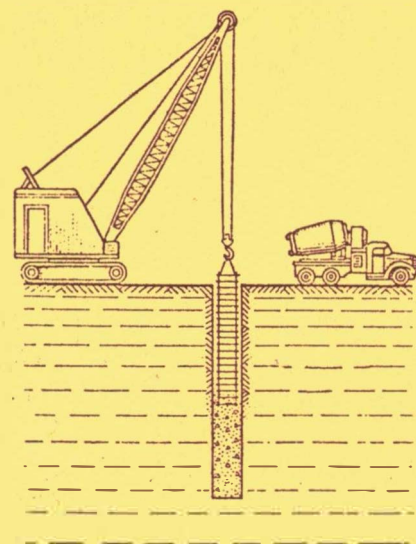
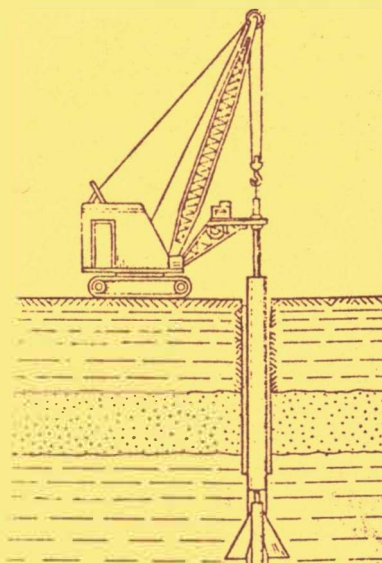
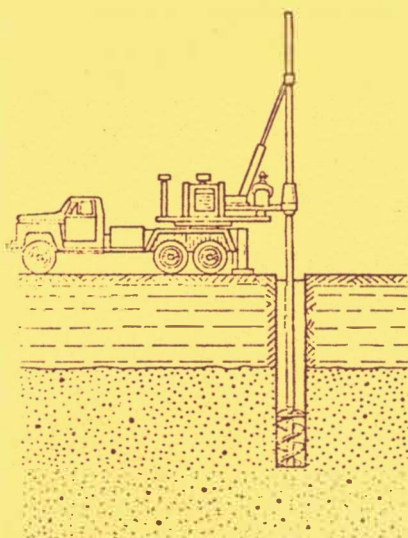


Implementation of Research Findings: **DRILLED SHAFT INVESTIGATION**

RESEARCH AND DEVELOPMENT DIVISION
OKLAHOMA DEPARTMENT OF TRANSPORTATION

Prepared in cooperation with the
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

DECEMBER, 1981

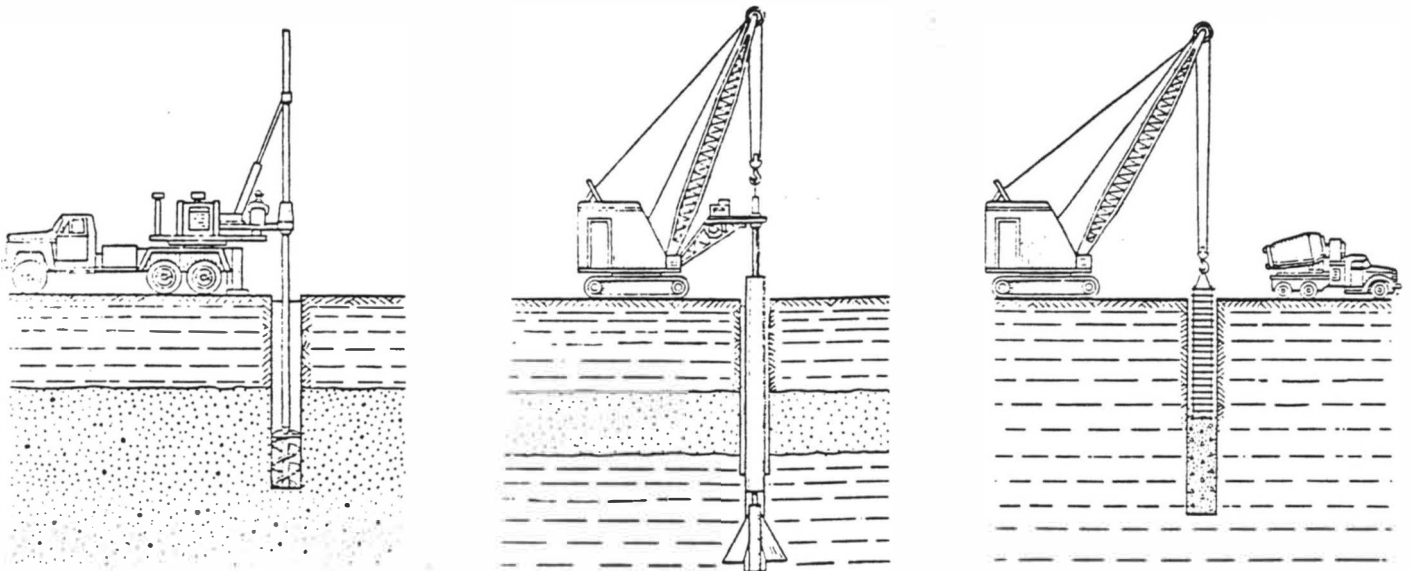


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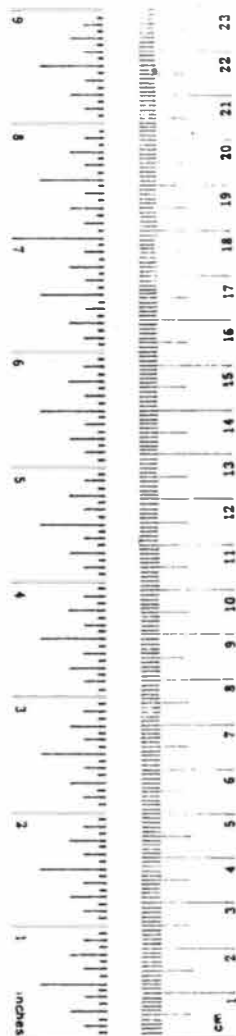
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
LENGTH				
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
AREA				
in ²	square inches	6.5	square centimeters	cm ²
ft ²	square feet	0.09	square meters	m ²
yd ²	square yards	0.8	square meters	m ²
mi ²	square miles	2.6	square kilometers	km ²
	acres	0.4	hectares	ha
MASS (weight)				
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
VOLUME				
teap	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.96	liters	l
gal	gallons	3.8	liters	l
ft ³	cubic feet	0.03	cubic meters	m ³
yd ³	cubic yards	0.76	cubic meters	m ³
TEMPERATURE (exact)				
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* 1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price \$2.25, SD Catalog No. C11.10.286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
LENGTH				
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
AREA				
cm ²	square centimeters	0.16	square inches	in ²
m ²	square meters	1.2	square yards	yd ²
km ²	square kilometers	0.4	square miles	mi ²
ha	hectares (10,000 m ²)	2.5	acres	
MASS (weight)				
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
VOLUME				
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m ³	cubic meters	36	cubic feet	ft ³
m ³	cubic meters	1.3	cubic yards	yd ³
TEMPERATURE (exact)				
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



**Implementation of Research Findings:
DRILLED SHAFT INVESTIGATION**

by

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Oklahoma City, Oklahoma

December 1981

The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the Oklahoma Department of Transportation or the Federal Highway Administration.

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INTRODUCTION

A drilled shaft is a foundation element formed by boring a cylindrical hole into soil and backfilling the hole with concrete. The last three decades have seen a world wide increase in the use of drilled shafts in areas which do not have a good surface soil for foundation. A drilled shaft is often preferred to a driven pile because it is more economical, especially in stiff clay. It also reduces ground heave, noise and vibration on the construction site. In Texas and in the Chicago area, drilled shaft foundations have been built for at least 25 percent less cost than a driven pile foundation.

A most common application of a drilled shaft is its use to resist large axial loads. However, drilled shafts have been used for retaining walls, offshore structures and as tiebacks and anchors. There is a considerable need for more precise information concerning soil strength parameters to be used in drilled shaft design.

Background

In the early part of this century, drilled shaft excavations were done by hand dug methods such as with the "Gow Caission." Gow cassions were cylindrical holes, sometimes several feet in diameter and they were cased with metal tubes that were withdrawn during concrete placement.

In the 1920's machine excavation began to be used for drilled shafts in the United States. The record shows that horse-driven rotary machines were used to dig holes in Texas about 1920 for shafts 25 feet (7.62 m) or more in depth.

In the late 1940's, drilling contractors introduced casing and drilling mud into boreholes, a procedure long established in the oil industry, to cut through permeable soils below the water table. Rotary drilling rigs became standardized and began to be mass produced, giving further assistance to drilled shaft construction. Today the two basic types are crawler-crane mounted rigs for larger boreholes and truck mounted rigs for smaller boreholes.

The first drilled shaft constructed by the Oklahoma Department of Transportation was done in 1962 and was 18 inches (457 mm) in diameter. The longest is 78 feet (23.77 m) with 6 feet (1.83 m) in diameter. A 74 ft. (22.55 m) long and 8 ft. (2.44 m) in diameter shaft now is under the construction along SH 20 in Osage County. The reasons for using drilled shafts instead of piles in Oklahoma are stability and faster construction.

Purpose

The purpose of this report is to characterize soil properties of Oklahoma flood plain (alluvial) soils for drilled shaft design. Also, to establish a correlation between shear strength (S_u) obtained from unconfined compression test and N values obtained from both the Texas Cone and Standard Penetration Test for Oklahoma flood plain soils.

Scope

This study evaluates a variety of flood plain soils at seven sites throughout the state of Oklahoma. For each site, Texas and Standard Penetrometers were used to find N values for different types of soils in flood plain deposits. Two holes were bored to obtain undisturbed and disturbed samples. The gradation, Atterberg limit tests and percentage of sand, silt, and clay were determined in the lab. Shear strength was determined by the unconfined compression test..

SAMPLING AND TEST METHODS

Sampling

Samples were taken by using Shelby tube, piston tube, and Denison core barrel devices.

Thin-walled (Shelby) tube sampling was done according to AASHTO T-207. The samples were then sealed in ULTRAFLEX-WAX and protected from shocking or jarring by foam rubber.

Piston tube sampling was similar to AASHTO T-207 procedure. The difference was that as the sampler is lowered into hole the piston inside the tube prevents filling of the tube.

Denison sampling was similar to AASHTO T-225 procedure except that samples were handled as described for thin-walled tube sampling.

Test Methods

Both the penetration tests were done according to AASHTO T-206 except for Texas Highway Department Cone Penetrometer test, a 170 pound (77 kg) hammer was used and the distance which the hammer fell was 24 inches (610 mm). (1) The cone was seated at the bottom of the hole by 12 blows.

The following table shows the type of tests and methods which were used to find the soil properties.

Type of Lab Test and Method

<u>Type of Test</u>	<u>Test Method</u>
Unconfined Compression	AASHTO T-208
Sieve Analysis	AASHTO T-88-72
Liquid Limit	AASHTO T 89-68
Plastic Limit	AASHTO T 90-70

SITE DESCRIPTION

In order to locate and study a variety of soils, seven sites were chosen throughout the state. These sites are designated as A, B, C, D, E, F and G, respectively. Figure 1 shows the approximate location of the test and sampling sites.

Locations

Site A is located in the center of the SE quarter of Sec 13, T26N, R6W at Osage Creek approximately 1/2 mile north of the town of Jefferson in Grant County.

Site B is located on Sec 28, T22N, R7W, at Clear Creek, 2 1/2 miles west of the city of Enid in Garfield County.

Site C is located in the SW corner of Sec 32, T16N, R11W at the Canadian River south of the city of Watonga in Blaine County.

Site D is located about 1200 ft. W of the NE corner of Sec 17, T16N, R3W in the flood plain approximately 6 1/2 miles west of the city of Guthrie in Logan County.

Site E is located in Sec 14, T3N, R26W at Sandy Creek north of the city of Hollis in Harmon County.

Site F is located in the north half of Sec 10, T3N, R6E in the flood plain approximately 2 miles SE of the city of Ada in Pontotoc County.

Site G is located in the center of the west side of Sec 18, T8N, R22E in the flood plain approximately 2 miles NE of the town of Cameron in LeFlore County.

Geology and Soils

Sites A, E, F, and G consist of soft and stiff silty clay layers which are present to a depth of approximately 20 feet (6.1 m). These materials are brown, reddish brown, gray and red in color. They become very stiff and moist with depth. These layers are resting on compact shale which is hard, moist, red or reddish brown.

Site D is as described above except the top layers to a depth of approximately 9 feet (2.7 m) are clayey sand and sand, loose, slightly moist, and brown in color. From 9 feet (2.7 m) to 29 feet (8.8 m) is silty clay, stiff, moist, and brown in color. Beneath the silty clay layer, there is 6 feet (1.8 m) of silty sand, medium compact, wet, and pale brown in color, which rests on sandstone.

Both Site B and C consist of silty sands, which are loose, moist, and pale brown in color with fewer fines with the layers becoming stiffer with depth. At approximately 20 feet (6.1 m) the boring encounters silty-shale which is hard, moist, and brown in color. For boring logs see Figures 2 through 8.

Depth Feet	Soil Symbol	Unified Classification	Description of Stratum	Texas Cone Penetrometer blows / ft.	Standard Penetrometer blows / ft.
5			Silty clay, loose, moist becoming more moist with depth, dark reddish brown		
10		CL CL CL	do	14	8
15					
20		CL CL CL	do	13	
25		SW	Well graded sands, with small gravels, medium compact, wet, pale brown		20
		CL	Shale, silty, hard, dry, reddish brown	114	68 171

Figure 2. Boring Log for Site A

Depth Feet	Soil Symbol	Unified Classification	Description of Stratum	Texas Cone Penetrometer blows / ft.	Standard Penetrometer blows / ft.
		SM	Silty sands, loose, slightly moist, pale brown, contains few clay balls	9	—
5		SM	 do	6	9
10		SM	do	36	Sunk 2.1 ft. under weight of tools
15		SP	Poorly graded sands, little to no fines, somewhat coarser than above, loose, pale brown		
20					
25					
		CL	Shale, silty clay, hard, brown	960	244

Figure 3. Boring Log for Site B

Depth Feet	Soil Symbol	Unified Classification	Description of Stratum	Texas Cone Penetrometer blows / ft.	Standard Penetrometer blows / ft.
5		SM	 Silty sands, loose, moist, reddish brown		
		SW SM	do with somewhat less fines, becoming yellowish red with depth	19	8
10					
		SW SM	do	7	7
15					
				400	171
20					
		ML	Silty shale, hard moist, reddish brown	370	—
25					

Figure 4. Boring Log for Site C

Depth Feet	Soil Symbol	Unified Classification	Description of Stratum	Texas Cone Penetrometer blows / ft.	Standard Penetrometer blows / ft.
5		SC SW	Clayey sand & sand, alternating layers; loose, slightly moist, brown	—	—
10		SP	Poorly graded sand, loose, slightly moist, pale brown	—	—
15		CL CL	Silty clay, stiff, moist, brown	17	10
20		CL CL	do	13	7
25					8
30		CL	 Silty sand, medium compact, wet, pale brown		16
35		SM SM		71 71	18 18
40			Sandstone, fine grained, poorly cemented, slightly moist		

Figure 5. Boring Log for Site D

Depth Feet	Soil Symbol	Unified Classification	Description of Stratum	Texas Cone Penetrometer blows / ft.	Standard Penetrometer blows / ft.
10		CL	Silty clay, very stiff, moist, becoming more moist with depth, yellowish red	39	24
20					
		CL	do except becoming hard, may be weathered shale	33	33
30		CL		35	21
		CL		51	19
40		CL	Shale, hard, moist, red, contains gypsum vein fillings, occasional hard-siltstone layers less than 0.1 ft. thick	350	
				556	53

Figure 6. Boring Log for Site E

DEPTH IN FEET	SOIL SYMBOL	UNIFIED CLASSIFICATION	DESCRIPTION OF STRATUM	TEXAS CONE PENETROMETER BLOWS / FT	STANDARD PENETROMETER BLOWS / FT.
		CL	SILTY CLAY HARD MOIST BROWN	—	—
5			SANDSTONE, MEDIUM HARD, GRAY	—	—
		CL	SILTY CLAY, STIFF, MOIST, BROWN		
10		CL	WEATHERED, SILTY CLAY, SHALE, HARD, MOIST, GRAY, OLIVE	35 —	— 46
15		CL	SILTY CLAY, HARD, MOIST, RED	51	—
			LIMESTONE, BROKEN, MOIST, RED		
20				50	86
25		CL	SILTY CLAY, SHALE, HARD, MOIST, RED	100 125 150	— 120 150
30					

Figure 7. Boring Log for Site F

Depth Feet	Soil Symbol	Unified Classification	Description of Stratum	Texas Cone Penetrometer blows / ft.	Standard Penetrometer blows / ft
		ML	Silty clay, stiff, slightly moist, highly mottled		
5		CL CL			
10		do CL CL CL CL CL SM	do becoming very stiff, moist progressively coarser with depth 	32	18
15				24	16
20			Soil & shale mixture, stiff, moist, brown	61	11
			Shale, sandy, hard, laminated		2000

Figure 8. Boring Log for Site G

TEST RESULTS AND DISCUSSION

Alluvial soils deposits throughout the State of Oklahoma are variable. It was important to choose an appropriate classification system and develop correlations with shear strength and resistance to penetration for each type of soil.

Soil Condition and Classification

The Unified system is one of the most widely used systems to classify soils. In order to determine the Unified soil classification, particle size analysis and Atterberg limits are needed. These and other properties are shown in Appendix A. Sites A, D, E, F and G were predominantly CL soils. Site B is predominantly SM and SW, and Site C is composed of SM and SP soils. Figure 9 shows the Unified classification of site D soils layers as an example.

The major objective of the report was to establish a correlation between shear strength, S_u and resistance to penetration, N value for each soil type.

The standard penetration test and Texas cone penetration test has been used by ODOT. The Soil Foundations Branch of the Materials Division is using the standard penetration test and the Bridge Division is using the Texas cone penetrometer to obtain data for foundation design. The standard penetration has the advantage of being able to obtain soil samples. The Texas cone penetrometer is being used primarily because the charts and curves from the Texas Department of Highways and Public Transportation are available for foundation design. (4) The Texas cone penetrometer also has the advantage of being able to penetrate soft rock as well as soil.

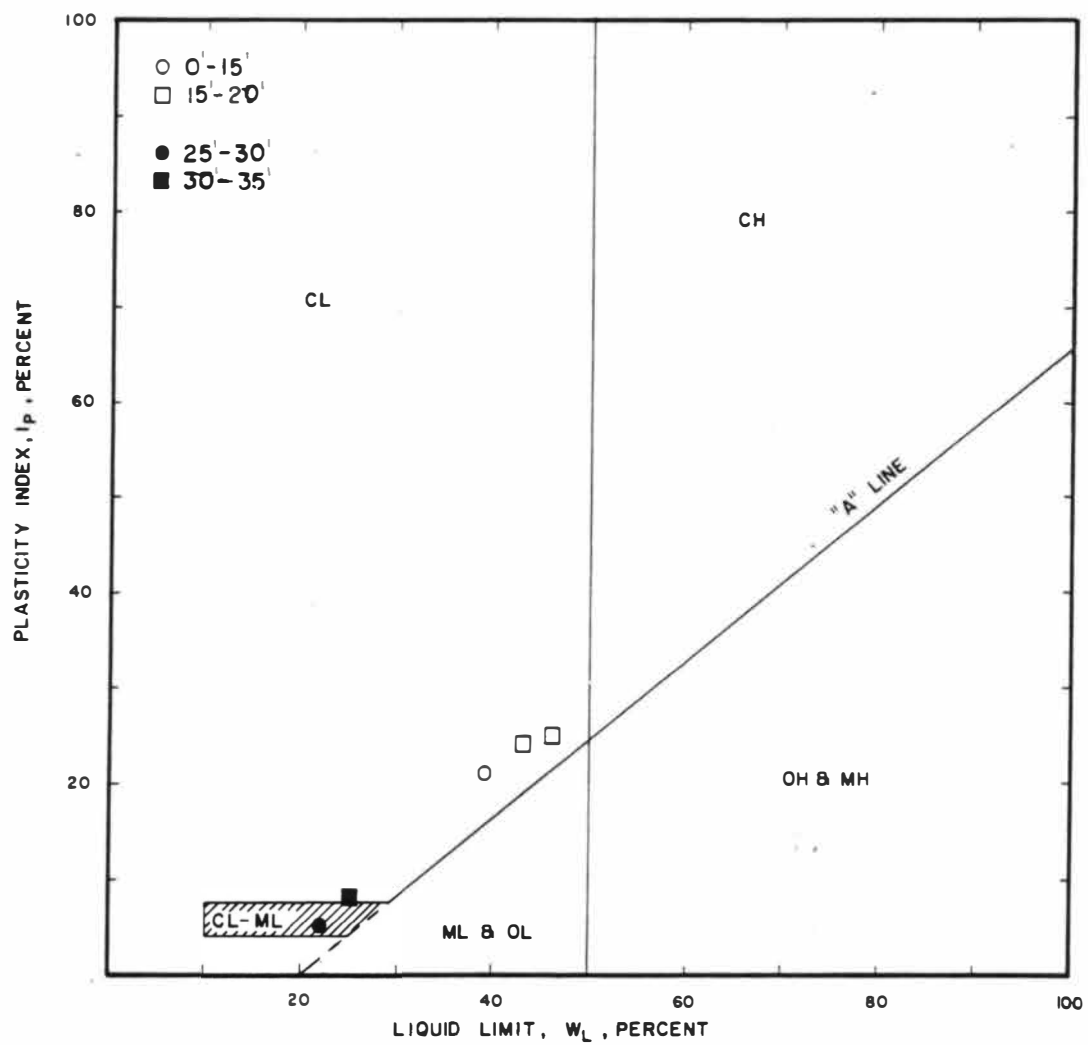


Figure 9. The Unified Classification of Site D Soil Layers

The data for each site is summarized in Tables A-1 through A-7 in Appendix A which shows N values and unconfined compressive strengths for each site.

Test Methods Discussion

The unconfined compression test is one of the simplest and most widely used to find shear strength for cohesive soils. (3) This test will generally give a conservative strength value for soils. There are opinions that shear strength obtained from unconfined compression tests is about 50 percent (range from 30 to 80 percent) smaller than actual in-situ strength determinations. Other investigators have reported that the unconfined compression test gives as valid soil values as a triaxial test. (3) One of the advantages that the unconfined compression test has over the direct shear test is that the shearing stress and strain distribution are more uniform than in direct shear. Another advantage of the unconfined compression test is that a failure surface will tend to develop in the weakest portion of the sample which is unlike the forced shear plane of the direct shear tests.

The triaxial test has superiority over the unconfined and the direct shear test. (13) The shear strengths obtained are closer to the in situ strength than those found from unconfined compression or direct shear. The Oklahoma Department of Transportation seldom uses the triaxial test for shear strength determination except for dam foundation purposes. Figure 10 shows a comparison of shear strength profiles obtained by various methods. (11)

Laboratory Test

The unconfined compression test method was chosen for this study, because it was felt that it would give conservative strength values and is simple and less expensive. The "Quick" or unconsolidated undrained (UU) shear

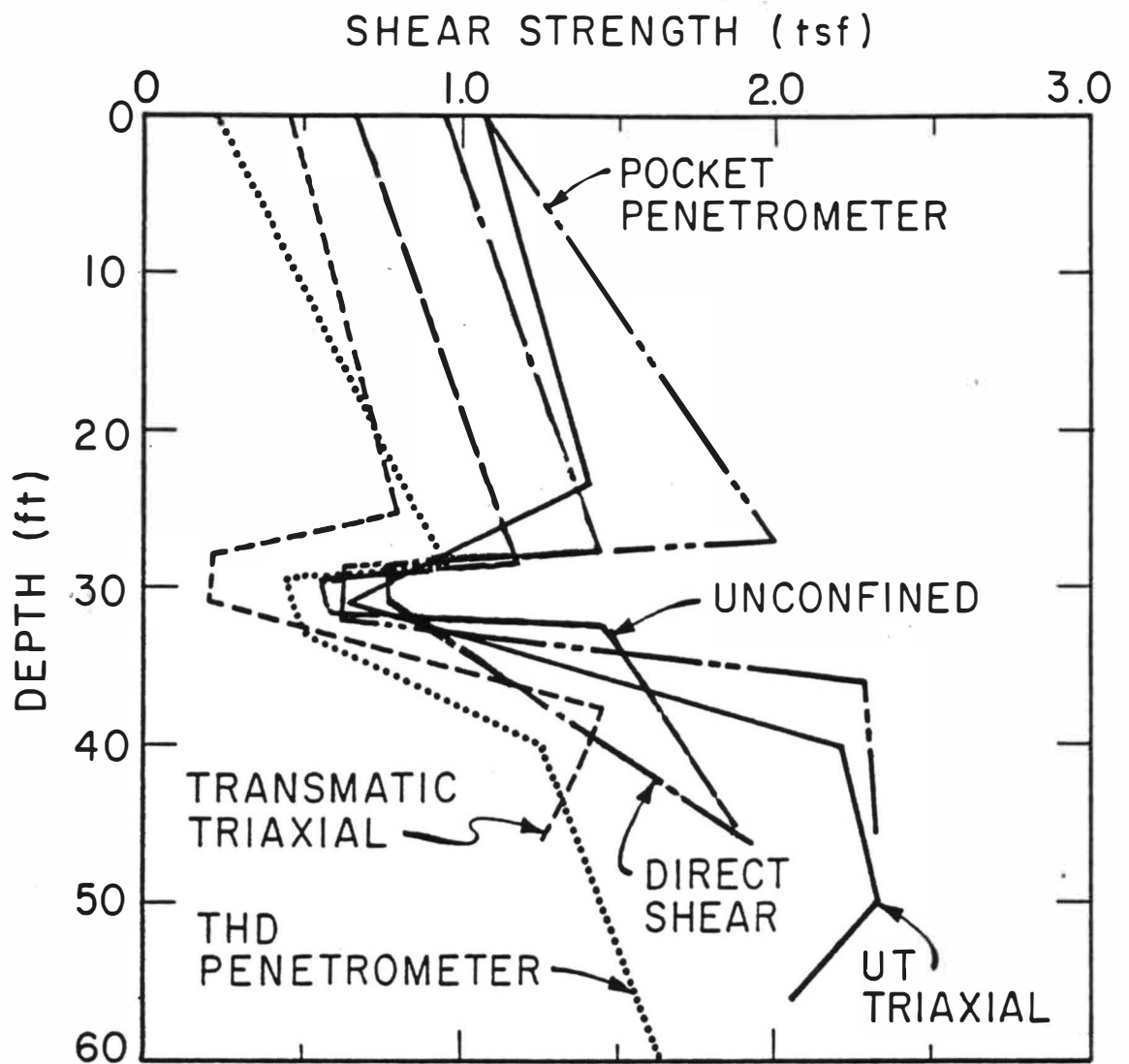


Figure 10. A Comparison of Shear Strength Profiles Obtained by Various Methods (from O'Neill and Reese)

strength test is the most commonly used test at the Oklahoma Department of Transportation to obtain the shear strength of the soil.

Soil shear strength in ton/sq. ft. for Sites A, D, E, F, and G was determined by the unconfined compression (UU) test. There was no data for site B and C, since these soils were too soft and weak to sample.

ANALYSIS OF RESULTS

The relationships between resistance to penetration, N value, and UU shear strength (S_u) are not always constant. It is necessary to discuss factors influencing N value, and S_u , before correlations are established.

Factors Affecting Resistance to Penetration, N.

Many researchers have investigated the factors affecting N value. There are many variables involved in the resistance to penetrometer penetration rates. DeSai (7) states that the driving of a cone would cause an upward displacement of the subsoil until a certain depth or surcharge pressure is reached which will not permit such displacement. He also concluded that the density, structure, depth, and ground water table will have a considerable effect on the cone penetrometer resistance. Gibbs and Holtz (8) conducted research with standard penetration tests in sand and concluded, "The overburden pressures were found to have the most pronounced and consistent effects on the penetration resistance values." Schultz and Knansenberger (12) report that "Dynamic penetrometers react very sensitively to any change of compactness or grain size". Although the researchers do not arrive at the same conclusion concerning the factors which have the most effect, they all agree that unit weight, grain size, moisture content, and overburden pressure are the major factors affecting the N value. These opinions are presented in the text of Dynamic and Static Sounding of Soils by Bodarik. (2)

The data from this study shows that there are recognizable effects, of moisture content, overburden pressure, and grain size on the N value for most sites. Figure 11 shows correlations between N value and unconfined compression shear strength. (6)

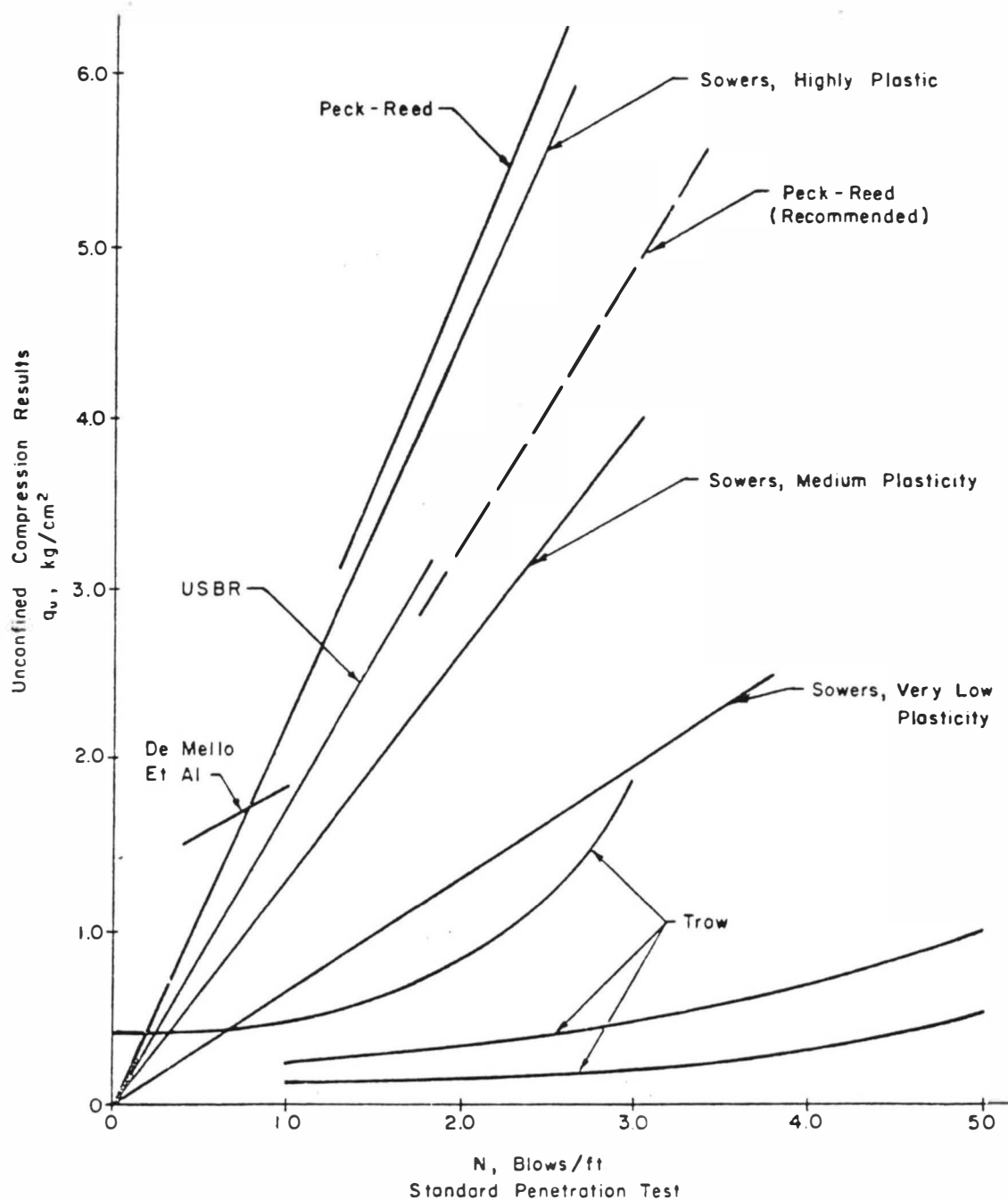


Figure 11. Correlation between N Value and Unconfined Compression Shear Strength for Clays attempted by Several Researchers
(from DeMello)

Factors Affecting Shear Strength

The shear strength of soil depends on many factors. One of the main factors is secondary structure of the soil. Lab tests usually will give low strength values when planes of failure in the test specimen follow joints or slickensides and yield higher shear strength values when planes of failure and joints intersect each other. There were no weakness planes observed in the test samples.

The test methods used in the lab influence the shear strength values. The shear strength results that were obtained in the lab may not represent the actual strength of the soil in situ. The shear strength of soils also depends on the angle of internal friction, and normal pressure (effective overburden pressure) acting on the soil. Means and Parcher (10) have reported that factors affecting the shear strength are density, void ratio, grain size and shape, gradation, and moisture content. Most of these factors are affected by the same factors influencing the resistance to penetration and have been described previously.

Correlation of N with Su

Since the resistance to penetration has been affected by most of the same factors as shear strength, a relationship should exist between shear strength, Su, and resistance to penetration, N value. It is necessary to evaluate the constant proportion between the two parameters by using the linear equation:

$$Su = KN$$

where K is a constant that varies for each type of the soil. Three steps were used to evaluate the constant K.

1. Soils were classified into groups with similar properties.
2. Plots were made for Su vs. N for each group.
3. A best fit linear curve was established for each group.

The first step was to place soils into groups of similar properties. The Unified classification was used to group the soils. The CH soils were placed in one group. The CL soils were divided into two groups, silty CL, and sandy CL. The SC materials were placed in another group. The cohesionless soils (SM, SP, SW) were placed in still another group.

The second step was to plot S_u vs. N value for each subgroup. The borings for sampling and borings for the penetration tests were in close proximity. There was no unconfined compression test data available for the sandy CL, CH, SC, SM, and SP soils. However, it was decided to use published data for sandy CL, CH, SC, SM, and SP soil properties. (5, 9)

The third step was to find the linear curve. The slope of the curve represents the constant K . The data for silty CL soils are given in Tables A-1, and A-4 through A-7. The data is plotted in Figure 12. There is a relatively good linear relationship existing between S_u and N value for silty CL soils. The equation obtained for silty CL soils is

$$S_u = 0.060 N_{THDP} \quad (1)$$

where S_u is unconsolidated undrained shear strength (ton/sq. ft.) obtained from unconfined compression test and N_{THDP} is resistance to penetration (blows/ft.) as obtained from the Texas Highway Department Cone penetrometer Test. This equation was compared to the equation that was obtained from the Texas Highway Department (THD) (9) for silty CL soils. The THD equation is as follows:

$$S_u = 0.063 N_{THDP} \quad (2)$$

where in this case, S_u , is unconsolidated undrained shear strength (ton/sq. ft.) obtained from ASTM 2850-7 triaxial test. Equations (1) and (2) are nearly equal. When comparing these two equations, it is worth noting that the equations resulting from the Texas Highway Department data (designated by

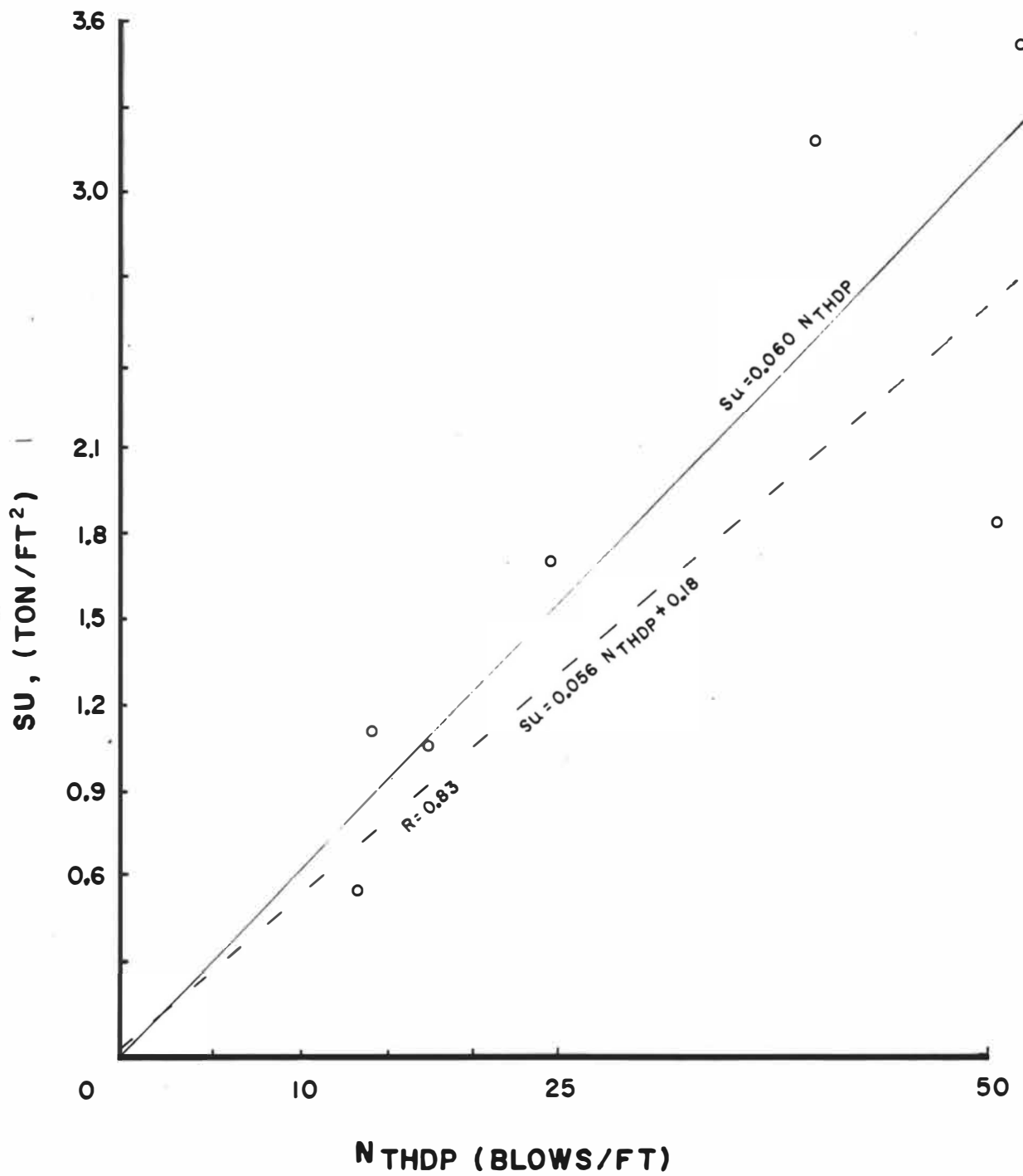


Figure 12. ODOT Correlation between N_{THDP} and Su for Silty CL Soils

asterisks) for CH, SP, sandy CL, and SM soils can be applied to Oklahoma soils. Therefore, the following equations will be valid.

$$\text{For homogeneous soil; } Su = 0.07 N_{THDP} \quad (3^*)$$

$$\text{For Sandy CL soil; } Su = 0.05 N_{THDP} \quad (4^*)$$

$$\text{For Cohesionless soil; } Su = 0.02 N_{THDP} \quad (5^*)$$

where Su is in ton/sq. ft. and N_{THDP} is blows/ft.

It was also desireable to determine the relationships between the Standard Penetration Test (N_{STDP}) and the Texas Highway Department Penetrometer (N_{THDP}), as well as the relationship between the Unconfined Compression (Su), and (N_{STDP}) values for Oklahoma soils. However, data were only available for silty CL soils. The Su and N values are shown in Tables A-1 and A-4 through A-8. These values for the silty CL soils were plotted to show their relationships (Figures 13, 14). The shear strength (Su) is ton/sq. ft. and N_{STDP} is blows/ft. The correlations are as follows:

$$N_{STDP} = 0.63 N_{THDP} \quad (6)$$

$$Su = 0.12 N_{STDP} \quad (7)$$

Touma and Reese (14) reported the correlations between N_{THDP} and N_{STDP} for cohesive and cohesionless soils (Figures 15 and 16) which are as follows:

$$N_{STDP} = 0.70 N_{THDP} \quad (8^*)$$

$$N_{STDP} = 0.50 N_{THDP} \quad (9^*)$$

Equation (8*) represents the cohesive soils having medium to high plasticity. Equation (9*) represents the cohesionless soils.

Equations (4*) and (6) were used to find the correlation between Su (tons/sq. ft.) and N_{STDP} (blows/ft.) for sandy CL, which is as follows:

$$\text{For sandy CL soils } Su = 0.08 N_{STDP} \quad (10)$$

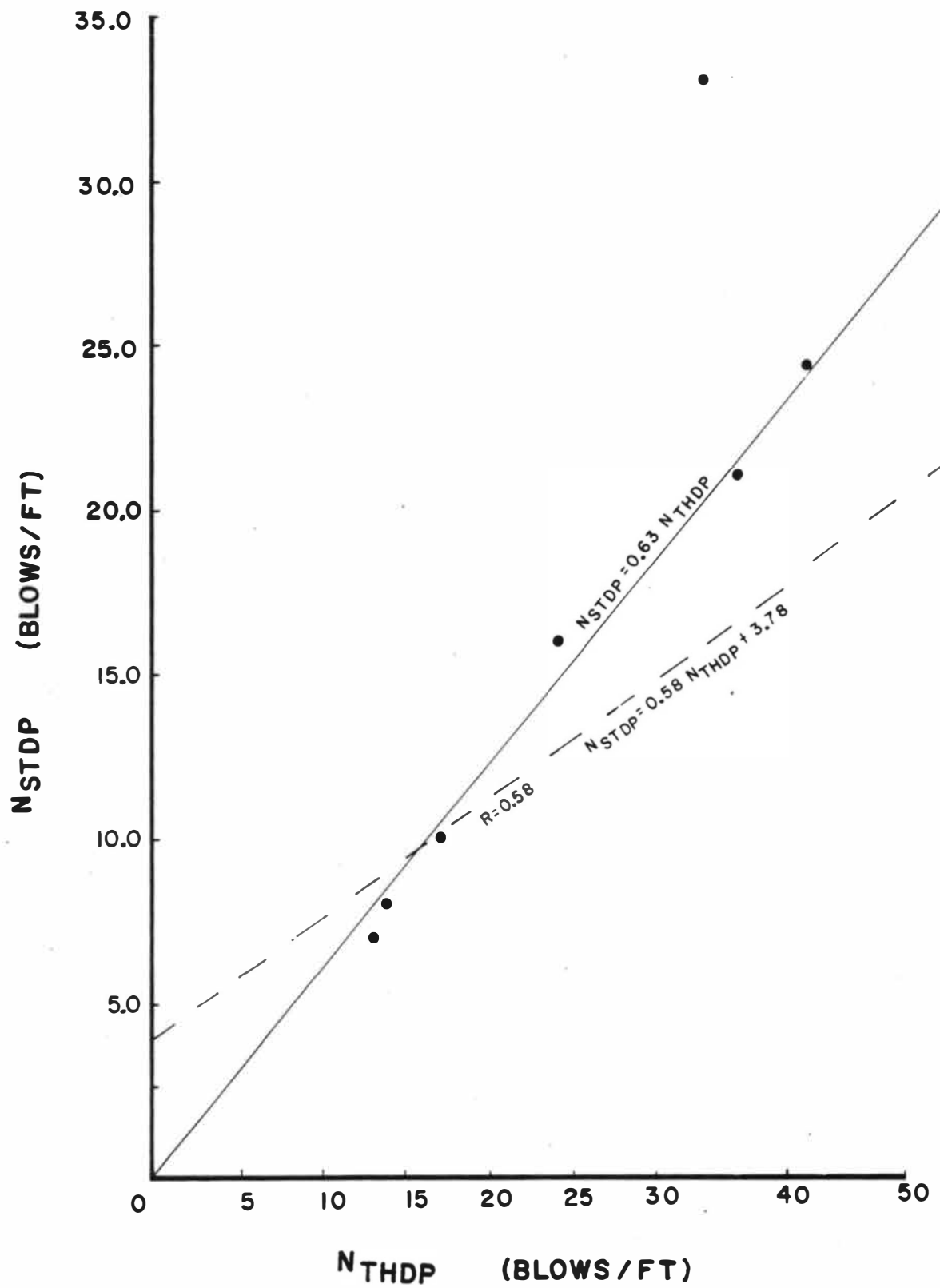


Figure 13. ODOT Correlation between N_{THDP} and N_{STDP} for Silty CL Soils

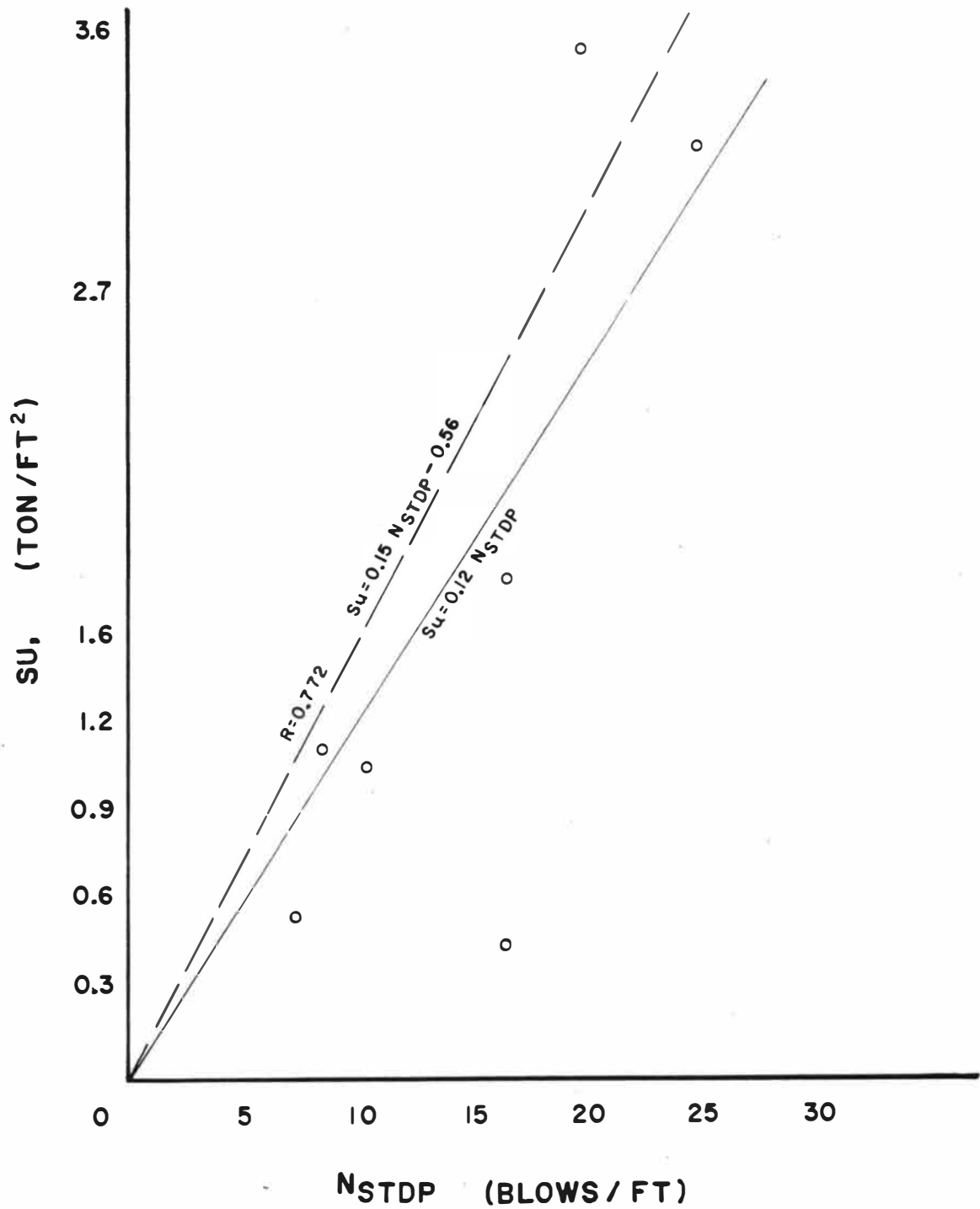
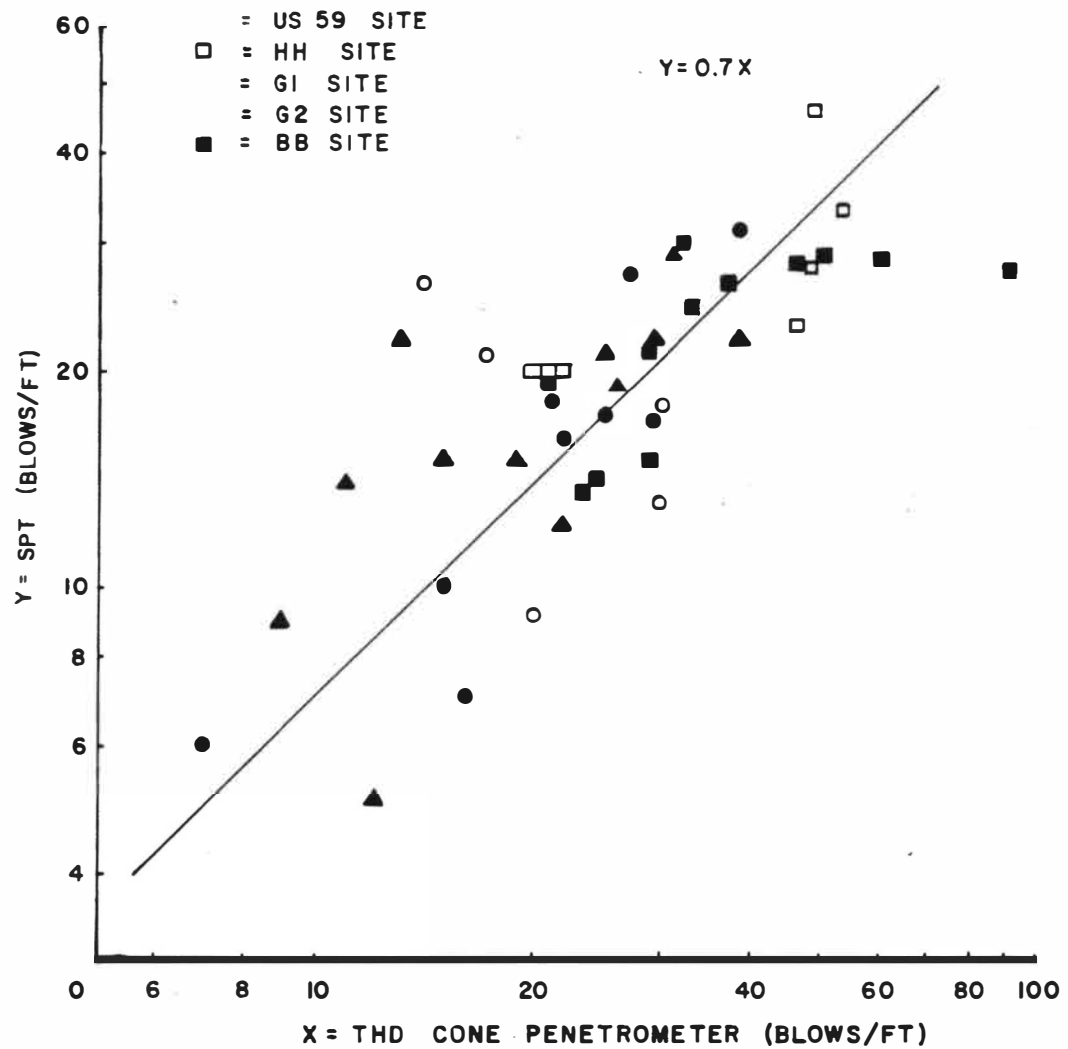


Figure 14. ODOT Correlation between N_{STDP} and S_u for Silty CL Soils



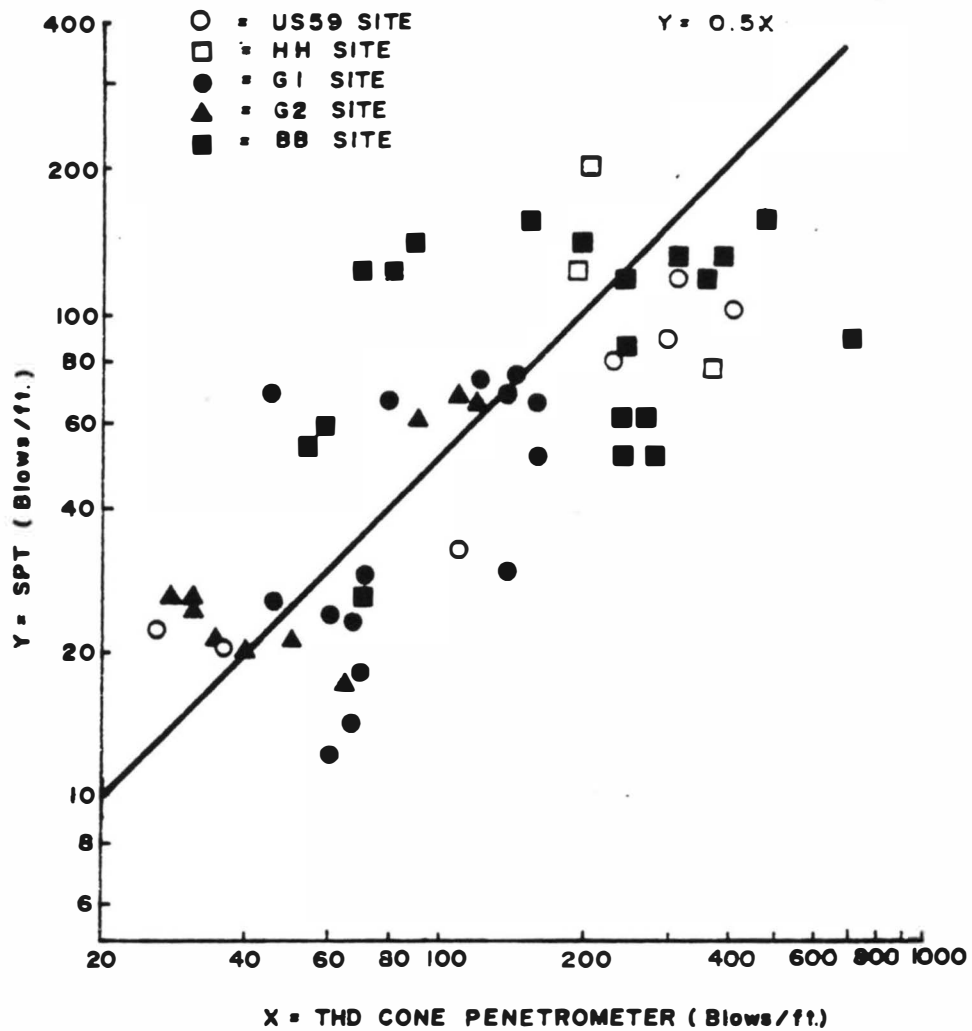


Figure 16. Correlation between the SPT and THD Cone Penetrometer in Sand as Obtained by Touma and Reese.

Equations (1) and (6) were used to obtain the correlation between S_u (ton/sq. ft.) and N_{STDP} (blows/ft.) for silty CL soils which is as follows:

$$S_u = 0.09 N_{STDP} \quad (11)$$

Comparing the two equations (7) and (11), it was decided to adopt equation (11) for silty CL soils, because it gives a conservative result. Equations (3*) and (8*) were used to obtain the correlation between S_u (ton/sq. ft.) and N_{STDP} (blows/ft.) for homogeneous CL soil in Oklahoma, which is as follows:

$$S_u = 0.10 N_{STDP} \quad (12)$$

Equations (5*) and (9*) were used to obtain the correlation between S_u (ton/sq. ft.) and N_{STDP} (blows/ft.) for cohesionless soil in Oklahoma, which is as follows:

$$S_u = 0.04 N_{STDP} \quad (13)$$

The correlation coefficient (r) for the relationship between S_u and N_{THDP} (Figure 12) is 0.83. The r -value is between 95 percent to 99 percent valid. It was assumed that when N value is zero, the resulting shear strength is also zero. The correlation becomes as in equation (1). When boundary conditions were specified as noted above, the r -value has no meaning.

The r -value for the relationship between N_{STDP} and N_{THDP} (Figure 13) is 0.58. The correlation coefficient is between 80 percent to 90 percent valid.

The r -value for the relation between N_{STDP} and S_u (Figure 14) is 0.772. The r -value is between 95 percent to 99 percent valid.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The correlations between S_u and N_{THDP} as well as S_u and N_{STDP} have been developed for homogeneous CH, silty CL, sandy CL, and cohesionless soils.

The following conclusions concerning this study are made.

1. The shear strength from unconfined compression tests can be predicted if the Texas Highway Department cone penetrometer N value is known by using the following equations:

$$S_u = 0.06 N_{THDP} \quad \text{for silty CL soils}$$

$$S_u = 0.07 N_{THDP} \quad \text{for homogeneous CH soils}$$

$$S_u = 0.05 N_{THDP} \quad \text{for sandy CL soils}$$

$$S_u = 0.02 N_{THDP} \quad \text{for cohesionless soils}$$

2. The relationship between the Texas Highway Department cone penetrometer and the standard penetration test for silty soils was established as follows:

$$N_{STDP} = 0.63 N_{THDP}$$

3. The equations for the relationship between S_u and N_{STDP} were developed as follows:

$$S_u = 0.09 N_{STDP} \quad \text{for silty CL soils}$$

$$S_u = 0.10 N_{STDP} \quad \text{for homogeneous CH soils}$$

$$S_u = 0.08 N_{STDP} \quad \text{for sandy CL soils}$$

$$S_u = 0.04 N_{STDP} \quad \text{for cohesionless soils}$$

4. The developed correlations between S_u and N_{THDP} as well as S_u and N_{STDP} are conservative and can be used to find soil strength.

Recommendations

The following recommendations are felt to be appropriate:

1. More research is needed in order to establish a correlation between resistance to penetration and shear strength for SC, SW, SP, SM soils in Oklahoma.
2. To further the validity of the correlations between the N values and shear strength, additional test data should be obtained and cataloged. After a large number of tests are recorded more accurate and precise correlations can be derived.
3. Additional tests, such as the Dutch cone penetrometer, are needed to better examine the factors that affect the N values of the Texas Highway Department cone penetrometer and standard penetration tests.
4. It is recommended that either the Texas Highway Department cone penetrometer test or the standard penetration test is adequate to calculate the soil shear strength for bridge foundation design purposes.
5. It is recommended that at least one soil sample per project site be tested for shear strength and this data used to modify the appropriate equation for that project. This data should be transmitted to the Research and Development Division for further modification of the equations as appropriate.

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APPENDIX A

Table A-1. Site A, Jefferson, Grant County

SOIL PROPERTIES

Boring No.	Depth, Feet	Moisture Percent	Wet Density PCF				% Passing Sieve Size			% Smaller than			Unified Class
				<u>L.L.</u>	<u>P.L.</u>	<u>P.I.</u>	<u>No.10</u>	<u>No.40</u>	<u>No.200</u>	<u>.05MM</u>	<u>.005MM</u>	<u>.002MM</u>	
A-1	8	22.0	120.6	-	-	-	-	-	-	-	-	-	-
B-1 & B-2	18	17.4	135.2	21	14	7	100	90	73.7	64	24	20	CL-ML
C-1 & C-2	33	15.8	136.4	27	17	10	100	99	98.8	94	40	28	CL

UNCONFINED COMPRESSIVE STRENGTH

Boring Number	Unconfined Compressive Strength TSF	Strain at Failure Percent	<u>N_{THDP}</u>	<u>N_{STDP}</u>
A-1	-	-	-	-
B-1 & B-2	1.09	3.9	14	8
C-1 & C-2	-	-	-	-

Table A-2. Site B, Enid, Garfield Co.

SOIL PROPERTIES

<u>Boring No.</u>	<u>Depth feet</u>	<u>Moisture Percent</u>	<u>Wet Density PCF</u>				<u>% Passing Sieve Size</u>			<u>Unified Class</u>
				<u>L.L.</u>	<u>P.L.</u>	<u>P.I.</u>	<u>No.10</u>	<u>No.40</u>	<u>No.200</u>	
2A	3.5-3.85	N/A*	N/A	NP**	NP	NP	100	67	22.4	SM
1A	5.3-6.8	N/A*	N/A	NP**	NP	NP	100	72	11.2	SW-SM
2B	12.0-13.0	N/A*	N/A	NP**	NP	NP	93	33	5.0	SW-SM
1B	12.5-13.5	N/A*	N/A	NP**	NP	NP	100	25	3.4	SW
1C	19.5-20.1	N/A*	N/A	NP**	NP	NP	100	97	91.1	ML

* Not Available

** Non Plastic

UNCONFINED COMPRESSIVE TEST

Not Available

Table A-3 Site C, Watonga, Blaine Co.

SOIL PROPERTIES

<u>Boring No.</u>	<u>Depth feet</u>	<u>Moisture Percent</u>	<u>Wet Density PCF</u>	% Passing Sieve Size						<u>Unified Class</u>
				<u>L.L.</u>	<u>P.L.</u>	<u>P.I.</u>	<u>No.10</u>	<u>No.40</u>	<u>No.200</u>	
2A	1.1-3.2	N/A*	N/A	NP**	NP	NP	100	95	18.6	SM
1A	5.0-6.5	--	--	NP	NP	NP	100	87	18	SM
2B	12.4-14.4	--	--	NP	NP	NP	100	88	17.2	SM
2F	18.5-20.0	--	--	NP	NP	NP	100	63	4.4	SP
1C	21.5-27.9	--	--	31	21	10	100	86	68.4	CL

* Not Available

** Non Plastic

UNCONFINED COMPRESSIVE TEST

Not Available

Table A-4. Site D, Guthrie, Logan County

SOIL PROPERTIES

Boring Number	Depth Feet	Moisture Percent	Wet Density PCF				%Passing Sieve Size			% Smaller Than			Unified Class
				L.L.	P.L.	P.I.	No.10	No.40	No.200	.05MM	.005MM	.002MM	
A-1	18.0	27.6	120.1	46	21	25	100	100	96.2	87	41	34	CL
A-2	19.0	30.4	120.7	43	19	24	100	100	93.5	84	38	32	CL
B-1	27.0	20.4	127.8	22	17	5	100	100	57.4	37	15	13	CL-ML
C-1 & C-2	13.0	25.0	121.8	39	18	21	100	100	92.3	80	35	30	CL
D-1	32.0	22.1	129.5	25	17	8	100	100	55.0	39	18	6	CL
B-2	27.0	20.4	133.8	--	--	--	--	--	--	--	--	--	--
D-2	32.0	26.2	125.6	--	--	--	--	--	--	--	--	--	--

UNCONFINED COMPRESSIVE STRENGTH

Boring Number	Unconfined Compressive Strength, TSF	Strain at Failure, Percent	N _{THDP}	N _{STDP}
A-1	1.04	3.2	17	10
A-2	0.52	2.8	13	7
C-1 & C-2	1.15	6.7	-	-
D-1	0.52	4.8	-	-
B-2	0.45	13.0	-	-
D-2	0.42	14.0	-	16

Table A-5. Site E, Hollis, Harmon County

SOIL PROPERTIES

Boring Number	Depth Feet	Moisture Percent	Wet Density PCF				%Passing Sieve Size			% Smaller than			Unified Class
				L.L.	P.L.	P.I.	No.10	No.40	No.200	0.5MM	.005MM	.002MM	
A-1-A & A-1-B	19	17.3	127.2	NP	NP	NP	100	96	26.6	23	16	14	SM
A-2	49	19.3	--	27	18	9	100	92	78.0	-	-	-	CL
B-1	19.20	13.9	--	19	17	2	100	100	48.0	30	17	14	SM
B-2	20-21	19.2	--	31	20	11	100	99	91.4	81	30	22	CL
B-3 & B-4	30-32	29.2	119.0	--	--	--	--	--	--	--	--	--	--
B-5 & B-6	49	19.1	132.2	31	21	10	100	89	74.2	65	32	22	CL

UNCONFINED COMPRESSIVE TEST

Boring Number	Unconfined Compressive Strength, TsF	Strain at Failure, percent	N _{THDP}	N _{STDP}
A-2	3.52	8.2	51	19
B-1 & B-2	3.18	4.2	39	24
-	-	-	35	21
-	-	-	33	33

Table A-6 Site F, Ada, Pontotoc County

SOIL PROPERTIES

<u>Boring Number</u>	<u>Depth, Feet</u>	<u>Moisture Percent</u>	<u>Wet Density PCF</u>				<u>% Passing Sieve Size</u>			<u>% Smaller than</u>			<u>Unified Class</u>
				<u>L.L.</u>	<u>P.L.</u>	<u>P.I.</u>	<u>No.10</u>	<u>No.40</u>	<u>No.200</u>	<u>.05MM</u>	<u>.005MM</u>	<u>.002MM</u>	
A-1	8.5	19.7	128.7	42	18	24	N/A	N/A	N/A	75	51	39	CL
A-2	13.5	16.8	137.0	47	19	28	--	--	--	85	52	43	CL
A-3	18.5	12.6	125.7	48	21	27	--	--	--	87	60	39	CL
A-4	23.5	16.9	137.8	41	23	18*	--	--	--	86	55	30	CL

* Ultrasonic Treatment: LL=47, PI=25

UNCONFINED COMPRESSIVE TEST

<u>Boring Number</u>	<u>Wet Density PCF</u>	<u>Unconfined Compressive Strength, TsF</u>	<u>Strain at Failure, Percent</u>	<u>N_{THDP}</u>	<u>N_{STDP}</u>
A-3	135.5	1.85	1.8	-	-
A-4	141.5	2.21	9.4	-	-

Table A-7. Site G, Cameron, LeFlore County

SOIL PROPERTIES

Boring Number	Depth, Feet	Moisture Percent	Wet Density PCF				% Passing Sieve Size			% Smaller than			Unified Class
				<u>L.L.</u>	<u>P.L.</u>	<u>P.I.</u>	<u>No.10</u>	<u>No.40</u>	<u>No.200</u>	<u>0.5MM</u>	<u>.005MM</u>	<u>.002MM</u>	
B1-A & B1-B	15.0	18.7	128.0	32	17	15	100	99	80.8	70	36	28	CL

UNCONFINED COMPRESSIVE TEST

Boring Number	Unconfined Compressive Strength, TsF	Strain at Failure, Percent	<u>N_{THDP}</u>	<u>N_{STDP}</u>
B1-A & B1-B	1.69	5.2	24	16
-	-	-	32	18

Unified Soil Classification System

Primary divisions			Group symbol	Secondary divisions	Laboratory classification criteria	Supplementary criteria for visual identification	
Coarse grained soils. (More than half of material is larger than No. 200 sieve size.)	Gravels. (More than half of the coarse fraction is larger than No. 4 sieve size.)	Clean gravels. (Less than 5% of material smaller than No. 200 sieve size.)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	Wide range in grain size and substantial amounts of all intermediate particle sizes.	
		... do Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.) ¹	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Not meeting all gradation requirements for GW.	Predominantly one size or a range of sizes with some intermediate sizes missing. Non-plastic fines or fines of low plasticity. Plastic fines.	
			GM	Silty gravels, and gravel-sand-silt mixtures, which may be poorly graded.	Atterberg limits below "A" line, or PI less than 4 ² Atterberg limits above "A" line, with PI greater than 7		Atterberg limits above "A" line with PI between 4 and 7 is borderline case GM-GC
			GC	Clayey gravels, and gravel-sand-clay mixtures, which may be poorly graded.			
	Do Sands. (More than half of the coarse fraction is smaller than No. 4 sieve size.) ... do Sands with fines. (More than 12% of material smaller than No. 200 sieve size.) ¹	Clean sands. (Less than 5% of material smaller than No. 200 sieve size.)	SW	Well graded sands, gravelly sands, little or no fines.	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	
		Sands with fines. (More than 12% of material smaller than No. 200 sieve size.) ¹	SP	Poorly graded sands, gravelly sands, little or no fines.	Not meeting all gradation requirements for SW	Predominately one size or a range of sizes with some intermediate sizes missing. Non-plastic fines or fines of low plasticity. Plastic fines.	
SM			Silty sands, and sand-silt mixtures, which may be poorly graded.	Atterberg limits below "A" line, or PI less than 4	Atterberg limits above "A" line with PI between 4 and 7 is borderline case SM-SC		
SC			Clayey sands, and sand-clay mixtures, which may be poorly graded.	Atterberg limits above "A" line, with PI greater than 7			

Unified Soil Classification System

Primary divisions		Group symbol	Secondary divisions	Laboratory classification criteria		Supplementary criteria for visual identification		
						Dry strength	Reaction to shaking	Toughness near plastic limit
Fine grained soils. (More than half of material is smaller than No. 200 sieve size.)	Silts and clays. (Liquid limit less than 50.)	ML	Inorganic silts, clayey silts, rock flour, silty very fine sands.	Atterberg limits below "A" line, or PI less than 4	Atterberg limits above "A" line with PI between 4 and 7 is borderline case ML-CL	None to slight	Quick to slow	None
	... do	CL	Inorganic clays of low to medium plasticity; silty, sandy or gravelly clays.	Atterberg limits above "A" line, with PI greater than 7		Medium to high	None to very slow	Medium
	... do	OL	Organic silts and organic silt-clays of low plasticity.	Atterberg limits below "A" line		Slight to medium	Slow	Slight
Do	Silts and clays. (Liquid limit greater than 50.)	MH	Inorganic silts, clayey silts, elastic silts, micaceous or diatomaceous silty or fine sandy soils.	Atterberg limits below "A" line		Slight to medium	Slow to none	Slight to medium
	... do	CH	Inorganic clays of high plasticity, fat clays.	Atterberg limits above "A" line		High to very high	None	High
	... do	OH	Organic clays and silty clays of medium to high plasticity.	Atterberg limits below "A" line		Medium to high	None to very slow	Slight to medium
Highly organic soils		Pt	Peat, meadow mat, highly organic soils.	High ignition loss, LL and PI decrease after drying		Organic color and odor, spongy feel, frequently fibrous texture.		

¹Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SM.

²See Ch. 3, Figure 3-1, for position on plasticity chart.