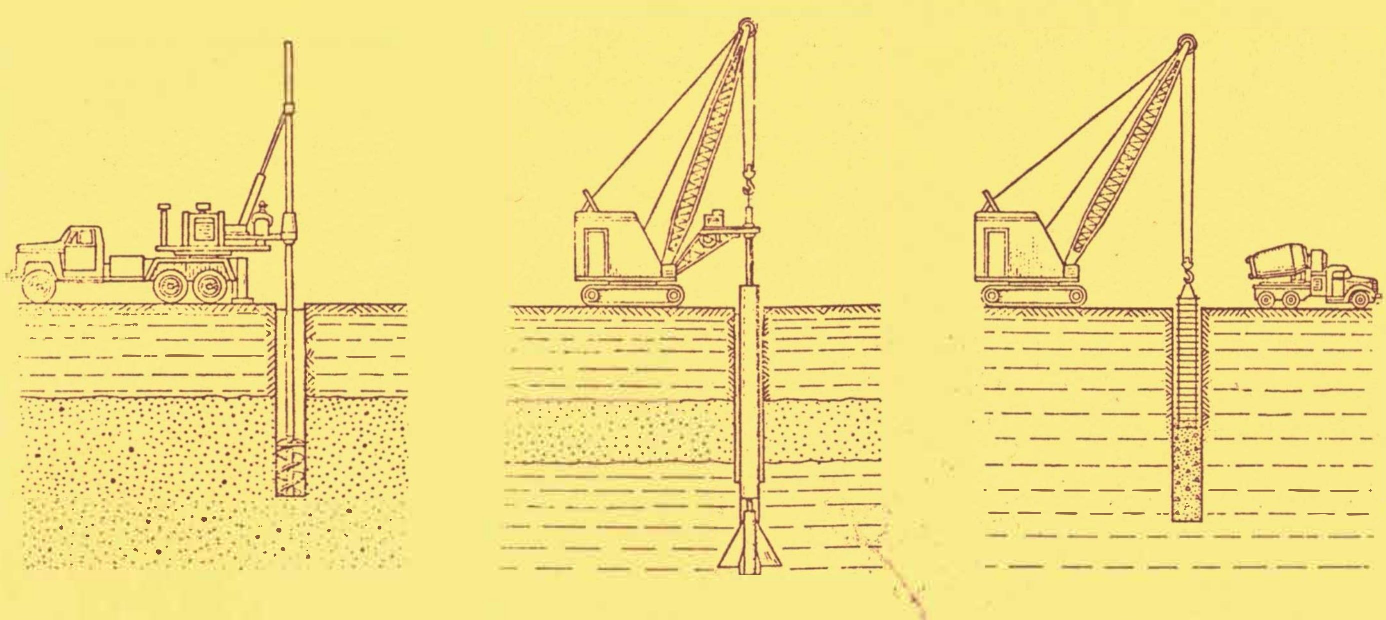
## Implementation of Research Findings:

# 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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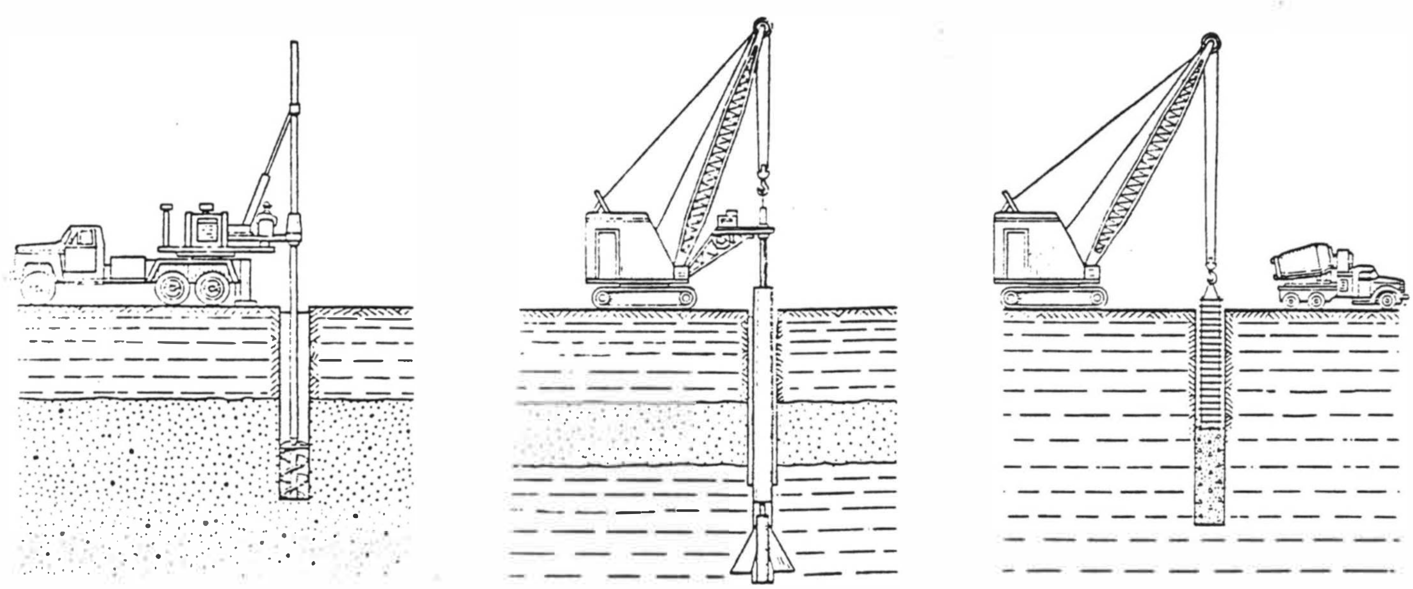

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The purpose of this report is to establish soil properties of Oklahoma flood plain (alluvial) soils for drilled shaft design. Also, to establish a correlation between shear strength (Su) obtained from unconfined compression test and N values obtained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

This study evaluates a variety of alluvial 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.



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Alluvial soils, cone penetrometer, standard penetration test, unconfined compression test.


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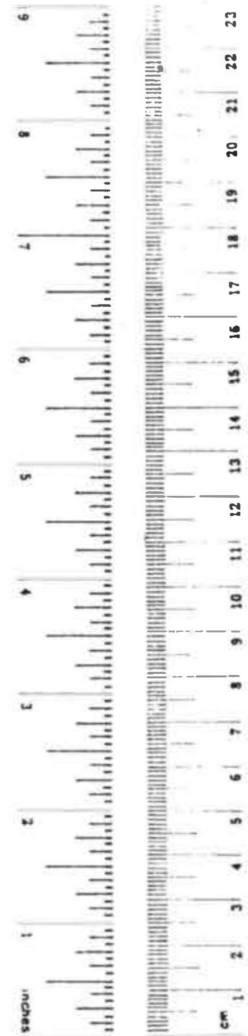
# METRIC CONVERSION FACTORS

## Approximate Conversions to Metric Measures

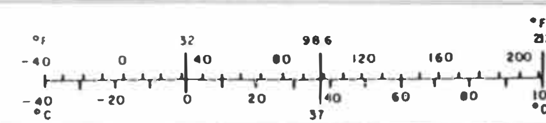
| Symbol | When You Know | Multiply by | To Find | Symbol |
|---|---|---|---|---|
| | **LENGTH** | | | |
| in | inches | *2.5 | centimeters | cm |
| ft | feet | 30 | centimeters | cm |
| yd | yards | 0.9 | meters | m |
| mi | miles | 1.6 | kilometers | km |
| | **AREA** | | | |
| $in^2$ | square inches | 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** | | | |
| tsp | 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 | 0.95 | 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. C13.10:286.




## 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 | 35 | 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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Under the Supervision of

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Research and Development Engineer
Research & Development Division
Oklahoma Department of Transportation

Oklahoma City, Oklahoma

December 1981

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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 (Su) 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

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

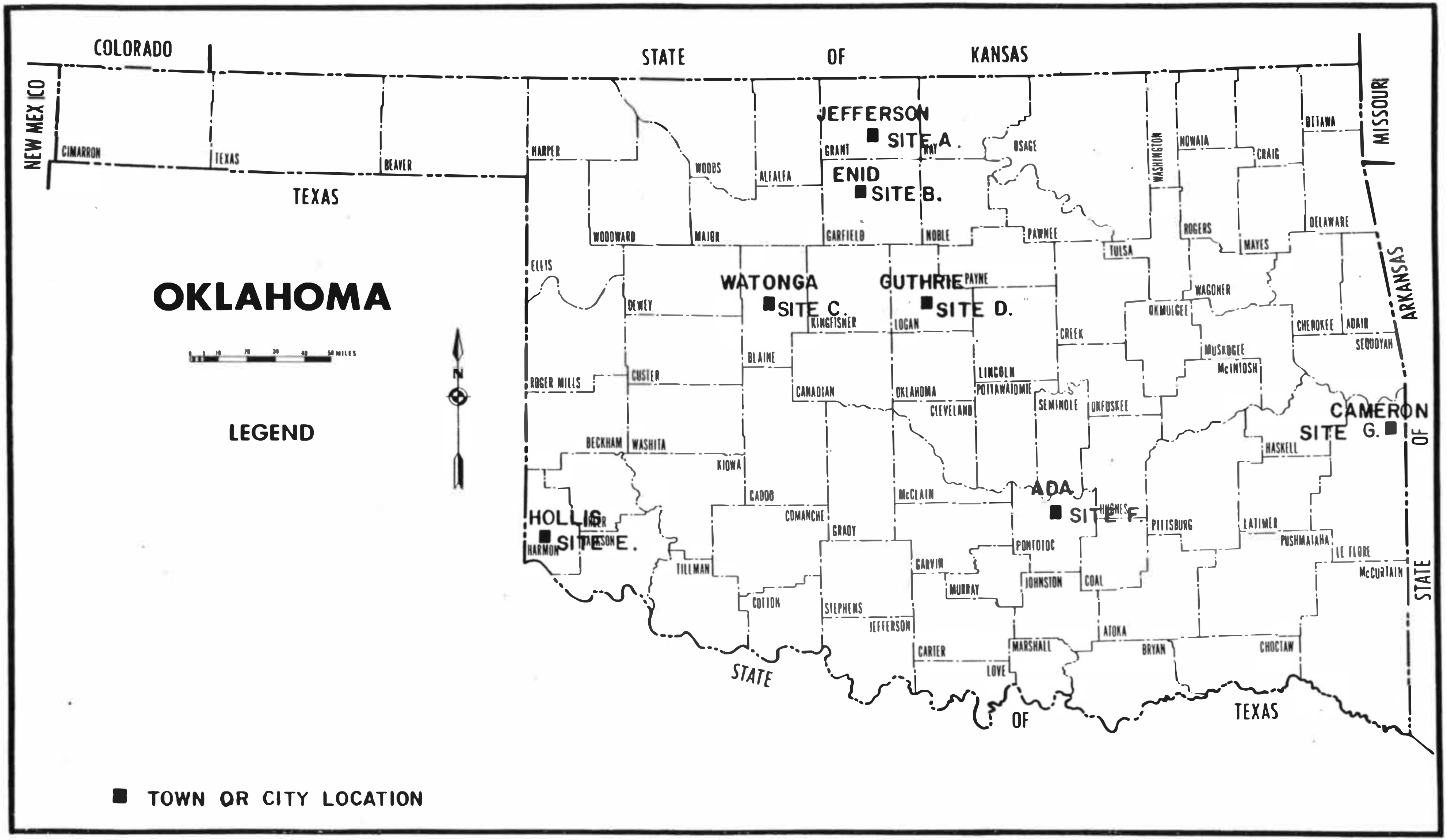


Figure 1. Location of Test and Sampling Sites

## 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.

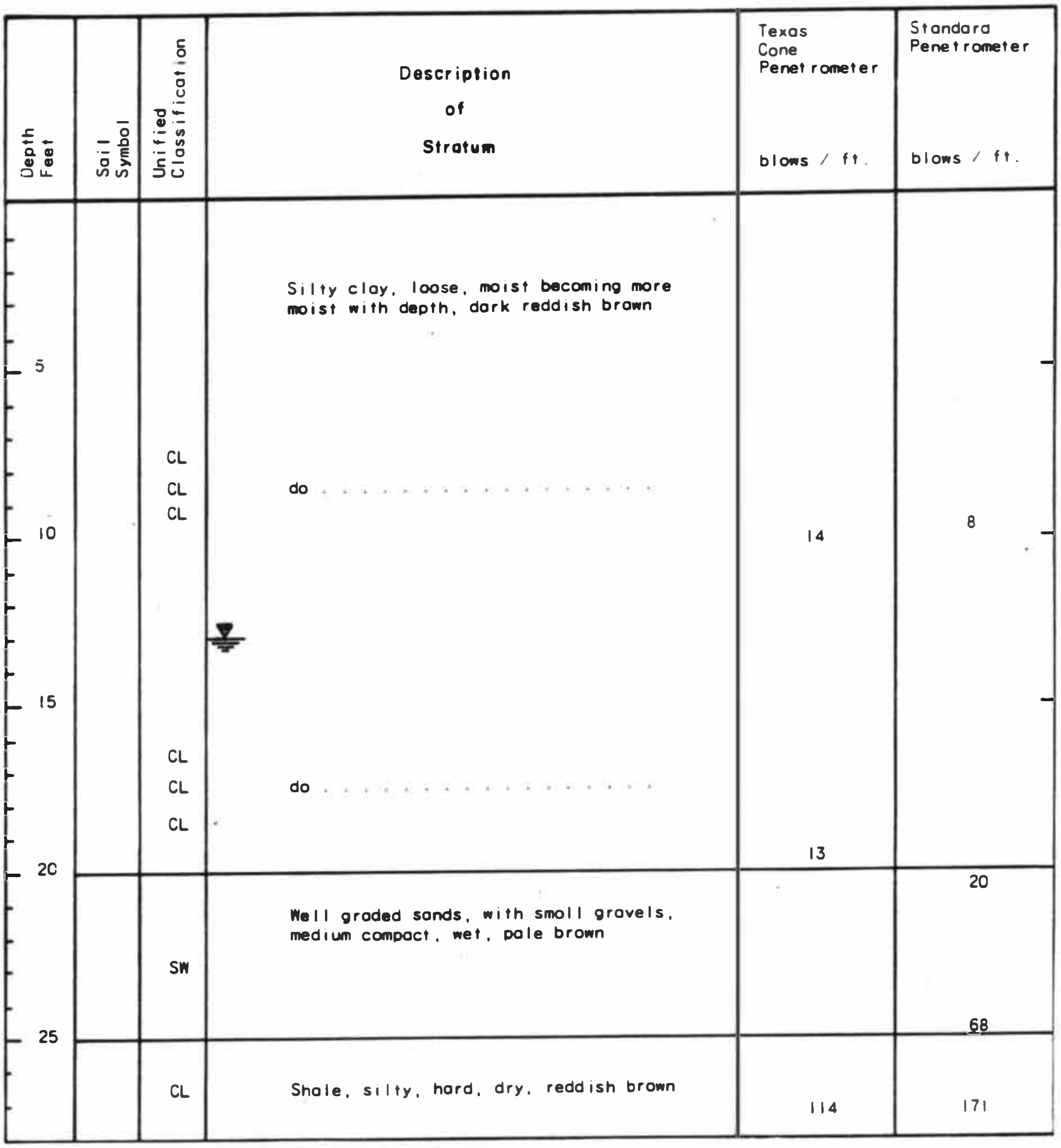


Figure 2. Boring Log for Site A

| 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 | | | | | Sunk 2.1 ft. under weight of tools |
| | | SM | do | | |
| | | | | 36 | |
| 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. |
|---|---|---|---|---|---|
| | | SM | Silty sands, loose, moist, reddish brown | | |
| 5 | | SW SM | do . . . . . . . . with somewhat less fines, becoming yellowish red with depth | 19 | 8 |
| 10 | | | | | |
| | | SW SM | do . . . . . . . . . . . . . . . . . . . . | 7 | 7 |
| 15 | | | | | |
| | | | | 400 | 171 |
| 20 | | | Silty shale, hard moist, reddish brown | 370 | — |
| | | ML | | | |
| 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 | — | — |
| | | CL<br>CL | Silty clay, stiff, moist, brown | | |
| 15 | | | | 17 | 10 |
| 20 | | CL<br>CL<br>CL | do . . . . . . . . . . . . . . . . | 13 | 7 |
| 25 | | | | | 8 |
| | | CL | | | |
| 30 | | | Silty sand, medium compact, wet, pale brown | | 16 |
| 35 | | SM<br>SM | | 71<br>71 | 18<br>18 |
| 40 | | | Sandstone, fine grained, poorly cemented, slightly moist | | |

Figure 5. Boring Log for Site D

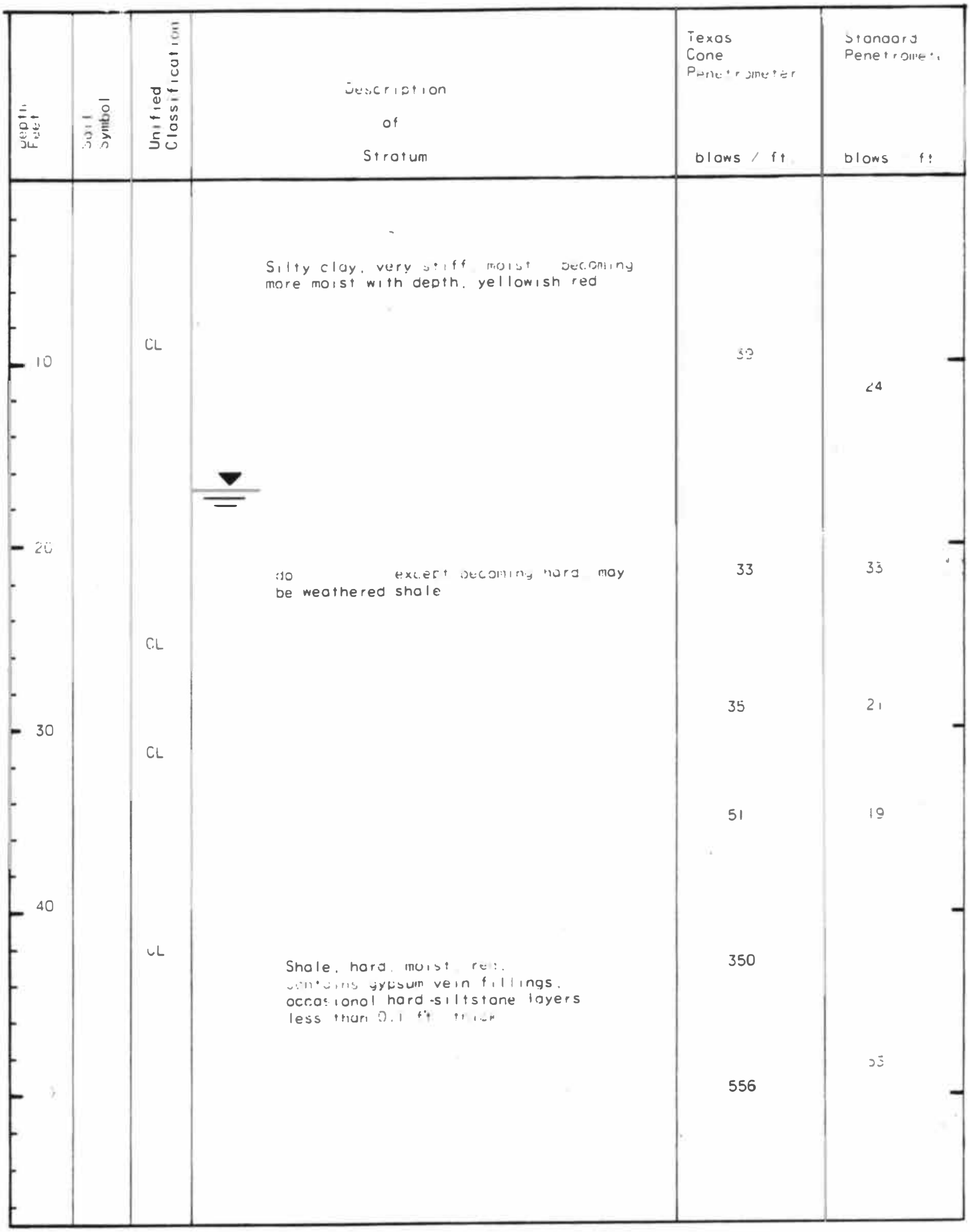


Figure 6. Boring Log for Site E

| DEPTH IN FEET | SOIL SYMBOL | UNIFIED CLASSIFICATION | DESCRIPTION OF STRATUM | TEXAS CONE PENETROMETER BLOWS / FT | STANDARD PENETROMETER BLOWS / FT. |
|---|---|---|---|---|---|
| 0–4 | | 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<br>— | —<br>46 |
| 15 | | CL | SILTY CLAY, HARD, MOIST, RED | 51 | — |
| | | | LIMESTONE. BROKEN, MOIST, RED | | |
| 20–30 | | CL | SILTY CLAY, SHALE, HARD, MOIST, RED | 50<br>100<br>125<br>150 | 86<br>—<br>120<br>150 |

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 | | | |
| | | | do . . . . becoming very stiff, moist progressively coarser with depth | 32 | 18 |
| 10 | | | | | |
| | | CL | | | |
| | | CL | | | |
| | | CL | | | |
| 15 | | | | 24 | 16 |
| | | CL | | | |
| | | SM | | | |
| 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, Su and resistance to penetration, N value for each soil type.

The standard penetration test 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.

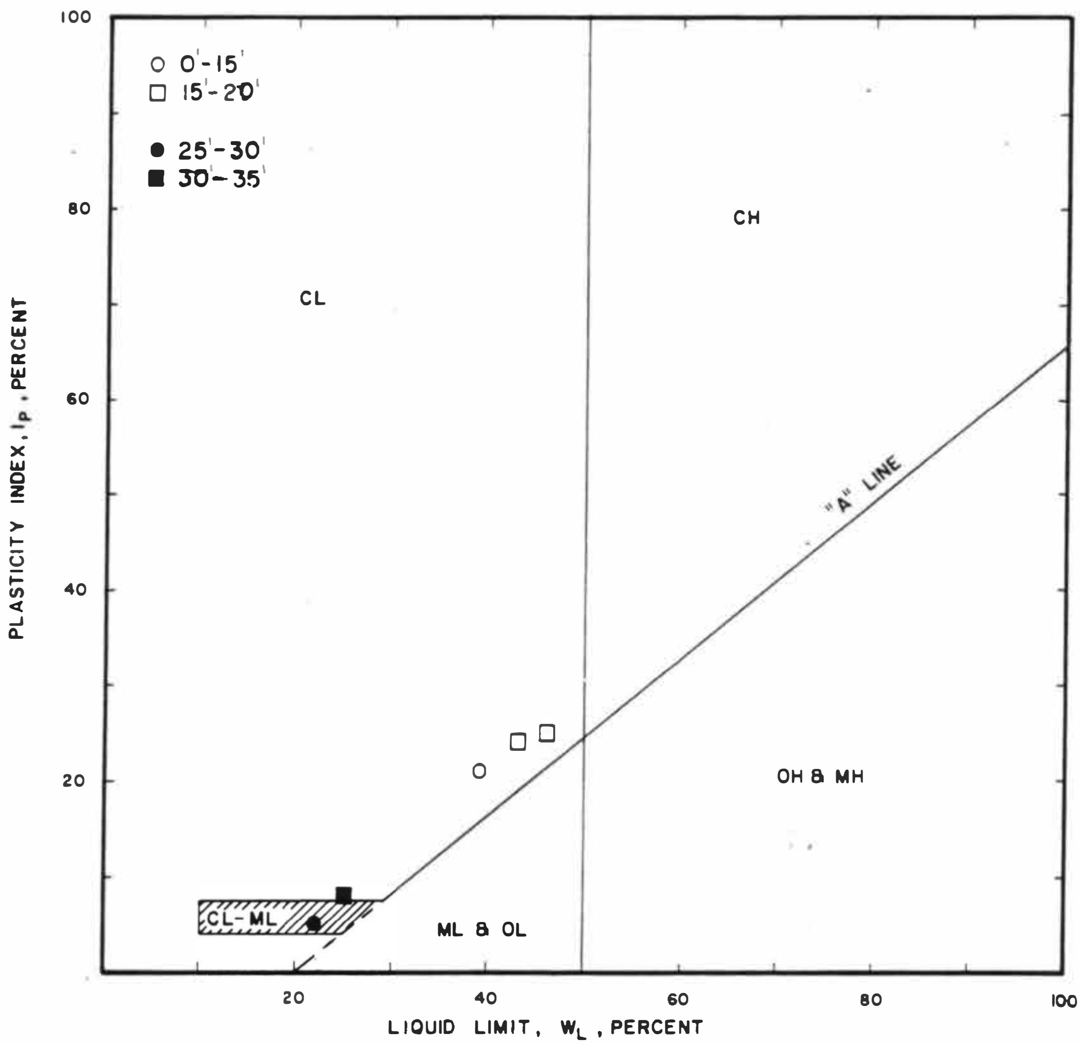


Figure 9. The Unified Classification of Site D Soil Layers

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

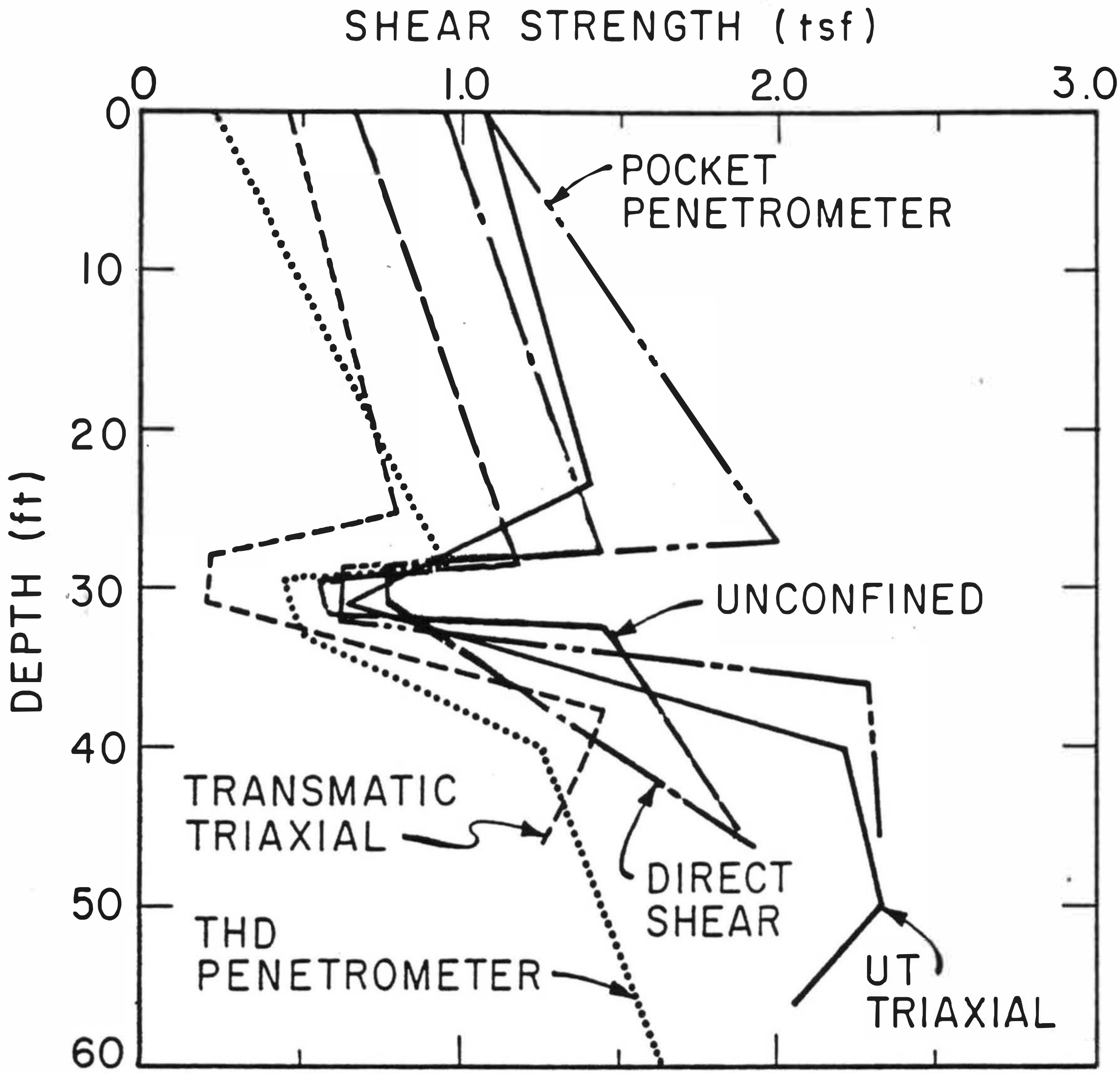


Figure 10. A Comparison of Shear Strength Profiles Obtained by Various Methods (from O'Neill and Reese)

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 (Su) are not always constant. It is necessary to discuss factors influencing N value, and Su, 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)

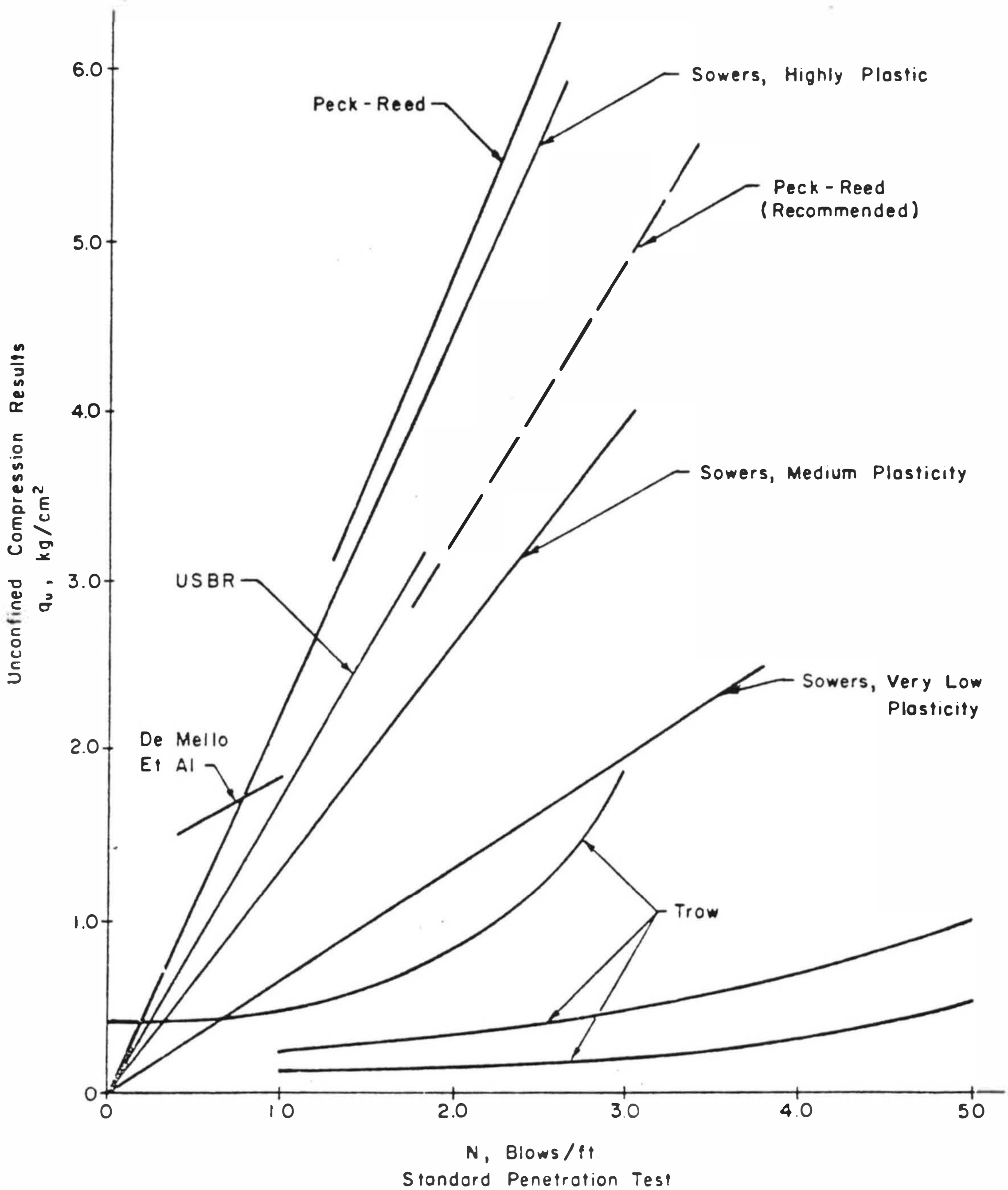


Figure 11. Correlation between N Value and Unconfined Compression Shear Strength for Clays attempted by Several Researchers (from DeMello)

## 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 stength 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 Su 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

$$Su = 0.060\ N_{THDP} \qquad (1)$$

where Su 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:

$$Su = 0.063\ N_{THDP} \qquad (2)$$

where in this case, Su, 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

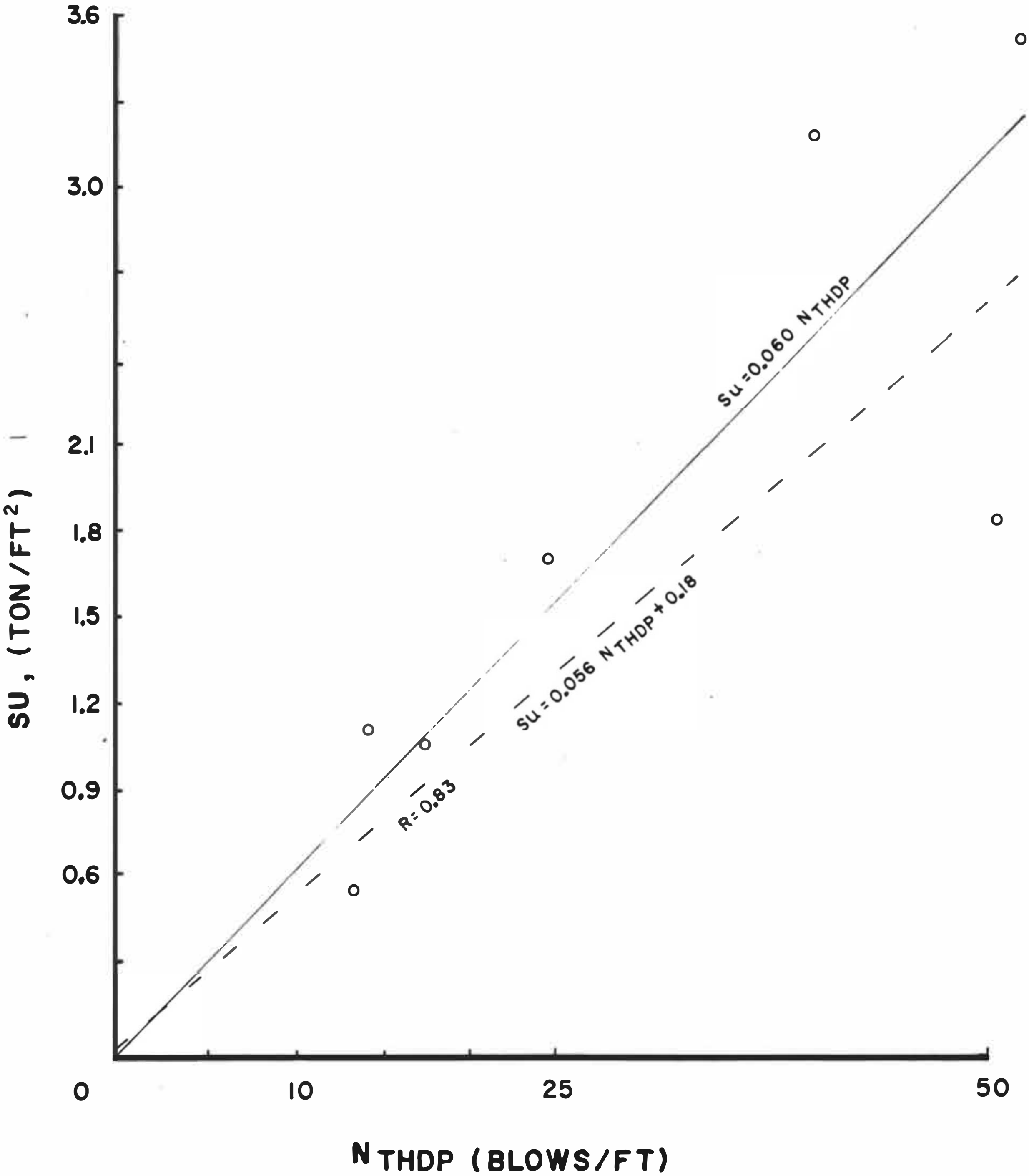


Figure 12. ODOT Correlation between $N_{THDP}$ and Su for Silty CL Soils

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

| | | |
|---|---|---|
| For homogeneous soil; | $Su = 0.07\ N_{THDP}$ | (3*) |
| For Sandy CL soil; | $Su = 0.05\ N_{THDP}$ | (4*) |
| For Cohesionless soil; | $Su = 0.02\ N_{THDP}$ | (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:

$$N_{STDP} = 0.63\ N_{THDP} \tag{6}$$

$$Su = 0.12\ N_{STDP} \tag{7}$$

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:

$$N_{STDP} = 0.70\ N_{THDP} \tag{8*}$$

$$N_{STDP} = 0.50\ N_{THDP} \tag{9*}$$

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:

For sandy CL soilsSu = 0.08 $N_{STDP}$ (10)

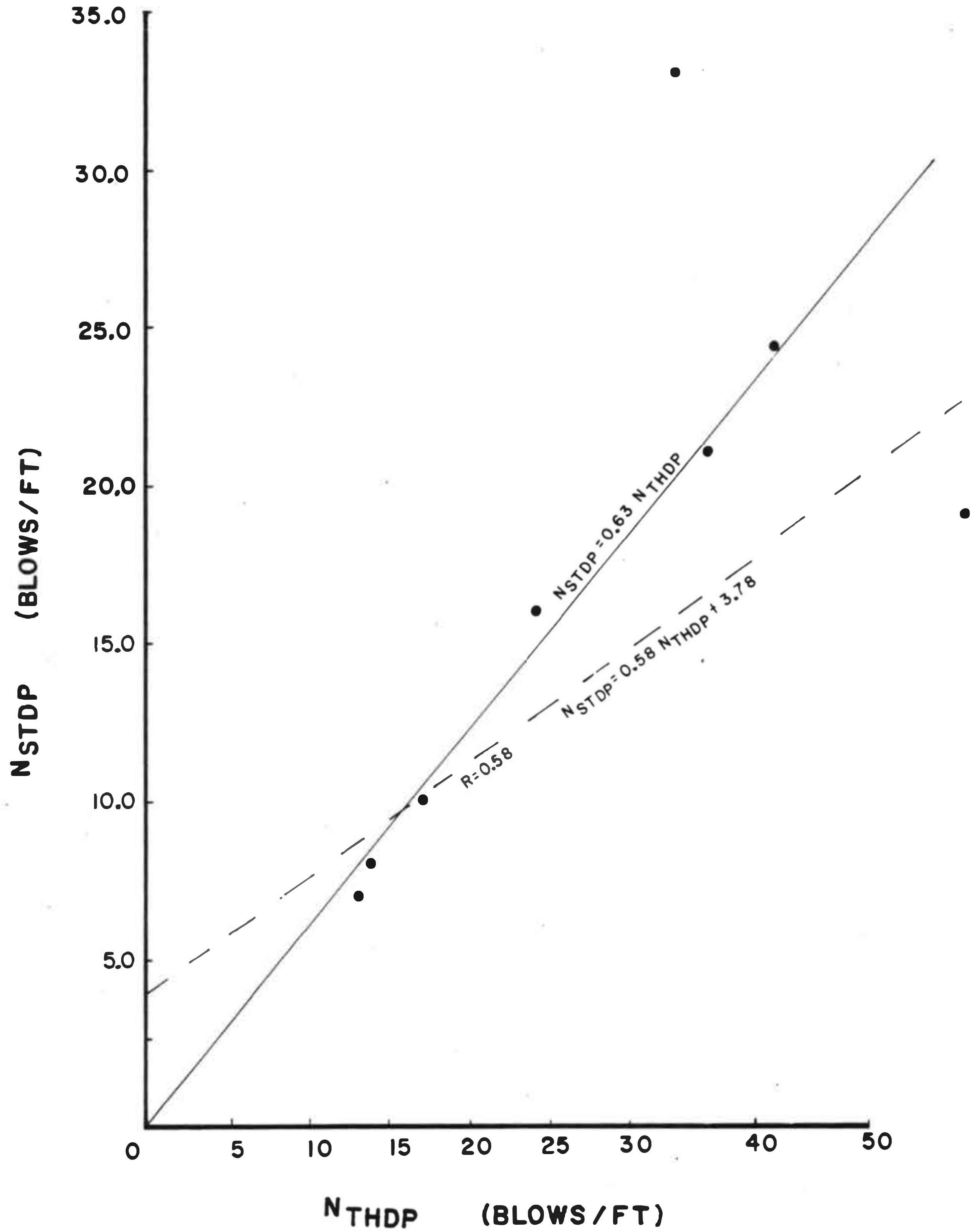


Figure 13. ODOT Correlation between $N_{THDP}$ and $N_{STDP}$ for Silty CL Soils

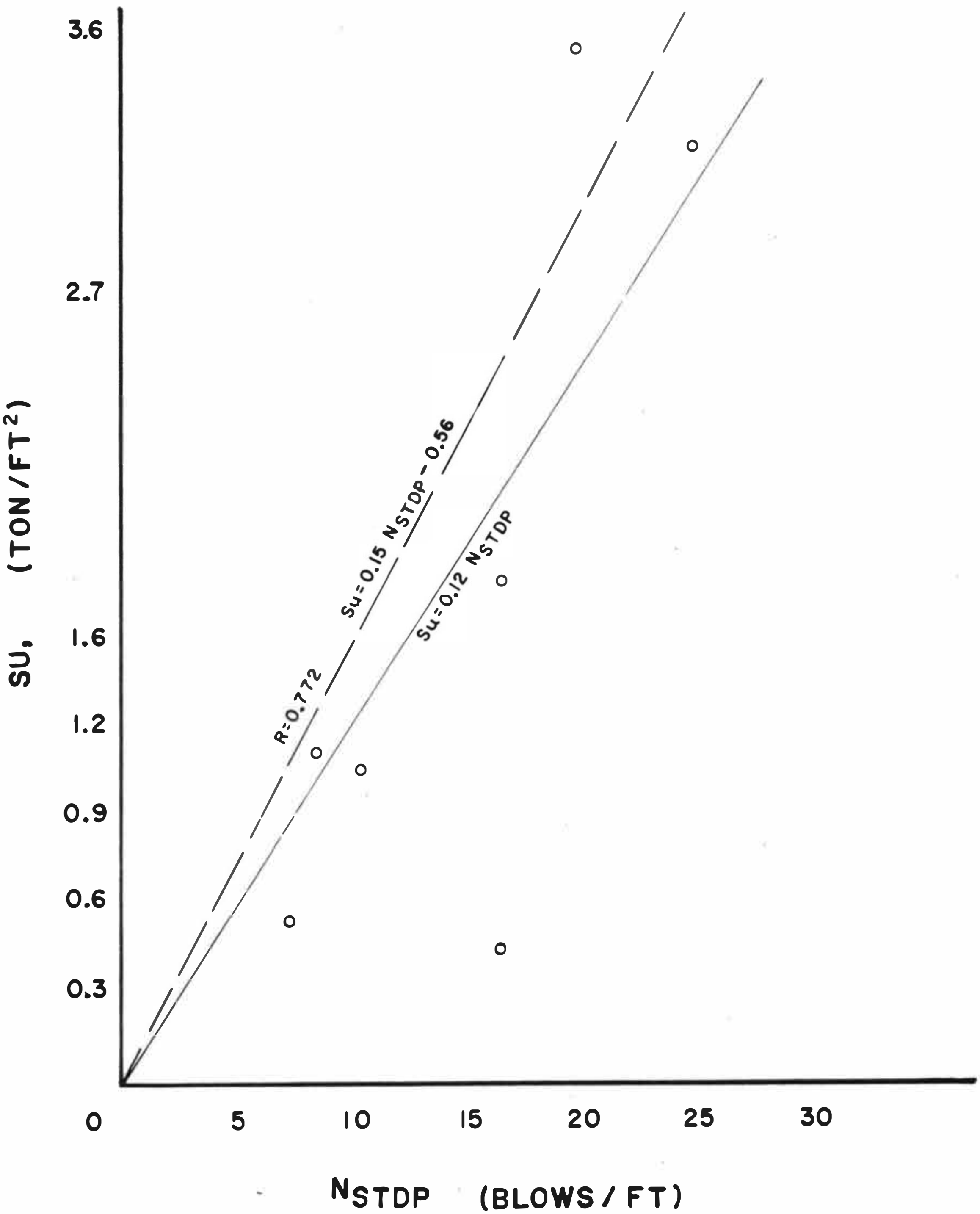


Figure 14. ODOT Correlation between $N_{STDP}$ and Su for Silty CL Soils

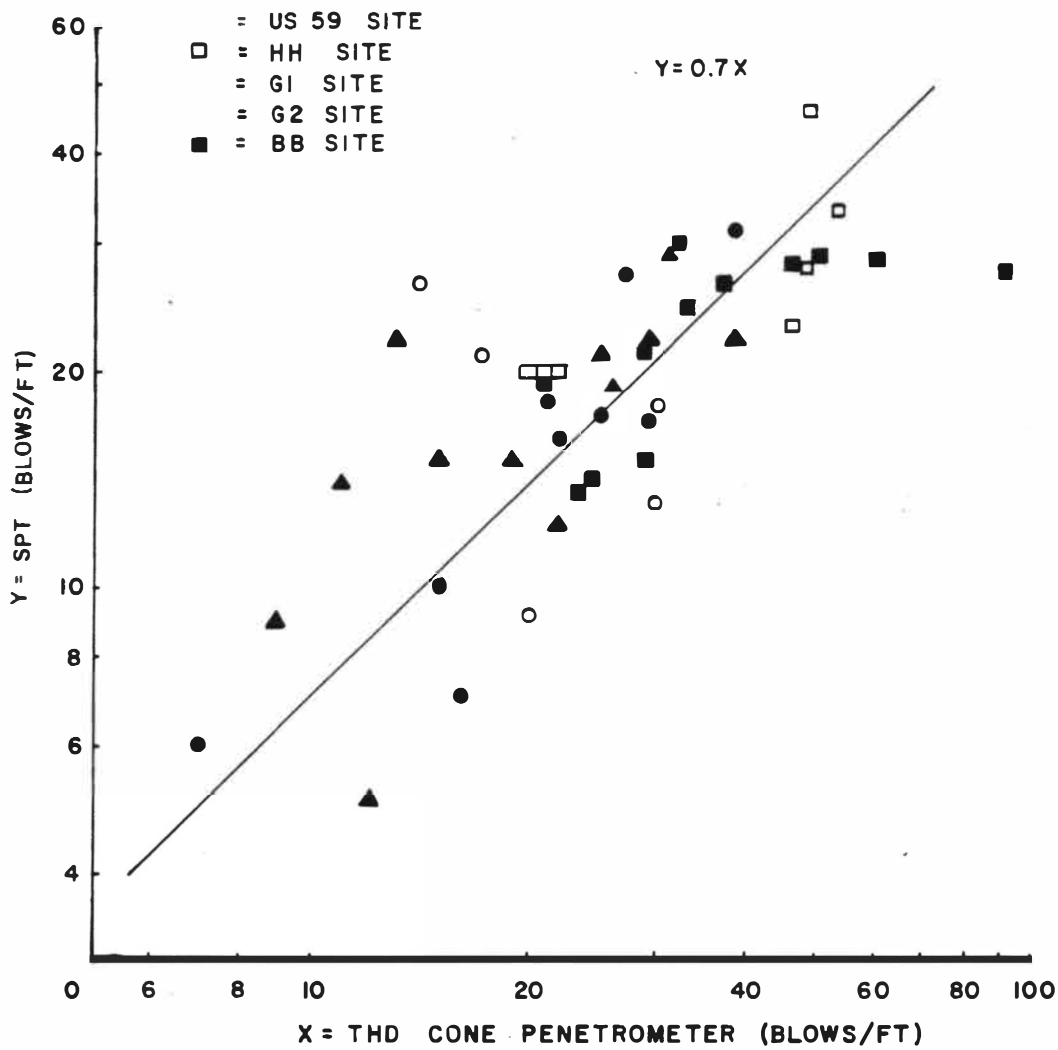


Figure 15. Correlation between the SPT and THD Cone Penetrometer in Clay as Obtained by Touma and Reese.

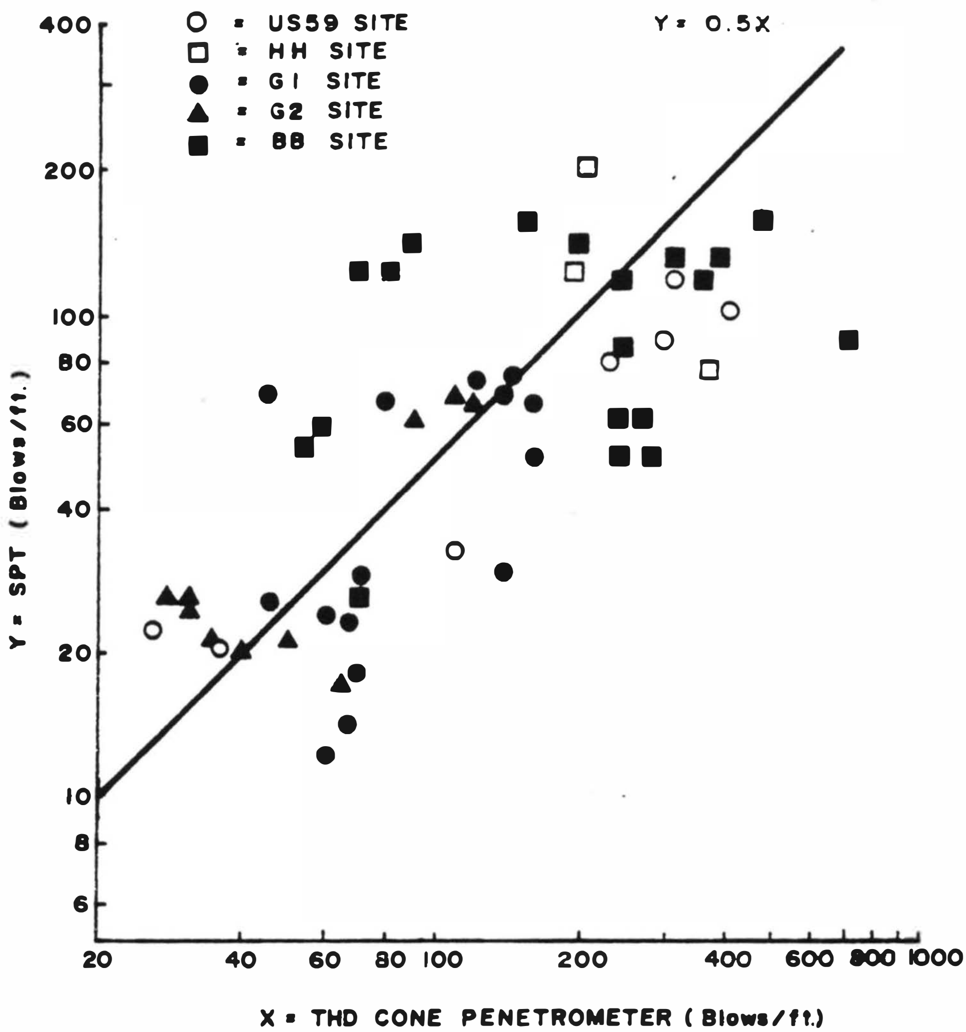


Figure 16. Correlation between the SPT and THD Cone Penetrometer in Sand as Obtained by Touma and Reese.

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

$$Su = 0.09\ N_{STDP} \qquad (11)$$

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 Su (ton/sq. ft.) and $N_{STDP}$ (blows/ft.) for homogeneous CL soil in Oklahoma, which is as follows:

$$Su = 0.10\ N_{STDP} \qquad (12)$$

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

$$Su = 0.04\ N_{STDP} \qquad (13)$$

The correlation coefficient (r) for the relationship between Su 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 Su (Figure 14) is 0.772. The r-value is between 95 percent to 99 percent valid.

# CONCLUSIONS AND RECOMMENDATIONS

## Conclusions

The correlations between Su and $N_{THDP}$ as well as Su 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:

   | | |
   |---|---|
   | $Su = 0.06\ N_{THDP}$ | for silty CL soils |
   | $Su = 0.07\ N_{THDP}$ | for homogeneous CH soils |
   | $Su = 0.05\ N_{THDP}$ | for sandy CL soils |
   | $Su = 0.02\ N_{THDP}$ | 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 Su and $N_{STDP}$ were developed as follows:

   | | |
   |---|---|
   | $Su = 0.09\ N_{STDP}$ | for silty CL soils |
   | $Su = 0.10\ N_{STDP}$ | for homogeneous CH soils |
   | $Su = 0.08\ N_{STDP}$ | for sandy CL soils |
   | $Su = 0.04\ N_{STDP}$ | for cohesionless soils |

4. The developed correlations between Su and $N_{THDP}$ as well as Su 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 | L.L. | P.L. | P.I. | % Passing Sieve Size No.10 | % Passing Sieve Size No.40 | % Passing Sieve Size No.200 | % Smaller than .05MM | % Smaller than .005MM | % Smaller than .002MM | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 | $N_{THDP}$ | $N_{STDP}$ |
|---|---|---|---|---|
| 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

| Boring No. | Depth feet | Moisture Percent | Wet Density PCF | L.L. | P.L. | P.I. | % Passing Sieve Size No.10 | No.40 | No.200 | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|
| 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

| Boring No. | Depth feet | Moisture Percent | Wet Density PCF | L.L. | P.L. | P.I. | % Passing Sieve Size No.10 | No.40 | No.200 | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|
| 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 | L.L. | P.L. | P.I. | %Passing Sieve Size No.10 | No.40 | No.200 | % Smaller Than .05MM | .005MM | .002MM | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 | L.L. | P.L. | P.I. | %Passing Sieve Size | | | % Smaller than | | | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 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

| Boring Number | Depth, Feet | Moisture Percent | Wet Density PCF | L.L. | P.L. | P.I. | % Passing Sieve Size No.10 | No.40 | No.200 | % Smaller than .05MM | .005MM | .002MM | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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

| Boring Number | Wet Density PCF | Unconfined Compressive Strength, TsF | Strain at Failure, Percent | $N_{THDP}$ | $N_{STDP}$ |
|---|---|---|---|---|---|
| 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 | L.L. | P.L. | P.I. | % Passing Sieve Size No.10 | No.40 | No.200 | % Smaller than 0.5MM | .005MM | .002MM | Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 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 | $N_{THDP}$ | $N_{STDP}$ |
|---|---|---|---|---|
| B1-A & BI-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<br>$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 | Wide range in grain size and substantial amounts of all intermediate particle sizes. |
| | | | 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. |
| | ... do ....... | Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.)[1] | GM | Silty gravels, and gravel-sand-silt mixtures, which may be poorly graded. | Atterberg limits below "A" line, or PI less than 4[2]<br>Atterberg limits above "A" line with PI between 4 and 7 is borderline case GM-GC | Non-plastic fines or fines of low plasticity. |
| | | | GC | Clayey gravels, and gravel-sand-clay mixtures, which may be poorly graded. | Atterberg limits above "A" line, with PI greater than 7 | Plastic fines. |
| Do ....... | Sands. (More than half of the coarse fraction is smaller than No. 4 sieve size.) | 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<br>$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 | Wide range in grain sizes and substantial amounts of all intermediate particle sizes. |
| | | | 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. |
| | ... do ....... | Sands with fines. (More than 12% of material smaller than No. 200 sieve size.)[1] | SM | Silty sands, and sand-silt mixtures, which may be poorly graded. | Atterberg limits below "A" line, or PI less than 4<br>Atterberg limits above "A" line with PI between 4 and 7 is borderline case SM-SC | Non-plastic fines or fines of low plasticity. |
| | | | SC | Clayey sands, and sand-clay mixtures, which may be poorly graded. | Atterberg limits above "A" line, with PI greater than 7 | Plastic fines. |

## 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. | | |

[1]Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SM.
[2]See Ch. 3, Figure 3-1, for position on plasticity chart.