TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWATX77-10-3F	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle CORRELATION OF THE TEXAS CONE PENETROMETER TEST N-VALUE WITH SOIL SHEAR STRENGTH		5. Report Date August, 1977 6. Performing Organization Code
7. Author(s) Franklin J. Duderstadt, Harry M. Coyle, and Richard E. Bartoskewitz		8. Performing Organization Report No. Research Report 10-3F
9. Performing Organization Name and Address Texas Transportation Institute Texas A&M University College Station, Texas 77843		10. Work Unit No. 11. Contract or Grant No. Research Study 2-5-74-10
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763		13. Type of Report and Period Covered Final - September, 1973 August, 1977 14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with DOT, FHWA.
Research Study Title: "Correlation of the THD Cone Penetrometer Test N-Value with Shear Strength of the Soil Tested."

16. Abstract

Improved correlations have been developed between the Texas Cone Penetrometer Test N-value and the shear strength of both cohesive and cohesionless soils. Correlations were also developed and compared with existing correlations for several shear strength parameters and the Standard Penetration Test N-value. Both field and laboratory investigations were conducted to obtain the necessary data to develop the correlations.

Penetrometer test data and undisturbed soil samples were obtained from five test sites for cohesive soils and six test sites for cohesionless soils. Reasonably good correlations were developed between the unconsolidated-undrained shear strength and the penetrometer test N-value for cohesive soils including homogeneous clays of high plasticity and silty or sandy clays of low plasticity. In addition, a reasonably good correlation was developed between the drained shear strength and the penetrometer test N-value for cohesionless soils including poorly graded sands and silty sands. The currently used relationship between the effective angle of shearing resistance of cohesionless soils and the penetrometer test N-value was found to be a lower bound for the data obtained in this study. Finally, correlations were attempted between unit skin friction and unit point bearing obtained from bored and driven pile tests and the penetrometer test N-value. These correlations are considered preliminary because only a limited amount of data was available from the instrumented pile load tests.

17. Key Words Penetrometer Test N-values, Cohesive Soils - Undrained Shear Strength, Cohesionless Soils - Drained Shear Strength.		18. Distribution Statement No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 124	22. Price

Form DOT F 1700.7 (8-69)

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ABSTRACT

Improved correlations have been developed between the Texas Cone Penetrometer Test N-value and the shear strength of both cohesive and cohesionless soils. Correlations were also developed and compared with existing correlations for several shear strength parameters and the Standard Penetration Test N-value. Both field and laboratory investigations were conducted to obtain the necessary data to develop the correlations.

Penetrometer test data and undisturbed soil samples were obtained from five test sites for cohesive soils and six test sites for cohesionless soils. Reasonably good correlations were developed between the unconsolidated-undrained shear strength and the penetrometer test N-value for cohesive soils including homogeneous clays of high plasticity and silty or sandy clays of low plasticity. In addition, a reasonably good correlation was developed between the drained shear strength and the penetrometer test N-value for cohesionless soils including poorly graded sands and silty sands. The currently used relationship between the effective angle of shearing resistance of cohesionless soils and the penetrometer test N-value was found to be a lower bound for the data obtained in this study. Finally, correlations were attempted between unit skin friction and unit point bearing obtained from bored and driven pile tests and the penetrometer test N-value. These correlations are considered preliminary because only a limited amount of data was

available from the instrumented pile load tests.

KEY WORDS: Penetrometer Test N-values, Cohesive Soils - Undrained Shear Strength, Cohesionless Soils - Drained Shear Strength.

SUMMARY

The information presented in this report was developed during a four-year study on "Correlation of the Texas Cone Penetrometer Test N-value with Shear Strength of the Soil Tested." The objective of the study was to develop an improved correlation between the Texas Cone Penetrometer Test N-value and the shear strength of different soil types to include sand, silt, and clay.

The first phase of the study dealt with cohesive soils. Field investigations for cohesive soils included eight borings taken at five different sites where the Texas Cone Penetrometer Test was conducted and undisturbed soil samples were obtained. The Texas Triaxial Test and the ASTM Triaxial Test were used in the laboratory investigation to obtain soil shear strength. Soils were classified and grouped by the Unified Soil Classification System. Correlations were developed between the unconsolidated-undrained shear strength, c_u, and the penetration resistance N-values for homogeneous CH soils, silty CL soils, and sandy CL soils.

The second phase of the study dealt with cohesionless soils. The field investigations for cohesionless soils included eight borings taken at six different test sites where the Texas Cone Penetrometer Test was conducted and undisturbed samples were obtained. The direct shear test was used to determine the effective angle of shearing resistance, ϕ', used in calculating the drained shear strength, s. Correlations were developed between the penetration test N-value and the drained shear strength, s, the effective overburden pressure, p', and the total

unit weight, γ_T. The relationship currently in use by the Texas State Department of Highways and Public Transportation (SDHPT) between ϕ' and the Cone Penetrometer N-value was examined and found to be a lower bound for the data obtained in this study. The soils tested were classified by the Unified Classification System and included SP, SM and SP-SM soil types.

During the third phase of the study correlations were developed relating both unit side friction and unit point bearing with the Texas Cone Penetrometer Test for bored and driven piles. The data used to develop the correlations for bored piles were obtained from eleven piles tested by researchers with the Center for Highway Research, University of Texas at Austin. The data used to develop the correlations for driven piles were obtained from five piles tested by researchers at Texas Transportation Institute. A limited amount of data was available for this phase of the study and there was considerable data scatter. The correlations developed are considered to be preliminary and more data from instrumented test piles are needed to verify the correlations.

IMPLEMENTATION STATEMENT

New correlations have been developed as a result of this study relating design stress (one-half soil shear strength) with the N-value obtained from the Texas Cone Penetrometer Test for several soil types. Fig. 33 in this report gives the new design curves for homogeneous CH soils; silty and sandy CL soils; and SP, SM, SP-SM soils. Also, Fig. 34 in this report gives a proposed new design curve relating the N-value from the Texas Cone Penetrometer Test to the angle of internal shearing resistance for cohesionless soils. It is recommended that these new design curves be implemented into the Texas Foundation Exploration and Design Manual. Implementation of these design curves should be limited to those soils possessing physical properties which are the same as the soils tested during this study.

ACKNOWLEDGMENTS

The writers gratefully acknowledge the support and assistance of the State Department of Highways and Public Transportation (SDHPT), and the Department of Transportation, Federal Highway Administration (FHWA), for their cooperative sponsorship which made the research possible.

A special note of thanks is extended to Mr. Horace E. Hoy of the Bridge Division, D-5, SDHPT, who was the study contact individual. Mr. Hoy was instrumental in coordinating many of the research activities and he very willingly provided load test and other data from his files whenever asked. The writers are especially grateful to Messrs. Robert E. Long, Charles L. McCulloch, and Bobby Wade of the District 17 laboratory, and Messrs. G. P. Berthelot, Jr., and Franklin Zaruba of the Houston Urban Project laboratory, for their spirit of cooperation and willingness to assist and provide whatever resources were necessary for field operations.

The contributions of research assistants Manaf M. Hamoudi and George D. Cozart are sincerely appreciated.

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INTRODUCTION

Present status of the problem - Soil soundings are used to measure the in situ resistance of a soil against dynamic penetration of a standard device. According to Wu (22)*, this resistance usually gives some indication of the strength and compressibility of the soil. Besides providing qualitative information for a subsoil, soundings can often be correlated with significant physical properties such as unit weight and shear strength.

In the United States the most widely used dynamic penetration test is the Standard Penetration Test (SPT). The results of the SPT can usually be correlated in a general way to the pertinent physical properties of sand. Meigh and Nixon (11) have reported the results of various types of in situ tests at several sites and have concluded that the SPT gives a reasonable, if not somewhat conservative, estimate of the allowable bearing capacity of fine sands. A relationship between the N-value and the angle of shearing resistance, ϕ', which has become widely used in foundation design procedures in sands is reported in the text by Peck, Hanson, and Thornburn (14). A correlation between the SPT N-value and the unconfined compressive strength of cohesive soils has been reported by other researchers (15, 17, 20).

The State of Texas currently uses a sounding test similar to the SPT for investigation of foundation materials encountered in bridge foundation exploration work. The Texas Cone Penetrometer (TCP) Test

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*Numbers in parentheses refer to the references listed in Appendix I.

2. To develop an improved correlation between the Texas Cone Penetrometer N-value and the drained shear strength of cohesionless soils.
3. To attempt the development of a correlation between the Texas Cone Penetrometer N-value and unit side friction and unit point bearing for driven and bored piles.

PENETROMETER CORRELATIONS FOR COHESIVE SOILS

During the period from September 1973 to August 1974, initial correlations were developed between the Texas Cone Penetrometer Test N-value and the unconsolidated-undrained shear strength of cohesive soils. A reasonably good correlation was established between the unconsolidated-undrained shear strength, c_u, and penetration resistance N-values, for homogeneous CH soils, silty CL soils, and sandy CL soils. The field investigation included seven borings taken at four different sites where the Cone Penetrometer Test was conducted and undisturbed soil samples were obtained. The Texas Triaxial Test and the ASTM Triaxial Test were used in the laboratory investigation to determine soil shear strength. The results of the 1973-74 phase of the study are reported in TTI Report 10-1 (9).

During the period from September 1975 to August 1976 soil samples and N-values from one additional site were obtained. These data are reported in detail in this section on cohesive soils. All laboratory and field test data are presented either in this section or in Appendix III. The correlations shown in this section are based on the combined data from all test sites.

Test Site - The 1975-76 test site was located at the SH87 overcrossing of the Intracoastal Canal south of Port Arthur, Texas. At this location undisturbed cohesive samples were obtained and penetration tests were conducted at corresponding depths. Samples were recovered using the equipment described in TTI Report 10-1 (9).

The Port Arthur test site is located within the outcrop of the Beaumont clay formation. The formation consists of poorly bedded plastic clay interbedded with silt and sand seams and some more or less continuous sand layers (16). The clays are overconsolidated by desiccation. Structurally, the clay is jointed and frequently contains slickensides created by nonuniform shrinkage and expansion. The predominant clay mineral is calcium montmorillonite, and the non-clay minerals are quartz and feldspar (13).

Field Investigation - The field investigation was conducted by a Texas State Department of Highways and Public Transportation soil investigation team under the direction of TTI personnel. Standard practices of field investigation as described in the Texas Foundation Exploration and Design Manual (3) were followed throughout the investigation. Samples were taken and penetration tests were performed continuously in adjacent bore holes.

The purposes of the field investigation were to:

1. Establish the location of the ground water table.
2. Obtain a soil description by visual inspection of samples.
3. Obtain Texas Cone Penetrometer Test N-values.
4. Obtain undisturbed samples for laboratory testing.

Fig. 2 shows the location of the ground water table, the soil description and the penetration test N-values for the Port Arthur test site.

Laboratory Investigation - The purpose of the laboratory investigation was to determine the unconsolidated-undrained shear strength of the undisturbed samples and to classify these samples according to the Unified Soil Classification System. Two types of test were used to determine soil shear strength. The shear strength was determined by the Texas Triaxial Test (TAT) and the ASTM Triaxial Test 2850-7 (ASTM). The Texas Triaxial Test was the primary means of determining the

DEPTH FEET	SOIL SYMBOL	DESCRIPTION OF STRATUM	TEXAS CONE PENETROMETER N-VALUE blows-per-foot
10		Black very soft silty clay with organics G.w.t. With grass roots 6'-8'	
		Dark gray very soft silty clay	
20		Tan and gray silty clay with small amount of sand at 20'	5 13
30		Clayey silt	10
40		Stiff tan and light gray silty clay	15 17
		Stiff dark gray clay	13
50 60		Plastic dark gray silty clay with shells 48'-52' with calcareous nodules 52'-60' fissured and slickensided 57'-62'	13 13 19
66		Gray and tan fine sandy clay	17

Fig. 2. BORING LOG OF PORT ARTHUR TEST SITE

unconsolidated-undrained shear strength of the samples tested. A confining pressure approximately equal to the effective overburden pressure that existed on the sample in situ was used for both tests. The total unit weight and natural moisture content were also determined for all samples.

A diagram of the Texas Triaxial Test apparatus is shown in Fig. 3. The apparatus includes a rubber membrane 0.051 in. (1.3 mm) thick fitted to a lightweight stainless steel cylinder. The sample is subjected to an air pressure applied between the cylinder and the membrane. A loading rate of 0.135 in. (3.429 mm) per minute was used to satisfactorily achieve the undrained condition. This is the same loading rate used during the 1973-74 phase of the study.

The ASTM testing apparatus is shown diagrammatically in Fig. 4. The apparatus includes a 0.012 in. (0.30 mm) thick rubber membrane that completely seals the sample. The sealed sample is enclosed in a cell where it is subjected to air pressure. A confining pressure equal to the effective overburden pressure that existed on the sample in situ was used. The ASTM Triaxial test was conducted on selected samples for purposes of comparing results. Samples tested by the ASTM method were paired with samples tested by the TAT method. The samples compared had the same Unified Soil Classification.

The sample testing in both the TAT and the ASTM procedures was performed using the same motorized press assembly. The same loading rate was used in all testing. Simultaneous readings of load and deformation were taken at intervals of 0.01 in. (.254 mm) deformation until the sample failed.

The soils were classified by the Unified Soil Classification System.

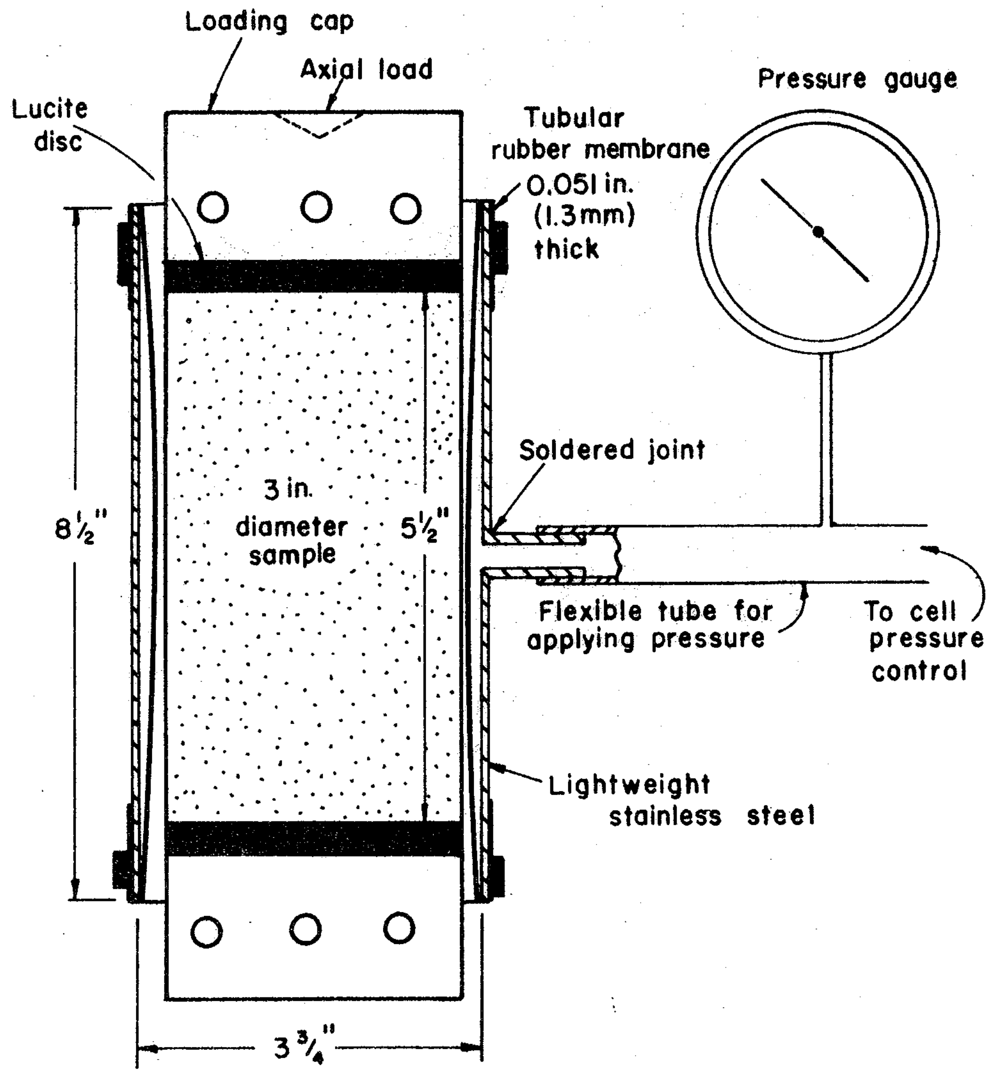


FIG. 3 — DIAGRAMMATIC LAYOUT OF THE TEXAS TRIAXIAL TEST (1.0in.= 25.4mm)

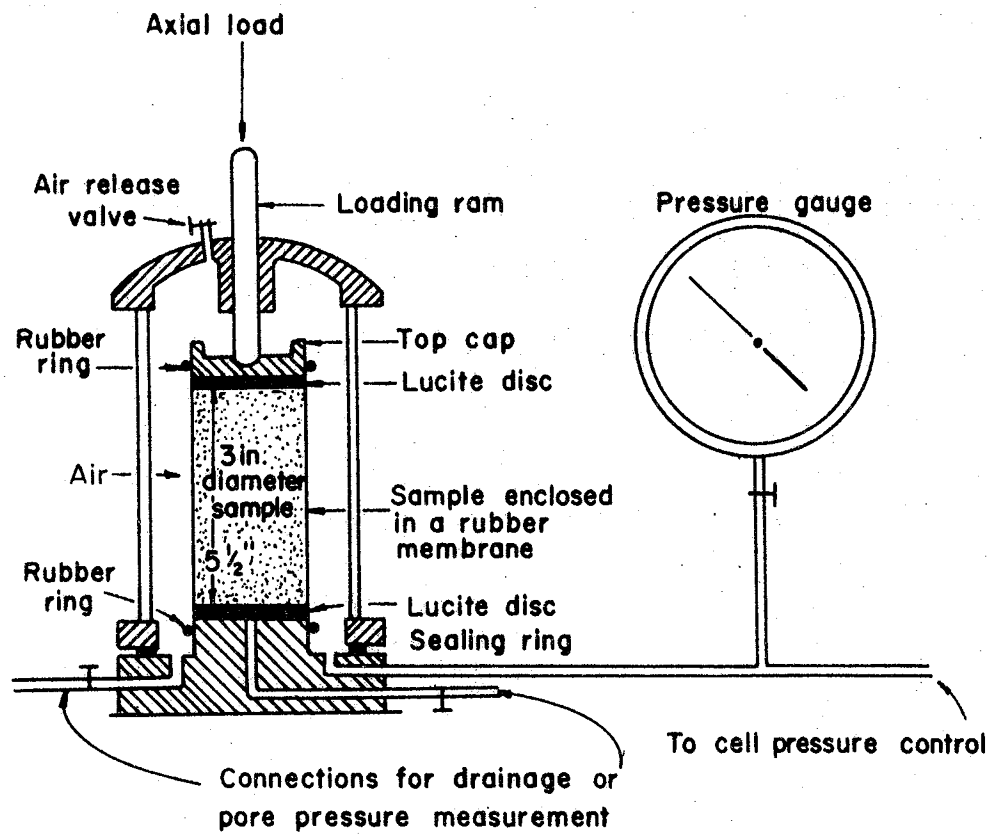


FIG.4 – DIAGRAMMATIC LAYOUT OF THE ASTM TRIAXIAL TEST (1.0in.= 25.4mm)

Standard laboratory equipment was used to perform the tests. The laboratory tests included:

1. Percent passing #200 sieve.
2. Liquid limit.
3. Plastic limit.
4. Plasticity index.

The moisture content of each sample before and after shear strength testing was determined. The total unit weight of each sample was also determined. A summary of all laboratory tests conducted on the Port Arthur samples is given in Appendix III.

Analysis of Test Results and Development of Correlations - After the completion of all laboratory tests the results were grouped according to type of shear strength test and soil classification. Table 1 summarizes the results of the laboratory tests conducted for the Port Arthur test site. The penetration test values are also tabulated to facilitate the correlation of unconsolidated-undrained shear strength, c_u, with penetration test N-values. Table 2 summarizes the same information for the test sites reported in TTI Report 10-1 (9). The test type given in Tables 1 and 2 indicate the test used to determine c_u. The soil classification given in the tables was determined by the Unified Soil Classification System. The N-values shown indicate the in situ resistance to penetration, in blows per foot, for the Texas Cone Penetrometer. The information in Tables 1 and 2 was used to develop all of the correlations in this section.

The values of c_u, expressed in tons per square foot, were computed for the Texas Triaxial Test using the following equation:

Table I- TEST DATA FOR THE PORT ARTHUR TEST SITE				
SAMPLE NUMBER	SOIL CLASSIFICATION	N-VALUE (blows-per-foot)	SHEAR STRENGTH(tsf)	
			TAT	ASTM
4	CL-Si	5	1.38	
5	CL-Si	7		1.18
8	CH-H	12	1.34	
10	CL-Si	12	1.62	
11	CH-H	15	1.51	
12	CH-H	16		1.03
15	CH-H	16	1.76	
16	CH-H	16		0.99
18	CH-H	15	1.75	
20	CH-H	13	1.81	
21	CH-H	13		0.98
23	CL-Si	13	1.91	
24	CL-Si	13	1.49	
25	CH-H	13		0.94
26	CH-H	13	1.84	
27	CH-H	13	2.38	
31	CH-H	16	2.34	
35	CH-H	19	2.44	
39	CH-H	17		1.13
40	CH-H	17	2.81	

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 2- TEST DATA FROM THE TTI REPORT 10-1 TEST SITES				
SAMPLE NUMBER	SOIL CLASSIFICATION	N-VALUE (blows-per-foot)	SHEAR STRENGTH (tsf)	
			TAT	ASTM
A-3	CH-H	36	4.54	
C-4	CH-H	32	3.17	
A-8	CH-H	22	2.82	
A-9	CH-H	18	2.32	
A-12	CL-Si	24	2.31	
A-13	CH-H	12	1.47	
A-14	CL-Si	28		0.98
A-15	CH-H	18		1.51
A-16	CH-H	18	2.21	
A-19	CH-H	14	1.45	
A-22	CH-H	12	1.25	
A-23	CH-H	12	0.74	
B-6	CL-Sa	26	2.03	
B-8	CL-Si	28	2.17	
B-9	CL-Si	32	3.27	
B-10	CL-Sa	30	3.60	
B-11	CL-Si	28	3.67	

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 2- (CONTINUED) TEST DATA FROM THE TTI REPORT 10-1 TEST SITES				
SAMPLE NUMBER	SOIL CLASSIFICATION	N-VALUE (blows-per-foot)	SHEAR STRENGTH(tsf)	
			TAT	ASTM
B-12	CL-Si	32		1.71
B-13	CL-Si	28	2.99	
B-15	CL-Si	26	2.82	
B-16	CL-Si	24		1.08
B-19	CL-Si	18	2.36	
B-30	CH-H	28	2.68	
B-33	CL-Si	28	2.09	
B-39	CH-H	32		1.33
B-40	CH-H	32	2.47	
B-43	CH-H	30		1.62
C-1	CH-H	10	1.78	
C-2	CL-Sa	40		2.43
C-3	CL-Sa	40	4.38	
C-5	CL-Sa	34	3.86	
C-6	CH-H	16	1.99	
C-8	CH-H	20		1.63
C-9	CH-H	18	2.05	
C-10	CH-H	18		1.50
C-12	CL-Sa	24	1.98	
C-13	CL-Sa	24		1.24
C-16	CL-Si	22	2.41	

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 2- (CONTINUED) TEST DATA FROM THE TTI REPORT 10-1 TEST SITES				
SAMPLE NUMBER	SOIL CLASSIFICATION	N-VALUE (blows-per-foot)	SHEAR STRENGTH(tsf)	
			TAT	ASTM
C-18	CL-Sa	22		1.50
C-19	CL-Sa	18	2.72	
C-22	CL-Si	30	2.42	
C-24	CL-Si	32		1.53
C-24	CL-Si	38	4.76	
C-30	CL-Sa	30	4.48	
C-32	CL-Sa	44	3.04	
C-33	CL-Sa	44		2.19
D-1	CH-H	10	1.03	
D-2	CH-H	22		1.03
D-3	CH-H	18	1.80	
D-7	CH-H	24	1.92	
D-9	CL-Sa	22	1.59	
D-10	CL-Sa	22		1.05
D-11	CL-Sa	32	2.50	
D-13	CL-Sa	32		1.95
D-14	CL-Sa	26	3.36	
D-17	CL-Sa	22	3.56	
D-19	CL-Sa	28		1.39
D-24	CL-Sa	46	2.47	

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

$$c_u = \left[\left(\frac{P_m}{A_c}\right) - \sigma_c \right] \times 0.5 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where P_m = the maximum observed load, i.e., the sum of the vertical load induced by the confining pressure and the applied vertical load in tons; A_c = the corrected area in square feet; and σ_c = the confining pressure in tons per square foot.

The values of c_u, expressed in tons per square foot, were computed for the ASTM Triaxial Test using the following equation:

$$c_u = \left(\frac{P_v}{A_c}\right) \times 0.5 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where P_v = deviator stress in tons; and A_c = the corrected area in square feet.

The difference between Eqs. 1 and 2 is due to the initial state of stress upon confinement. The initial state of stress, in the Texas Triaxial Test, is anisotropic. The initial state of stress, in the ASTM Triaxial Test, is isotropic.

The Port Arthur soils tested included only two classifications. The first was found to be homogeneous CH materials (i.e. clays of high plasticity) by the Unified Soil Classification System. These soils contained no secondary structures and hereafter will be referred to as homogeneous CH soils or simply CH-H. The second classification was the CL materials (i.e. clays of low plasticity). These soils contained some silt and were categorized silty CL or CL-Si. The silty CL soils are those clays with less than 20% retained on the No. 200 sieve and not containing sand or silt seams.

Two other soil types were included in TTI Report 10-1 (9). These soils were sandy CL or CL-Sa, and CH soils with secondary structure

CH-W. The sandy CL soils are those clays that contain more than 20% retained on the No. 200 sieve and do not contain sand or silt seams. None of the soils tested at the Port Arthur site fell into these classifications. Therefore, new correlations for the CL-Sa and the CH-W soils are not included in this section.

Shown in Fig. 5 is a plot of unconsolidated-undrained shear strength, c_u, based on the Texas Triaxial Test (TAT), herein referred to as c_u (TAT), and resistance to penetration of the Texas Cone Penetrometer, in blows per foot, N_{TCP}, for homogeneous CH soils. A least square curve fit was used to develop the constant of proportionality that relates c_u (TAT) and N_{TCP}. The equation developed is:

$$c_u \text{ (TAT)} = 0.11\ N_{TCP} \quad \ldots\ldots\ldots \quad (3)$$

where c_u (TAT) is shear strength expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. Eq. 3 may be used to predict the soil shear strength based on the Texas Triaxial Test if the resistance to penetration, N_{TCP}, is available, and provided that the soil tested is a homogeneous CH soil. In order to predict the shear strength of a homogeneous CH soil based on the ASTM Triaxial Test Fig. 6 should be used. The shear strength equation now becomes:

$$c_u \text{ (ASTM)} = 0.067\ N_{TCP} \quad \ldots\ldots\ldots \quad (4)$$

where c_u (ASTM) is shear strength expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. Eq. 4 may be used to predict the shear strength of a homogeneous CH soil based on ASTM Triaxial Test, if the resistance to penetration, N_{TCP}, is available.

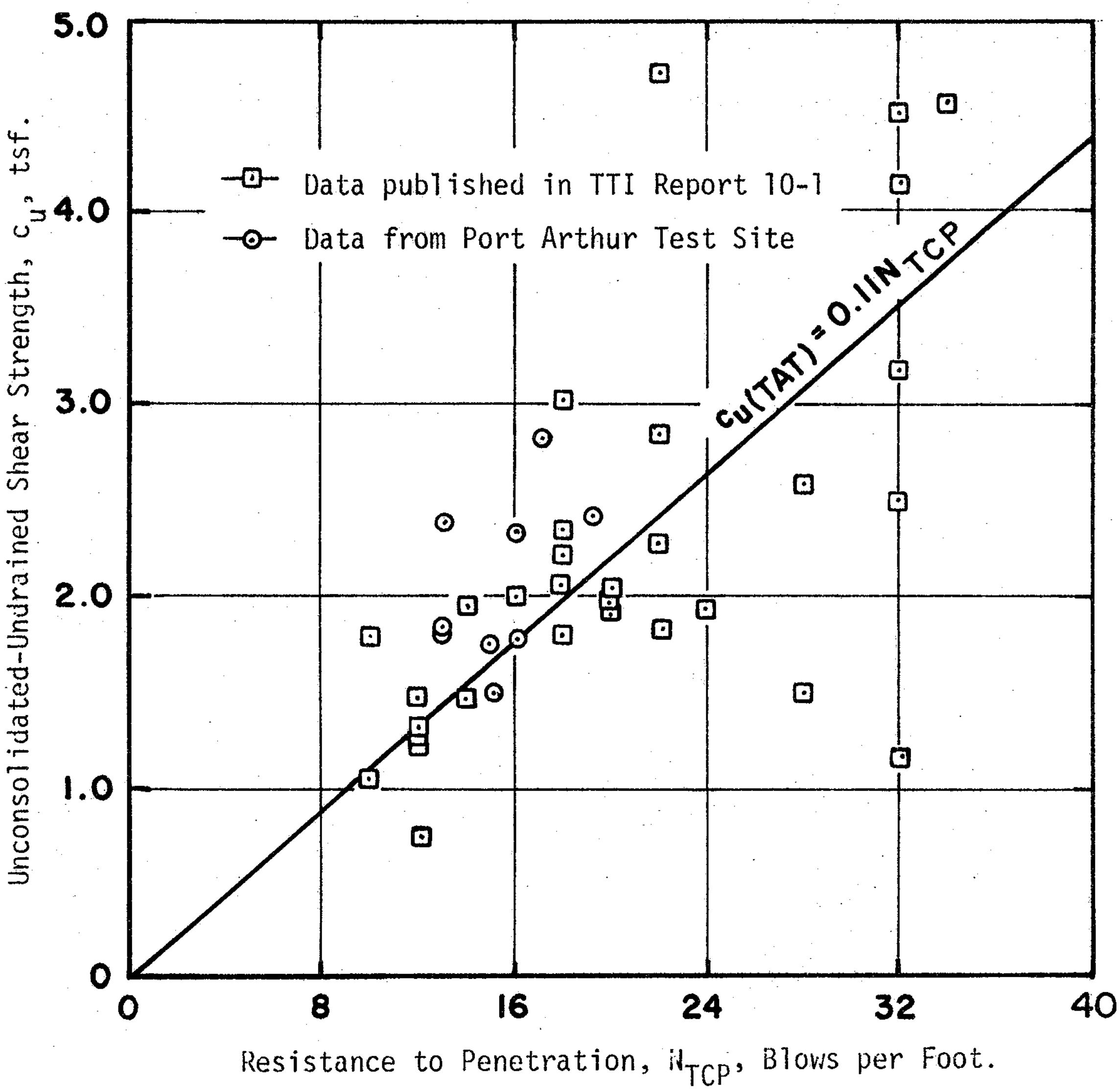


Fig. 5 RELATIONSHIP BETWEEN UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR HOMOGENEOUS CH SOILS - TEXAS TRIAXIAL TEST

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

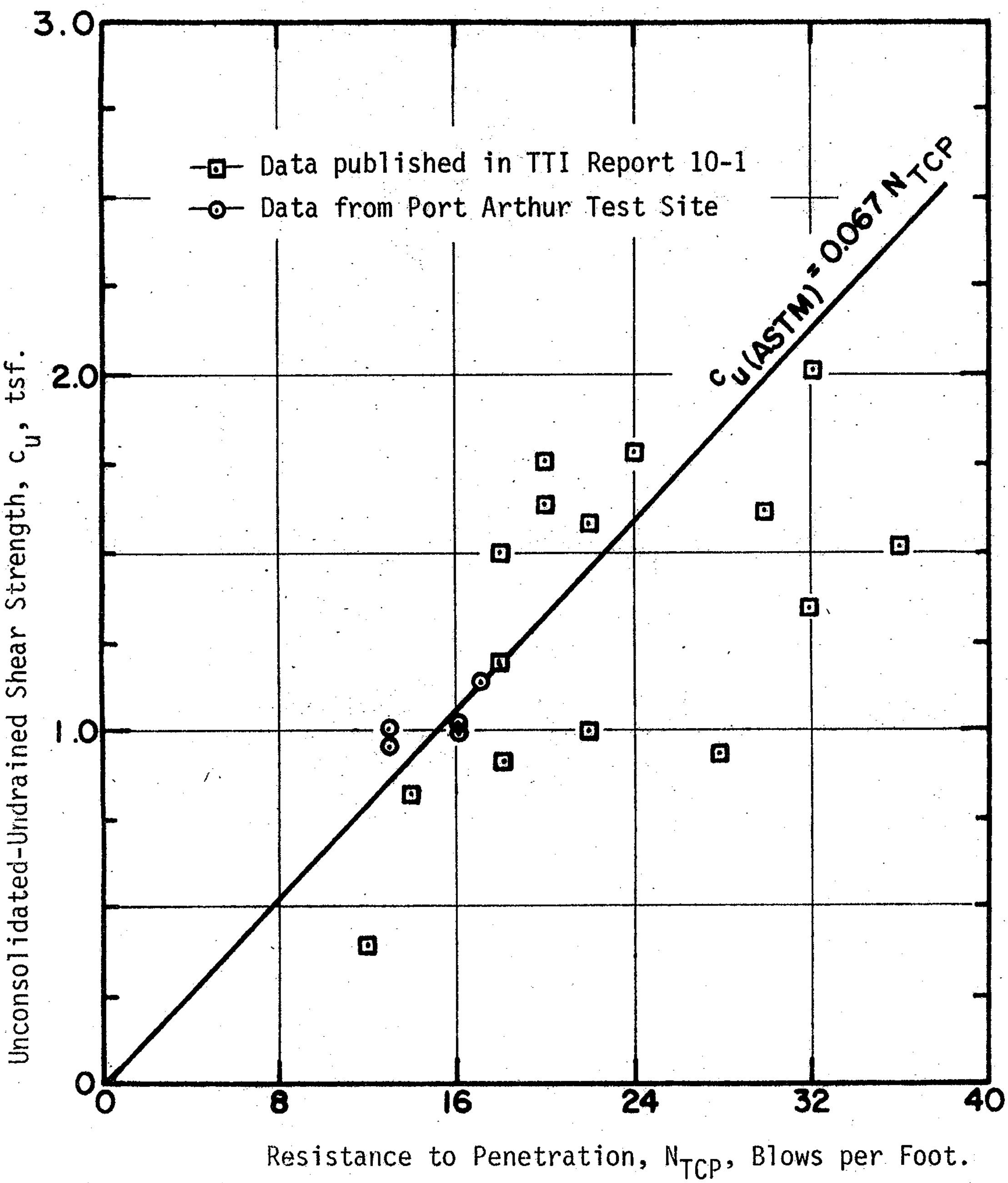


Fig. 6. RELATIONSHIP BETWEEN UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR HOMOGENEOUS CH SOILS-ASTM TRIAXIAL TEST
(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Similarly a correlation was developed for silty CL soils. Fig. 7 presents the result of this correlation. In this case the equation is:

c_u (TAT) = 0.11 N_{TCP} . . . . (5)

where c_u (TAT) is the shear strength, based on the Texas Triaxial Test, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. This correlation makes it possible to predict the shear strength of a silty CL soil based on the Texas Triaxial Test provided that the resistance to penetration, N_{TCP}, is available. Fig. 8 relates c_u (ASTM) for silty CL soils with N_{TCP}. The equation now becomes:

c_u (ASTM) = 0.054 N_{TCP} . . . . (6)

where c_u (ASTM) is the shear strength, based on the ASTM Triaxial Test expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. With this correlation it is possible to predict the shear strength of a silty CL soil based on the ASTM Triaxial Test if penetration test N-values are available.

It can be seen that Eqs. 3 and 5 are identical. Both of these equations are based on the Texas Triaxial Test. These results indicate the possibility of using only one correlation for all cohesive soils. Eqs. 4 and 6 on the other hand are not the same. These equations were based on shear strengths obtained using the ASTM Triaxial Test. Eqs. 4 and 6 indicate a range of shear strengths for a given N_{TCP} value.

It should also be noted that the shear strengths predicted by the Texas Triaxial Test are higher than those predicted by the ASTM Triaxial Test. Fig. 9 shows a plot of c_u (ASTM) versus c_u (TAT) for all of the soils listed in Tables 1 and 2. The samples compared were paired

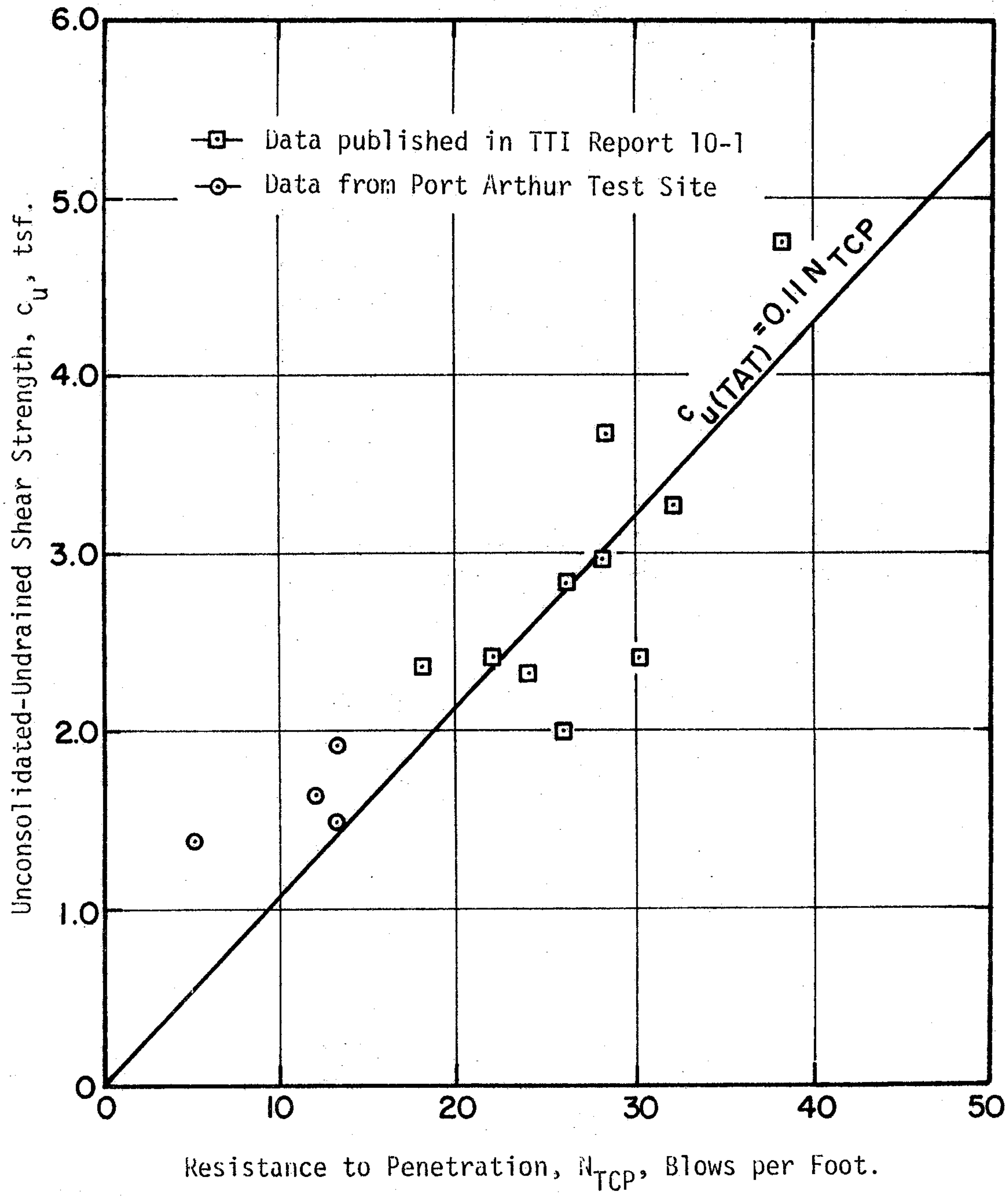


Fig. 7 RELATIONSHIP BETWEEN UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR SILTY CL SOILS - TEXAS TRIAXIAL TEST

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

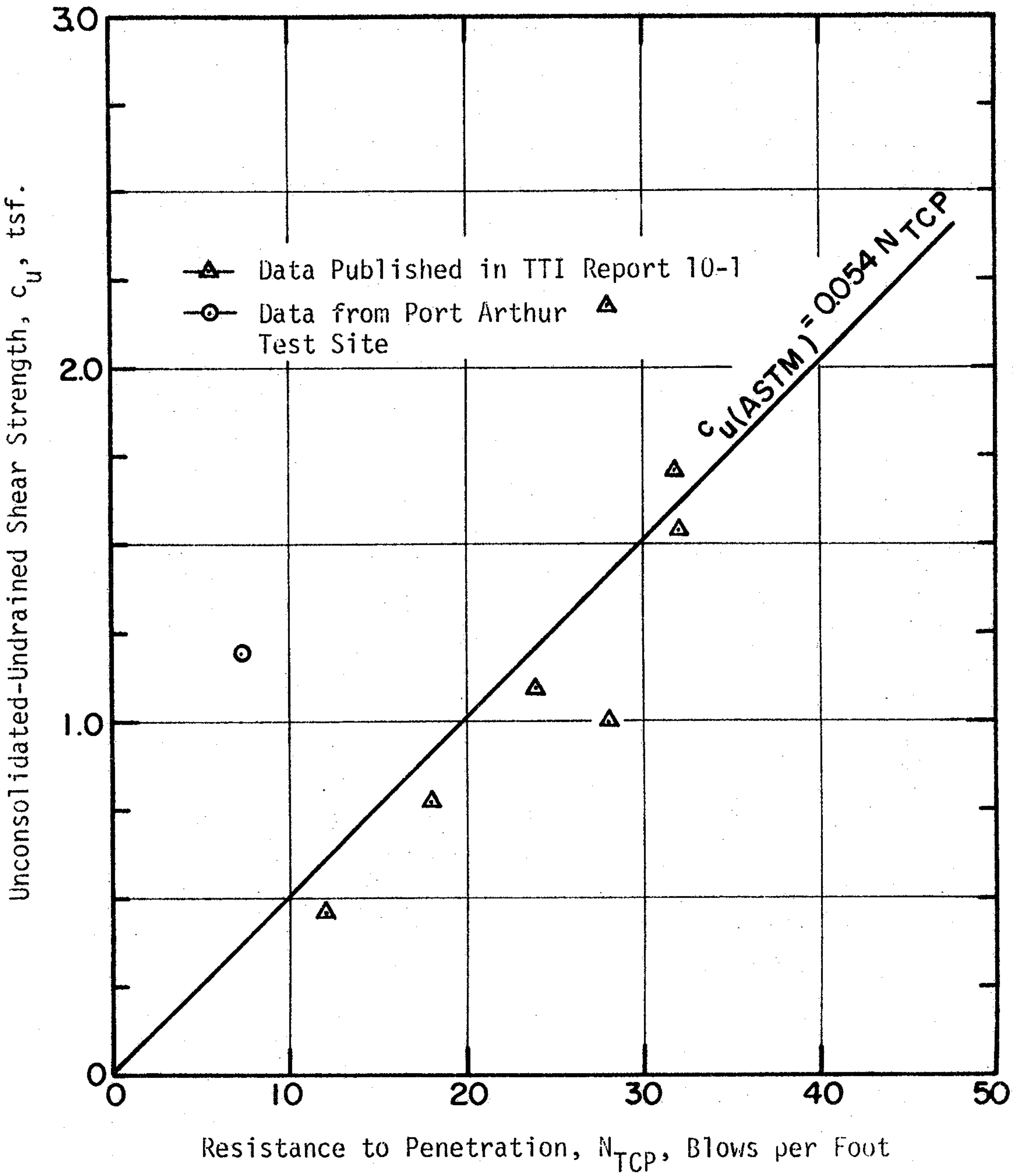


Fig. 8 RELATIONSHIP BETWEEN UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR SILTY CL SOILS - ASTM TRIAXIAL TEST
(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

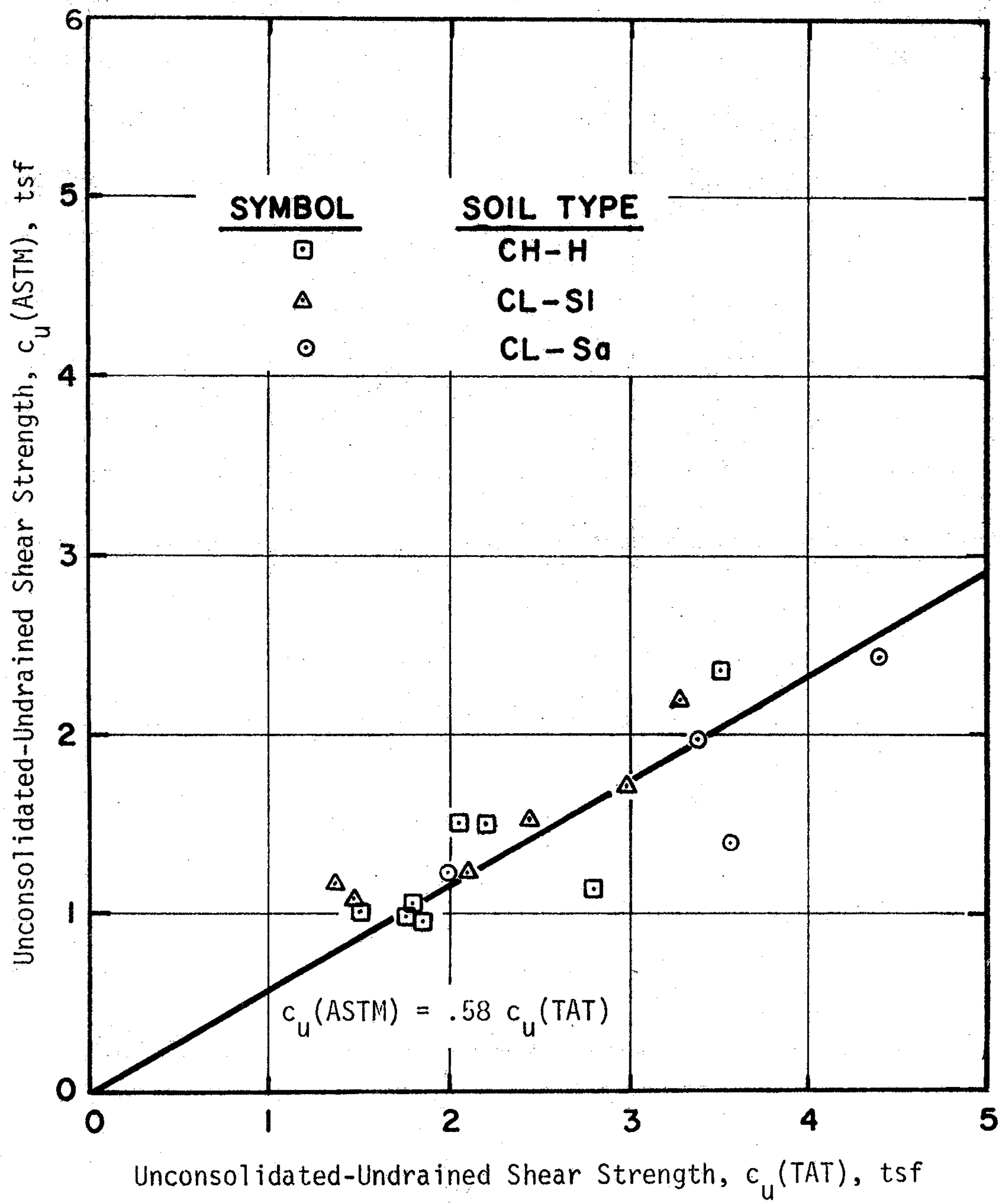


Fig. 9 RELATIONSHIP BETWEEN TEXAS TRIAXIAL AND ASTM TRIAXIAL SHEAR STRENGTH.
(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

samples and care was exercised to ensure that the soil in each pair had the same properties. The equation relating the shear strength tests is:

$$c_u \text{ (ASTM)} = 0.58\ c_u \text{ (TAT)} \quad \ldots \quad (7)$$

where c_u (ASTM) is the shear strength as determined by the ASTM Triaxial Test, and c_u (TAT) is the shear strength as determined by the Texas Triaxial Test. c_u (ASTM) and c_u (TAT) must be expressed in the same units. Probable reasons for the differences in the shear strength predicted by the ASTM Triaxial Test and the TAT Triaxial Test are explained in detail in TTI Report 10-1 (9).

Other researchers have developed correlations between the unconsolidated-undrained shear strength of cohesive soils and the Standard Penetration Test N-value (17, 20). It is possible to compare data from this study with the correlations developed for the SPT. Touma and Reese (18) have developed a relationship for cohesive soils between the N-values obtained by the Texas Cone Penetrometer Test, N_{TCP}, and N-values obtained by the Standard Penetration Test, N_{SPT}. For clay soils the relationship is:

$$N_{SPT} = 0.7\ N_{TCP} \quad \ldots \quad (8)$$

Combining Eq. 8 with the correlation equations developed in this study yields the equations and the plots shown in Fig. 10. The correlations are compared graphically with the results from the other studies (17, 20). The correlations in Fig. 10 compare favorably. It is important to note that the results of other research indicates a single curve is valid for both CH and CL soils. On the other hand, the results of this study indicate a range of shear strength for a given N-value depending

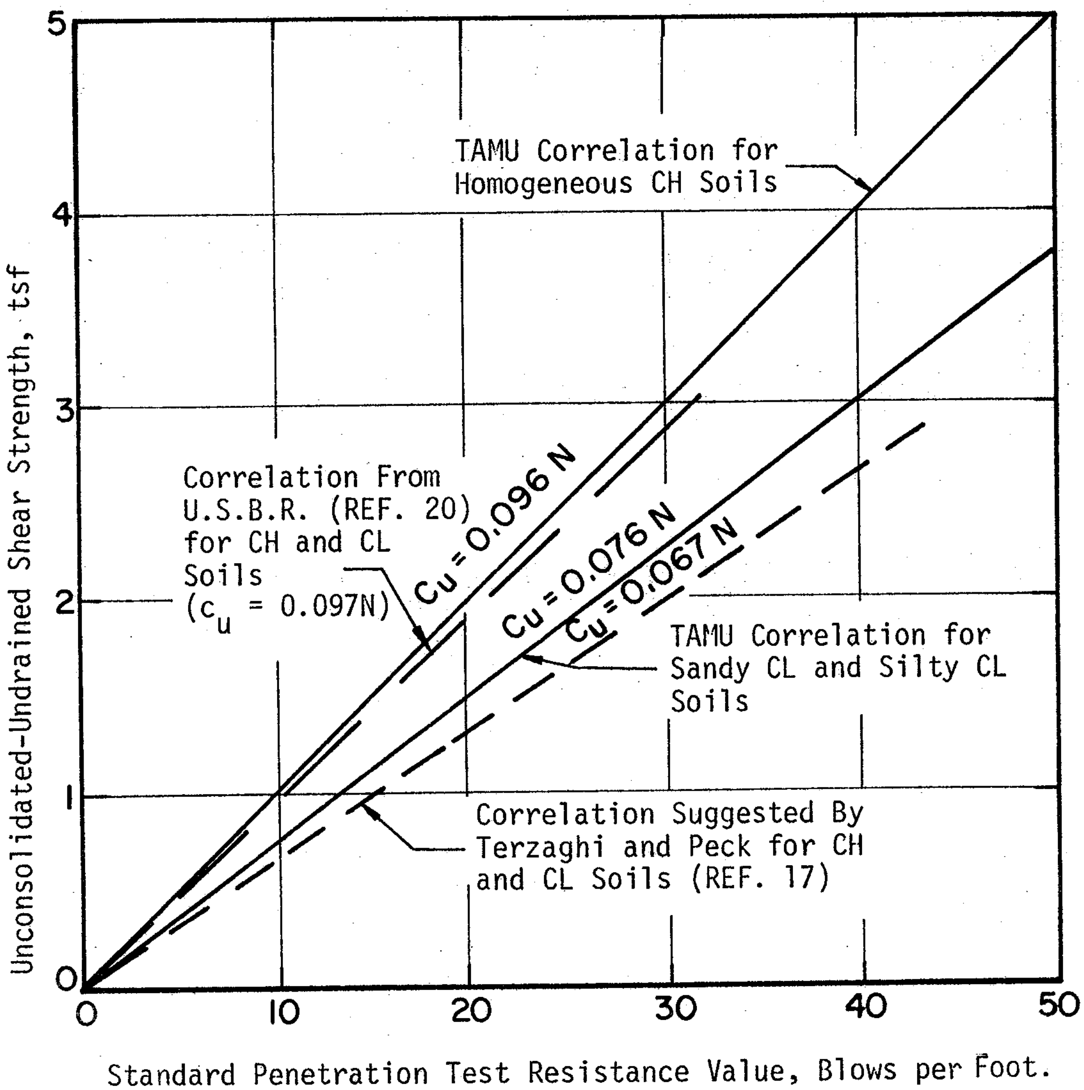


Fig. 10 RELATIONSHIP BETWEEN UNCONSOLIDATED-UNDRAINED SHEAR STRENGTH AND THE STANDARD PENETRATION TEST RESISTANCE VALUE (1 psi = 6.9 kN/m^2 ; 1 ft = .305 m).

on soil type. This range of shear strength is located within an upper and lower bound established by the other researchers (17, 20).

PENETROMETER CORRELATIONS FOR COHESIONLESS SOILS

During the period from September 1974 to August 1975 initial correlations were developed between the Texas Cone Penetrometer Test N-value and drained shear strength, s, as well as several other parameters for cohesionless soils. The field investigation included five test sites and eight borings and the results of the 1974-75 phase of the study are reported in TTI Report 10-2 (6).

During the period from September 1975 to August 1976 soil samples and N-values from one additional test site were obtained. These data are presented in this section on cohesionless soils. All laboratory and field test data are presented either in this section or in Appendix II. The correlations shown in this section are based on the combined data from all test sites.

Test Site. - The 1975-76 test site was located at the Park Road 22 overcrossing of the Intracoastal Canal southeast of Corpus Christi, Texas. At this location undisturbed sand samples were obtained and penetration tests were conducted at corresponding depths. The samples were recovered using the methods and equipment described in TTI Report 10-2 (6). This test site will hereafter be referred to as the Corpus Christi site.

Corpus Christi is located in an area of coastal prairies underlain by Pleistocene river, delta, and shoreline sediments deposited more than 30,000 years ago during one or more interglacial periods. River-fed deltas built gulfward across marine embayments where coastal prairies now occur. A relict shoreline deposit that lies along the main shore of Laguna Madre and Redfish-Aransas Bays marks the position of the youngest

Pleistocene shoreline in the Corpus Christi area (4).

Field Investigation. - The field investigation was conducted by a Texas State Department of Highways and Public Transportation soil investigation team under the direction of TTI personnel. Standard practices of field investigation as described in the Texas Foundation Exploration and Design Manual (3) were followed throughout the investigation. Samples were taken and penetration tests were performed in adjacent bore holes.

The purposes of the field investigation were to:

1. Establish the location of the ground water table.
2. Obtain a soil description by visual inspection of samples.
3. Obtain Texas Cone Penetrometer N-values.
4. Obtain undisturbed samples for laboratory testing.

Fig. 11 shows the location of the ground water table, the soil description, and the penetration test N-values for the Corpus Christi test site.

Undisturbed cohesionless samples were obtained using a small diameter sampling tube. Fig. 12 shows a cross section of the sampling apparatus. The sampler has an area ratio of 9.23 percent. The area ratio is computed as follows:

$$\text{Area Ratio} = \frac{\text{volume of displaced soil}}{\text{volume of soil}} = \frac{D_w^{\,2} - D_e^{\,2}}{D_e^{\,2}}\ (100) \quad . \ . \ . \ . \quad (9)$$

where D_w = outside diameter of sample tube, and D_e = inside diameter of sample tube. The area ratio of the sampler used satisfies the requirement of minimum disturbance as described by Hvorslev (10).

Laboratory Investigation. - The purpose of the laboratory investigation was to determine the drained shear strength of the cohesionless samples and to classify these samples according to the Unified Soil

Depth Feet	Soil Symbol	Description of Stratum	Texas Cone Penetrometer N-Value Blows per foot
		No Recovery	
4'		Some clay from 4'–5'. Loose dark gray sand with small amount of clay and organic and shells.	5
12 1/2'		Very soft dark gray clay with organics.	
15 1/2'		Very loose fine dark gray clay w/org.	41
19'		Green and gray dense clayey sand w/organics. Becomes firm at 25'	53 49
26'		With organics at 26' Loose green silty and with shell at 30'	26 24
32'		Firm brown very fine clayey sand with shells.	44 56
36'			

Fig. 11. BORING LOG OF CORPUS CHRISTI TEST SITE

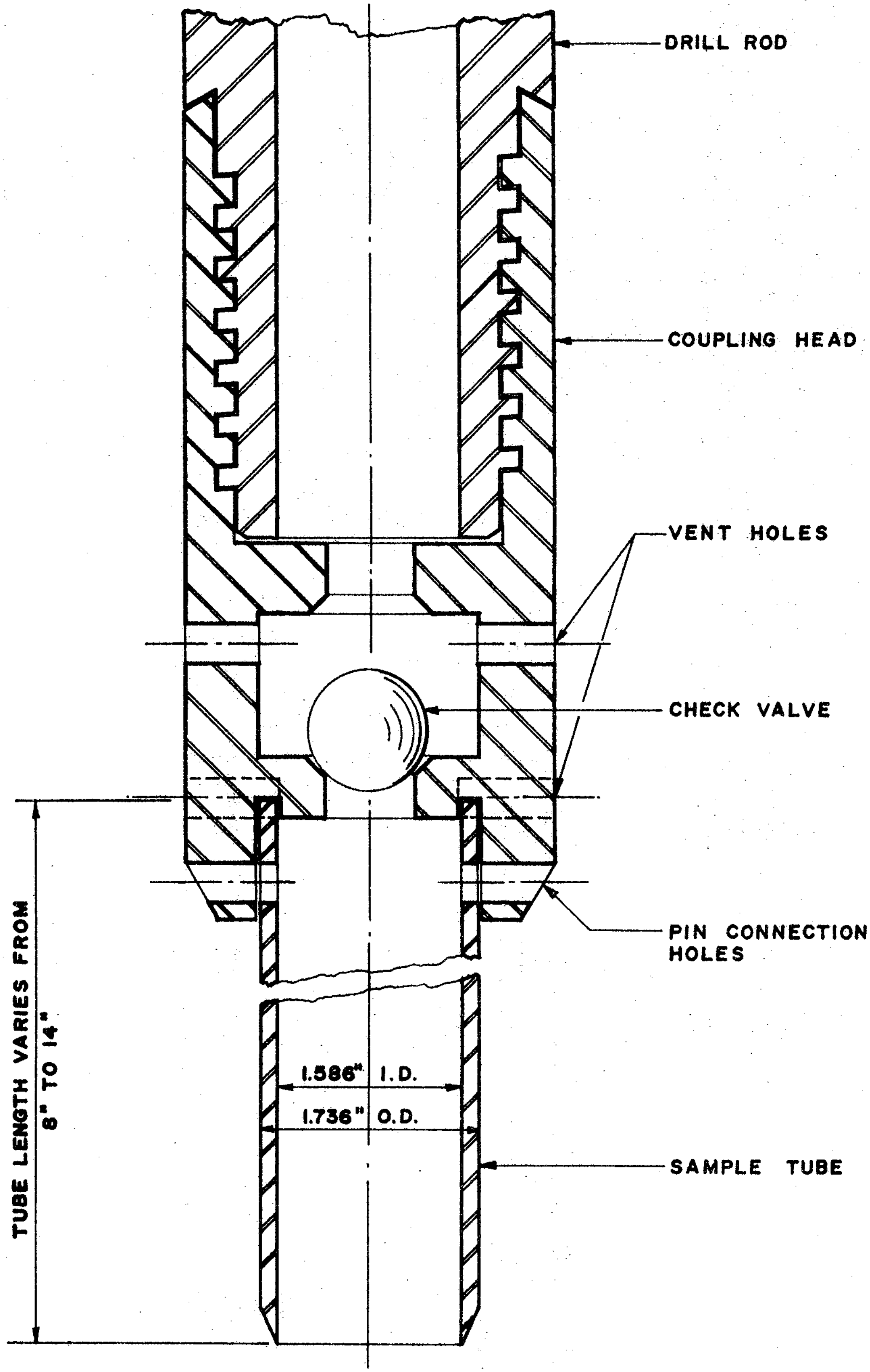


FIG. 12. - CROSS SECTION OF SAMPLING APPARATUS.
(1.0 in = 25.4 mm)

Classification System. The direct shear test was used to determine the effective angle of shearing resistance used in calculating drained shear strength. Mechanical analyses and Atterberg limits were used to classify the soils tested.

Direct shear tests were performed on small diameter samples using the equipment and procedure described in TTI Report 10-2 (6). The samples were extruded, using a hand operated hydraulic jack, directly into the direct shear box using the extrusion device shown in Fig. 13. Before placing the sample into the extruding device, cuttings were removed from both ends of the sample. At this time the total unit weight of the sample was determined. The sample tube was then placed in the extruding device. The direct shear box was placed inverted over the tube complete with bottom plates. The sample was then extruded into the box until the bottom plates made contact with the restraining pins in the base of the shear box. The samples were trimmed using the 0.001 in. thick (.025 mm) trimming device. The box was then removed from the extrustion device and placed upright into the direct shear loading apparatus for testing.

The direct shear box assembly used for testing the samples is shown in Fig. 14. The box uses a 1.58 in. (40.28 mm) diameter sample. The shear box assembly was adapted for use with the Wykeham Farrance equipment used in the Texas A&M Soil Mechanics Laboratory.

The loading assembly used is shown in Fig. 15. A constant speed motor was used to achieve a strain rate of 0.005 in./min (.127 mm/min). The strain rate used for the Corpus Christi samples was the same strain rate used to obtain the data presented in TTI Report 10-2 (6). In most

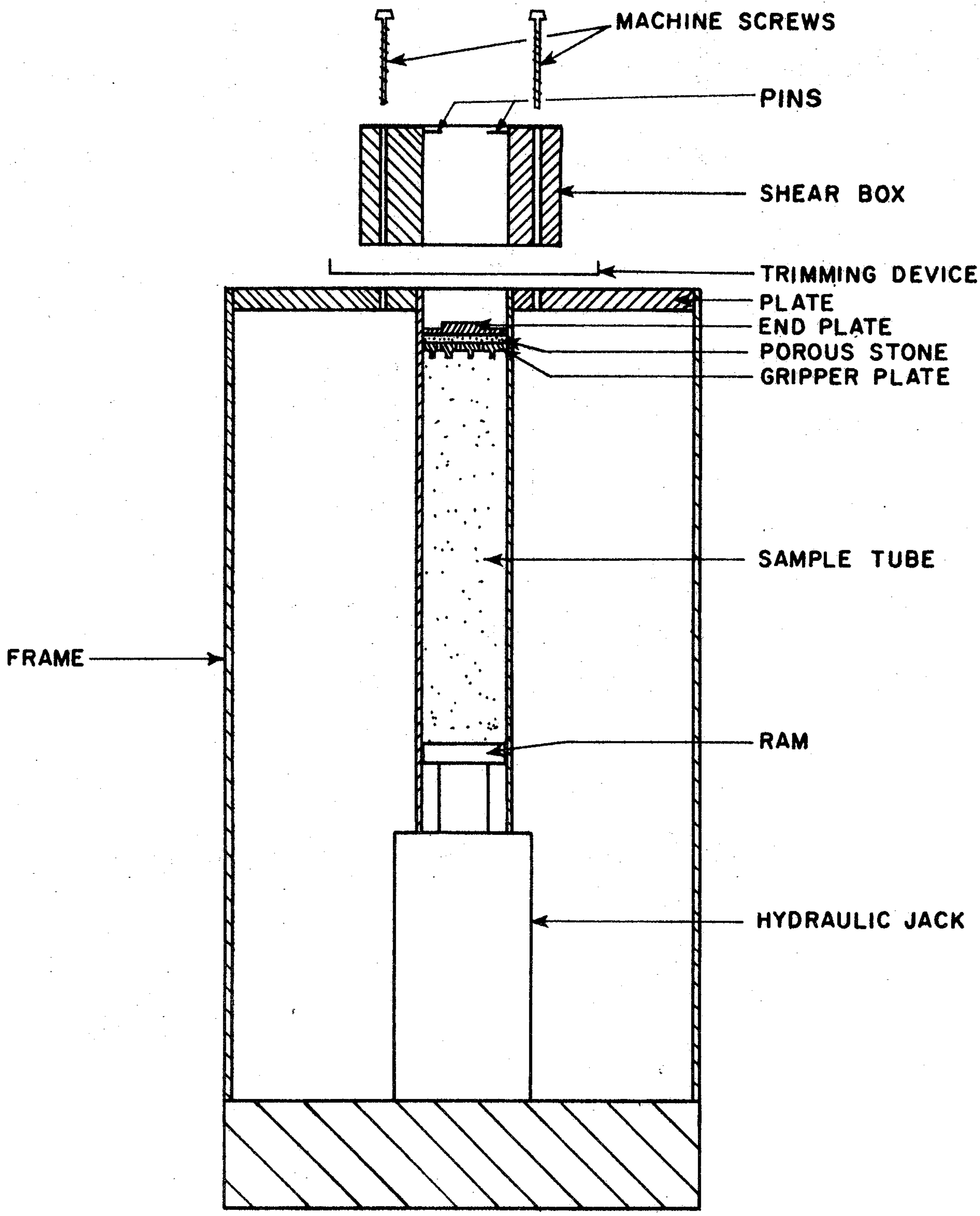


FIG. 13 - CROSS SECTION OF EXTRUSION ASSEMBLY.

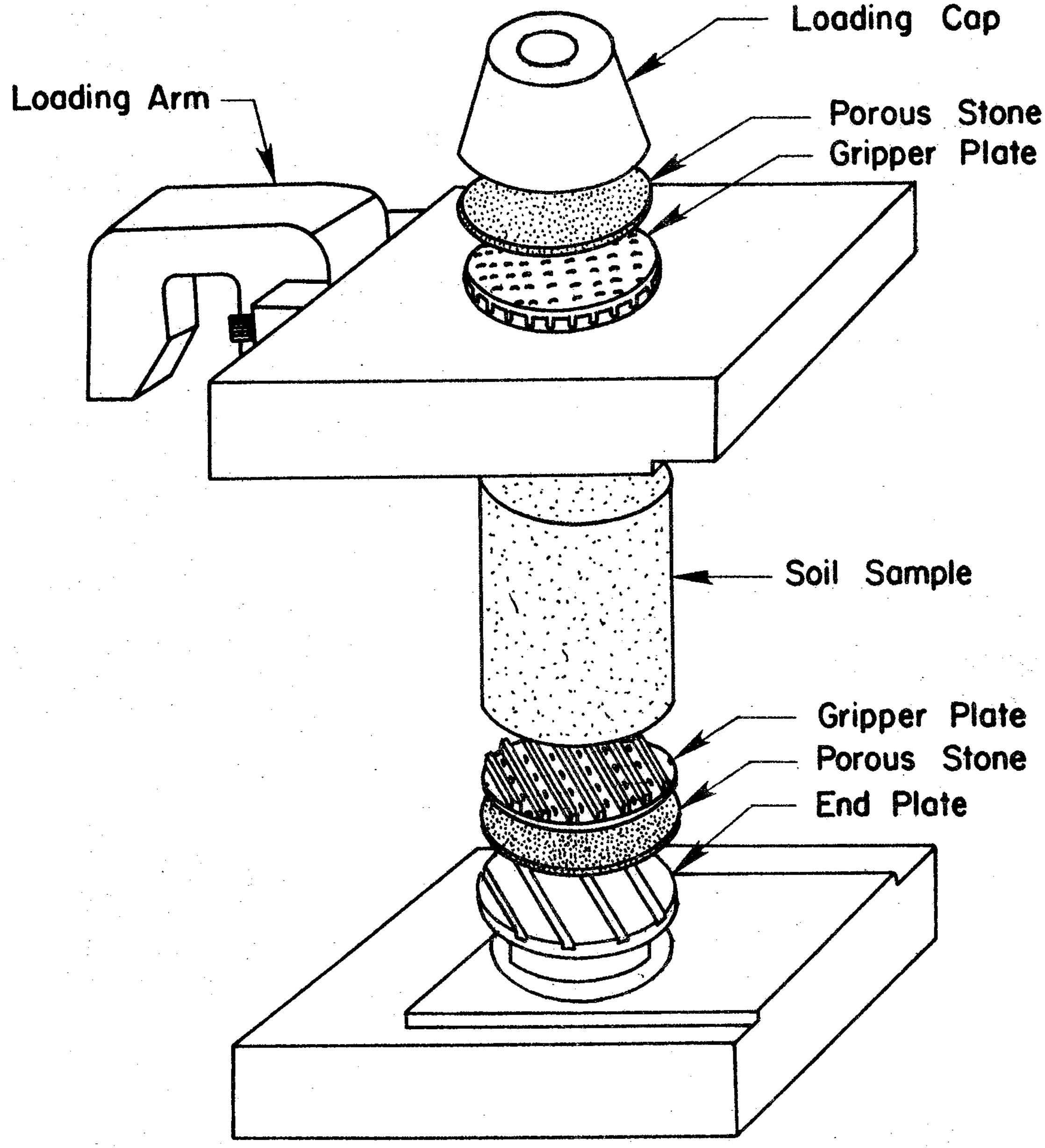
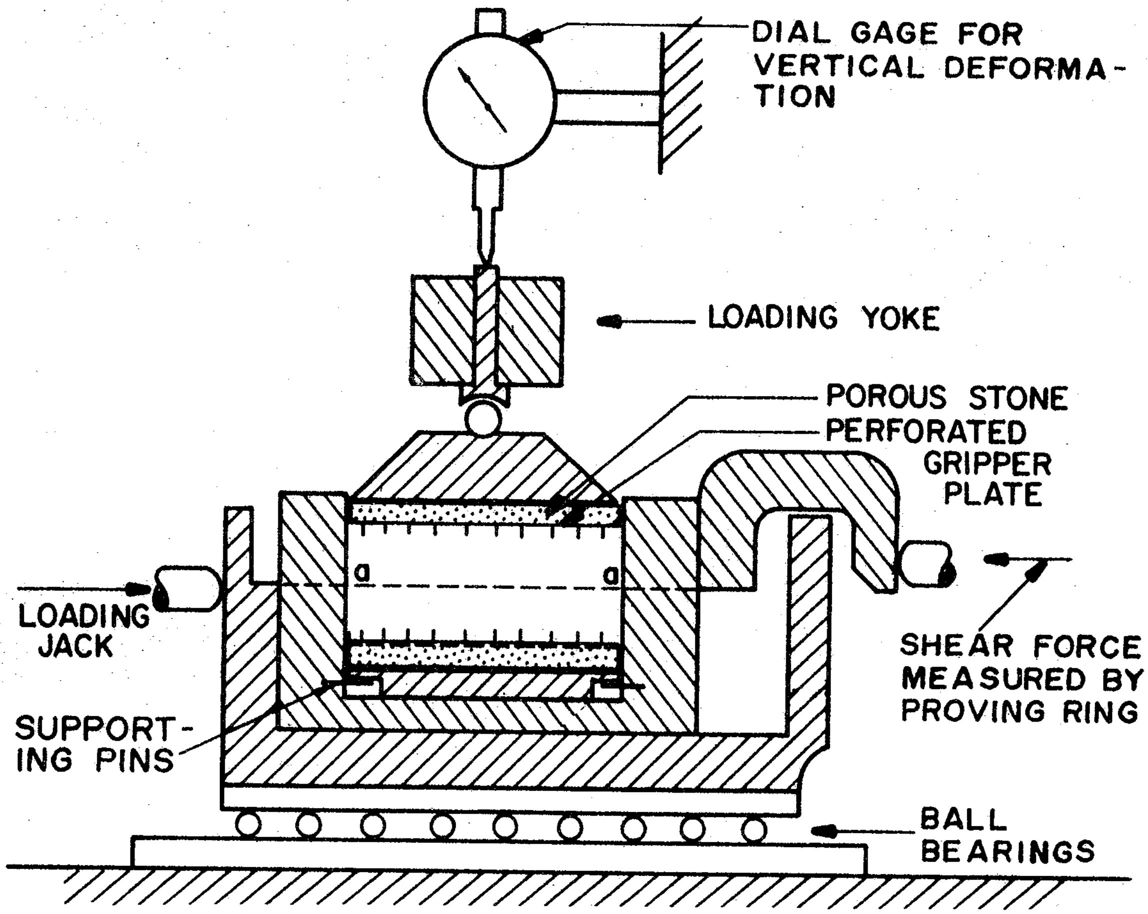
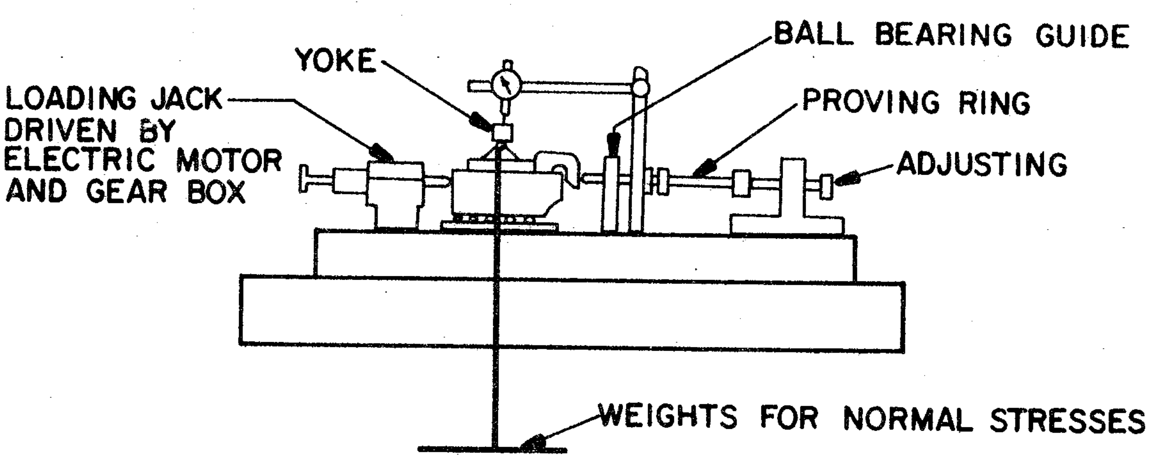


FIG. 14.- SHEAR BOX ASSEMBLY.



SECTION OF SHEAR BOX



LAYOUT OF LOADING SYSTEM

FIG. 15.- LOADING ASSEMBLY FOR DIRECT SHEAR TESTS (AFTER TTI REPORT 10-2)

cases three tests were performed on each tube sample. Normal stresses of 10, 20, and 30 psi (69, 138, and 207 kN/m^2) respectively were used for samples in each tube.

The shear strength of the sample was determined by dividing the maximum force required to shear the sample by the cross sectional area of the sample. The failure envelope was then plotted using the shear stresses at failure and the corresponding normal stresses. The effective angle of shearing resistance, ϕ', is the angle formed by the failure envelope and the horizontal.

The shear strength at depths corresponding to the depths where penetrometer tests were conducted was determined from the general Mohr-Coulomb relationship:

$$s = c' + \sigma_\eta' \tan \phi' \qquad (10)$$

where s = effective shear strength of soil, c' = effective cohesion, σ_η' = effective normal stress, and ϕ' = effective angle of shearing resistance. The cohesion equals zero for drained tests involving cohesionless soils. Therefore, Eq.10 becomes:

$$s = \sigma_\eta' \tan \phi' \qquad (11)$$

Table 3 contains the summary of N-values, the effective angle of shearing resistance, and the drained shear strength for the Corpus Christi test site. Table 4 contains the same information as Table 3 but the data is taken from TTI Report 10-2 (6). Tables 3 and 4 contain the information used to develop Figs. 16 through 23.

The soils were classified using the Unified Soil Classification System. Standard laboratory equipment was used to perform the tests necessary for classification. The laboratory tests needed for

Table 3.-- SUMMARY OF N-VALUES, EFFECTIVE ANGLE OF SHEARING RESISTANCE, DRAINED SHEAR STRENGTH CORPUS CHRISTI TEST SITE

Sample number	N-value Blows per foot				Effective Angle of Shearing Resistance (ϕ)	Shear Strength, S (tsf)
	N_{TCP}	N'_{TCP}	N_{SPT}	N'_{SPT}		
1	5	5	3	3	38.7	0.190
2	2	2	1	1	31.3	0.236
4	41	36	21	18	36.3	0.482
6	53	42	27	21	41.0	0.674
7	49	40	25	20	38.5	0.685
8	26	26	13	13	34.0	0.638
9	24	24	12	12	35.5	0.734
10	44	37	22	19	32.5	0.701
11	56	43	28	22	45.0	1.180

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 4 -- SUMMARY OF N-VALUES, EFFECTIVE ANGLE OF SHEARING RESISTANCE, DRAINED SHEAR STRENGTH TTI REPORT 10-2 TEST SITES

Sample number	N value, Blows per Foot				Effective Angle of Shearing Resistance (ϕ')	Drained Shear Strength, S (tsf)
	N_{TCP}	N'_{TCP}	N_{SPT}	N'_{SPT}		
A-1-2	35	33	18	17	42.0	.411
A-1-3	60	45	30	23	40.0	.450
A-2-1	4	4	2	2	36.5	.212
A-2-2	5	5	3	3	31.5	.209
A-2-3	9	9	5	5	37.5	.307
A-3-1	6	6	3	3	34.5	.187
A-3-2	6	6	3	3	30.0	.199
A-3-3	20	20	10	10	36.5	.323
B-1-9	33	32	17	16	34.0	.433
C-1-13	19	19	9	9	36.0	.442
C-1-18	18	18	9	9	39.0	.637
D-1-5	22	22	11	11	41.0	.855
D-1-6	48	39	24	20	40.0	.961

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 4.-- (Cont.) SUMMARY OF N-VALUES, EFFECTIVE ANGLE OF SHEARING RESISTANCE, DRAINED SHEAR STRENGTH TTI REPORT 10-2 TEST SITES

Sample number	N value, Blows per Foot				Effective Angle of Shearing Resistance (ϕ')	Drained Shear Strength, S (tsf)
	N_{TCP}	N'_{TCP}	N_{SPT}	N'_{SPT}		
D-1-7	33	32	17	16	43.0	1.153
D-1-12	30	30	15	15	37.5	1.278
D-1-19	80	55	40	28	41.0	1.766
D-1-22	68	49	34	25	38.5	1.722
E-1-11	64	47	32	24	39.0	1.183
E-1-12	80	55	40	28	38.0	1.816
E-1-17	74	52	37	26	42.0	2.076

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

classification included:

1. Mechanical grain size analysis.
2. Liquid limit.
3. Plastic limit.
4. Plasticity index.

Other laboratory tests conducted included:

1. Moisture content of sample before and after testing.
2. Total unit weight before testing.

The results of the laboratory tests for all Corpus Christi samples are given in Appendix IV.

Analysis of Test Results and Development of Correlations. - Bowles (2) recommends the use of the following equation for very fine or silty, saturated sand if the measured penetration number, N, is greater than 15:

$$N'_{SPT} = 15 + \frac{1}{2}(N_{SPT} - 15) \quad \ldots \ldots \ldots \quad (12)$$

where N'_{SPT} = adjusted penetration number, and N_{SPT} = measured penetration number. This equation is based on penetration numbers obtained from the Standard Penetration Test (SPT). Eq. 12 was developed based on the assumption that the critical void ratio occurs at approximately N_{SPT} equal to 15, and in fine-grained materials the coefficient of permeability is so low that the change in pore pressure created by the expansion of the soil impedes penetration by the split spoon, thus increasing the penetration number.

Touma and Reese (18) also developed a relationship for cohesionless soils between the Standard Penetration Test N-value and the Texas Cone Penetrometer Test N-value. This relationship indicates that the penetration test N-values obtained by the TCP are twice those obtained for the same soil using the SPT. In equation form this relationship is expressed as follows:

$$N_{TCP} = 2N_{SPT} \quad \ldots\ldots \quad (13)$$

where N_{TCP} = Texas Cone Penetrometer Test N-value, expressed in blows per foot, and N_{SPT} = Standard Penetration Test N-value, expressed in blows per foot. Eq. 13 can be used to establish a value of N_{TCP} equal to 30 at the critical void ratio. If the same relationship indicated by Eq. 12 is applied to the Texas Cone Penetrometer Test, the following equation is developed:

$$N'_{TCP} = 30 + \frac{1}{2}(N_{TCP} - 30) \quad \ldots\ldots \quad (14)$$

where N'_{TCP} = adjusted penetration number, and N_{TCP} = measured penetration number. Eq. 14 is limited to very fine or silty saturated sands with a penetration number N_{TCP} greater than 30. Separate correlations were developed using both the corrected and the uncorrected N-values.

Fig. 16 shows a plot of the drained shear strength, s, versus the corresponding Texas Cone Penetrometer Test N-value, N_{TCP}. The values of N_{TCP} are the uncorrected values measured in the field. Using a least square type of statistical analysis, a constant of proportionality for the two soil parameters was developed. The relationship can be expressed in equation form as follows:

$$s = 0.021\ N_{TCP} \quad \ldots\ldots \quad (15)$$

where s = drained shear strength, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. This correlation applies only to the soil types tested. The soil types include SP, SM, and SP-SM soils. Eq. 15 can be used to determine the

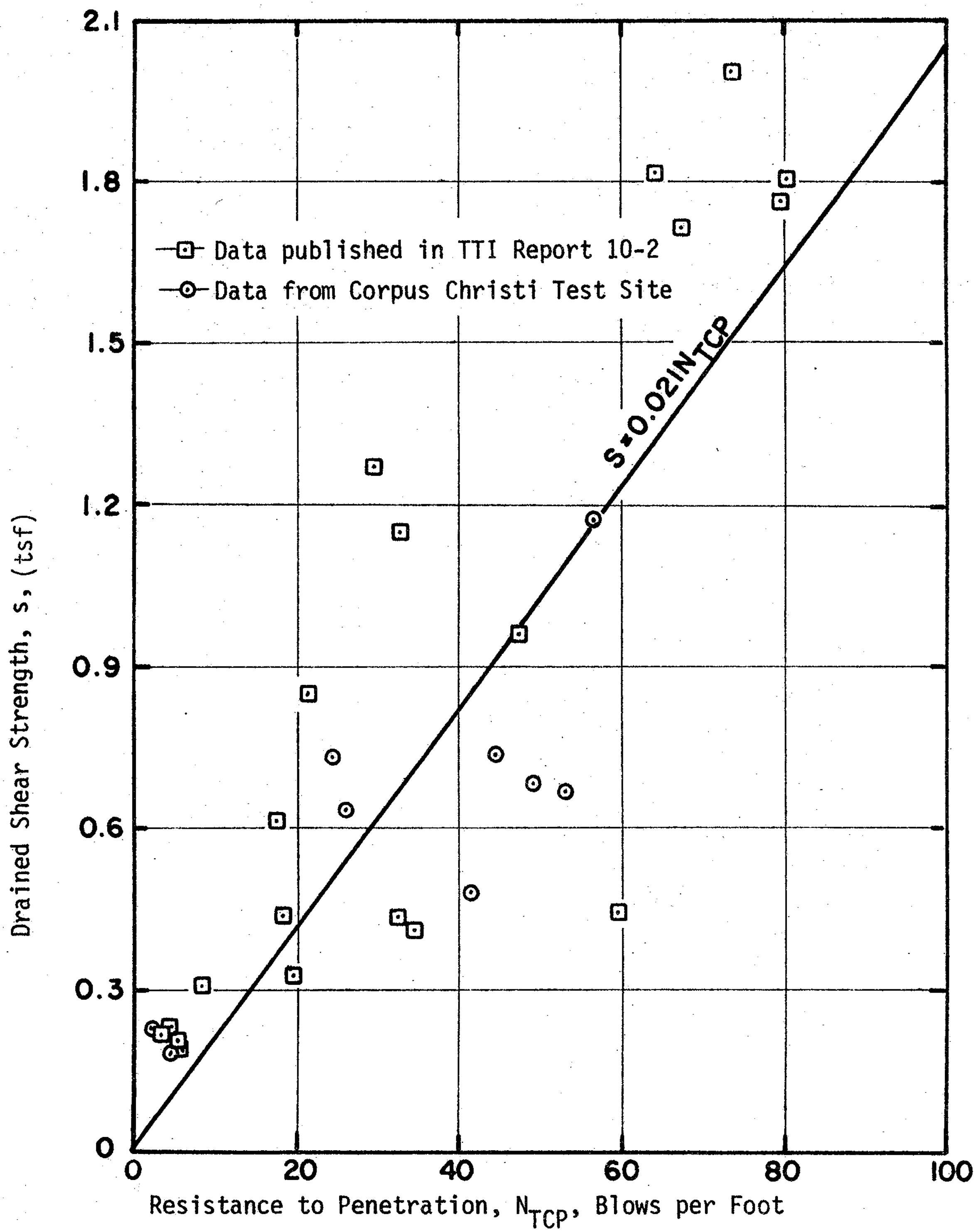


Fig. 16 RELATIONSHIP BETWEEN DRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR SP, SM AND SP-SM SOILS -N_{TCP}

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

drained shear strength of these soil types if N_{TCP} is known. Plotting the values of shear strength, and N'_{TCP} given in Tables 3 and 4 yields the relationship shown in Fig. 17. The relationship between the parameters s, and N'_{TCP} now becomes:

$$s = 0.026\ N'_{TCP} \quad (16)$$

where s = drained shear strength, expressed in tons per square foot, and N'_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. Eq. 16 should only be used with a corrected value of N_{TCP}. It should be noted that when a corrected value of N_{TCP} is used in Eq. 16 the value of s obtained will not differ greatly from the value of s obtained from Eq. 15 using the measured value of N_{TCP}. In coarse sands or nonsaturated sands, the value of s obtained from Eq. 16 will be greater than the value of s obtained from Eq. 15. This indicates that for nonsaturated or coarse sands Eq. 15 is more conservative than Eq. 6 .

A correlation between s and N_{SPT} was also developed. The values of N_{SPT} were determined using Eq. 13 to convert the measured values of N_{TCP} into the appropriate values of N_{SPT}. Fig. 18 is the plot of s versus N_{SPT}. The realtionship between s and N_{SPT} can be expressed in equation form as follows:

$$s = 0.041\ N_{SPT} \quad (17)$$

where s = drained shear strength, expressed in tons per square foot, and N_{SPT} = Standard Penetration Test N-value, expressed in blows per foot. If Eq. 12 is used to correct the values of N_{SPT} where the soil conditions warrant, the following equation is developed:

$$s = .052\ N'_{SPT} \quad (18)$$

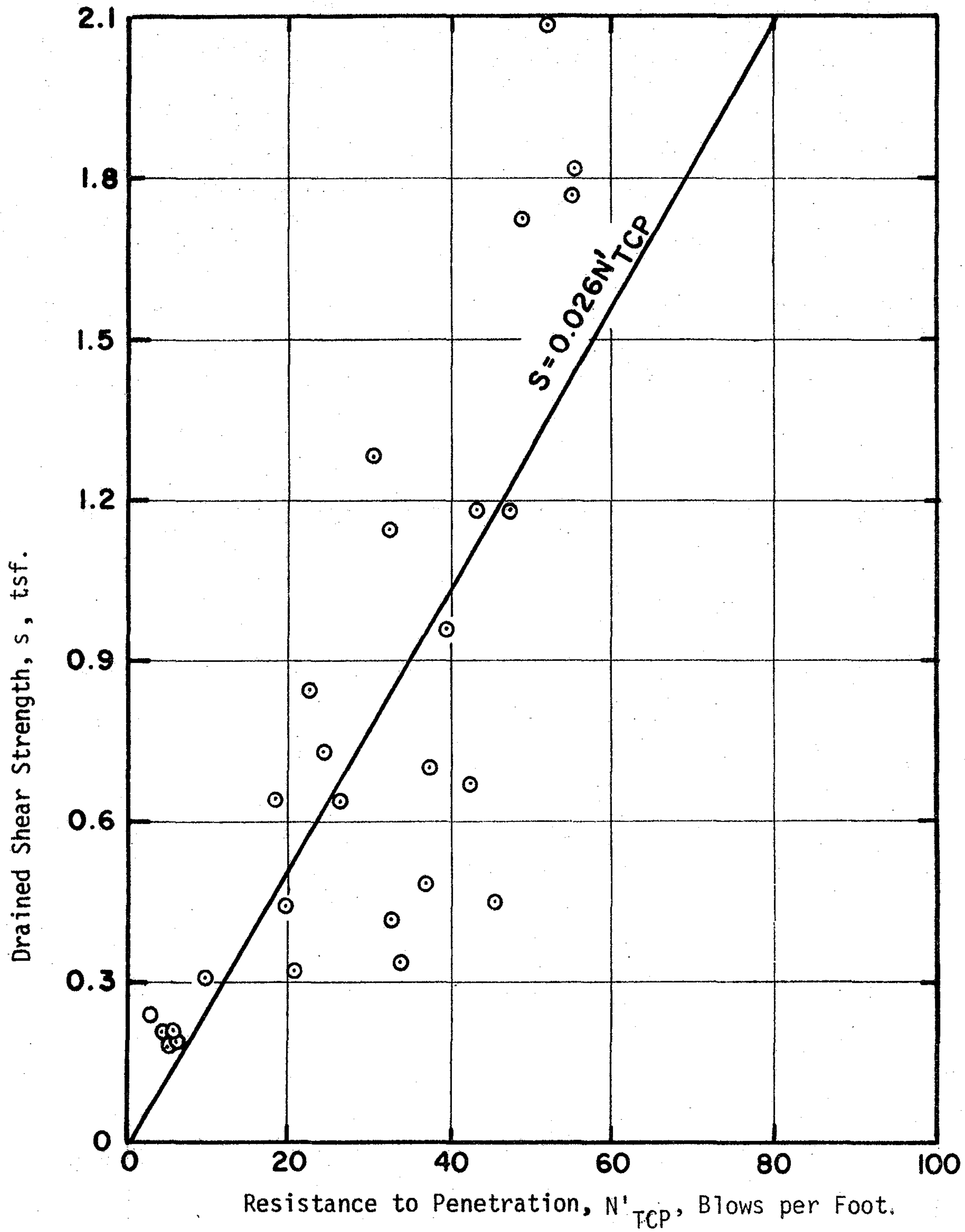


Fig. 17. RELATIONSHIP BETWEEN DRAINED SHEAR STRENGTH AND CORRECTED RESISTANCE TO PENETRATION FOR SP, SM, AND SP-SM SOILS - N'_{TCP}.

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

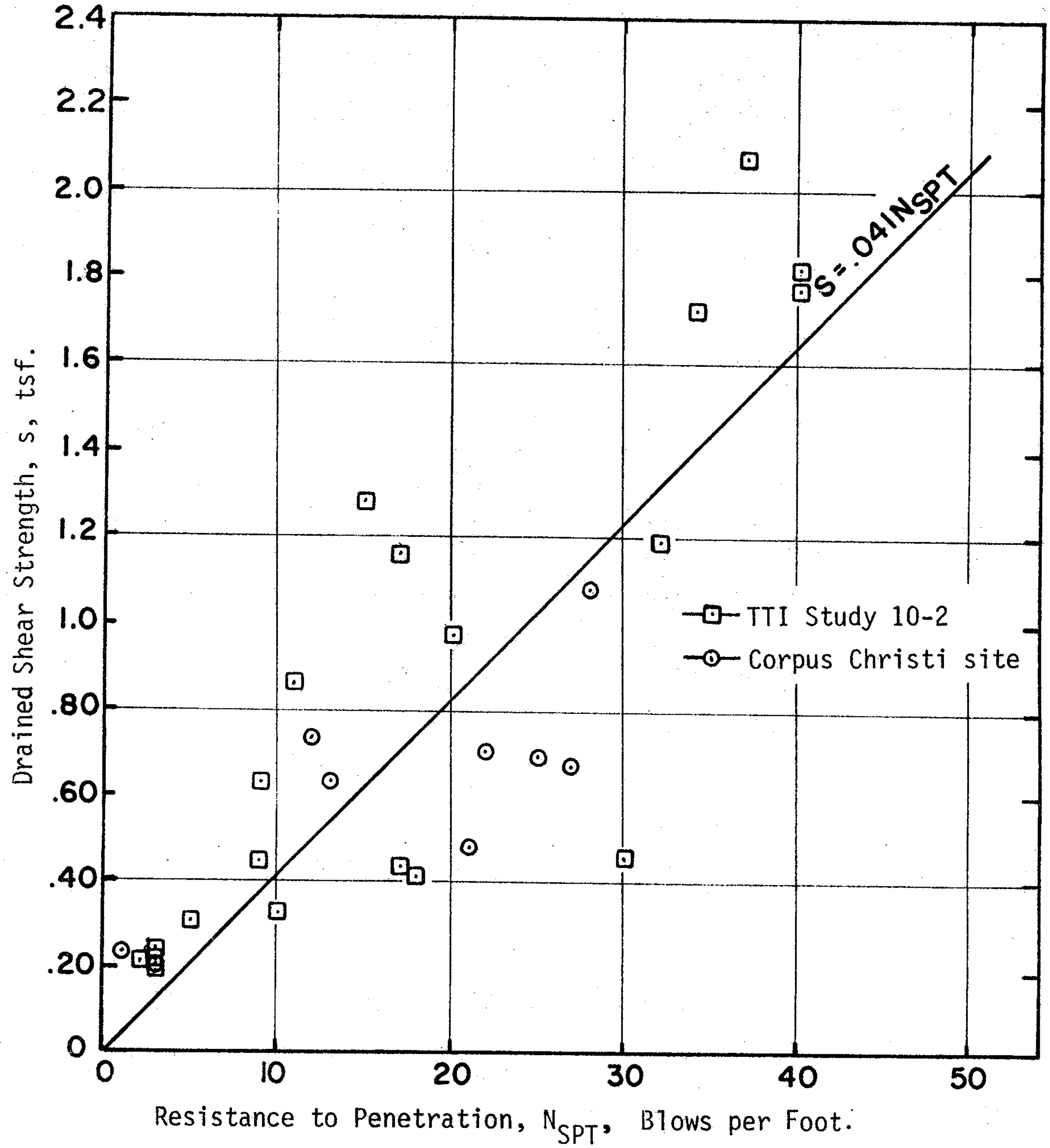


Fig. 18 RELATIONSHIP BETWEEN DRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR SP, SM, AND SP-SM SOILS -N_{SPT}.
(1 ft = .305 m, 1 tsf = 9.58 x 10^2 N/m^2)

where s = drained shear strength, expressed in tons per square foot, and N'_{SPT} = Standard Penetration Test N-value, expressed in blows per foot, corrected using Eq. 12 where applicable. Fig. 19 shows the plotted data which was used to develop the relationship expressed in Eq.18.

An effort was also made to correlate N_{TCP} with the shear strength parameter, ϕ'. The solid curve predicting the relationship between N_{TCP} and ϕ' as shown in Fig. 20 was taken from the Texas Foundation Exploration and Design Manual (3). It can be seen from Fig. 20 that the relationship between N_{TCP} and ϕ' used by the Texas State Department of Highways and Public Transportation forms a lower bound for the data obtained in this study. The plot of N'_{TCP} versus ϕ' is shown in Fig. 21. The solid curve shown in Fig. 21 is the same curve shown in Fig. 20. Many of the data points in Fig. 21 have been moved upward and are further away from the solid curve. The dashed curve is a proposed new lower bound for these data. The dashed curve should only be used with the corrected N-value, N'_{TCP}. The proposed curve yields larger values of ϕ' than the existing curve.

A relationship between the Standard Penetration Test N-value and the effective angle of shearing resistance, ϕ', which has become widely used for foundation design in sands is presented in the text by Peck, Hanson, and Thornburn (14). Eq. 13 was used to convert N_{TCP} to N_{SPT} so that data from this study could be compared with the existing relationship. Fig. 22 is a plot of N_{SPT} and the effective angle of shearing resistance, ϕ'. The solid curve shown is the widely accepted curve taken from Peck, Hanson, and Thornburn (14). The data were also plotted in Fig. 24 using values of N'_{SPT}. In Fig. 23 the solid curve is the

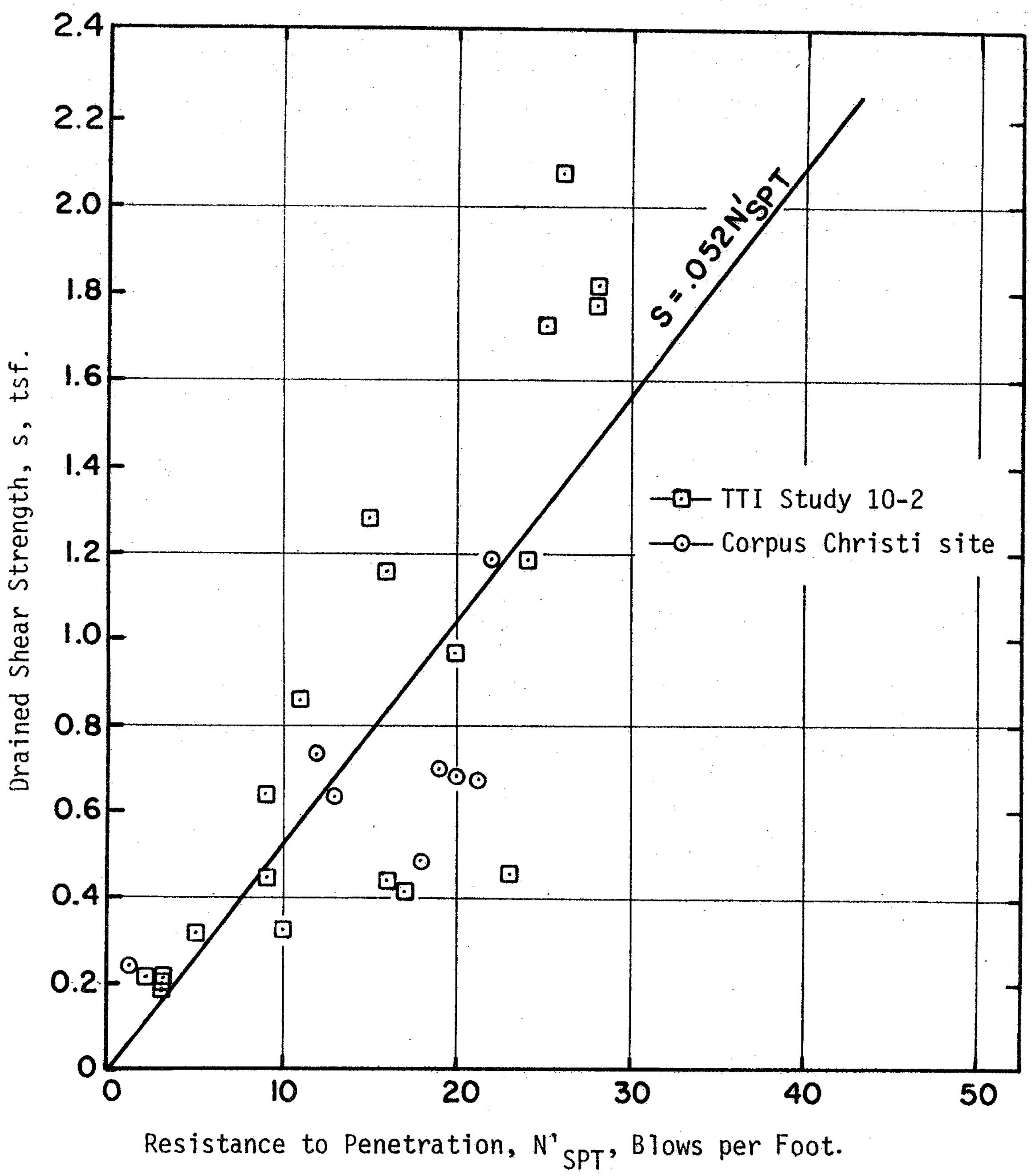


Fig. 19 RELATIONSHIP BETWEEN DRAINED SHEAR STRENGTH AND CORRECTED RESISTANCE TO PENETRATION FOR SP, SM, AND SP-SM SOILS -N'_{SPT}. (1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

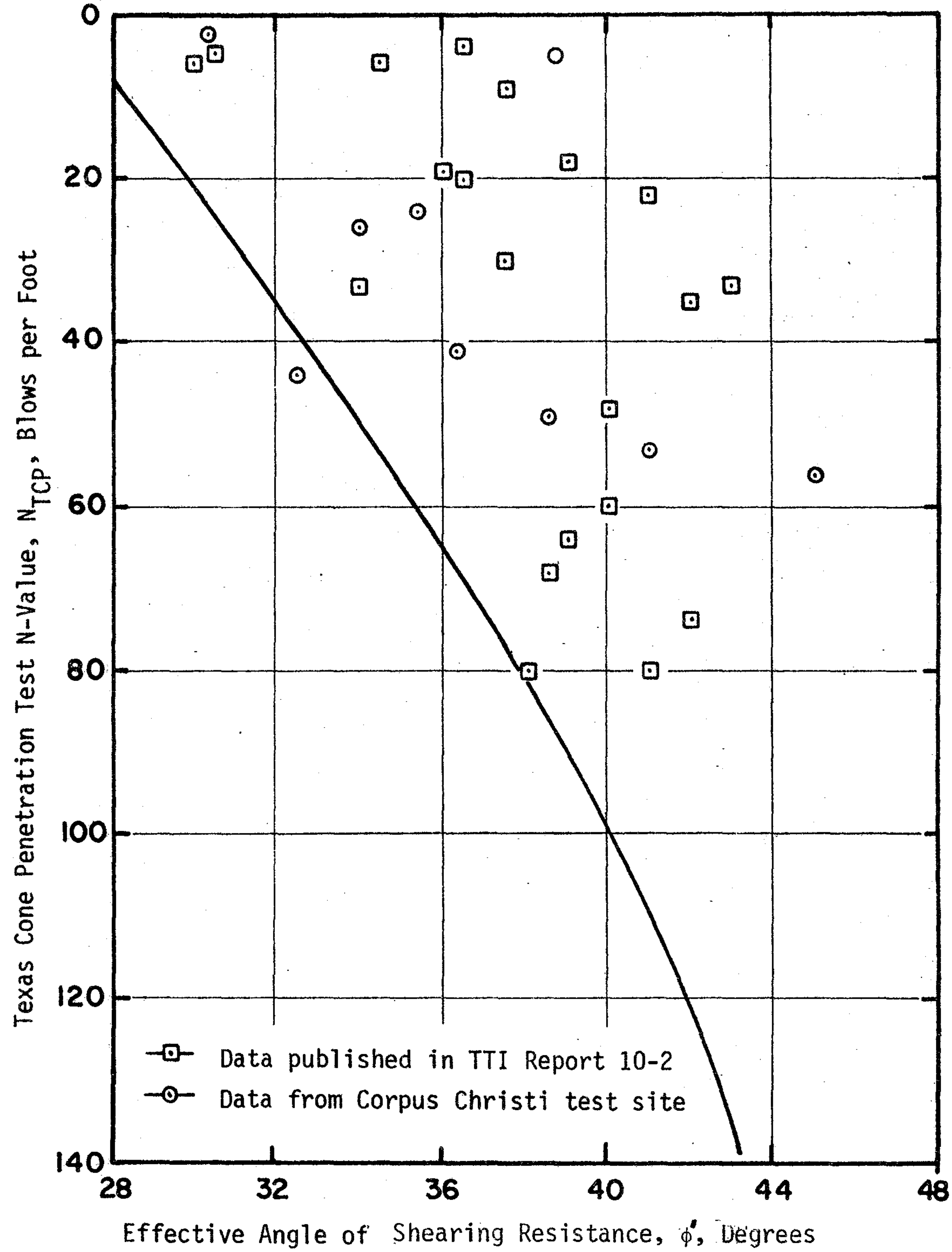


Fig. 20. RELATIONSHIP BETWEEN TEXAS CONE PENETRATION TEST N-VALUE AND EFFECTIVE ANGLE OF SHEARING RESISTANCE FOR SP, SM, AND SP-SM SOILS -N_{TCP}

(1 ft = .305 m; 1 tsf = 9.58×10^2 N/m^2)

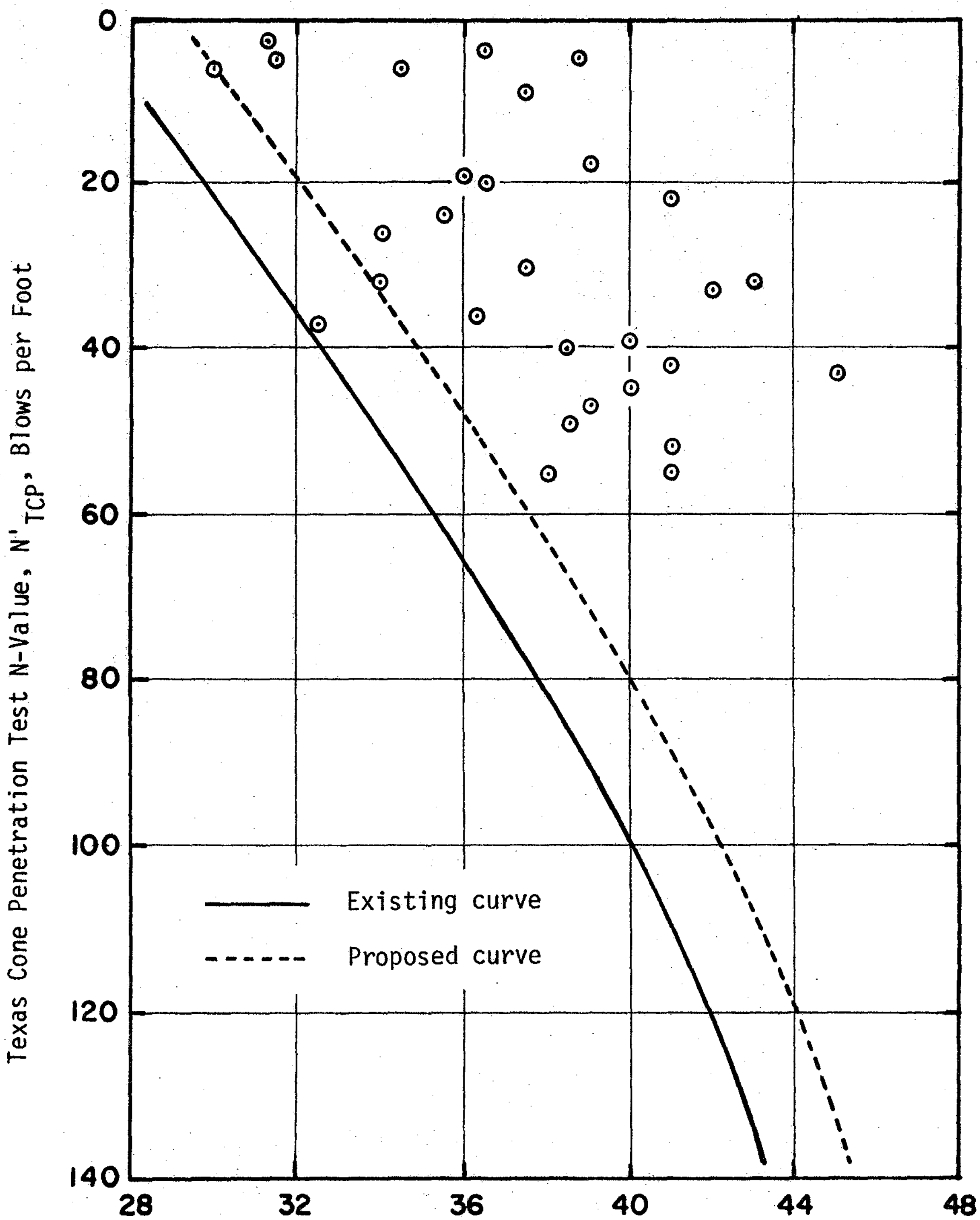


Fig. 21 RELATIONSHIP BETWEEN TEXAS CONE PENETRATION TEST N-VALUE AND EFFECTIVE ANGLE OF SHEARING RESISTANCE FOR SP, SM, AND SP-SM -N'$_{TCP}$

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

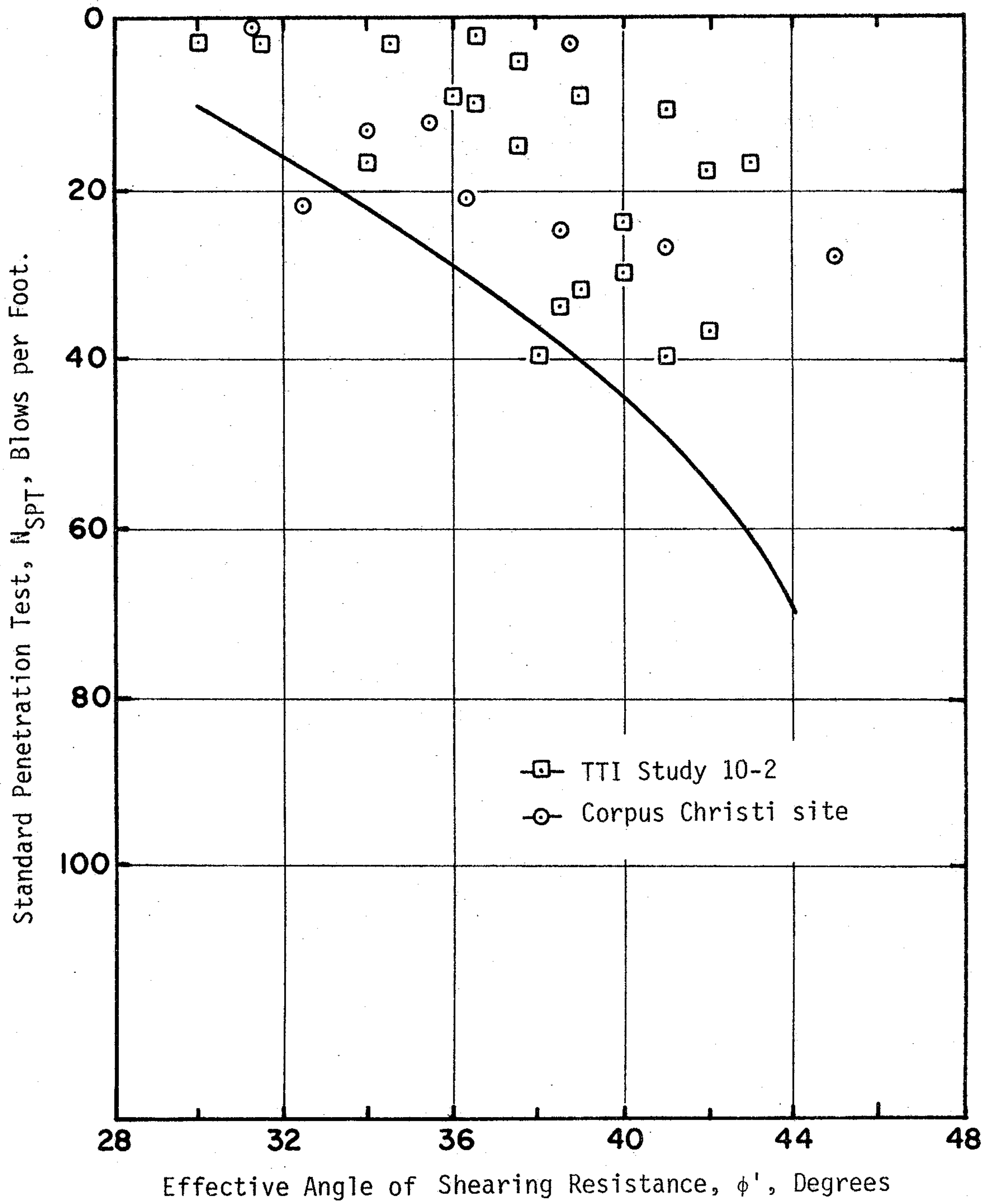


Fig. 22 RELATIONSHIP BETWEEN STANDARD PENETRATION TEST N-VALUE AND EFFECTIVE ANGLE OF SHEARING RESISTANCE FOR SP, SM, AND SP-SM SOILS -N_{SPT}.

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

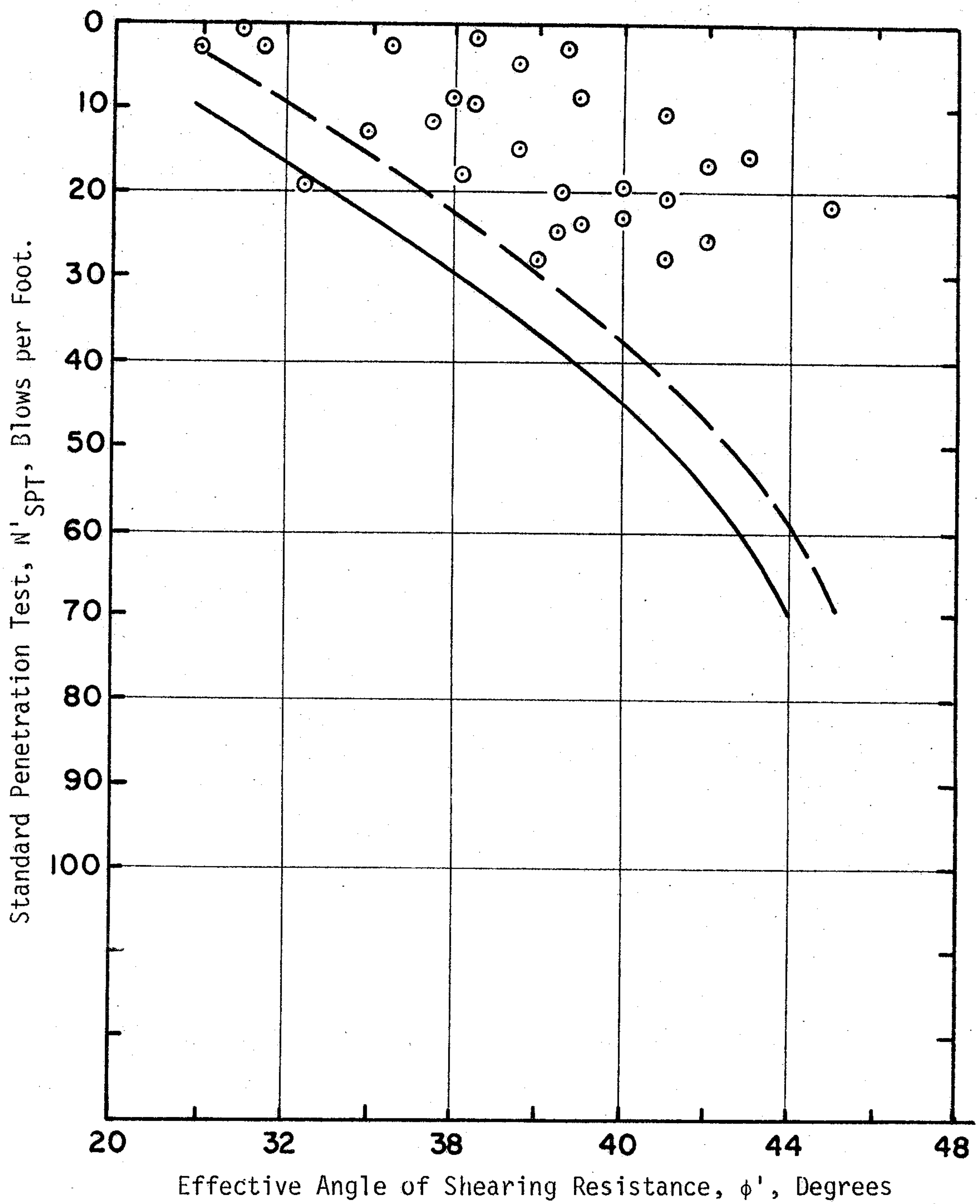


Fig. 23 RELATIONSHIP BETWEEN STANDARD PENETRATION TEST N-VALUE AND EFFECTIVE ANGLE OF SHEARING RESISTANCE FOR SP, SM, AND SP-SM SOILS -N'_{SPT}.
(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

same curve shown in Fig. 22, and the broken curve is a proposed new bound for these data.

The total unit weight, γ_T, and the in situ effective overburden pressure, p' , for each sample had to be determined in order to calculate the drained shear strength, s. Since the values of γ_T and p' were available, an attempt was made to correlate these values with N_{TCP}. Tables 5 and 6 contain a summary of the data used to develop Fig. 24 through 27. Fig. 24 shows a plot of p' versus N_{TCP}. The relationship shown in Fig. 24 is expressed in equation form as follows:

$$p' = 0.172 + 0.023\, N_{TCP} \quad (19)$$

where p' = the effective overburden pressure, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. Fig. 25 shows the relationship between p' and N'_{TCP} when the correction for a very fine or silty saturated sand is made using Eq. 14 The relationship shown in Fig. 25 is expressed in equation form as follows:

$$p' = 0.05 + 0.02\, N'_{TCP}. \quad (20)$$

where p' = the effective overburden pressure, expressed in tons per square foot, and N'_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. Fig. 26 shows the relationship between γ_T and N_{TCP}. The relationship shown in Fig. 26 is expressed in equation form as follows:

$$\gamma_T = 111.0 + 0.231\, N_{TCP} \quad (21)$$

where γ_T = total unit weight, expressed in pounds per cubic foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot.

Table 5-- SUMMARY OF N-VALUES, EFFECTIVE OVERBURDEN PRESSURE, TOTAL UNIT WEIGHT CORPUS CHRISTI TEST SITE

Sample number	N-value, blows / foot		Total Unit Weight (lbs/ft^3)	Effective Overburden Pressure (tsf)
	N_{TCP}	N'_{TCP}		
1	5	5	125.5	0.238
2	2	2	118.6	0.388
4	41	36	131.1	0.658
6	53	42	133.0	0.775
7	49	40	133.6	0.860
8	26	26	127.4	0.945
9	24	24	123.0	1.028
10	44	37	123.0	1.100
11	56	43	124.9	1.180

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 6.-- SUMMARY OF N-VALUES, EFFECTIVE OVERBURDEN PRESSURE, TOTAL UNIT WEIGHT TTI REPORT 10-2 TEST SITES

Sample Number	N-Value, blows/foot		Total Unit Weight (lbs/ft^3)	Effective Overburden Pressure (tsf)
	N_{TCP}	N'_{TCP}		
A-1-2	35	33	111.4	.457
A-1-3	60	45	118.6	.536
A-2-1	4	4	104.3	.287
A-2-2	5	5	106.6	.341
A-2-3	9	9	111.4	.400
A-3-1	6	6	98.7	.270
A-3-2	6	6	103.9	.345
A-3-3	20	20	105.8	.437
B-1-9	33	32	120.2	.643
C-1-13	19	19	118.6	.608
C-1-18	18	18	120.4	.780
D-1-5	22	22	124.7	.960
D-1-6	48	39	123.3	1.145

(1 pcf = 16.01 kg/m^3; 1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

Table 6.-- (Continued) SUMMARY OF N-VALUES, EFFECTIVE OVERBURDEN PRESSURE, TOTAL UNIT WEIGHT TTI REPORT 10-2 TEST SITES				
Sample Number	N-Value, blows/foot		Total Unit Weight (lbs/ft^3)	Effective Overburden Pressure (tsf)
	N_{TCP}	N'_{TCP}		
D-1-7	33	32	122.2	1.235
D-1-12	30	30	134.7	1.665
D-1-19	80	55	125.5	2.032
D-1-22	68	49	119.9	2.165
E-1-11	64	47	119.3	2.270
E-1-12	80	55	123.5	2.325
E-1-17	74	52	130.3	2.755

(1 psf = 16.01 kg/m^3; 1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

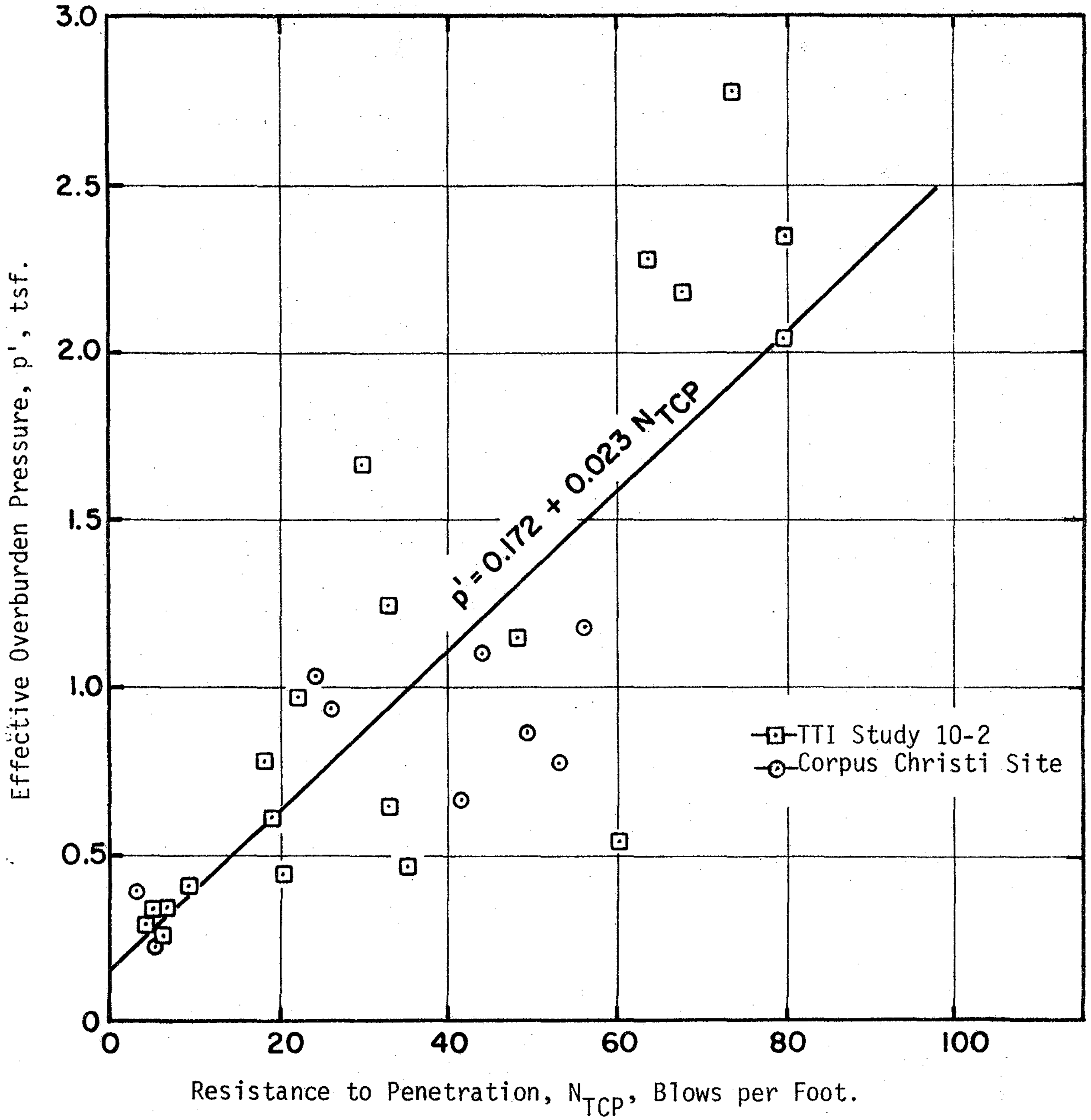
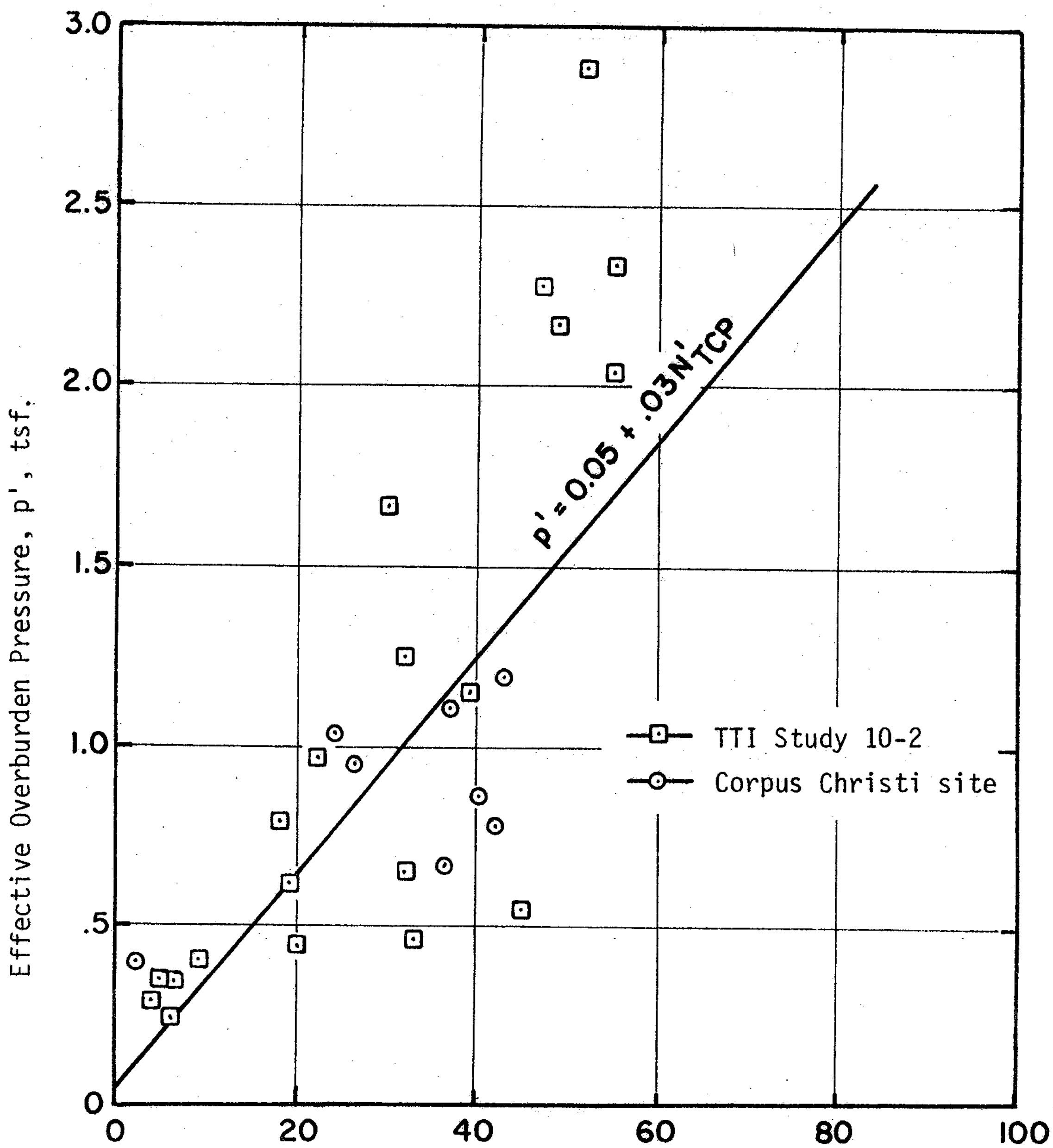


Fig. 24 RELATIONSHIP BETWEEN EFFECTIVE OVERBURDEN PRESSURE AND RESISTANCE TO PENETRATION FOR SP, SM, AND SP-SM SOILS - N_{TCP}.
(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)



Resistance to Penetration, N'_{TCP}, Blows per Foot.

Fig. 25 RELATIONSHIP BETWEEN EFFECTIVE OVERBURDEN PRESSURE AND RESISTANCE TO PENETRATION FOR SP, SM AND SP-SM SOILS -N'_{TCP}.

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

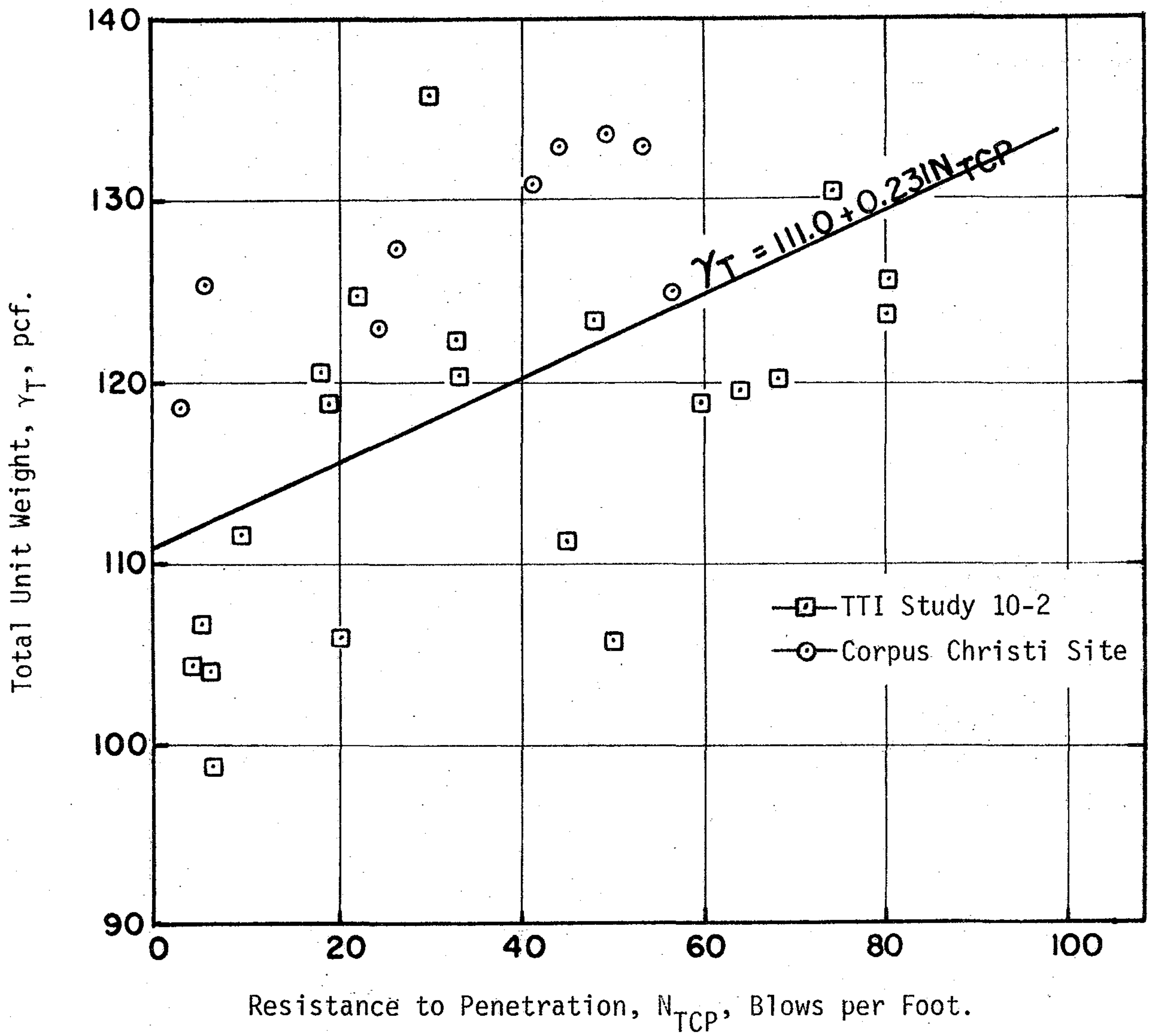


Fig. 26 RELATIONSHIP BETWEEN TOTAL UNIT WEIGHT AND RESISTANCE TO PENETRATION FOR SP, SM, AND SP-SM SOILS - N_{TCP}.

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

Fig. 27 shows the relationship between γ_T and N'_{TCP}. The equation relating γ_T and N'_{TCP} is as follows:

$$\gamma_T = 110.48 + 0.34\ N'_{TCP} \qquad (22)$$

where γ_T = total unit weight, expressed in pounds per cubic foot, and N'_{TCP} = Texas Cone Penetrometer N=value, expressed in blows per foot.

In this section equations have been presented relating the Texas Cone Penetrometer N-value, N_{TCP}, with the drained shear strength, s, the effective overburden pressure, p', and the total unit weight γ_T. Better relationships were developed for s and p' than for γ_T. Also, it has been shown that there is less scatter in the data when the same relationships were developed using N'_{TCP}, i.e. the corrected N-value for very fine or silty saturated sands. The reduced data scatter seems to indicate that the use of a corrected N-value would be appropriate.

A relationship was also developed during this study, which could be compared with the relationship currently used by the Texas State Department of Highways and Public Transportation (SDHPT), between the effective angle of shearing resistance, ϕ', and N_{TCP}. This currently used relationship was shown to be a lower bound for the study data presented. The currently used relationship appears to be even more conservative when compared to the data from this study using corrected values of N_{TCP}.

Using the relationship developed by Touma and Reese (18) to convert the Texas Cone Penetrometer N-values into Standard Penetration Test N-values, correlations were developed for both s and ϕ' versus N_{STP}. Both corrected and uncorrected Standard Penetration Test N-values

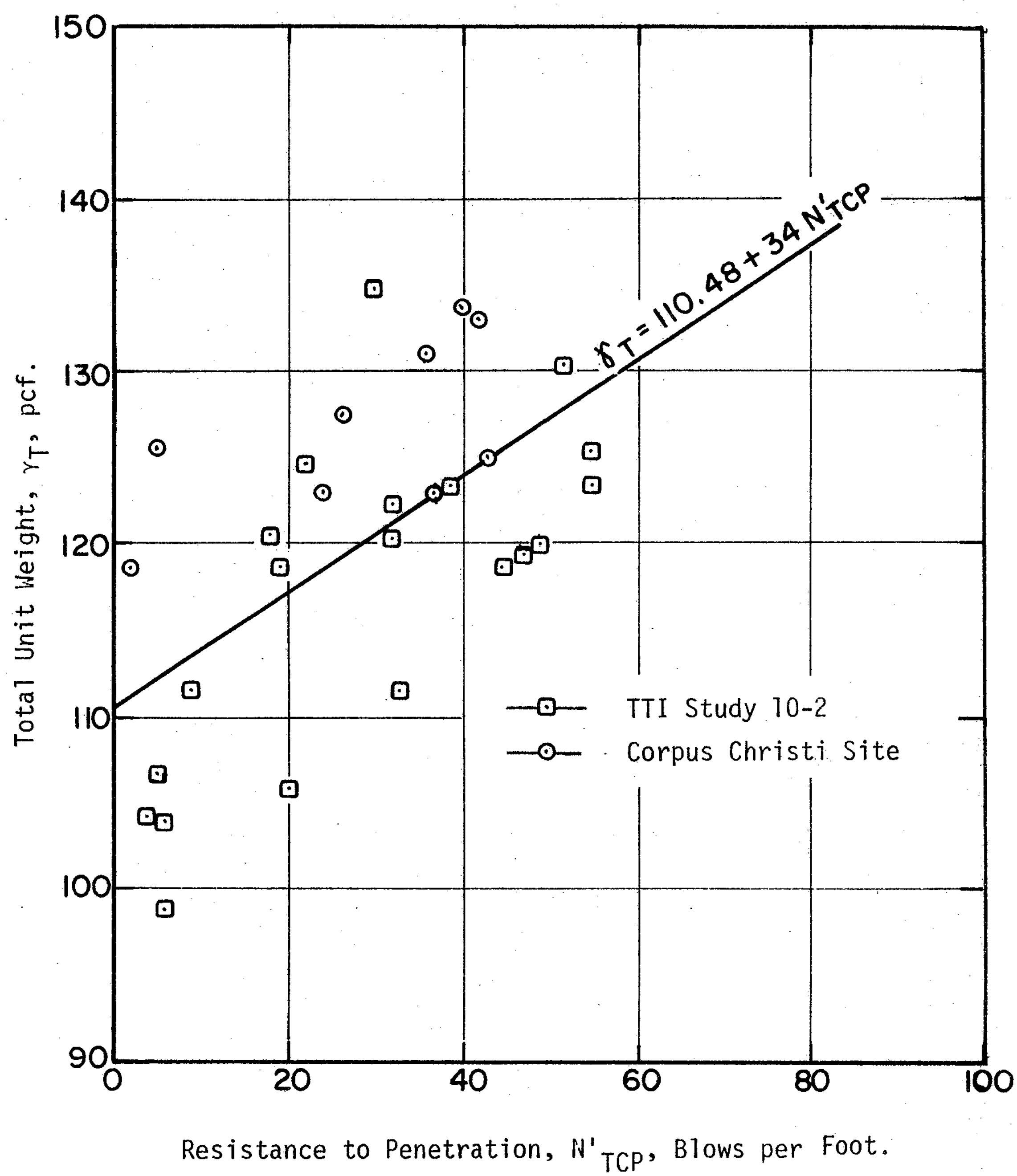


Fig. 27 RELATIONSHIP BETWEEN TOTAL UNIT WEIGHT AND RESISTANCE TO PENETRATION FOR SP, SM, AND SP-SM SOILS

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

were used to develop these correlations. Data scatter was reduced when corrected N-values were used. It was possible to compare the data from this study with the widely used relationship between ϕ' and the Standard Penetration Test N-value appearing in the text by Peck, Hanson, and Thornburn (14). This widely used relationship was shown to be a lower bound for the study data. Also, when the corrected N-value, N'_{SPT}, was used the study data plotted even further above the widely used N_{SPT} versus ϕ' curve.

It has been shown that both the widely used relationship between N_{SPT} and ϕ' and the relationship currently used by the SDHPT between N_{TCP} and ϕ' are conservative based on the results of this study. There appears to be ample justification towards modification of the existing relationships. Furthermore, the study data indicates that corrected N-values should be used where appropriate with the proposed new relationships.

Finally, it should be noted that the samples tested in this study were classified by the Unified Classification System as SP, SM, and SP-SM soils. This may be a limitation to the new correlations, in the sense that the new correlations are not proven for well-graded or coarse sands. It is felt, however, that many of the cohesionless soils that exist in nature will fall into one of the classification categories covered in this study.

PENETROMETER CORRELATIONS FOR DRIVEN AND BORED PILES

The third objective of this study was the development of a correlation between N_{TCP} and unit side friction, f, and unit point bearing, q, for driven and bored piles. Data from previous research efforts were used to develop the correlations. The data used to develop the correlations for bored piles are reported in detail in a series of reports produced by The Center for Highway Research (CFHR) for the Texas State Department of Highways and Public Transportation (SDHPT) (1, 7, 13, 18, 19, 21). The data used to develop the correlations for driven piles are reported in detail in TTI Report 125-8F (5). The research associated with TTI Report 125-8F was also conducted for the SDHPT. Table 7 contains a list of the test site locations used for the bored piles and includes the references containing the detailed information for the tests. Table 8 contains a list of the test site locations used for driven piles and includes the references containing the detailed information for the tests.

Unit Side Friction and Unit Point Bearing. - In order to correlate f and q with N_{TCP} it is necessary to have load transfer data so that both side load and point load can be determined. Therefore, during this phase of the research it was necessary to find data from instrumented load tests of full scale piles. It was also necessary to have soil profiles complete with N-values for each test site. The piles used in this study were instrumented with strain gages. In most cases the gages were placed at the top of the pile, near the bottom of the pile, and along the pile at locations of major changes in soil types. The use of strain gages made it possible to measure the load transfer between

Table 7. LIST AND LOCATIONS OF BORED PILES

Test Pile	Location of Test Site	Reference No.
G1	Houston, Texas - South middle bay of bent 12 of I 610 - I 45 Interchange East-bound structure	(19)
G2	Houston, Texas - North bay of bent 27 of I 610 - I 45 Interchange East-bound structure	(19)
BB	Houston, Texas - West bay of bent 5 of left frontage street SH 288 and Brays Bayou structure	(19)
LB	Ten miles west of Bryan, Texas adjacent to State Highway 21	(7)
US59	Live Oak County, Texas - West bay of bent no. 3 of the left roadway of IH-37 and US 59 structure	(18)
HH	Live Oak County, Texas - North bay of bent no. 2 of the left main lane of IH 37 and Hailey Hollow structure	(18)
US90	San Antonio, Texas - Intersection of S.W. Military Drive and U.S. Highway 90	(21)

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 7. (CONTINUED) LIST AND LOCATIONS OF BORED PILES

Test Pile	Location of Test Site	Reference No.
HB&T	Houston, Texas - I 610 - HB&T Railroad overpass structure	(1)
S1T1	Houston, Texas - I 610 - SH 225 intersection	(13)
S2T1	Houston, Texas - I 610 - SH 225 intersection	(13)
S3T1L1	Houston, Texas - I 610 - SH 225 intersection	(13)

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 8. LIST AND LOCATIONS OF DRIVEN PILES		
Test Pile	**Location of Test Site**	**Reference No.**
PA1	Intracoastal Canal Bridge on SH 87 south of Port Arthur, Texas	(5)
PA2	Intracoastal Canal Bridge on SH 87 south of Port Arthur, Texas	(5)
CC	Park Road 22 on the Intracoastal Waterway near Corpus Christi, Texas	(5)
H-99R	US 77 at the North Floodway near Harlingen, Texas	(5)
H-4L	US 77 at the North Floodway near Harlingen, Texas	(5)

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

various points along the pile. The load measured in the bottom strain gage is the point load. Since the N-values for the soil layers between gages were known, a correlation could be attempted. Fig. 28 is a schematic of a pile showing typical locations of strain gages. The unit side friction, f, between the top two gages can be computed as follows:

$$f = \frac{\text{load in gage 1 - load in gage 2}}{\text{contact area}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

where f = unit side friction, expressed in tons per square foot, gage loads are expressed in tons, and contact area = the perimeter of the pile x (depth to gage 2 - depth to gage 1), expressed in square feet. The unit point load, q, can be computed as follows:

$$q = \frac{\text{load in gage 4}}{\text{area of pile point}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

where q = unit point bearing, expressed in tons per square foot, load in gage 4 is the point load expressed in tons, and area of pile point is the cross sectional area expressed in square feet. Appendix V contains tables showing the location of the strain gages, the value of N_{TCP} for each soil layer, and the corrected value of N_{TCP}. The correction of N_{TCP} was made using Eq. 14 where applicable.

The correlations presented in this section are divided into two groups. Group I is the correlation for bored piles. Group I includes correlations of f and q for both cohesionless and cohesive soils. Group II is the correlation for driven piles and includes f and q for both cohesionless and cohesive soils.

In order to determine the ultimate bearing capacity, P_{ult}, of the piles, it was necessary to use the load settlement curves from the pile

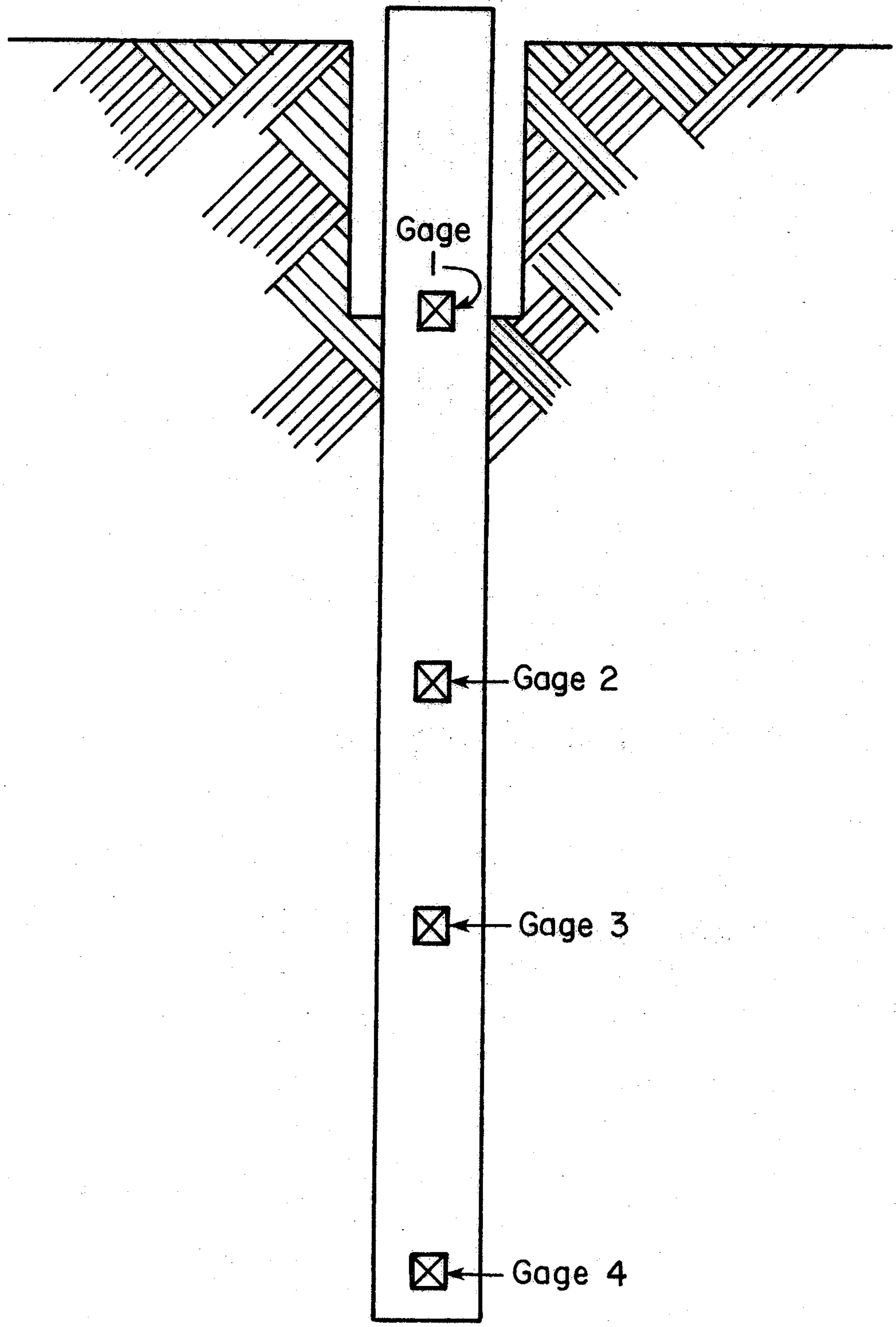


Fig. 28 SCHEMATIC VIEW OF INSTRUMENTED TEST PILE

load tests. The magnitude of P_{ult} was determined using a method outlined in the Texas Foundation Exploration and Design Manual (3). This method is referred to in this study as the method of tangents. The method of tangents was used in developing the correlations presented herein. The magnitude of P_{ult} was also determined by another method using the maximum load applied to the piles during the load tests. This method is referred to in this study as the maximum applied load method. Plotted data are not presented herein for the maximum applied load method. However, the values of f and q obtained by this method were determined and are presented in Appendix V. The correlations given in this section for f and q in cohesionless soils utilizes the values of N_{TCP} as measured in the field. Although the plotted data are not presented, correlations were also made based on values of N_{TCP} corrected using Eq. 12 where applicable. Table 9 is a summary of all correlations developed for bored piles and Table 10 is a summary of all correlations developed for driven piles.

All piles analyzed in this study were load tested in the same manner. This method is described by Fuller and Hoy (8) and is referred to as The Texas Highway Department Quick-Load Test Method. Some of the piles analyzed were subjected to several load tests. In most cases the final load test was used.

Bored Piles. - Piles G1, G2, and BB were tested in Houston, Texas and were installed using the slurry displacement method. All three piles completely penetrated a layer of clay and were bored into a layer of sand. Pile LB was tested west of Bryan, Texas. The soil type at this test location was predominantly clay to the total depth of pile

Table 9. SUMMARY OF CORRELATIONS DEVELOPED FOR BORED PILES

Method Used to Determine $P_{ult.}$	N-value	Soil Type	Correlation
Method of tangents	Measured	Cohesive	f = 0.022 N
Method of tangents	Measured	Cohesionless	f = 0.014 N
Maximum applied load	Measured	Cohesive	f = 0.023 N
Maximum applied load	Measured	Cohesionless	f = 0.015 N
Method of tangents	Measured	Cohesive	q = 0.32 N
Method of tangents	Measured	Cohesionless	q = 0.10 N
Maximum applied load	Measured	Cohesive	q = 0.357 N
Maximum applied load	Measured	Cohesionless	q = 0.167 N
Method of tangents	Corrected	Cohesionless	f = 0.024 N
Maximum applied load	Corrected	Cohesionless	f = 0.025 N
Method of tangents	Corrected	Cohesionless	q = 0.18 N
Maximum applied load	Corrected	Cohesionless	q = 0.30 N

f = unit side friction, expressed in tsf.

N = Texas Cone Penetrometer N-value, expressed in blows per foot.

(1 psi = 6.9 N/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

Table 10. SUMMARY OF CORRELATIONS DEVELOPED FOR DRIVEN PILES

Method Used to Determine $P_{ult.}$	N- value	Soil Type	Correlation
Method of tangents	Measured	Cohesive	f = 0.031 N
Method of tangents	Measured	Cohesionless	f = 0.033 N
Maximum applied load	Measured	Cohesive	f = 0.032 N
Maximum applied load	Corrected	Cohesionless	f = 0.035 N
Method of tangents	Measured	Cohesive	q = 0.103 N
Method of tangents	Measured	Cohesionless	q = 1.330 N
Maximum applied load	Measured	Cohesive	q = 0.173 N
Maximum applied load	Corrected	Cohesionless	q = 1.620 N

f = unit side friction, expressed in tons per square foot

N = Texas Cone Penetrometer N-value, expressed in blows per foot

(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

penetration. The piles designated US 59 and HH were tested in Live Oak County, Texas. Both piles completely penetrated a clay layer and were bored into a sand layer. The pile designated US 90 was tested in San Antonio, Texas. This pile completely penetrated a clay layer and was bored into a clay shale layer. Pile HB & T was tested in Houston, Texas and penetrated a predominantly clay soil with intermittent layers of silt and silty sand. The piles designated S1T1, S2T1, and S3T1L1 were tested in Houston, Texas. These test piles were installed in a predominantly clay soil.

The unit side friction, f, versus N_{TCP} for bored piles is plotted in Fig. 29 and includes friction data for both cohesive (clay) and cohesionless (sand) soils. As mentioned previously, the friction data were determined using the method of tangents to determine P_{ult} and the values of N_{TCP} are the values measured in the field. The correlation equation for clay soils is:

$$f = 0.022\, N_{TCP} \quad (25)$$

where f = unit side friction, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. The equation relating, f, and N_{TCP} for sands is expressed as:

$$f = 0.014\, N_{TCP} \quad (26)$$

where f = unit side friction, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. As indicated on Fig. 29 some of the values of unit side friction were obtained from piles installed by the slurry displacement method. The data points for these piles fall in the same range of values as the

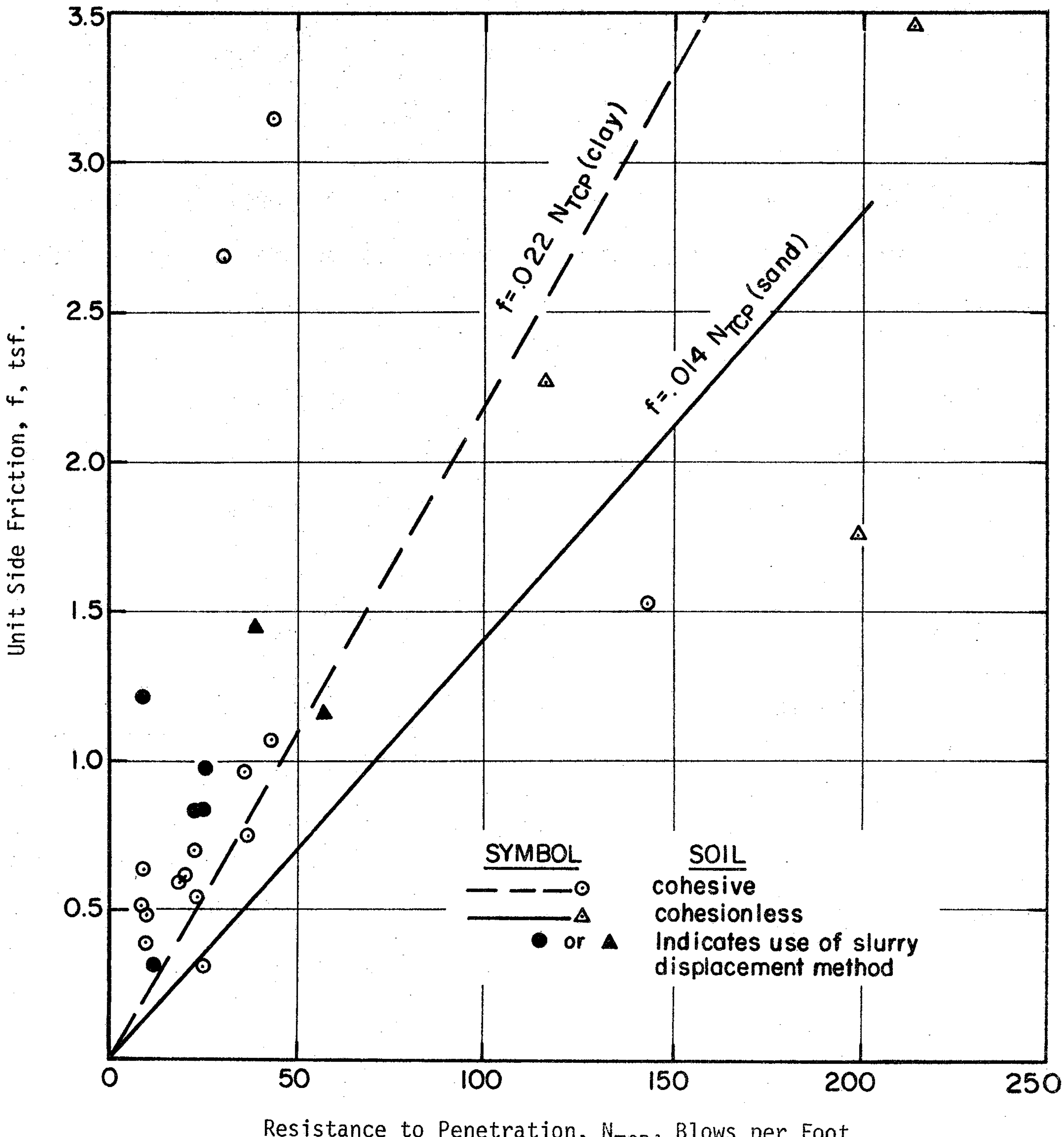


Fig. 29 RELATIONSHIP BETWEEN UNIT SIDE FRICTION AND RESISTANCE TO PENETRATION FOR BORED PILES.
(1 ft = 0.305 m; 1 tsf = 9.58 x 10^2 N/m^2)

points from the other piles and it was decided to combine them for purposes of making the correlations. Table 11 contains a summary of the data plotted in Fig. 29.

The correlations developed for f and N_{TCP} are considered preliminary. There are only five data points for f in sand and these data points are very scattered. Because of the scatter and the limited amount of data for f in sand, the correlation is not good. There was more data available for f in clay, but the data also exhibit considerable scatter. There does seem to be a better trend developing between f and N_{TCP} for the clay soils. With the addition of data from future research it may be possible to develop a better correlation between f and N_{TCP} for clays.

Fig. 30 shows the plotted data for unit point bearing, q, and N_{TCP}. Fig. 30 includes values of q for both sands and clays. The values of q were determined using the method of tangents and the values of N_{TCP} are the values measured in the field. The relationship between q and N_{TCP} for clay soils is:

$$q = 0.32\, N_{TCP} \qquad (27)$$

where q = unit point bearing, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value., expressed in blows per foot. The relationship between q and N_{TCP} for sands is:

$$q = 0.10\, N_{TCP} \qquad (28)$$

where q = unit point bearing, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. The data plotted in Fig. 30 are summarized in Table 12. Again because of

Table II - SUMMARY OF VALUES OF SIDE FRICTION AND N_{TCP} FOR BORED PILES

Side Friction tons per square foot	N_{TCP} blows per foot	Soil Type
0.32	12	CLAY
.83	23	CLAY
1.22	9	CLAY
.84	25	CLAY
.95	27	CLAY
.38	9	CLAY
.53	23	CLAY
.64	9	CLAY
.61	20	CLAY
.51	9	CLAY
.70	23	CLAY
.59	18	CLAY
3.14	44	CLAY
2.69	30	CLAY
.32	25	CLAY
1.53	143	CLAY
.48	10	CLAY
.75	37	CLAY
1.07	43	CLAY

(1 ft = .305 m; 1 tsf = 9.58×10^2 N/m^2)

Table II (CONTINUED) SUMMARY OF VALUES OF SIDE FRICTION AND N_{TCP} FOR BORED PILES

Side Friction Tons Per Square Foot	NTCP Blows Per Foot	Soil Type
.96	36	CLAY
1.15	57	SAND
1.45	39	SAND
1.76	199	SAND
2.27	115	SAND
3.45	213	SAND

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

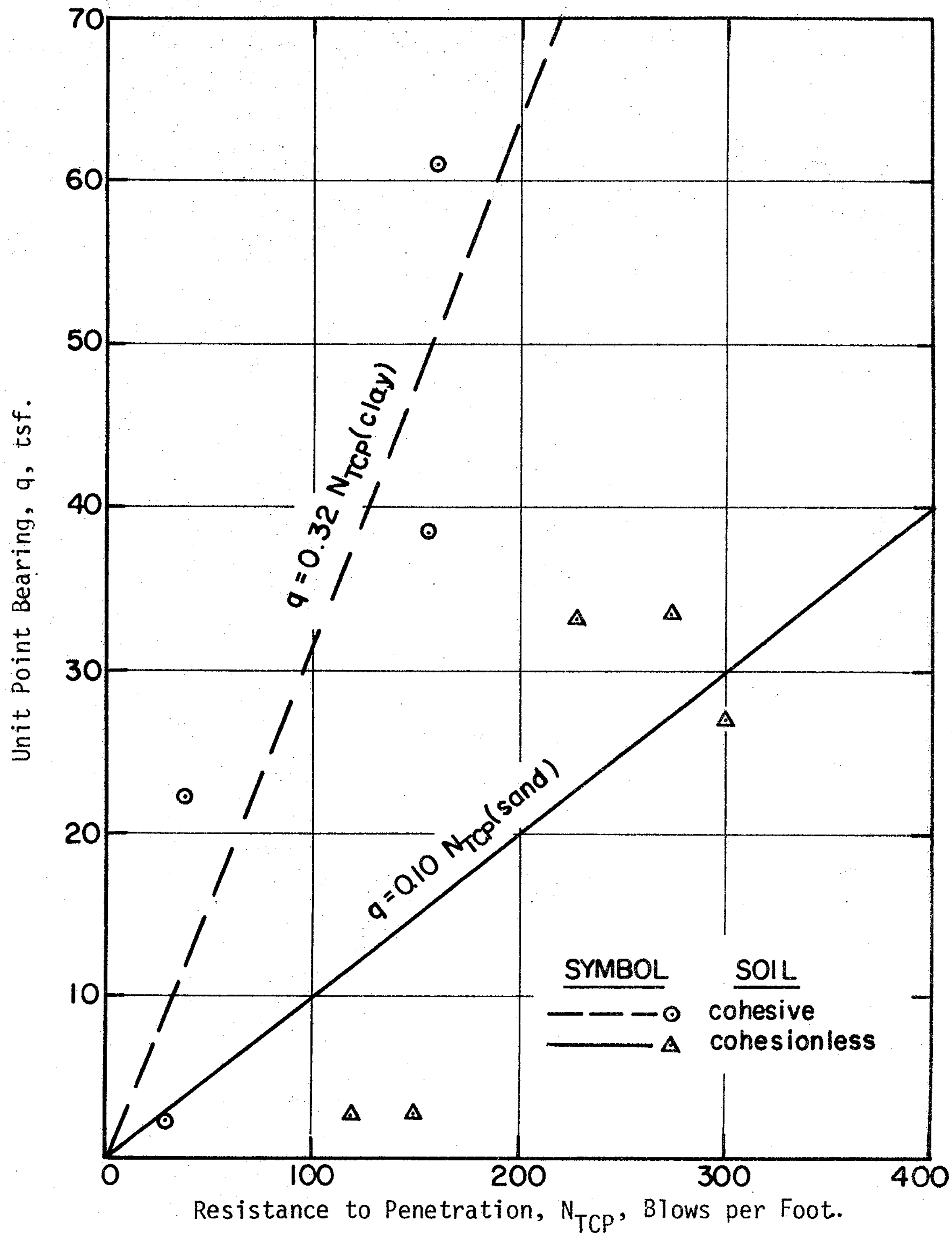


Fig. 30 RELATIONSHIP BETWEEN UNIT POINT BEARING AND RESISTANCE TO PENETRATION FOR BORED PILES
(1 ft = 0.305 m; 1 tsf = 9.58 x 10^2 N/m^2)

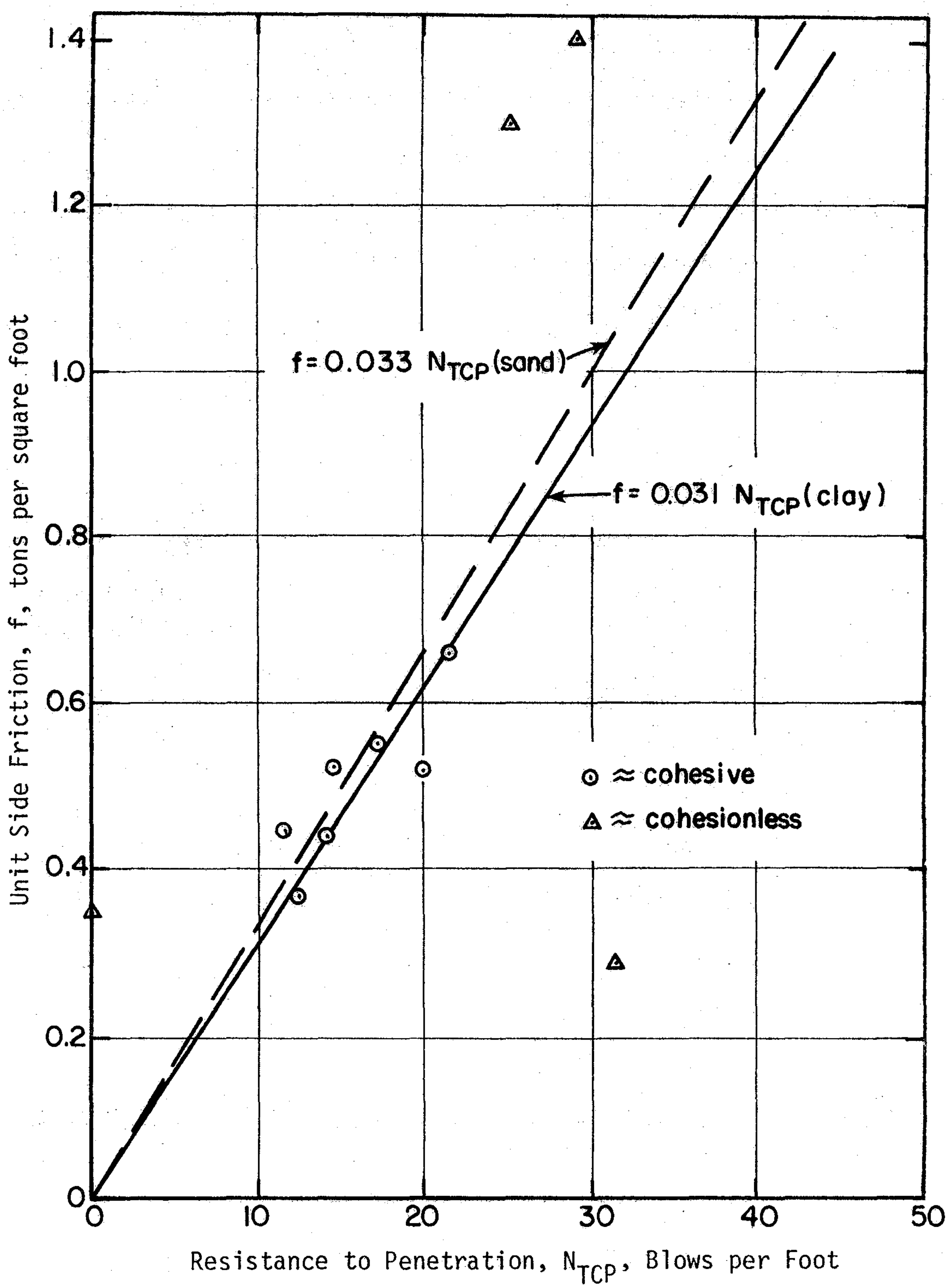


Fig. 31 RELATIONSHIP BETWEEN UNIT SIDE FRICTION AND RESISTANCE TO PENETRATION FOR DRIVEN PILES
(1 ft. = .305 m; 1 tsf = 9.58×10^2 N/m^2)

Table 13 - SUMMARY OF VALUES OF UNIT SIDE FRICTION AND N_{TCP} FOR DRIVEN PILES

Side Friction tons per square foot	N_{TCP} blows per foot	Soil Type
0.521	14.5	CLAY
0.441	14.0	CLAY
0.523	20.0	CLAY
0.447	11.5	CLAY
0.555	17.0	CLAY
0.662	21.5	CLAY
0.368	12.5	CLAY
0.345	0.0	SAND
0.285	31.5	SAND
1.400	29.0	SAND
1.300	25.0	SAND

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

limited. Although a reasonably good correlation between f and N_{TCP} was developed, this correlation should be considered preliminary. There is considerable scatter in the data used to correlate f and N_{TCP} for sands and the data used for this correlation were very limited. Again, there is a need for additional data in order to verify and improve these correlations.

Fig. 32 is a plot of unit point bearing, q, versus N_{TCP} for driven piles. Values of q for both sand and clay soils are plotted. The method of tangents was used to determine q, and values of N_{TCP} measured in the field were used to develop the correlations. The relationship between q and N_{TCP} for clays is:

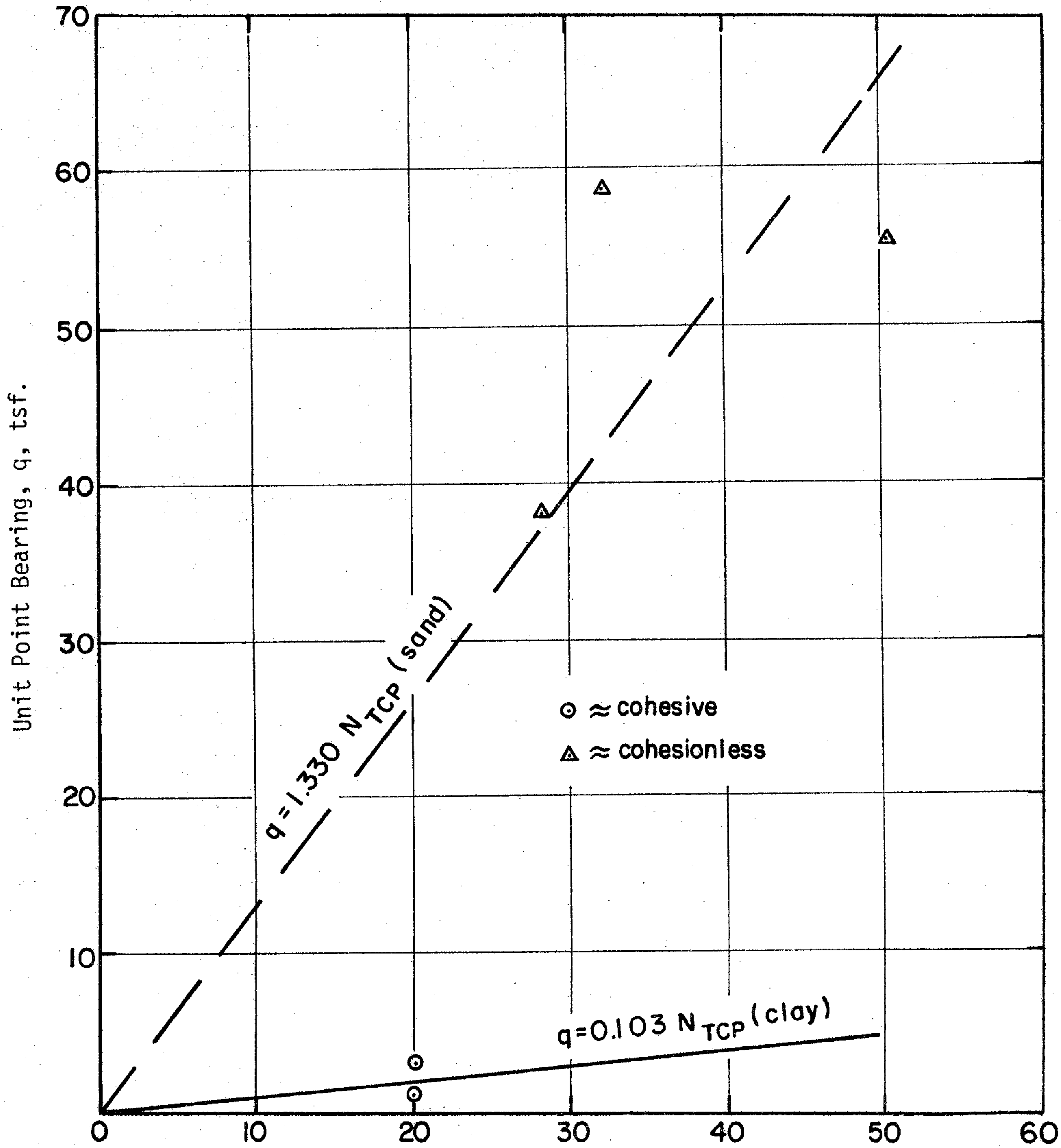
$$q = 0.103\ N_{TCP} \qquad (31)$$

where q = unit point bearing, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. The relationship between q and N_{TCP} for sands is:

$$q = 1.330\ N_{TCP} \qquad (32)$$

where q = unit point bearing, expressed in tons per square foot, and N_{TCP} = Texas Cone Penetrometer N-value, expressed in blows per foot. Table 14 contains the data plotted in Fig. 32. The data used to develop the correlations between q and N_{TCP} for driven piles are really limited and these correlations should be considered very preliminary.

Correlations were developed in this section relating both unit side friction, f, and unit point bearing, q, with N_{TCP} for bored and driven piles. These correlations were developed using a limited amount of data and in most cases there was considerable data scatter. Therefore,



Resistance to Penetration, N_{TCP}, Blows per Foot.

Fig. 32 RELATIONSHIP BETWEEN UNIT POINT BEARING AND RESISTANCE TO PENETRATION FOR DRIVEN PILES.

(1 ft. = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

TABLE 14 - SUMMARY OF VALUES OF UNIT POINT BEARING FOR DRIVEN PILES

Unit Point Bearing tons per square foot	N_{TCP} blows per foot	Soil Type
1.045	20.0	CLAY
3.072	20.0	CLAY
55.670	50.0	SAND
58.769	32.0	SAND
38.127	28.0	SAND

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

these correlations should be considered preliminary in nature.

A reasonably good correlation was developed relating q and N_{TCP} for bored piles in clay in the sense that there was not much data scatter. However, this correlation is based upon only four data points. A reasonable correlation was also developed between f and N_{TCP} for driven piles in clay. This correlation has the limitation that all but one of the seven data points used to develop the correlation came from the same test site.

Tables 9 and 10 contain a list of all of the correlations developed for f and q. It is interesting that, with only a few exceptions, the constants of proportionality do not change greatly when different methods are used to determine the ultimate bearing capacity, P_{ult}, of the test piles. This is primarily due to the manner in which the piles were load tested. That is, an attempt was made to reach a plunging failure for each load test.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions. - Correlations have been developed between the Texas Cone Penetrometer Test N-value and the unconsolidated-undrained shear strength for a group of cohesive soils. The soil shear strengths used in the correlations were determined using both the Texas Triaxial Test and the ASTM Triaxial Test. The correlations were developed for three soil subgroups which include homogeneous CH soils (i.e. soils with no secondary structure), silty CL soils, and sandy CL soils. The following conclusions are made for cohesive soils:

1. The shear strengths of identical samples were higher when determined by the Texas Triaxial Test (TAT) than those shear strengths determined by the ASTM Triaxial Test (ASTM). The equation relating these shear strengths is as follows:

   c_u (ASTM) = 0.58 c_u (TAT)

2. (a) The following equations can be used to predict the unconsolidated-undrained shear strength, based on the Texas Triaxial Test, when the Texas Cone Penetrometer (TCP) Test N-value is known:

   c_u (TAT) = 0.11 N_{TCP} - Homogeneous CH soils
   c_u (TAT) = 0.11 N_{TCP} - Silty CL soils
   c_u (TAT) = 0.095 N_{TCP} - Sandy CL soils

   (b) Equations were also developed relating the unconsolidated-undrained shear strength, as determined by the ASTM Triaxial Test, to the TCP Test N-value. These equations are as follows:

c_u (ASTM) = 0.067 N_{TCP} - Homogeneous CH soils

c_u (ASTM) = 0.054 N_{TCP} - Silty CL soils

c_u (ASTM) = 0.053 N_{TCP} - Sandy CL soils

3. Results obtained by Touma and Reese (18) were used to develop equations which can be used to predict the unconsolidated-undrained shear strength from the Standard Penetration Test N-value. The ASTM shear strength can be predicted using the following equations:

   c_u (ASTM) = 0.096 N_{SPT} - Homogeneous CH soil

   c_u (ASTM) = 0.076 N_{SPT} - CL soils

Correlations were developed between the drained shear strength of cohesionless soils and the Texas Cone Penetrometer Test N-value. In order to calculate the shear strength, it was necessary to determine the effective angle of shearing resistance, ϕ', the effective overburden pressure, p', and the total unit weight, γ_T. Correlations were also developed between these parameters and the TCP N-value. The following conclusions are made for SP, SM, and SP-SM soils:

1. The drained shear strength can be predicted using the following equation if the Texas Cone Penetrometer Test N-value, N_{TCP}, is known:

   $s = 0.021\ N_{TCP}$

2. The effective overburden pressure can be predicted using the following equation if N_{TCP} is known:

   $p' = 0.172 + 0.023\ N_{TCP}$

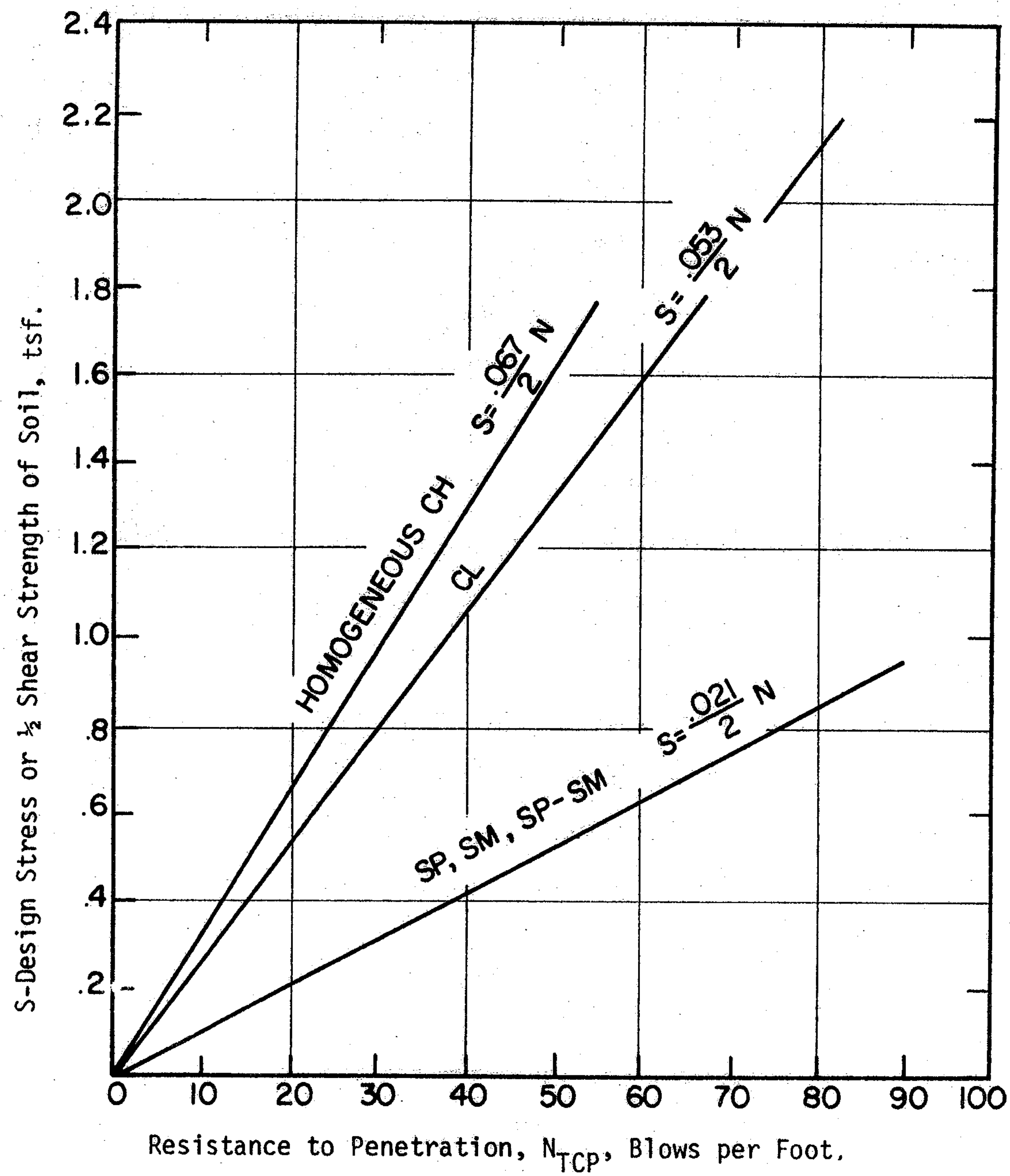


Fig. 33. RELATIONSHIP BETWEEN DESIGN STRESS AND RESISTANCE TO PENETRATION FOR THE TEXAS CONE PENETROMETER.

(1psi = 6.9 kN/m^2; 1pcf = 16.01 kg/m^3; 1 ft = .305 m)

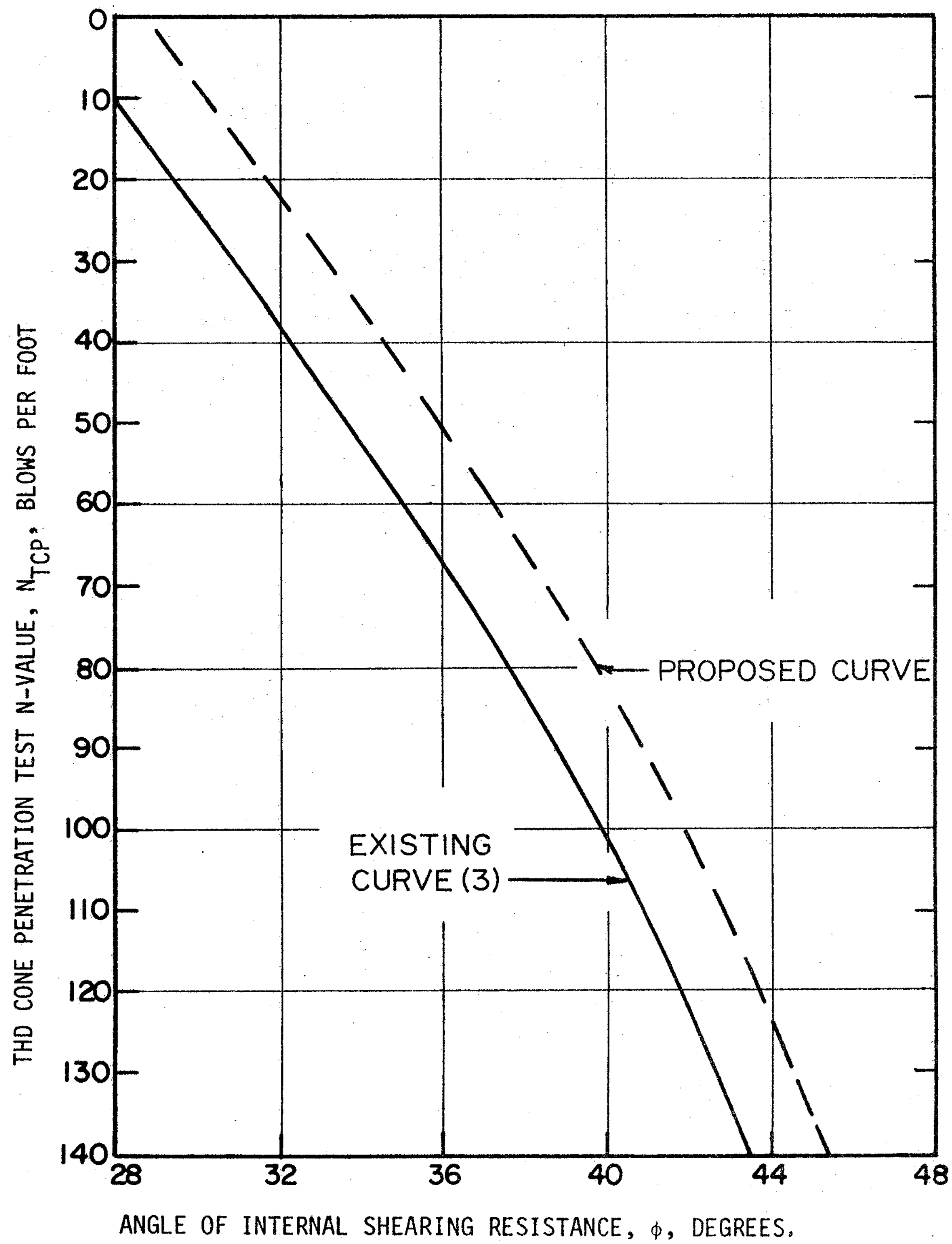


Fig. 34. RELATIONSHIP BETWEEN THE EFFECTIVE ANGLE OF SHEARING RESISTANCE AND RESISTANCE TO PENETRATION FOR THE TEXAS CONE PENETROMETER.
(1 psi = 6.9 kN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft = .305 m)

N-value to the effective angle of shearing resistance. The solid curve is the relationship currently used by the Texas State Department of Highways and Public Transportation. The broken curve is the proposed new curve based upon the results of this study. The broken curve forms a lower bound to test data when corrected values of N_{TCP} are used. It is felt that the relationship proposed should only be used with corrected values of N_{TCP}. That is, Eq. 14 should be applied to the measured values of N_{TCP} when the soil is a very fine or silty saturated sand with a measured value of N_{TCP} greater than 30. Also, the new curve should only be used for SP, SM, and SP-SM soils.

There is a need for additional data from other test sites. These new test sites should contain soil types not tested in this study. This would make it possible to develop curves for a more complete range of soil types.

No final correlations are recommended relating unit side friction and unit point bearing with N_{TCP} for driven and bored piles. Additional data needs to be added to the data used in this study. The addition of more data from instrumented piles might make it possible to predict the bearing capacity of piles directly from the results of the Texas Cone Penetrometer Test.

APPENDIX I. - REFERENCES

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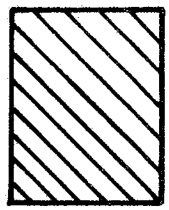
APPENDIX II- NOTATION

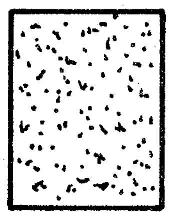
The Symbols Used on Borings Logs Are:

Soil Type

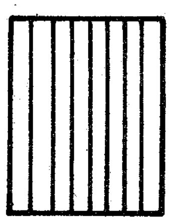
Clay



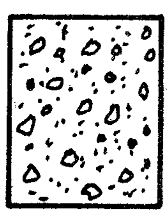
Sand



Silt



Fill



The following symbols are used in this paper:

A_c = the corrected area in square feet;

c' = effective cohesion, in tons per square foot;

c_u = unconsolidated-undrained shear strength; in tons per square foot;

$c_u(ASTM)$ = unconsolidated-undrained shear strength as determined by the ASTM Triaxial Test, in tons per square foot;

$c_u(TAT)$ = unconsolidated-undrained shear strength as determined by the Texas Triaxail Test, in tons per square foot;

D_e = inside diameter of sample tube;

D_w = outside diameter of sample tube;

f = unit side friction capacity of a pile, in tons per square foot;

N = the number of blows required to drive a penetrometer one foot;

N_{SPT} = the measured number of blows required to drive the standard split spoon one foot;

N'_{SPT} = the corrected number of blows required to drive the standard split spoon one foot;

N_{TCP} = the measured number of blows required to drive the Texas Cone Penetrometer one foot;

N'_{TCP} = the corrected number of blows required to drive the Texas Cone Penetrometer one foot;

p' = effective overburden pressure, in tons per square foot;

P_m = the sum of the vertical load induced by the confining pressure and the applied vertical load, in tons;

P_v = the deviator stress, in tons;

P_{ult} = the ultimate bearing capacity of a pile; in tons;

q = unit point bearing capacity of a pile, in tons per square foot;

s = drained shear strength, in tons per square foot;

S = design stress or ½ shear strength, in tons per square foot;

γ_T = total unit weight, in pounds per cubic foot;

ϕ' = effective angle of shearing resistance, in degrees;

σ_c = the confining pressure, in tons per square foot;

$\sigma\eta'$ = effective normal stress, in tons per square foot.

APPENDIX III

SUMMARY OF PORT ARTHUR TEST DATA

TABLE. SUMMARY OF TEST RESULTS

			4	5	8	10	11	12	13
SAMPLE NUMBER AND SITE			4	5	8	10	11	12	13
PENETRATION, FT			19-20	21-21.5	26-27	31-32	35-36	36-37	38-39
PENETRATION RESISTANCE, N*			5	7	12	12	15	16	17
CLASSIFICATION TESTS	Liquid Limit, %		45.9	45.7	72.4	42.9	64.0	63.9	80.0
	Plastic Limit, %		19.3	18.8	26.7	21.5	22.2	27.0	29.0
	Plasticity Index, %		26.6	26.9	45.7	21.4	41.8	36.9	51.1
	Percent Passing No. 200 Sieve		78.3	80.4	95.1	99.6	98.9	99.9	--
	Unified Classification		CL	CL	CH	CL	CH	CH	CH
	Subgroup		Si	Si	H	Si	H	H	H
TRIAXIAL COMPRESSION	Type of Test		1	3	--	1	1	3	--
	WATER CONTENT	Initial	25.8	22.1	33.9	34.3	47.5	31.4	26.7
		Final	27.3	21.4	29.5	31.4	33.7	34.0	26.5
	Total Unit Wt.lb/ft^3		121.1	129.2	119.2	128.0	118.7	119.2	--
	Cohesion, ton/ft^2		1.38	1.18	--	1.62	1.51	1.03	--
	Lateral Pressure, PSI		8.5	9.0	--	12.8	14.5	14.8	--
OTHER SOIL PROPERTIES	Specific Gravity								
	Percent Saturation								

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained ASTM Triaxial

N* = Blow count for twelve inches penetration
(1 PSI = 6.9 KN/M^2; 1 PCF = 16.01 kg/M^3; 1 ft. = .305 m)

SITE:
Port Arthur, Texas

TABLE. SUMMARY OF TEST RESULTS

			15	16	18	20	21	23	24
SAMPLE NUMBER AND SITE			15	16	18	20	21	23	24
PENETRATION, FT			40-40.5	40.5-41	42-43	45-46	46-47	49-50	49-50
PENETRATION RESISTANCE, N*			16	16	15	13	13	13	13
CLASSIFICATION TESTS	Liquid Limit, %		85.9	86.7	101.4	92.9	63.1	49.0	47.9
	Plastic Limit, %		37.2	36.8	33.6	35.0	22.9	20.4	20.7
	Plasticity Index, %		48.7	49.9	67.8	57.9	40.1	28.6	27.1
	Percent Passing No. 200 Sieve		94.1	99.8	88.8	97.2	92.7	73.6	72.2
	Unified Classification		CH	CH	CH	CH	CH	CL	CL
	Subgroup		H	H	H	H	H	Si	Si
TRIAXIAL COMPRESSION	Type of Test		1	3	1	1	3	1	1
	WATER CONTENT	Initial	47.5	32.7	32.9	51.6	38.5	32.3	23.1
		Final	47.0	45.1	33.9	51.6	38.3	32.3	23.8
	Total Unit Wt. lb/ft^3		109.3	111.8	108.6	108.0	116.7	116.4	121.7
	Cohesion, ton/ft^2		1.76	.99	1.75	1.81	.98	1.91	1.49
	Lateral Pressure, PSI		16.2	16.5	17.3	18.3	18.8	19.8	20.0
OTHER SOIL PROPERTIES	Specific Gravity								
	Percent Saturation								

<u>Legend and Notes</u>

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained ASTM Triaxial

N* = Blow count for twelve inches penetration
(1 PSI - 6.9 KN/M^2; 1 PCF = 16.01 kg/M^3;
1 ft. = .305 m)

SITE:

Port Arthur, Texas

TABLE. SUMMARY OF TEST RESULTS

			25	26	27	31	35	39	40
SAMPLE NUMBER AND SITE			25	26	27	31	35	39	40
PENETRATION, FT			50-51	51-52	52-53	57-58	60-61	64-65	65-66
PENETRATION RESISTANCE, N*			13	13	13	16	19	17	17
CLASSIFICATION TESTS	Liquid Limit, %		55.4	78.4	61.58	79.7	74.3	72.7	75.6
	Plastic Limit, %		21.0	24.0	21.2	26.5	26.7	24.8	27.0
	Plasticity Index, %		34.4	54.4	40.4	53.3	47.6	48.0	49.0
	Percent Passing No. 200 Sieve		81.6	93.1	90.8	95.9	99.4	96.7	97.2
	Unified Classification		CH	CH	CH	CH	CH	CH	CH
	Subgroup		H	H	H	H	H	H	H
TRIAXIAL COMPRESSION	Type of Test		3	1	1	1	1	3	1
	WATER CONTENT	Initial	25.5	29.3	21.5	25.2	36.8	30.8	30.0
		Final	27.11	28.1	21.5	23.0	33.1	32.4	29.3
	Total Unit Wt.lb/ft^3		123.0	122.4	124.2	120.5	119.9	118.6	121.1
	Cohesion, ton/ft^2		.94	1.84	2.38	2.34	2.44	1.13	2.81
	Lateral Pressure, PSI		20.5	21.0	21.5	23.5	25.0	27.0	27.5
OTHER SOIL PROPERTIES	Specific Gravity								
	Percent Saturation								

Legend and Notes

1 = Unconsolidated-undrained Texas Triaxial
2 = Unconsolidated-undrained ASTM Triaxial

N* = Blow count for twelve inches penetration

(1 PSI = 6.9 KN/M^2; 1 PCF - 16.01 kg/M^3; 1 ft. = .305 m)

SITE:
Port Arthur, Texas

APPENDIX IV

SUMMARY OF CORPUS CHRISTI TEST DATA

TABLE - Summary of Tests Results								
Site and Sample Number		S-1	a	b	c	S-2	a	b
Depth (ft)		4-5				9-10		
Penetration Resistance, N		5				2.3		
Classification Tests	Percent Passing No. 200 Sieve	9.0				29.6		
	Uniformity Coef., C_u							
	Curvature Coef., C_c							
	Plastic Limit							
	Liquid Limit							
	Unified Classification	SP-SM				SM		
Direct Shear Test	Shear Strength at Failure (psi)		9.52	16.44	21.20		6.19	10.38
	Moisture Content: Before test (%)		24.5	23.5	22.9		23.3	23.2
	Moisture Content: After test (%)		23.3	24.2	23.7		24.3	38.9
	Unit Weight[1] (pcf)		116.2	113.1	114.9		116.2	105.6
	Angle of Internal Friction	38.7				31.3		
Total Unit Weight[2] (pcf)		125.5				118.6		

Notes

a = Normal Stress = 10 psi
b = Normal Stress = 20 psi
c = Normal Stress = 30 psi
1 = Measured in Shear Box
2 = Measured in Sample Tube

Site

Corpus Christi, Texas

(1 psi = 6.9 KN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

TABLE - Summary of Tests Results

		S-3	a	b	S-4	a	b	c
Site and Sample Number		S-3	a	b	S-4	a	b	c
Depth (ft)		12.5-13.5			19-20			
Penetration Resistance, N		0			41			
Classification Tests	Percent Passing No. 200 Sieve	82.2			19.0			
	Uniformity Coef., C_u	43.2						
	Curvature Coef., C_c	22.7						
	Plastic Limit							
	Liquid Limit							
	Unified Classification	CL			SM			
Direct Shear Test	Shear Strength at Failure (psi)					8.65	13.84	22.06
	Moisture Content: Before test (%)		40.7	54.5		19.3	21.6	20.7
	Moisture Content: After test (%)						18.7	18.2
	Unit Weight[1] (pcf)					121.2	122.4	122.4
	Angle of Internal Friction				36.3			
Total Unit Weight[2] (pcf)		104.3			131.1			

Notes

a = Normal Stress = 10 psi
b = Normal Stress = 20 psi
c = Normal Stress = 30 psi
1 = Measured in Shear Box
2 = Measured in Sample Tube

Site

Corpus Christi, Texas

(1 psi = 6.9 KN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

TABLE - Summary of Tests Results								
Site and Sample Number		S-6	a	b	c	S-7	a	b
Depth (ft)		22.5-23.5				25-26		
Penetration Resistance, N		53				49		
Classification Tests	Percent Passing No. 200 Sieve	13.8				12.8		
	Uniformity Coef., C_u							
	Curvature Coef., C_c							
	Plastic Limit							
	Liquid Limit							
	Unified Classification	SM				SM		
Direct Shear Test	Shear Strength at Failure (psi)		7.52	18.14	25.22		7.96	13.72
	Moisture Content: Before test (%)		20.0	17.2	16.5		21.1	20.2
	Moisture Content: After test (%)		16.9	18.2	16.6		20.3	20.7
	Unit Weight[1] (pcf)		122.4	118.7	117.4		121.2	118.7
	Angle of Internal Friction	41.01				38.5		
Total Unit Weight[2] (pcf)		133.0				133.6		

Notes

a = Normal Stress = 10 psi
b = Normal Stress = 20 psi
c = Normal Stress = 30 psi
1 = Measured in Shear Box
2 = Measured in Sample Tube

Site

Corpus Christi, Texas

(1 psi = 6.9 KN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

TABLE - Summary of Tests Results								
Site and Sample Number		S-7c	S-8	a	b	c		
Depth (ft)			27.5-28.5					
Penetration Resistance, N			26					
Classification Tests	Percent Passing No. 200 Sieve		13.7					
Classification Tests	Uniformity Coef., C_u							
Classification Tests	Curvature Coef., C_c							
Classification Tests	Plastic Limit							
Classification Tests	Liquid Limit							
Classification Tests	Unified Classification		SM					
Direct Shear Test	Shear Strength at Failure (psi)	23.89		6.64	14.16	19.91		
Direct Shear Test	Moisture Content: Before test (%)	19.4		22.9	23.9	21.9		
Direct Shear Test	Moisture Content: After test (%)	19.2		21.6	22.0	19.9		
Direct Shear Test	Unit Weight[1] (pcf)	119.9		122.4	123.6	124.9		
Direct Shear Test	Angle of Internal Friction		34.0					
Total Unit Weight[2] (pcf)			127.4					

Notes

a = Normal Stress = 10 psi
b = Normal Stress = 20 psi
c = Normal Stress = 30 psi
1 = Measured in Shear Box
2 = Measured in Sample Tube

Site

Corpus Christi, Texas

(1 psi = 6.9 KN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

TABLE - Summary of Tests Results

		Site and Sample Number	S-9	a	b	c	S-10	a	b
		Depth (ft)	30-31				32.5-33.5		
		Penetration Resistance, N	24				44		
Classification Tests		Percent Passing No. 200 Sieve	8.5				13.9		
Classification Tests		Uniformity Coef., C_u							
Classification Tests		Curvature Coef., C_c							
Classification Tests		Plastic Limit							
Classification Tests		Liquid Limit							
Classification Tests		Unified Classification	SP-SM				SM		
Direct Shear Test		Shear Strength at Failure (psi)		7.52	15.49	19.02		6.19	12.39
Direct Shear Test	Moisture Content	Before test (%)		25.1	24.6	27.1		24.2	26.5
Direct Shear Test	Moisture Content	After test (%)		25.7	25.8	27.7		25.2	32.5
Direct Shear Test		Unit Weight[1] (pcf)		113.0	113.6	113.0		113.0	113.0
Direct Shear Test		Angle of Internal Friction	35.5				32.5		
		Total Unit Weight[2] (pcf)	123.0				123.0		

Notes

a = Normal Stress = 10 psi
b = Normal Stress = 20 psi
c = Normal Stress = 30 psi
1 = Measured in Shear Box
2 = Measured in Sample Tube

Site

Corpus Christi, Texas

(1 psi = 6.9 KN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

TABLE - Summary of Tests Results							
Site and Sample Number		S-10c		S-11	a	b	c
Depth (ft)				35-36			
Penetration Resistance, N				56			
Classification Tests	Percent Passing No. 200 Sieve			22.2			
	Uniformity Coef., C_u						
	Curvature Coef., C_c						
	Plastic Limit						
	Liquid Limit						
	Unified Classification			SM			
Direct Shear Test	Shear Strength at Failure (psi)	20.35			8.41	21.2	--
	Moisture Content: Before test (%)	25.5			28.5	22.4	23.1
	Moisture Content: After test (%)	29.1			28.6	23.8	24.1
	Unit Weight[1] (pcf)	111.7			117.4	117.4	--
	Angle of Internal Friction			45			
Total Unit Weight[2] (pcf)				124.9			

Notes

a = Normal Stress = 10 psi
b = Normal Stress = 20 psi
c = Normal Stress = 30 psi
1 = Measured in Shear Box
2 = Measured in Sample Tube

Site

Corpus Christi, Texas

(1 psi = 6.9 KN/m^2; 1 pcf = 16.01 kg/m^3; 1 ft. = .305 m)

APPENDIX V

SUMMARY OF PILE DATA

TABLE - SUMMARY OF TEST PILE G1

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE f(TSF) MT	COHESIVE f(TSF) AL	COHESIVE q(TSF) MT	COHESIVE q(TSF) AL	COHESIONLESS f(TSF) MT	COHESIONLESS f(TSF) AL	COHESIONLESS q(TSF) MT	COHESIONLESS q(TSF) AL	N-value N_{TCP}	N-value N'_{TCP}
0	⊠										
		.32	.37							12	
23	⊠										
		.83	.80							23	
36	⊠										
						1.15	1.15			57	44
57	⊠										
								2.83	2.40	141	90
61.8	⊠										

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE G2

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE f(TSF) MT	COHESIVE f(TSF) AL	COHESIVE q(TSF) MT	COHESIVE q(TSF) AL	COHESIONLESS f(TSF) MT	COHESIONLESS f(TSF) AL	COHESIONLESS q(TSF) MT	COHESIONLESS q(TSF) AL	N-VALUE N_{TCP}	N-VALUE N'_{TCP}
3	⊠										
		1.22	1.17							9	
15	⊠										
		.84	.81							25	
56	⊠										
					1.45	1.51				39	35
78	⊠							3.05	14.26	31	75

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE BB

DEPTH (FT.)	STRAIN GAGE LOCATION	COHESIVE				COHESIONLESS				N-VALUE	
		f(TSF)		q(TSF)		f(TSF)		q(TSF)		N_{TCP}	N'_{TCP}
		MT	AL	MT	AL	MT	AL	MT	AL		
6	⊠										
		.95	1.16							27	
30	⊠										
						1.76	1.84			199	115
48	⊠							33.0	55.0	229	130

MT - Method of Tangents
AL - Maximum Applied Load Method
(1 ft = .305 m, 1 tsf = 9.58×10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE LB

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE f(TSF) MT	COHESIVE f(TSF) AL	COHESIVE q(TSF) MT	COHESIVE q(TSF) AL	COHESIONLESS f(TSF) MT	COHESIONLESS f(TSF) AL	COHESIONLESS q (TSF) MT	COHESIONLESS q (TSF) AL	N-VALUE N_{TCP}	N-VALUE N'_{TCP}
		.32	.30							25	
32	⊠										
		1.53	1.66							143	
42	⊠			38.7	44.8					155	

MT - Method of Tangents
AL - Maximum Applied Load Method
(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE US59

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE				COHESIONLESS				N-VALUE	
		f(TSF)		q(TSF)		f(TSF)		q(TSF)		N_{TCP}	N'_{TCP}
		MT	AL	MT	AL	MT	AL	MT	AL		
		.59	.79							18	
15	⊠										
25	⊠					2.27	2.39			115	73
25.4	G.W.T. NOT ENCOUNTERED							33.6	67.2	275	153

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft = .305 m; 1 tsf = 9.58×10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE HH

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE				COHESIONLESS				N-value	
		f(TSF)		q(TSF)		f(TSF)		q(TSF)			
		MT	AL	MT	AL	MT	AL	MT	AL	N_{TCP}	N'_{TCP}
17	⊠	3.14	3.09							44	
						3.45	3.45			213	122
20	⊠							27.1	31.9	300	165
	G.W.T. NOT ENCOUNTERED										

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft - .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE SITI

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE f (TSF) MT	COHESIVE f (TSF) AL	COHESIVE q (TSF) MT	COHESIVE q (TSF) AL	COHESIONLESS f (TSF) MT	COHESIONLESS f (TSF) AL	COHESIONLESS q (TSF) MT	COHESIONLESS q (TSF) AL	N-VALUE N_{TCP}	N-VALUE N'_{TCP}
		.38	.21							9	
8	⊠										
		.53	.63							23	
23				2.44	10.79					28	

MT - Method of Tangents
AL - Maximum Applied Load Method
(1 ft = .305 m; 1 tsf = 9.58×10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE S2T1

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE				COHESIONLESS				N-VALUE	
		f (TSF)		q (TSF)		f (TSF)		q (TSF)		N_{TCP}	N'_{TCP}
		MT	AL	MT	AL	MT	AL	MT	AL		
		.64	.88							9	
8	⊠										
		.61	.42							20	
18.5	⊠										

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft = .305 m; 1 tsf = 9.58×10^2 N/m^2)

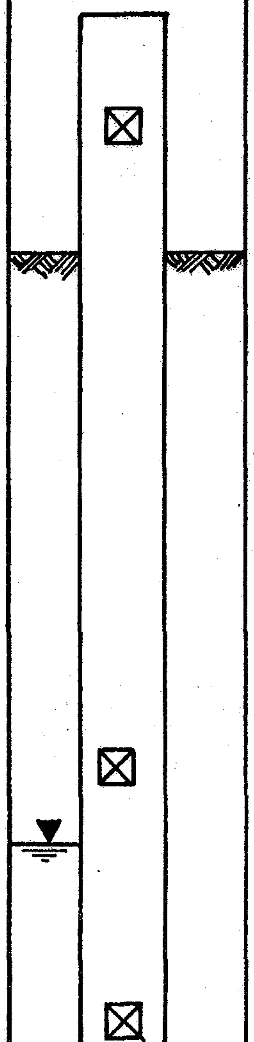
TABLE - SUMMARY OF TEST PILE H-99R											
DEPTH (FT.)	STRAIN GAGE LOCATION	COHESIVE				COHESIONLESS				N-VALUE	
		f(TSF)		q(TSF)		f(TSF)		q(TSF)		N_{TCP}	N'_{TCP}
		MT	AL	MT	AL	MT	AL	MT	AL		
2.5	⊠										
4.8											
		.368	.328							12.4	
9.6	⊠										
						1.41	1.40			29.1	
18.4	⊠							58.8	62.4	32	31

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft = .305 m; 1 tsf = 9.58 x 10^2 N/m^2)

TABLE - SUMMARY OF TEST PILE H-4L

DEPTH (FT)	STRAIN GAGE LOCATION	COHESIVE				COHESIONLESS				N-VALUE	
		f(TSF)		q(TSF)		f(TSF)		q(TSF)			
		MT	AL	MT	AL	MT	AL	MT	AL	N_{TCP}	N'_{TCP}
											
4.8											
		.119	.140							20.6	
15.4											
						1.30	1.30			25	
21.1											
								38.1	44.4	28	

MT - Method of Tangents

AL - Maximum Applied Load Method

(1 ft = .305 m; 1 tsf = 9.58×10^2 N/m^2)

NOTES