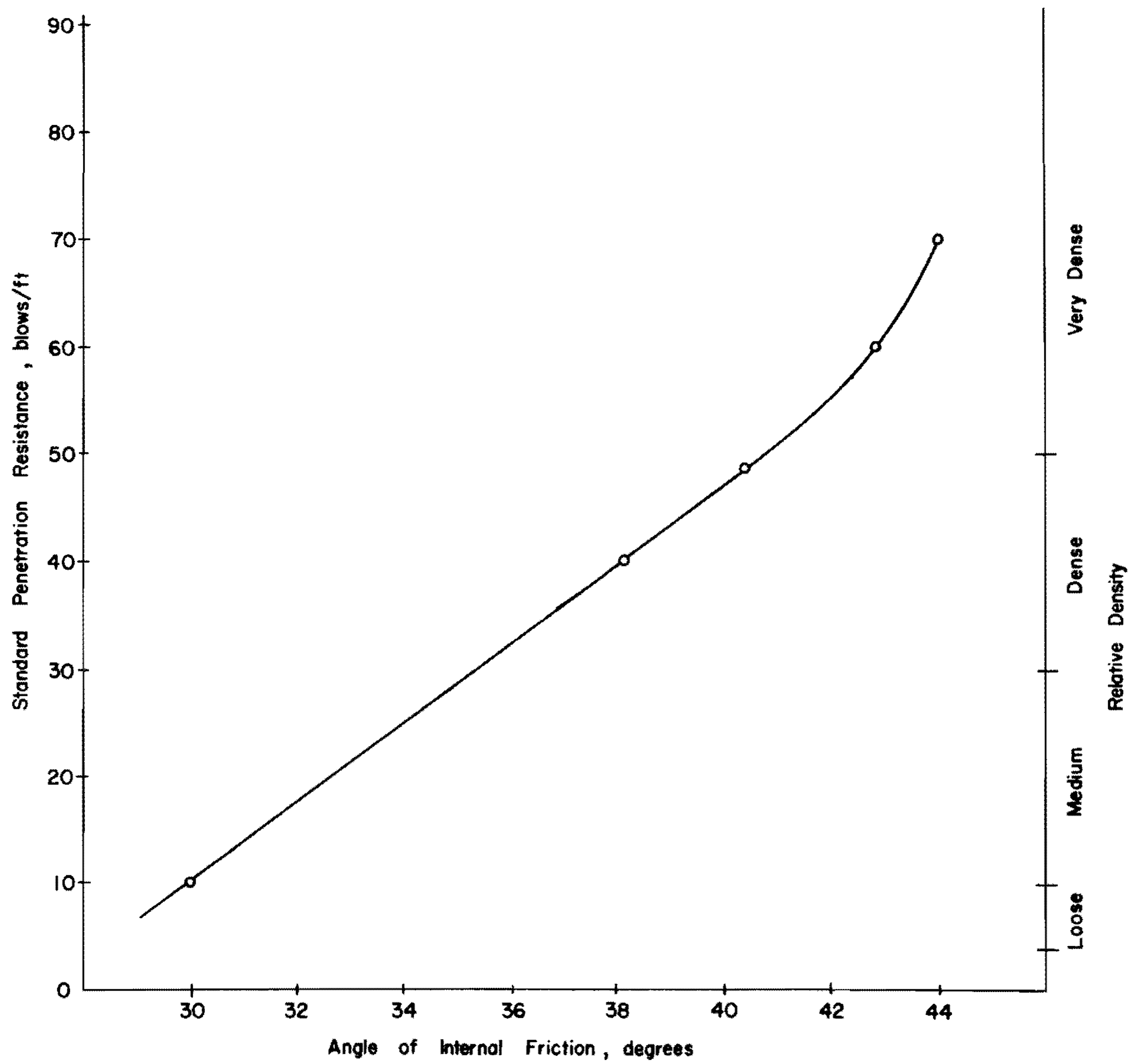

Fig. 3.4. Relationship Between Penetration Resistance and Angle of Internal Friction of Cohesionless Soils (After Peck, et al)

obtained by a test method commonly used by that Department, referred to as the standard THD cone penetration test.

The THD cone penetration test (Ref. 29) utilizes a penetrometer cone 3.0 inches in diameter, as shown in Fig. 3.5. The cone is attached to a drill stem of 2-3/8 inches in diameter and lowered to the bottom of the test hole. An anvil is attached to the top of the drill stem, and an automatic tripping mechanism with a 170-pound hammer is placed on top of the anvil. This arrangement is shown in Fig. 3.6. The drop of the hammer is regulated at two feet. The actual testing is begun by seating the cone with 12 blows of the hammer, except in soft soils, in which the seating is determined by the operator. If the soil to be tested is relatively soft, the cone is driven one foot, and the number of blows required for each six inches of penetration of the cone is recorded. However, in harder material, where the blow count exceeds 100 blows per foot, the procedure is to apply two sets of 50 blows each to the cone and to record the corresponding penetration for each set of blows.

The THD cone penetration test is used quite extensively for obtaining estimates of <u>in situ</u> shear strengths in Texas. Its use in this study involves determination of material properties at the test sites of the Drilled Shaft Project. Discussion of these test results is found in Chapters 5 and 6.

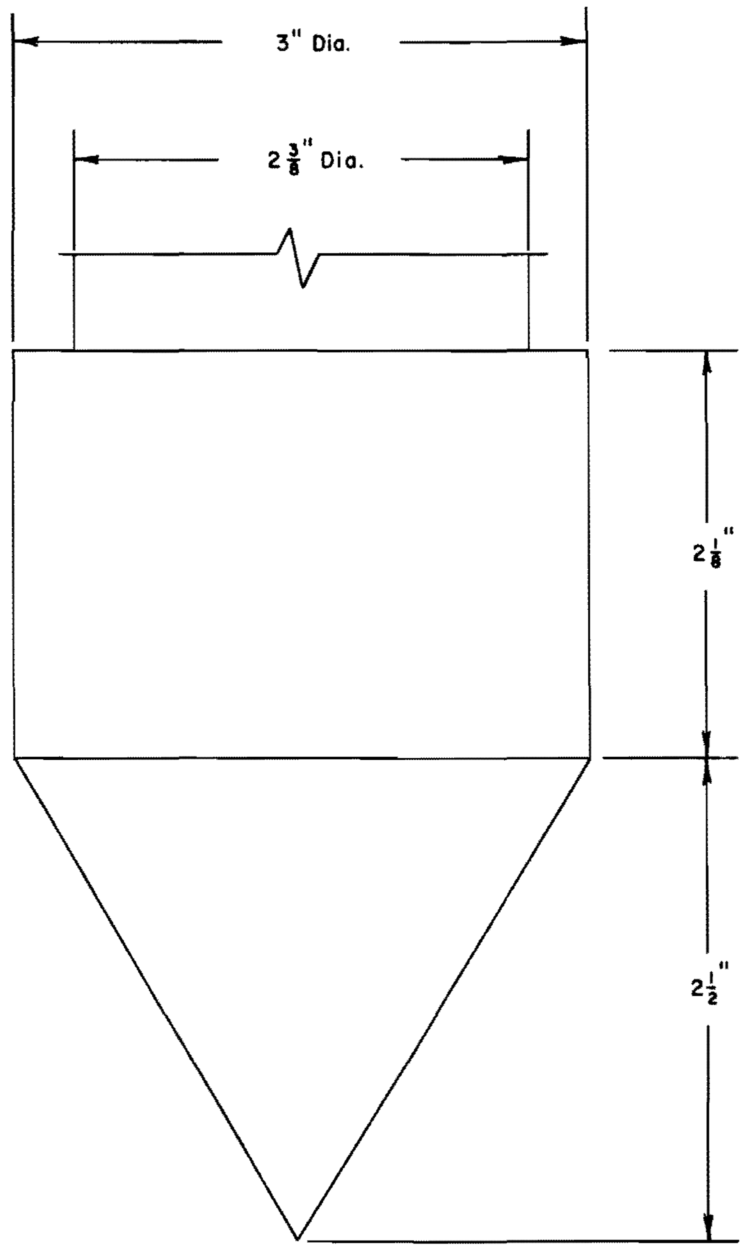

Fig. 3.5. Details of THD Cone Penetrometer

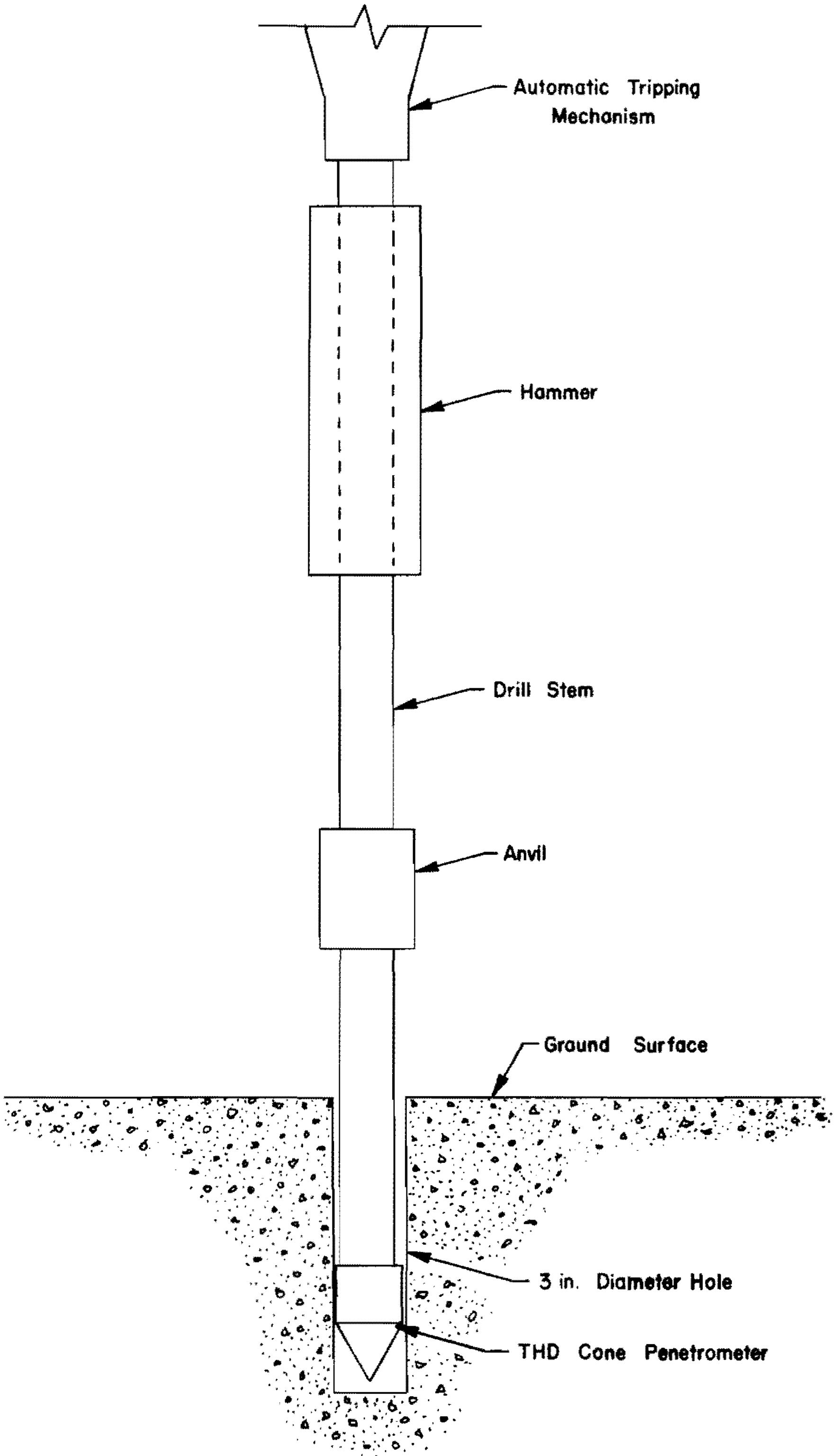

Fig. 3.6. THD Cone Penetration Test Arrangement

CHAPTER 4

UNIVERSITY OF TEXAS *IN SITU* SHEAR TEST DEVICE

In May 1966 considerable sampling difficulty was encountered at the site of a drilled shaft test in San Antonio. Strata composed of gravel, shells, and sandstone existed to such an extent as to inhibit satisfactory sampling. For this reason *in situ* testing seemed desirable and necessary for determining the soil properties at the site. For use in obtaining these soil strength values in San Antonio and for further use in connection with the Drilled Shaft Project (Ref. 22), a device was developed to measure soil shear strength *in situ*. The device is identified as The University of Texas *in situ* device.

Operational Concept

The University of Texas *in situ* device (Fig. 4.1) was designed to shear the soil under investigation and to measure the maximum shearing resistance offered by the soil under known normal pressures.

The first step in the procedure is to lower into a borehole a hydraulic ram with shear plates attached to the ends. During lowering, the ram and plates should be maintained in a horizontal position by adjusting the cables (Fig. 4.1). When the assembly reaches the desired depth of testing, the plates are forced against the walls of the hole, and the hydraulic ram is used to exert a desired normal pressure on the soil in contact with the plates. A hydraulic jack at the surface then applies an upward force on

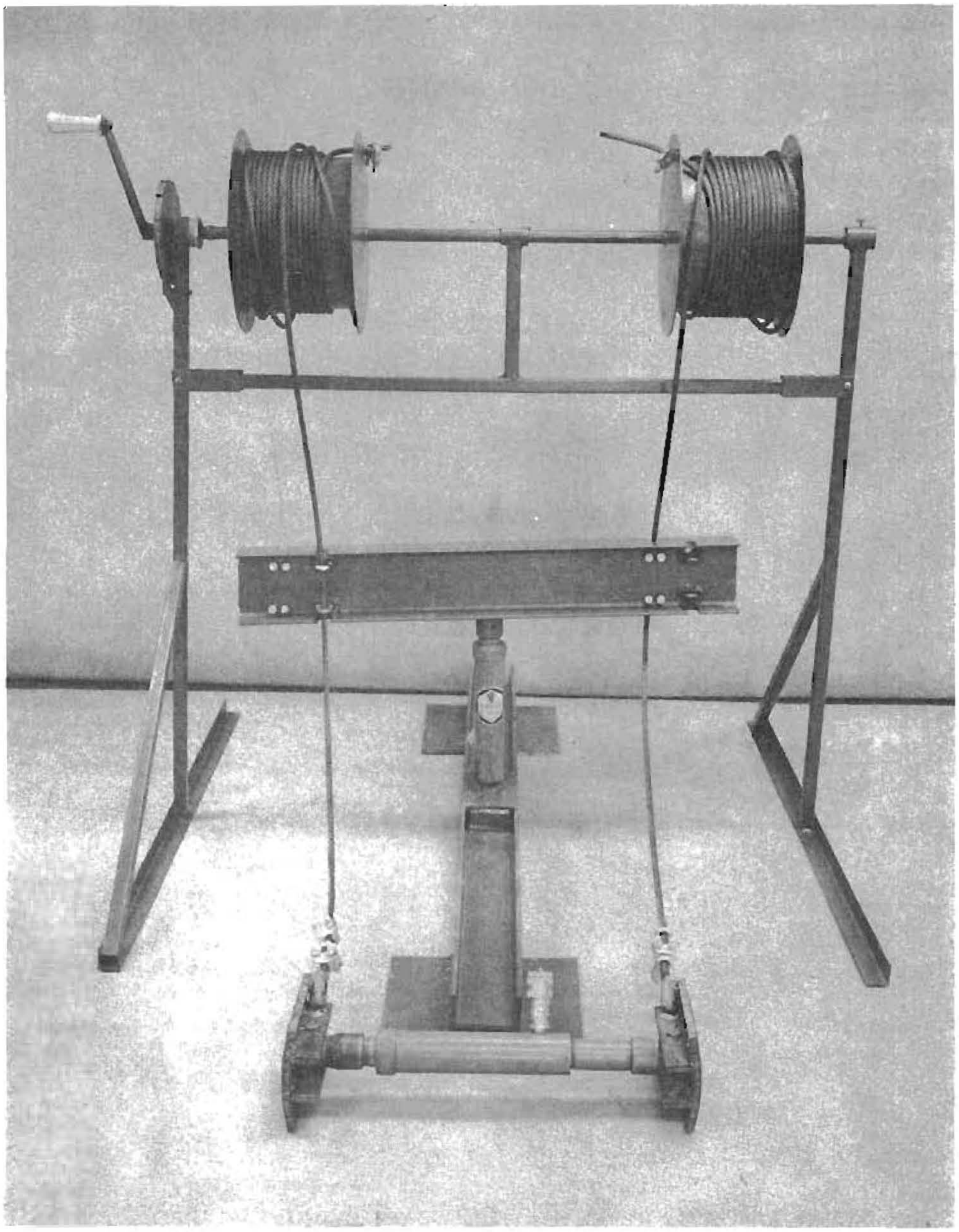

Fig. 4.1. The University of Texas *In Situ* Device

the plates. The plates and ram are pulled upward, thus shearing the soil. A hydraulic gage measures the shearing resistance offered by the soil.

Equipment

The *in situ* device is composed of three primary components: the shear plates, the normal force system, and the pull-out system.

The shear plates are the most important of the parts. One of the two identical plates is shown in Fig. 4.2. They are steel plates which are curved to conform with the walls of the borehole. For a 24-inch-diameter drilled hole, the plates used are 6 inches long, 6 inches wide, and 5/18-inch thick. Curved plates were obtained by cutting segments of a 24-inch-diameter pipe. The width of the shear plates is less than one-tenth of the circumference of the borehole to allow the plates to be used several times around the periphery of the hole at any particular depth.

In an effort to assure uniform contact with the soil over the entire area of the plates and thereby obtain a homogeneous failure in the soil, various attachments to the convex sides of the plates were considered. Among the attachments utilized were several thin steel ribs which were equally spaced along the horizontal direction and welded perpendicular to the surfaces of the plates. These ribs were later replaced by horizontal beams welded to the surfaces. The present attachments are small spikes (shown in Fig. 4.2) welded to the plates. The spikes are 1/4 inch long and 3/16 inch in diameter, and fourteen are equally spaced on each plate. The spikes are believed to be the most effective of the three attachments investigated.

Fig. 4.2. Shear Plate for UT *In Situ* Device

In each case, the ratio of the surface area of the attachments to the plate area in contact with the soil is less than five percent. This is to minimize any friction force or bearing force which might develop between the attachments and the soil. Bearing of the soil on the top edge of each plate is reduced by the presence of a steel piece welded across the top edge of each plate and sloping away from the convex surface (see Fig. 4.2). On the concave side of each plate, two 1/2-inch-thick, 2-inch-wide, and 6-inch-long stiffeners are fixed for transferring normal loads uniformly to the plate. At the top of the stiffeners, cables are attached and extended to the ground surface for transmitting the shear force.

The normal force system (Fig. 4.3) used by the _in situ_ device consists of a hydraulic ram on the ends of which the two shear plates are connected. A ram of ten-ton capacity is used to apply the various normal loads to the plates. A hydraulic pressure gage connected in series with the hydraulic ram measures the normal loads.

The pull-out system (Fig. 4.4) for the _in situ_ device consists of the equipment necessary for applying the shear force to the soil and measuring that force during testing. The cables connected to the shear plates extend above the ground surface and are clamped to a 24-inch-long beam fastened to the top of a vertical hydraulic ram. The ram is centered on a second reaction beam which is positioned over the test hole. The shearing force is applied from the hydraulic ram and transferred to the shear plates by the cables. The shearing force is measured by a hydraulic pressure gage connected to the ram.

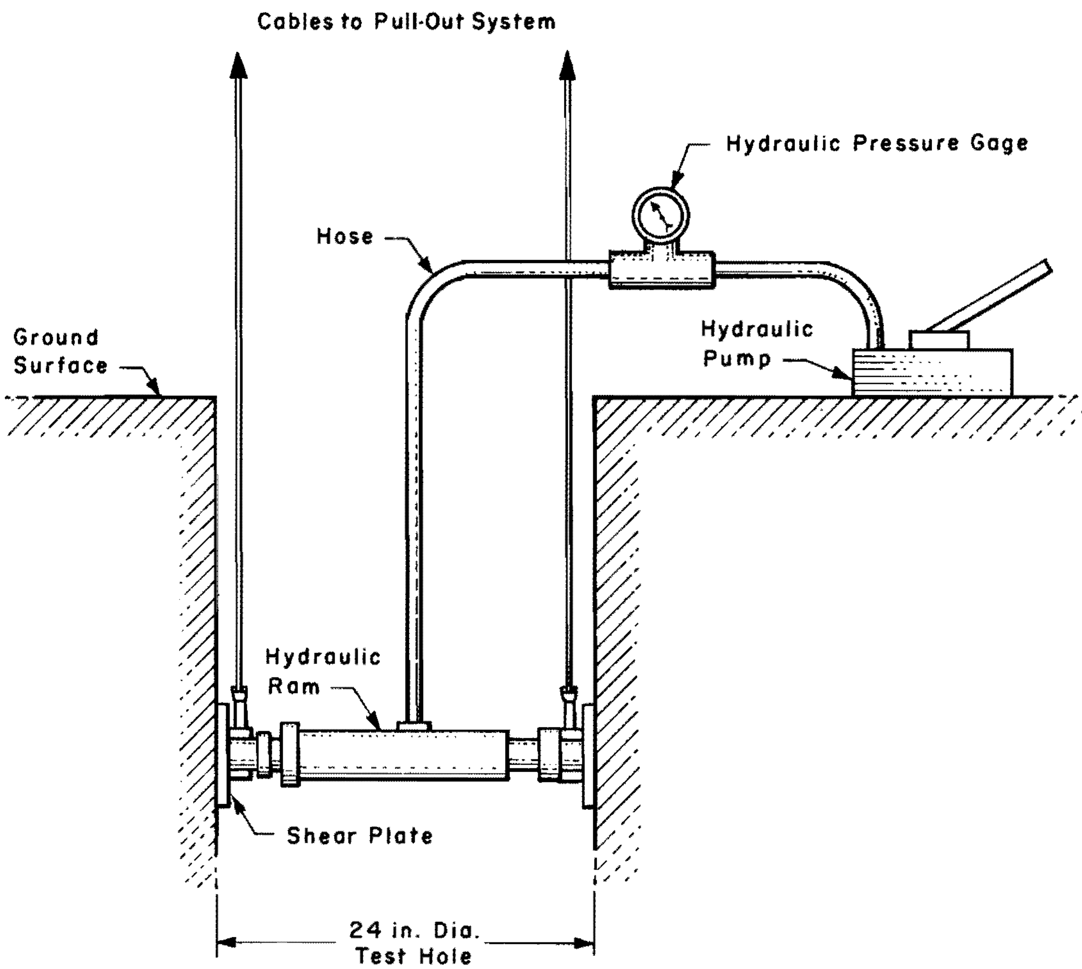

Fig. 4.3. Normal Force System for UT *In Situ* Device

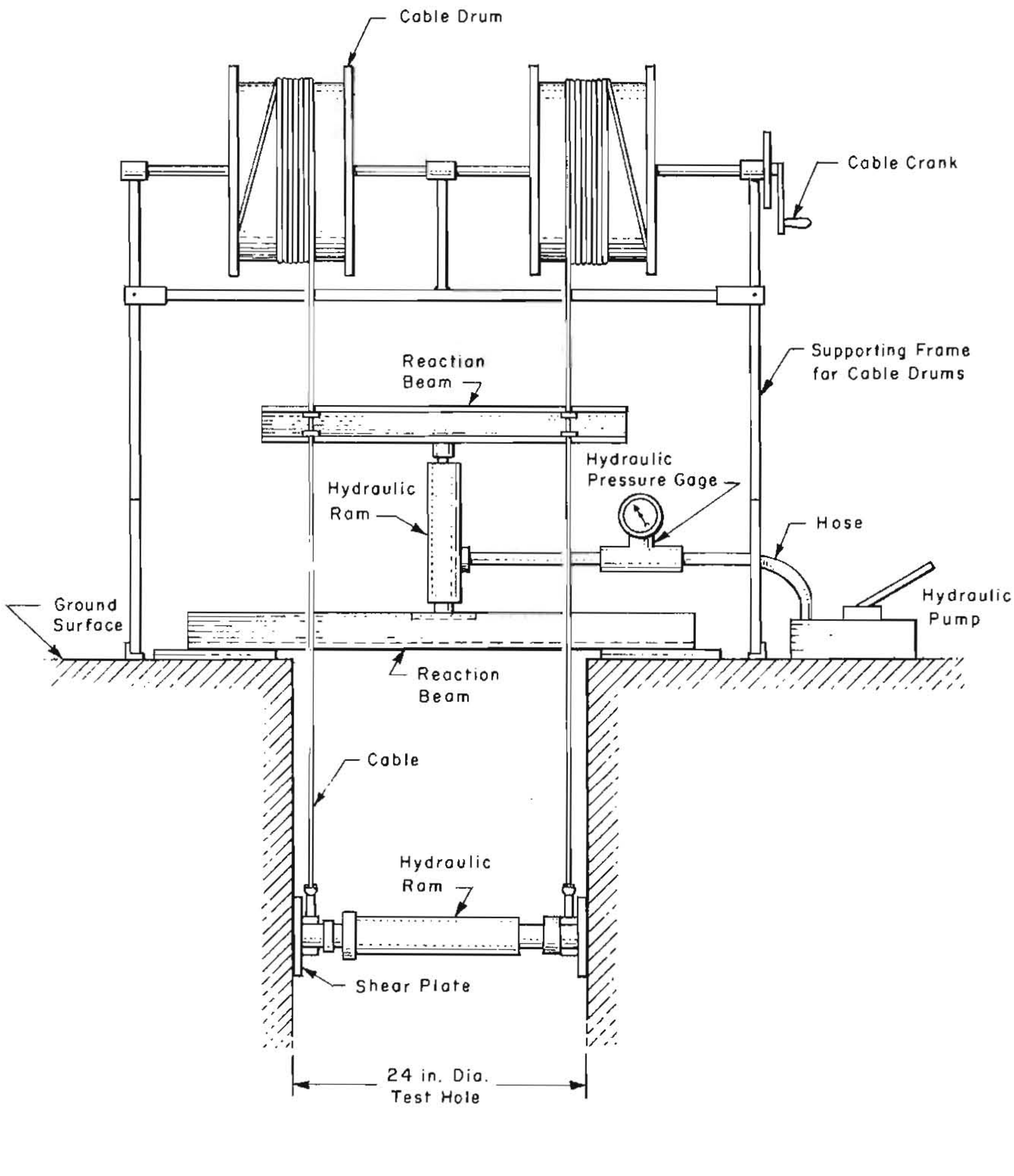

Fig. 4.4. Pull-out System for UT <u>In Situ</u> Device

Testing Procedure

The procedure used for the *in situ* device is as follows:

(1) Drill the test hole at the selected site. The present *in situ* equipment allows testing at depths of up to 30 feet in holes with diameters of 20 to 30 inches. If laboratory soil testing is to accompany the use of the *in situ* device, the hole used for the *in situ* tests should coincide with that from which the samples were obtained.

(2) Center the reaction beam and the frame supporting the cable drum over the hole. It is necessary to lower the horizontal ram into the hole so that the reaction beam may be placed between the two cables.

(3) Connect a hydraulic pump and pressure gage in series to each hydraulic ram.

(4) Lower the shear plates to the desired level of testing and clamp the cables firmly to the upper reaction beam. One-foot intervals are marked on the cables to determine the depth to the shear plates.

(5) Apply the selected normal pressure to the shear plates. Although any magnitude of normal pressure may be applied to the plates, it is believed that an appropriate normal pressure is one which just brings the surfaces of the plates and the soil in good contact. Further discussion of normal pressure selection occurs later in this chapter.

(6) Apply the pull-out force to the shear plates at a slow, constant rate while holding the selected normal pressure constant. Continue to apply the pull-out force until a steady decrease in pressure is noted by the pressure gage measuring the shearing force. The maximum pressure reached is the pressure required to cause a shear failure of the soil.

(7) Release the normal pressure and rotate the plates 60° for further testing at the same depth and repeat Steps 5 and 6. The shear plates may be used three times at each depth with the normal pressure remaining constant or being varied.

Analysis of Data

The shear failure is assumed in the soil, not at the interface between the plates and the soil. The shearing resistance of the soil under the particular normal pressure may be easily calculated when the pressure required for the shear failure is obtained. This maximum pressure is converted into a force, F , in pounds by use of the calibration curve for the vertical hydraulic ram and pressure gage combination. The shearing resistance in tons per square foot is

$$R = \frac{F}{A} \tag{4.1}$$

where

F = maximum shearing force, tons, and

A = total area of both shear plates, square feet.

If several different normal pressures are used, estimates of the cohesion and angle of internal friction of the soil may be obtained from a plot of shearing resistance versus normal pressure. The actual normal force, and thereby the actual normal pressure, on the plates is obtained by converting the normal pressure, as read on the pressure gage, by use of a similar calibration curve.

Limitations

Several difficulties have been encountered in the use of the *in situ* device, as follows:

(1) Perfect contact between the soil surface and the shear plates is not always obtained. This may be due to inclusions in the soil or to the nonhomogeneous nature of the soil in contact with the plates. In these instances it is difficult to determine the effective contact area.

(2) At depths greater than 15 feet it is difficult to place the horizontal ram in a level position. The difficulty is a result of poor illumination and having to level the ram by judgment based on observations from the surface. If the ram is in an unlevel position, part of the shear plates tilts away from the soil when the normal pressure is applied. This also results in poor contact between the soil and the shear plates.

(3) The selection of a proper normal pressure is necessary for obtaining reliable estimates of the shearing resistance of the soil. This selection is difficult and can be determined only by visual inspection from the surface; thus, it becomes more difficult with

increasing depth. Care should be taken to embed the spikes fully into the soil, but not to embed the shear plates, which results in a bearing pressure being developed. The selection of the normal pressure is made easier through experience in working with the *in situ* device. However, the selection will always be based on the judgment of the operator, and thus will be subject to some error.

(4) Soil frequently becomes attached to the shear plates and must be removed to avoid erroneous measurements. The process of pulling the plates up and cleaning them after each test is very time consuming.

(5) The pull-out force cannot be applied at a steady rate, as the pumping of the hydraulic fluid to the vertical ram is done by hand.

(6) The cables need to be tightly clamped before beginning the test. Occasionally one of the cables slipped during testing. This resulted in a shifting of the load to the other cable and a possible error in the measurement of the shearing resistance.

(7) Eccentric loading exists on the horizontal ram when one shear plate moves more than the other.

(8) The measured force applied to the horizontal ram could be incorrect due to the ram being unlevel.

CHAPTER 5

RESULTS OF CORRELATION STUDIES

In order to analyze the findings of this study, several correlations were made of *in situ* test results. Shearing resistances estimated from the UT *in situ* device were correlated with

(1) strengths obtained by conventional laboratory tests, and

(2) penetration resistance measured with the THD cone penetrometer.

Maximum load transfer results obtained from tests of instrumented drilled shafts were correlated with

(1) shearing resistances measured with the *in situ* device, and

(2) penetration resistance measured with the THD cone penetrometer.

In Situ Device Versus Laboratory Tests

The *in situ* device was used on two occasions at a test site near Montopolis, Texas, the location of the first drilled shaft test. A soil profile of the Montopolis site is shown in Fig. 5.1.

The initial use of the *in situ* device was in a 24-inch-diameter hole, ten feet in depth. The shearing resistance results from this initial field testing were compared with one-half of the unconfined compression test results reported by Eulalio Juarez-Badillo (Ref. 11) in his soil investigation report for this particular drilled shaft test. This comparison is shown in Fig. 5.2.

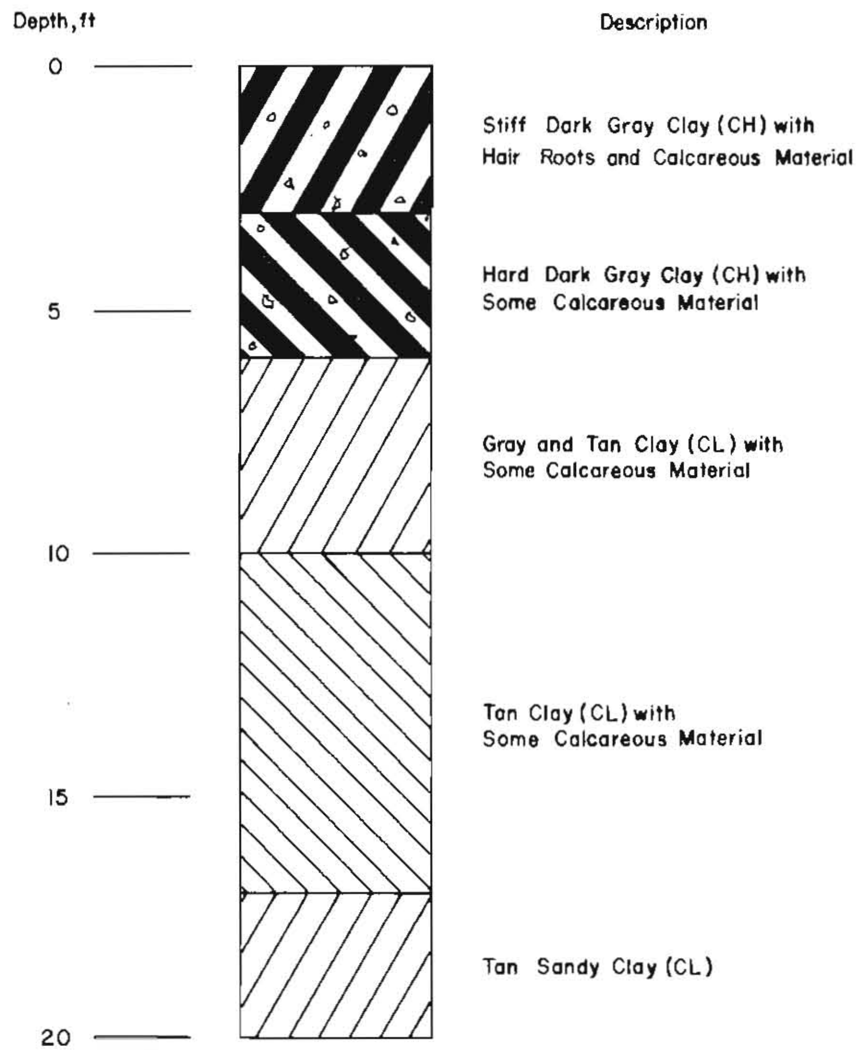

Fig. 5.1. Soil Profile for Test Site at Montopolis

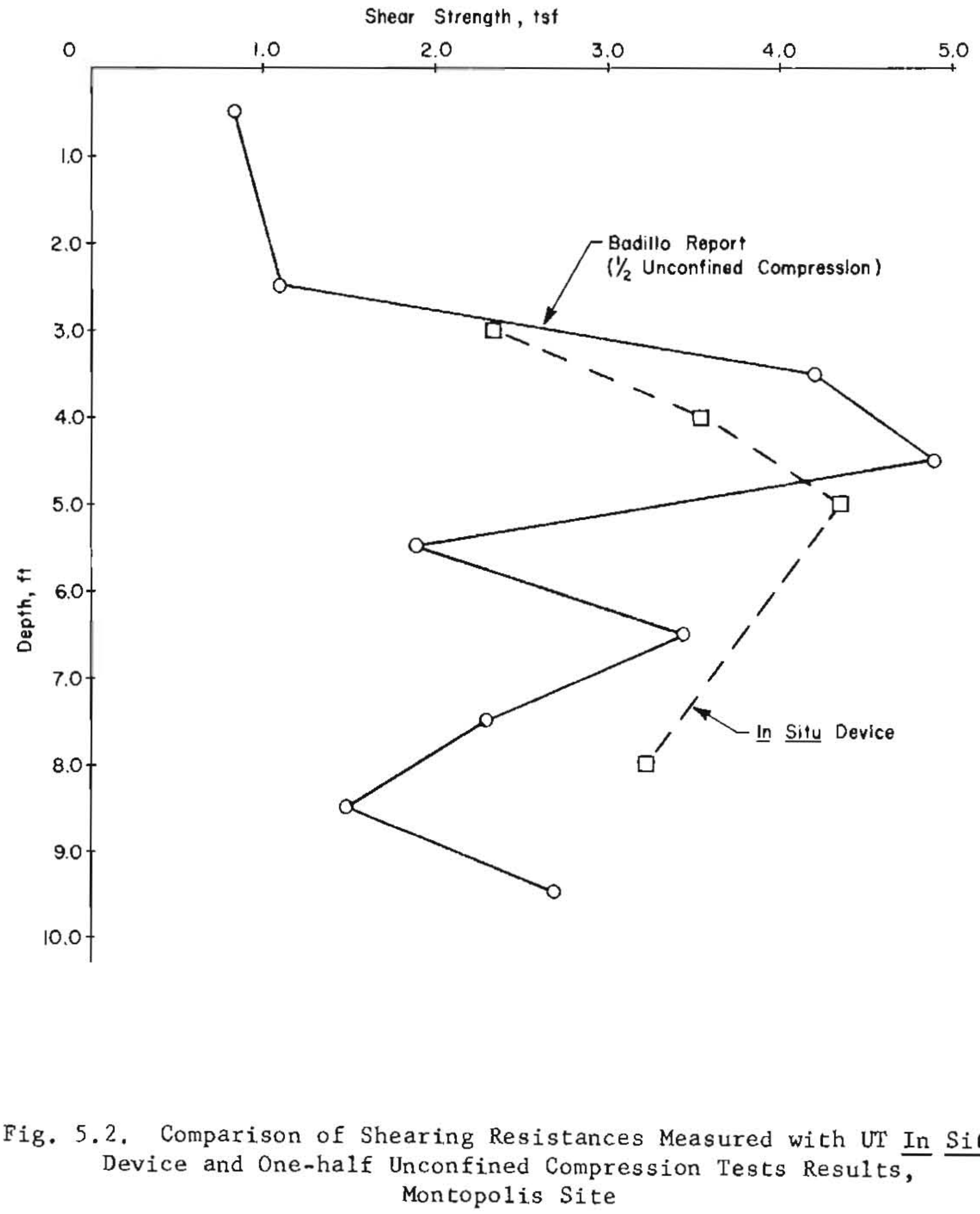

Fig. 5.2. Comparison of Shearing Resistances Measured with UT *In Situ* Device and One-half Unconfined Compression Tests Results, Montopolis Site

The second use of the *in situ* device at Montopolis was in a test hole 24 inches in diameter and 20 feet in depth. Undisturbed samples were recovered from this hole, and direct shear tests were performed on them at normal pressures approximately the same as those used with the *in situ* device at the corresponding depths. The correlation of these direct shear test results and shearing resistances obtained with the *in situ* device is shown in Fig. 5.3.

As seen from the plots, the data are quite scattered for each correlation. This scatter is attributed in part to the large number of rocks and nodular inclusions existing in the soil, which prevented good contact between the shear plates and the soil. Erroneous measurements with the *in situ* device also resulted at times when the shear plates became embedded in the soil, which caused a bearing pressure to be developed on the top edge of the plate. The method of testing also contributed to the variation in the test results. Whereas samples tested in unconfined compression will fail at their weakest point, direct shear specimens are forced to fail on a certain plane, which quite possibly is not the weakest plane in the sample. In contrast to these, the shearing resistances measured with the UT device are average strengths of the soil in contact with the shear plates. That is, the soils in contact with the plates are of varying strengths, and they are all involved in the shear failure, resulting in an average shearing resistance being measured.

In Situ Device Versus THD Penetrometer

The *in situ* device was also used at a drilled shaft test site in San Antonio, Texas. The test hole in San Antonio was 30 inches in diameter

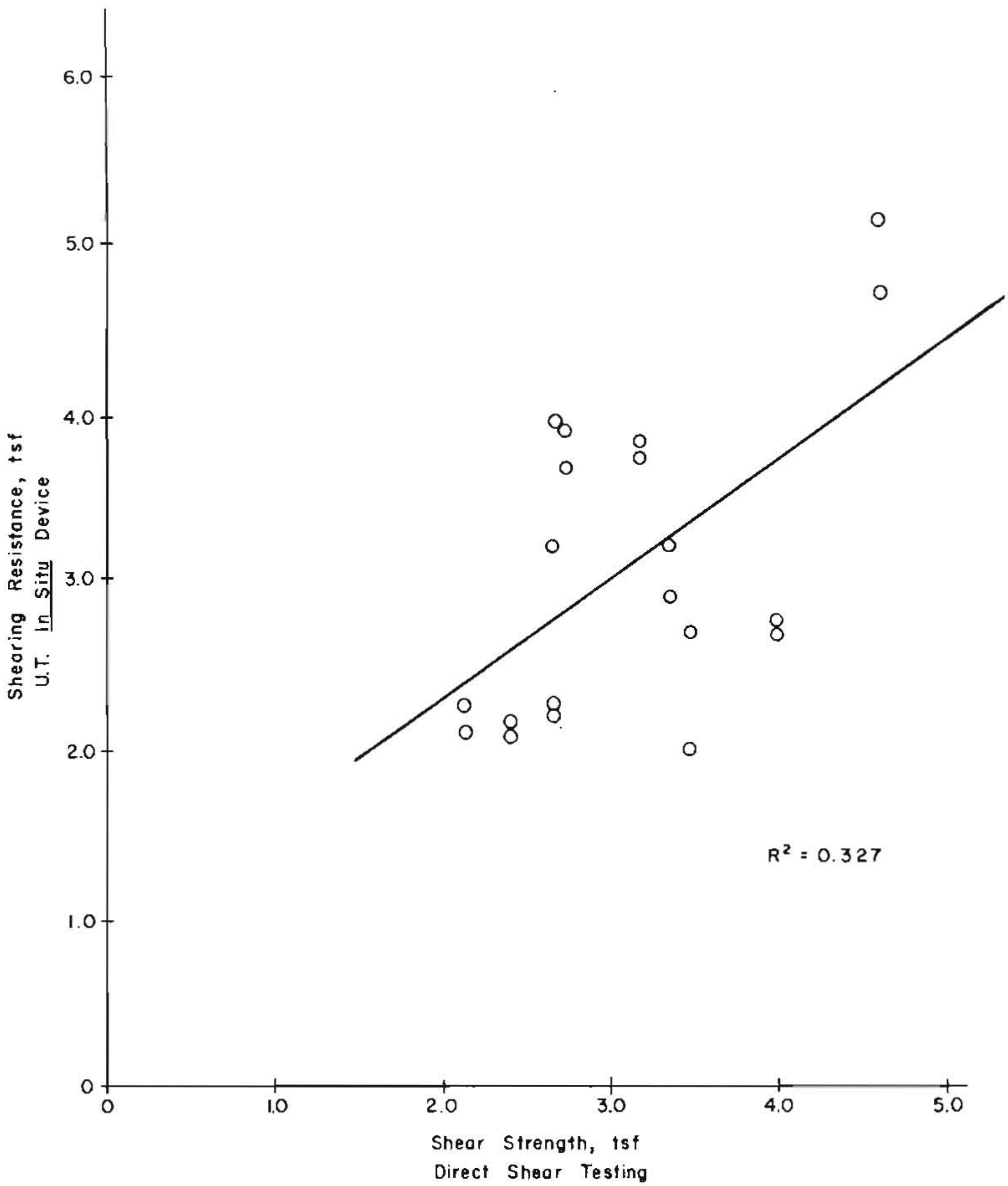

Fig. 5.3. Correlation of Shearing Resistances Measured with UT *In Situ* Device and Direct Shear Test Results, Montopolis Site

and 30 feet in depth. A soil profile of this site is shown in Fig. 5.4. The shearing resistance results of this test plus those from the Montopolis tests were correlated with penetration resistance as measured by the THD penetrometer, which was also used to estimate soil shear strengths at each site. This correlation (Fig. 5.5) was made to compare the UT *in situ* device with a more established method of *in situ* testing.

The low correlation is attributed primarily to the difficulties encountered with the UT device, such as embedding the shear plates and not achieving good contact between the plates and the soil. Such conditions can result in excessive resistances being recorded in relatively weak soils and resistances below their actual values being recorded in stronger soils. However, the THD penetration resistances (Refs. 22 and 32) are affected by a number of factors, which can cause the values to be unreliable. The arbitrary placement of the penetrometer for testing may or may not result in reliable average values of penetration resistance being measured for the soils encountered. The presence of rocks and shells can result in the recording of resistances in excess of those actually existing in the soil strata. The nonhomogeneous nature of soils and varying moisture contents also add to the scatter of penetration resistance values which can be measured in a soil stratum.

In Situ Device Versus Load Transfer

Essential to this study was the determination of how well the load transfer in a drilled shaft could be estimated by using *in situ* testing apparatus. A correlation was made between shearing resistances measured with the *in situ* device and maximum load transfer values obtained from

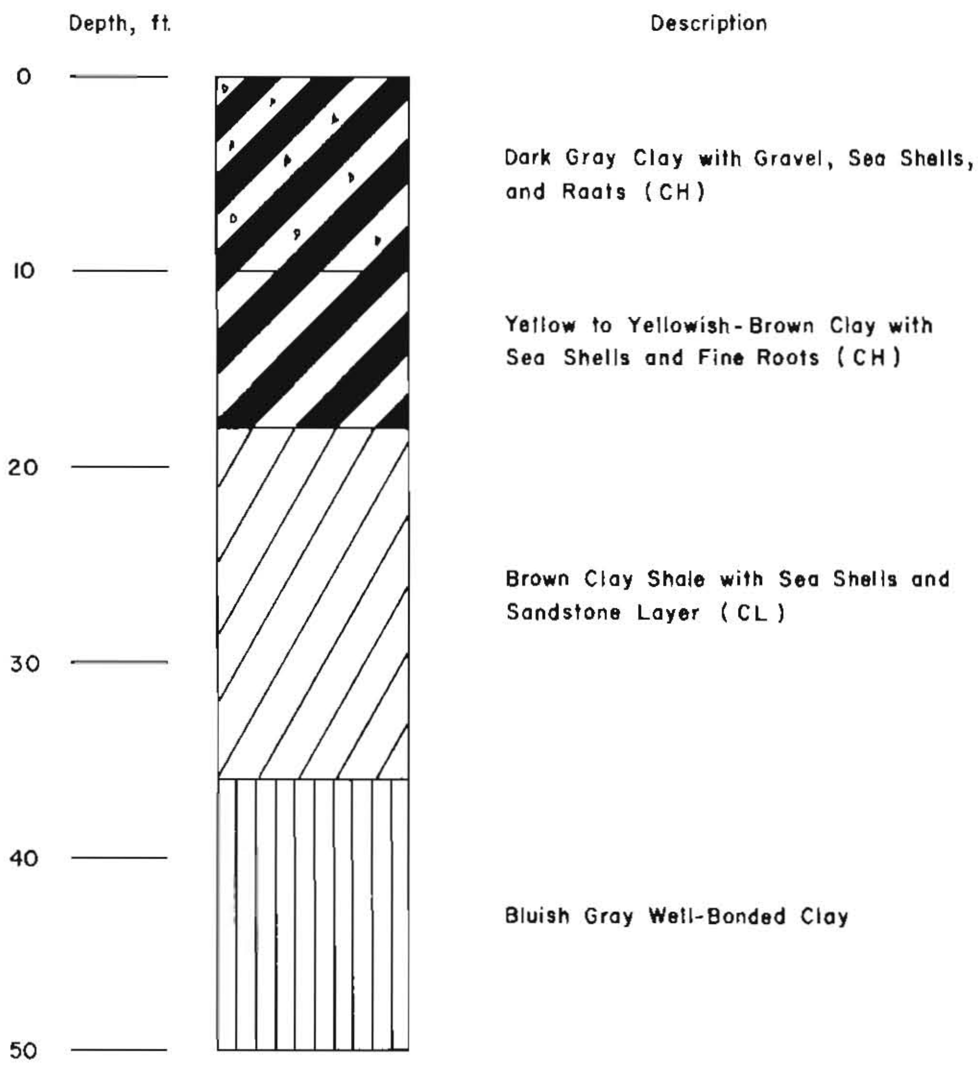

Fig. 5.4. Soil Profile for Test Site at San Antonio

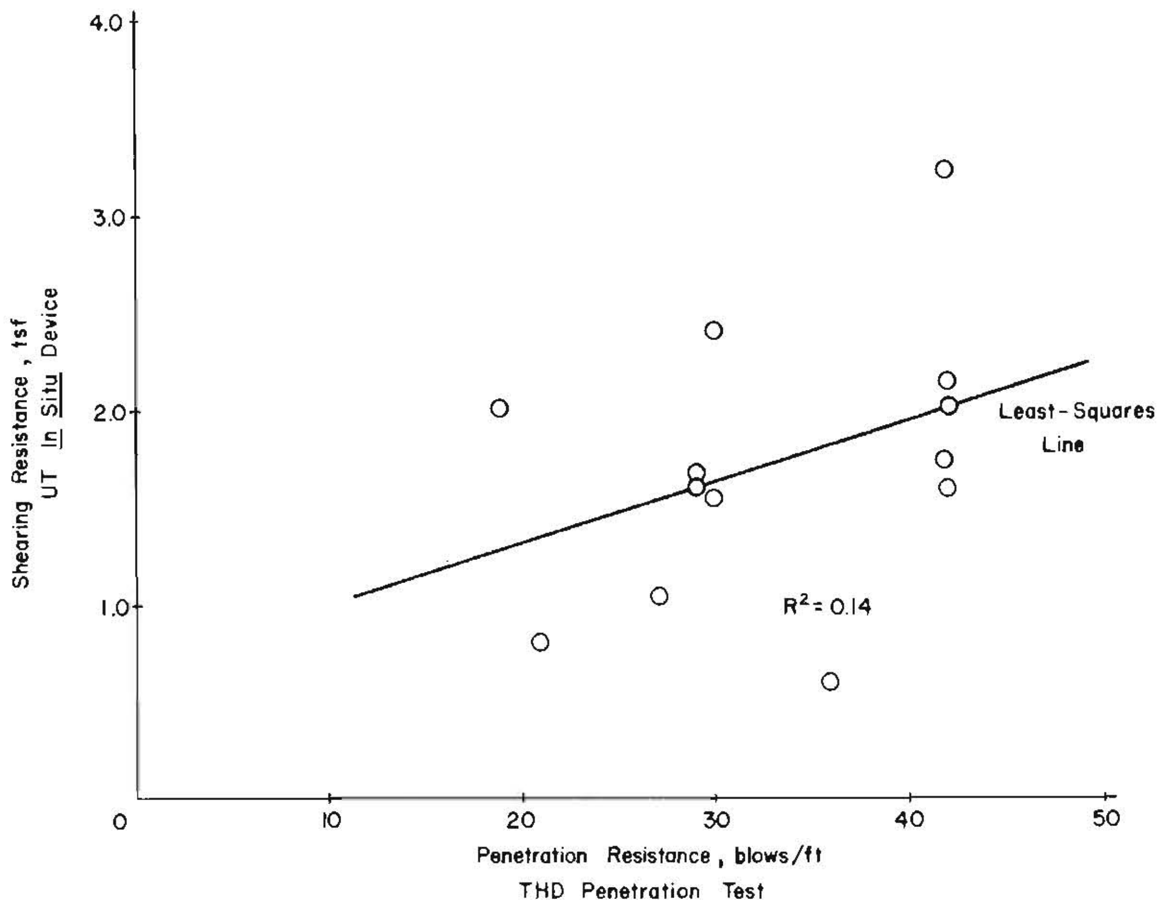

Fig. 5.5. Correlation of Shearing Resistances Measured with UT In Situ Device and Penetration Resistance Measured by THD Penetrometer, Montopolis and San Antonio Sites

instrumented drilled shafts. This correlation was performed for both the Montopolis and San Antonio test sites. The results are shown in Figs. 5.6 and 5.7, respectively.

The correlation for the Montopolis site is very poor and also negative. This lack of significant correlation is attributed primarily to the operational difficulties of the UT device noted above, since the load transfer values (Ref. 22) were measured with accurate instrumentation and appear to be reliable.

The correlation for the San Antonio site is positive, but also quite low. Two straight-line trends are noted in this data. The lower three data points, which fall in a nearly horizontal line, represent test results in the clay strata near the surface, and the increasing shearing resistances measured with the UT device are primarily attributed to the partial embedment of the shear plates during testing. In the harder shale and sandstone strata at deeper depths, load transfer values increased appreciably with depth. The <u>in situ</u> device resistances increased likewise but at a lower rate. Nevertheless, the correlation of the data at this depth is fairly good. The load transfer values (Ref. 32) are measured with accurate instrumentation and appear to be reliable.

<u>THD Penetrometer Versus Load Transfer</u>

A correlation was likewise made between the THD penetrometer results and load transfer test results in an effort to determine the extent that load transfer could be estimated by penetration resistance. These correlations for the two sites are shown in Figs. 5.8 and 5.9. At each test site, the correlation is quite high, as shown by the R^2 values of 0.968 and

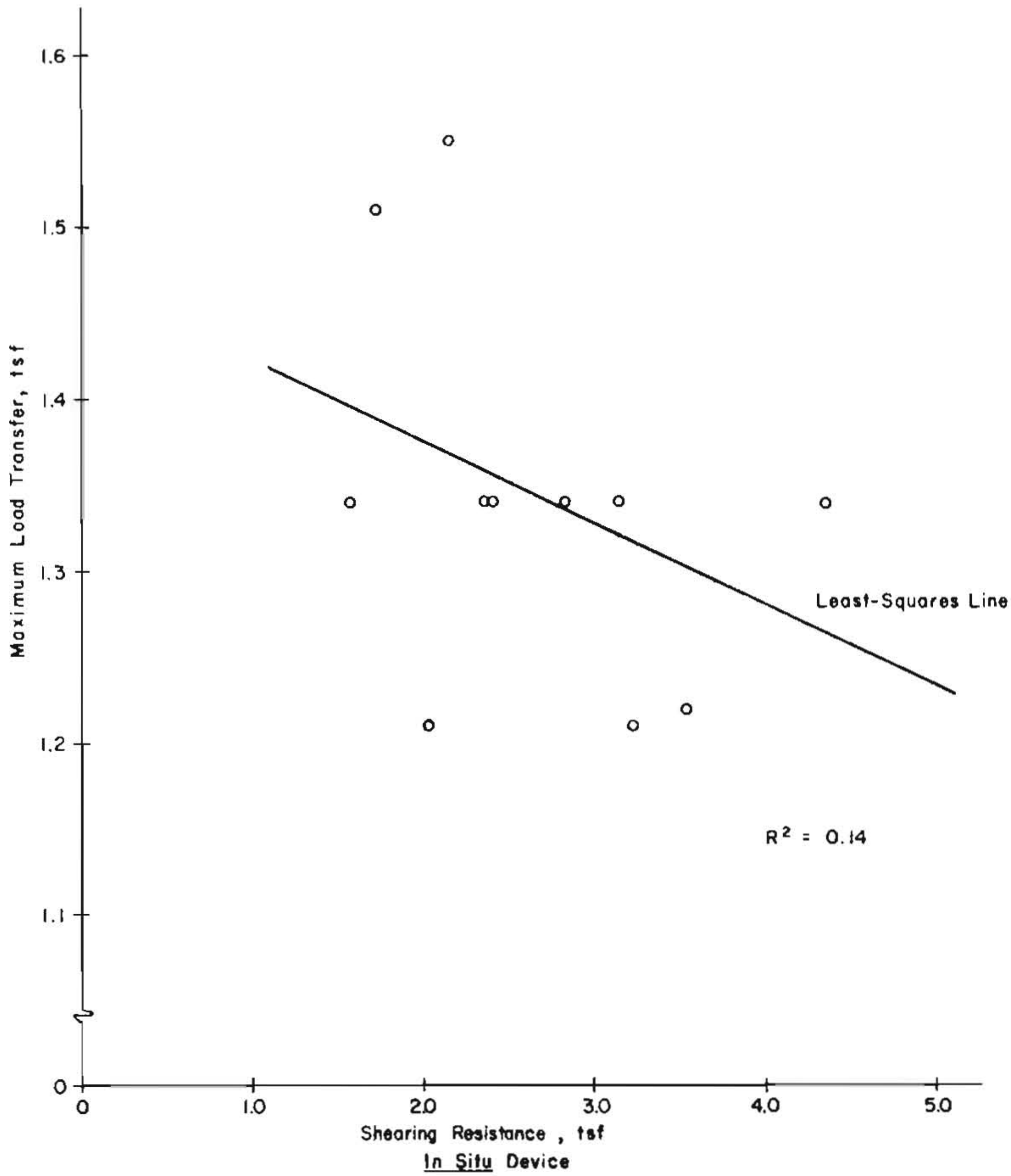

Fig. 5.6. Correlation of Load Transfer and Shearing Resistances Measured with UT *In Situ* Device, Montopolis Site

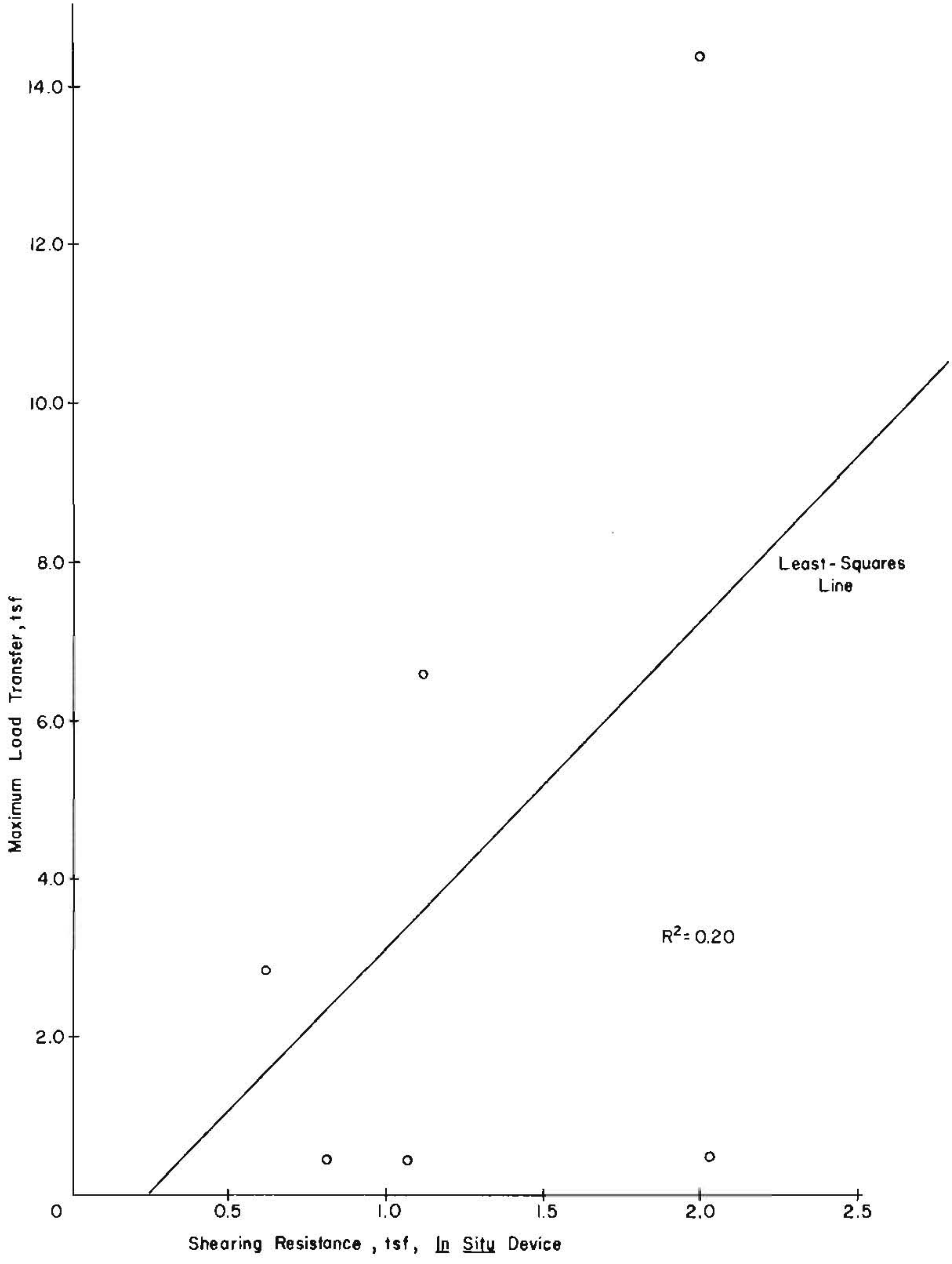

Fig. 5.7. Correlation of Load Transfer and Shearing Resistances Measured with UT *In Situ* Device, San Antonio Site

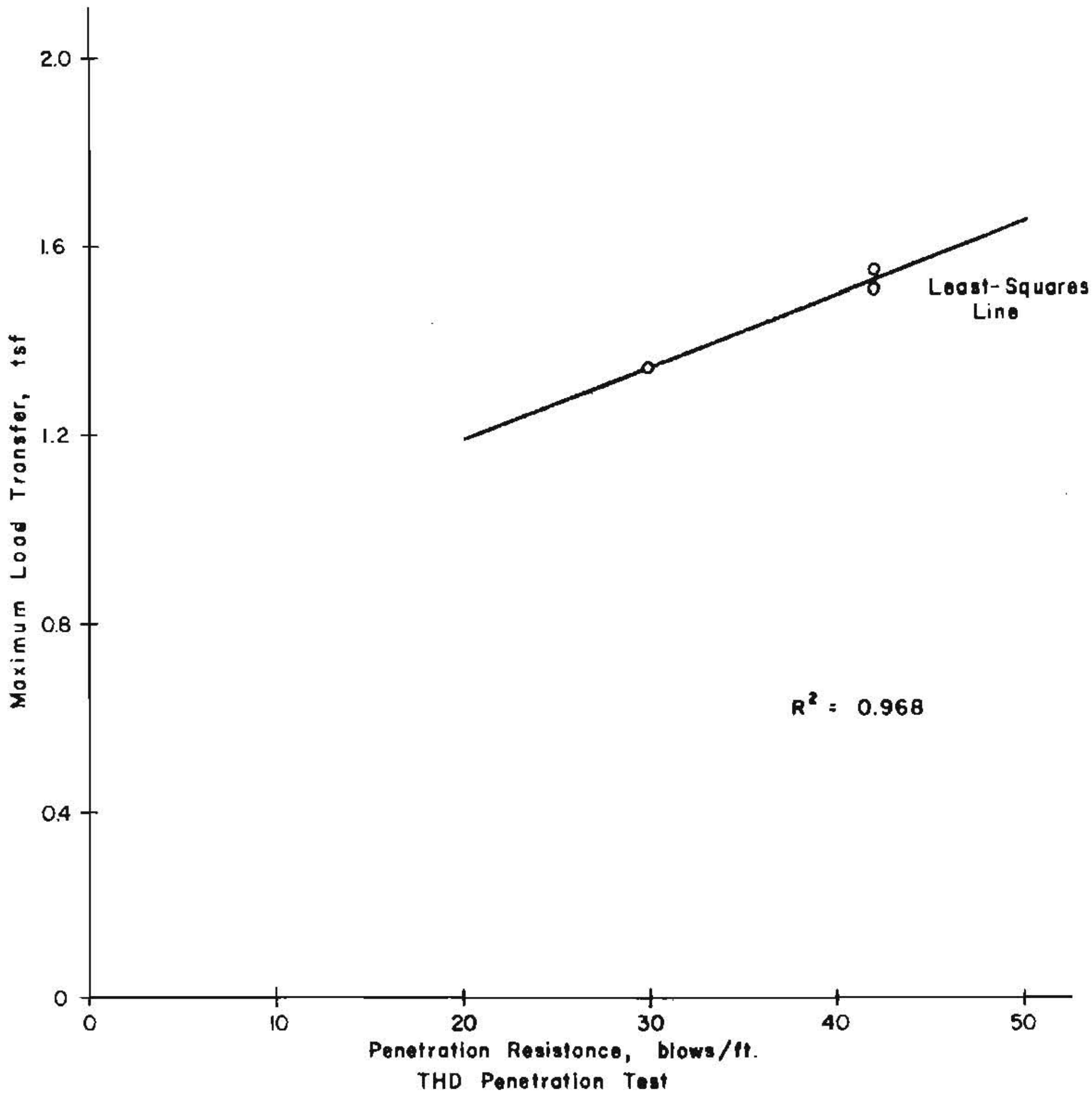

Fig. 5.8. Correlation of Load Transfer and Penetration Resistance, Montopolis Site

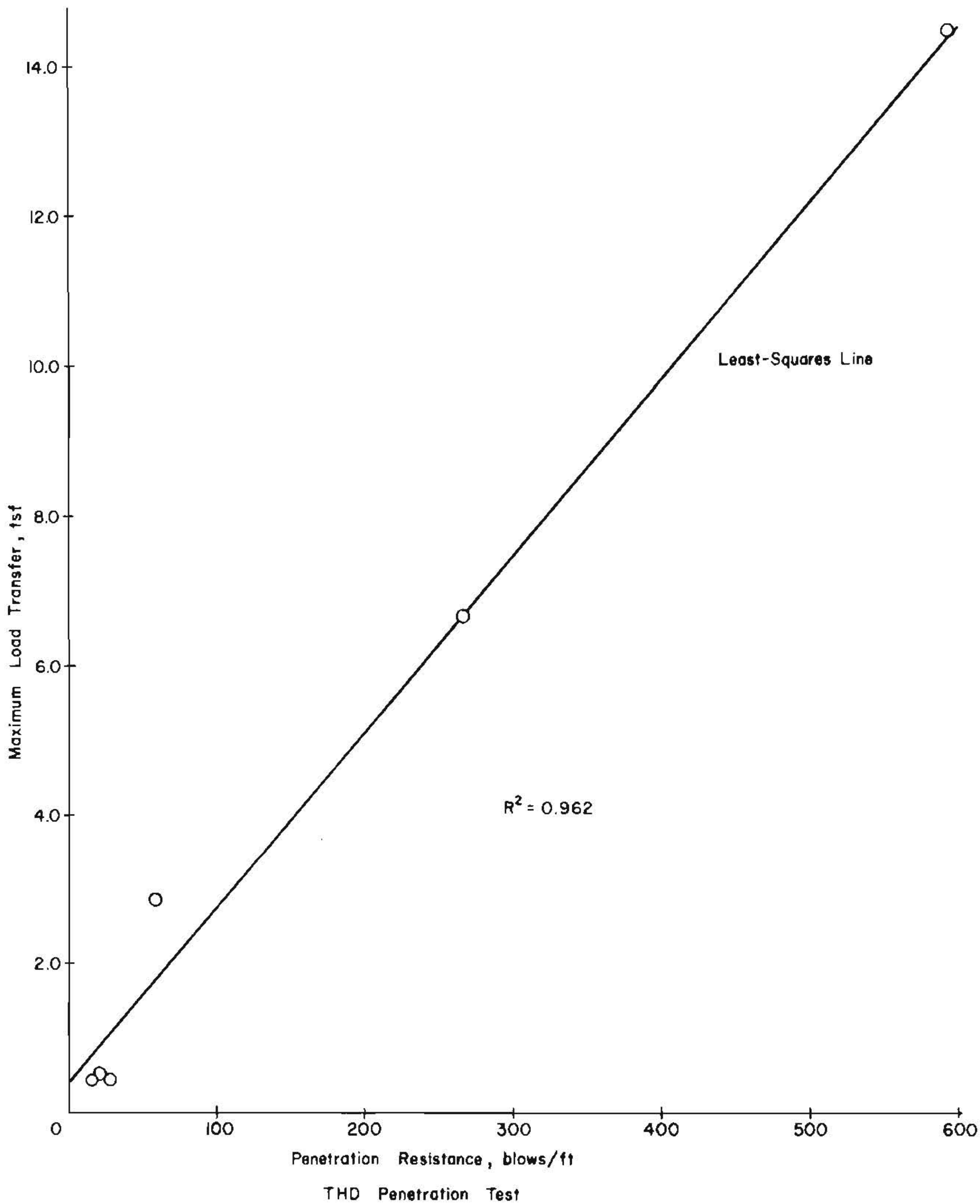

Fig. 5.9. Correlation of Load Transfer and Penetration Resistance, San Antonio Site

0.962. From these R^2 values, it may be interpreted that approximately 96 per cent of the variation in load transfer values is linearly associated with the variation in penetrometer values. The penetrometer was consistent in estimating load transfer values at each site.

Vijayvergiya (Ref. 32) in an earlier study correlated the same load transfer and penetrometer data from the San Antonio site together with values of shaft movement during testing and by a regression analysis arrived at a nonlinear relationship given in the form

$$T = \frac{N}{35} \left(2\sqrt{\frac{s}{s_o}} - \frac{s}{s_o} \right) \tag{5.1}$$

where

T = load transfer at any depth, tons per square foot,

N = penetration resistance, blows per foot,

s = movement of drilled shaft at any depth, inches, and

s_o = maximum settlement of drilled shaft, inches, assumed to be six per cent of the shaft diameter.

The relationship between T, N, and s given by Eq. 5.1 was devloped for the specific soil conditions existing at the San Antonio site. These soil conditions are far from homogeneous, as indicated by Fig. 5.4. The suggested value of s_o may be much higher for soft clays and much smaller for hard clay shale. The values of the constants are based on the study of only a limited number of cases. More case studies involving different soil conditions and different load test procedures are necessary before any firm recommendations can be made regarding the estimation of load transfer values in this manner (Ref. 32).

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Conclusions are drawn from these correlation studies and recommendations are made concerning the future use of these three methods of in situ soil testing as follows.

(1) Data available from other sources indicate that the Menard Pressuremeter readings compare well with shear strengths measured by conventional laboratory tests, and therefore the device may be useful for determining in situ properties of homogeneous fine-grained soils, but probably not for gravelly materials.

(2) Penetration resistance measured with the Texas Highway Department cone penetrometer appears to correlate well with maximum load transfer values obtained from field tests of drilled shafts.

(3) The shearing resistances measured with The University of Texas in situ device are not well correlated with either direct shear test results or with THD penetrometer results.

(4) There appears to be no significant correlation between the shearing resistances measured with the UT in situ device and load transfer tests results.

(5) In its present configuration, the UT in situ device has not proven to be an effective means for determining soil properties.

Recommendations

Based on the above conclusions, the following recommendations are made:

(1) The Menard Pressuremeter should be obtained for further study in the Drilled Shaft Project, if possible. Shear strengths measured with the pressuremeter should be correlated with laboratory soil strengths and with load transfer field data.

(2) Use of the THD penetration test should continue at drilled shaft test sites, with further correlations being made between penetration resistance and load transfer values.

(3) Use of the UT *in situ* device in its present design should be discontinued unless and until it can be greatly improved. Two needed revisions are (a) a means of unwinding equal lengths of cable on each end of the horizontal ram to assure that the ram is in a level position when at the proper depth for testing, and (b) a means of applying the pull-out force at a steady rate to assure a constant shearing rate of the soil.

In summary, no additional work seems warranted on the device at the present time.

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