Prediction of Shear Strength of Sand by Use of Dynamic Penetration Tests

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Texas cone penetrometer tests were conducted at six sites in the middle and upper Texas gulf coast region. The soils tested were cohesionless and included poorly graded sands and silty sands. The direct-shear test method was used to determine the effective angle of internal shearing resistance of the soils, and an empirical relationship was used to obtain standard penetration test values from the measured Texas cone penetrometer test data. The standard penetration test N-values of fine and silty saturated sands were corrected to account for the development of pore pressures during driving. Both the Texas cone and the standard penetration test N-values were correlated with the shear strengths and with the effective angles of internal shearing resistance of the sands. The new correlations were compared with existing correlations commonly used in the geotechnical profession, and it was found that the currently used relationships between the N-value and the effective angle of internal shearing resistance are a lower bound for these test data.

Soil sounding or probing consists of forcing a rod into the soil and observing the resistance to penetration. According to Hvorslev (1), "variation of this resistance indicates dissimilar 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 in this dynamic test is defined as the number of blows (N) that produces a penetration of 1 ft.

In the United States, the most commonly used dynamic penetration test is the standard penetration test (SPT). The results of the SPT can usually be correlated with the pertinent physical properties of a sand. Meigh and Nixon (2) 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 a fine sand. Gibbs and Holtz (3) have found that a definite relationship exists between the N-value as determined from the SPT and the relative density of a sand. A relationship between the N-value and the effective angle of shearing resistance, which has become widely used in foundation design procedures in sands, has been reported by Peck, Hanson, and Thornburn (4).

The Texas State Department of Highways and Public Transportation (TSDHPT) currently uses a penetration test similar to the SPT for investigation of foundation materials encountered in bridge-foundation exploration work. The penetration test is especially useful in investigations in cohesionless soils because of difficulties in obtaining undisturbed samples for laboratory testing. According to the Texas foundation manual (5), "the design of foundations in cohesionless soils is generally based upon visual classification and penetrometer test data." The Texas cone penetrometer (TCP) test consists of dropping a 756-N (170-lbf) hammer 0.61 m/blow (2 ft/blow) to drive a 7.6-cm (3.0-in) diameter cone that is attached to the end of the drill pipe. The details of the cone penetrometer are shown in Figure 1. The penetrometer is first lowered tc the bottom of the bore hole and driven 12 blows to seat it in the soil. Then, the penetrometer test is started and the number of blows (the N-value) required to produce the next 1 ft of penetration is recorded.

The objective of the study reported in this paper was to develop an improved correlation between the N-value (in blows per foot) obtained by using the TCP test or the SPT and the drained shear strength of a cohesionless soil. Correlations were developed for two types (as defined by the Unified Soil Classification system) of soils:

1. SP—poorly graded sands, gravelly sands, and little or no fines and
2. SM—silty sands and poorly graded sand-silt mixtures.

SAMPLING PROGRAM

Correlating shear strengths with penetration test N-values requires that undisturbed sand samples be collected and penetration tests be carried out at corresponding depths at the same test site. This requires a sampling procedure in which a relatively large number of samples can be recovered and tested with minimal disturbance.

Previously used methods for collecting undisturbed samples of cohesionless soils were investigated first. Methods such as solidification of the lower end of the sample by chemical injection or freezing (6) and solidification of the sand before sampling by asphalt injection or by freezing the soil by the use of a cooling mixture in auxiliary pipes (1) do not always produce undisturbed samples and are very elaborate and expensive. Also, according to Bishop (7), mechanical core retainers, such as that used in the Denison sampler, cause excessive disturbance in clean sands.

With the aid of personnel from TSDHPT, a sampling apparatus similar to a small-diameter Shelby tube sampler was developed. This sampling device (see Figure 2) consists of a thin-walled sampler that has a coupling head to adapt the sampler to the drilling rod. A check valve in the coupling head allows the drilling fluid to escape while the sample tube is lowered to the bottom of the borehole and prevents the water pressure in the drilling rod from forcing the sample out of the sampler during extraction. Two vent holes above the check valve allow the drilling fluid to drain from the drilling rod while the sample tube is being extracted from the bore hole.

The sample tubes were made of either stainless or galvanized steel and had an outside diameter of 44.09 mm (1.736 in) and a wall thickness of 1.91 mm (0.075 in). For minimum disturbance (1), it is preferable that area ratio of the sampler not exceed 10-15 percent as computed by using Equation 1:

$$\text{Area ratio} = \text{volume of displaced soil/volume of soil} = (D_w^2 - D_e^2)/D_e^2 \qquad (1)$$

where D_w = outside diameter of sample tube and D_e = inside diameter of sample tube. The area ratio of the chosen sampler was 20 percent, very close to this limit. In a preliminary field study, the 254-mm (10-in) and 305-mm (12-in) diameter samplers were found to permit the best recovery.

The borings were made by using a truck-mounted Failing 1500 rotary-core drilling rig. As the hole ad-

Figure 1. Details of Texas cone penetrometer.

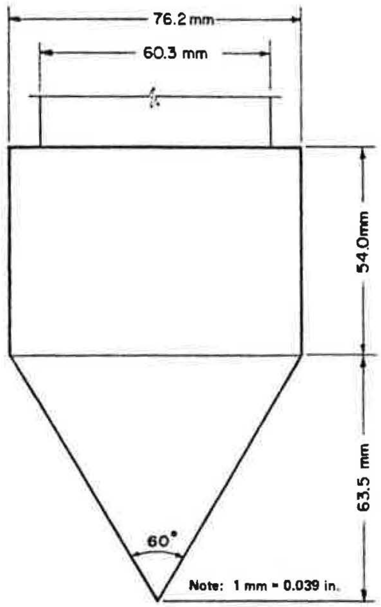
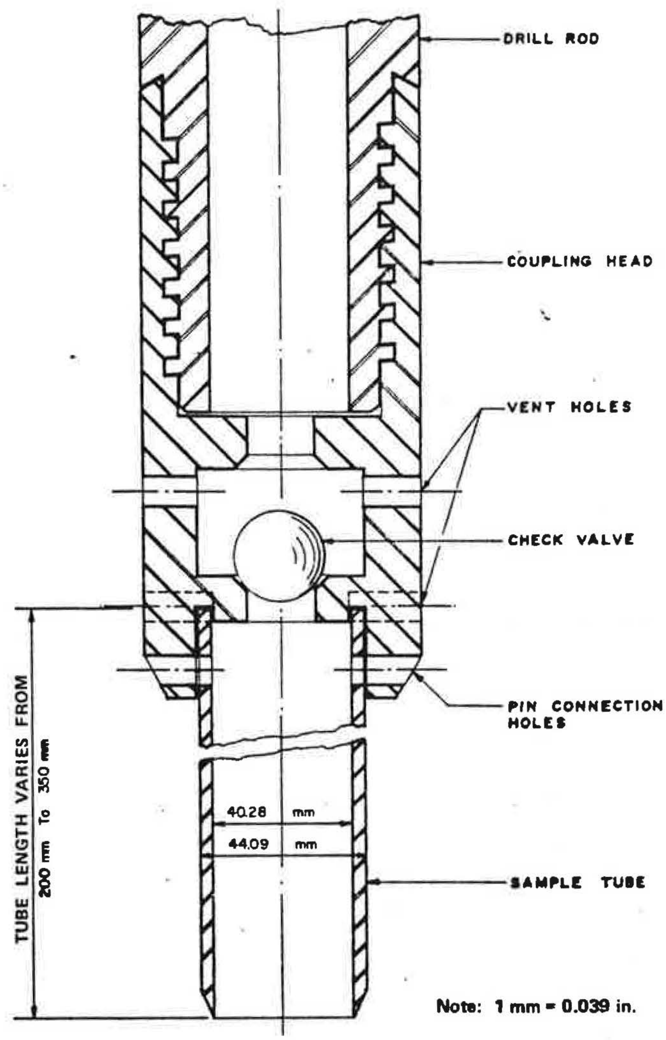


Figure 2. Cross section of sampling apparatus.



vanced 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 from which the undisturbed samples were to be taken was encountered, 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 by 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 the depth of the sample and its visual classification. The sample tube was then sealed on each end, covered with paraffin, and packaged for transportation to the soils laboratory.

TEST PROGRAM AND SITES

The test program was conducted by a TSDHPT soil investigation team in cooperation with Texas Transportation Institute personnel. Standard practices of field investigation as described in the foundation manual (5) were followed. The purposes of the field investigation were to

1. Establish the location of the groundwater table,
2. Obtain a soil description by visual inspection of samples,
3. Obtain TCP N-values, and
4. Obtain undisturbed samples for laboratory testing.

After the undisturbed sand samples were obtained, the TCP test was performed at corresponding depths at each test site. The penetrometer tests were conducted in new boreholes located not more than 3.05 m (10 ft) from the holes from which the soil samples had been obtained.

Because the boreholes were advanced by using a 76-mm Shelby tube sampler, samples of the cohesive soils could be kept for determination of their unit weight whenever there was an indication of change in soil properties. The unit weights and moisture contents were determined from the Shelby tube samples in the conventional manner. The Unified Soil Classifications, moisture contents, and total unit weights of the cohesionless soils were determined from the small-diameter samples. To determine the Unified Soil Classification, mechanical sieve analysis and Atterberg limits were conducted.

Five test sites were investigated and eight borings were made during the period of September 1974-August 1975. [Complete laboratory and field data for these sites are reported elsewhere (8).] One additional test site was investigated and one boring was made during the period of September 1975-August 1976. [Laboratory and field data for this site are reported elsewhere (9).] The test sites investigated in 1974-1975 were located in Brazos County near College Station (sites A, B, and C) and in Harris County near Houston (sites D and E). The test site investigated in 1975-1976 was located in Nueces County near Corpus Christi (site F).

Typical boring logs for two of the test sites are shown in Figures 3 and 4. Figure 3 shows the log of boring no. 3 at test site A, where the soil was essentially all sand. (Because the penetration resistance is defined in terms of U. S. customary units, SI units are not given for this quantity and the depth below ground at which it is measured in Figures 3, 4, 7, and 8 and in Table 1.) Figure 4 shows the log of boring no. 1 at test site D, where alternating layers of clay and sand occurred. Overall, penetration tests were conducted at depths of 1.8-21.4 m (6-70 ft), and N-values ranged from 20 to 330 blows/m (6 to 100 blows/ft).

Figure 3. Log of boring 3: site A—TX-30.

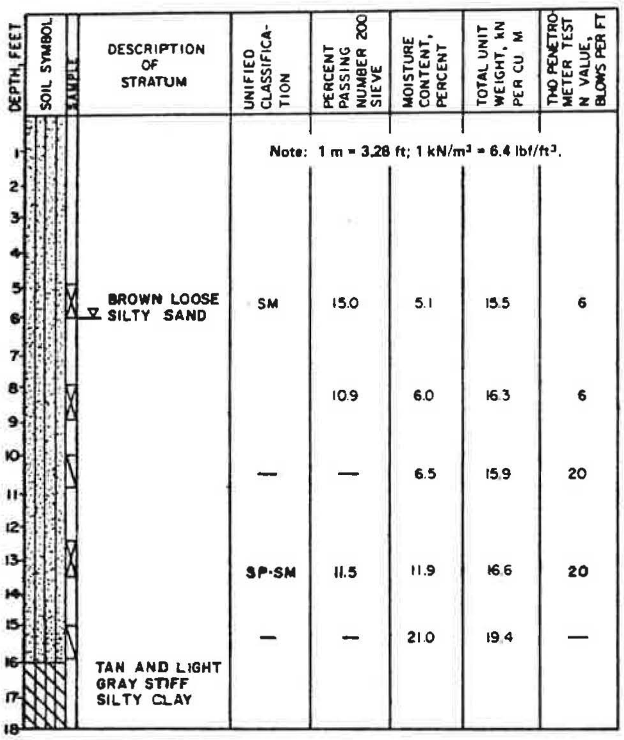
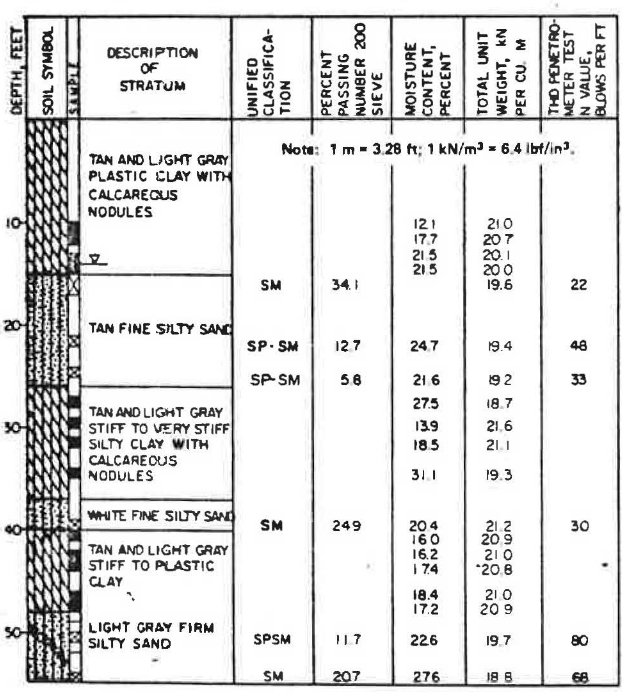


Figure 4. Log of boring 1: site D—Woodridge Road.



LABORATORY INVESTIGATION

The purpose of the laboratory investigation was to determine the drained shear strength of the cohesionless samples. The direct shear test was used to determine the effective angle of shearing resistance which, in turn, was used to calculate the drained shear strength. First, cuttings were removed from both ends of the sample, and the total unit weight of the sample remaining in the tube was determined. Then the sample tube was placed in the extrusion device shown in Figure 5, and the direct shear box was inverted and placed over the tube. The sample was extruded into the box until the end plates made contact with the restraining pins in the base of the shear box. The samples were trimmed by using a 0.02-mm (0.001-in) thick trimming device. The box was then removed from the extrusion device and placed upright in the direct shear loading apparatus for testing.

The test setup used for the drained direct shear tests to determine the angle of shearing resistance is shown in Figure 6. A normal stress was applied on plane a-a through the loading frame by a constant-speed motor that turned the lower half of the shear box while the upper half was held in place by a horizontal arm and thus caused a relative motion between the two halves. The force required to hold the arm was determined by readings on a proving ring. The shearing force was increased until the sample failed along plane a-a. Three tests were performed at normal stresses of 69, 138, and 207 kPa (10, 20, and 30 lbf/in^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. A failure envelope was then plotted by using the shear stresses at failure and the corresponding normal stresses. The angle of shearing resistance (ϕ') is the angle between the failure envelope and the horizontal.

In this test, it is necessary to use a strain rate that allows drainage during testing. As noted by Means and Parcher (10), a number of investigators have shown that the strength of a soil tested in the laboratory depends "to a remarkable extent upon the rate and duration of loading employed in the test." In his text (11), Lambe states that "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 percent passing the 75-µm (no. 200) sieve] from test site E was used to investigate the effect of the rate of loading, and it was found that varying the strain rate from 0.002 to 0.13 mm/min (0.0001 to 0.005 in/min) resulted in a difference in the angle of shearing resistance of only 1°. Thus, a strain rate of 0.13 mm/min was considered suitable to allow drainage and thereby prevent pore pressure from building up.

Unit weights of both small-diameter and standard Shelby tube samples were determined. In general, samples taken at approximately equal depths had unit weights that were in very close agreement, independent of the method of sampling. At test sites where several consecutive small-diameter samples were taken, consistency in the unit weights was observed. This consistency was especially noticeable at test site E where an obviously dense material (N > 100) was encountered; for this test site, the three samples tested had unit weights (determined from the small-diameter samplers) of 21.49-21.62 kN/m^3 (136.8-137.6 lbf/ft^3). These two factors—the independence of the unit weights from the method of sampling and the consistency of the unit weights at each test site—indicate that the unit weights determined from the small-diameter samplers are of acceptable accuracy.

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

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

Figure 5. Cross section of extrusion assembly.

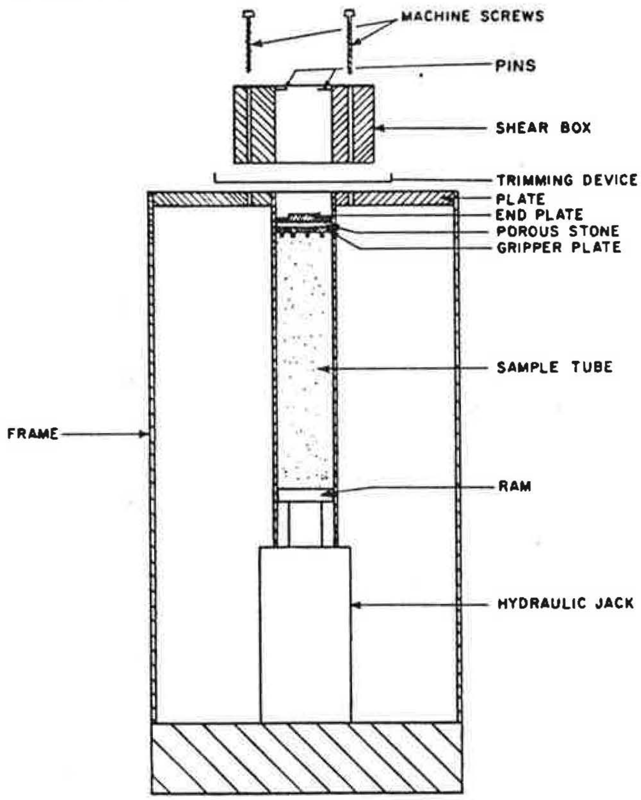
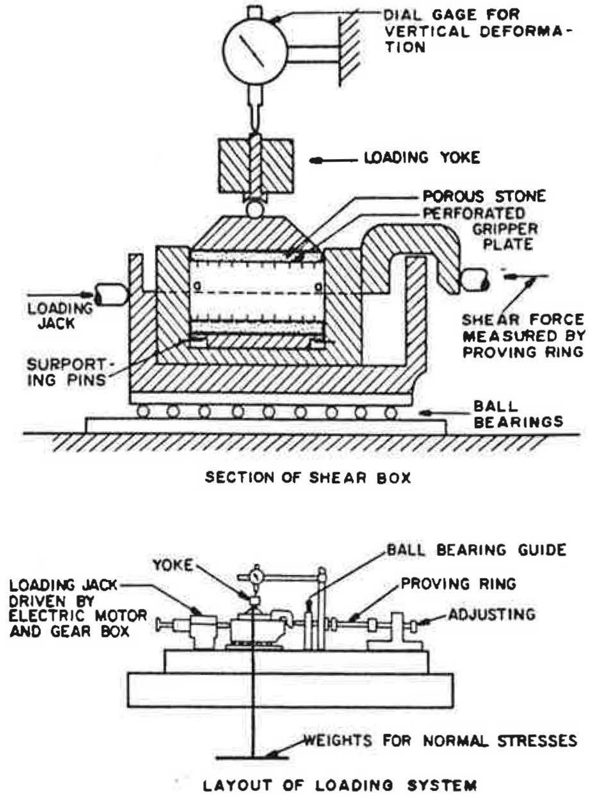


Figure 6. Direct shear equipment.



where

s = effective shear strength of soil,
c' = effective cohesion,
σ_η' = effective normal stress, and
ϕ' = effective angle of shearing resistance.

For drained tests conducted on cohesionless soils, $c' = 0$, and therefore

$$s = \sigma_\eta' \tan\phi' \tag{3}$$

The normal stress at a point above the groundwater level is equal to the overburden pressure (p), which is calculated by using the relationship:

$$\sigma_\eta = p = \gamma h \tag{4}$$

where

σ_η = normal stress,
γ = unit weight of soil, and
h = depth below ground surface.

The stress below the groundwater level, however, must be calculated by using the effective overburden pressure (p'). If it is assumed the pore-water pressure is hydrostatic, this can be expressed as

$$p' = (\gamma - \gamma_\omega)h \tag{5}$$

where γ_ω = unit weight of water and h = depth below the groundwater level. The shear strength is then calculated by combining the overburden pressure (based on averaged unit weights for the soil strata) contributed by each soil stratum above and below the groundwater level with the effective angle of shearing resistance as in Equation 3.

For various depths at test sites A and D, typical average unit weights, angles of shearing resistance, shear strengths calculated by using these data and information about the position of the groundwater level, and corresponding N-values are summarized in Figures 7 and 8, respectively.

DEVELOPMENT OF CORRELATIONS

The relationship between TCP test N-value (N_{TCP}) and ϕ' for sand used by the TSDHPT is represented by the solid curve shown in Figure 9 (5). As can be seen from this figure, this relationship forms a lower bound for the data obtained in this study, although the scatter in the data does not warrant the establishment of a new curve. However, the current relationship is apparently conservative.

Based on the data shown in Figure 10, Touma and Reese (12) have proposed the following general relationship between N_{TCP} and the SPT N-value (N_{STP}):

$$N_{SPT} = 0.5N_{TCP} \tag{6}$$

where N_{SPT} and N_{TCP} are both expressed in blows per foot.

Bowles (13) recommends the use of Equation 7 for very fine or silty saturated sand (below the water table) if the measured penetration number (N) is greater than 15:

$$N'_{SPT} = 15 + (1/2)(N_{SPT} - 15) \tag{7}$$

where N'_{SPT} = adjusted penetration number and N_{SPT} = measured penetration number. Equation 7 is based on the assumption that N_{SPT} is approximately 15 when the in

Figure 7. Summary of shear strength data: boring 3—site A.

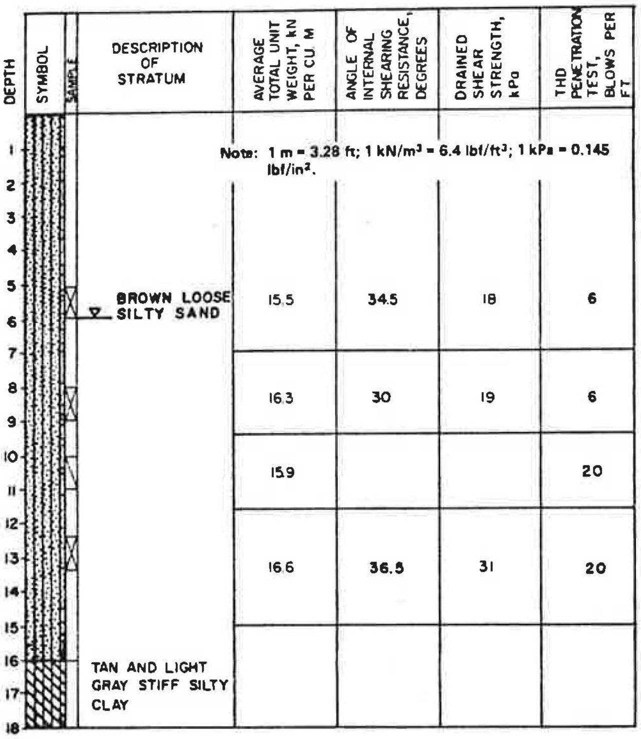
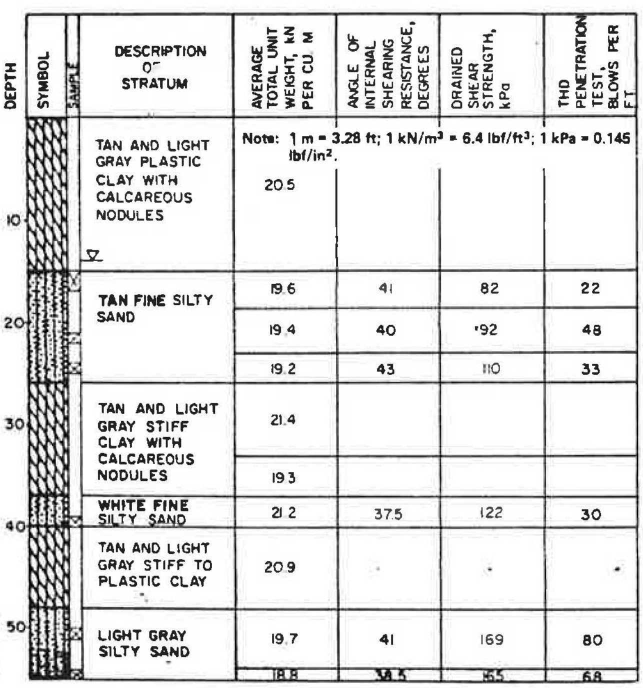


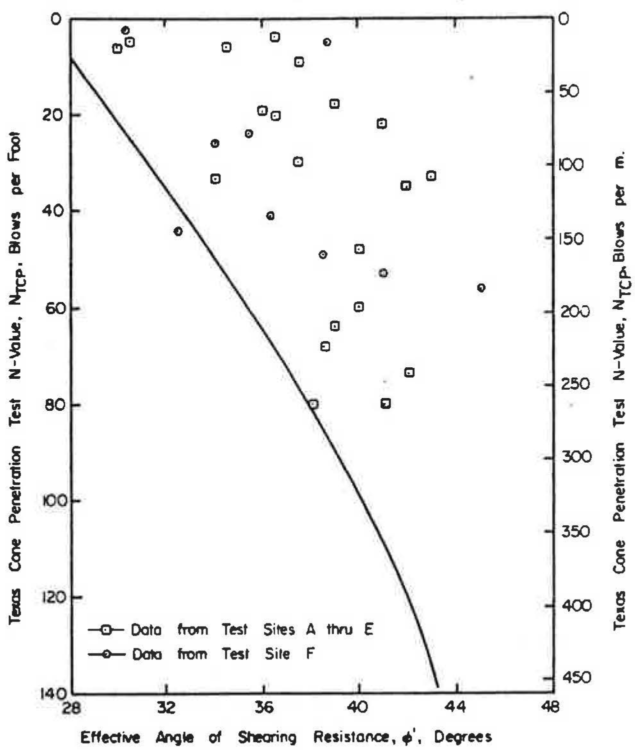
Figure 8. Summary of shear strength data: boring 1—site D.



situ void ratio equals the critical void ratio of the soil. Also, in fine-grained materials, the coefficient of permeability is so low that the change in pore pressure created by the expansion of the soil impedes penetration by the split spoon and thus increases the penetration number.

In this study Equation 6 was used to evaluate the N_{SPT} values for each N_{TCP} value obtained from all study test sites and, where appropriate, Equation 7 was used to determine the adjusted N-value (N'_{SPT}). The N-values

Figure 9. Relationship between TCP test N-value and effective angle of shearing resistance for SP, SM, and SP-SM soils.



and the other significant study data are given in Table 1. The relationship between N_{SPT} and ϕ' (which is widely used for foundation design in sands) given by Peck, Hanson, and Thornburn (4) is shown by the solid curve in Figure 11. A plot of $\overline{N}'_{SPT}$ values versus the ϕ' values obtained in this study is shown in Figure 12; it would appear that the dashed curve is a more accurate lower bound for the relationship. However, the dashed curve can only be used with the adjusted N-value (N'_{SPT}).

It has been shown that the shear strength of a cohesionless soil depends on the angle of shearing resistance and the normal pressure acting on the failure plane. Means and Parcher (10) have reported that the factors affecting the angle of shearing resistance are degree of density, void ratio or porosity, particle size and shape, gradation, and moisture content. Because the resistance to penetration is also reported to be affected by these same factors and especially by the normal pressure, a relationship should exist between penetration resistance and shear strength.

The effect of shear strength on penetration resistance has been verified by several workers. According to DeMello (14), "The shear resistance is the principal parameter at play in resisting penetration." Desai (15) concluded that shear strength was one of the main factors affecting penetration resistance. Jonson and Kavanagh (16) have summarized their findings by stating that the resistance to penetration is a function of the shearing resistance of the soil.

A plot of the drained shear strength (s) versus the corresponding N_{TCP} value is shown in Figure 13. Least-squares statistical analysis was used to develop a constant of proportionality for the two soil parameters. The relationship can be expressed as follows:

$$s = 2.0N_{TCP} \quad (8)$$

The coefficient of correlation for this relationship is

Figure 10. Correlation between standard penetration and TCP test N-values in sands.

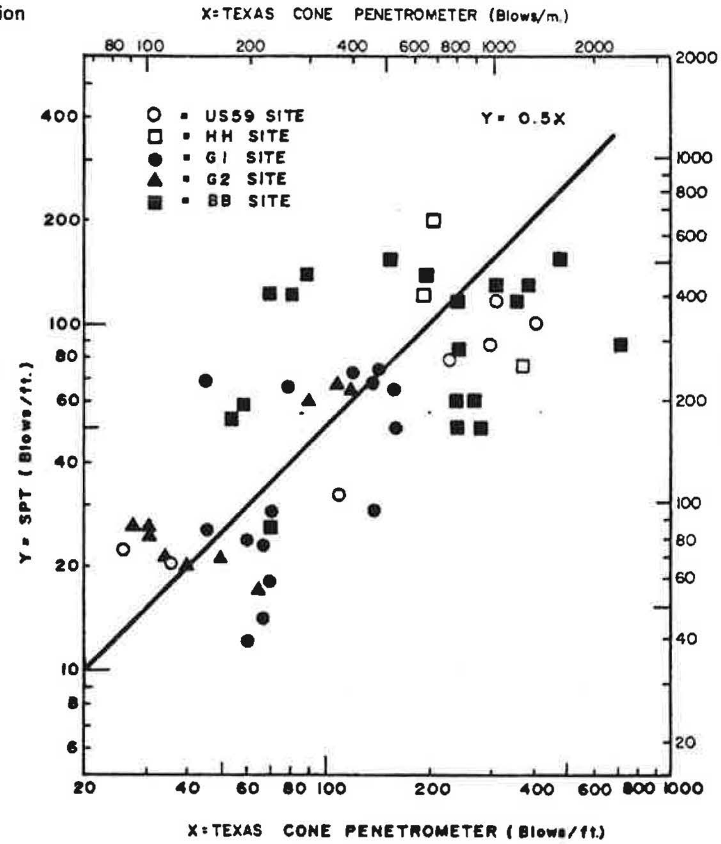


Table 1. Summary of N-values, effective angles of shearing resistance, and drained shear strengths.

Test Site	N-Value (blows per foot) N_{TCP}	N_{SPT}	N'_{SPT}	ϕ' (°)	s (kPa)	Test Site	N-Value (blows per foot) N_{TCP}	N_{SPT}	N'_{SPT}	ϕ' (°)	s (kPa)
A	35	18	17	42.0	39.4	D	80	40	28	41.0	169.2
A	60	30	23	40.0	43.1	D	68	34	25	38.5	164.9
A	4	2	2	36.5	20.3	E	64	32	24	39.0	113.3
A	5	3	3	31.5	20.0	E	80	40	28	38.0	173.9
A	9	5	5	37.5	29.4	E	74	37	26	42.0	198.9
A	6	3	3	34.5	17.9	F	5	3	3	38.7	18.2
A	6	3	3	30.0	19.1	F	2	1	1	31.3	22.6
A	20	10	10	36.5	30.9	F	41	21	18	36.3	46.9
B	33	17	16	34.0	41.5	F	53	27	21	41.0	64.6
C	19	9	9	36.0	42.3	F	49	25	20	38.5	65.5
C	18	9	9	39.0	61.0	F	26	13	13	34.0	61.1
D	22	11	11	41.0	81.9	F	24	12	12	35.5	70.4
D	48	24	20	40.0	92.0	F	44	22	19	32.5	67.1
D	33	17	16	43.0	110.4	F	56	28	22	45.0	113.0
D	30	15	15	37.5	122.4						

Note: 1 kPa = 0.145 lbf/in².

r^2 = 0.67. Equation 8 can be used to predict the drained shear strength of these sands if N_{TCP} is known.

A correlation between s and N_{SPT} was also developed. Equation 6 was used to convert the measured values of N_{TCP} into the appropriate values of N_{SPT}. The plot of s versus N_{SPT} is shown in Figure 14. The relationship can be expressed as follows:

$$s = 3.9N_{SPT} \quad (9)$$

The coefficient of correlation for this relationship is also r^2 = 0.67.

If Equation 7 is used to adjust the N_{SPT} values where the soil conditions warrant, a correlation can be developed between s and N'_{SPT}. The plot of s versus N'_{SPT} is shown in Figure 15. The relationship can be expressed as follows:

$$s = 5.0N'_{SPT} \quad (10)$$

The coefficient of correlation for this relationship is r^2 = 0.64. Therefore, the use of N'_{SPT} did not lead to an improved correlation.

FACTORS AFFECTING PENETRATION RESISTANCE

A number of workers have investigated the factors affecting resistance to penetration. Although many variables are involved, a certain amount of agreement exists as to the major ones affecting resistance to penetration in sands. Desai (15), in an effort to present a rational analysis of the penetration phenomenon, stated that "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." He concluded that density, structure, depth, and groundwater table will have significant effects on resistance. In a study of the SPT in sands, Gibbs and Holtz (3) concluded that "The overburden pressures were found to have the most pronounced and consistent effects on the penetration resistance values." Schultz and Knausenberger (17) report that "Dynamic penetrometers react very sensitively to any changes of compactness or grain size."

The consensus seems to be that unit weight, particle size, moisture content, and overburden pressure are the major factors affecting resistance to penetration in sands. This opinion is substantiated by the summary of the con-

Figure 11. Relationship between standard penetration test N-value (N_{SPT}) and effective angle of shearing resistance for SP, SM, and SP-SM soils.

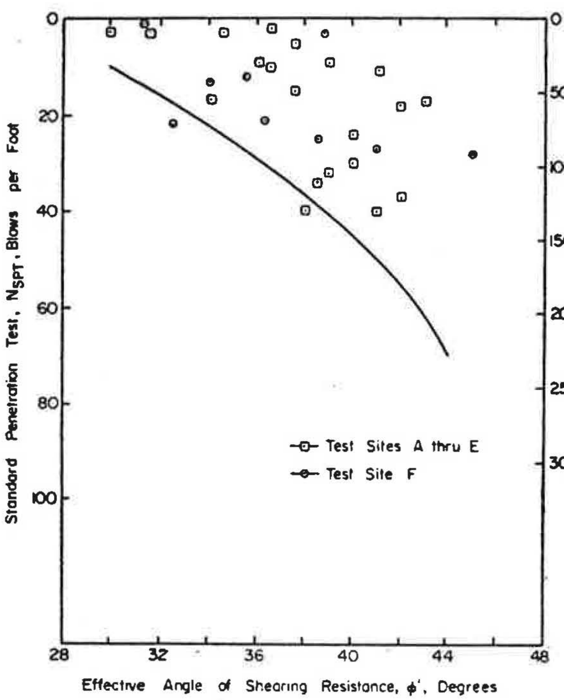


Figure 12. Relationship between standard penetration test N-value (N'_{SPT}) and effective angle of shearing resistance for SP, SM, and SP-SM soils.

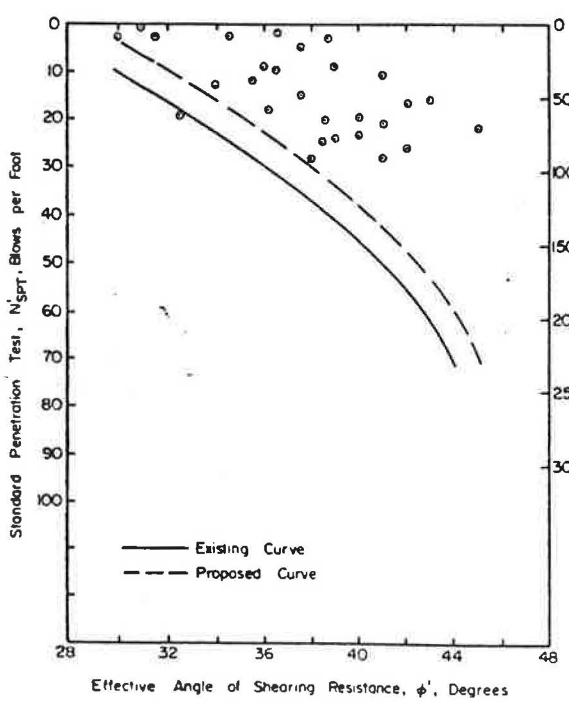


Figure 13. Relationship between drained shear strength and resistance to penetration (N_{TCP}) for SP, SM, and SP-SM soils.

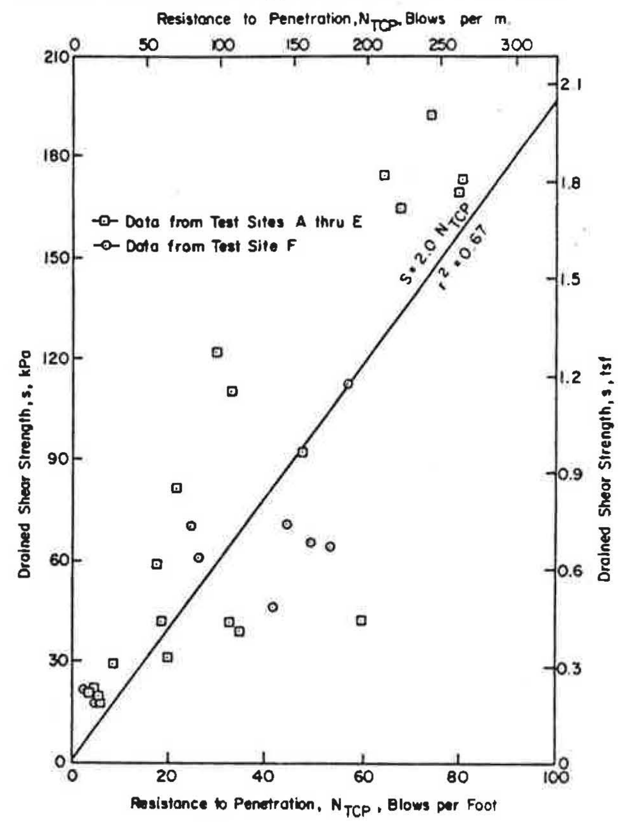


Figure 14. Relationship between drained shear strength and resistance to penetration (N_{SPT}) for SP, SM, and SP-SM soils.

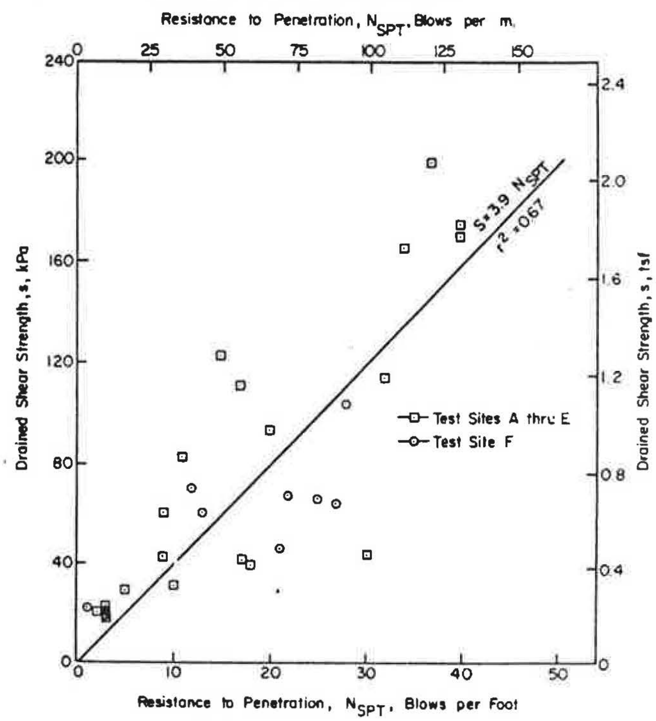
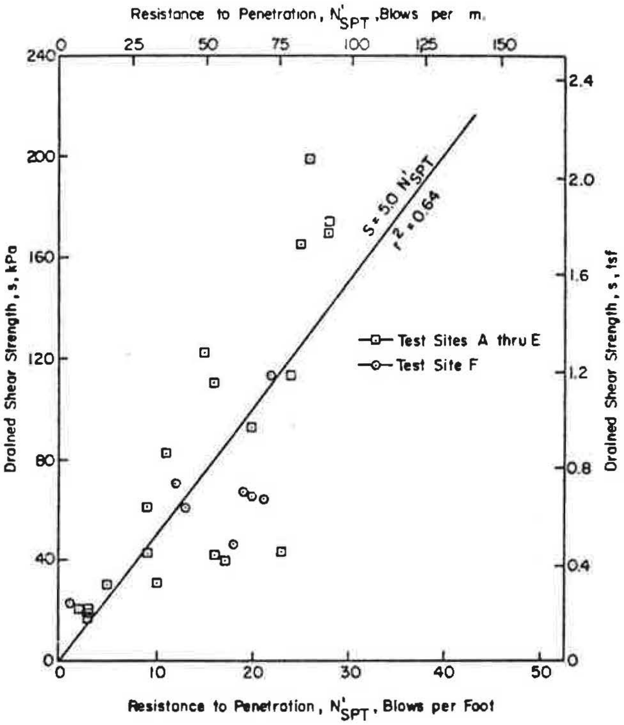


Figure 15. Relationship between drained shear strength and corrected resistance to penetration for SP, SM, and SP-SM soils.



clusions of 21 workers given by Bodarik (18); although there is not complete agreement concerning the factors that have the most effect, there is general agreement concerning what factors affect the resistance to dynamic penetration in sands.

The effect of overburden pressure on penetration resistance is probably best explained by Bodarik, who states that

> 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, cause an appreciable increase in density and affect the results of the sounding.

Some field observations have confirmed the effect of overburden pressure on the results of the SPT. Fletcher (19) reported that the removal of 4.6 m (15 ft) 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 workers [for example, Bowles (13)] to correct the N-value at shallow depths to include the effect of overburden. Gibbs and Holtz (3) have shown that "for two cohesionless soils of the same density, the one with the greatest overburden pressure has the higher penetration number." Several cases were observed in this study where N-values increased with increasing overburden pressure. However, variations in other factors may also have affected the resistance to penetration.

Terzaghi and Peck (20) have suggested that, in loose, very fine or silty sands below the groundwater 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 (21), "These positive pore-water pressures would reduce the 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, for dense, very fine or silty sands, the penetration test might induce negative pore-water pressures that would increase the resistance to penetration and thus increase the N-value. The effect of the groundwater level was noted at two test sites in this study. In neither case could a definite conclusion be drawn concerning the effect of the groundwater level on the N-value because of the variations in other factors that affect the resistance to penetration. However, an increase was observed in the resistance to penetration of relatively loose materials below the water table, which is not in agreement with the statement made by Terzaghi and Peck.

Another factor thought to have a major effect on the resistance to penetration is particle-size distribution. According to Desai (15), "Grain size distribution has a considerable effect on the penetration resistance for a given relative density." Because it has been shown (3, 22) that penetration resistance can be related to relative density and relative density is a function of particle size, it can be concluded that particle size does have an effect on penetration resistance. A sand composed of a large amount of gravel, according to Desai, will have a relatively low resistance to penetration, because the round gravel will act like ball bearings and thus reduce friction and penetration resistance considerably. Sands that have a large amount of fine material will experience positive or negative pore-water pressures (depending on the state of compactness), which will result in increases or decreases in the N-values. In natural sand deposits where the particle-size characteristics are not uniform, the effect of particle size is not so easily determined. As in the case of unit weights, it is suspected that the particle size will affect the N-value, but this effect is not obvious. Several situations were encountered in this study in which the penetrated soil had a large percentage of material passing the 75-μm (no. 200) sieve and correspondingly high N-values. However, other factors (such as overburden pressure, position of the groundwater table, and unit weight) were not constant among these situations and, thus, the cause of the increased N-value could not be attributed to any one factor.

CONCLUSIONS

The relationship between the drained shear strength and the resistance to penetration of cohesionless soils was studied by the use of new techniques in sampling and testing. The following conclusions are made:

1. The TCP test N-value (N_{TCP}) and the drained shear strength (s) of poorly graded and silty sands (SP, SM, and SP-SM) can be correlated by using Equation 8.
2. For the same sands, the SPT N-value (N_{SPT}) and the adjusted SPT N-value (N'_{SPT}) can be correlated with the drained shear strength (s) by using Equations 8 and 9, respectively.
3. The relationship between the effective angle of shearing resistance (ϕ') and the N_{TCP} currently used by the TSDHPT was found to be a lower bound for the data obtained in this study.
4. A widely used relationship between ϕ' and N_{SPT} was found to be a lower bound for the data obtained in this study; a new lower-bound curve was developed based on the relationship between ϕ' and N'_{SPT}.
5. Other factors that might affect penetration resistance in a cohesionless soil (e.g., overburden pressure, unit weight, particle-size characteristics, and position of the groundwater level) were also con-

sidered in this study, but no correlations or trends were established. Rather, it is shown that, in a field study such as this, control of individual factors is not possible. Therefore, because individual factors cannot be separated, it is probable that interaction occurs and a combination of several factors actually affects the resistance to penetration.

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Prediction of Permanent Strain in Sand Subjected to Cyclic Loading

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The trend toward ever-increasing axle loads on highways and airport pavements requires that new methods for pavement design and rehabilitation be developed. This paper introduces a simple and economical procedure whereby permanent strain in sand subjected to cyclic loading can be characterized by using stress and strain parameters from the universally accepted static triaxial test. To develop the procedure, duplicate samples were tested by using both a static triaxial apparatus and a closed-loop electrohydraulically actuated triaxial system. The dynamic test results