JOURNAL OF THE
GEOTECHNICAL
ENGINEERING DIVISION

Statics of SPT

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Introduction

An engineer may have many reasons to wish to better understand the statics of the widely used American Society for Testing and Materials (ASTM) D-1586 Standard Method for Penetration Test and Split-Barrel Sampling of Soils (SPT). Such understanding would offer a starting point for understanding the dynamics of this dynamic test. Understanding SPT statics might allow some theoretical comparisons with modified SPT or with other types of soil penetration tests, or allow a more informed and effective application to static design problems. Further, an engineer might then improve his or her ability to differentiate good from poor SPT practice and data and to understand the effects of intentional or unintentional changes in the test.

The recent availability of the friction-cone tip in the Dutch quasi-static (q-s) cone penetration test, (CPT), ASTM D-3441 has made it possible to perform experiments and field observations that provide useful knowledge about the statics of the SPT. This paper organizes much of this knowledge.

Components of SPT Sampler Penetration Resistance

Fig. 1 shows the vertical forces involved when the SPT sampler penetrates into the bottom of a borehole. From vertical force equilibrium, Eq. 1 expresses that the additional force F, added to the bouyant weight of rods + sampler $= W'$, required for penetration equals the sum of end bearing resistance, F_e, plus the sum of the outside and inside soil friction or adhesion forces, F_o and F_i

$$F + W' = F_e + (F_o + F_i) \quad \text{(1)}$$

Note.—Discussion open until October 1, 1979. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 105, No. GT5, May, 1979. Manuscript was submitted for review for possible publication on July 17, 1978.

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Assuming the average sampler inside and outside unit friction or adhesion equals f for both the inside and outside sampler walls over the length of penetration L, and that the unit bearing pressure at the ends of the sampler equals q, then substitution into Eq. 1 gives Eq. 2

$$F + W' = qA_c + (d_i + d_o)\pi Lf \quad (2)$$

Make the further assumption that the SPT q over the 10.7 cm² end area of the sampler equals a constant, C_1, times the static cone end bearing, q_c over the 10 cm² area of the cone, or $q = C_1 q_c$. After also assuming that the SPT friction, f, equals a constant, C_2, times the local friction on the CPT friction-cone, or $f = C_2 f_c$, then Eq. 2 becomes Eq. 3

$$F + W' = C_1 q_c A_e + (d_i + d_o)\pi L C_2 f_c \quad (3)$$

Recognizing that the CPT ratio f_c/q_c equals the friction ratio, denoted R_f,

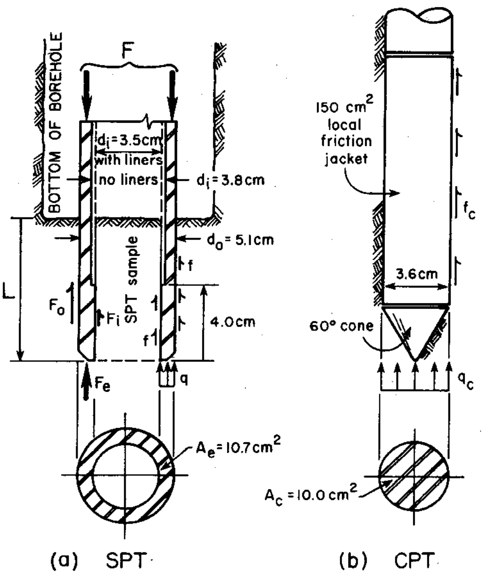

FIG. 1.—Comparison of SPT and CPT Components of Penetration Resistance

and substituting this into Eq. 3 produces Eq. 4

$$F + W' = [C_1 A_e + (d_i + d_o)\pi L C_2 R_f]\, q_c \quad (4)$$

The energy required, in addition to the potential energy from W', for static penetration of the SPT sampler equals the length increment of penetration, ΔL, multiplied by the average additional static force required over this penetration interval, $\bar{F}$. Because Eq. 4 shows that F increases linearly with L, this average force equals the value of F when L has an average value $\bar{L}$ over ΔL. After designating ΔN as that the increment of SPT blow count over the penetration interval ΔL, and making the further assumption (verified subsequently) that this ΔN depends linearly on the added energy required for quasi-static penetration over the same interval, one obtains Eq. 5

$$\Delta N \sim \bar{F} \Delta L \qquad (5)$$

In accord with the ASTM D-1586, engineers must obtain the ΔN information over the penetration increments of 0 in.–6 in., 6 in.–12 in., and 12 in.–18 in. For these cases $\bar{L}$ equals 3 in., 9 in., and 15 in., respectively. After assuming that q_c remains constant over all three ΔL increments if in the same soil, and noting that ΔL equals 6 in. for all cases, the following Eqs. 6 express the ratio of each 6-in. ΔN in comparison to the final ΔN from 12 in.–18 in.

$$X_1 = \frac{\Delta N_{0\text{in.}-6\text{in.}}}{\Delta N_{12\text{in.}-18\text{in.}}} = \frac{[10.7\, C_1 + 2.052\, C_2\, R_f(\%)]\, q_c - W'}{[10.7\, C_1 + 10.26\, C_2\, R_f(\%)]\, q_c - W'} \qquad (6a)$$

$$X_2 = \frac{\Delta N_{6\text{in.}-12\text{in.}}}{\Delta N_{12\text{in.}-18\text{in.}}} = \frac{[10.7\, C_1 + 6.156\, C_2\, R_f(\%)\, q_c - W'}{[10.7\, C_1 + 10.26\, C_2\, R_f(\%)]\, q_c - W'} \qquad (6b)$$

$$X_3 = \frac{\Delta N_{12\text{in.}-18\text{in.}}}{\Delta N_{12\text{in.}-18\text{in.}}} = 1.0 \qquad (6c)$$

In Eqs. 6 q_c has the units of kilogram force per square centimeter and W' of kilogram force.

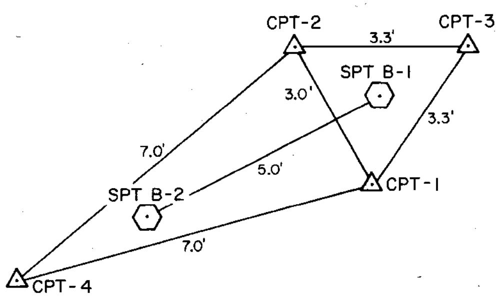

FIG. 2.—Location Plan for Comparative SPT and CPT

The writer will show subsequently that the X ratios predicted by this theory will agree with those observed in the field and in the laboratory, and thus provide evidence to support the theory. It first becomes necessary to evaluate the assumed SPT-CPT proportionality constants C_1 and C_2.

Field Experiment to Evaluate C_1 and C_2

The writer performed two special SPT borings within a pattern of four Begemann friction-cone CPT soundings, in the following order: CPT-1, -2, -3, SPT B-1, CPT-4, and finally SPT B-2, all near the Palacios (9) research site on the University of Florida campus. Fig. 2 shows their relative positions. In these SPT borings the writer alternated, at 2-1/2-ft intervals, between an ordinary dynamic SPT test with 18 in. of sampler penetration and (q-s) penetration of the SPT sampler over the 0-in. to 18-in. penetration interval. Fig. 3 shows the detailed q_c logs from each of the four CPT soundings, logs of the average friction ratio from each group of three soundings surrounding one of the SPT borings, and the depth pattern of standard (SPT) and q-s testing with the SPT sampler. This same pattern applies to both borings. However, in B-1 the writer used a sampler designed for liners, but did not use liners. In this condition the sampler had an initial 1.375-in. inside diameter length for 1.6 in. above the cutting edge, and then the inside diameter enlarged to 1.50 in. For boring B-2 the writer used the same sampler, but with liners to provide a constant inside diameter of 1.375 in. as required by ASTM D-1586.

The writer used the same hydraulic load cell during the q-s (2 cm/s) penetration

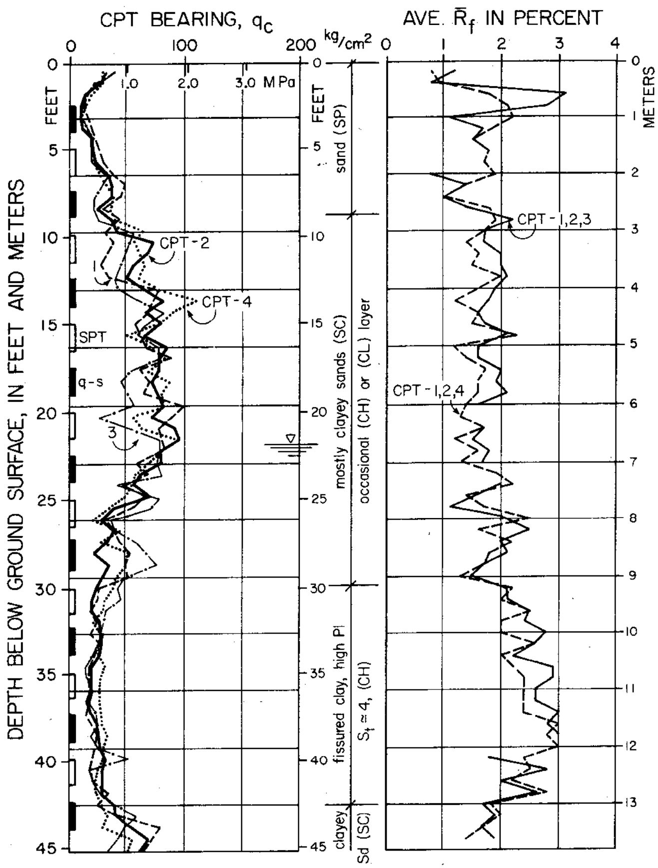

FIG. 3.—Logs of q_c and Average R_f Data from CPT Fig. 2, Also Showing Alternate Depths of Parallel Q-S and *N*-SPT

of the sampler as ordinarily used with the CPT to measure q_c and with it made measurements of F at 3-in., 6-in., 9-in., 12-in., 15-in., and 18-in. penetration. Fig. 4 shows example data with an approximately linear increase in F with increasing L. Estimating a linear fit through these data permits an extrapolation of F to L equals 0, which when added to W' should equal the sampler end bearing resistance, F_e. The writer then changed this extrapolated F_e to q and compared q with the average q_c from the surrounding static cone tests at the same elevation. Fig. 5 shows the results of these $q/q_c = C_1$ comparisons. The average C_1 from these available 16 comparisons equals approx 1.0 and does not appear to vary greatly with the types of soil tested at this site. These did not include weak clays. On this basis the writer tentatively suggests using $C_1 = 1.0$ for both the mechanical and electrical CPT tips described in ASTM D-3441. The writer guesses $C_1 \simeq 0.7$ for the mechanical tips in weak clays.

For those tests where the SPT quasi-static F increased with depth over ΔL from 6 in.–18 in., and the matching q_c and R_f data seemed constant enough to make a meaningful comparison between the CPT f_c and the SPT f, the writer made such comparisons assuming F_e did not change over these 6 in.–18 in. Four comparisons in B-1 gave C_2 equal 0.56–0.87, with an average of 0.67. Five comparisons in B-2 gave a C_2 range of 0.51–0.80, with an average of 0.69. Based on these limited data, the writer tentatively recommends the use of $C_2 = 0.7$ in all soils when using the Begemann friction-cone tip. With this tip the writer suspects that f_c exceeds f because of parasitic soil bearing resistance

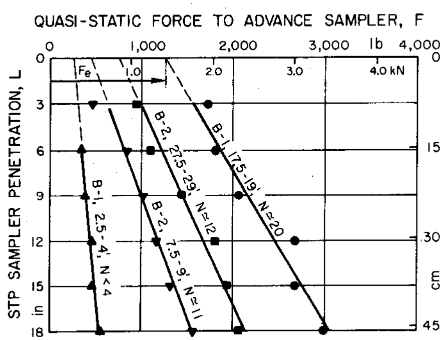

FIG. 4.—Examples of Extrapolation for F_e

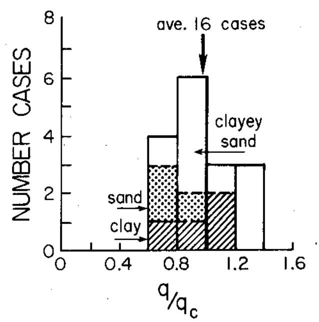

FIG. 5.—Comparison of End Bearing on SPT Sampler and CPT Begemann Cone Tip

that occurs at the bevel at the bottom of the friction jacket of this tip, and thus increases the measured f_c when using this tip. With the cylindrical tip of the type shown in Fig. 1, where such parasitic bearing cannot occur, the writer suggests using $C_2 = 1.0$.

ΔN Ratios

The writer (11) used the theory presented herein to make predictions of the Eq. 6 X values, and then compared them to the then available field data. These predictions contained the assumptions that C_1 and $C_2 = 1.0$ and that $W' = 0$. Table 1 includes these predictions and field data. Table 1 also includes the Eq. 6 predictions when $C_2 = 0.7$ and $W' = 0$. When taking $W' = 0$ the X ratios do not depend on q_c. But, they do depend on q_c when taking $W' \neq 0$. The writer made a parameter study of Eqs. 6 to check the impact of estimates of W' for various SPT depths on the predicted X values, using an R_f range of 1%–8% and appropriate q_c for strong and weak sands, silts and clays at depths from 10 ft–100 ft. The results indicate that the soil strength has a more important effect on the X values than does depth. Table 1 includes the results digested from this parameter study.

The extensive, large-chamber, SPT laboratory research work at the U.S. Army Engineer Waterways Experiment Station reported by Bieganousky and Marcuson (3) using uniform fine sands, both dry and near-saturated, provides further data against which to check the predictions in Table 1. The writer averaged the 6-in. incremental blow count results from their 51 tests in dry sand, plus 20

TABLE 1.—Predicted and Measured ΔN Ratios for Constant d_i Sampler

Assumed				$\Delta N/\Delta N_{12\text{ in.}-18\text{ in.}}$		
Begemann R_f (1)	C_2 (2)	W' (3)	Soil type (4)	X_1 (5)	X_2 (6)	Notes (7)
1.0%	1.0	0	any with R_f	0.61	0.80	Ref. 11
	0.7	0	any with R_f	0.68	0.84	
	0.7	$\neq 0$	loose sand	0.60	0.80	
	0.7	$\neq 0$	dense sand	0.66	0.83	
	loose to dense Florida sands above and below water table			0.63	0.81	average 33 tests Ref. 11
	WES Lab fine sand			0.46	0.79	average 71 tests Ref. 3
	WES Lab med-cse sand			0.60	0.83	average 34 tests Ref. 4
2.5%	1.0	0	any with R_f	0.37	0.68	Ref. 11
	0.7	0	any with R_f	0.50	0.75	
	0.7	$\neq 0$	loose clayey silts	0.30	0.65	
	0.7	$\neq 0$	dense clayey silts	0.48	0.74	
	Florida silty marl $\bar{w} \simeq 45\%$ $LI \simeq 1.00$			0.46	0.72	average 25 tests Ref. 11
4.0%	1.0	0	any with R_f	0.37	0.68	Ref. 11
	0.7	0	any with R_f	0.42	0.71	
	0.7	$\neq 0$	NC clay	0	0.4	
	0.7	$\neq 0$	highly OC clay	0.38	0.69	
	stiff, fissured, high PI clay $\bar{S}_t \simeq 4$, $\overline{LI} \simeq 0$			0.44	0.66	average 33 tests Ref. 11
8.0%	1.0	0	any with R_f	0.29	0.645	Ref. 11
	0.7	0	any with R_f	0.32	0.66	
	0.7	$\neq 0$	NC clay	0.09	0.54	
	0.7	$\neq 0$	highly OC clay	0.30	0.65	

tests in near-saturated sand. They conducted these tests at depths of 2.3 ft and 3.7 ft into a 6-ft deep chamber. These tests gave $\bar{X}_1 = 0.46$ and $\bar{X}_2 = 0.79$. Bieganousky and Marcuson (4) also reported the detailed results from additional chamber SPT Tests into two medium to coarse sands, using a relative density range of 14%–96% and a vertical effective stress range of 10 psi–80 psi. Their results from 34 tests produced an average $\bar{X}_1 = 0.60$ and $\bar{X}_2 = 0.83$, as also shown in Table 1. Although Bieganousky and Marcuson did not include any R_f information in their papers, for the purpose of comparisons in Table 1 the writer assumed their sands had a typical Begemann R_f of 1%.

Inspection of Table 1 will show that the available field and laboratory evidence shows good agreement with the various predictions based on Eq. 6, including the presumed most accurate one using $C_2 = 0.7$ and $W' \neq 0$. Even the prediction for soft, normally consolidated (NC) clay matches field experience. Here engineers often observe the penetration of the sampler by the weight of rods alone and thus would observe the predicted $X_1 = 0$.

One could argue that the observed low X_1 could result from the "seating" of the sampler into a loose soil residue at the bottom of the borehole, due either to loosening during the drilling or due to sedimenting through the drilling fluid, or both. But, such would not easily explain the distinct X_2. More likely, SPT borings that use drilling mud and rotary drilling (all actual data in Table 1) either leave little loosened soil for the sampler to penetrate or the sampler-rod-hammer system easily penetrates it under its own weight before the $\Delta N_{0\text{in.}-6\text{in.}}$ count begins, or both. The writer concludes that the Table 1 comparisons provide strong support for the validity of the static theory and SPT-CPT interrelationships presented.

Importance of Sampler Friction

Some time ago Awkati (1) made the then surprising observation that his SPT N-values seemed to correlate better with the CPT R_f than with q_c. Begemann (2) later made a similar observation. These observations suggest that inside and outside sampler friction-adhesion play a dominant, or at least a major, role in determining N. This would also help explain the pronounced stair-step pattern of increasing ΔN often observed in the blow count record for successive 6-in. increments of penetration during the same penetration of the sampler. The writer (11) presented a dramatic example of such a stair-step pattern when using a 5-ft long SPT-type sampling operation with 1-ft increments of penetration. The important increase in sampler side friction, $F_o + F_i$, with increased penetration, L, could account for the preceding observations.

Having the sequence of X values in Table 1 permits a simple computation for the division of $N = (\Delta N_{6\text{in.}-12\text{in.}} + \Delta N_{12\text{in.}-18\text{in.}})$ into bearing and friction components. Table 2 and Fig. 6(*a*) show the result.

It appears from Table 2 and Fig. 6(*a*) that end bearing dominates in low friction ratio soils but that side friction resistance dominates in high friction ratio soils. This helps explain the possibility of a better correlation between SPT N-values and CPT f_c rather than against q_c, as mentioned at the beginning of this section.

Soils with high R_f, such as relatively insensitive clays, have their penetration resistance dominated by side friction-adhesion. As an extreme, all soils weak enough to have W' alone cause sampler end bearing penetration would have 100% of N due to side friction. As noted by the writer (11) this helps greatly to explain the misleadingly low N-values often obtained in sensitive clays. For example, after the partial remolding due to sampler or cone displacement of clay an insensitive clay with $R_f = 8\%$, might have R_f reduced to 1% if it had a sensitivity of about 10–15. Assuming end bearing remains constant, this would drop the total resistance to about 25% of that of the insensitive clay.

Actually, end bearing would also drop somewhat as a result of the sensitive clay structure and the average quasi-static $\bar{F}$ over L = 6 in.–18 in., and therefore also N, would drop to approx 1/5 its magnitude in insensitive clay. Casagrande (5) and deMello (6) have reported such drops in the field.

Effects of Removing Liners

Unlike the SPT samplers used to obtain the real SPT data reported in Table 1, almost all SPT samplers used in the United States today have an enlarged

TABLE 2.—Components of N when Using 1.375 in ID Sampler

Sequence			Typical Begemann	Percentage N due to	
X_1 (1)	X_2 (2)	X_3 (3)	R_f (4)	Bearing (5)	Friction (6)
0.6	0.8	1.0	1%	56	44
0.5	0.75	1.0	2-1/2%	43	57
0.4	0.7	1.0	4%	29	71
0.3	0.65	1.0	8%	15	85

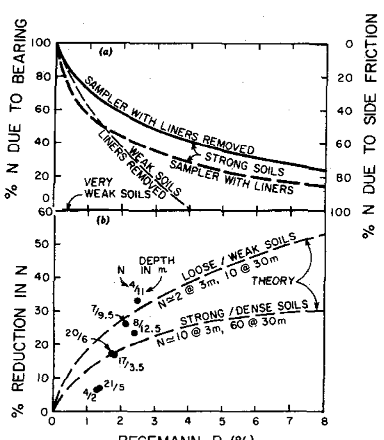

FIG. 6.—Some Effects of Enlarging SPT Inside Diameter by Removing Liners: (*a*) Relative Values of Quasi-Static Components of Sampler Penetration; (*b*) Reduction in N-Values

inside diameter to hold liners (shown dashed in Fig. 1). But, drillers almost always use them without the liners. This, of course, might greatly reduce the F_i inside friction resistance. After inspecting many SPT samples for the apparent clearance between sample and inside walls of the sampler, and also noting the apparent shear resistance between sample and sampler, the writer believes that engineers can reasonably assume that f = 0 along that part of the inside of the sampler with the liners removed. Making this assumption, allowing a

1.6-in. cutting shoe length with the 1.375-in. inside diameter before the liners begin, and using the same theory as presented for the constant 1.375-in. inside diameter sampler, results in Table 3.

Comparing Tables 2 and 3 shows that removing the liners decreases the relative importance of sampler side friction and thus raises the X_1 and X_2 values. As done for Table 2, the writer then converted these new ratios into relative end bearing and side friction components of sampler penetration resistance. Fig. 6(*a*) presents a picture of the degree to which the present theory suggests that removing the liners has made the SPT more of an end bearing test.

The reduction of only inside sampler friction, F_i, due to the removal of the liners must, from Eq. 1, also reduce F. This in turn would reduce $\bar{F}$ for the SPT ΔL interval of 6 in.–18 in. which from Eq. 5 would also reduce N. To produce Tables 2 and 3 the writer first had to compute the $\bar{F}$ value for each 6-in. penetration from 0 in.–18 in. The comparison of the 6-in.–18-in. $\bar{F}$ values

TABLE 3.—Predicted ΔN Ratios for Sampler with Liners Removed C_1 = 1, C_2 = 0.7

Begemann R_f (1)	W' (2)	Soil type (3)	$\Delta N/\Delta N_{12\text{ in.}-18\text{ in.}}$ X_1 (4)	$\Delta N/\Delta N_{12\text{ in.}-18\text{ in.}}$ X_2 (5)
1%	0	all with R_f	0.78	0.89
	≠0	loose sand	0.70	0.85
	≠0	dense sand	0.76	0.88
2-1/2%	0	all with R_f	0.61	0.81
	≠0	loose/weak	0.40	0.70
	≠0	strong/dense	0.59	0.795
4%	0	all with R_f	0.505	0.74
	≠0	NC clay	0	0.2
	≠0	highly OC clay	0.46	0.73
8%	0	all with R_f	0.415	0.71
	≠0	NC clay	0	0.5
	≠0	highly OC clay	0.37	0.69

between the constant d_i and removed-liner samplers allows computing the percent reduction in F, and therefore also N, due to removing the liners. Fig. 6(*b*) presents the resulting N-value reductions as a function of Begemann friction ratio. This theory predicts important percent reductions in N, which become even higher as soils become weaker and N decreases.

The only available data to check Fig. 6(*b*) comes from seven comparisons from the field experiment described previously in connection with Figs. 2–5. Fig. 6(*b*) includes these field data. Although sparse and subject to considerable uncertainty because of variable soil conditions between borings B-1 and B-2, they nevertheless tend to confirm the theory.

From the driller's viewpoint removing the liners not only makes easier the removal of the soil in the sampler but also increases sample recovery. Less inside friction should allow more soil to enter before inside friction exceeds the added end bearing across the 1.375-in. opening and the sampler "plugs." The writer measured recovery in all q-s and ordinary SPT samples in both

B-1 and B-2 and then compared recoveries at the same depth. In all nine comparisons involving q-s sampling, as well as in all eight with ordinary SPT sampling, the writer measured a greater recovery when not using the liners (B-1). With the ordinary SPT sampling the writer measured a no-liner (B-1) average recovery of 17.8 in. and an average with-liner (B-2) recovery of 11.9 in. The average 99% sample recovery without the liners versus the 66% with liners suggests that a constant d_i sampler has significantly more tendency to "plug." Plugging would transfer the inside resistance from friction to additional end bearing. Thus, from this viewpoint removing the liners can reduce end bearing and at least partially offset the increased percent end bearing effect presented in Fig. 6(*a*).

The almost identical average recoveries from the q-s sampling in the same

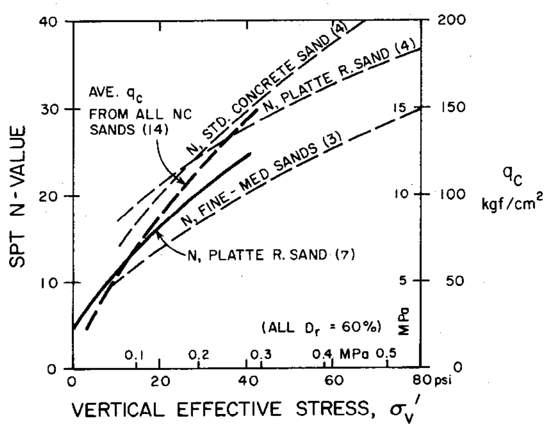

FIG. 7.—Comparison of Influence of σ_v' on N and q_c Values

borings also support the present theory and provide further evidence of the basic similarity between q-s and dynamic sampling with the SPT sampler.

Importance of Vertical and Radial Effective Stresses

The recent papers by Bieganousky and Marcuson (3,4) tend to confirm the much earlier findings by Gibbs and Holtz (7) concerning the importance of the vertical effective stress existing at the level of the SPT in determining the N-value at that level. In Fig. 7 the writer took data from the B & M tests in fine and in medium to coarse sands, both dry and near-saturated, and plotted N-values against vertical effective stress for $D_r = 60\%$. The writer then estimated an average curve for each set of data. Fig. 7 also includes the common Gibbs and Holtz correlation for $D_r = 60\%$. Note that for this case these separate studies agree quite well.

Fig. 7 also includes the writer's (14) lab chamber test correlations between q_c and σ_v' for the Fugro tip in NC fine sands. The good agreement between the N and q_c curves emphasizes that the vertical effective stress influences both the SPT and the CPT in a similar manner. The values N and q_c vary approximately with $\sigma_v'^{(0.5-0.7)}$, with all other variables held constant.

Horizontal effective stresses probably have an even greater influence on N-values than vertical stresses. Bieganousky and Marcuson (3) performed only five tests on overconsolidated sand, all with an overconsolidation ratio (OCR) $\simeq 2.9$. These tests showed an average increase in N-value of 30%. The limited number of tests, plus the scatter of these data, did not permit refined conclusions but did definitely indicate a significant increase in N with an increase in horizontal stress due to overconsolidation, while holding σ'_v constant. Some test chamber work at Duke University (10) (Vesić, personal note) showed that N varies with the level of octahedral effective stress. The much more extensive and controlled large-chamber testing research with the CPT indicates the dominant role of the radial effective stress in determining q_c (11,12,16). In view of the evidence already presented herein to demonstrate the similarity between penetration of the SPT sampler and the CPT cone, it seems reasonable to expect that the horizontal effective stress before and during sampling will also dominate the N-value. The present evidence suggests N will vary with the octahedral σ', and thus the radial effective stress has twice the effect of the vertical. In view of our present inability to measure in-situ radial stress, it may also prove very difficult to separate in the field the similar effects on N of the often simultaneously changing density and changing effective stress magnitudes.

Considering the well documented importance of effective stress levels on N-values, it becomes critical that the SPT boring technique not disturb the in-situ effective stresses before performing the SPT.

Energy Comparisons

As noted previously, for both the liner and no-liner SPT borings in Fig. 2 the writer obtained alternate samples using q-s penetration with measurement of the force required for penetration. Fig. 4 presented some of these q-s penetration force data. The area under these records, over the ΔL depth of 6 in.–18 in., represents the q-s energy added to W' (12 in.) required for penetration, $\bar{F}_{6\text{in.}-18\text{in.}}$ [(12 in.) $= E'$]. The writer plotted the variation in E' with depth for both the no-liner and liner borings and then interpolated halfway between the q-s sampling points to obtain an estimate of E' at the intermediate depth where we had an ordinary SPT sample. Fig. 8 shows a comparison of the ordinary N-values and the interpolated E' values. These data demonstrate the approximately linear proportionality between N and E' for both the with-liner and no-liner cases. The two cases produced similar results and the scatter does not appear excessive when considering natural soil variation at this site and the need to interpolate for E'. These data support the concept, expressed in Eq. 5, that N varies proportionately to the additional energy required for q-s sampling.

Note that the additional potential energy delivered to the sampler by the weight of the rod system, which $= W'$ (12 in.), occurs in both the research q-s and ordinary dynamic methods of SPT sampling. It does not form a part of the additional energy resulting from the use of a hammer. Therefore, in accord with Eq. 5, the writer has not included this potential energy in the definition of E'.

Let E^* equal the maximum possible amount of dynamic energy in the 140 lb (623 N) SPT hammer at impact after falling the specified 30 in. (0.76 m),

= 4,200 in.-lb (474 J). The maximum amount of energy that the hammer could possible supply for the SPT sampling over N blows would then equal (NE^* + 1,680 in.-lb). The 1,680 in.-lb (190 J) constant results from the 140 lb (623 N) hammer always having a net drop of 12 in. (30.5 cm) during the SPT blow count. Fig. 8 includes lines of constant fractions, α, of maximum deliverable hammer energy, or $\alpha\ NE^*$. Comparison of the data with these α lines show that the q-s energy required to obtain the SPT sample varies from 0.25–0.53 of the maximum dynamic energy available, with an average of $\bar{\alpha} = 0.38$. The nine comparisons in primarily cohesionless soils gave an average $\alpha = 0.42$ and the eight in primarily cohesive soils gave $\alpha = 0.33$.

The writer, Smith, and Ho (15) made a dynamic calibration of the drill rig used for the tests in Fig. 8 a few months after the tests. They measured the percent of E^* energy that reached the SPT sampler. At that time this rig delivered an average energy efficiency, η, of 54% E^*, and Fig. 8 also shows this line.

The energy required for the ordinary dynamic penetration of the sampler usually exceeds that required for q-s penetration because of special dynamic

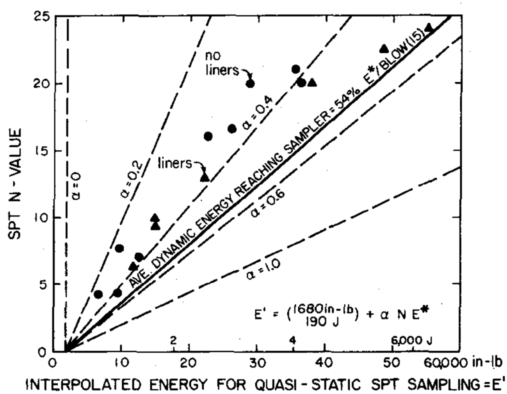

FIG. 8.—Quasi-Static Energy Needed for SPT Sampling When $\bar{\eta}$ = 0.54

losses due to ground quake, viscous effects, etc. Therefore, assuming the calibration valid during the earlier Fig. 8 tests, all the points in Fig. 8 should fall to the left of the $NE^* = 54\%$ line, and they do. Because of greater viscous effects in cohesive soils, the β ratio of dynamic to quasi-static energy required for SPT sampling in cohesive soils should exceed that required in cohesionless soils. The average $\beta = 0.54/0.33 = 1.64$ in cohesive soils exceeds $\beta = 0.54/0.42 = 1.29$ in cohesionless soils. The energy data in Fig. 8 seem to provide good support for the q-s SPT resistance components theory presented herein.

Eq. 7 also expresses the positions of the α-lines in Fig. 8, where $E'' = E' - 1{,}680$ in.-lb or $E' - 190$ J

$$E'' = \alpha\ NE^* \quad \quad (7)$$

After introducing η = the efficiency with which the hammer-anvil-rod-sampler system delivers E^* to the sampler, and noting that $\alpha = \eta/\beta$, one can write Eq. 8

$$N = \frac{E''\beta}{\eta\, E^*} \quad (8)$$

Now consider the data in Fig. 8 together with Eq. 8. These data suggest that one can consider α as an approximate constant, independent of N, in a given soil. The η efficiency also does not depend on N because almost all of the hammer energy enters the rods before the stress wave reaches the sampler (13). The η efficiency should remain approximately constant with a given driller and drill rig. Thus, one can consider β as an approximate constant in a given soil, independent of N. With β, E'' and E^* constants, Eq. 8 makes the very important prediction that N varies inversely with the energy efficiency η. Any meaningful standardization of the SPT must standardize η.

Note that the data in Fig. 8 also permit an estimate of the average added q-s force during SPT sampling, $E' = \bar{F}$ (12 in. or 0.305 m), which in turn equals an average 0.33 of $N\,E^*$ for cohesive and 0.42 for cohesionless soils. Combining this information produces Eqs. 9

$$\bar{F} = (140 + \alpha\, 350 N)\ \text{lb} \quad \text{or} \quad (623 + \alpha\ 1{,}557 N)\, N \quad (9a)$$

$$\bar{F} = 140 + (115 N \text{ to } 147 N)\ \text{lb} \quad (9b)$$

The results of the wave equation studies of the SPT by McLean, et al. (8) indicate that $F(\text{lb}) \simeq 140N$ when $N \geq 10$. This agrees well with Eq. 9*b*.

Estimating q and f from SPT

Consider the following hypothetical problem as an example of how the reader might use the SPT theory presented herein to estimate the q-s SPT sampler resistance components in a particular soil layer. Given: three parallel SPT tests at 25 ft (8 m) depth in a clayey sand layer, with successive 6-in. ΔN values of 5-8-9, 6-7-9 and 8-8-10, using a sampler with the liners removed and a drill rig delivering an energy efficiency η = 45%. Find: the average q and f for this layer at this depth.

From the data the average N = 17.0, X_1 = 0.68 and X_2 = 0.82. Comparison with Table 3 suggests an average Begemann $R_f \simeq 2\%$ for the layer. Using this R_f, Fig. 6(*a*) indicates that in relatively strong soils, such as clayey sand with N = 17, about 57% of N, and therefore also of $\bar{F}_{6\text{in.}-18\text{in.}}$, results from end bearing. Because $\alpha = \eta/\beta$, with β an approximate constant, any change in η results in a proportional change in α. Thus, for this example, and based on the present research, α for clayey sand = 0.42 (45/54) = 0.35. Using Eq. 9*a* gives $\bar{F}$ = 2,222 lb (9.88 kN) when using α = 0.35. After allowing 25 ft × 4 lb/ft = 100 lb (0.45 kN) for W', the total F_e = 100 + (0.57)(2,222) = 1,366 lb (6.08 kN). The total $(F_i + F_o)$ = (0.43)(2,222) = 955 lb (4.25 kN). Then q = 1,366/(2.204)(10.7) = 58 kgf/cm^2, or 5.7 MN/m^2. To get f one needs first to compute the outside sampler friction area at the $\bar{L}$ = 12 in. [= 75.4 sq in. (486 cm^2)] and add it to the inside, 1.375-in. diameter area [= 6.9 sq in. (45 cm^2)], giving a total area of 82.3 sq in. Then f = 955/(82.3)(14.22) = 0.82 kgf/cm^2 (80 kN/m^2).

The reader can then further convert to $q_c = q/1.0$ = 58 kgf/cm^2 and $f_c = f/0.7$ = 1.17 kgf/cm^2 values and use them to help check the reasonableness of the preceding q and f. Then $R_f = 1.17/58 = 2.0\%$, which checks with the assumed 2%. Also, q_c (kilogram force per square centimeter)$/N = 58/17 = 3.4$, which also seems reasonable for a clayey sand. Of course, this approach would give less accurate CPT data than when obtained directly from a CPT.

This paper and the preceding example help to demonstrate the convertibility between SPT and CPT data. However, the reader must also consider a tacit assumption included in the theory and example. The writer applied the C_1 and C_2 ratios to the dynamic penetration case, while he obtained them from the quasi-static research. This assumes no significantly different effective stress effects between the two modes of sampler penetration.

Conclusions

The writer draws the following conclusions from this paper:

1. End bearing and side friction/adhesion have similar magnitudes in q-s SPT sampling as in Dutch friction-cone penetration tests (CPT).
2. The simple static SPT theory presented herein, based on vertical force equilibrium and the CPT bearing and friction resistance components, seems well confirmed by its correct predictions of: (a) The relative magnitudes of the incremental, 6-in., ΔN values, or X values, in various types of soils; (b) the major importance of the side friction contribution to N in cohesive soils; (c) the magnitude of the decrease in N when removing the liners from an SPT sampler designed for liners, and the qualitative prediction of an increase in sample recovery; (d) the effect of changes in both vertical and radial in-situ effective stresses; and (e) the energy required for SPT sampling.
3. The characteristic low ΔN for the first 6 in. of SPT penetration may result almost entirely from the much smaller side friction during the first 6 in.
4. Removing the liners from an SPT sampler designed for liners improves sample recovery and removal, but it also produces a significant reduction in N and tends to make the SPT more dependent on the sampler end bearing resistance. The percent reduction in N increases with decreasing N in any type soil.
5. The N-value varies approximately directly with the q-s energy required for the same sampler penetration, and inversely with the efficiency with which the drill rig delivers hammer energy to the sampler.
6. An example shows how an engineer can estimate the quasi-static bearing and friction resistance stresses against the penetration of the SPT sampler using ordinary SPT data.

Acknowledgments

The Florida Department of Transportation, under the project direction of Robert Ho, and the University of Florida Engineering and Industrial Experiment Station sponsored the research described in this paper. William J. Whitehead, from the University of Florida Department of Civil Engineering, and Allain Gallet, former graduate student, ably performed much of the associated field work.

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Appendix II.—Notation

The following symbols are used in this paper:

A_c = projected end area of CPT cone;
A_e = projected end area of SPT sampler;
CPT = Dutch cone penetration test;

C_1 = proportionality constant relating q and q_c;
C_2 = proportionality constant relating f and f_c;
D_r = relative density, void ratio basis;
d_i = inside diameter of SPT sampler;
d_o = outside diameter of SPT sampler;
E' = quasi-static energy required, added to potential energy of rods, to move sampler over 6-in.–18-in. (15.2-cm–45.7-cm) sampling increment;
E'' = net q-s SPT sampling energy after subtracting from E' the hammer potential energy loss during the SPT N blow count;
E^* = maximum SPT hammer energy per blow at impact, = 4,200 in.-lb (474 J);
F = total quasi-static force to cause penetration of SPT sampler, in addition to W';
F_e = total end bearing resistance force against q-s SPT sampler penetration;
F_i = total inside friction resistance force against q-s SPT sampler penetration;
F_o = total outside friction resistance force against q-s SPT sampler penetration;
f = unit soil friction/adhesion along sides of SPT sampler;
f_c = unit soil friction/adhesion along CPT friction jacket;
L = depth of SPT sampler penetration below bottom of borehole;
LI = liquidity index;
N = SPT blow count, N-value;
NC = normally consolidated;
OC = overconsolidated;
OCR = overconsolidation ratio;
PI = Atterberg plasticity index;
q = unit end bearing resistance to q-s penetration of SPT sampler;
q_c = unit end bearing resistance to CPT penetrometer;
q-s = quasi-static, constant rate of penetration of approx 2 cm/s;
R_f = friction ratio, $= f_c/q_c$;
SPT = standard penetration test;
S_t = shear strength sensitivity to mechanical remolding;
W' = buoyant weight of rod and sampler system;
w = water content;
X_1 = ratio of ΔN for first 6 in. (15.2 cm) of SPT sampler penetration to N for third 6 in.;
X_2 = ratio of ΔN for second 6 in. (15.2 cm) of SPT sampler penetration to N for third 6 in.;
X_3 = 1.0;
α = ratio of SPT hammer energy required for q-s penetration of SPT sampler over L = 6 in.–18 in. (15.2 cm–45.7 cm) compared to NE^*;
β = ratio of (total dynamic hammer energy delivered to sampler)/(E'') $= \eta/\alpha$;
Δ = increment of, as ΔN;
η = efficiency with which E^* delivered to sampler;
σ' = effective stress;
σ'_v = effective vertical stress; and
$\bar{}$ = bar over letter, as $\bar{F}$, denotes average.