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Subsurface Exploration Using the Standard Penetration Test and the Cone Penetrometer Test

J. DAVID ROGERS

*Department of Geological Sciences & Engineering, 125 McNutt Hall,
University of Missouri–Rolla, Rolla, MO 65409-0230*



Key Terms: *Site Exploration, Site Characterization, Subsurface Exploration, Drilling, Standard Penetration Test, Cone Penetrometer*

ABSTRACT

The Standard Penetration Test (SPT) and Cone Penetrometer Test (CPT) have become industry standards for subsurface geotechnical investigations using small diameter (<8-in. [20-cm]) borings and soundings. Both procedures have evolved over a period of 100 and 70 years, respectively, and have been adopted as ASTM standards. Each procedure has certain advantages over the other, but both can elicit incorrect data under particular subsurface conditions that are often overlooked, depending on the experience of field personnel operating or logging the tests. This paper seeks to explain the operative assumptions employed in both procedures, highlight the various corrections that are commonly employed, and warn the reader of common errors in interpretation. The article concludes by stating that, under most conditions, the joint employment of SPT and CPT together has the greatest potential for characterizing sites correctly.

INTRODUCTION

The purpose of a subsurface exploration program is to investigate those underlying features suggested by office research and field reconnaissance so as to confirm, modify, or deny the assumed conditions existing below ground surface. The program of exploration should provide a reasonable idea as to the underlying geologic structure and geomorphic factors shaping a site. The overall goal should be to confirm 1) the geologic setting, such as soil development horizons; 2) presence of colluvium, alluvium, terrace deposits, lacustrine sediments, or landslide debris; 3) depth and style of weathering; 4) the underlying stratigraphy of layered units; 5) geologic structures, such as joints, folds, faults, and disconformities; 6) hydrogeologic characterization, such as groundwater, likelihood of perched groundwater or different “groundwater compartments,” and surface

water hydrologic regimen; and 7) the appropriate soil and rock parameters that are of interest to engineering evaluations, such as strength, compressibility, hydrologic conditions, and the likely variability of all the above across the site. Of these, assessing the future variations in hydrologic regimen is the most difficult and requires the utmost degree of professional judgment.

STANDARD PENETRATION TEST

In 1902, Charles R. Gow, owner of the Gow Construction Co. in Boston, began making exploratory borings using 1-in. (2.5-cm)-diameter drive samplers driven by repeated blows of a 110-lb hammer to aid in estimating the cost of hand excavating belled caissons (Fletcher, 1965; Mohr, 1966). The dimensions and layout of this device are shown in Figure 1A. Until that time, contractors used wash borings with cuttings, similar to the methods presently used in advancing water wells. In 1922, Gow was absorbed as a subsidiary of the Raymond Concrete Pile Co. (RCPC) under the direction of Linton Hart (Fletcher, 1965). The Gow Division of RCPC continued to employ the pipe sampler, which was hand powered and operated by three-man crews. The split-spoon soil sampler was introduced by Sprague and Henwood, Inc., of Scranton, PA, in the mid-1920s and marketed nationally. It was manufactured in variety of sizes, with outside diameters of 2.0 in. (5.1 cm), 2.5 in. (6.35 cm), 3.0 in. (7.6 cm), and 3.5 in. (8.9 cm). The inside diameters of these samplers were 0.50 in. (1.27 cm) less than the outer dimensions cited above. Two-inch (5-cm)-diameter split-spoon sampler was introduced in 1927 through the cooperative efforts of three Gow engineers: Linton Hart in New York, Harry Mohr in Boston, and Gordon Fletcher in Philadelphia. During this same period (late 1920s), Harry Mohr measured the numerical values of driving force employed by Boston area drilling crews, determined to be 140 lbs (69.3 kg) average driving weight with an average 30 in. (76.2 cm) drop, recording the number of blows required to drive the sampler 12 in. (30.5 cm) (Fletcher, 1965; Mohr, 1966). Other firms used a variety of split-spoon and push tube samplers with outside diameters varying between 2.0 and 4.5 in. (5 and 12.7 cm) and hammer weights between

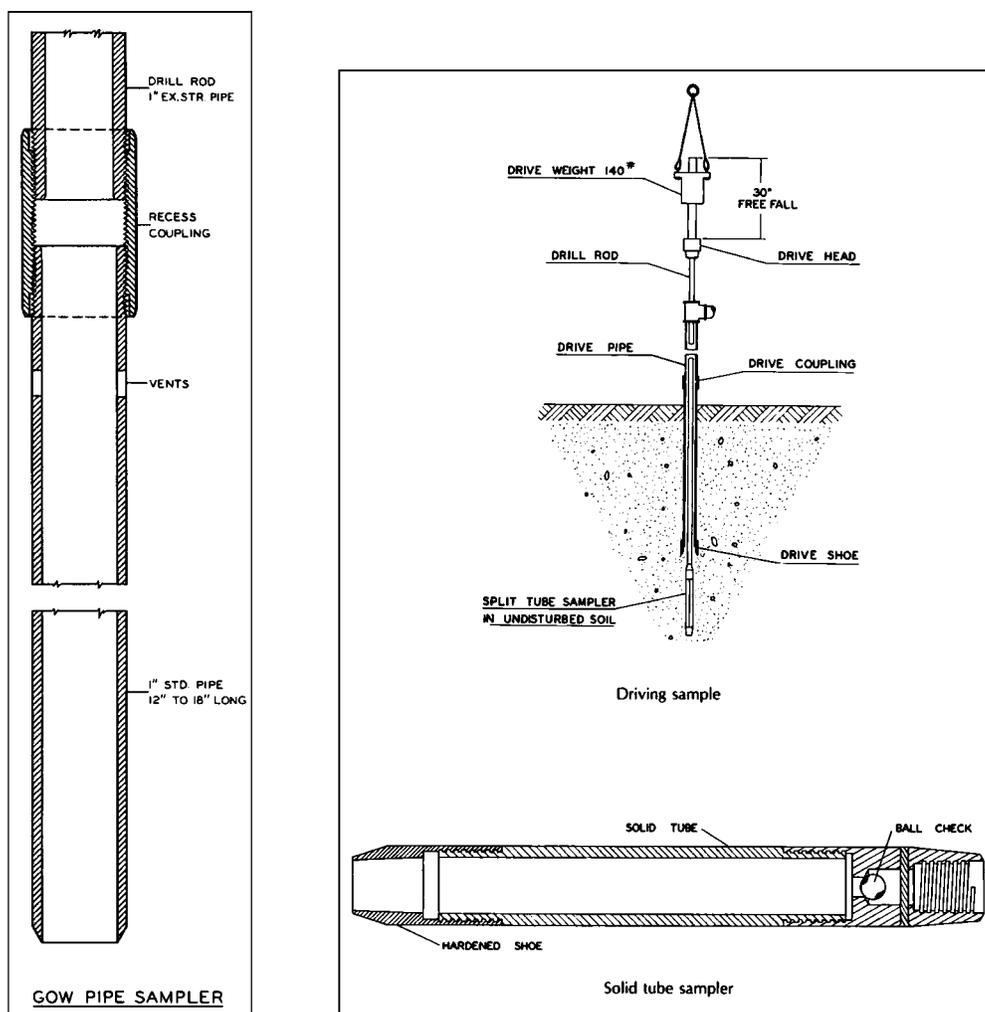


Figure 1. (A, left). The original Gow pipe sampler was the first dry sampling method. It utilized 1-in. (2.5-cm)-diameter pipe drill rod with a recessed coupling attached to a 1-in.-diameter pipe with a beveled cutting tip. (B, right) The components of a 2-in. (5.1-cm)-diameter Standard Penetration Test (SPT) split-spoon sampler, developed around 1927, after the Charles R. Gow Co. had been absorbed by the Raymond Concrete Pile Company. Figures taken from Hvorslev (1949).

100 and 350 lbs (45.5 and 159 kg) (Mohr, 1936, 1943; Stanton, 1936; and Acker, 1974).

This modified Gow sampler recovered 1½ in. (3.5-cm)-diameter samples. Even though disturbed, drive samples were able to recover thin seams of material in the correct stratigraphic sequence, providing important details that wash borings could not. Improvements to the sample barrel were made over the years, including the introduction of a ball check valve to prevent sample loss. In 1945, the split-spoon sampler barrel was augmented by the introduction of size A hollow drill rods (wall thickness of 0.234 in. [0.59 cm]), which were equipped with Jackbit threads (coarse flat threads, without deep or sharp undercuts). These replaced the old 1-in. extra pipe that had been used until that time (Fletcher, 1965). By 1940, the Gow split-spoon sampler essentially appeared as we know it today but accommodated only a 12-in. (30.5-cm)-long sample, as shown in Figure 1B. During the early

1940s, RCPC lengthened their sampler to accommodate 22 in. (55.9 cm) of sample, and this apparatus became known as Raymond Sampler (Hvorslev, 1949). In 1954, James D. Parsons of Moran, Proctor, Freeman, and Mueser in New York introduced the conventional procedure wherein blows are recorded for each of three 6-in. (15.2-cm) increments (Fletcher, 1965). The value recorded for the first round of advance is usually discarded because of fall-in and contamination in the borehole (Lo Pinto, 1966). The second pair of numbers are then combined and reported as a single value for the last 12 in. (30.5 cm). This value is reported as the raw (uncorrected) Standard Penetration Test (SPT) blow-count value, commonly termed N or, more recently, as N_{SPT} .

Karl Terzaghi liked the Raymond Sampler because Harry Mohr had collected more than 30 years of sub-surface penetration data around Boston, and since 1927, Raymond had been employing the standardized

Subsurface Exploration Using the SPT and CPT

Table 1. Prior to the introduction of energy corrections for depth, uncorrected blow counts, N_{SPT} , were compared with charts like those reproduced here, from Peck et al. (1953). These correlations were used in classifying soils and used to determine allowable bearing capacity and friction parameters for soils.

Relative Density		Consistency		
Sands and Gravels	Blows/Ft (N_{SPT})	Silts and Clay	Strength (tsf)	Blows/Ft (N_{SPT})
Very loose	0-4	Very soft	0-1/4	0-2
Loose	4-10	Soft	1/4-1/2	2-4
Medium	10-30	Firm	1/2-1	4-8
Dense	30-50	Stiff	1-2	8-16
Very dense	Over 50	Very stiff	2-4	16-32
		Hard	Over 4	Over 32

penetration procedure and apparatus out of all their offices across the United States. Terzaghi and Professor Arthur Casagrande of Harvard University vigorously sponsored adoption of the split-spoon sampling procedure through the auspices of the Committee on Sampling and Testing of the Soil Mechanics and Foundations Division of ASCE, which was formed in 1938. The work of this committee was carried out at Harvard by Juul Hvorslev, a former doctoral student of Terzaghi's in Vienna. Subsurface sampling procedures were more or less standardized by 1940, when Hvorslev (1940) wrote "The Present Status of the Art of Obtaining Undisturbed Samples of Soils," included as an 88-page appendix to the Purdue Conference on Soil Mechanics and Its Applications.

Terzaghi realized that the penetration resistance of the split-spoon sampler could provide useful *in situ* test data that might be correlated with the consistency and density of the soils encountered. While he was writing the text of *Soil Mechanics in Engineering Practice*, he sat down with Harry Mohr and developed correlations between the number of blows, N , and a number of salient properties of soils, including the relative density of sands, consistency and unconfined compressive strength of clays, and allowable bearing pressure on sands and clays. In 1947, Terzaghi christened the Raymond Sampler procedure as the "Standard Penetration Test" (SPT) in a presentation titled "Recent Trends in Subsoil Exploration," which he gave at the 7th Conference on Soil Mechanics and Foundation Engineering at the University of Texas at Austin. The first published SPT correlations appeared in Terzaghi and Peck (1948). These were soon followed by correlations relating SPT blow counts to consistency for silts and clays and relative density for sands in Peck et al. (1953), who noted that the data for sands were more reliable than those for silts or clays. These classification charts are shown in Table 1. The SPT procedure and its simple correlations quickly became soil classification standards across the United States. By 1960, published

Table 2. Estimated values of soil friction and cohesion based on uncorrected Standard Penetration Test (SPT) blow counts, taken from Karol (1960). Although bereft of overburden corrections, these simple correlations were generally used for shallow foundation investigations.

Soil Type and SPT Blow Counts		Undisturbed Soil	
		Cohesion (psf)	Friction Angle (°)
Cohesive soils			
Very soft	(<2)	250	0
Soft	(2-4)	250-500	0
Firm	(4-8)	500-1,000	0
Stiff	(8-15)	1,000-2,000	0
Very stiff	(15-30)	2,000-4,000	0
Hard	(>30)	4,000	0
Cohesionless soils			
Loose	(<10)	0	28
Medium	(10-30)	0	28-30
Dense	(>30)	0	32
Intermediate soils			
Loose	(<10)	100	8
Medium	(10-30)	100-1,000	8-12
Dense	(>30)	1,000	12

charts were appearing that also estimated soil strength based on SPT blow counts, presented in Table 2.

The "standard drive sampler" test was subsequently adopted by ASCE and the Corps of Engineers. In the early 1950s, Sprague and Henwood began producing an 18-in. (45.7-cm) version of the 22-in. (55.9-cm) Raymond Sampler, and the barrel length was left open ended (between 18 and 30 in. [45.7 and 76.2 cm]) when the SPT was adopted as ASTM Standard D 1586-84 in 1958 (Figure 2). Since this time, it has more or less become a nationwide standard. Despite standardized barrel dimensions, SPT samplers still employ an array of cutting shoes with varying tapers and lip widths, shown in Figure 3. Variations on the SPT procedure have been adopted by many foreign countries, and the International Reference Test Procedure for the SPT was adopted by the International Society for Soil Mechanics and Foundation Engineering in 1988. Figure 4 shows an SPT sampler being disassembled (left) and the opposing hemispheres of the split-spoon sampler open, exposing the recovered soil (right).

Disadvantages of the SPT Procedure

The SPT procedure evolved out of the Boston area, where Charles Gow used it to probe conditions preparatory to constructing his Gow Caissons, as was custom elsewhere (Mohr, 1964). The procedure is most reliable in granular soils, such as sand and granule gravel. A major shortcoming with the SPT has been that silts and clays exhibit different driving resistances when dry or moist. If

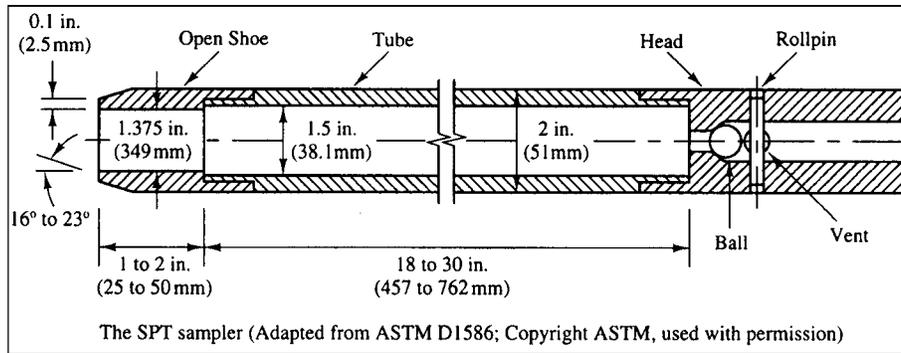


Figure 2. Standard dimensions for the Standard Penetration Test (SPT) sampler, as given in ASTM D 1586-84.

these materials become more moist at a later date, they may not exhibit the stiffness predicted by the SPT.

Another common source of interpretive error in the SPT procedure is when the sampler encounters rocks slightly larger than the sample barrel's sleeve diameter, as sketched in Figure 5. In these cases, very high blow counts can be recorded, and these horizons can easily be misinterpreted to be "bedrock" or "drilling refusal" when, in fact, the object may be a "floater" within the colluvium (Figure 5). Seasoned operators usually take their borings

a good 10 ft (3 m) into supposed "rock" to be sure of the interpretation, usually through drilling resistance, not according to drive sampling resistance alone.

A less recognized problem is the influence of strata thickness and changes in stiffness, sketched in Figure 6. As the sample barrel approaches an appreciably stiffer horizon, the penetration resistance will increase, even though the sampled material remains more or less constant throughout the softer horizon. This can lead to overestimates of strength, density, and compressibility



Figure 3. Drive samplers usually employ three types of cutting heads, or shoes, shown here. The sharp tapered heads are intended for soil sampling, while the more blunt tips are designed for greater longevity when sampling granular soils.



Figure 4. A split-spoon sampler barrel must be recovered from the hole, detached from the drill rod, and mechanically broken down, as shown at left. The two halves of a typical Standard Penetration Test (SPT) sampler with entrained soil are shown at right.

based solely on blow-count values. A big disadvantage of the SPT procedure is that it reports the average blows per foot during any given sample round, so the measurement would only be valid for horizons >12 in. (30.5 cm) thick, plus the influence area beneath the sampler shoe sketched in Figure 6. This is the zone of influence in front of the sampler shoe (sketched in Figure 6). This influence zone usually encompasses an additional 8–14 in. (20–36 cm), equal to four to seven times the sample barrel diameter. Overall, the SPT procedure tends to “average” the penetration resistance of the materials sampled, and the material effects of low strength horizons less than 20–26 in. (51–66 cm) thick may altogether be missed, though their existence may be gleaned from visual inspection of the recovered samples.

Baseline References on the SPT Procedure

Many of the SPT correlations have been explored, and there exists no small number of problems, requiring considerable judgment. Most of these problems are discussed in the following publications: Fletcher (1965), Ireland et al. (1970), de Mello (1971), LaCroix and Horn (1973), Liao and Whitman (1986), Skempton (1986), Clayton (1990), Robertson and Wride (1997), and Youd and Idriss (1997).

In 1986, a series of new correlations and corrections were introduced, and these are in current usage. These included Liao and Whitman (1986), Riggs (1986), Skempton (1986), and Robertson and Wride (1997). In 1990, Clayton (1990) presented an expanded listing of SPT hammer efficiencies (E_m) and rod energy ratios (C_E) that have been widely accepted and were reproduced in

Clayton et al. (1995). For evaluation of liquefaction potential, raw SPT blow counts are normally corrected to $(N_1)_{60}$ values, described later.

Burmister's Input Energy Correction

Despite all the encouragement to adopt Terzaghi's SPT, most people went on using whatever devices they had previously until more and more of Casagrande's students infiltrated the ranks of foundation engineering. In the New York area, the favored device was the 3.625-in.-diameter Moran & Proctor or, M & P Sampler, which had been developed by Carlton Proctor for the firm's exploration of the San Francisco Bay Bridge project (Proctor, 1936). The M & P sampler (Figure 7) allowed recovery of much larger 3-in. (7.6-cm)-diameter samples, using 5,000 in.-lbs (5,760 cm-kg) per blow in lieu of the SPT's 4,200 in.-lbs (4,838 cm-kg). Proctor engaged Professor Donald Burmister of Columbia University to develop a scheme for correlating M & P sampler blow counts with those of Mohr's smaller SPT sampler, commonly employed in New England. Burmister assumed that SPT blow counts were simply proportional to driving weight and energy input versus the annular cross-sectional area of the sampler shoe. This reasoning ignored the increase in skin friction area with larger-diameter samplers and increasing skin friction with depth.

Burmister's relationship considered energy input as the weight of hammer multiplied by drop height, the size of the recovered sample (D_i), and sample barrel diameter (D_o). These factors were combined to provide input energy and diameter correction for other tests to correlate

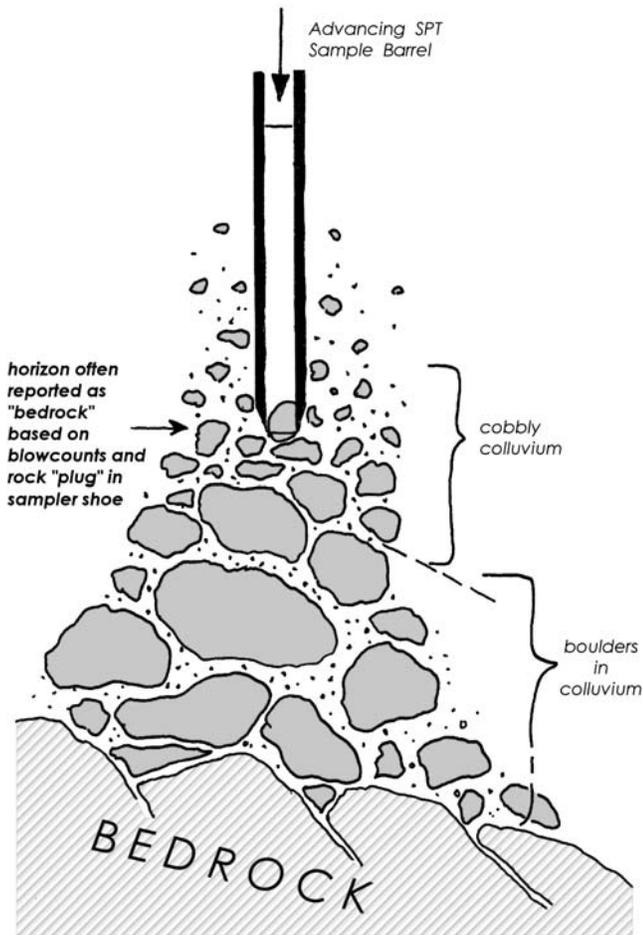


Figure 5. One disadvantage of the Standard Penetration Test (SPT) method is the small diameter of the sampler shoe. Although most suited to granular soils, the SPT cannot recover clasts greater than 1 3/8 in. across, as shown. This often leads to erroneous assumptions about the position of “encountered bedrock” or “drilling refusal.”

with the same values used in the SPT procedure (ASTM, 1984, D 1586-84) as

$$N^* = N_R \frac{(W \text{ lbs})(H \text{ in.})}{(140 \text{ lbs})(30 \text{ in.})} \left[\frac{(2.0 \text{ in.})^2 - (1.375 \text{ in.})^2}{(D_o)^2 - (D_i)^2} \right]$$

where W is the hammer weight, H is the height of the drop, D_o is the outside diameter of the sample barrel, D_i is the diameter of the drive sample, N_R is the raw blow count, and N* is the blow count reported as the equivalent SPT value. The Burmister energy correction takes the raw SPT blow-count value and multiplies it by an appropriate fraction, derived from this relationship. The corrected blow-count value is usually denoted by an asterisk (*) on the boring log, with a note explaining that the blow counts have been adjusted.

Like New York, California has long employed the SPT method along with employment of a larger-diameter drive

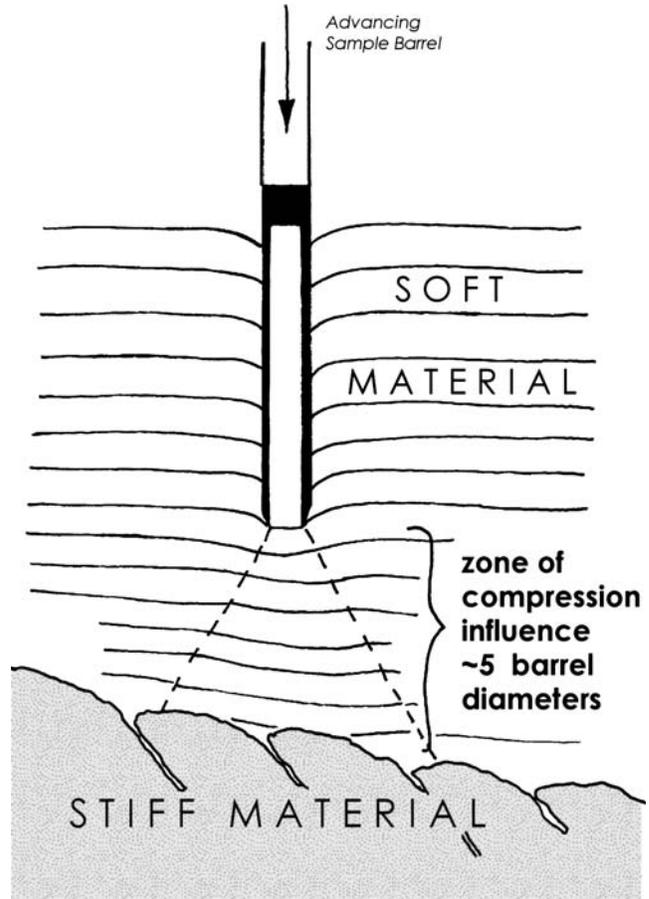


Figure 6. As the Standard Penetration Test (SPT) sample barrel approaches stiff contacts, it senses increasing resistance to penetration, and the blow counts will increase markedly, even though sampling in soft material.

sampler that uses brass rings or 6-in. (15.2-cm)-long brass liners inside the sample barrel to allow easy removal of soil samples back to the lab, where they can be extruded and evaluated. California routinely employs two different sized samplers, both called “Modified California,” or “Mod Cal,” samplers. The smaller version has a D_o = 2.5 in. (6.35 cm) with D_i = 1.875 in. (4.76 cm) with brass liners. The larger version employs a D_o = 3.0 in. (7.62 cm) with D_i = 2.4 in. (6.1 cm).

If we apply Burmister’s energy-area equation to the larger Mod Cal sample barrel, with D_o = 3.0 in. and D_i = 2.4 in., the correction would be 0.65. The correction for the smaller Mod Cal barrel would be 0.77. This means that the “equivalent SPT N values” would be about 65–77 percent of those recorded with either of the Mod Cal samplers. This energy correction does not make a distinction between cohesive (i.e., clay) and noncohesive (i.e., sands and gravels) materials, which should affect the side friction of the sample barrel markedly. Nevertheless, there existed nothing better, so most professionals cited Burmister’s 1948 “correction” for adjusted blow counts

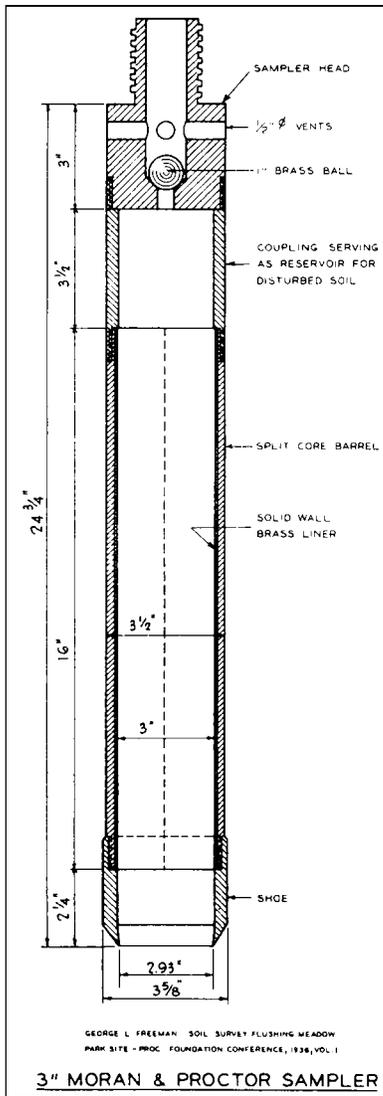


Figure 7. The M & P drive sampler was developed by Carlton Proctor of Moran & Proctor in the early 1930s. It was the preferred drive sampler in the New York City area because it recovers a much more disturbed sample than the smaller-diameter SPT.

recorded with larger-diameter drive samplers or for lower-energy hammers (de Mello, 1971).

Lacroix and Horn (1973) Correction

In 1973, Yves Lacroix and Harry Horn of Woodward-Clyde wrote an article titled “Direct Determination and Indirect Evaluation of Relative Density and Its Use on Earthwork Construction Projects,” published in ASTM STP 523. This ASTM conference had been convened in Los Angeles in June 1972, and this correction was subsequently adopted by many geotechnical engineers, especially when sampling materials with variable stiffness or in proximity to contacts between soft and stiff materials, as shown in Figure 6.

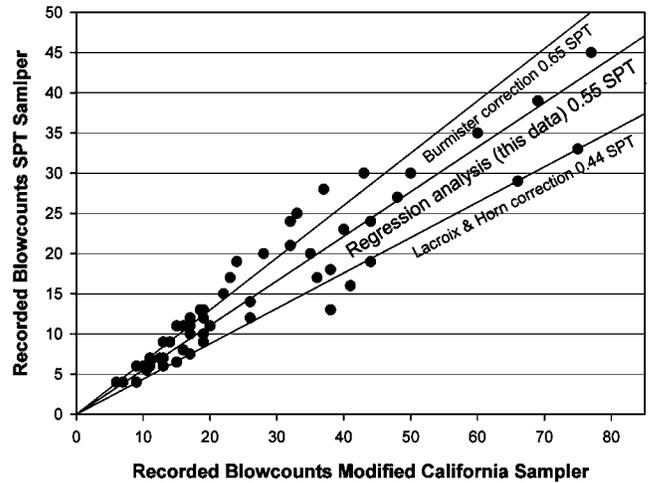


Figure 8. Comparison of uncorrected blow counts (N) for 57 drive samples using the Standard Penetration Test (SPT) and larger-diameter Modified California samplers at the same depths and locations in the San Francisco Bay area. A regression analysis of these data suggests a correction of 0.55, while the methods proposed by Burmister (1948) and LaCroix and Horn (1973) are plotted for comparison.

Lacroix and Horn (1973) proposed that nonstandard penetration resistance, N_1 , recorded using nonstandard split-spoon or solid cone, could be correlated with standard penetration resistance, N , by considering the differences in driving energy and distance of penetration. They reasoned that the energy required to drive the sampler or cone a given distance, or “depth” (L), was directly proportional to the square of the outside diameter (D) and the distance of penetration and inversely proportional to the energy per blow (weight of hammer multiplied by the height of drop, WH):

$$N = N_1(2 \text{ in.}/D_1)^2 \times 12 \text{ in.}/L_1 \times W_1/140 \text{ lbs} \times H_1/30 \text{ in.}$$

$$= \frac{2 N_1 W_1 H_1}{175 D^2 L_1}$$

When this correction is applied to the larger Mod Cal sampler ($D_o = 3.0$ in. and $D_i = 2.4$ in.), the predicted correction is reduced to $0.44 N_1$, where N_1 is the non-standard blow count recorded during driving of the larger Mod Cal sampler. The predicted correction for the smaller Mod Cal sampler would be $0.64 N_1$. The depth of penetration (L_1) in the denominator is the distance the sampler was advanced during the sampling round (typically, 18 in. [45.7 cm]). In most instances, this is given as the last 12 in. (30.5 cm) of an 18-in. (45.7-cm) sampling round (so, $L_1 = 12$ in. [30.5 cm]).

The Lacroix and Horn correction gives a more conservative estimate of the corrected SPT blow counts than by Burmister’s (1948) energy correlation (Figure 8). The LaCroix and Horn correction appears most valid when sampling is undertaken within 5–10 sample barrel

diameters (15–30 in. [38–76 cm]) of contacts with stiffer horizons (such as silty or clayey materials above cemented sands or gravels). The stiffness of the base layer causes the blow counts to become elevated because the tip of the sampler senses the stiffer mass beneath its cutting tip.

Side-by-Side Correlations

Between 1991 and 1996, the author supervised 57 side-by-side sampling rounds using the SPT and 3.0-in. Mod Cal samplers in residual soils, colluvium, and Tertiary-age sediments in northern and southern California to estimate the correction factor that seemed most appropriate under general conditions of use. These tests were carried out using the same drill rigs and crews, recording SPT and Mod Cal blow counts at depths of 3.5–33.5 ft (1–10 m). A linear regression analysis of the data points shown in Figure 8 gives a correction factor of 0.553. This would be applied to raw Mod Cal blow counts (N_1) for the larger Mod Cal sampler ($D_o = 3.0$ in. [7.62 cm]). Note how the regression analyzed figure falls between the Burmister and the LaCroix and Horn predictions, also shown on Figure 8. The data become increasingly dispersed with depth and increasing blow counts, likely because of increasing density, confinement, penetration into weathered bedrock materials (which are highly variable) and length of drill rods. No attempt was made to differentiate between cohesive and noncohesive materials. In addition, none of these data were collected in relatively clean sands, for which much emphasis has been placed in SPT correlations over the past two decades because of their recognized susceptibility to liquefaction and strength loss.

Standardized SPT Corrections

The SPT data can be corrected for a number of site-specific factors to improve its repeatability. Burmister's energy correction assumed that the hammer percussion system was 100 percent efficient (a 140-lb hammer dropping 30 in. = 4,200 ft-lbs raw input energy). In Skempton (1986), the procedures for determining a standardized blow count were presented, allowing hammers of varying efficiency to be accounted for. This corrected blow count is referred to as " N_{60} " because the original SPT hammer had about 60 percent efficiency, being comprised of a donut hammer, a smooth cathead, and worn hawser rope, and this is the "standard" to which other blow-count values are compared. Trip release hammers and safety hammers typically exhibit greater energy ratios (ER) than 60 percent (Skempton, 1986). N_{60} is given as

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.60}$$

where N_{60} is the SPT N-value corrected for field procedures and apparatus, E_m is the hammer efficiency, C_B is the borehole diameter correction, C_S is the sample

barrel correction, C_R is the rod length correction, and N is the raw SPT N-value recorded in the field. Skempton (1986) created a chart for estimating the appropriate values of C_B , C_S , and C_R . Clayton (1990) provided an expanded list of SPT hammer efficiencies (E_m).

In 1969, D'Appolonia et al. (1969) presented conclusive evidence that confinement increased with depth in sands, and this, along with stress history, affected SPT penetration noticeably (Horn, 2000). Gibbs and Holtz (1957) had presented a method for correcting SPT blow counts for a "standard effective overburden" pressure, N' , so that blow counts in similar materials at different depths could be compared. They defined $N' = C_N N$, where N is the uncorrected SPT blow-count value taken in the ground and C_N is a correction factor. The standard effective overburden pressure was given as an effective stress, σ'_v (overburden confinement). The correction factor C_N is taken as the reciprocal of the square root of σ'_v .

The Gibbs and Holtz overburden correction was found lacking when applied to situations where samples were taken from near the bottom of uniform soil deposits, which exhibit higher blow counts because the sampler senses the stiffer material lying below (Figure 6). Liao and Whitman (1986) presented the overburden correction to Skempton's energy-corrected value (N_{60}), known as $(N_1)_{60}$, to account for increasing confinement with depth. The corrected $(N_1)_{60}$ blow count was given as

$$(N_1)_{60} = N_{60} \frac{\sqrt{2,000 \text{ psf}}}{\sigma'_v}$$

where σ'_v is vertical effective stress where the sample was recovered.

Robertson and Wride (1997) have modified Skempton's chart and added additional correction factors to those proposed by Liao and Whitman (1986). This chart is reproduced in Table 3. The overburden stress corrected blow count, $(N_1)_{60}$, provides a consistent reference value for penetration resistance. This has become the industry standard in assessments of liquefaction susceptibility (Youd and Idriss, 1997). Robertson and Wride (1997) defined $(N_1)_{60}$ as

$$(N_1)_{60} = N C_N C_E C_B C_R C_S$$

where N is the raw SPT blow-count value, $C_N = (P_a / \sigma'_{vo})^{0.5}$ (with the restriction that $C_N \leq 2$) is the correction for effective overburden stress (Liao and Whitman, 1986), P_a is a reference pressure of 100 kPa, σ'_{vo} is the vertical effective stress, $C_E = ER/60\%$ is the correction to account for rod energy, ER is the actual energy ratio of the drill rig used in percent, C_B is a correction for borehole diameter, C_R is a correction for length of the drill rod, and C_S is a correction for the sampling method.

Table 3. Recommended corrections for Standard Penetration Test (SPT) blow-count values, taken from Robertson and Wride (1997), as modified from Skempton (1986).

Factor	Equipment Variable	Term	Correction
Overburden pressure		C_N	$(Pa/\sigma'_{vo})^{0.5}$ but $C_N \leq 2$
Energy ratio	Donut hammer	C_E	0.5–1.0
	Safety hammer		0.7–1.2
	Automatic hammer		0.8–1.5
Borehole diameter	65–115 mm	C_B	1.0
	150 mm		1.05
	200 mm		1.15
Rod length	3–4 m	C_R	0.75
	4–6 m		0.85
	6–10 m		0.95
	10–30 m		1.0
	>30 m		<1.0
Sampling method	Standard sampler	C_s	1.0
	Sampler without liners		1.1–1.3

Since 1986, published correlations have increasingly used corrected $(N_1)_{60}$ values with other parameters, such as relative density and angle of internal friction. $(N_1)_{60}$ values are useful in assessments of liquefaction potential, although the most important factor appears to be how the borehole is stabilized. Mud rotary borings have been found to be vastly superior to hollow stem auger borings in loose saturated sands (Holtz, 2005).

Drive Sample Disturbance

Thin-wall samplers are defined as those with a wall thickness less than 2.5 percent of the diameter, such as Shelby tubes. Drive samples less than 2.5 in. (6.35 cm) in diameter should be regarded as “disturbed” and their reported moisture and bulk density values judged accordingly. Drive samples of 2.5 in. (6.35 cm) or greater are also disturbed, but less than SPT samples.

When drive samplers are driven, under each blow the sampling tube advances downward, then rebounds slightly. This upward rebound action stresses the soil at the bottom of the sampler in tension and often causes separations, as shown in Figure 9. This induced tension creates a series of tensile fractures/discontinuities between zones of compression. The contrast shows up well on some X-rays, as shown in Figure 10. This is why most experienced geotechnical engineers favor pushed samples over driven samples (discussed in the following section). The exception to this premise is when sampling fibrous soils like peat. In such cases, the driving process is favored because there is a need to sever the fibers by driving a sharp edge; pushing would deform the peat to much larger strains before cutting. Trimming samples of peat is best done with a double-blade vibrating electric knife that shears the fibers (Briaud, 2005).

Special care should be exercised when taking drive



Figure 9. Five-inch (12.7-cm)-diameter drive sample of soil in an embankment dam recovered using a hollow stem flight auger drill rig. Note disturbance caused by multiple tensile separations. Image courtesy of Jeffrey A. Farrar, U.S. Bureau of Reclamation.

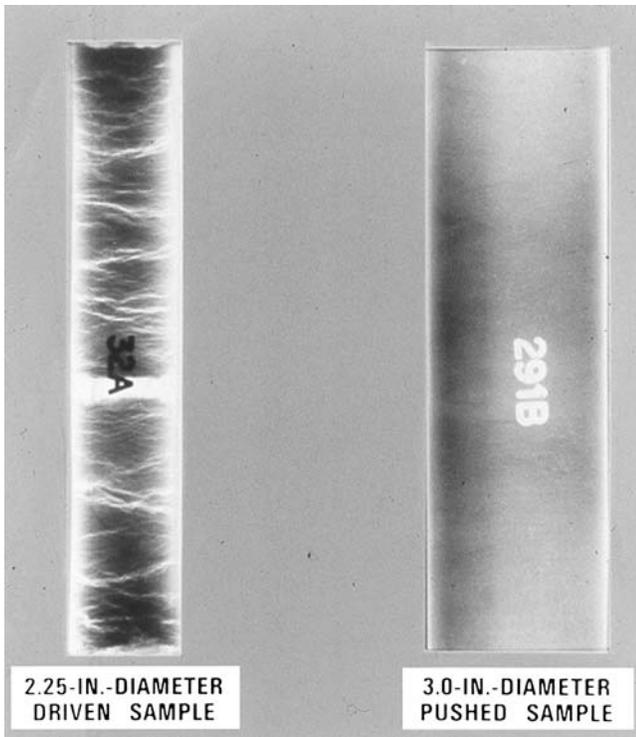


Figure 10. X-rays contrasting fabric of soils samples recovered using drive sampler (at left) and thin-wall piston sampler (at right). Tensile fractures are common in drive samples through stiff soils with low tensile strength. Images courtesy of Fugro-McClelland, Inc., via Jean-Louis Briaud.

samples in the rooted zone of natural slopes, commonly in the upper 6.5 ft (2 m). This is because the rooted zone is typically of lower density because of root action and creep (Burmister, 1936). When advancing drive samples within the upper 6.5 ft (2 m) of a native slope, it is advisable to mark the Kelly bar with chalk or crayon and compare the distance advanced during the sampling round with the actual thickness of the recovered soil taken from the sample barrel sleeves. A careful comparison usually shows that the sample has been densified during the sampling process, often leading to erroneous conclusions about soil strength in these upper zones.

Sample disturbance has also been described by the area ratio, A_r , originally described by Mohr (1936) and Hvorslev (1940); A_r was defined by Hvorslev (1949) as

$$A_r(\%) = \frac{(D_o)^2 - (D_i)^2}{(D_i)^2} \times 100$$

Shelby Tube Sampler

A drive sample can be considered undisturbed if the area ratio is less than or equal to 10 percent. In 1936, Harry Mohr developed the “Shelby tubing” or “thin-wall sampler” in response to a request made by Professor Arthur Casagrande at Harvard University. Casagrande

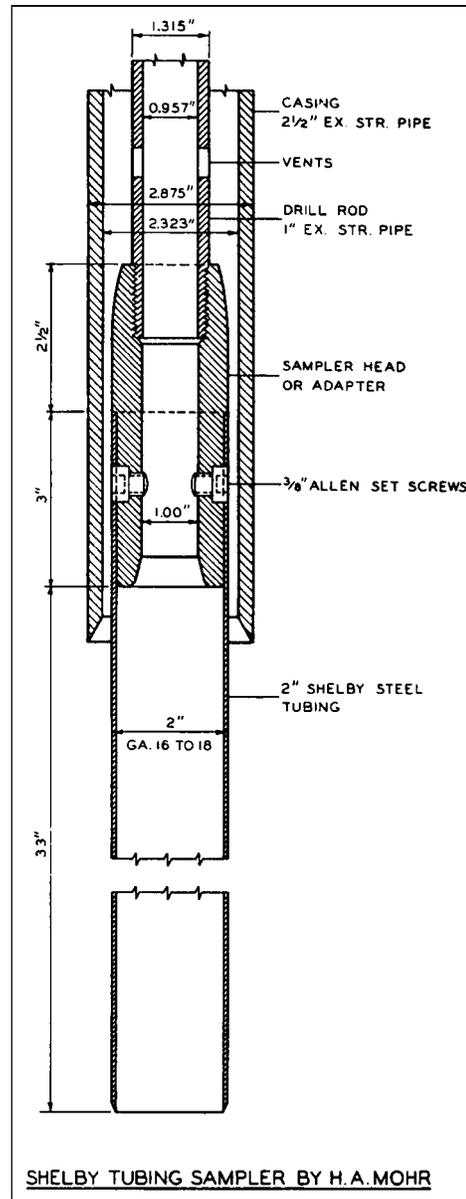


Figure 11. Original concept for Shelby tube thin-wall piston sampler, developed by Harry A. Mohr in 1936. Modern Shelby tubes are 3.0-in. (7.6-cm) diameter.

wanted a less disturbed sample within the standard 2½-in. (6.35-cm) casing size then employed in most exploratory borings around Boston. The term “Shelby tubing” was a trade name for hard-drawn, seamless steel tubing, manufactured by the National Tube Company of Lorain, Ohio, and was available nationwide (because National Tubing was owned by US Steel). Shelby tubes have an outside diameter of 3.0 in. (7.6 cm) and use 16-gauge (0.0578-in.) to 18-gauge (0.0451-in.) wall thickness tubes advancing a 36-in. (91.4-cm) section of tubing ahead of the sampler, recovering a 33-in. (83.8-cm)-long sample (see Figure 11).

The area ratio for an SPT is 110 percent, while that for a thin-wall Shelby tube is about 13.7 percent. Despite this, soft soils recovered in Shelby tubes are generally assumed to be more or less undisturbed, while all split-spoon samplers should be regarded as disturbed samples (Terzaghi, 1940). The dimensions of a typical Shelby tube sampler are shown in Figure 11. The impact of drive sampling disturbance on laboratory indices was recognized early on and summarized by Philip Rutledge in his research with Casagrande at Harvard (Rutledge, 1944). Rutledge advised that caution should be employed when performing strength and compressibility tests on drive samples because of sample disturbance and densification.

Soil behavior is greatly influenced by sampling disturbance. Nevertheless, disturbed samples may be adequate for indices tests, such as Atterberg limits and grain size distribution, but slightly more suspect for bulk density and water content (because of densification).

A great many piston and drive samplers of larger than 4-in. (10.2-cm) diameter were developed by various workers beginning in the late 1930s in an attempt to recover less disturbed samples of compressible materials, such as the Boston Blue Clay and soft peaty soils in the lower Mississippi Valley. These alternative samplers are summarized in Hvorslev (1949) and Mohr (1962).

CONE PENETROMETER SOUNDINGS

Introduction

The standardized cone-penetrometer test (CPT) involves pushing a 1.41-in. (3.58-cm)-diameter 60° cone (Figures 12 and 13) through the underlying ground at a rate of 0.40–0.78 in./s (1–2 cm/s). The CPT soundings can be very effective in site characterization, especially sites with discrete stratigraphic horizons or discontinuous lenses. The methodology was adopted by ASTM (1994) as Test Designation D 3441 in 1974. The CPT is a valuable method of assessing subsurface stratigraphy associated with soft materials, discontinuous lenses, organic materials (peat), potentially liquefiable materials (silt, sands, and granule gravel), and landslides. Cone rigs can usually penetrate normally consolidated soils and colluvium but have also been employed to characterize weathered Quaternary- and Tertiary-age strata. Cemented or unweathered horizons, such as sandstone, conglomerate, or massive volcanic rock, can impede advancement of the probe, but the author has always been able to advance CPT cones in Tertiary-age sedimentary rocks. The cone is able to delineate discrete low strength horizons, easily missed in conventional (small-diameter) sampling programs. An example of a CPT electronic log through a bedrock landslide complex is shown in Figure 20, along with hand-drawn lithologic interpretations.

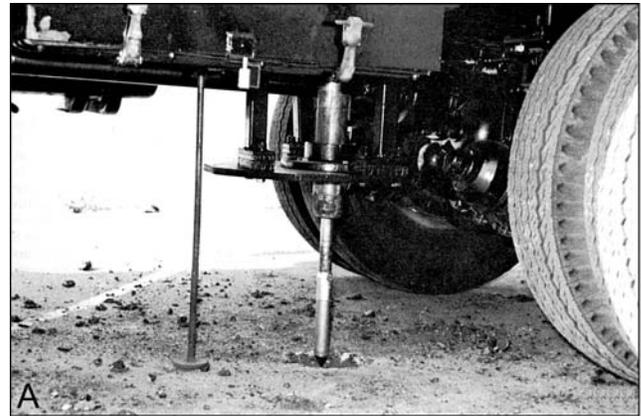
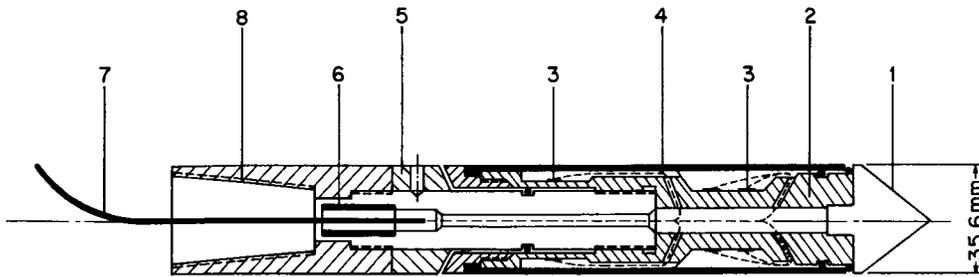


Figure 12. (A, left). Cone tip exposed beneath truck, just before being advanced into the ground at a rate between 1 and 2 cm/s. The cone has an area of either 10 or 15 cm² and is typically advanced in 1-m increments because the rods are of that length. (B, right) A Hogentogler Cone Penetrometer Test (CPT) rig operated by Ertec in Long Beach, CA. Truck-mounted rigs weigh about 18 tons and are capable of exerting considerable normal force on the advancing rods.

Most of the commercially available CPT rigs operate electronic friction cone and piezocone penetrometers, whose testing procedures are outlined in ASTM (1995), D 5778, adopted in 1995. These devices produce a computerized log of tip and sleeve resistance, the ratio between the two, induced pore pressure just behind the cone tip, pore pressure ratio (change in pore pressure divided by measured pressure), and lithologic interpretation of each 0.78-in. (2-cm) interval, which are continuously logged and printed out.

Tip Resistance

The tip resistance is measured by load cells located just behind the tapered cone (Figure 14). The tip resistance is theoretically related to undrained shear strength of a saturated cohesive material, while the sleeve friction is theoretically related to the friction of the



- 1 Conical point (10 cm²)
- 2 Load cell
- 3 Strain gages
- 4 Friction sleeve (150 cm²)
- 5 Adjustment ring
- 6 Waterproof bushing
- 7 Cable
- 8 Connection with rods

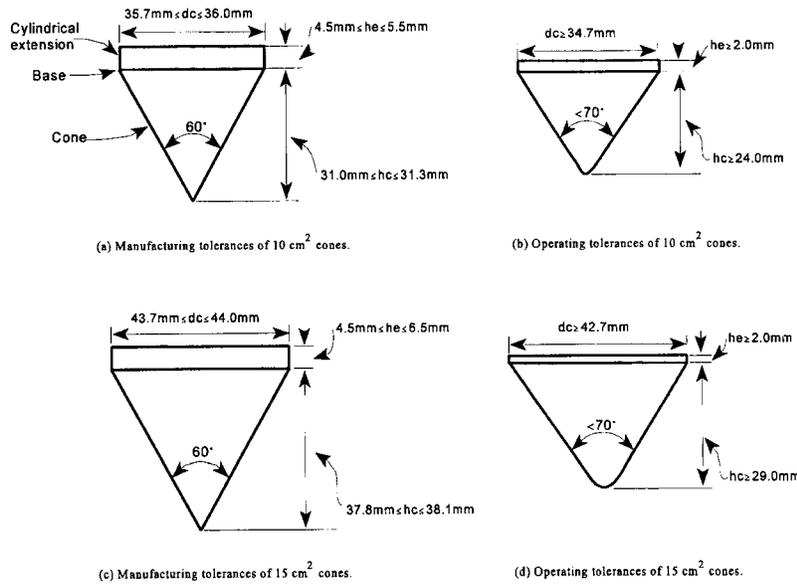
Electric Friction-Cone Penetrometer Tip

Figure 13. Manufacturing and operating tolerances of cones, taken from ASTM D 5778-95.

horizon being penetrated (Robertson and Campanella, 1986). The tapered cone head forces failure of the soil about 15 in. (38 cm) ahead of the 10-cm² tip, and the resistance is measured with an embedded load cell in tons/ft² (tsf [kg/cm²]).

Local Friction

The local friction is measured by tension load cells embedded in the sleeve for a distance of 4 in. (10.2 cm) behind the tip (Figures 13 and 15). They measure the



CONE BASE AREA	NOMINAL			TOLERANCE MANUFACTURED (OPERATIONS)		
	BASE DIAMETER	CONE HEIGHT	EXTENSION	dc	hc	he
cm ²	dc mm	hc mm	he mm	dc mm	hc mm	he mm
10	35.7	31.0	5.0	+0.3 - 0.0 (≥ 34.7)	+0.3 - 0.0 (≥ 24.0)	+0.0 - 0.5 (≥ 2.0)
15	43.7	37.8	5.0 - 6.0	+0.3 - 0.0 (≥ 42.7)	+0.3 - 0.0 (≥ 29.0)	+0.0 - 0.5 (≥ 2.0)

Manufacturing and Operating Tolerances of Cones (2)

Figure 14. Schematic section through an electric friction-cone penetrometer tip, taken from ASTM D 3441-94 (ASTM, 1994).

average skin friction as the probe is advanced through the soil. If cohesive soils are partially saturated, they may exert appreciable skin friction, complicating the programmed interpretation, which is based on behavior, not lithology.

Friction Ratio

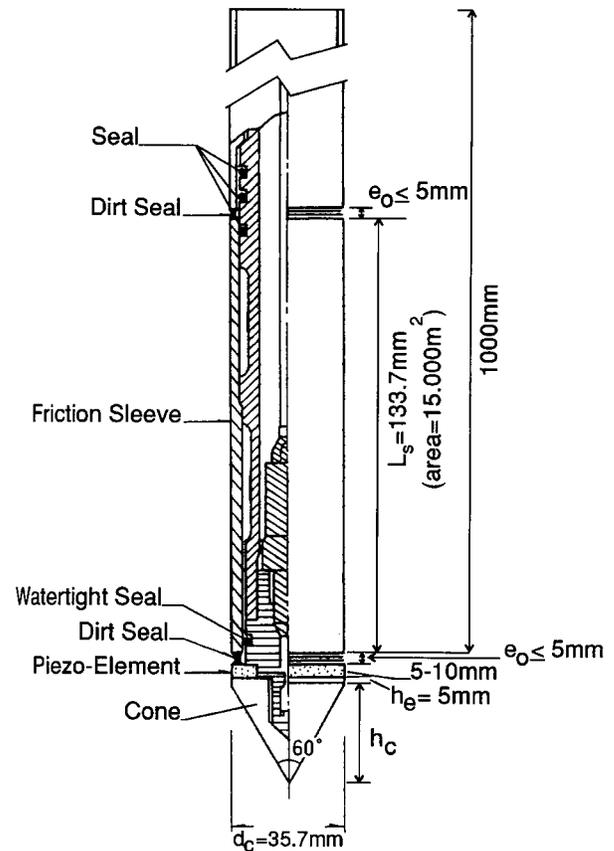
The friction ratio is the ratio of skin friction divided by the tip resistance (both in tsf), given in percent. It is used to classify the soil by its behavior or reaction to the cone being forced through the soil. High ratios generally indicate clayey materials (high c , low ϕ), while lower ratios are typical of sandy materials (or dry desiccated clays). Typical friction ratios are between 1 and 10 percent. The ratio seldom, if ever, exceeds 15 percent, unless the cone is being pushed through organic debris, like wood. Sands and highly sensitive clays are generally identified by exhibiting ratios <1 percent.

Pore Pressure

Piezocoones also measure *in situ* pore pressure (in psi [kPa]) in either dynamic (while advancing the cone) or static (holding the cone stationary) modes. Piezocoones employ a porous plastic insert just behind the tapered head that is made of hydrophilic polypropylene, with a nominal particle size of 120 microns (Figure 15). The piezocell must be saturated with glycerin or peanut oil prior to its employment. The filter permeability is about 0.01 cm/s. When using the cone to penetrate dense layers, such as cemented siltstone, sandstone, or conglomerate, the piezo-filter element can become compressed, thereby inducing high positive pore pressures. However, the plastic filters do not exhibit this tendency, though they do become brittle with time and may need to be replaced periodically. In stiff overconsolidated clays, the pore pressure gradient around the cone may be quite high, although the pore pressure itself is usually negative. There is a danger of desaturation and believing erroneous pore-water pressure measurements when characterizing such materials. This pore pressure gradient often results in dissipations recorded behind the CPT tip that initially increase before decreasing to the equilibrium value.

Differential Pore Pressure

The differential pore pressure ratio is the ratio between the generated pore pressure and the cone resistance at a given depth level. It is used to aid in soil classification according to the Unified Soil Classification System (USCS) because the sleeve friction measurement is inherently inaccurate, being subject to rate of advance. When the cone penetrates dense materials like sand, the sand dilates, and the pore pressure drops. In clayey



Example of a Reference Penetrometer With a Fixed Cone and With Friction Sleeve

Figure 15. Schematic section through a piezocone head, showing the piezo-element and friction sleeve. Taken from ASTM D 5778-95.

materials, high pore pressures may be induced, advancing the cone head at the typical rate of 0.787 in./s (2 cm/s), but in normally consolidated soils this may not occur. If transient pore pressures are being recorded that seem nonhydrostatic, experienced operators will ask that the penetration be halted and allowed at least 5 minutes to equilibrate so that a quasi-static pore pressure reading can be recorded. Sometimes equilibration can take 10–30 minutes, depending on the soil type. In practice, experienced operators try to stop the advance and take pore pressure measurements in recognized aquifers and just above or adjacent to indicated aquitards.

Temperature Sensor

A significant advantage of the electric cone is the temperature sensor. This has been found to be very useful in assessing the precise position of the zone or zones of saturation, which is of great import in slope stability and consolidation studies. A temperature shift of about 6°F (3.3°C) is common at the groundwater interface, even

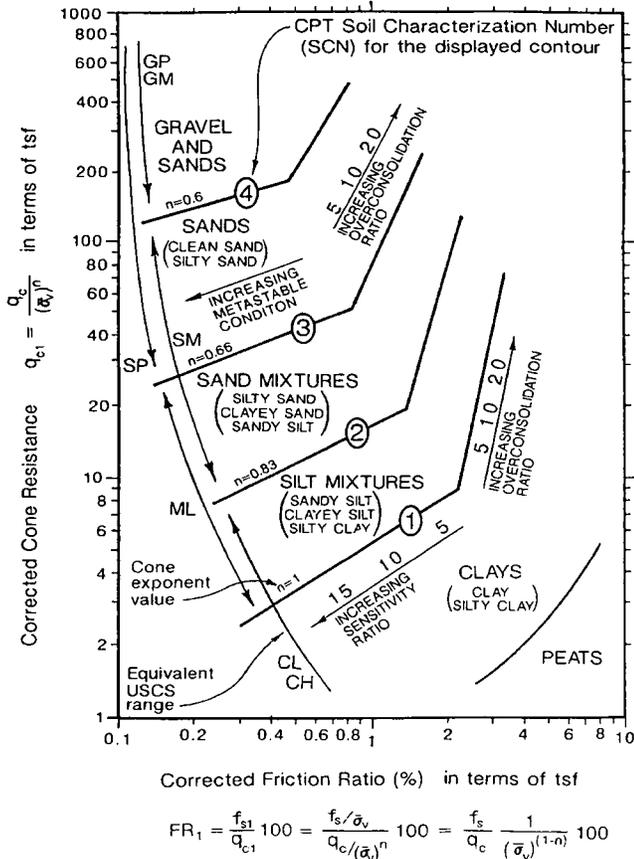


Figure 16. Cone Penetrometer Test (CPT) classification chart developed by Olsen (1988).

perched horizons within landslides. The temperature sensor is also crucial during calibration tests because the embedded strain gauges are very temperature sensitive.

Inclinometer

Most CPT cones are equipped with embedded inclinometers that detect and record the inclination of the advancing cone tip from vertical. This is an important aspect to monitor when advancing soundings in heterogeneous materials, such as old landfills, which may contain stiff inclusions that can easily deflect the advancing cone, which has a high slenderness ratio. If the cone is advanced beyond 90° tilt, it may not be recoverable.

Corrected Logs

Most CPT rigs are equipped with one or several automated interpretation programs, which arbitrarily classify 0.4-in. (1-cm) horizons according to the USCS. The most widely employed routine has been that orig-

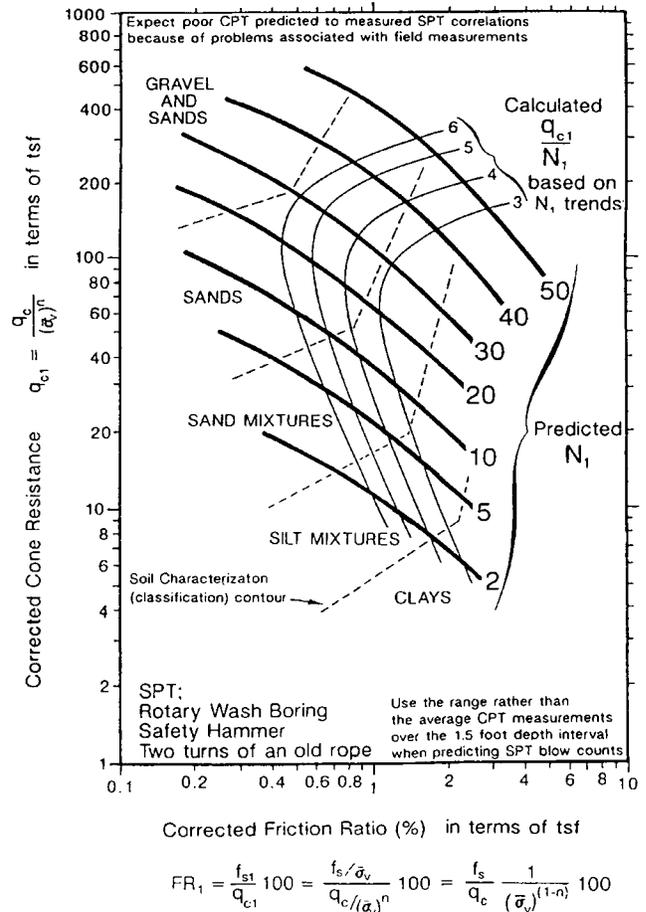


Figure 17. Cone Penetrometer Test (CPT) to Standard Penetration Test (SPT) conversion chart proposed by Olsen (1988).

inally developed by Robertson and Campanella (1986), available from Hogentogler & Co., of Gaithersburg, MD, or from the Natural Sciences and Engineering Research Council of Canada. A more refined CPT classification chart was developed by Olsen (1988), reproduced in Figure 16. It has been the author's experience that this chart is more accurate than those that preceded it. Most CPT interpretive programs employ the methods suggested in Campanella and Robertson (1988), Olsen (1988), Robertson (1990), or Olsen and Mitchell (1995).

The interpretation programs evaluate all the measured properties and classify the horizon according to its behavior (in lieu of petrology). For instance, when classifying a clayey material the interpretive programs consider undrained shear strength, tip resistance, and differential pore pressure. A high differential pore pressure is assumed diagnostic of more clayey materials.

Olsen (1988) also developed a useful chart that relates CPT data to equivalent SPT data, shown in Figure 17. This is useful when comparing or correlating subsurface data acquired using both methods on the same site.

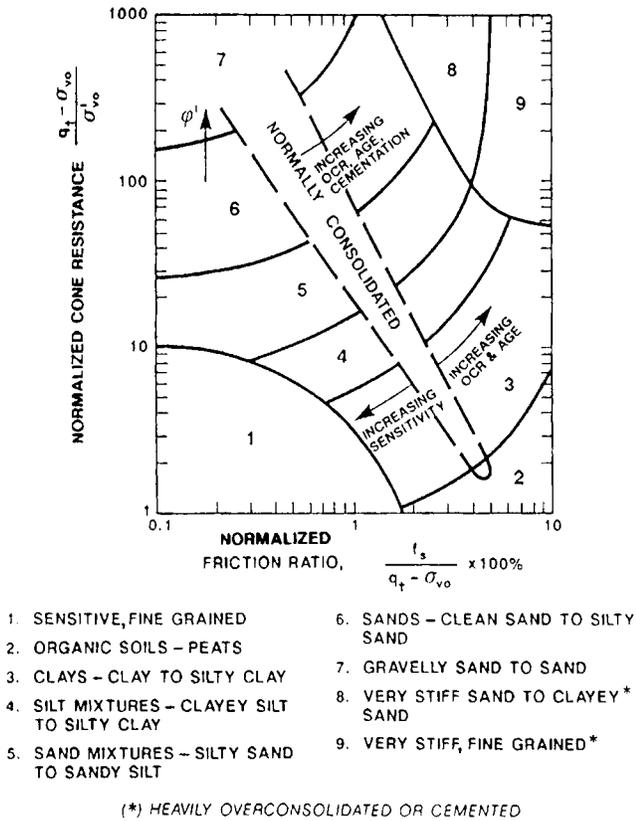


Figure 18. Normalized Cone Penetrometer Test (CPT) soil behavior type chart proposed by Robertson (1990).

Robertson (1990) developed a useful soil classification chart that estimates soil type based on CPT data, presented in Figure 18. The friction ratio (ratio of the CPT sleeve friction to the tip resistance) tends to increase with increasing fines content and plasticity. Robertson's classification scheme estimates grain characteristics directly from "filtered" CPT data and, though somewhat tedious, can be valuable when working with over-consolidated or sensitive materials.

Importance of "Ground-Truthing"

Like geophysical techniques, CPT soundings are most meaningful when "ground-truthed" with established lithologic horizons. The easiest method to ground-truthing CPT data is to advance a sounding next to a bucket auger or conventional boring from which subsurface samples are collected. In this way, the electronic "signature" of the sounding can be compared with the various lithologies already identified in the substory. This comparison can prove especially valuable in identifying potentially liquefiable materials and old landslide slip surfaces. Once the CPT sounding is ground-truthed, the CPT rig can crisscross the job site, commonly advancing 6-12 soundings in a single day.

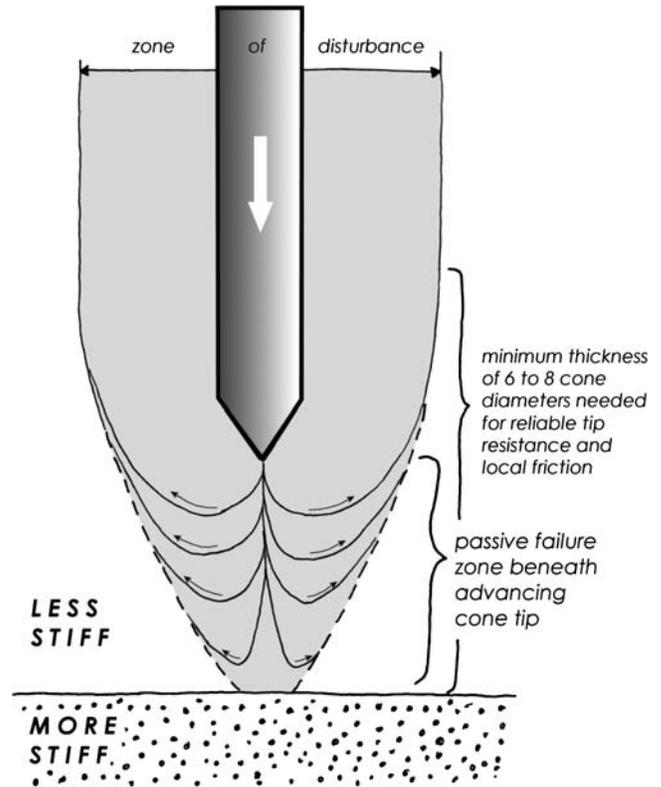


Figure 19. The tip of the cone penetrometer senses out ahead of itself as it induces a local bearing failure of the soil it passes through. The tip resistance recorded by the instrument is an average across this tip influence zone. Therefore, caution should be exercised when evaluating *in situ* strength parameters for horizons less than 14-28 in. (36-72 cm) thick, such as landslide slip surfaces.

This allows for an expanded data set and thus for superior three-dimensional characterization of the site under evaluation and construction of reliable geologic cross sections and fence diagrams.

Notes of Caution

Some notes of caution are advised when applying the CPT method to evaluating discrete low strength horizons or partings, such as landslide slip surfaces. The 60° tip of the cone forces a passive failure of the ground in front of the advancing tip. The instrumented tip senses soil resistance about 5-10 cone diameters ahead and behind the advancing tip, as shown schematically in Figure 19. This means that the tip resistance reported as "undrained shear strength" is actually an average value, taken over the zone within 7-14 in. (18-36 cm) of the 1.55-in.² (10-cm²) cone tip. For the smaller 1.55-in.² (10-cm²) electric cone (Figure 15), the minimum layer thickness to ensure full tip and skin friction response is somewhere between 14 and 28 in. (36 and 71 cm). If the tip penetrates low strength horizons less than this thickness, such as a landslide slip surface, the tip

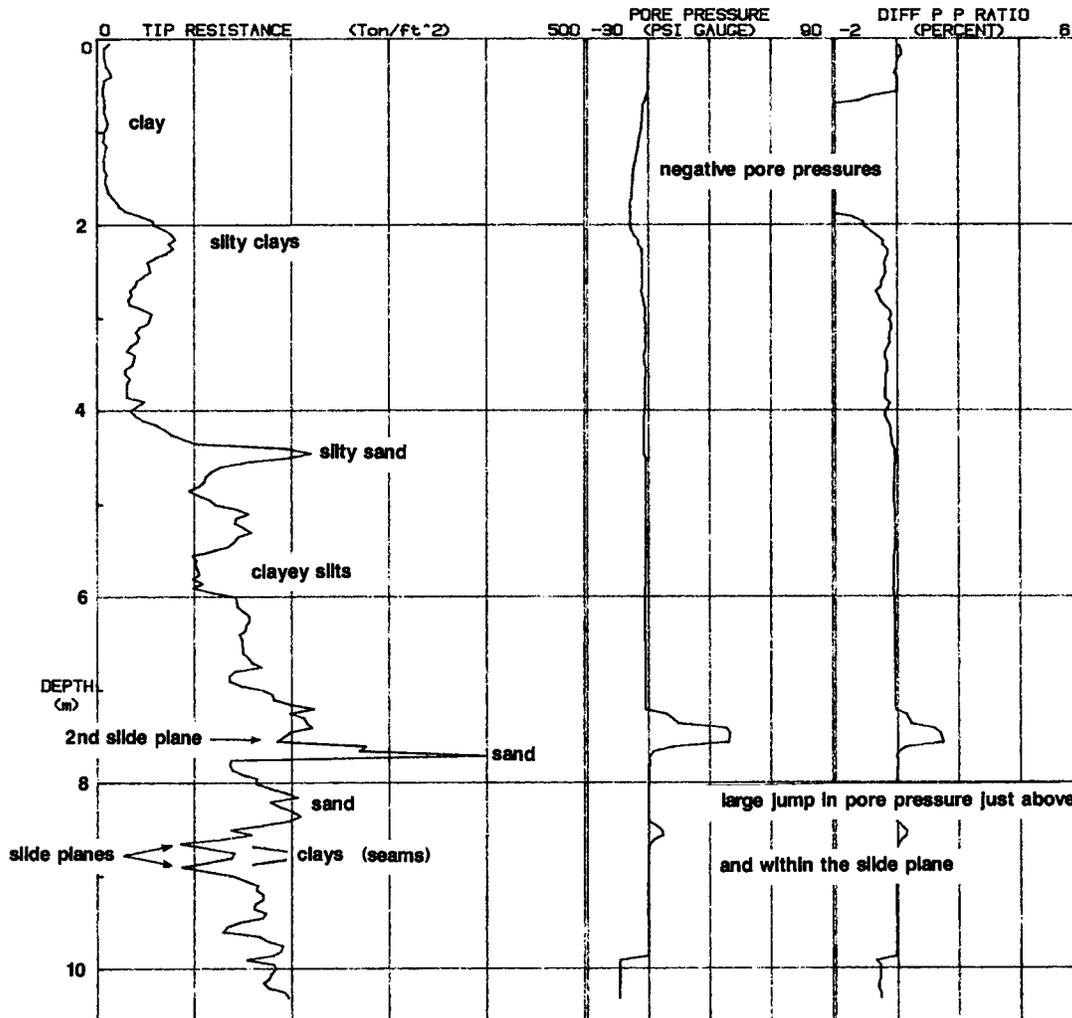


Figure 20. Cone Penetrometer Test (CPT) graphic log through a compound bedrock landslide, showing a series of slide planes encountered at depths of 7.5, 8.6, and 8.9 m, subsequently confirmed by excavation. Note the corresponding jumps in pore pressure with two of the slip surfaces. The tip resistance values cannot be used for back-analysis because the horizons were not of sufficient thickness to allow a full sensing by the cone. Despite this shortcoming, CPT soundings are valuable in that they can identify discrete low strength horizons often responsible for the landslide. Sounding taken in Orinda, CA.

resistance reported on the CPT log may be much higher than actually exists on the discrete plane of slippage, which may be only 0.5–4 in. (1–10 cm) thick. So, these thin layers are “sensed” by the CPT but not “fully sensed” in that the values of tip resistance and skin friction will be artificially high.

Another problem with the CPT method is that cone soundings advanced through desiccated clay will often be interpreted as sand or silt mixtures (by the computerized lithologic interpretation routine) because of recorded sleeve friction. The opposite problem occurs when reporting SPT blow counts after advancing drive samples through clayey horizons. Penetration resistance in desiccated clays depends on degree of saturation and load history. The SPT works best in granular materials but tends to give erroneous answers in soft clays. Blow counts in desiccated clayey materials must be regarded

with some degree of skepticism, as they may shift dramatically on later absorption of moisture.

Sample CPT Logs

The attached logs (Figures 20 to 22) display some of the distinguishing traits of electric friction cone soundings, which can detect subtle changes in stiffness that are characteristic of discrete horizons. Figure 20 shows raw data sensed by the cone as it is pushed through the ground. This includes friction ratio, local friction, tip resistance, pore pressure, differential pore pressure ratio, and an interpreted lithologic profile (often printed out on a separate sheet, depending on which interpretation program is being utilized). Figure 21 illustrates how compacted fill lifts can be discerned with CPT soundings, while Figure 22 shows how these stiffness variances can

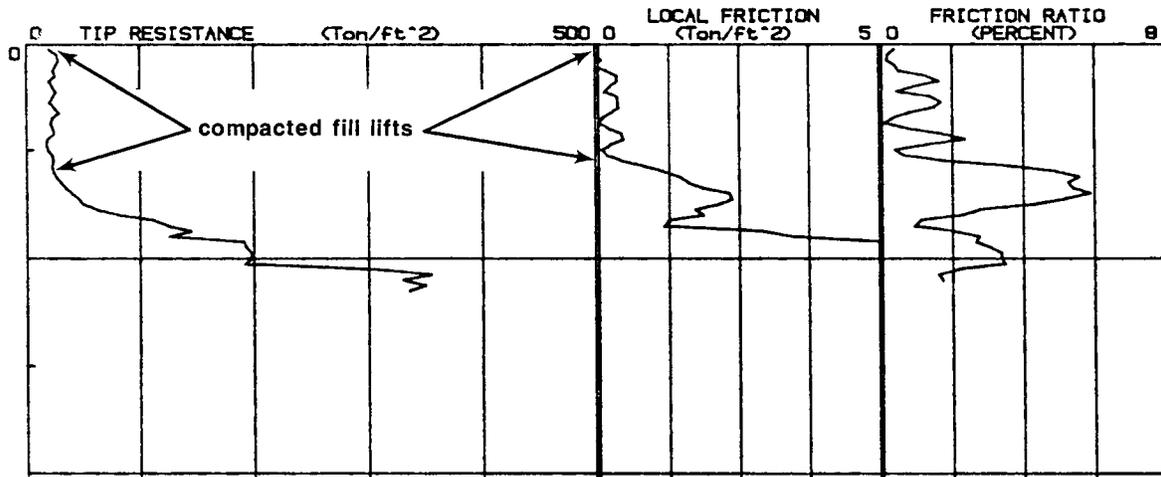


Figure 21. CPT soundings can detect seemingly minute details, such as these fill lifts in an engineered embankment, confirmed by subsequent excavation. Note how easily discerned these are in local (skin) friction but not in tip resistance. Sounding taken in Danville, CA.

subsequently change through normal processes of aging, such as saturation (swell) and desiccation (shrinkage) cycles. The CPT procedure can be valuable in judging changes in soil stiffness and behavior over time, especially beneath pavements and irrigated areas.

CONCLUSIONS

Engineering geologists are most often entrusted with characterizing difficult sites for subsequent analysis by geotechnical engineers. Our ability to develop the most effective program of exploration, sampling, and testing is built on each person's unique pedigree of experience.

The most effective means of characterizing complex sites includes a thorough background work-up on the area under investigation, followed by a well-conceived program of subsurface exploration that commonly includes small-diameter borings. Cone penetrometer soundings are being employed with increasing regularity, especially in evaluation of soil liquefaction potential (Robertson and Wride, 1997; Youd and Idriss, 1997; Martin and Lew, 1999; and Lew, 2001). The engineering geologist should consider employing both techniques whenever possible because each has slight advantages over the other, but they are most powerful when combined on the same sites. The SPT allows a firsthand look at subsurface materials

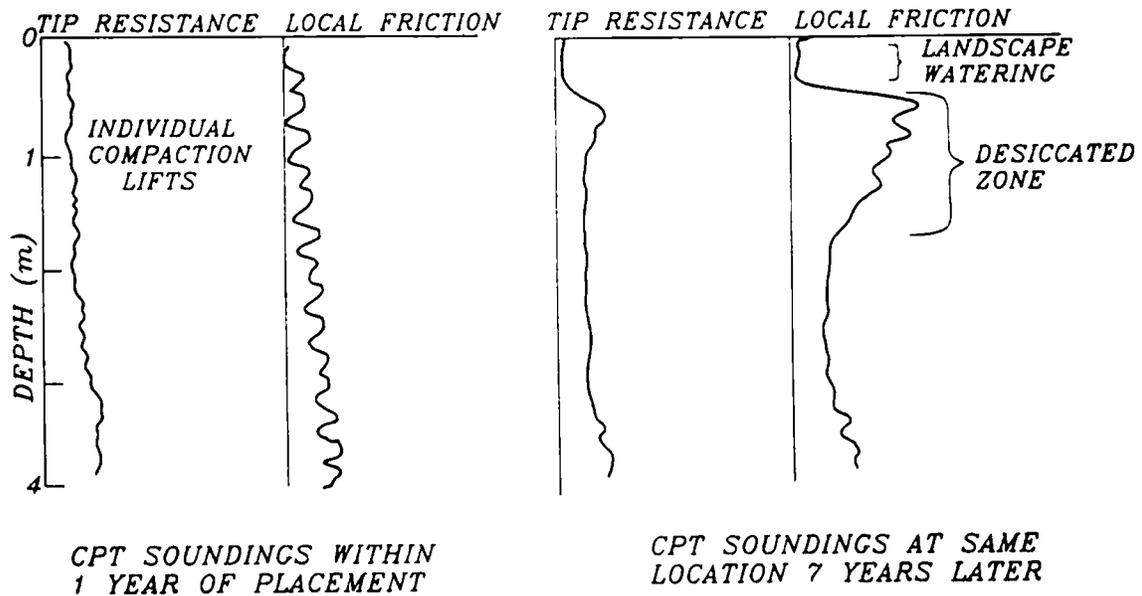


Figure 22. Comparisons of cone penetrometer soundings in expansive silty clay of an engineered fill embankment taken 7 years apart. Note how the initial effects of compaction lifts were erased by moisture absorption, desiccation, and swell over a 7-year period. Sounding taken in Blackhawk, CA.

(the CPT does not) and can provide crucial ground-truthing as to the type of subsurface material, especially cohesionless materials that include fines.

The CPT procedure is often capable of detecting discrete horizons that would normally be missed using drive samples at specific depth intervals. However, the absolute values of tip resistance, skin (local) friction and pore pressure must be evaluated with a great degree of judgment because these are simply measurements of material behavior during penetration of the cone head, subject to limitations of loading rate, geometry, pore pressures, and changes in hydrologic regimen discussed earlier.

Concluding Remarks

Sites underlain by natural geologic structures should not be approached like a foundation investigation, taking subsurface samples at fixed intervals; they must be attacked individually, with a focused program of exploration that employs a realistic working model of the site's evolution, focused on validating the assumptions used to construct such models. The greatest danger we face as a profession is the inherent tendency to make the site exploration fit our preconceived notions of site conditions, then employ insufficient exploration to confirm or deny such assumptions.

ACKNOWLEDGMENTS

The author is indebted to many individuals who shared their knowledge of the evolution of the Standard Penetration and Cone Penetration Tests. The history of the SPT procedures came from interviews with Ralph B. Peck, who worked with Karl Terzaghi and Harry Mohr. Additional insights on the Burmister and LaCroix/Horn corrections came from interviews with Harry Horn, who was a graduate student under Burmister at Columbia. Alec Skempton provided information on the evolution of energy corrections for the SPT procedure. The history of the SPT and CPT procedures evolved from conversations with Donald P. Coduto, William L. Houston, Richard L. Meehan, James K. Mitchell, Daniel Pradel, Richard S. Olsen, and Robert B. Rogers. Jean-Louis Briaud, Jeff Ferrar, and John Walkinshaw provided several of the images used in this paper. Kevin James and Jae-won Chung drafted several of the excellent figures, while Conor Watkins scanned photographs. Their technical support was invaluable. Other aspects and procedural checks mentioned in the article evolved from the author's experience using these techniques to characterize difficult sites. Lastly, the author wishes to thank Robert D. Holtz and William Gates for their detailed review of this article and many helpful comments and suggestions.

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