TTI-2-5-74-10-2

TEXAS
TRANSPORTATION
INSTITUTE

STATE DEPARTMENT
OF HIGHWAYS AND
PUBLIC TRANSPORTATION

COOPERATIVE
RESEARCH

CORRELATION OF THE TEXAS HIGHWAY DEPARTMENT CONE PENETROMETER TEST WITH THE DRAINED SHEAR STRENGTH OF COHESIONLESS SOILS

in cooperation with the
Department of Transportation
Federal Highway Administration

RESEARCH REPORT 10-2
STUDY 2-5-74-10
THD CONE PENETROMETER TEST

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	
4. Title and Subtitle CORRELATION OF THE TEXAS HIGHWAY DEPARTMENT CONE PENETROMETER TEST WITH THE DRAINED SHEAR STRENGTH OF COHESIVE SOILS		5. Report Date August, 1975
		6. Performing Organization Code
7. Author(s) George D. Cozart, Harry M. Coyle, and Richard E. Bartoskewitz		8. Performing Organization Report No. Reserach Report 10-2
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 Interim - September 1973 - August 1975
		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 Highway Department (THD) Cone Penetrometer Test N-value and the drained shear strength of cohesionless soils. Cone penetrometer test data and undisturbed sand samples were obtained at five different test sites. To develop the correlations new techniques in sampling and testing of cohesionless soils were implemented.

From the results of field and laboratory investigations reasonably good correlations were developed for both drained shear strength, s, and effective overburden pressure, p', with the THD Cone Penetrometer Test N-value. A trend was noted in the relationship between total unit weight, γ, and the N-value. The relationship currently in use by the THD between the effective angle of internal friction, ϕ', and the N-value was found to be a lower bound for the data obtained from this study. An attempt to determine the effects of individual factors upon the N-value resulted in the conclusion that an interaction of many factors influences the resistance to penetrometer penetration.

17. Key Words	18. Distribution Statement
THD Cone Penetrometer Test, N-value, Cohesionless Soils, Drained Shear Strength, Effective Overburden Pressure, Total Unit Weight, Effective Angle of Internal Friction.	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	83	

Form DOT F 1700.7 (8-69)

CORRELATION OF THE TEXAS HIGHWAY DEPARTMENT CONE PENETROMETER TEST WITH THE DRAINED SHEAR STRENGTH OF COHESIONLESS SOILS

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Sponsored by the
State Department of Highways and Public Transportation
in Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

August 1975

TEXAS TRANSPORTATION INSTITUTE
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ABSTRACT

Improved correlations have been developed between the Texas Highway Department (THD) Cone Penetrometer Test N-value and the drained shear strength of cohesionless soils. Cone penetrometer test data and undisturbed sand samples were obtained at five different test sites. To develop the correlations new techniques in sampling and testing of cohesionless soils were implemented.

From the results of field and laboratory investigations reasonably good correlations were developed for both drained shear strength, s, and effective overburden pressure, p', with the THD Cone Penetrometer Test N-value. A trend was noted in the relationship between total unit weight, γ, and the N-value. The relationship currently in use by the THD between the effective angle of internal friction, ϕ', and the N-value was found to be a lower bound for the data obtained from this study. An attempt to determine the effects of individual factors upon the N-value resulted in the conclusion that an interaction of many factors influences the resistance to penetrometer penetration.

KEY WORDS: THD Cone Penetrometer Test, N-value, Cohesionless Soils, Drained Shear Strength, Effective Overburden Pressure, Total Unit Weight, Effective Angle of Internal Friction.

SUMMARY

The information presented in this report was developed during the second year of a three-year study on the determination of in-situ soil shear strength by means of dynamic sub-surface sounding tests. During the first year of this study improved correlations were developed between the THD Cone Penetrometer Test N-value and the unconsolidated-undrained shear strength of homogeneous CH, silty CL and sandy CL soils. The objective of this 2nd year phase of the study is to develop improved correlations between the THD Cone Penetrometer Test N-value and the drained shear strength of cohesionless soils.

The equipment and techniques used to recover undisturbed samples of cohesionless soil are presented together with a description of the THD Cone Penetrometer Test. The sampling technique consists of using 1.6-in. diameter sampling tubes which vary in length from 8 to 12-in. The sampling tubes are designed to be attached directly to the end of standard THD drill stem with no additional equipment required. The use of this technique permits the recovery of relatively undisturbed samples in all but very loose and highly saturated sands.

The laboratory test methods used to determine various soil properties, and the soil conditions as determined from these laboratory tests, are described for each test site. The direct shear test is used to determine the drained shear strength. The sand samples are extruded directly from the sampling tube into the direct shear box, thereby minimizing sample disturbance. The laboratory direct shear test equipment is a commercially available item with the exception of the shear box which was specially built by TTI to fit the small diameter samples.

The factors reported to have the most affect upon the resistance to penetration are analysed. Correlations are developed for some of these factors (i.e. unit weight, effective overburden pressure and effective angle of internal friction). The drained shear strength of SP, SM, and SP-SM soils is correlated with the THD Cone Penetrometer Test N-value.

IMPLEMENTATION STATEMENT

The correlation of the drained shear strength of cohesionless soils with the THD Cone Penetrometer Test N-value developed in this study can be incorporated in the Texas Highway Department Foundation Exploration and Design Manual (Bridge Division). No change is recommended in the relationship currently in use by the THD between the angle of internal friction and the N-value. The relationship presented in this study between total unit weight and the N-value is not recommended for implementation owing to excessive scatter.

Implementation of the findings of this study should be limited to soils possessing physical properties similar to soils for which the correlations were developed, i.e., SP, SM and SP-SM classification. Since the correlations are based on a limited amount of data they should be subject to change upon the addition of new data.

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INTRODUCTION

Present Status of the Problem.-- Soil sounding or probing consists of forcing a rod into the soil and observing the resistance to penetration. According to Hvorslev (14)*, "variation of this resistance indicates dissimiliar soil layers, and numerical values of this resistance permit an estimate of some of the physical properties of the strata". The oldest and simplest form of soil sounding consists of driving a rod into the ground by repeated blows of a hammer. The penetration resistance of this dynamic test is the number of blows, N, to produce a penetration of one foot.

In the United States the most widespread dynamic penetration test is the Standard Penetration Test (SPT). The results of the SPT can usually be correlated in a general way with the pertinent physical properties of sand. Meigh and Nixon (18) 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. Research conducted by Gibbs and Holtz (13) indicates a definite relationship exists between the N-value as determined from the SPT and the relative density of 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 (19).

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

The Texas Highway Department (THD) currently uses a penetration test similiar to the SPT for investigation of foundation materials encountered in bridge foundation exploration work. The penetration test is especially applicable in investigations of cohesionless soils because of difficulties in obtaining undisturbed samples for laboratory testing. According to the THD Foundation Manual (4) "the design of foundations in cohesionless soils is generally based upon visual classification and penetrometer test data." Penetrometer test N-values have been correlated with the shear strength of CH, CL, and SC soils based on the results of triaxial tests. Correlations developed for the THD Cone Penetrometer Test with the shear strength of SP and SM soils would result in improved design procedures for foundations in cohesionless soils.

Objective.-- The objective of this research effort is to develop an improved correlation between the N-value (blows/ft) obtained from the THD Cone Penetrometer Test and the drained shear strength of cohesionless soils. Correlations will be developed for the following soil types as defined by the Unified Soil Classification system:

SP - Poorly graded sands, gravelly sands, little or no fines

SM - Silty sands, poorly graded sand silt mixture.

SAMPLING PROGRAM

In order to correlate shear strength with the THD Cone Penetrometer Test N-value, undisturbed sand samples and penetration tests were needed at corresponding depths at the same test site. A sampling procedure was required in which a relatively large number of samples could be recovered and tested with minimal disturbance.

Development of Sampling Equipment.-- Before developing a sampling procedure, previously used methods of undisturbed sampling of cohesionless soils were investigated. Methods such as solidification of the lower end of the sample by chemical injection or freezing (11), solidification of the sand before sampling by asphalt injection, or freezing the ground by the use of a cooling mixture in auxiliary pipes (14) do not always produce undisturbed samples and are very elaborate and expensive. Also, according to Bishop (1), mechanical core retainers, such as utilized in the Denison Sampler, cause excessive disturbance in clean sands.

With the aid of THD personnel a sampling apparatus similiar to a small diameter Shelby tube sampler was developed. As seen in Fig. 1 the sampling device consisted of a thin walled sampler with a coupling head which adapts the sampler to the drilling rod. A check valve in the coupling head allowed the escape of drilling fluid while lowering the sample tube to the bottom of the borehole and prevented the water pressure in the drilling rod from forcing the sample out of the sampler during extraction. Two vent holes were provided above the check valve to allow the drilling fluid to drain from the drilling rod while the sample tube was being extracted from the bore hole.

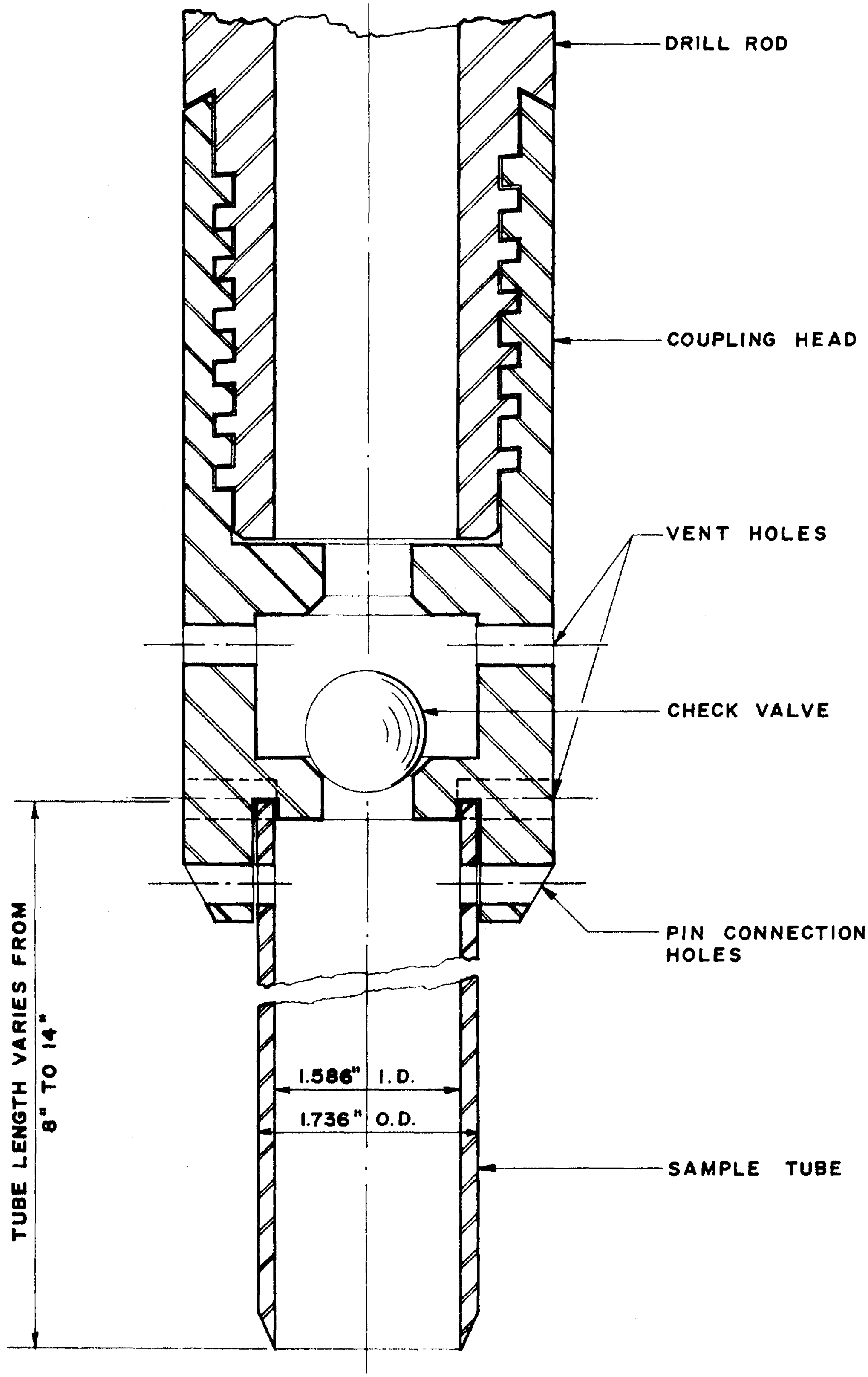


FIG. 1. - CROSS SECTION OF SAMPLING APPARATUS.
(1.0 in = 25.4 mm)

Both stainless and galvanized steel sample tubes were used with an outside diameter of 1.736 in. (44.09 mm) and a wall thickness of .075 in. (1.91 mm). According to Hvorslev (14), for minimum disturbance, the area ratio of the sampler should preferably not exceed 10 to 15 percent as computed by the formula:

$$\text{Area Ratio} = \frac{\text{volume of displaced soil}}{\text{volume of soil}} = \frac{D_w^2 - D_e^2}{D_e^2} \quad . . .(1)$$

where: D_w = outside diameter of sample tube

D_e = inside diameter of sample tube

The area ratio of the chosen sampler was 9.23 percent, thus meeting Hvorslev's requirements for minimizing disturbance. The inside friction could have been reduced by making the diameter of the cutting edge slightly smaller than the inside diameter. However, Hvorslev states that for short samples this is not necessary. Also, Hvorslev recommends that a detachable shoe and cutting edge not be used to reduce the outside wall friction in cohesionless soils. As a result of a preliminary field study the 10 in. (254 mm) and 12 in. (304.8 mm) samplers were found to permit the best recovery.

Sampling Procedure.-- A truck mounted Failing 1500 rotary core drilling rig was used to make each boring. When advancing the hole through cohesive material continuous Shelby tube samples were taken and selected samples were kept for visual observation and unit weight determination. Once the sand stratum was encountered in which undisturbed samples were to be taken, cuttings were removed by washing through the Shelby tube. The small diameter sampler and coupling head were then attached to the drilling rod. The sampler was pushed in a rapid continuous motion with a hydraulically powered pull-down. After extraction from the bore hole the sampler was removed from the coupling head and the cuttings at the top of the sample tube were observed. Any indication of overpushing was recorded along with sample depth and visual classification. The sample tube was sealed on each end, covered with paraffin, and packaged for transportation to the soils laboratory.

SOIL CONDITIONS AT TEST SITES

Site Selection.-- To find suitable sand deposits for undisturbed sampling and penetrometer testing, a preliminary site investigation was undertaken. Soil Conservation Service Soil Surveys (27) and THD boring logs were used to locate potential test sites in Brazos and Harris Counties.

After locating a number of potential test sites, hand auger samples were taken and classified according to the Unified Soil Classification System. Many of the potential sites were eliminated owing to an excessive amount of fine grained material or to erratic stratigraphy which was undesirable for correlation purposes. The preliminary site investigation resulted in the selection of five test sites; sites A, B, and C in Brazos County and sites D and E in Harris County.

THD Cone Penetrometer Test.-- After obtaining undisturbed sand samples by the method outlined earlier the THD Cone Penetrometer Test was performed at corresponding depths at each test site. The penetrometer tests were conducted in new boreholes located not more than 10 ft. (3.05 m) from the holes wherein the soil samples were obtained.

The procedure used to obtain the resistance to penetration, N-values, is described in detail in the THD Foundation Manual (4). The test consists of driving the three inch (76.2 mm) diameter cone shown in Fig. 2, with a 170 lb. (74 Kg) hammer which is dropped a regulated two feet (.61 m). According to the THD Manual, "In relatively soft materials, the penetrometer cone should be driven one foot and the

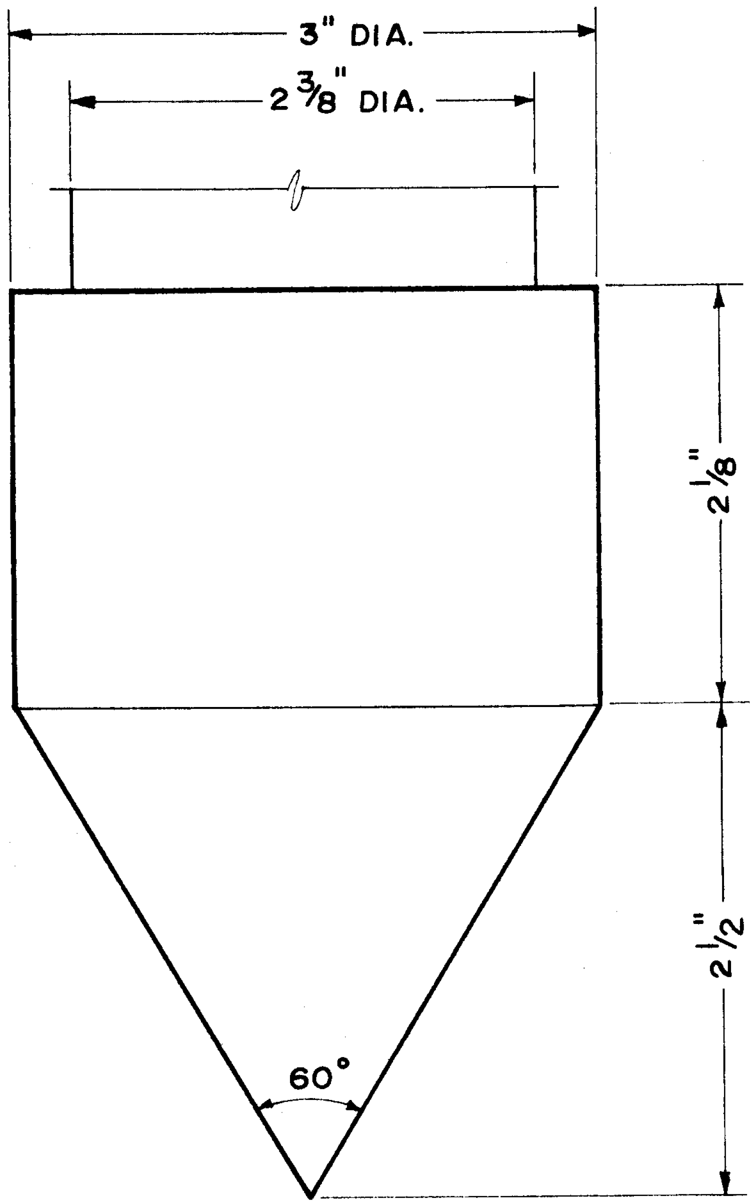


FIG. 2.-DETAILS OF THD CONE PENETROMETER.
(1.0 IN. = 25.4 mm)

number of blows for each 6 in. (150 mm) increment noted on the log. In hard materials, including rock, the penetrometer cone is driven with the resulting penetration in inches accurately recorded for the first and second 50 blows for a total of 100 blows." Although the specifications require the penetrometer to "be driven 12 blows in order to seat it in the soil or rock" (4), the penetrometer was seated a maximum of 12 blows or 4-in.,(102 mm) whichever occurred first. Thus, when possible, samples and penetrometer test data were obtained every 2.5 ft. (.76 m). This allowed the penetrometer to be seated 4 in. (102 mm), driven 1 ft. (.305 m) and the disturbed material removed to a depth of 1 ft. (.305 m) below the penetrated soil.

Soil Properties Other than Shear Strength.-- The soil properties other than shear strength are included on the boring logs for each boring. Symbols used on the borings logs to indicate soil and sample types are summarized in Appendix II.

Since the bore holes were advanced with a three inch (76 mm) Shelby tube sampler, samples could be kept for unit weight determination of cohesive soils when there was an indication of change in soil properties. The unit weights were determined from the Shelby tube samples in the conventional manner. Moisture contents for all of the soil samples in this study were determined in accordance with the THD Manual of Testing Procedures (25), Test Method Tex-103-E.

Unified Soil Classification, moisture content, and total unit weight of the cohesionless soils were determined from the small diameter samples. In order to determine the Unified Soil Classification the percent passing the number 200 sieve, mechanical analysis, and Atterberg limits were needed. The method used for each test is summarized in Table 1.

TABLE 1.-- Test Methods According to the THD Manual of Testing Procedures

Type of Test	Test Method Used
Moisture Content	TEX 103-E
Plastic Limit	TEX 104-E
Liquid Limit	TEX 105-E
Mechanical Analysis	TEX 110-E
Percent Passing No. 200 Sieve	TEX 111-E

The total unit weight of the sand samples was determined from measurements while the samples were in the sample tubes. All soil properties are tabulated in Appendix III for each sample tested.

The ground water level was measured at test sites A, B, and C in the open boreholes approximately 48 hours after drilling and again after two weeks. No appreciable variation in the ground water level was found after the two week period. At site D the ground water level 24 hours after drilling was 14 ft. (4.3m) below the ground level. This was in fairly close agreement with the THD boring logs for borings made earlier on April 30, 1973. The bore hole at test site E could not be left open. However, according to THD personnel, recent borings in the area indicated the ground water level to be 18 ft. (5.5 m) below the ground surface.

Summary of Data for Test Sites.-- Test site A is located in a borrow area along State Highway 30 approximately 4.8 miles (7.7 km) east of the intersection of State Highway 30 and Farm-to-Market road 158 in Brazos County. This site was approximately the same location as the test sites reported by Coyle and Wright (5) and Dunlap and Ivey (10). Logs of the three borings made at test site A are shown in Fig. 3, 4, and 5.

The underlying materials at borings 2 and 3 were mostly light gray, poorly graded, fine silty sands A range of penetrometer test values of 4-20 blows per foot indicates a relatively loose material at borings 2 and 3 whereas higher values of 23-60 blows per foot at boring 1 indicates a denser material. A layer of sandy clay was encountered at boring 1 from 3 to 6.5 ft. (.92 to 1.98 m) below the ground surface which was not present at borings 2 or 3. At depths of around 16 ft. (4.88 m) the sand became very loose and wet and could not be recovered with the small diameter sampler. Below this loose material, a light gray stiff silty clay was encountered at each boring.

Test site B was located near the overpass of Briarcrest Drive at State Highway 6 in Bryan, Texas. The log of the boring made at site B is shown in Fig. 6. The sand to be sampled was overlain by alternating layers of sandy clay and silty to clayey sand. A sandy clay fill at the surface overlaid a hard silty sand of very low moisture content and correspondingly high unit weight. A 2 ft. (.61 m) layer of plastic sandy clay was encountered at 4 ft. (1.20 m) which became more firm and clayey at 6 ft. (1.80 m). A uniform deposit of firm silty sand was encountered at 8 ft. (2.40 m). The sample taken at 10 ft. (3.10 m) indicated 25.7 percent of the material passed the number of 200 sieve. At 13.5 ft. (4.12 m) the sand became less silty,

DEPTH, FEET	SOIL SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N-VALUE BLOWS PER FT.
1			BROWN LOOSE					
2			SILTY SAND			8.7	125.2	
3								
4			RED AND BROWN PLASTIC SANDY					
5			CLAY			31.4	119.7	
6			▽			40.4	110.2	
7								
8			LIGHT TAN FIRM SILTY SAND			21.4	102.0	23
9								
10								
11				SP-SM	8.6	20.1	111.4	35
12								
13								
14				SP-SM	10.5	23.6	118.6	60
15			LIGHT GRAY LOOSE					
16			SAND	—	—	10.3	124.6	10
17			TAN AND LIGHT GRAY FIRM SILTY					
18			CLAY					
19								
20								
21								

FIG. 3.- LOG OF BORING I SITE A - STATE HIGHWAY 30.
(1 ft = .305 m; 1 pcf = 16.01 kg/m^3)

DEPTH, FEET	SOIL SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N VALUE BLOWS PER FT.
0–17			LIGHT GRAY LOOSE SILTY SAND					
6		X		SM	13.4	12.6	104.3	4
8		X		SM	13.4	13.8	106.6	5
10		X		SM	16.3	26.0	111.6	9
15		X		—	—	18.0	115.0	10
17–18			TAN FIRM SILTY CLAY					

FIG. 4.-LOG OF BORING 2 SITE A - STATE HIGHWAY 30.
(1 ft = .305 m; 1 pcf = 16.01 kg/m^3)

DEPTH, FEET	SOIL SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N VALUE BLOWS PER FT.
1								
2								
3								
4								
5								
6			BROWN LOOSE SILTY SAND	SM	15.0	5.1	98.7	6
7								
8					10.9	6.0	103.9	6
9								
10				—	—	6.5	101.3	20
11								
12								
13				SP-SM	11.5	11.9	105.8	20
14								
15				—	—	21.0	123.3	—
16			TAN AND LIGHT GRAY STIFF SILTY CLAY					
17								
18								

FIG. 5. - LOG OF BORING 3 SITE A - STATE HIGHWAY 30.
(1ft = .305m; 1pcf = 16.01kg/m^3)

DEPTH, FEET	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N VALUE BLOWS PER FT.
1	TAN AND RED SANDY CLAY FILL					
2	TAN HARD SILTY SAND			8.7	125.2	
3						
4						
5	TAN AND DARK BROWN PLASTIC SANDY CLAY			31.3	119.7	
6						
7	TAN AND LIGHT GRAY FIRM CLAYEY SAND			40.0	110.2	
8	▽					
9						
10	DARK BROWN FIRM SILTY SAND	SM	25.7	23.5	120.2	33
11						
12						
13						
14		SP-SM	10.8	23.5	120.0	-
15		SP-SM	12.0	—	—	—

FIG. 6. - LOG OF BORING 1 SITE B - BRIARCREST DRIVE.
(1ft = .305m; 1pcf = 16.01kg/m^3)

with approximately 12 percent of the material passing the number 200 sieve, but much more dense as indicated by the resistance offered to the penetration of the small diameter sampler. Because this material was very dense, excess pore water pressures were induced in the sample during the extrusion process. According to Taylor (23), "if changes in the water volume are prevented, stresses will inevitably be thrown into the (pore) water. The dense sample will attempt to expand..." This dilation of the sample while in the sampler made extrusion without considerable disturbance impossible. Thus the data obtained from the two samples at 13.5 ft. (4.12 m) and 15 ft. (4.58 m) at test site B were not used in the correlations.

Test site C was located at the East end of a new bridge that crosses the Little Brazos River on State Highway 21 approximately 12 miles (19 km) southwest of Bryan, Texas. As seen in Fig. 7, the stratigraphy at test site C was typical of a floodplain deposit of the Brazos River. Uniform deposits of dark brown and tan soft clay overlaid a uniformly graded layer of silty sand. The layer of silty sand was encountered at 8 ft. (2.44 m). The sand was very wet having a moisture content of 22.3 percent. This material was classified as a SP-SM material with 7.7 percent of the material being finer than the number 200 sieve. At approximately 12 ft. (3.66 m) a sand of medium coarse texture was encountered. Less than one percent of this material passed the number 200 sieve. The sand became very loose at 14 ft. (4.27) and could not be recovered with the small diameter sampler. A uniformly graded gravel was encountered at 18 ft. (5.49 m). After

DEPTH, FEET	SOIL SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N VALUE BLOWS PER FT.
1			DARK BROWN AND TAN SOFT CLAY			19.6	122.6	
2						18.2	128.4	
3			DARK BROWN AND TAN PLASTIC CLAY WITH CALCAREOUS NODULES			18.0	130.9	
4						20.0	130.5	
5			DARK BROWN SOFT SANDY CLAY			19.4	125.2	
6						17.6	131.0	
7						12.2	133.4	
8						15.4	131.7	
9			TAN FIRM SILTY SAND	SPSM	7.7	22.3	118.6	19
10								
11								
12			TAN LOOSE MEDIUM SAND	SP	0.7	20.4	120.4	18
13								
14			∇					
15								
16								
17								
18			WITH GRAVEL AT 18'					

FIG. 7 –LOG OF BORING I SITE C – LITTLE BRAZOS RIVER.
(1ft = .305m; 1 pcf = 16.01 kg/m^3)

attempts to recover the gravel with a 3 in. (76 mm) Shelby tube sampler failed the small diameter sampler was successfully used to recover a disturbed sample for visual observation.

Test site D was located at the intersection of Woodridge Road and Interstate Highway 45 in Houston, Texas. The log of the boring made at site D is shown in Fig. 8 . The stratigraphy at site D consisted of alternating layers of tan and light gray plastic to very stiff clay and layers of fine silty sand. Silty clay of moderate plasticity was present to a depth of 15 ft. (4.58 m). At 15 ft. (4.58 m) a tan fine silty sand was encountered. This sand stratum existed from 15-25 ft. (4.58 - 7.63 m) below the ground surface becoming less silty with increasing depth. The amount of material passing the number 200 sieve ranged from 34.1 percent at 15 ft. (4.58 m) to 5.9 percent at 25 ft. (4.63 m). The material in this stratum was very wet as indicated by an average moisture content of approximately 23 percent. Underlying this strata was a very stiff silty clay extending to a depth of 37 ft. (11.29 m) where a thin layer of fine silty sand was encountered. From 35 ft. (11.50 m) to 48 ft. (14.14 m) there existed a stiff to plastic silty clay. Underlying this silty clay was a very firm silty sand. Considerable pressure was applied to the sampler to achieve full penetration at 50 ft. (15.25m). The corresponding N value of 80 blows per foot indicates the firm consistency of this material. All of the sand samples taken at test site D were successfully recovered and tested.

Test site E was located near the Interstate Highway 610 and Interstate Highway 45 interchange in Houston, Texas at approximately the

DEPTH, FEET	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N VALUE BLOWS PER FT.
	TAN AND LIGHT GRAY PLASTIC CLAY WITH CALCAREOUS NODULES					
10				12.1	133.6	
				17.7	131.9	
				21.5	128.1	
				21.5	127.0	
	TAN FINE SILTY SAND	SM	34.1		124.7	22
20		SP-SM	12.7	24.7	123.3	48
		SP-SM	5.8	21.6	122.2	33
	TAN AND LIGHT GRAY STIFF TO VERY STIFF SILTY CLAY WITH CALCAREOUS NODULES			27.5	119.2	
30				13.9	137.8	
				18.5	134.3	
				31.1	123.1	
40	WHITE FINE SILTY SAND	SM	24.9	20.4	134.7	30
	TAN AND LIGHT GRAY STIFF TO PLASTIC CLAY			16.0	133.3	
				16.2	133.6	
				1 7.4	132.5	
				18.4	133.6	
				17.2	132.8	
50	LIGHT GRAY FIRM SILTY SAND	SPSM	11.7	22.6	125.5	80
		SM	20.7	27.6	119.9	68

FIG.8.- LOG OF BORING I SITE D - WOODRIDGE ROAD.
(1ft =.305m ; 1pcf = 16.01 kg/m^3)

same location as a test site reported by Reese and Touma (26). The log of the boring at this site is shown in Fig. 9. The stratigraphy at this test site was more uniform than at site D. A layer of plastic silty clay existed to a depth of 15 ft (4.58 m). At 15 ft. (4.58 m) a stiff red and light gray silty clay was encountered. The moisture content of this stratum was much lower but not near the plastic limit. Consequently the unit weight was higher than that of the above strata. A very soft sandy silt was encountered at 30 ft. (9.15 m). The silt content decreased and the material became a soft silty sand at 43 ft. (13.12 m). At 50 ft. (15.25 m) the sand became coarser. Two samples taken between 50 and 60 ft. (15.25 to 18.30 m) indicated approximately nine percent of the material passed the number 200 sieve. The material was relatively dense as indicated by N-values averaging 72 blows per foot. Below 60 ft. (18.30 m) the sand became clayey exhibiting some plasticity. The increase in moisture content and unit weight was not significant but the N-values increased to an excess of 100 blows per foot.

DEPTH, FEET	SOIL SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	UNIFIED CLASSIFICATION	PERCENT PASSING NUMBER 200 SIEVE	MOISTURE CONTENT, PERCENT	TOTAL UNIT WEIGHT, LBS. PER CU. FT.	THD PENETROMETER TEST, N VALUE BLOWS PER FT.
10			TAN AND LIGHT GRAY PLASTIC SILTY CLAY			31.9	118.9	
20			RED AND LIGHT GRAY STIFF SILTY CLAY			24.3	125.7	
			RED AND LIGHT GRAY PLASTIC SILTY CLAY			31.1	119.4	
						31.5	120.4	
30								
			YELLOW AND TAN VERY SOFT SANDY SILT			21.3	127.3	
						19.2	132.6	
40						21.5	132.7	
			YELLOW AND TAN VERY SOFT SILTY SAND			32.4	125.7	
50								
			TAN COARSE SILTY SAND	SP-SM	6.5	18.7	119.3	64
				SP-SM	11.3	20.7	123.5	80
60								
			TAN COARSE SAND			23.2	137.6	100/8$\frac{1}{4}$
						22.9	136.8	100/7
						20.9	136.8	100/4$\frac{1}{4}$
70				SM	32.3	20.3	130.3	74

FIG. 9. - LOG OF BORING I SITE E - LOOP 610 OVERPASS.
(1ft = .305m; 1pcf = 16.01kg/m^3)

SOIL SHEAR STRENGTH

Since the objective of this research study was to develop a correlation between the drained shear strength of cohesionless soils and THD Cone Penetrometer Test N-value, a method was developed and implemented to test the sand samples from each test site. From the data made available from these tests the drained shear strength of the soil at depths corresponding to depths at which penetrometer tests were conducted could be determined.

Development of Testing Equipment.-- A basic requirement of the testing method was that disturbance be minimized during preparation of the samples. The direct shear test was chosen instead of the triaxial test because the samples could be extruded directly into the shear box and tested without the use of a membrane, thus minimizing disturbance.

The design of the shear box, shown in Fig. 10, was patterned after the Wykeham Farrance shear boxes which are used in the laboratory. The inside diameter was the same as that of the sample tube. Drainage was facilitated by holes drilled in the gripper plates placed beneath a porous stone. In order to eliminate friction between the two halves of the shear box during the shearing process, the plates could be separated by two screws which were backed off before the test was started. The two sliding surfaces of the shear box were also machined to reduce friction.

A series of preliminary tests were performed using dry Ottawa sand placed at a void ratio of .58 which resulted in an angle of internal friction, ϕ' = 37 degrees. Means and Parcher (17), in their text,

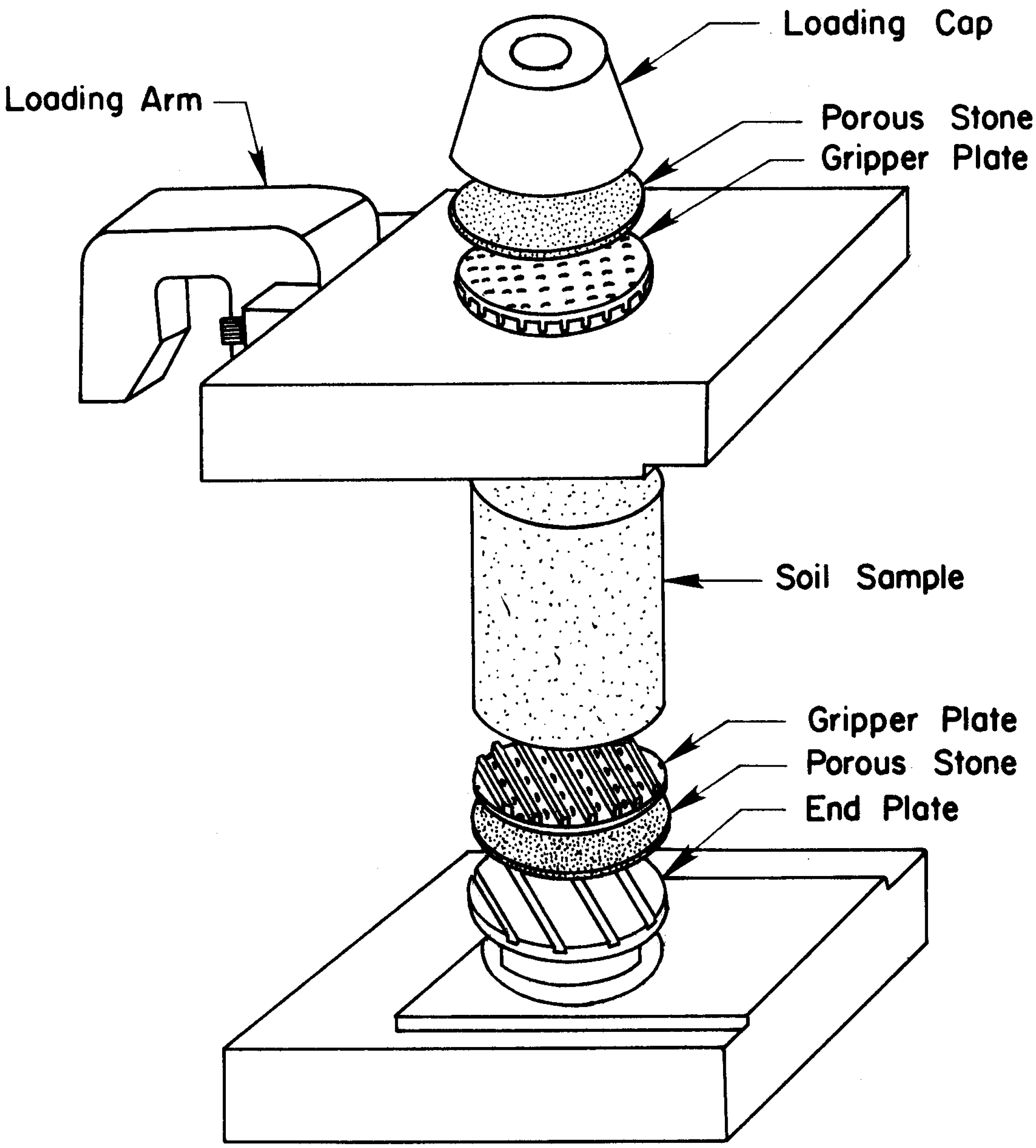


FIG. 10.- SHEAR BOX ASSEMBLY.

have reported an angle of internal friction of $\phi' = 35$ degrees for Ottawa sand at a void ratio of .53. Since the results of the preliminary tests were in close agreement with the values reported by Means and Parcher, the direct shear box was considered suitable.

In order to extrude the sample from the sample tubes into the shear box an extrusion device was developed as seen in Fig. 11. This device was designed to extrude the sample, by means of a hand operated hydraulic jack, directly into the shear box. Before extruding the sample the cuttings were trimmed from both ends and the sample tube was fitted into the extrusion device. The ram was brought into contact with the bottom end of the sample and the gripper plate, porous stone, and end plate were positioned at the top. The shear box was then inverted and secured to the plate with two machine screws. The sample was extruded until the end plate came in contact with the four pins which supported it at the bottom of the shear box. After extruding the sample it was trimmed flush with the top of the shear box. The trimming device was made of sheet metal with a thickness of .001 in. (.025 mm) fitted into the plate so that it could be pulled in a straight, continuous motion. After trimming, the screws were removed, the shear box turned upright, loading arm attached, and the assembly was placed in the loading apparatus for testing.

Direct Shear Test.-- A series of drained direct shear tests were performed on the sand samples to determine the angle of internal friction used in calculating shear strength. The test set up is shown in Fig. 12. The soil sample is enclosed in the shear box discussed

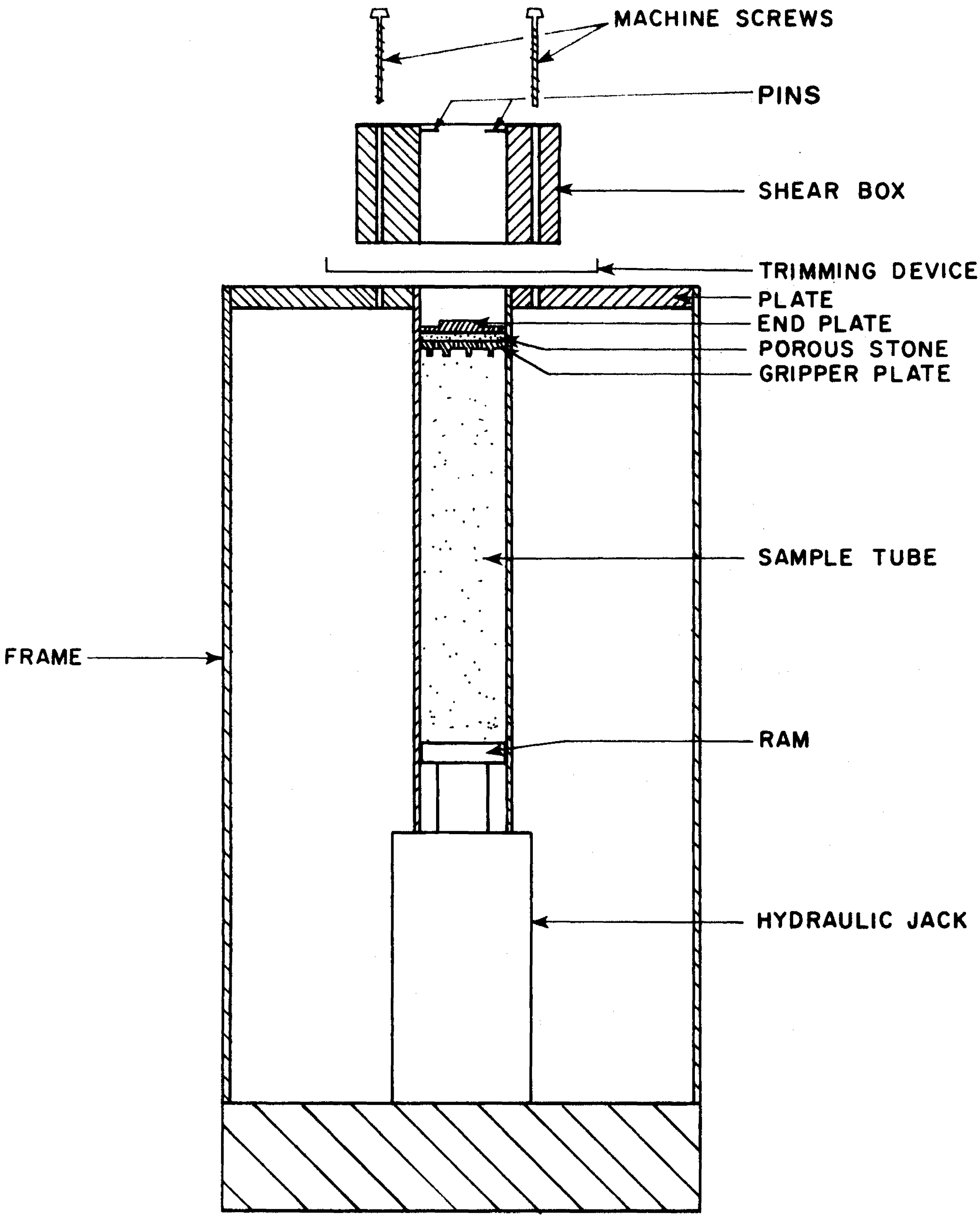
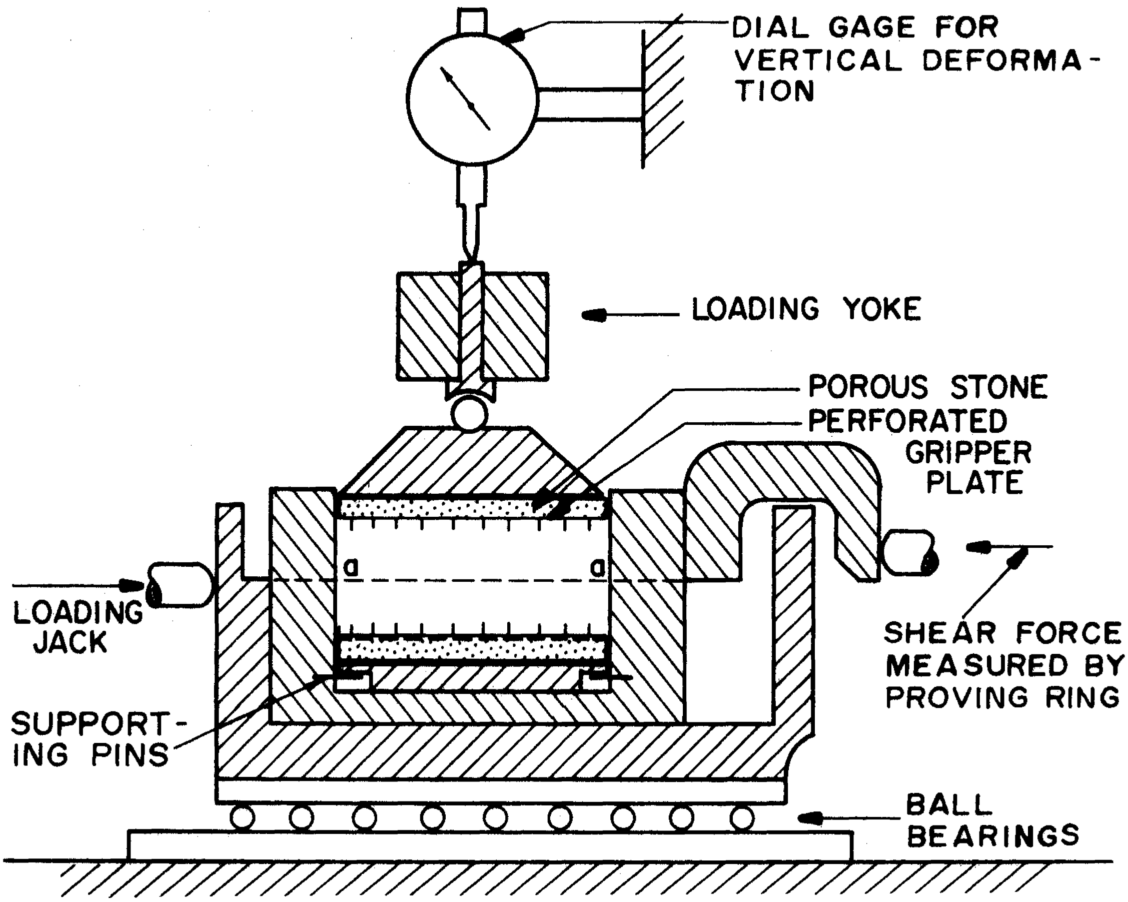
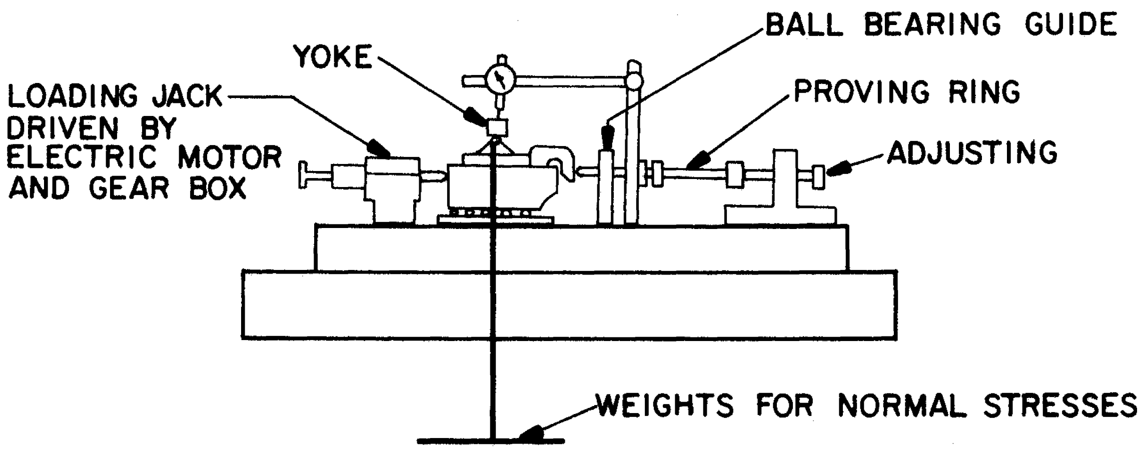


FIG. 11. - CROSS SECTION OF EXTRUSION ASSEMBLY.



SECTION OF SHEAR BOX



LAYOUT OF LOADING SYSTEM

FIG. 12. - (AFTER SKEMPTON AND BISHOP (22)).

above. A normal stress, σ_n, is applied on plane a-a through a loading frame. A constant speed motor causes a relative motion between the upper and lower half of the shear box. The upper half is held in place by a horizontal arm. The force required to hold this arm is determined by readings on a proving ring. The shearing force is increased until the sample fails along plane a-a. In most cases three tests were performed at normal stresses of 10, 20, and 30 psi (69, 138 and 207 kN/m^2). The shear strength of the sample corresponding to each normal stress 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 angle of internal friction, ϕ', is the angle formed by the failure envelope with the horizontal. The results of the direct shear tests are tabulated in Appendix III.

A strain rate must be used which will allow drainage during testing. According to Means and Parcher (17), a number of investigators have shown the strength of the soil tested in the laboratory depends "to a remarkable extent upon the rate and duration of loading employed in the test". In his text (16), Lambe states, "rapid shear of saturated (cohesionless) soil may throw stresses into the pore water, thereby causing a decrease in strength of a loose soil or an increase in the strength of a dense soil." A sample of silty sand (21% passing the #200 sieve) from test site E was used to investigate the effect of the rate of loading. The strain rate was varied from .0001 in/min (.0025 mm/min) to .005 in/min (.127 mm/min) resulting in only one degree difference in the angle of internal friction. Thus, a strain rate of .005 in/min

(.127 mm/min) was considered suitable to allow drainage and thereby prevent pore pressure build up.

Determination of Shear Strength.-- The shear strength at depths corresponding to depths at which penetrometer tests were conducted was determined from the general Mohr-Coulomb relationship:

$$s = c' + \sigma_n' \tan \phi' \quad \ldots \quad (2)$$

where: s = soil shear strength

c' = cohesion intercept

σ_n' = effective normal stress

ϕ' = effective angle of internal friction

Considering the cohesion intercept to be zero, which is common practice in drained tests involving cohesionless soils, the equation becomes:

$$s = \sigma_n' \tan \phi' \quad \ldots \quad (3)$$

The normal stress at some point above the ground water level is equal to the overburden pressure as calculated by the relationship:

$$\sigma'_n = p' = \gamma h. \quad \ldots \quad (4)$$

where: γ = unit weight of soil

h = depth below ground surface

Below the ground water level the effective overburden pressure must be used. Considering the pore water pressures to be hydrostatic the effective overburden pressure can be expressed as:

$$p' = (\gamma - \gamma\omega)\ h \quad \ldots \quad (5)$$

where: $\gamma\omega$ = unit weight of water

h = depth below the ground water level

The procedure for calculating shear strength is to combine the overburden pressure contributed by each soil strata above and below the ground water level with the effective angle of internal friction as in Eq. 3.

The unit weights used in calculating overburden pressures were obtained from laboratory measurements of three inch (76.2 mm) Shelby tube samples. The unit weights determined from these samples are tabulated in Figs. 3 through 9. These unit weights were averaged for each soil strata and the average values summarized in Figs. 13 through 19. With these average unit weights, position of the ground water levels, and angles of internal friction the shear strengths could be calcualted. These data and the corresponding N-values have also been tabulated in Figs. 13 through 19.

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS. PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
0–3			BROWN LOOSE SILTY SAND	125			
3–6.6			RED AND BROWN PLASTIC SANDY CLAY	115			
6.6–9.5			LIGHT TAN LOOSE SILTY SAND	102			23
9.5–11.9			TAN FIRM SILTY SAND	111.4	42	.411	35
11.9–15			LOOSE LIGHT GRAY CLAYEY SAND	118.6	40	.450	60
15–16.5			LOOSE LIGHT GRAY SAND				
16.5–21			TAN LIGHT GRAY STIFF SILTY CLAY				

FIG. 13-SUMMARY OF SHEAR STRENGTH DATA
BORING I SITE A
(1ft. = .305m; 1pcf = 16.01 kg/m^3; 1tsf = 9.58 x 10^4 N/m)

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
1 2 3 4 5 6			▽ LIGHT GRAY LOOSE SILTY SAND	104.3	36.5	.212	4
7 8 9				106.6	31.5	.209	5
10 11 12 13				111.4	37.5	.307	9
14 15 16							

FIG. 14 - SUMMARY OF SHEAR STRENGTH DATA
BORING 2 - SITE A
(1ft. = .305m; 1pcf = 16.01kg/m^3; 1tsf = 9.58 x 10^4 N/m^2)

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
0–16			BROWN LOOSE SILTY SAND	98.7	34.5	.187	6
				103.9	30	.199	6
				101.3			20
				105.8	36.5	.323	20
16–18			TAN AND LIGHT GRAY STIFF SILTY CLAY				

FIG. 15 - SUMMARY OF SHEAR STRENGTH DATA
BORING 3 SITE A
(1ft. = .305m; 1 pcf = 16.01 kg/m^3; 1tsf = 9.58 x 10^{4}N/m^2)

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS. PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
1			TAN AND RED SANDY CLAY FILL	125.2			
2, 3			TAN HARD SILTY SAND	125.2			
4, 5			TAN AND DARK BROWN PLASTIC SANDY CLAY	119.7			
6, 7			TAN AND LIGHT GRAY FIRM CLAYEY SAND	110.2			
8, 9, 10, 11, 12, 13, 14, 15, 16			DARK BROWN FIRM SILTY SAND	120.0	34	.433	33

FIG.16 - SUMMARY OF SHEAR STRENGTH DATA

BORING I SITE B

(1ft. = .305 m ; 1pcf = 16.01 kg/m^2; 1tsf = 9.58 x 10^4 N/m^2)

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS. PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
0–2			DARK BROWN AND TAN SOFT CLAY	125.5			
2–8			DARK BROWN AND TAN PLASTIC CLAY WITH CALCAREOUS NODULES DARK BROWN SOFT SANDY CLAY CLAY	131.1			
8–12			TAN FIRM SILTY SAND	118.6	36	.442	19
12–18			TAN LOOSE MEDIUM SAND WITH GRAVEL AT 18'	120.4	39	.637	18

FIG. 17 - SUMMARY OF SHEAR STRENGTH DATA
BORING I SITE C
(1 ft. = .305m; 1 pcf = 16.01 kg/m³; 1 tsf = 9.58x10⁴ N/m²)

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS. PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
10			TAN AND LIGHT GRAY PLASTIC CLAY WITH CALCAREOUS NODULES	130.2			
			TAN FINE SILTY SAND	124.7	41	.855	22
20				123.3	40	.961	48
				122.2	43	1.153	33
30			TAN AND LIGHT GRAY STIFF CLAY WITH CALCAREOUS NODULES	136.1			
				123.1			
40			WHITE FINE SILTY SAND	134.7	37.5	1.278	30
			TAN AND LIGHT GRAY STIFF TO PLASTIC CLAY	133.3			
50			LIGHT GRAY SILTY SAND	125.5	41	1.766	80
				119.9	38.5	1.722	68

FIG. 18 - SUMMARY OF SHEAR STRENGTH DATA
BORING 1 SITE D
(1 ft. = .305 m; 1 pcf = 16.01 kg/m^3; 1 tsf = 9.58 x 10^4 N/m^2)

DEPTH	SYMBOL	SAMPLE	DESCRIPTION OF STRATUM	AVERAGE TOTAL UNIT WEIGHT, LBS. PER CU. FT.	ANGLE OF INTERNAL FRICTION, DEGREES	DRAINED SHEAR STRENGTH, TONS PER SQ. FT.	THD PENETRATION TEST BLOWS PER FT.
0–15			TAN AND LIGHT GRAY PLASTIC CLAY WITH CALCAREOUS NODULES	118.9			
15–25			RED AND LIGHT GRAY STIFF SILTY SAND	125.7			
25–30			RED AND LIGHT GRAY PLASTIC SILTY CLAY	119.9			
30–42.5			YELLOW AND TAN VERY SOFT SANDY SILT	130.9			
42.5–50			YELLOW AND TAN VERY SOFT SILTY SAND	125.7			
50–60			TAN SILTY SAND	119.3 123.5	39 38	1.183 1.816	64 80
60–70			TAN COARSE SAND	137.6 136.8 136.8 130.3	42	2.076	74

FIG. 19-SUMMARY OF SHEAR STRENGTH DATA
BORING I SITE E
(1 ft. = .305m; 1 pcf = 16.01 kg/m^3; 1 tsf = 9.58 x 10^4 N/m^2)

ANALYSIS OF TEST RESULTS AND DEVELOPMENT OF CORRELATIONS

Before developing correlations involving the THD Cone Penetrometer Test N-value, a review of the factors reported by various researchers to affect the resistance to dynamic penetration in sands is presented.

Previous Research.-- A great number of researchers have investigated the factors affecting the resistance to penetrometer penetration. Although many variables come into play, a certain amount of agreement exists among the researchers as to major factors affecting the resistance to penetration in sands. Desai (8), in an effort to present a rational analysis of the penetration phenomenon, states, "The driving of the cone would cause an upward displacement of the subsoil till a certain depth or surcharge pressure is reached which will not permit such displacement". Desai concludes that the density, structure, depth, and ground water table will have considerable effect on the cone resistance. In research conducted with the Standard Penetration Test in sands, Gibbs and Holtz (13), conclude, "The overburden pressures were found to have the most pronounced and consistent effects on the penetration resistance values". Schultze and Knausenberger (21) report, "Dynamic penetrometers react very sensitively to any changes of compactness or grain size".

The consensus of the opinion seems to be that unit weight, grain size, moisture content, and overburden pressure are the major factors affecting the resistance to penetration in sands. This opinion was substantiated in a summary of the conclusions of 21 researchers presented in the text, Dynamic and Static Sounding of

Soils, by Bodarik (2). Although the researchers do not arrive at the same conclusions concerning the factors which have the most effect, they all agree that these factors do affect the resistance to dynamic penetration in sands. The available data from this study have thus been analysed to investigate the effect of these factors.

Unit Weight.-- Unit weights were determined from both small diameter and three inch (76 mm) Shelby tube samples. In general, samples taken by each method at approximately the same depths resulted in unit weights which were in very close agreement. At test sites where several consecutive small diameter samples were taken a consistency in the unit weights was observed. Although this consistency applies to each test site it is especially noticeable at test site E where an obviously dense material was encountered with N-values well in excess of 100 blows/foot. Unit weights determined from the small diameter samplers fell within the narrow range of 136.8 to 137.6 lb/ft^3 (2190 to 2203 Kg/m^3) for the three samples tested. In view of the close agreement with the unit weights determined from Shelby tube samplers and the consistency in unit weights at each test site, the unit weights determined from the small diameter samplers seem to be accurate.

Several researchers have indicated a relationship does exist between unit weight and penetration resistance in sands. These relationships, however, are based upon laboratory observations in which a material of uniform unit weight is penetrated. The relationship determined from data obtained in this field study, shown in Fig. 20, reveals considerable scatter. An equation, however, was determined for

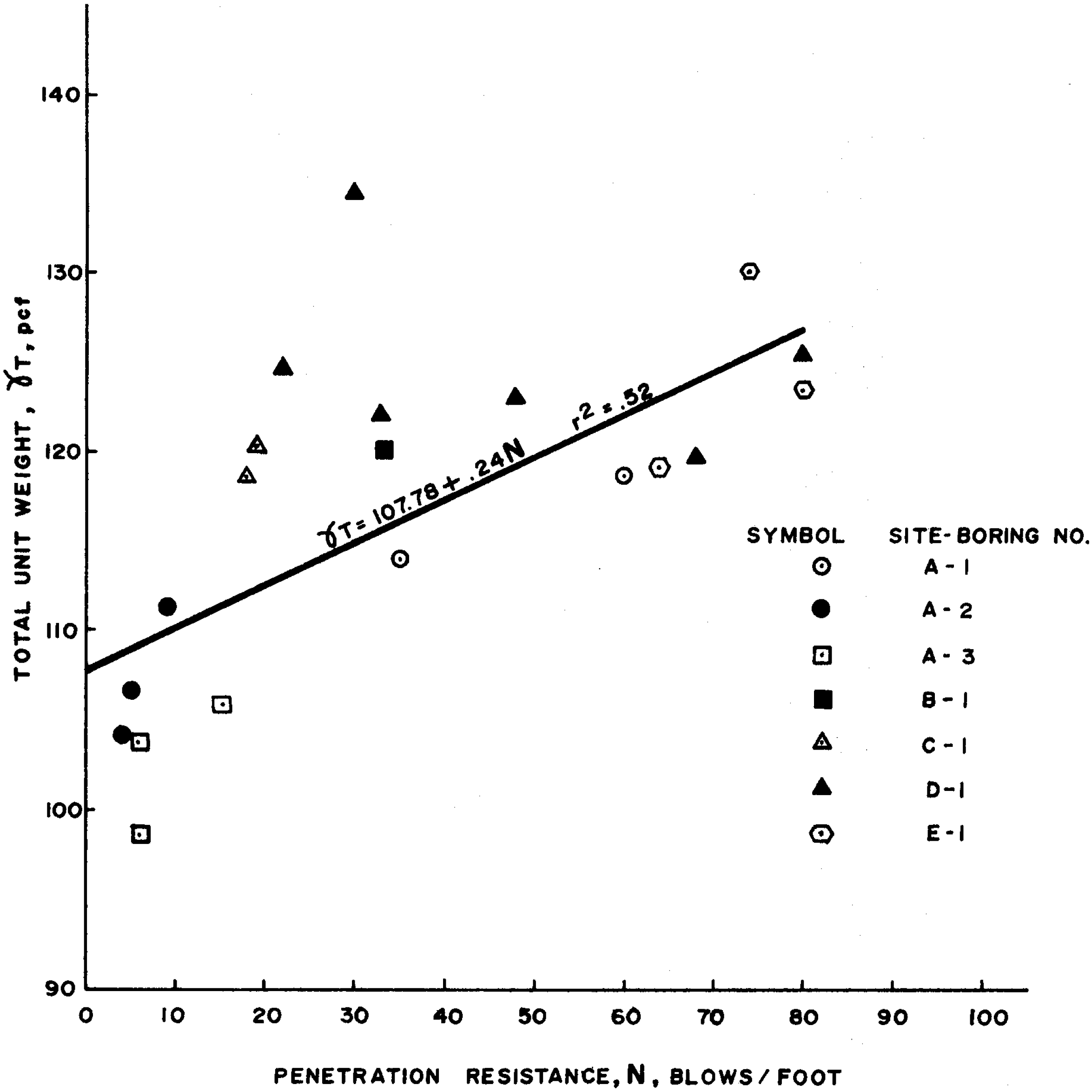


FIG. 20. - RELATIONSHIP BETWEEN TOTAL UNIT WEIGHT AND RESISTANCE TO PENETRATION FOR ALL SOILS TESTED. (1 ft = .305 m; 1 pcf = 16.01 kg/m^3)

the best fit linear relationship and is expressed as:

$$\gamma_t = 107.78 + .24N \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

where: γ_t = total unit weight

The coefficient of correlation for the above relationship of $r^2 = 0.52$ is indicative of the poor correlation. A simple regression analysis using quadratic, cubic, and logarithmic models resulted in no significant improvement in the r^2 value.

Effective Overburden Pressure. -- The effect of overburden pressure upon penetration resistance is probably best explained by Bodarik (2), who states, "The stress caused by the weight of the overburden presses the particles together and greatly delays their displacement during penetration. Since compressive forces in sands are transmitted from grain to grain through points of contact, increases in earth pressures, even in loose sands, causes an appreciable increase in density and affects the results of the sounding".

Some field observations have confirmed the effect of overburden pressure on the results of the Standard Penetration Test. Fletcher (12) reported that the removal of 15 ft. (4.6 m) of overburden from a sand deposit will "relieve pressure noticeably and thus affect the N-value at shallow depths by underestimating relative density and hence the bearing capacity". Attempts have been made by various researchers to correct the N-value at shallow depths to include the effect of overburden. Bowles (3) presents such a relationship in his text by stating in reference to research conducted by Gibbs and Holtz, "for two cohesionless soils of the same density the one with the greatest overburden pressure has the higher penetration number".

Several cases can be cited from the data obtained in the study and presented in Table 2, where N-values increased with increasing overburden pressure. However, the actual effect of the overburden cannot be determined because of variations in other factors which also affect the resistance to penetration. The relationship observed between overburden pressure and the THD Cone Penetration Test N-value is shown in Fig. 21. The best linear relationship determined by the least squares method is:

$$p' = .150 + .026\ N \qquad (7)$$

The coefficient of correlation, r^2, was .73. As can be expected in field observations, there does exist a considerable amount of scatter, however, a definite trend has been established.

Angle of Internal Friction.-- A relationship has been developed by Touma and Reese (26) between the N-value from the Standard Penetration Test (N_{SPT}) and the N-value from the THD Cone Penetrometer Test (N_{THD}). The data from which this relationship was developed are shown in Fig. 22. Although there is a considerable amount of scatter in the data by Touma and Reese, the following general relationship was proposed:

$$N_{SPT} = .5\ N_{THD} \qquad (8)$$

The N-values from the THD Cone Penetrometer Test were adjusted by Eq. 8 and are presented in Table 3. These adjusted N-values were then superimposed on the plot of N_{SPT} and angle of internal friction reported by Peck, Hanson, and Thornburn (19). As seen in Fig. 23 the curve

TABLE 2.- Summary of N-Values and Effective Overburden Pressures			
Site	Sample Number	N-Values Blows/Ft.	Effective Overburden Pressure, tsf
A	2-10-11	35	.457
	2-13-14	60	.536
	1-5-6	4	.287
	2-7.5-8.5	5	.341
	3-10-11	9	.400
	3-1	6	.270
	3-2	6	.345
	3-3	20	.437
B	9-12	33	.643
C	13-9-10	19	.608
	18-12-13	18	.780
D	5-15-17	22	.960
	6-21-22	48	1.145
	7-24-25	33	1.235
	12-39-40	30	1.665
	19-49-50	80	2.032
	22-54-55	68	2.165
E	11-55-56	64	2.270
	12-57-58	80	2.325
	17-69-70	74	2.755

Note: 1 ft. = .305 m; 1 tsf = 9.58×10^4 N/m^2

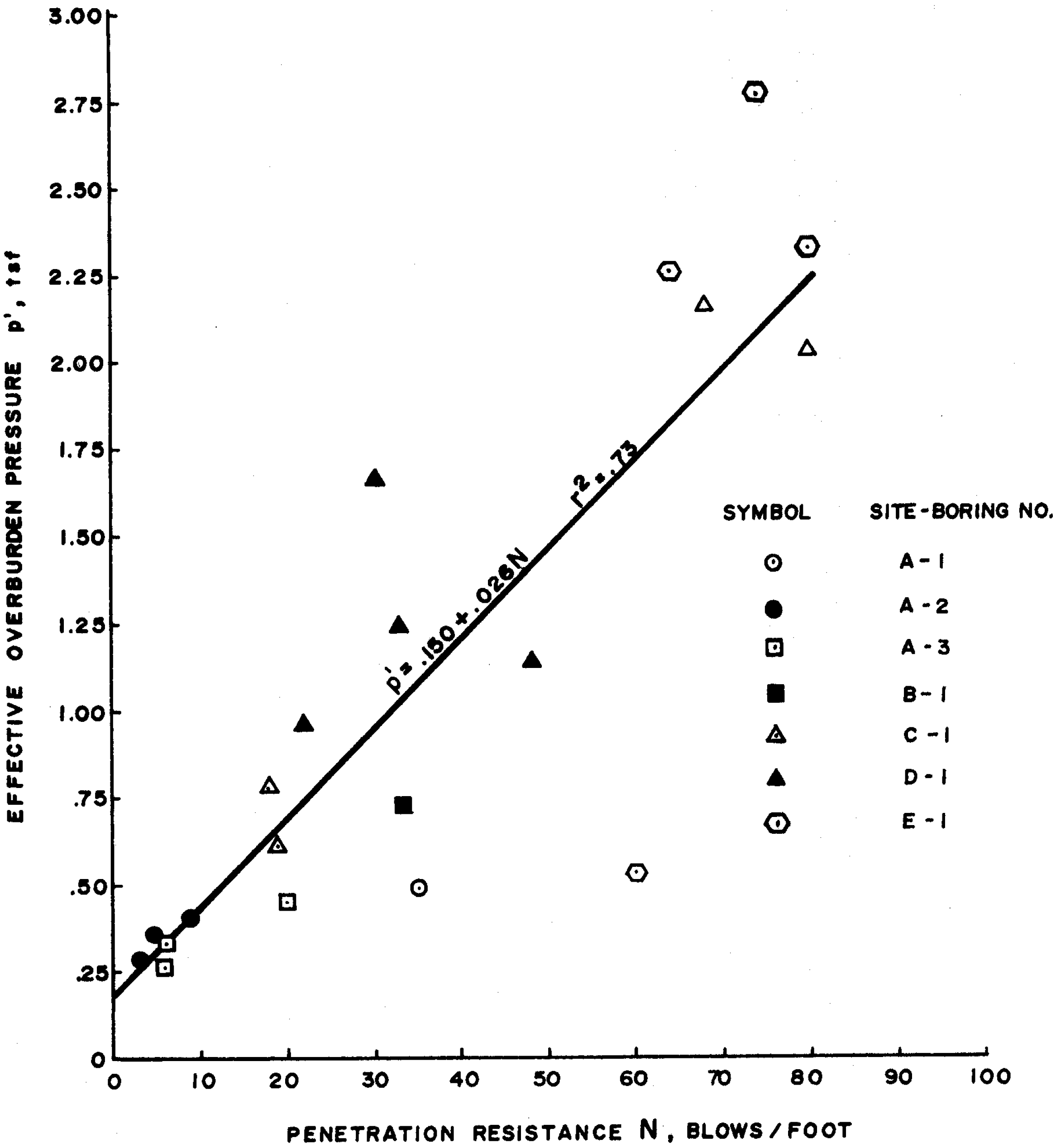


FIG.21. - RELATIONSHIP BETWEEN PENETRATION RESISTANCE AND EFFECTIVE OVERBURDEN PRESSURE FOR SP, SM, AND SPSM SOILS. (1ft. = .305m; 1 tsf = 9.58 x 10^4 N/m^2)

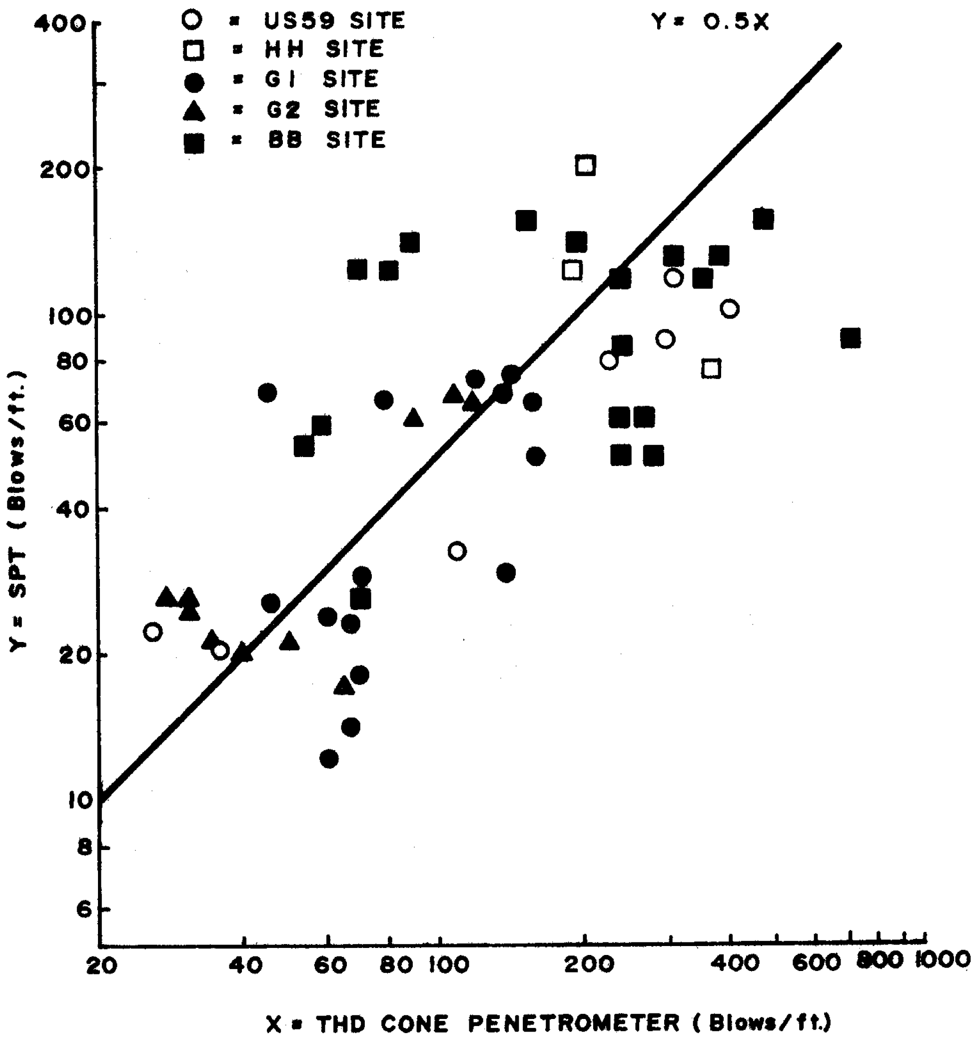


FIG. 22.- CORRELATION BETWEEN THE SPT AND THD CONE PENETROMETER TESTS IN SANDS. (AFTER TOUMA AND REESE (26)) (1ft = .305m)

TABLE 3.-- Summary of N-Values and Effective Angles of Internal Friction				
Site	Sample Number	N-Value N_{THD}	Blows/Ft. N_{SPT}	Effective Angle of Internal Friction, Degrees
A	2-10-11	35	18	42.0
	3-13-14	60	30	40.0
	1-5-6	4	2	36.5
	2-7.5-8.5	5	3	31.5
	3-10-11	9	5	37.5
	3-1	6	3	34.5
	3-2	6	3	30.0
	3-3	20	10	36.5
B	9-12	33	17	34.0
C	13-9-10	19	9	36.0
	18-12-13	18	9	39.0
D	5-15-17	22	11	41.0
	6-21-22	48	24	40.0
	7-24-25	33	17	43.0
	12-39-40	30	15	37.5
	19-49-50	80	40	41.0
	22-54-55	68	34	38.5
E	11-55-56	64	32	39.0
	12-57-58	80	40	38.0
	17-69-70	74	37	42.0
1 ft. = .305 m				

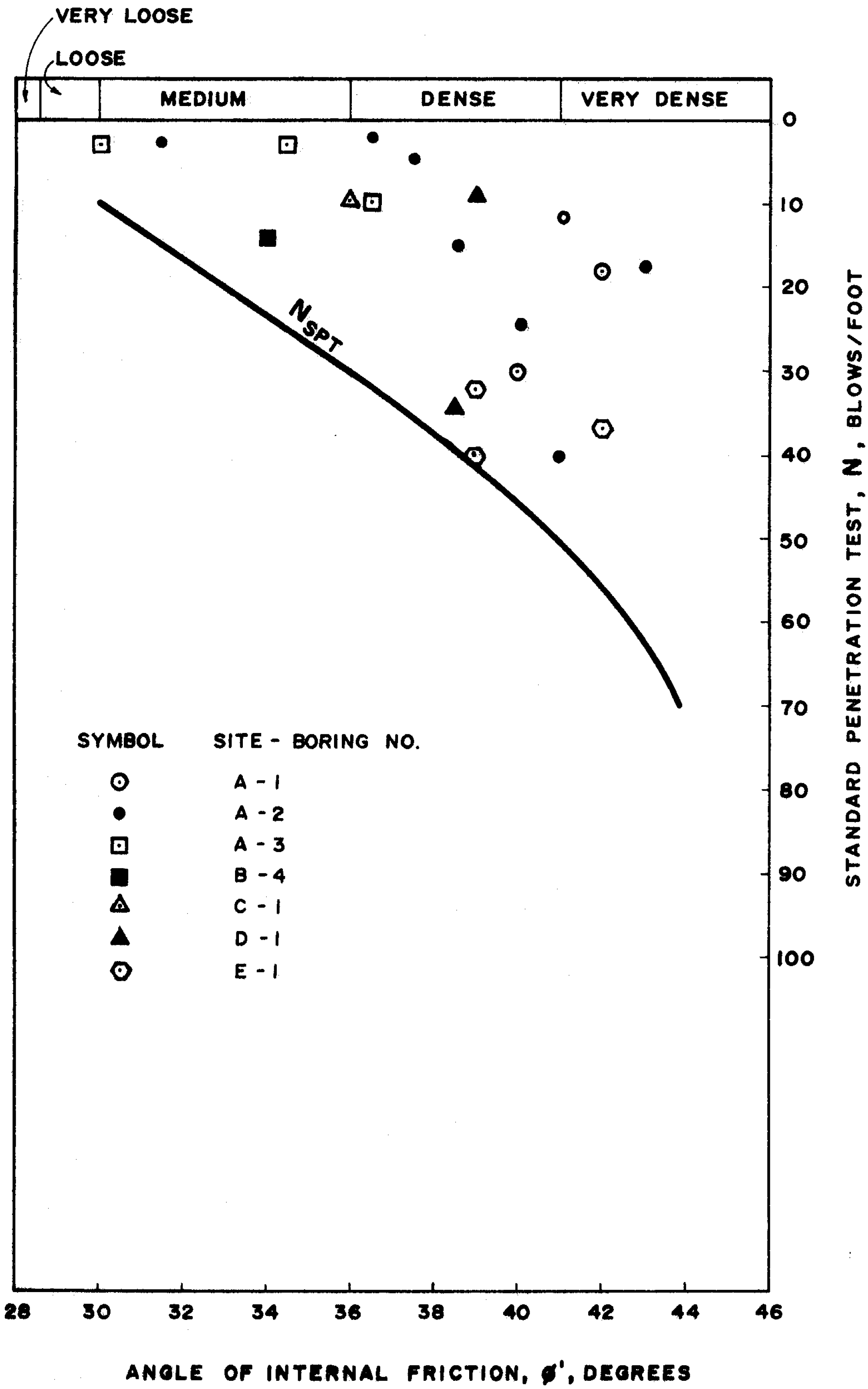


FIG. 23. - COMPARISON OF ADJUSTED N_{THD}-VALUES WITH PECK, HANSON, AND THORNBURN'S (19) RELATIONSHIP FOR THE STANDARD PENETRATION TEST. (1ft. = .305m)

reported by Peck, Hanson and Thornburn is a lower bound to the data obtained in this study. The Texas Highway Department currently uses a relationship between N_{THD} and the angle of internal friction. The N-values when related to this curve are shown in Fig. 24. The scatter in the data does not warrant the fitting of a new curve. However, these relationships are significant since all of the data obtained as a result of this study fall above Peck, Hanson, and Thornburn's curve and above the THD curve, and are thus an indication of the conservative nature of these relationships.

Since the angles of internal friction used in Figs. 23 and 24 were obtained using new techniques in sampling and testing of cohesionless soils, it is appropriate to discuss the limitations which might affect the correlations with resistance to penetration. Although the method of testing is sound and the results fairly reproducible it is difficult to determine the actual effect of disturbance upon the end result. Terzaghi and Peck (24) list sample disturbance as one of the principal factors leading to the misjudgement of soil conditions. In full recognition of this fact, attempts were made to determine the relative order of magnitude of disturbance for each sample tested. An unsuccessful attempt was made to examine the samples by X-ray photography before extrusion. After testing, a cross section of each sample was allowed to air dry and the amount of disturbance, indicated by discontinuities in its stratification, was observed. Several samples from test sites A and C were eliminated. These samples were primarily very loose sands. Three different samples from test sites C and E were very

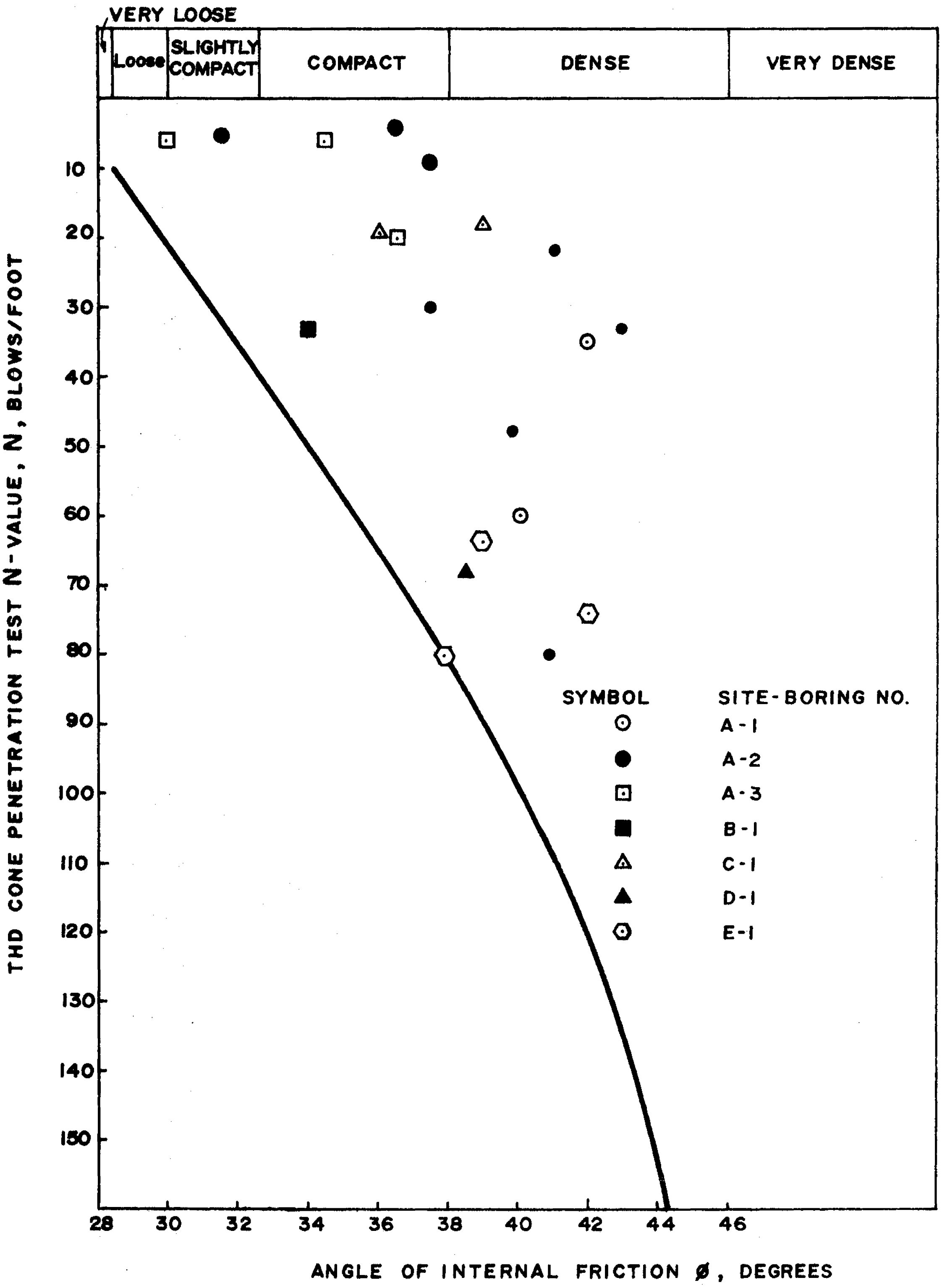


FIG. 24. - COMPARISON OF THE DATA OBTAINED FROM THIS STUDY WITH THE RELATIONSHIP CURRENTLY IN USE BY THE TEXAS HIGHWAY DEPARTMENT (1ft = .305 m).

dense and could not be extruded from the sampler. These samples were allowed to air dry for approximately 8 hours and were then easily extruded. Air dry sections of each sample indicated excessive disturbance. Whether the sampling or extrusion process was the major cause of disturbance could not be determined. However, there is little doubt that very dense sands which dilate while being extruded from the sampler undergo a excessive amount of disturbance resulting in erroneous test results.

Shear Strength. -- The shear strength of cohesionless soils depends upon the angle of internal friction and the normal pressure acting on the failure plane. Means and Parcher (17) have reported that the factors affecting the angle of internal friction are degree of density, void ratio or porosity, grain size and shape, gradation, and moisture content. Since the resistance to penetration has been reported to be affected by most of these same factors and especially the normal pressure (effective overburden pressure), a relationship should exist between penetration resistance and shear strength.

The effect of shear strength upon the penetration resistance has been verified by several researchers (7,8,15.20). According to DeMello (7), "The shear resistance is the principle parameter at play in resisting penetration". Desai (8) concludes that shear strength was one of the main factors affecting penetration resistance. Jonson and Kavanagh (15) have summarized their findings by stating that the resistance to penetration is a function of the shearing resistance of the soil.

The calculated values of drained shear strength and the THD Cone Penetrometer N-values, tabulated in Fig. 13 through 19, are correlated in Fig. 25. The equation of the best fit linear relationship is:

$$s = .114 + .020N \quad (9)$$

with $r^2 = .73$. If it is assumed that when the N-value is zero the resulting shear strength is also zero the relationship becomes:

$$s = .022N \quad (10)$$

Since the boundary conditions were specified the coefficient of correlation, r^2, has no meaning.

The shear strength as calculated by Eq. 3, is most affected by the effective overburden pressure, p'. In several instances when small N-values have been found at relatively large depths a correspondingly low value of ϕ' has been observed. An example of this occurred at test site D. At approximately 40 ft. (12.2 m) the N-value was found to be 30 blows per foot. The friction angle was found to be 37.5°. A relatively large overburden pressure of 1.76 tsf (169 kN/m^2) was calculated. The shear strength, however, from Eq. 3 was 1.20 tsf (115 kN/m) which fit the trend of the other observed data. Thus, the effect of the relatively low friction angle when combined with a large overburden pressure resulted in a correlateable value of shear strength. Other such instances, although not as pronounced, were observed at test site E with samples taken from 57 and 58 ft. (17.4 and 17.7 m).

<u>Ground Water Level</u>.-- Terzaghi and Peck (24) suggested that, in loose very fine or silty sands below the ground water level, positive pore water pressures might develop in the soil due to dynamic application of the load and the low permeability of the soil. According to Sanglerat (20), "These positive pore water pressures would reduce the

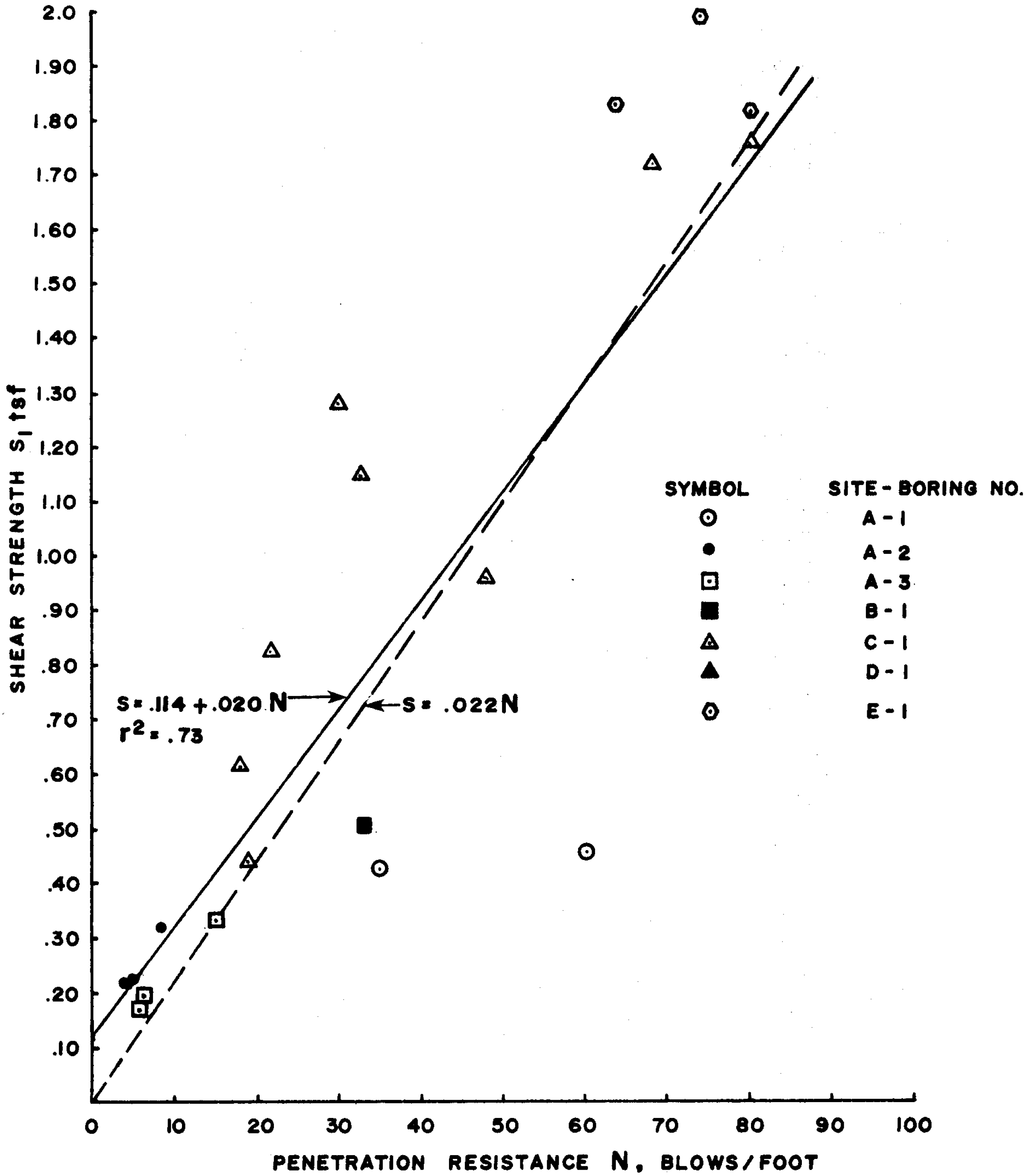


FIG. 25.- RELATIONSHIP BETWEEN DRAINED SHEAR STRENGTH AND RESISTANCE TO PENETRATION FOR SP, SM, AND SPSM SOILS. (1ft. = .305m; 1tsf = 9.58 x 10^2 N/m^2)

shearing resistance of the soil which opposes the penetration of the sampling spoon, hence the standard penetration value of these loose soils would decrease upon submergence." On the other hand it was suggested that for dense, very fine or silty sands the penetration test might induce negative pore water pressures which would increase the resistance to penetration and thus increase the N-value. The effect of the ground water level was noted at two test sites. At boring 3 of test site A, as seen in Fig. 5, the N-value slightly above the ground water level of 6 blows/foot indicated a very loose material. Approximately 2 ft. (.6 m) below the water table the N-value increased from 22 blows/foot at the ground water level to 48 blows/foot approximately 6 ft. (1.8 m) below the water table. In neither case can a definite conclusion be drawn concerning the effect of the ground water level upon the N-value because of the variation in other factors which affect the resistance to penetration. However, an increase has been observed in the resistance to penetration of relatively loose materials which is not in agreement with the statement made by Terzaghi and Peck.

Grain Size.-- Another factor thought to have a major effect upon the resistance to penetration is grain size distribution. According to Desai (8), "Grain size distribution has a considerable effect on the penetration resistance for a given relative density." Since it has been shown by other researchers (9, 13) that penetration resistance can be related to relative density and relative density is a function of grain size it can be concluded that grain size does have an effect upon penetration resistance. A sand composed of a large amount of gravel, according to Desai, will have a relatively low resistance to penetration, the round gravel acting like ball bearings will reduce friction and penetration resistance considerably. Sands with a large amount of fine material will experience positive or negative pore water pressures (depending upon the state of compactness) resulting in an increase or decrease in the N-value. In natural sand deposits where grain size characteristics are not uniform, the effect of grain size is not so easily determined. As in the case of unit weights, the grain size is suspected to influence the N-value but this effect is not obvious. There were several situations encountered in this study where the penetrated soil had a large percentage of material passing the number 200 sieve and correspondingly high N-values. However, other factors such as overburden pressure, position of the ground water table, and unit weight were not the same in each situation. Thus, the effect of the increased N-value could not be attributed to any one factor.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions.-- A study of the relationship between the drained shear strength and the resistance to penetration of cohesionless soils has been made by the implementation of new techniques in sampling and testing. The following conclusions concerning this study can be made:

1. An improved correlation has been established between the N-value from the THD Cone Penetrometer Test and the drained shear strength of SP, SM and SP-SM soils as defined by the Unified Soil Classification System. The shear strength can be predicted if the N-value is known by using the following equation:

   $$s = .114 + .020N$$

   If the boundary condition ($s = 0$ when $N = 0$) is stipulated, the equation is:

   $$s = .022\ N$$

2. The drained shear strength has been shown to be affected mostly by the effective overburden pressure. A correlation of effective overburden pressure with the THD Cone Penetrometer Test N-value has been developed. The equation of the best fit linear relationship is:

   $$p' = .150 + .026N$$

3. A relatively poor correlation exists between total unit weight and the THD Cone Penetrometer Test N-value. However, a trend was noted. The equation of the best fit linear relationship for this trend is:

   $$\gamma_T = 107.78 + .24N$$

4. By adjusting the values of N_{THD} to N_{SPT} using the equation developed by Touma and Reese (26), the angles of internal friction and the N-values from this study have been compared to the relationship developed for the Standard Penetration Test by Peck, Hanson, and Thornburn. Peck, Hanson, and Thornburn's curve is a lower bound to the data obtained in this study. Since the plot of N_{THD} and angle of internal friction currently used by the Texas Highway Department is also a lower bound to the N-values obtained during this study, the conservatism of the THD plot has also been substantiated.
5. Other factors which might affect penetration resistance in cohesionless soil, such as grain size characteristics and position of the ground water level, have been considered in this study. However, no correlations or trends for these factors have been established. Rather, it has been shown that in a field study such as this one, control of individual factors is not possible. Therefore, since individual factors cannot be separated, it is probable that some interaction occurs and a combination of several factors actually affects the resistance to penetration.

Recommendations.-- The following recommendations are made concerning additional research in this area:

1. Considering the limited amount of data available for use in this study, additional data are needed to ascertain the validity of the correlations.
2. The possibility of developing separate correlations for SW, SP, SM, and SC materials should be investigated.

3. Additional data are needed to establish a better correlation between N_{THD} and the angle of internal friction.
4. A field study is needed to determine the effect of the ground water level and shallow depths upon the magnitude of the N-value. Adjustment factors should be developed as for the SPT.

APPENDIX I.-REFERENCES

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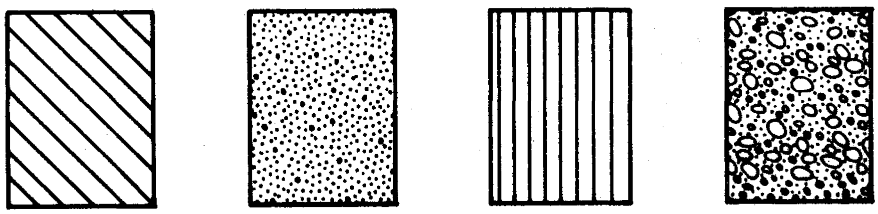
APPENDIX II.-- NOTATION

The symbols used on boring logs are:

SOIL TYPE

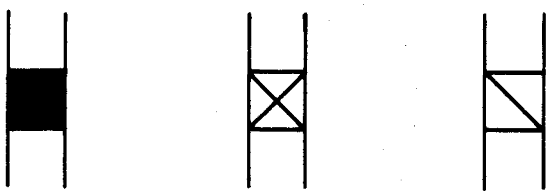
(shown in symbol column)



CLAY SAND SILT FILL

SAMPLER TYPES

(shown in samples column)



SHELBY TUBE SAMPLES

THD CONE PENETROMETER TEST AND SMALL DIA. SAMPLES

NO RECOVERY OR NOT USED

The following symbols are used in this paper:

c' = cohesion intercept;

D_e = inside diameter of sample tube;

D_w = outside diameter of sample tube;

F = percent passing no. 200 sieve;

h = depth below ground level; also depth below ground water level;

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

N_{THD} = the number of blows required to drive the THD Cone Penetrometer a depth of one foot;

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

p' = effective overburden pressure, also noted as p';

r^2 = coefficient of correlation;

s = drained shear strength;

γ_t = total unit weight; also noted γ;

γ_ω = unit weight of water

σ_n' = effective normal stress

ϕ' = effective angle of internal friction; also angle of internal friction

APPENDIX III

SUMMARY OF TEST DATA

TABLE.- Summary of Tests Results

		A-1-2	a	b	A-1-3	a	A-2-1	a
Site and Sample Number		A-1-2	a	b	A-1-3	a	A-2-1	a
Depth		10-11			13-14		5-6	
Penetration Resistance, N		35			60		4	
Classification Tests	Percent Passing No. 200 Sieve	8.6			10.5		13.4	
	Uniformity Coef., C_u	2.75			3.30			
	Curvature Coef., C_c	1.36			1.20			
	Plastic Limit	-			-		NONPLASTIC	
	Liquid Limit	-			-			
	Unified Classification	SP-SM			SP-SM		SM	
Direct Shear Test	Shear Strength at Failure (psi)		9.2	18.4		8.6		6.65
	Moisture Content: Before test (%)		18.7	18.5		23.6		11.1
	Moisture Content: After test (%)		17.0	14.4		20.0		9.8
	Unit Weight[1] (pcf)		105.3	111.3		110.1		95.6
	Angle of Internal Friction	42"			40		36.5	
Total Unit Weight[2] (pcf)		111.4			118.6		104.3	

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 - Boring

A-1-State Highway 30
Boring 1

A-2-State Highway 30
Boring 2

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

TABLE.- Summary of Tests Results

		Site and Sample Number	b	A-2-2	a	b	A-2-3	a	b
		Depth		7.5 - 8.5			10-11		
		Penetration Resistance, N		5			9		
Classification Tests		Percent Passing No. 200 Sieve		13.4			16.3		
		Uniformity Coef., C_u							
		Curvature Coef., C_c		NONPLASTIC			NONPLASTIC		
		Plastic Limit							
		Liquid Limit							
		Unified Classification		SM			SM		
Direct Shear Test		Shear Strength at Failure (psi)	15.6		6.3	11.6		7.71	15.7
	Moisture Content	Before test (%)	11.5		13.5	23.6		11.1	12.9
		After test (%)	10.6		11.7	15.9		11.1	16.1
		Unit Weight[1] (pcf)	98.0		97.9	107.7		100.4	111.3
		Angle of Internal Friction		31.5			37.5		
		Total Unit Weight[2] (pcf)		106.6			111.4		

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

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

TABLE.- Summary of Tests Results

			A-3-1	a	b	A-3-2	a	b	A-3-3
Site and Sample Number			A-3-1	a	b	A-3-2	a	b	A-3-3
Depth			5-6			8-9			12.5-13.5
Penetration Resistance, N			6			6			20
Classification Tests		Percent Passing No. 200 Sieve	15.0			10.9			11.5
		Uniformity Coef., C_u				2.67			3.10
		Curvature Coef., C_c				.96			1.01
		Plastic Limit	NONPLASTIC			-			-
		Liquid Limit				-			-
		Unified Classification	SM			SP-SM			SP-SM
Direct Shear Test		Shear Strength at Failure (psi)		6.8	14.1		4.7	13.0	
	Moisture Content	Before test (%)		5.1	4.9		6.0	10.4	
		After test (%)		-	-				
		Unit Weight[1] (pcf)		93.3	-				
		Angle of Internal Friction	34.5			30			36.5
Total Unit Weight[2] (pcf)			98.7			103.9			105.8

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

A-3 - State Highway 30
Boring 3

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

TABLE.- Summary of Tests Results

Site and Sample Number		a	b	B-1-9	a	b	c	c-1-13
Depth				10-105				9-10
Penetration Resistance, N				33				19
Classification Tests	Percent Passing No. 200 Sieve			25.7				7.7
	Uniformity Coef., C_u			--				2.33
	Curvature Coef., C_c			--				.96
	Plastic Limit			23.9				--
	Liquid Limit			25.0				--
	Unified Classification			SM				SP-SM
Direct Shear Test	Shear Strength at Failure (psi)	5.9	13.9		6.6	11.9	20.8	
	Moisture Content: Before test (%)	11.9	13.8		23.5	29.2	20.0	
	Moisture Content: After test (%)	--	--		--	--	--	
	Unit Weight[1] (pcf)	98.1	105.4		113.2	107.2	106.0	
	Angle of Internal Friction			34				36
Total Unit Weight[2] (pcf)				120.2				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

B-1 - Intersection of Briarcrest Drive and State Highway 6, Bryan, Texas

C-1 - Stak Highway 21 and Little Brazos River

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

TABLE.- Summary of Tests Results

Site and Sample Number		a	b	c-1-18	a	b	D-1-5	a
Depth				12-13			15-17	
Penetration Resistance, N				18			22	
Classification Tests	Percent Passing No. 200 Sieve			.7			34.1	
	Uniformity Coef., C_u			1.74				
	Curvature Coef., C_c			.92				
	Plastic Limit			--			NONPLASTIC	
	Liquid Limit			--				
	Unified Classification			SP			SM	
Direct Shear Test	Shear Strength at Failure (psi)	4.3	14.9		8.5	15.7		10.0
	Moisture Content: Before test (%)	12.5	7.8		18.3	17.1		22.2
	Moisture Content: After test (%)	9.3	7.0		17.3	15.2		21.5
	Unit Weight[1] (pcf)	111.3	105.3		114.7	111.3		119.9
	Angle of Internal Friction			39			41	
Total Unit Weight[2] (pcf)				120.4			124.7	

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

D-1 - Intersection of Woodridge Road and Interstate Highway 45, Houston, Texas

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

TABLE.- Summary of Tests Results

	Site and Sample Number	b	a	D-1-6	a	b	a	D-1-7
	Depth			21-22				24-25
	Penetration Resistance, N			48				33
Classification Tests	Percent Passing No. 200 Sieve			12.7				5.85
Classification Tests	Uniformity Coef., C_u			1.85				2.22
Classification Tests	Curvature Coef., C_c			.82				.99
Classification Tests	Plastic Limit			--				--
Classification Tests	Liquid Limit			--				--
Classification Tests	Unified Classification			SP-SM				SP-SM
Direct Shear Test	Shear Strength at Failure (psi)	17.8	8.8		10.8	18.7	10.1	
Direct Shear Test	Moisture Content: Before test (%)	21.6	25.3		24.7	23.4	22.2	
Direct Shear Test	Moisture Content: After test (%)	24.7	25.8		23.2	21.6	21.9	
Direct Shear Test	Unit Weight[1] (pcf)	115.0	112.6		122.3	122.3	122.3	
Direct Shear Test	Angle of Internal Friction			40				43
	Total Unit Weight[2] (pcf)			123.3				122.2

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

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

TABLE.- Summary of Tests Results

	Site and Sample Number	a	b	a	D-1-12	a	b	b
	Depth				39-40			
	Penetration Resistance, N				30			
Classification Tests	Percent Passing No. 200 Sieve				24.9			
Classification Tests	Uniformity Coef., C_u							
Classification Tests	Curvature Coef., C_c							
Classification Tests	Plastic Limit				NONPLASTIC			
Classification Tests	Liquid Limit							
Classification Tests	Unified Classification				SM			
Direct Shear Test	Shear Strength at Failure (psi)	10.22	18.7	8.9		8.5	16.1	17.2
Direct Shear Test	Moisture Content: Before test (%)	21.6	20.1	21.4		20.4	20.5	19.8
Direct Shear Test	Moisture Content: After test (%)	22.3	22.4	22.1		20.1	19.7	18.8
Direct Shear Test	Unit Weight[1] (pcf)	117.4	111.4	110.2		122.3	122.3	121.1
Direct Shear Test	Angle of Internal Friction				37.5			
	Total Unit Weight[2] (pcf)				134.7			

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

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

TABLE.- Summary of Tests Results								
Site and Sample Number		a	D-1-19	a	b	D-1-22	a	b
Depth			49.50			54-55		
Penetration Resistance, N			80			68		
Classification Tests	Percent Passing No. 200 Sieve		11.7			20.7		
	Uniformity Coef., C_u		3.69			NONPLASTIC		
	Curvature Coef., C_c		1.26					
	Plastic Limit		--					
	Liquid Limit		--					
	Unified Classification		SP-SM			SM		
Direct Shear Test	Shear Strength at Failure (psi)	9.6		9.0	19.4		8	15.5
	Moisture Content: Before test (%)	17.6		22.6	20.4		22.6	20.6
	Moisture Content: After test (%)	17.9		21.5	16.3		21.6	21.5
	Unit Weight[1] (pcf)	124.7		119.9	117.4		117.4	115.0
	Angle of Internal Friction		41°			38.5		
Total Unit Weight[2] (pcf)			125.5			119.9		

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

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

TABLE.- Summary of Tests Results

		Site and Sample Number	a	E-1-11	a	b	c	E-1-12	a
		Depth		55-56				57-58	
		Penetration Resistance, N		64				80	
Classification Tests		Percent Passing No. 200 Sieve		6.5				11.3	
Classification Tests		Uniformity Coef., C_u		2.8				3.6	
Classification Tests		Curvature Coef., C_c		1.3				1.85	
Classification Tests		Plastic Limit		--				--	
Classification Tests		Liquid Limit		--				--	
Classification Tests		Unified Classification		SP-SM				SP-SM	
Direct Shear Test		Shear Strength at Failure (psi)	8.1		7.2	17.0	25.5		9.4
Direct Shear Test	Moisture Content	Before test (%)	18.2		18.7	20.7	17.5		20.7
Direct Shear Test	Moisture Content	After test (%)	22.4		--	--	--		19.6
Direct Shear Test		Unit Weight[1] (pcf)	115.0		109.0	120.0	127.1		123.5
Direct Shear Test		Angle of Internal Friction		39				38	
		Total Unit Weight[2] (pcf)		119.3				123.5	

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

E-1 - Interstate Highway 610 and Interstate Highway 45 Interchange, Houston, Texas

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

TABLE.- Summary of Tests Results

Site and Sample Number			E-1-17	a	b				
Depth			69-70						
Penetration Resistance, N			74						
Classification Tests	Percent Passing No. 200 Sieve		32.32						
	Uniformity Coef., C_u		--						
	Curvature Coef., C_c		--						
	Plastic Limit		22.1						
	Liquid Limit		28.2						
	Unified Classification		SM						
Direct Shear Test	Shear Strength at Failure (psi)			9.0	16.2				
	Moisture Content	Before test (%)		20.3	20.0				
		After test (%)		21.0	--				
	Unit Weight[1] (pcf)			118.5	123.5				
	Angle of Internal Friction		42						
Total Unit Weight[2] (pcf)			130.3						

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

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

NOTES