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«A critical review on soil penetration tests»

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«ΠΟΛΙΤΙΚΗ ΜΗΧΑΝΙΚΗ ΚΑΙ ΑΕΙΦΟΡΟΣ ΣΧΕΔΙΑΣΜΟΣ»

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Abstract

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 variety in geological conditions and range of design and construction problems make the subject complex. This has led to the development of a considerable number of different in situ test techniques. This dissertation offers a critical review on the most commonly used, in civil engineering practice, soil penetration tests, namely, the Standard Penetration Test (SPT) and Cone Penetrometer Test (CPT). Both procedures have evolved over a period of 100 and 70 years, respectively, and have been adopted as ASTM and EN 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.

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CHAPTER 1 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 variety in geological conditions and range of design and construction problems make the subject complex. This has led to the development of a considerable number of different in situ test techniques.

Mitchell et al. (1978) listed the following four main reasons for a growing interest in the use of in situ testing techniques:

- (1) Their ability to determine properties of soils, such as sands and offshore deposits that cannot be easily sampled in the undisturbed state.
- (2) Their ability to test a larger volume of soil than can conveniently be tested in the laboratory.
- (3) Their ability to avoid some of the difficulties of laboratory testing, such as sample disturbance and the proper simulation of in situ stresses, temperature, and chemical and biological environments.
- (4) The increased cost effectiveness of an exploration and testing program using these in situ methods.

Correct selection of the appropriate in situ test technique, however, is governed by the following main factors: geological conditions, project requirements, type of construction, and method of analysis intended for design.

This dissertation discusses the present status of in situ testing and its application to geotechnical engineering and mainly foundation engineering.

CHAPTER 2 SOIL EXPLORATION

2.1 General

A fairly accurate assessment of the characteristics and engineering properties of the soils at a site is essential for proper design and successful construction of any structure at the site. The field and laboratory investigations required to obtain the necessary data for the soils for this purpose are collectively called *soil exploration*.

The choice of the foundation and its depth, the bearing capacity, settlement analysis and such other important aspects depend very much upon the various engineering properties of the foundation soils involved.

Soil exploration may be needed not only for the design and construction of new structures, but also for deciding upon remedial measures if a structure shows signs of distress after construction. The design and construction of highway and airport pavements will also depend upon the characteristics of the soil strata upon which they are to be aligned.

The primary objectives of soil exploration are:

1. Determination of the nature of the deposits of soil,
2. Determination of the depth and thickness of the various soil strata and their extent in the horizontal direction,
3. The location of groundwater and fluctuations in GWT,
4. Obtaining soil and rock samples from the various strata,
5. The determination of the engineering properties of the soil and rock strata that affect
6. the performance of the structure, and
7. Determination of the *in-situ* properties by performing field tests.

2.2 Soil Exploration

The subsoil exploration should enable the engineer to draw the soil profile indicating the sequence of the strata and the properties of the soils involved.

In general, the methods available for soil exploration may be classified as follows:

1. Direct methods ... Test pits, trial pits or trenches
2. Semi-direct methods ... Borings
3. Indirect methods ... Soundings or penetration tests and geophysical methods

In an exploratory program, one or more of these methods may be used to yield the desired information.

2.2.1 Test Pits

Test pits or trenches are open type or accessible exploratory methods. Soils can be inspected in their natural condition. The necessary soils samples may be obtained by sampling techniques and used for ascertaining strength and other engineering properties by appropriate laboratory tests.

Test pits will also be useful for conducting field tests such as the plate-bearing test.

Test pits are considered suitable only for small depths—up to 3m; the cost of these increases rapidly with depth. For greater depths, especially in pervious soils, lateral supports or bracing of the excavations will be necessary. Ground water table may also be encountered and may have to be lowered.

Hence, test pits are usually made only for supplementing other methods or for minor structures.

2.2.2 Boring

Making or drilling bore holes into the ground with a view to obtaining soil or rock samples from specified or known depths is called 'boring'.

The common methods of advancing bore holes are:

1. Auger boring
2. Auger and shell boring
3. Wash boring
4. Percussion drilling
5. Rotary drilling strata

Auger Boring

'Soil auger' is a device that is useful for advancing a bore hole into the ground. Augers may be hand-operated or power-driven; the former are used for relatively small depths (less than 3 to 5 m), while the latter are used for greater depths. The soil auger is advanced by rotating it while pressing it into the soil at the same time. It is used primarily in soils in which the borehole can be kept dry and unsupported. As soon as the auger gets filled with soil, it is taken out and the soil sample collected.

Two common types of augers, the post hole auger and the helical auger, are shown in figure 1.

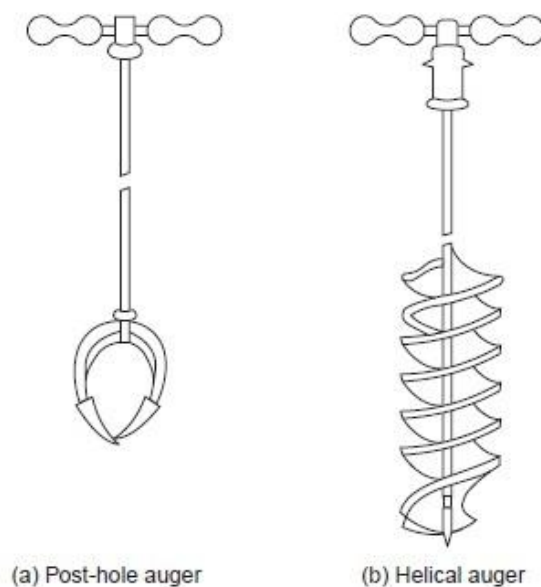


Fig. 1 Soil augers

Auger and Shell Boring

If the sides of the hole cannot remain unsupported, the soil is prevented from falling in by means of a pipe known as 'shell' or 'casing'. The casing is to be driven first and then the auger; whenever the casing is to be extended, the auger has to be withdrawn, this being an impediment to quick progress of the work.

An equipment called a 'boring rig' is employed for power-driven augers, which may be used up to 50 m depth (A hand rig may be sufficient for borings up to 25 m in depth). Casings may be used for sands or stiff clays. Soft rock or gravel can be broken by chisel bits attached to drill rods. Sand pumps are used in the case of sandy soils.

Wash Boring

Wash boring is commonly used for exploration below ground water table for which the auger method is unsuitable. This method may be used in all kinds of soils except those mixed with gravel and boulders. The set-up for wash boring is shown in Figure 2.

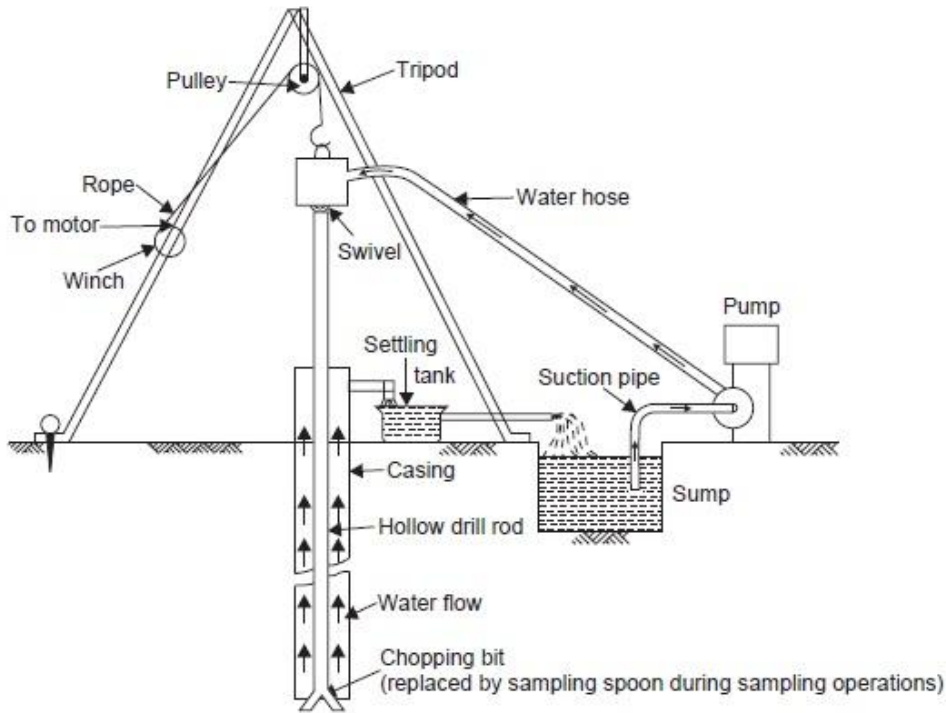


Fig. 2 Set-up for Wash Boring

Initially, the hole is advanced for a short depth by using an auger. A casing pipe is pushed in and driven with a drop weight. The driving may be with the aid of power. A hollow drill bit is screwed to a hollow drill rod connected to a rope passing over a pulley and supported by a tripod. Water jet under pressure is forced through the rod and the bit into the hole. This loosens the soil at the lower end and forces the soil-water suspension upwards along the annular surface between the rod and the side of the hole. This suspension is led to a settling tank where the soil particles settle while the water overflows into a sump. The water collected in the sump is used for circulation again.

The soil particles collected represent a very disturbed sample and is not very useful for the evaluation of the engineering properties. Wash borings are primarily used for advancing bore holes; whenever a soil sample is required, the chopping bit is to be replaced by a sampler. The change of the rate of progress and change of color of wash water indicate changes in soil strata.

Percussion Drilling

A heavy drill bit called 'churn bit' is suspended from a drill rod or a cable and is driven by repeated blows. Water is added to facilitate the breaking of stiff soil or rock. The slurry of the pulverized material is bailed out at intervals. The method cannot be used in loose sand and is slow in plastic clay.

The formation gets badly disturbed by impact.

Rotary Drilling

This method is fast in rock formations. A drill bit, fixed to the lower end of a drill rod, is rotated by power while being kept in firm contact with the hole. Drilling fluid or bentonite slurry is forced under pressure through the drill rod and it comes up bringing the cuttings to the surface. Even rock cores may be obtained by using suitable diamond drill bits. This method is not used in porous deposits as the consumption of drilling fluid would be prohibitively high.

2.2.3 Planning an Exploration Program

The planning of an exploration program depends upon the type and importance of the structure and the nature of the soil strata. The primary purpose the cost involved should be borne in mind while planning a program. The depth, thickness, extent, and composition of each of the strata, the depth of the rock, and the depth to the ground water table are important items sought to be determined by an exploration program. Further, approximate idea of the strength and compressibility of the strata is necessary to make preliminary estimates of the safety and expected settlement of the structure.

The planning should include a site plan of the area, a layout plan of proposed structures with column locations and expected loads and the location of bore holes and other field tests. A carefully planned program of boring and sampling is the crux of any exploration job. Resourceful and intelligent personnel trained in the principles of geology and geotechnical engineering are necessary.

The two important aspects of a boring program are 'spacing of borings' and 'depth of borings'.

2.2.4 Spacing of Borings

The spacing of borings, or the number of borings for a project, is related to the type, size, and weight of the proposed structure, to the extent of variation in soil conditions that permit safe interpolation between borings, to the funds available, and possibly to the stipulations of a local building code.

It is impossible to determine the spacing of borings before an investigation begins, since it depends on the uniformity of the soil deposit. Ordinarily a preliminary estimate of the spacing is made. Spacing is decreased if additional data are necessary and is increased if the thickness and depth of the different strata appear about the same in all the borings.

The following spacings are recommended in planning an exploration program:

<i>S.No.</i>	<i>Nature of the project</i>	<i>Spacing of borings (metres)</i>
1.	Highway (subgrade survey)	300 to 600
2.	Earth dam	30 to 60
3.	Borrow pits	30 to 120
4.	Multistorey buildings	15 to 30
5.	Single story factories	30 to 90

Note : For uniform soil conditions, the above spacings are doubled; for irregular conditions, these are halved.

Table 1. Spacing of Borings (Sowers and Sowers, 1970)

"IS: 1892-1979—Code of Practice for Subsurface Investigation for Foundations" has made the following recommendations:

For a compact building site covering an area of about 0.4 hectare, one bore hole or trial pit in each corner and one in the centre should be adequate. For smaller and less important buildings even one bore hole or trial pit in the centre will suffice. For very large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of bore holes or trial pits. Cone penetration tests may be performed at every 50 m by dividing the area in a grid pattern and number of bore holes or trial pits decided by examining the variation in penetration curves. The cone penetration tests may not be possible at sites having gravelly or boulderous strata. In such cases geophysical methods may be suitable.

2.2.5 Depth of Borings

In order to furnish adequate information for settlement predictions, the borings should penetrate all strata that could consolidate significantly under the load of the structure. This necessarily means that, for important and heavy structures such as bridges and tall buildings, the borings should extend to rock. For smaller structures, however, the depth of boring may be estimated from the results of previous investigations in the vicinity of the site, and from geologic evidence.

Experience indicates that damaging settlement is unlikely to occur when the additional stress imposed on the soil due to the weight of the structure is less than 10% of the initial stress in the soil due to self-weight. E.De Beer of Belgium adopted this rule for determining the so-called 'critical depth of boring' (Hvorslev, 1949). Based on this, recommended depths of borings for buildings are about 3.5 m and 6.5 m for single- and two-storey buildings. For dams and embankments, the depth ranges between half the height to twice the height depending upon the foundation soil.

According to IS: 1892-1979: "The depth of exploration required depends upon the type of the proposed structure, its total weight, the size, shape and disposition of the loaded area, soil profile and the physical properties of the soil that constitutes each individual stratum. Normally, it should be one and half times the width of the footing below foundation level. If a number of loaded areas are in close proximity, the effect of each is additive. In such cases, the whole area may be considered as loaded and exploration should be carried out up to one and half times the lower dimension. In any case, the depth to which seasonal variations affect the soil should be regarded as the minimum depth for the exploration of the sites. But, where industrial processes affect the soil characteristics, this depth may be more. The presence of fast-growing and water-seeking trees also contributes to the weathering processes..."

The depth of exploration at the start of the work may be decided as given in Table 2, which may be modified as exploration proceeds, if required.

<i>S.No.</i>	<i>Type of foundation</i>	<i>Depth of exploration</i>
1.	Isolated spread footings or raft or adjacent footings with clear spacing equal or greater than four times the width	One and half times the width
2.	Adjacent footings with clear spacing less than twice the width	One and half times the length
3.	Adjacent rows of footings	
	(i) With clear spacing between rows less than twice the width	Four and half times the width
	(ii) With clear spacing between rows greater than twice the width	Three times the width
	(iii) With clear spacing between rows greater than or equal to four times the width	One and half times the width
4.	Pile and Well foundations	One and half times the width of structure from bearing level (toe of pile or bottom of well)
5.	Road cuts	Equal to the bottom width of the cut
6.	Fill	Two metres below the ground level or equal to the height of the fill whichever is greater

Table 2. Depth of Exploration (IS: 1892-1979)

2.2.6 Boring Log

Information on subsurface conditions obtained from the boring operation is typically presented in the form of a boring record, commonly known as “boring log”. A continuous record of the various strata identified at various depths of the boring is presented. Description or classification of the various soil and rock types encountered, and data regarding ground water level have to be necessarily given in a pictorial manner on the log. A “field” log will consist of this minimum information, while a “lab” log might include test data presented alongside the boring sample actually tested.

Sometimes a subsurface profile indicating the conditions and strata in all borings in series is made. This provides valuable information regarding the nature of variation or degree of uniformity of strata at the site. This helps in delineating between “good” and “poor” area. The standard practice of interpolating between borings to determine conditions surely involves some degree of uncertainty.

A sample record sheet for a boring is shown in Fig. 3. A site plan showing the disposition of the borings should be attached to the records.

RECORD OF BORING [IS : 1892-1979]

Bored for	Name of boring organization:	Location-site
Ground level.....		Boring No.
Type of boring.....		Soil sampler used
Diameter of boring.....		Date started
Inclination: Vertical.....		Date completed
Bring:		Recorded

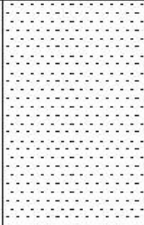
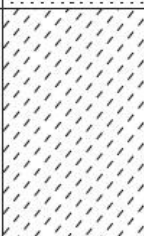
Description of strata	Soil classification	Thickness of stratum	Depth from GL	R.L. of lower contact	Samples			GWL	Remarks
					Type	No.	Depth and thickness of sample		
Fine to medium sand with practically no binder	SP		1 m		Undisturbed	1	1 m		
			2 m				1.4 m		
			2.7 m				1.7 m		
			3 m				2 m		
			4 m						
Silty clays of medium plasticity no coarse or medium sands	CI		5 m		Undisturbed	2	4 m	Not struck upto 6 m depth	
							4.3 m		
							5 m		

Fig. 3. Sample boring log

CHAPTER 3 SOUNDING AND PENETRATION TESTS

3.1 General

Subsurface soundings are used for exploring soil strata of an erratic nature. They are useful to determine the presence of any soft pockets between drill holes and also to determine the density index of cohesionless soils and the consistency of cohesive soils at various desired depths from the ground surface.

Methods of sounding normally consist of driving or pushing a standard sampling tube or a cone. The devices involved are also termed 'penetrometers', since they are made to penetrate the subsoil with a view to measuring the resistance to penetration of the soil strata, and thereby try to identify the soil and some of its engineering characteristics. The necessary field tests are also called 'penetration tests'.

If a sampling tube is used to penetrate the soil, the test is referred to as the Standard Penetration Test (SPT, for brevity). If a cone is used to penetrate the soil, the test is called a 'Cone penetration test'. Static and dynamic cone penetration tests are used depending upon the mode of penetration—static or dynamic.

A field test called 'Vane Test' is used to determine the shearing strength of the soil located at a depth below the ground.

3.2 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is widely used to determine the parameters of the soil *in-situ*. The test is especially suited for cohesionless soils as a correlation has been established between the SPT value and the angle of internal friction of the soil.

The test consists of driving a split-spoon sampler (Fig. 18.6) into the soil through a bore hole 55 to 150 mm in diameter at the desired depth. A hammer of 640 N (65 kg) weight with a free fall of 750 mm is used to drive the sampler. The number of blows for a penetration of 300 mm is designated as the "Standard Penetration Value" or "Number" N . The test is usually performed in three stages. The blow count is found for every 150 mm penetration. If full penetration is obtained, the blows for the first 150 mm are ignored as those required for the seating drive. The number of blows required for the next 300 mm of penetration is recorded as the SPT value. The test procedure is standardised by ISI and set out in "IS: 2131-1986—Standard Penetration Test".

Usually SPT is conducted at every 2 m depth or at the change of stratum. If refusal is noticed at any stage, it should be recorded.

In the case of fine sand or silt below water-table, apparently high values may be noted for N . In such cases, the following correction is recommended (Terzaghi and Peck, 1948):

$$N = 15 + \frac{1}{2} (N' - 15)$$

Where N' = observed SPT value,
 N = corrected SPT value.

For SPT made at shallow levels, the values are usually too low. At a greater depth, the same soil, at the same density index, would give higher penetration resistance.

The effect of the overburden pressure on SPT value may be approximated by the equation:

$$N = N' \cdot \frac{350}{(\sigma + 70)}$$

Where N' = observed SPT value,
 N = corrected SPT value, and

σ = effective overburden pressure in KN/m², not exceeding 280 KN/m². This implies that no correction is required if the effective overburden pressure is 280 kN/m². Terzaghi and Peck also give the following correlation between SPT value, D_r , and ϕ :

S. No.	Condition	N	D_r	ϕ
1.	Very loose	0 – 4	0 – 15%	Less than 28°
2.	Loose	4 – 10	15 – 35%	28° – 30°
3.	Medium	10 – 30	35 – 65%	30° – 36°
4.	Dense	30 – 50	65 – 85%	36° – 42°
5.	Very dense	Greater than 50	Greater than 85%	Greater than 42°

Table 3. Correlation between N, D_r , and ϕ

For clays the following data are given:

S. No.	Consistency	N	q_u (kN/m ²)
1.	Very soft	0 – 2	Less than 25
2.	Soft	2 – 4	25 – 50
3.	Medium	4 – 8	50 – 100
4.	Stiff	8 – 15	100 – 200
5.	Very stiff	15 – 30	200 – 400
6.	Hard	Greater than 30	Greater than 400

Table 4. Correlation between N and q_u

The correlation for clays is rather unreliable. Hence, vane shear test is recommended for more reliable information.

3.3 Static Cone Penetration Test (Dutch Cone Test)

The Static cone penetration test, which is also known as Dutch Cone test, has been standardized by the ISI and given in “IS: 4968 (Part-III)-1976—Method for subsurface sounding for soils—Part III Static cone penetration test”.

Among the field sounding tests the static cone tests in a valuable method of recording variation in the *in-situ* penetration resistance of soils, in cases where the *in-situ* density is disturbed by boring operations, thus making the standard penetration test unreliable especially under water. The results of the test are also useful in determining the bearing capacity of the soil at various depths below the ground level. In addition to bearing capacity values, it is also possible to determine by this test the skin friction values used for the determination of the required lengths of piles in a given situation. The static cone test is most successful in soft or loose soils like silty sands, loose sands, layered deposits of sands, silts and clays as well as in clayey deposits. In areas where some information regarding the

foundation strata is already available, the use of test piles and loading tests thereof can be avoided by conducting static cone penetration tests.

Experience indicates that a complete static cone penetration test up to depths of 15 to 20 m can be completed in a day with manual operations of the equipment, making it one of the inexpensive and fast methods of sounding available for investigation.

The equipment consists of a steel cone, a friction jacket, sounding rod, mantle tube, a driving mechanism and measuring equipment.

The steel cone shall be of steel with tip hardened. It shall have an apex angle of $60^\circ \pm 15'$ and overall base diameter of 35.7 mm giving a cross-sectional area of 10 cm². The friction jacket shall be of high carbon steel. These are shown in Fig. 4.

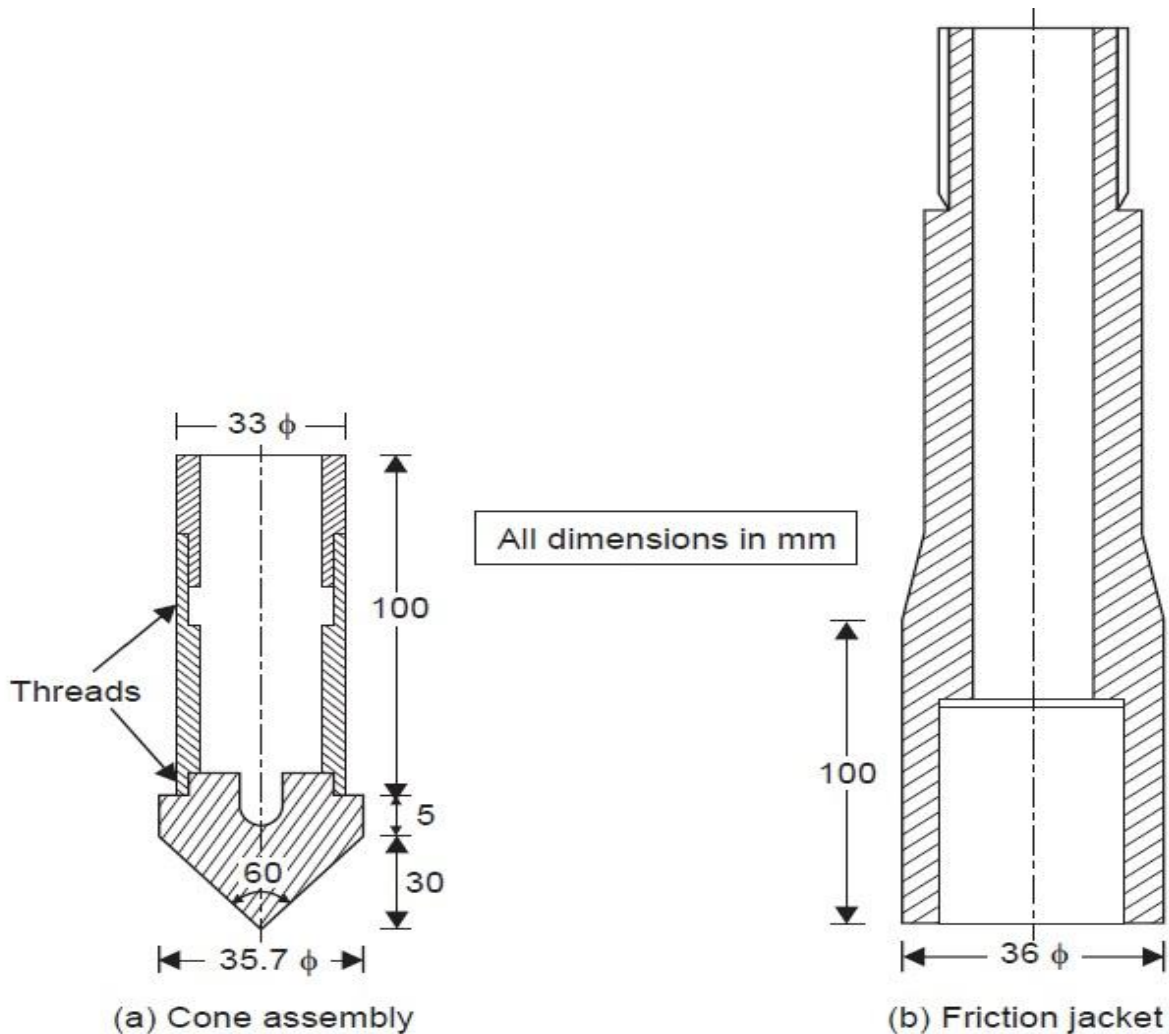


Fig 4. Cone assembly and friction jacket for static cone penetration test (IS)

The sounding rod is a steel rod of 15 mm diameter which can be extended with additional rods of 1 m each in length. The mantle tube is a steel tube meant for guiding the sounding rod which goes through it. It should be of one meter in length with flush coupling.

The driving mechanism should have a capacity of 20 to 30 KN for manually operated equipment and 100 KN for the mechanically operated equipment. The mechanism essentially consists of a rack and pinion arrangement operated by a winch. The reaction for the thrust may be obtained by suitable devices capable of taking loads greater than the capacity of the equipment.

The hand operated winch may be provided with handles on both sides of the frame to facilitate driving by four persons for loads greater than 20 kN. For the engine driven equipment the rate of travel should be such that the penetration obtained in the soil during the test is 10 to 15 mm/s.

Hydraulic pressure gauges should be used for indicating the pressure developed. Alternatively, a proving ring may also be used to record the cone resistance. Suitable capacities should be fixed for the gauges.

Basically, the test procedure for determining the static cone and frictional resistances consists of pushing the cone alone through the soil strata to be tested, then the cone and the friction jacket, and finally the entire assembly in sequence and noting the respective resistance in the first two cases. The process is repeated at predetermined intervals. After reaching the deepest point of investigation the entire assembly should be extracted out of the soil.

The results of the test shall be presented graphically, in two graphs, one showing the cone resistance in kN/m² with depth in meters and the other showing the friction resistance in kN/m² with depth in meters, together with a bore hole log.

The cone resistance shall be corrected for the dead weight of the cone and sounding rods in use. The combined cone and friction resistance shall be corrected for the dead weight of the cone, friction jacket and sounding rods. These values shall also be corrected for the ratio of the ram area to the base area of the cone.

The test is unsuitable for gravelly soils and for soils with standard penetration value *N* greater than 50. Also, in dense sands the anchorage becomes too cumbersome and expensive and for such cases dynamic cone penetration tests may be carried out.

3.4 Dynamic Cone Penetration Test

The dynamic cone penetration test is standardised by the ISI and given in “IS: 4968 (Part I)-1976—Method for Subsurface Sounding for Soils—Part I Dynamic method using 50 mm cone without bentonite slurry”.

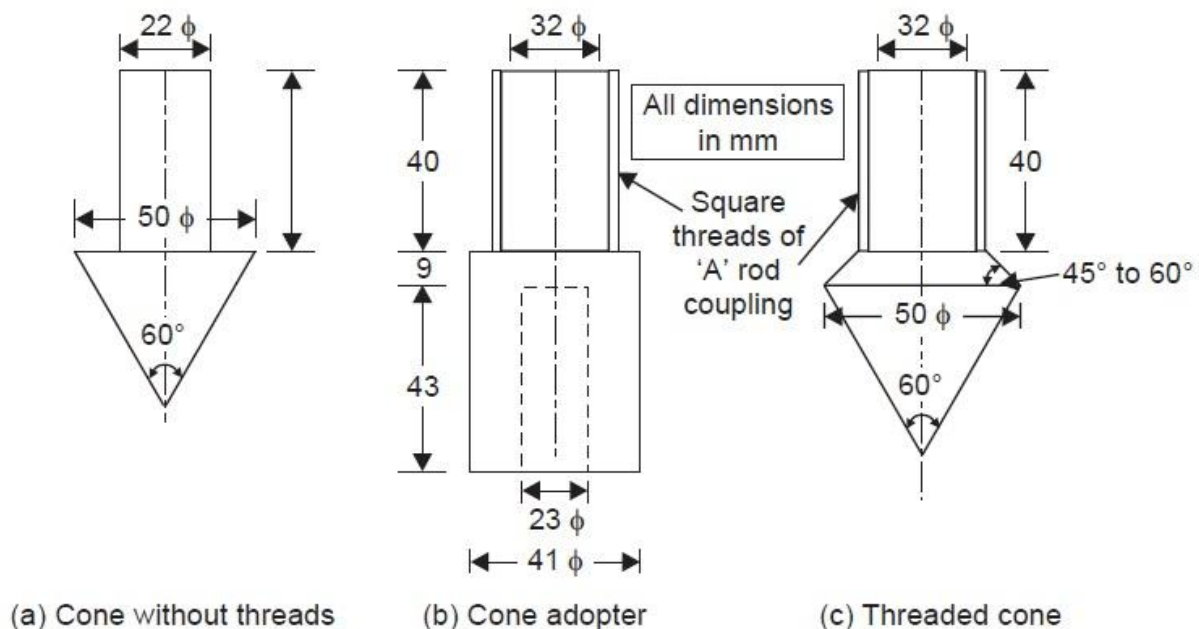


Fig. 5. Cone details for dynamic cone penetration test (IS)

The equipment consists of a cone, driving rods, driving head, hoisting equipment and a hammer.

The cone with threads (recoverable) shall be of steel with tip hardened. The cone without threads (expendable) may be of mild steel. For the cone without threads, a cone adopter shall be provided. These are shown in Fig. 5.

The driving rods should be A rods of suitable length with threads for joining A rod coupling at either end. The rods should be marked at every 100 mm.

The driving head shall be of mild steel with threads at either end for a rod coupling. It shall have a diameter of 100 mm and a length of 100 to 150 mm.

Any suitable hoisting equipment such as a tripod may be used. A typical set-up using a tripod is shown in Fig. 6.

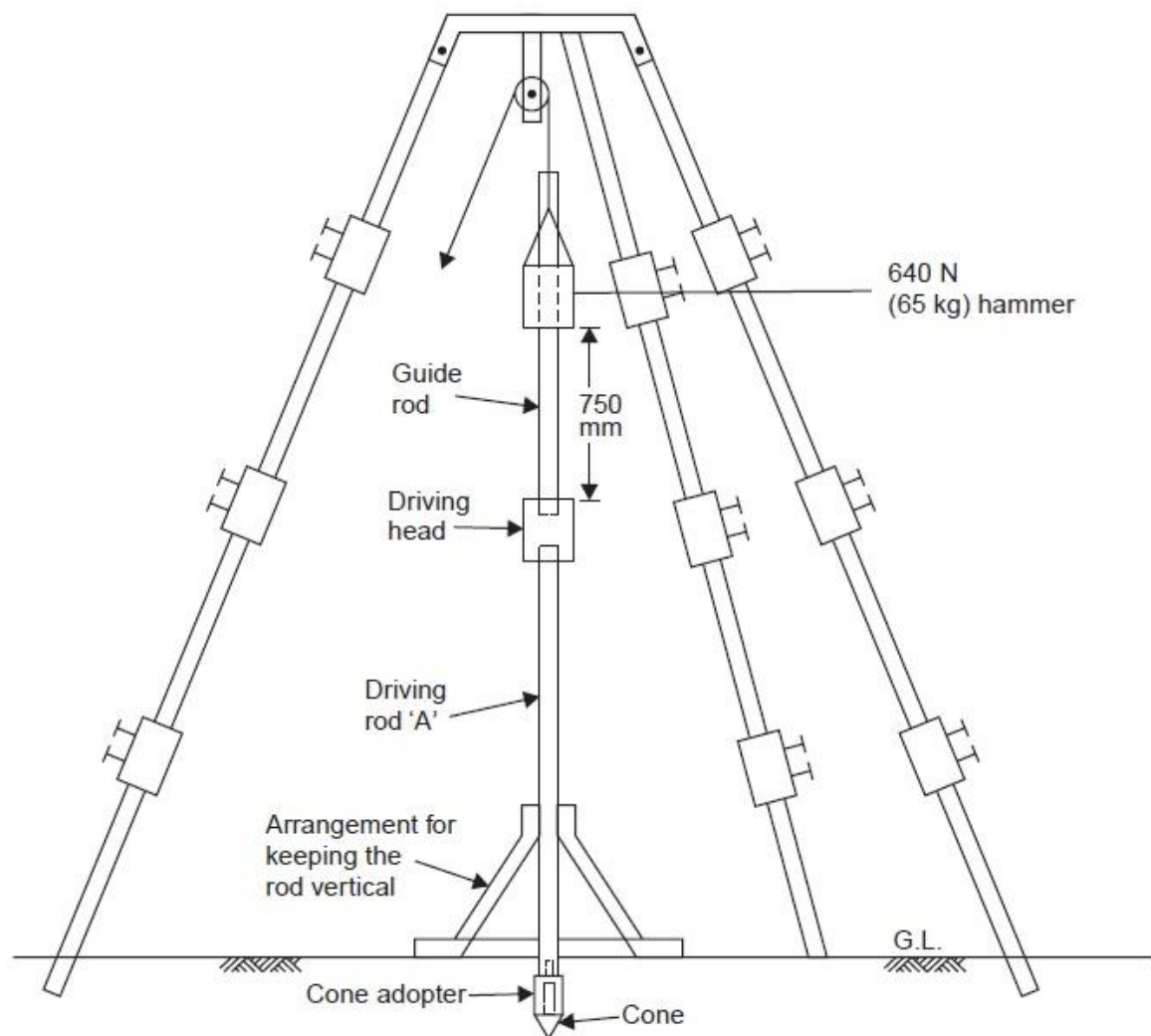


Fig. 6. Typical set-up for dynamic cone penetration test (IS)

The hammer used for driving the cone shall be of mild steel or cast-iron with a base of mild steel. It shall be 250 mm high and of suitable diameter. The weight of the hammer shall be 640 N (65 kg).

The cone shall be driven into the soil by allowing the hammer to fall freely through 750 mm each time. The number of blows for every 100 mm penetration of the cone shall be recorded. The process shall be repeated till the cone is driven to the required depth. To save the equipment from damage, driving may be stopped when the number of blows exceeds 35 for 100 mm penetration.

When the depth of investigation is more than 6 m, bentonite slurry may be used for eliminating the friction on the driving rods. The cone used in this case is of 62.5 mm size and the details of the dynamic method using bentonite slurry, as standardized by the ISI, are available in “IS: 4968 (Part-II)-1976—Method for subsurface Sounding for Soils—Part II Dynamic method using 62.5 mm cone and bentonite slurry”.

Dynamic Cone Penetration test is a simple device for probing the soil strata and it has an advantage over the standard penetration test in that making of a bore hole is avoided. Moreover, the data obtained by cone test provides a continuous record of soil resistance. Efforts are being made to correlate the cone resistance with the SPT value for different zones.

3.5 In-situ Vane Shear Test

In-situ vane shear test is best suited for the determination of shear strength of saturated cohesive soils, especially of sensitive clays, susceptible for sampling disturbances. The vane shear test consists of pushing a four-bladed vane in the soil and rotating it till a cylindrical surface in the soil fails by shear. The torque required to cause this failure is measured and this torque is converted to a unit shearing resistance of the cylindrical surface. The test may be conducted from the bottom of a bore hole or by direct penetration from ground surface.

The test, as standardized by the ISI, is given in “IS: 4434-1978—Code of Practice for *In-situ* Vane Shear Test for Soils”.

The equipment consists of a shear vane, torque applicator, rods with guides, drilling equipment and jacking arrangement.

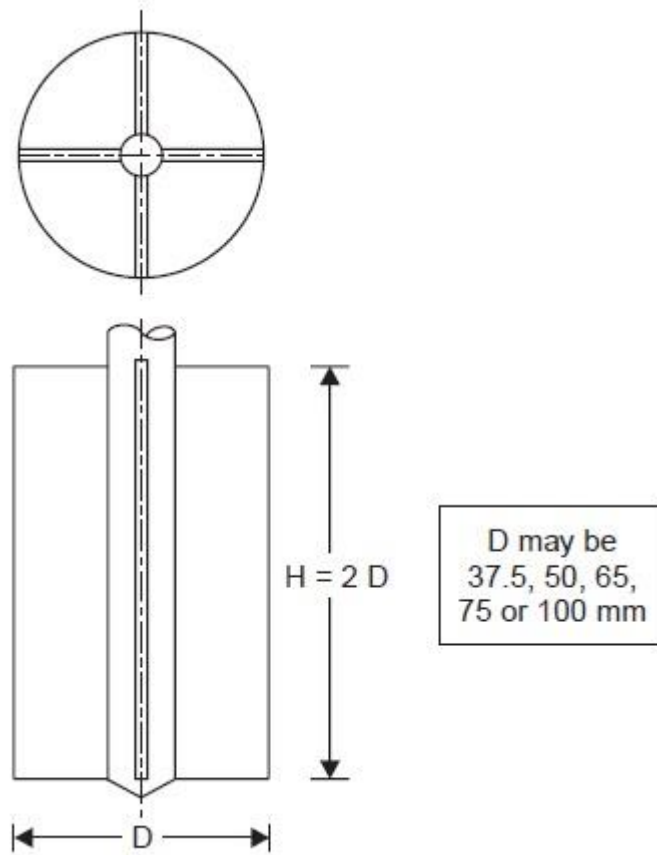


Fig. 7. Field vane (IS)

The area ratio of the vane shall be kept as low as possible, and shall not exceed 15%, calculated as follows:

$$A_r = \frac{8t(D - d) + \pi d^2}{\pi D^2}$$

Where, A_r = area ratio,

t = thickness of the blades of the vane,

D = overall diameter of the vane, and

d = diameter of central vane rod including any enlargement due to welding.

A diagrammatic vane test arrangement is shown in Fig. 8.

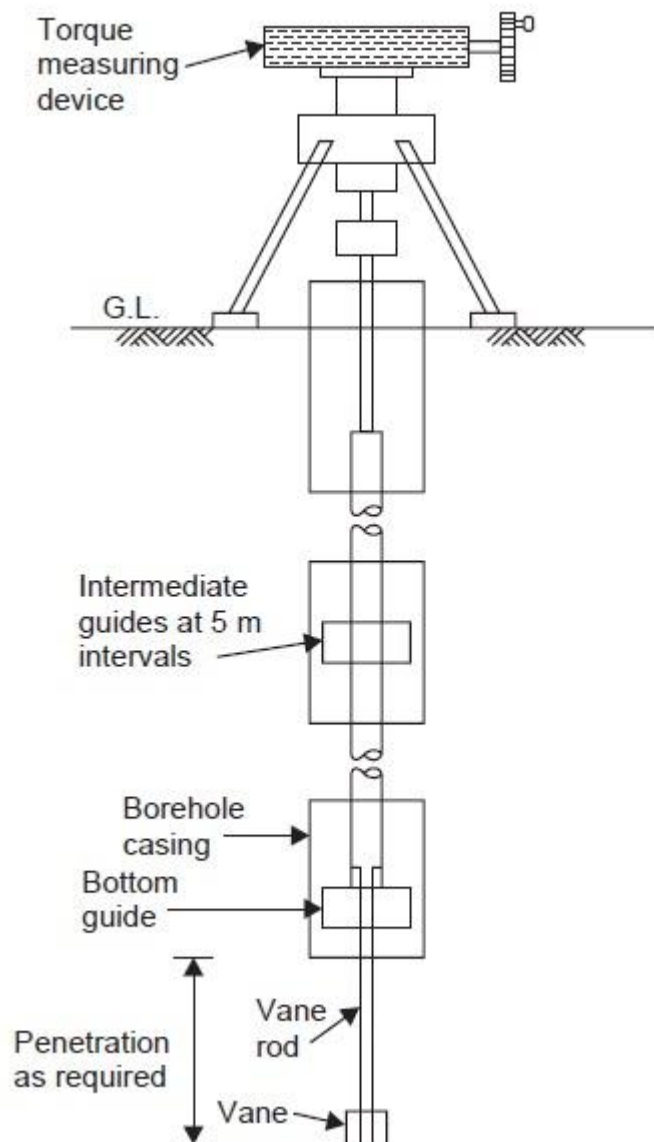


Fig. 8 Arrangement for vane test, from bottom of bore hole (IS)

The vane is pushed with a moderately steady force up to a depth of four-times the diameter of the bore hole or 50 cm, whichever is more, below the bottom. No torque shall be applied during the thrust. The torque applicator is tightened to the frame properly. After about 5 minutes, the gear handle is turned so that the vane is rotated at the rate of 0.1°/s. The maximum torque reading is noted when the reading drops appreciably from the maximum.

For a rectangular vane the shear strength of the soil is computed from the following formula:

$$\tau = \frac{T}{\pi D^2 [(H / 2) + (D / 6)]}$$

Where, τ = shear strength (N/mm²)

T = torque in mm-N,

D = overall diameter of the vane in mm, and

H = height of the vane in mm.

The assumptions involved are:

1. shearing strengths in the horizontal and vertical directions are the same;
2. at the peak value, shear strength is equally mobilised at the end surface as well as at the centre;
3. at the centre;
4. the shear surface is cylindrical and has a diameter equal to the diameter of the vane;
5. vane; and
6. the shear stress distribution on the vane is as shown in Fig. 9.

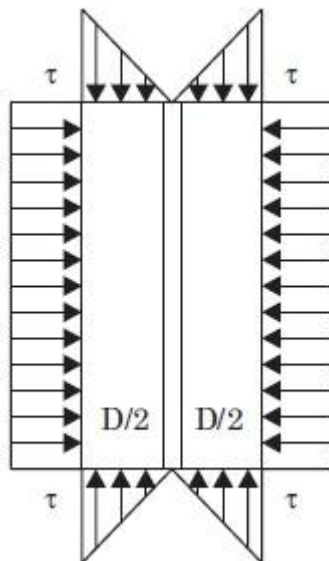


Fig. 9 Assumed stress distribution on blades of vane

For equilibrium, the applied torque, T = moment of resistance of the blades of the vane.
 $\therefore T$ = surface area \times surface stress \times lever arm + end areas \times average stress \times lever arm.

$$= \pi DH \times \tau \times \frac{D}{2} + 2 \left[\frac{\pi D^2}{4} \times \frac{\tau}{2} \times \frac{2}{3} D \right]$$

$$= \tau \left[\frac{\pi D^2 H}{2} + \frac{\pi D^3}{6} \right]$$

This leads to Eq. 18.7 for τ .

If $H = 2D$, Eq. 18.7 reduces to

$$\tau = \frac{6T}{7\pi D^3} \approx \frac{3T}{11.D^3} \quad \dots(\text{Eq. 18.8})$$

A shoe is used for protecting the vane if it is to penetrate direct from the ground surface. A 100-mm vane is recommended for very soft soils. For moderately firm saturated soils, a 75-mm vane is recommended. The 50-mm vane is used infrequently but is intended for firm saturated soils.

The vane test may have a laboratory version also, the vane being relatively much smaller than the field vane.

CHAPTER 4

Interpretation of cone penetration tests in sand and clay.

4.1 General

Significant advances have been made in recent years in research, development, interpretation, and application of cone penetration testing. The addition of pore pressure measurements during cone penetration testing has added a new dimension to the interpretation of geotechnical parameters.

The cone penetration test induces complex changes in stresses and strains around the cone tip. No one has yet developed a comprehensive theoretical solution to this problem. Hence, the cone penetration test provides indices which can be correlated to soil behavior. Therefore, the interpretation of cone penetration data is made with empirical correlations to obtain required geotechnical parameters.

This chapter discusses the significant recent developments in cone penetration testing and presents a summarized work guide for practicing engineers for interpretation for soil classification, and parameters for drained conditions during the test such as relative density, drained shear strength, and deformation characteristics of sand. Factors that influence the interpretation are discussed and guidelines provided. Clay considers undrained conditions during the test and summarizes recent developments to interpret parameters for clay soils, such as undrained shear strength, deformation characteristics of clay, stress history, consolidation characteristics, permeability, and pore pressure. The advantages and use of the piezometer cone are also discussed.

4.2 Soil classification - Sand

The most comprehensive recent work on soil classification using electric cone penetrometer data is that by Douglas and Olsen (1981). A copy of their proposed soil-behavior type classification chart is shown in Fig. 10. The chart shows how cone penetration test data have been correlated with other soil type indices, such as those provided by the Unified Soil Classification System. The correlation was based on extensive data collected from areas in California, Oklahoma, Utah, Arizona, and Nevada (USGS 1980). Figure 11 is a simplified working version for identifying soil type.

The usual progression of site investigation using cone penetration test (CPT) is to perform the CPT soundings, develop detailed site profiles with the soil classification charts (Figs. 10 and 11), and then selectively sample and test to provide any additional information regarding ambiguous classifications. With local experience this latter step is often not necessary.

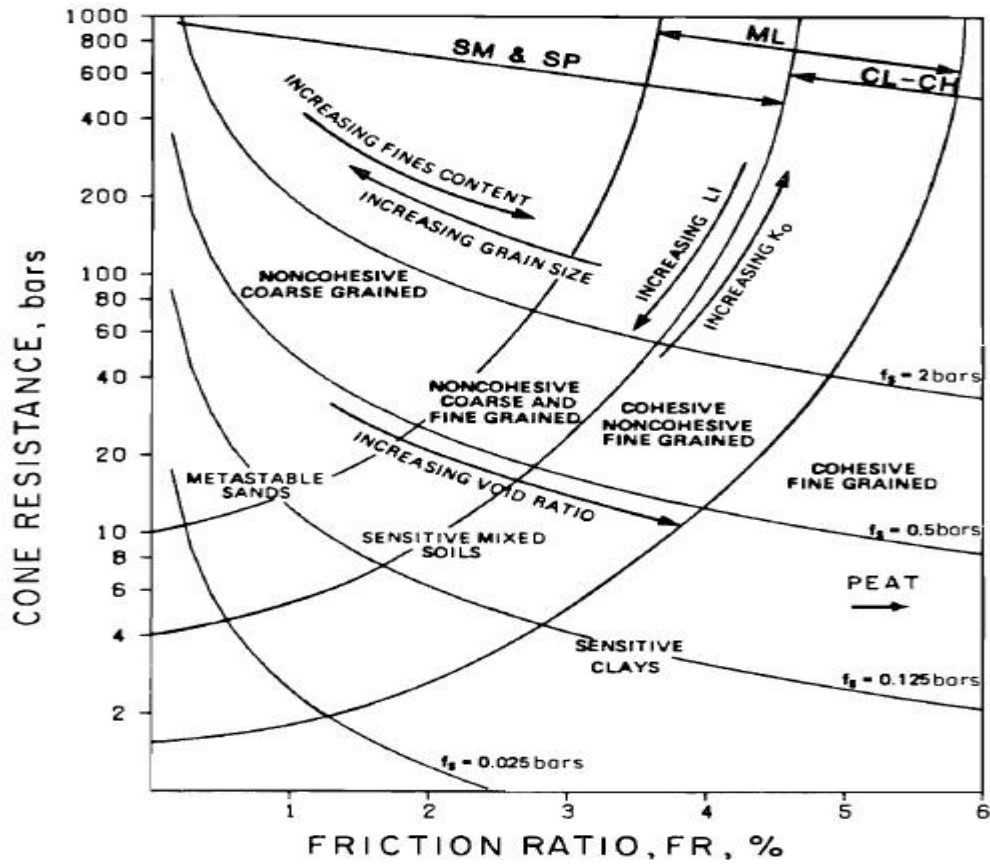


FIG. 10. Soil classification chart for standard electric friction cone (after Douglas and Olsen 1981);

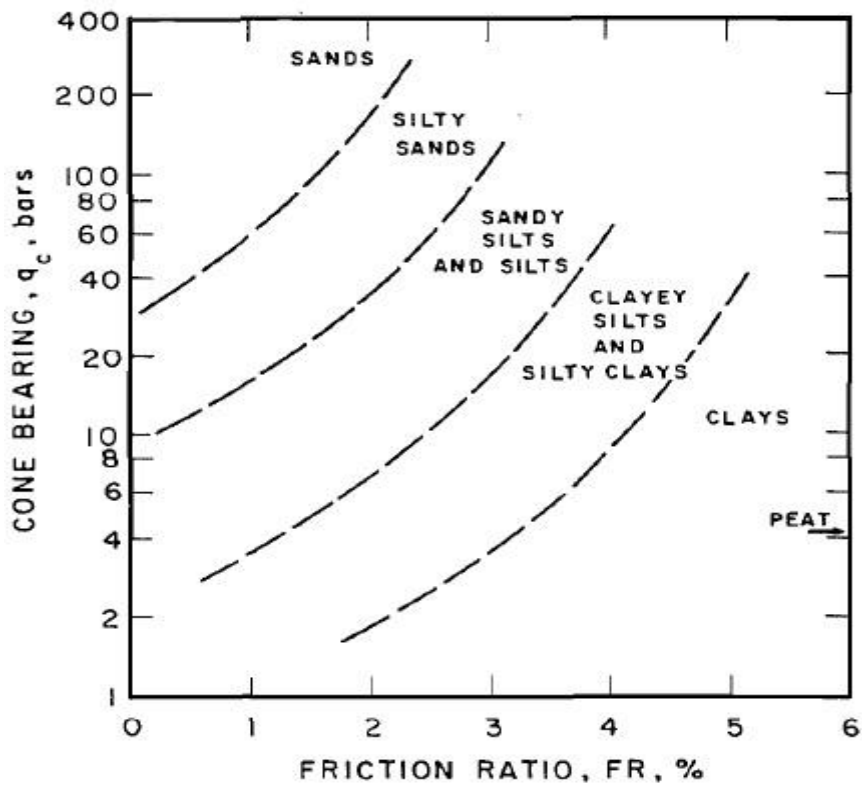
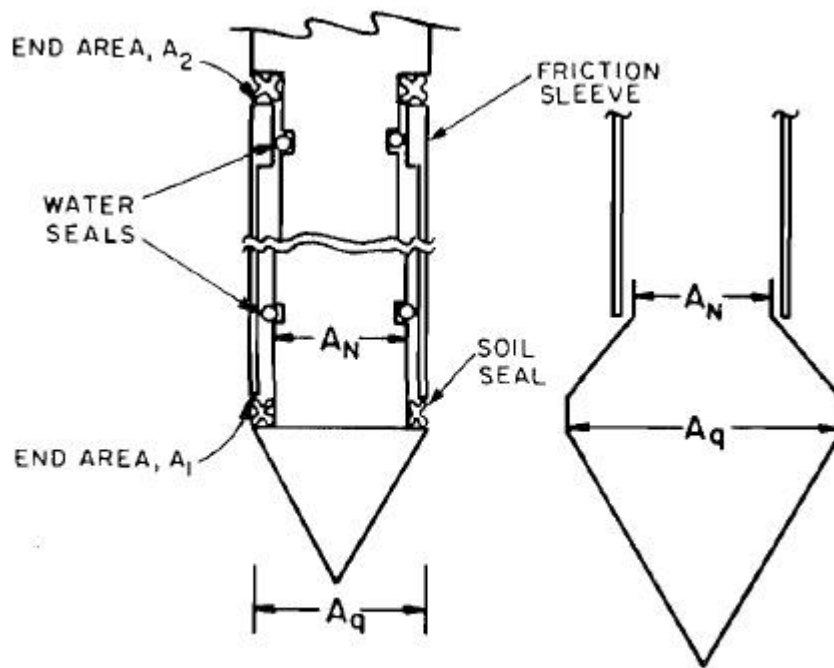


FIG. 11. Simplified soil classification chart for standard electric friction cone.

Recent work by Campanella *et al.* (1983) has illustrated the importance of cone design and the effect that water pressures have on the measured bearing and friction due to unequal end areas (Fig. 12). An all around water pressure causes a thrust on the friction sleeve which in turn causes a zero offset on the load cell output. The friction sleeve offset can be calculated as the total water pressure during penetration multiplied by the difference in end area ($A_1 - A_2$) and can be either positive or negative depending upon which end area is larger. With respect to bearing, the tip should measure total stress or intergranular pressure plus water pressure. However, the water pressure is only partially sensed by the tip or bearing load cell because the area A_N is for current designs always less than A_q , (see Fig. 12). A detailed discussion concerning cone design is also given by Schaap and Zuidberg (1982). The data used to compile the classification chart (Fig. 10) used bearing and friction values that had not been corrected for pore pressure effects, since, in general, pore pressure measurements were not made. It appears that there is little difference between corrected and uncorrected friction ratios for most soil types except for those soils that classify in the lower left portion of the chart (Fig. 10). These soils usually generate large positive pore pressures during penetration and have very low measured bearing (q , < 10 bars) (1 bar = 100 kPa) and small friction values where corrections become very significant. These soils also tend to have high liquidity index values, as noted by Douglas and Olsen (1981). Also, in off-shore investigations where significant hydrostatic water pressures exist, it may be important to account for these effects for most soil types. Most standard electric cone data do not include pore pressure measurements and the measured bearing and friction values are therefore not corrected for pore pressure effects. For this type of data the chart in Fig. 10 can be used directly to provide a reasonable estimate of soil type. If pore pressure measurements are included and the necessary corrections applied to the data, Figs. 10 and 11 should be used with caution, especially for soft saturated soils, and should always be adjusted to reflect local experience. Some electric cones, like the one developed at UBC, have equal end area friction sleeves where the friction measurement requires no correction. Increased use of these cones will eventually lead to further improvements in the soil classification chart shown in Fig. 10. Unfortunately, there is currently no cone design which eliminates the need to correct bearing for net area ratio.



Bearing Net Area Ratio = A_N / A_q
 Friction Unequal End Area, $A_1 \neq A_2$

FIG. 12. Influence of unequal end areas.

A comprehensive classification chart for use with a mechanical cone was proposed by Searle (1979). The chart is similar to that proposed by Schmertmann (1978~) although considerably more information is contained on Searle's chart.

Recently the CPT has also been used for classification and interpretation of unconventional soils such as carbonate sediments (Searle 1979; Power 1982).

Robertson and Campanella (1983) suggest that the friction ratio for some fine grained soils may decrease with increasing effective overburden pressure. Thus, the accuracy of Figs. 10 and 11 may be reduced somewhat for CPT data from very deep soundings. Of course there is no substitute for improvements to Figs. 10 and 11 based on local experience.

4.3 Stratigraphy - Sand

The cone penetration tip resistance is influenced by the soil properties ahead and behind the tip. Chamber studies by Schmertmann (1978a) showed that the tip senses an interface between 5 and 10 cone diameters ahead and behind. The distance over which the cone tip senses an interface increases with increasing soil stiffness. Since stiffness increases with increasing overburden pressure, the distance over which the cone tip senses an interface tends to increase with depth (Treadwell 1975). For interbedded relatively shallow deposits, the thinnest stiff layer for which the measured cone bearing represents a full response (i.e., q_c to reach full value characteristic for the soil within the layer) is about 10-20 cone diameters. For the standard 10 cm' electric cone, the minimum stiff layer thickness to ensure full or plateau value of tip resistance is between 36 and 72 cm. Since the cone tip is advanced continuously, the tip resistance will sense much thinner layers, but not fully. This has significant implications when interpreting cone bearing, for example, relative density determination in sand. If a sand layer is less than about 70 cm thick and located between,

say, two soft clay deposits, the cone penetration resistance may not reach its full value within the sand because of the close proximity of the adjacent interfaces. Thus, the relative density in the sand may be severely underestimated.

The continuous monitoring of pore pressures during cone penetration can significantly improve the identification of soil stratigraphy (Campanella et al. 1983). The pore pressure responds to the soil type in the immediate area of the cone tip. To aid in the identification of very thin silt or sand layers within clay deposits, some researchers (Torstensson 1982) have proposed and successfully used thin (2.5 mm) pore pressure elements located immediately behind the cone tip.

4.4 Drained soil

4.4.1 Density

For cohesionless soils the density, or more commonly the relative density, is often used as an intermediate soil parameter. Recent research has shown that the stress-strain and strength behavior of cohesionless soils is too complex to be represented solely by the relative density of the soil. Several papers in ASTM (1973) have discussed difficulties associated with determination of maximum, minimum, and *in-situ* densities as well as problems in correlating relative density with measured soil properties. However, because many engineers continue to use relative density as a guide in design some discussion is given here on recent work relating cone penetration resistance to soil relative density.

Recent work in large calibration chambers (Veismanis 1974; Chapman and Donald 1981; Baldi *et al.* 1981; Parkin *et al.* 1980; Villet and Mitchell 1981) has provided numerous correlations of cone resistance (q_c) with soil relative density (D_r). Most of these works have also shown that no single unique relationship exists between relative density, *in-situ* effective stress, and cone resistance for all sands. Recent work by Parkin and Lunne (1982) has also shown that the measured relationship between relative density and cone resistance is influenced by the small calibration chamber size, particularly at higher densities.

It is not surprising that no unique relationship exists between cone resistance, *in-situ* effective stress, and relative density since other factors such as soil compressibility also influence cone resistance.

A review of the numerous calibration chamber tests performed on a variety of different sands shows a significant range of D_r versus q_c relationships. However, on detailed inspection part of the variation can be accounted for due to differences in chamber size and boundary conditions. All the chamber test results, however, show that the curves are all similar in shape and most show that the cone resistance can be more uniquely related to relative density, for any given sand, if correlated with the *in-situ* initial horizontal effective stress (σ'_{ho}). If the horizontal effective stress is used the relationship can be expected to apply to both normally and overconsolidated sand. Figure 13 shows a comparison between the curves proposed by Schmertmann (1978b), Villet and Mitchell (1981), and Baldi *et al.* (1981) for two levels of relative density. All the curves have been corrected for chamber size. Details of the sands used in the calibration chamber studies are given in Table 5.

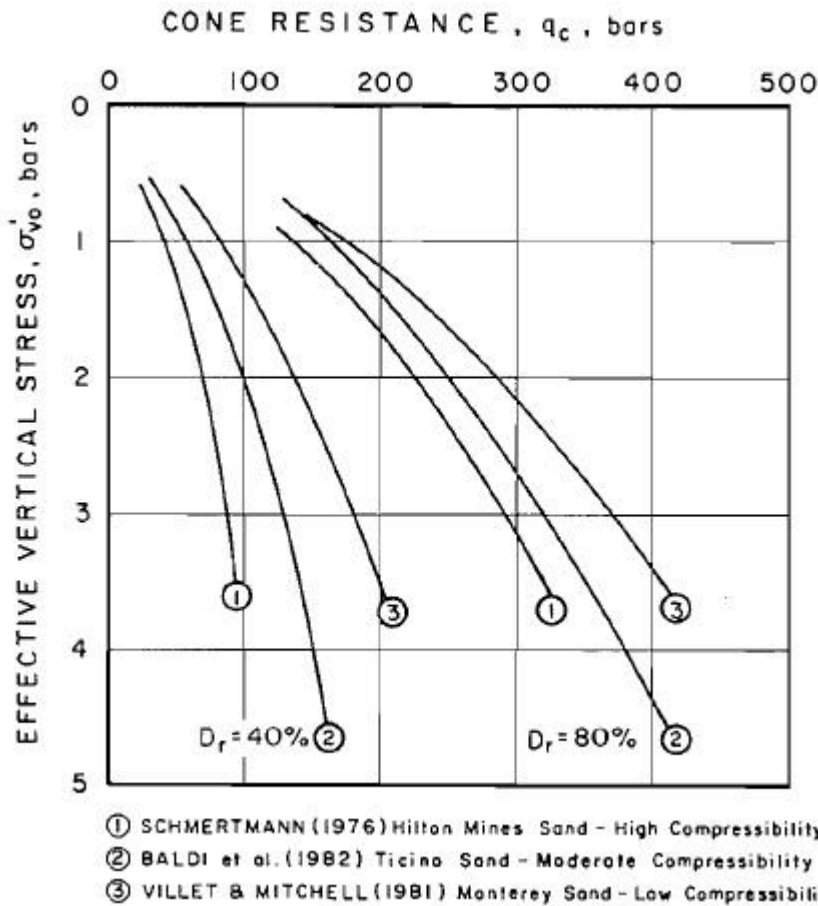


FIG. 13. Comparison of different relative density relationships.

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TABLE 1. Properties of sand tested in calibration chamber studies

Reference	Sand name	Mineralogy	Shape	Gradation (mm)		Porosity	
				D_{60}	D_{10}	n_{max}	n_{min}
Baldi <i>et al.</i> (1981, 1982)	Ticino	Mainly quartz 5%* mica	Subangular to angular	0.65	0.40	0.50	0.41
Villet and Mitchell (1981)	Monterey	Mainly quartz some feldspar	Subrounded to subangular	0.40	0.25	0.45	0.36
Schmertmann (1978 <i>b</i>)	Ottawa #90	Quartz	Rounded	0.24	0.13	0.44	0.33
Schmertmann (1978 <i>b</i>)	Hilton mines	Quartz + mica + feldspar	Angular	0.30	0.15	0.44	0.30
Parkin <i>et al.</i> (1980)	Hokksund	35% quartz 45% feldspar 10%* mica	Rounded to subangular	0.5	0.27	0.48	0.36
Veismanis (1974)	Edgar	Mainly quartz	Subangular	0.5	0.29	0.48	0.35
Veismanis (1974)	Ottawa	Quartz	Subangular	0.54	0.45	0.42	0.32
Holden (1971)	South Oakleigh	Quartz	Subangular	0.19	0.12	0.47	0.35
Holden (1971)	South Oakleigh	Quartz	Subangular	0.37	0.17	0.43	0.29
Chapman and Donald (1981)	Frankston	Mainly quartz	Rounded to subangular	0.37	0.18	—	—

TABLE 5. Properties of sand tested in calibration chamber studies

The calibration test data (Fig. 13) shows the importance of sand compressibility. The curves by Schmertmann (1978) represent the results of tests on Hilton Mines sand, which is a highly compressible quartz, feldspar, mica mixture with angular grains. The curves by Villet and Mitchell (1981) represent results on Monterey Sand which is relatively incompressible quartz sand with subrounded particles. Schmertmann (1978) also performed tests on Ottawa sand, which is also incompressible quartz sand with rounded particles, and obtained curves almost identical to those of Villet and Mitchell (1981). Thus, it appears that sands with a low compressibility have a D - q relationship similar to that shown by Villet and Mitchell (1981) and sands with a high compressibility have a relationship similar to that shown by Schmertmann (1978). The sand used by Baldi *et al.* (1981) (Ticino Sand) was a quartz, feldspar, mica mixture with subangular particles. The Ticino Sand appears to have a moderate compressibility somewhere between the two extremes of Hilton Mines and Monterey Sand.

4.4.2 Drained shear strength of sand

Many theories and empirical or semi-empirical correlations for the interpretation of drained shear strength of sand from cone resistance have been published. The theories can be divided into two categories, namely those based on bearing capacity theory (Janbu and Senneset 1974; Durgunoglu and Mitchell 1975) and those based on cavity expansion theory (Vesic 1972).

Work by Vesic (1963) has shown that no unique relationship exists between friction angle for sands and cone resistance, since soil compressibility influences the cone resistance. The curvature of the Mohr-Coulomb failure envelope for granular soils has been observed repeatedly by numerous investigators and is presently recognized as a typical material behavior. Most of the available bearing capacity theories on deep penetration neglect both the curvature of the shear strength envelope and the compressibility of the soil. The increasing influence of these two factors tends to reduce the tip resistance.

Based on cavity expansion concepts, Vesic (1972) developed a theory for tip resistance taking account of soil compressibility and volume change characteristics. Baligh (1976) developed this further to incorporate the curvature of the strength envelope. The comprehensive calibration chamber test work by Baldi *et al.* (1981) showed that the cavity expansion theory appeared to model the measured response extremely well. The cavity expansion theory by Vesic, however, cannot incorporate volume expansion. At first this appears to be a major disadvantage since almost all sands dilate (expand) during shear. Work by Vesic and Clough (1968) showed that at high stresses (greater than 50 bars) dense sand ($D_r = 80\%$) will compress during shear. The stresses immediately in front of the cone tip during penetration into sand often exceed 200 bars. Thus it seems reasonable that the high stresses developed during cone penetration in sands cause a compressive punching failure beneath the tip. This agrees well with the observed behavior from model tests of deep penetration (Robinsky and Morrison 1964; Mikasa and Takada 1974). The cavity expansion analysis, however, is complex and requires considerable input data regarding compressibility and shear strength. Calibration chamber results illustrate the complex nature of cone penetration in sands and show that simple closed form solutions to derive the shear strength are not possible. In addition, chamber tests provide valuable insight into the relative importance of the various factors that influence cone penetration in sands.

In general, it would be expected that the bearing capacity theories, which cannot take account of soil compressibility, could not provide reliable predictions of friction angle. However, the work by Villet and Mitchell (1981) showed that the bearing capacity theory developed by Durgunoglu and Mitchell (1975) provided reasonable predictions for a variety of different sands. The chamber test study by Baldi *et al.* (1981) showed that Durgunoglu and Mitchell's theory gave excellent agreement with measured friction angle at a failure stress level approximately equal to the average stress around the cone. The average stress

around the cone was assumed to be about 9 times the *in-situ* horizontal stress (Baldi *et al.* 1982). Thus, due to the curvature of the strength envelope, the Durgunoglu and Mitchell theory appears to underestimate the friction angle at the *in-situ* stress level by about 2 degrees.

As discussed earlier, the two main parameters that control penetration resistance in sands are shear strength and compressibility. Work by Al-Awkati (1975) showed that, for the predominantly quartz sands he tested, shear strength had significantly more influence on cone resistance than compressibility. This is probably due to the fact that for the most natural quartz sands the variation in compressibility is not that large, especially when compared with the possible variation of shear strength. This observation is particularly important when one considers that a large portion of natural sands encountered in the northern hemisphere consist predominantly of quartz and feldspars with small amounts of mica. These sands are similar to those tested in the calibration chamber studies (see Table 5). Thus it has been possible to use bearing capacity theories, in which the influence of compressibility is neglected, and produce reasonable estimates of friction angle. It is interesting to note that such theories will give conservatively low estimates of friction angle for more compressible sands (i.e., carbonate sands).

A review of the calibration chamber test results was carried out to compare the measured cone penetration resistance with measured friction angle from drained triaxial tests. The friction angle values were obtained from triaxial tests performed at confining stresses approximately equal to the horizontal effective stress in the calibration chamber before cone penetration (i.e., *in-situ* σ'_{ho}). The results of the comparison are shown on Fig. 14. Details of the sands used in the studies are given in Table 5. The scatter in the results illustrates the limited influence of soil compressibility. Also shown in Fig. 14 are the theoretical relationships proposed by Janbu and Senneset (1974) and Durgunoglu and Mitchell (1975). The Durgunoglu and Mitchell method includes the effect of *in-situ* horizontal stresses. The difference between the normally consolidated state, where $K_o = 1 - \sin\phi$, and the overconsolidated state (OCR = 6), where $K_o = 1 .O$, is less than 2 degrees, as shown on Fig. 14.

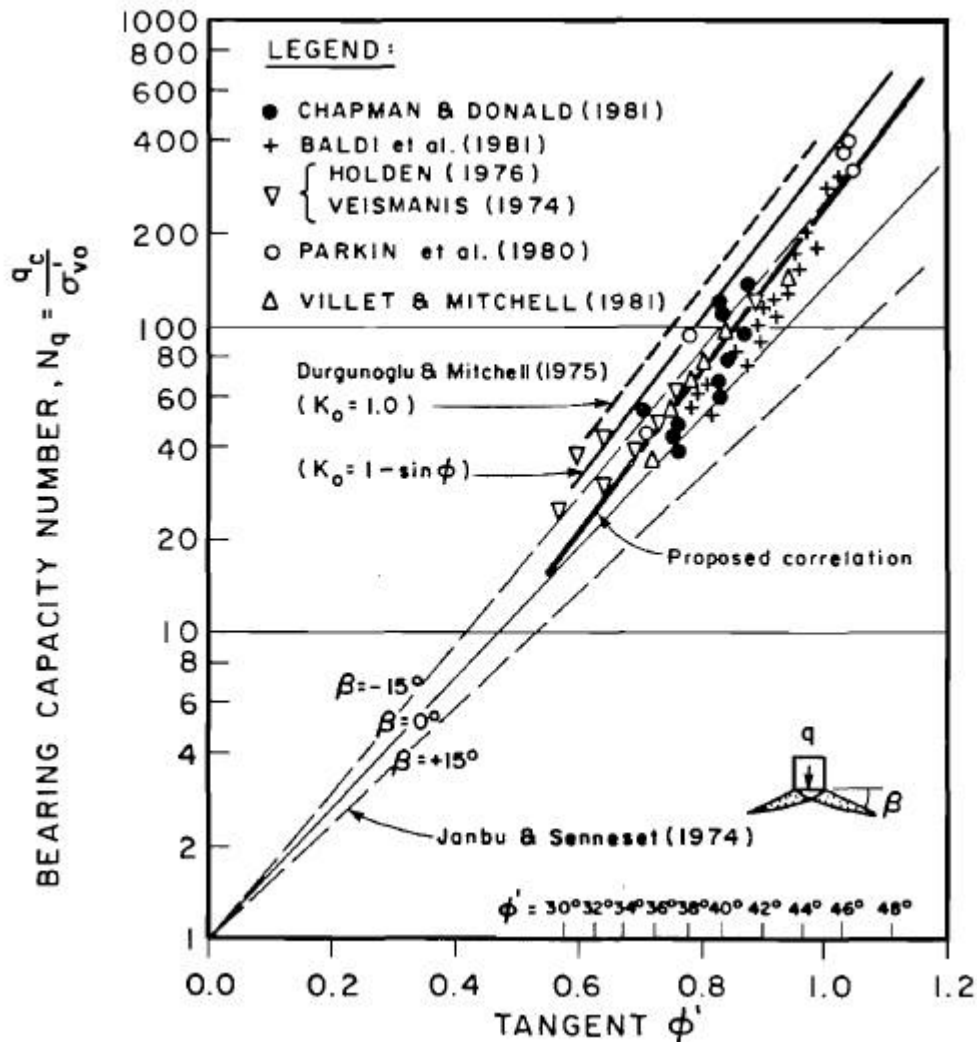


FIG. 14. Relationship between bearing capacity number and peak friction angle from large calibration chamber tests.

Since the solution by Janbu and Senneset (1974), for $\beta = 0$ (see Fig. 14), tends to slightly overestimate ϕ' and Durgunoglu and Mitchell tends to underestimate ϕ' , an average empirical relationship is proposed by the writers, as shown on Fig. 14. If the average relationship is taken, a useful design chart for estimation of friction angle from cone penetration resistance can be developed, as shown in Fig. 15. The proposed chart in Fig. 15 can be expected to provide reasonable estimates of friction angle for normally consolidated, moderately incompressible, predominantly quartz sands, similar to those used in the chamber studies. For highly compressible sands, the chart would tend to predict conservatively low friction angles. Durgunoglu and Mitchell's theory shows that there is little change in predicted friction angle for relatively large changes in stress history. It is important to note that the friction angle predicted from Fig. 15 is related to the *in-situ* initial horizontal stress level before cone penetration.

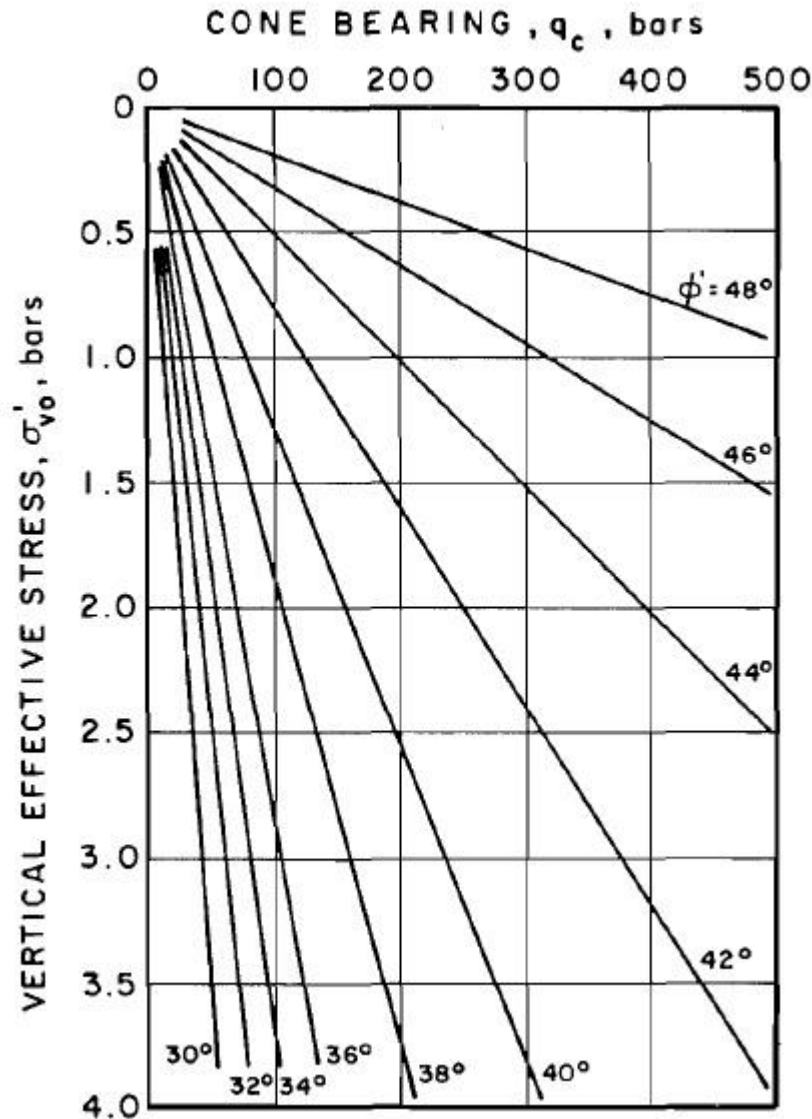


FIG. 15. Proposed correlations between cone bearing and peak friction angle for uncemented, quartz sands.

It is interesting to note that the friction ratio for sands increases with increasing compressibility. Many compressible carbonate sands have friction ratios as high as 3% (Joustra and de Gijt 1982), whereas typical incompressible quartz sands have friction ratios of about 0.5%. Thus, the presence of compressible sands may be identified using the friction ratio.

The author recommends that, for sands where the friction ratio is about 0.5%, the peak friction angle can be estimated using Fig. 15. In overconsolidated sands, Fig. 15 may slightly overestimate the friction angle (see Fig. 14). Care should be exercised in interbedded deposits where the cone resistance may not have reached the full value within a layer.

4.4.3 Deformation characteristics of sand

4.4.3.1 Constrained modulus

As already discussed, the cone penetration resistance in sand is a complex function of both strength and deformation properties. Hence, no generally applicable analytical solution for cone resistance as a function of deformation modulus is available. Instead, many empirical

correlations between cone resistance and deformation modulus have been established. Mitchell and Gardner (1975) made a comprehensive review of the existing correlations for sand.

The simple fact that soil is not a linear elastic material makes the assumption of a constant modulus unrealistic. This is further complicated by the fact that many of the correlations were derived from non-one-dimensional loading conditions for which "elastic" solutions were applied to back-figure a modulus. Thus, reasonable agreement can be expected only if the required problem involves similar boundary conditions and the same theoretical method is reapplied. Schmertmann's (1970) CPT method for predicting settlements in sand under spread foundations is a typical example. Schmertmann applied his strain influence elastic theory to analyse the results of screw plate tests. An equivalent Young's modulus (E_s) was calculated using a secant slope over the 1 bar to 3 bars (1 tsf to 3 tsf) increment of plate loading. This interval was chosen principally because real footing pressures commonly fall within this interval. Thus, Schmertmann's design method, where $E_s = 29$, can be expected to produce good results if the proposed design problem has similar loading conditions to the screw plate (i.e., circular spread footing loaded from 1 to 3 bars) and the same strain influence theory is reapplied.

4.4.3.2 Young's modulus

A common problem, however, appears to be the use of the one-dimensional constrained modulus (M_c) applied to non-one-dimensional loading conditions. For non-one-dimensional cases an equivalent Young's modulus, as suggested by Schmertmann (1970), would appear to be a more logical parameter. A review, performed by the writers, of the calibration chamber results (Baldi et al. 1981) provides a relationship between the drained secant Young's modulus at the 50 and 25% failure stresses, E_{50} and E_{25} , respectively, and cone resistance, q_c , for different levels of vertical effective stress (Fig. 16). Since the overall safety factor against bearing capacity failure is usually around 4 for foundations on sand, the designer is usually interested in a Young's modulus for an average mobilized stress level around 25% of the failure stress. Thus, the calibration chamber results on normally consolidated sand give values of E_{25}/q_c varying between 1.5 and 3.0 which are in good agreement with the recommended value of 2 by Schmertmann (1970) for computation of settlements of shallow foundations on sand. Schmertmann (1978a) has changed the value to 2.5 and 3.0 to allow for the variation of shape factors for square and strip footings, respectively. A careful review of Fig. 16 shows that in Schmertmann's study the load increment of 1-3 bars was probably closer to the 50% failure stress level for loose to medium dense sands and closer to the 25% stress level for medium dense to dense sands. For very dense sands the load increment (1-3 bars) was only a small percentage of the failure stress.

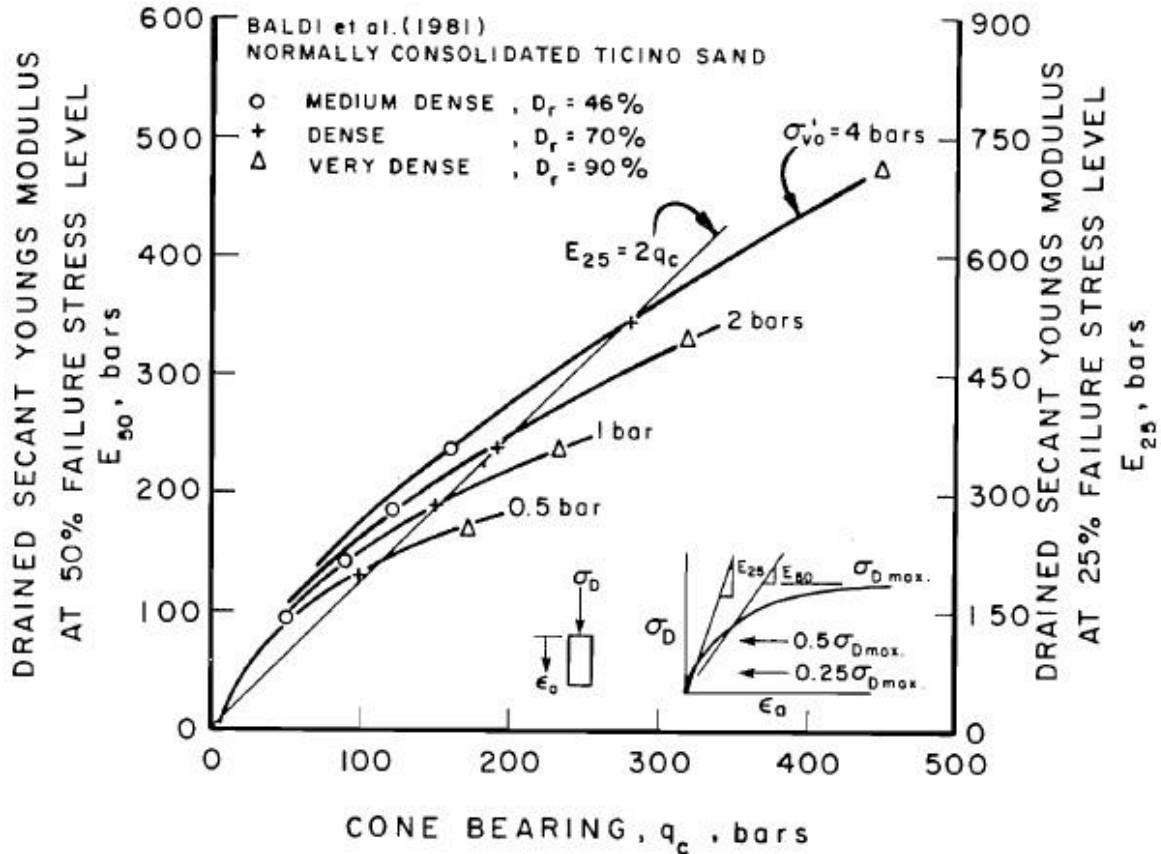


FIG. 16. Relationship between cone bearing and drained Young's modulus for normally consolidated, uncemented, quartz sands (based on data from Baldi *et al.* 1981).

The use of Fig. 16 may underestimate the *in-situ* Young's modulus because it is based on laboratory measured moduli using reconstituted samples. Many *in-situ* sand deposits have had some past stress history that can cause a significant increase in soil stiffness.

4.4.3.3 Shear modulus

A similar approach can be applied to develop a correlation between cone resistance and shear modulus, G , for sands. Extensive laboratory work has been conducted by several researchers (Seed and Idriss 1970; Hardin and Drnevich 1972) to relate dynamic shear modulus, G_{max} , to soil index properties. When expressed in the form :

$$G_{max} = k_G Pa \left(\frac{\sigma'_m}{Pa} \right)^{0.5}$$

Where k_G = modulus number,

σ'_m = mean effective stress, and

Pa = reference stress (i.e., $Pa = 1$ bar),

the empirical equations can be compared, as shown on Fig. 17. If the proposed relationship for k_G shown in Fig. 17 is combined with the relative density, cone resistance relationship developed by Baldi *et al.* (1981) a series of curves relating G_{max} , to q_c , can be developed.

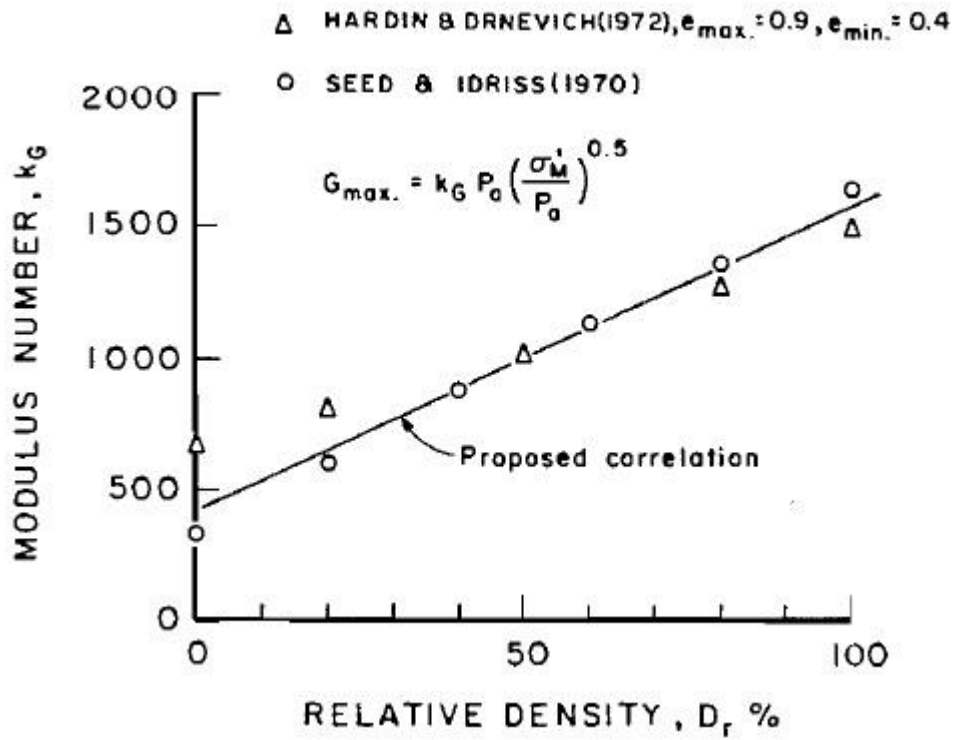


FIG. 17. Correlation between dynamic shear modulus number and relative density.

Once a correlation has been developed for the dynamic shear modulus it should be possible to estimate the shear modulus at any strain level by using the reduction curves suggested by Seed and Idriss (1970).

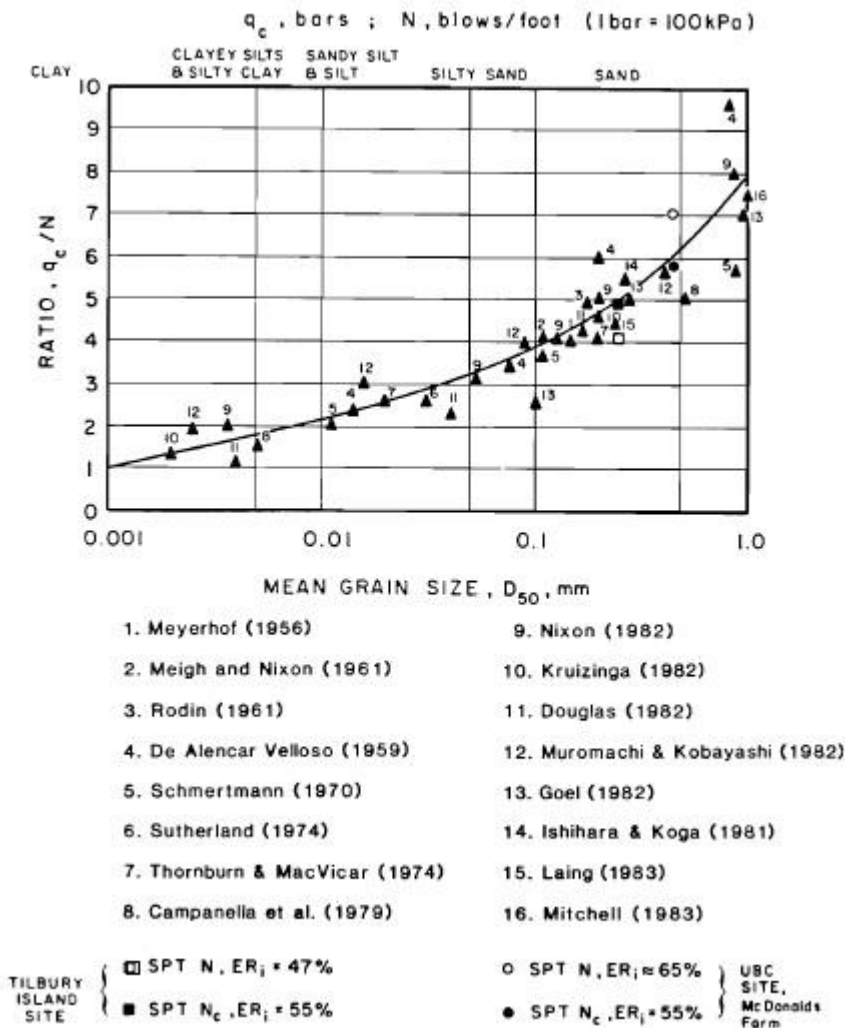


FIG. 18. Variation of q_c/N ratio with mean grain size (after Robertson *et al.* 1982).

The CPT is a fast, economic logging test that can provide continuous measurements of parameters like bearing, friction, and dynamic pore pressure. These parameters can be used to provide a preliminary estimate of geotechnical parameters based on the correlations presented here. The continuous soundings enable detailed site profiles to be developed and critical areas or zones to be identified. These critical areas may then require further testing with specific test methods that may include sampling and laboratory testing.

In interbedded deposits, it is important to remember that the cone resistance may not obtain its full value within a layer if the layer has a thickness less than about 70 cm.

The main correlations presented here are applicable to cone penetration test data in drained soil. The continuous measurement of pore pressure during cone penetration can significantly aid in the assessment of the applicability of these correlations to different soil types.

The correlations for density and moduli are approximate and should be used as a guide. The density correlation can be improved significantly if the compressibility of the sand can be accessed from the grain characteristics. It should be possible in the near future to quantify the grain characteristics by performing simple compressibility tests on disturbed bulk samples and comparing the results with similar tests on the chamber test sands.

The moduli correlations are likewise influenced by variations in grain characteristics because the parameter is a small strain measurement. The cone resistance, on the other hand, is a large strain measurement. Thus, the peak friction angle correlation appears less influenced by grain characteristics because it also is a large strain parameter. Therefore, when interpreting CPT data in sands more reliance can be placed on the estimated friction angle than density or moduli.

The equivalent SPT N value can be estimated from CPT data using Figs. 11 and 18. If local design correlations have been developed based on SPT data obtained using alternate procedures with resulting different average energy levels (i.e., **ERI** \neq **55%**), Fig. 18 should be adjusted to reflect local practice and experience.

4.5 Undrained soil

4.5.1 Undrained shear strength of clay

One of the earliest applications of the cone penetration test was in the evaluation of undrained shear strength (c_u) of clays (Schmertmann 1975). Comprehensive reviews of c_u evaluation from CPT data have been presented by Baligh *et al.* (1980), Jamiolkowski *et al.* (1982), and Lunne and Kleven (1981). Note that the undrained shear strength of clay is not a unique parameter and depends significantly on the type of test used, the rate of strain, and the orientation of the failure planes.

Estimates of c_u , from CPT results usually employ an equation of the following form:

$$q_c = c_u N_k + \sigma_o$$

where σ_o is the *in-situ* overburden total pressure and N_k is the cone factor.

The contribution of the total overburden pressure (σ_o) has been interpreted as either the *in-situ* vertical total stress (σ_{vo}), or the *in-situ* horizontal total stress (σ_{ho}), or the *in-situ* octahedral stress ($\sigma_{oct} = 1/3(\sigma_{vo} + 2 \sigma_{ho})$). Theoretical solutions for **N_k** have been based on bearing capacity theories (e.g., Meyerhof 1961) and more recently by use of cavity expansion theories (e.g., Ladanyi 1967; Vesic 1972). Baligh (1975) combined these two approaches in approximate form to obtain the results shown in Fig. 19. The rigidity index (I_r) is the ratio of the undrained shear modulus to undrained strength and the vertical axis gives **N_k** similar in form to [1]. Note the use of the *in-situ* total horizontal stress (σ_{ho}) rather than σ_{vo} and that the theory applies to the standard (Fugro) type electric cone. Figure 19 clearly demonstrates that **N_k** should not be a constant for all clays and for the case of the standard cone, where $2\theta = 60^\circ$, **N_k** = 16 ± 2 over the full range of likely I_r values.

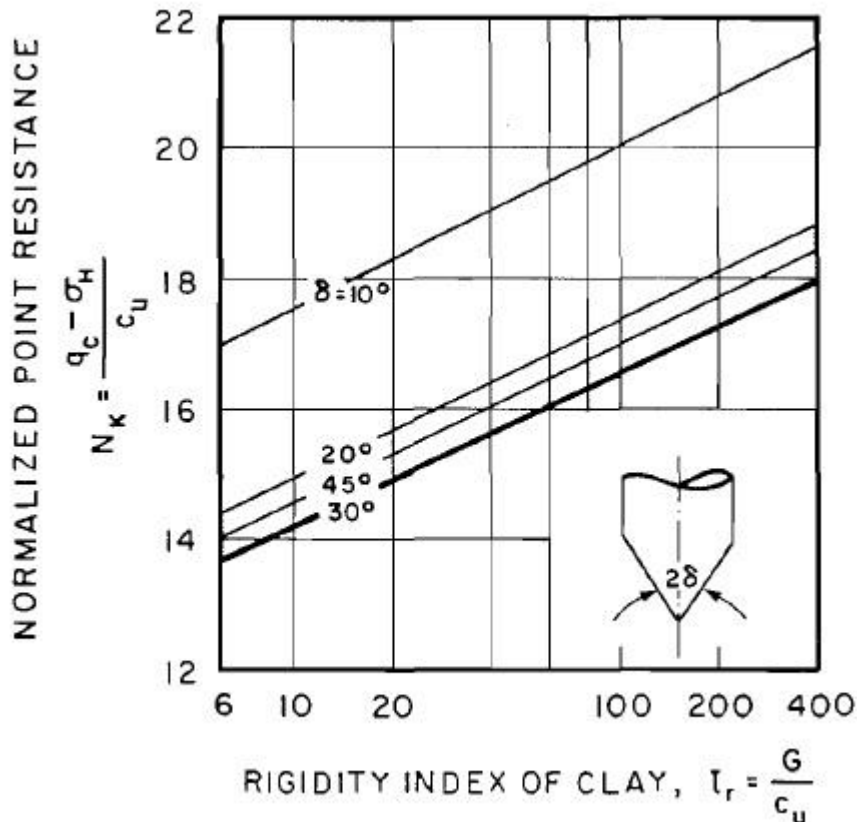


FIG. 19. Effect of rigidity index and cone angle on the penetration resistance of clay (after Baligh 1975).

The solution by Baligh (1975) involves several simplifying assumptions, such as neglect of undrained strength anisotropy and strain softening behavior. The former can be adequately approximated by using the average of the vertical and horizontal strengths. Neglecting strain-softening, on the other hand, can lead to a serious error for sensitive clays, Ladanyi (1972). Other factors such as cone type and rate of penetration may significantly affect the penetration resistance.

However, Nk is generally obtained from empirical correlations. The reference c_u is usually measured from field vane tests but sometimes half the unconfined compression test is used. The overburden pressure (σ_o) is usually taken as the in-situ total vertical stress (σ_{v0}) since the in-situ horizontal stress is usually not known.

Data presented by Lunne and Kleven (1981) show that for normally consolidated marine clays using corrected field vane strength (i.e., Bjerrum's (1972) correction for plasticity index), the cone factor Nk falls between 11 and 19 with an average of 15. These results were obtained using a standard (Fugro) type electric cone at a standard rate of penetration of 2 cm/s.

It is more difficult to establish similar correlations in stiff overconsolidated clays because of the effects of fabric and fissures on the response of the clay.

Robertson and Campanella suggest using [1] with an Nk value of 15 for preliminary assessment of c_u . For sensitive clays, the Nk value should be reduced to around 10 or less depending on the degree of sensitivity. The overburden pressure can be taken as the total vertical stress. With local experience individual correlations for Nk can be developed for specific clays. The writers also recommend that Nk be defined for a specific method of evaluating c_u such as by field vane corrected for PI, since c_u is not a unique soil parameter.

4.5.2 Drained shear strength of clay

Senneset et al. (1982) have suggested a method to determine the drained effective stress shear strength parameters (c' , ϕ'), from the cone penetration resistance and the measured total pore pressures. However, their method, as with any method for determining effective stress parameters from undrained cone penetration data, can be subject to serious problems. Any method of analyses must make assumptions as to the distribution of total stresses and pore pressures around the cone. Unfortunately, the distribution of stresses and pore pressures around a cone is extremely complex in all soils and has not adequately been modelled or measured to date except perhaps in soft normally consolidated clays. Also, an important problem, which is not identified by Senneset et al. (1982), is the location of the porous element, since different locations give different measured total pore pressures.

A detailed discussion about limitations of the theories relating to interpretation of CPT data in clays is given by Tavenas et al. (1982). As mentioned previously, the subject of applying an effective stress interpretation to CPT data in undrained soil is a high priority for intense study by many researchers.

4.6 Deformation characteristics of clay

4.6.1 Constrained modulus and compression index

Sanglerat *et al.* (1972) developed a comprehensive array of a values for different cohesive soil types with different cone resistance values. Mitchell and Gardner's (1975) summary of Sanglerat's a values are given in Table 6. Schmertmann developed a slightly more logical method that related the cu/σ_{vo}' ratio to the overconsolidation ratio (OCR) and then to the one-dimensional compression index of the soil, C_c as shown on Table 7.

The volumetric compressibility (m_v) and the compression index (C_c) are related by:

$$m_v = \frac{0.435 C_c}{(1 + e_o) \sigma_{va}}$$

Where e_o = initial void ratio and
 σ_{va} = average of initial and final stresses.

This method provide only a rough estimate of soil compressibility. The values by Schmertmann in Table 7 appear to give very conservative estimates of C_c . Additional data from Atterberg limit tests (PI) or undisturbed sampling and oedometer tests are required for more reliable estimates.

$$M = \frac{1}{m_v} = \alpha q_c$$

$q_c < 7$ bars	$3 < \alpha < 8$	} Clay of low plasticity (CL)
$7 < q_c < 20$ bars	$2 < \alpha < 5$	
$q_c > 20$ bars	$1 < \alpha < 2.5$	
$q_c > 20$ bars	$3 < \alpha < 6$	} Silts of low plasticity (ML)
$q_c < 20$ bars	$1 < \alpha < 3$	
$q_c < 20$ bars	$2 < \alpha < 6$	Highly plastic silts and clays (MH, CH)
$q_c < 12$ bars	$2 < \alpha < 8$	Organic silts (OL)
$q_c < 7$ bars:		} Peat and organic clay (Pt, OH)
$50 < w < 100$	$1.5 < \alpha < 4$	
$100 < w < 200$	$1 < \alpha < 1.5$	
$w > 200$	$0.4 < \alpha < 1$	

NOTE: 1 bar = 100 kPa.

TABLE 6. Estimation of constrained modulus, M (after Mitchell and Gardner 1975)

c_u/σ_{vo}'	Approx. OCR	$C_c/(1 + e_1)$
0–0.1	Less than 1	Greater than 0.4 (still consolidating)
0.1–0.25	1	0.4
0.26–0.50	1 to 1.5 (assume 1)	0.3
0.51–1.00	3	0.15
1–4	6	0.10
Over 4	Greater than 6	0.05

TABLE 7. Estimation of compression index, C_c , from c_u/σ_{vo}' ratio (after Schmertmann 1978)

4.6.2 Young's modulus

The estimation of undrained Young's modulus, E_u is usually made using empirical correlations with the undrained shear strength, c_u in the form:

$$E_u = n c_u$$

Where n is a constant that depends on stress level, overconsolidation ratio, clay sensitivity, and other factors (Ladd et al. 1977). As discussed earlier, because soil behavior is non-linear, the choice of relevant stress level is very important. Figure 20a presents data for normally consolidated soils from Ladd et al. (1977) that show the variation of the ratio E_u/c_u , with stress level for seven different cohesive soils ($1.5 < PI < 75$). Figure 20b shows the variation of E_u/c_u , with overconsolidation ratio (OCR) at two stress levels for the same soil types shown in Fig. 20a.

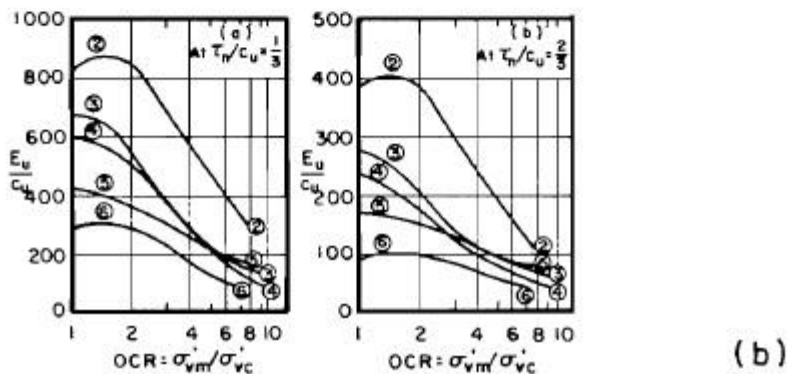
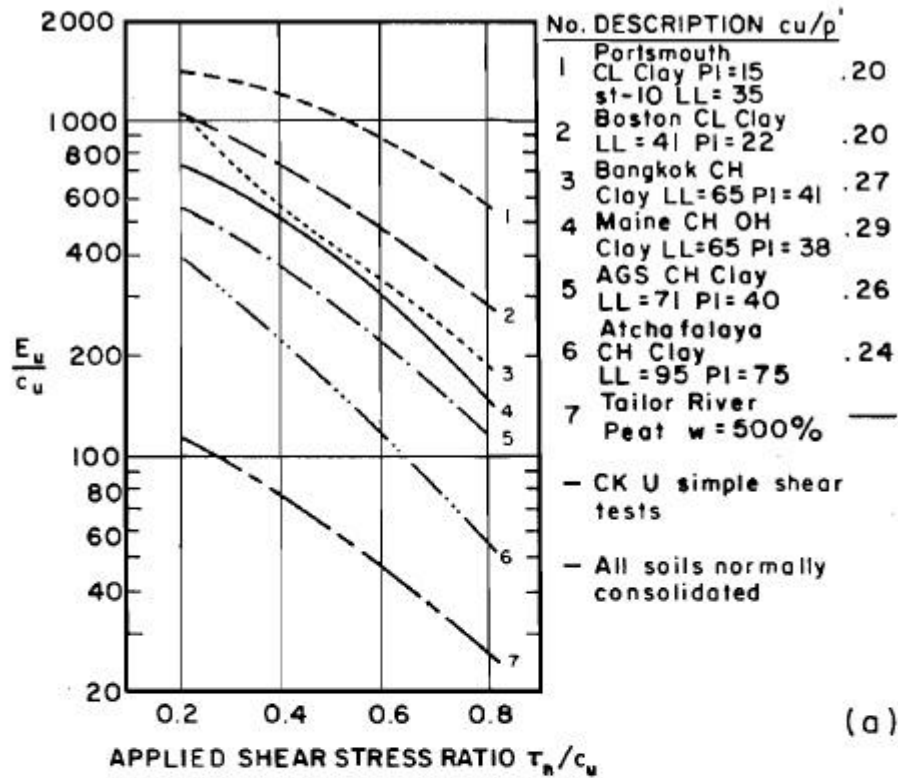


FIG. 20. Selection of soil stiffness ratio for clays (adapted from Ladd *et al.* 1977).

4.7 Piezometer cone

4.7.1 Pore pressure measurements

During cone penetration, soils tend to generate pore pressures. For sandy soils these pore pressures dissipate almost as fast as they are generated and the high cone resistance (q_c) in sands generally gives differential pore pressure ratios ($\Delta u/q_c$) of essentially zero. Silty and clayey soils, because of their relatively low permeability, can generate significant excess pore pressures during cone penetration.

The volume change characteristics are a direct measure of a soil's stress history. Normally consolidated silts and clays tend to develop large positive pore pressures during shear, whereas overconsolidated silts and clays tend to develop smaller positive or even negative pore pressures during shear. Therefore, if the permeability of a soil deposit is relatively low such that drainage during cone penetration is small, the excess pore pressure (u_o) may be a

direct measure of the soil deposit stress history. Thus, the excess pore pressure can give an excellent indication of both the volume change characteristics and relative permeability.

This logic applies equally well to sandy soils in that loose sands tend to generate positive pore pressures and dense sands negative pore pressures during shear. However, because of their relatively high permeability these pore pressures often dissipate as fast as they are generated and very little excess pore pressures are recorded.

Use of the pore pressure or differential pore pressure ratio is very dependent on the details of the cone design. The three significant aspects of cone design in relation to pore pressure measurements are:

- (i) Pore pressure element location.
- (ii) Unequal end area effects.
- (iii) Saturation of pore pressure measuring system.

4.7.2 Soil type and stress history

No clear correlation between soil type and pore pressure measurements during CPT has yet emerged. The recently suggested soil classification chart based on pore pressure and bearing data (Jones and Rust 1982) shows considerable promise. Most researchers believe that it is just a matter of time before sufficient field correlations are obtained to allow a soil classification chart to be developed. Unfortunately, this cannot be achieved until some standard is accepted for pore pressure element location. The writers believe that a location immediately behind the tip will soon become the standard.

Several researchers have recently shown that the pore pressure ratio can be related in a quantitative manner to OCR for clay soils (Smits 1982; Tumay et al. 1982). However, there are several factors that influence any correlation. The first is standardization of the cone design and pore pressure element location, since these have a significant influence on measured cone bearing, q_c and pore pressure, u_T . The second is standardization in definition of pore pressure ratio. The main definitions proposed to date are:

1. u_T/q_c (Baligh et al. 1981).
2. $\Delta u/q_T$ (Campanella and Robertson 1981).
3. $\Delta u/(q_c - u_o)$ (Smits 1982).
4. $\Delta u/(q_c - \sigma_{vo})$ (Senneset et al. 1982; Jones and Rust 1982; Jefferies and Funegard 1983).

The latter two definitions are important for offshore CPT work. *“The last definition (4) may become the standard provided the measured cone bearing, q_c is corrected for unequal area effects (q_T). If q_T is used, definitions (2) and (3) are very similar for shallow cone profiles where u_T is small”.* (Robertson & Campanella 1983)

The third factor that will influence any correlation between stress history and pore pressure ratio is related to the fact that even for normally consolidated clays, the measured excess pore pressure (Δu) is not unique for all clays. The excess pore pressures depend on the rigidity index ($I_r = G/cu = E/3cu$). Generally, for the same OCR, I_r increases with decreasing plasticity index, PI (see Fig. 20). High excess pore pressures are generally generated by normally consolidated soils with a high rigidity index (i.e., a low PI). The rigidity index also tends to decrease as OCR increases (see Fig. 20). Thus, it may not be clear if a low excess pore pressure is related to a normally consolidated soil with a high PI or a soil with a high OCR. Therefore, any future correlation is likely to contain some relation to PI, similar to that shown in Fig. 19. The sensitivity of a soil may also complicate any correlation.

The previous discussion should provide a guide as to how soil permeability, stress history, and cone design influence the measured pore pressures during cone penetration.

4.7.3 Coefficient of consolidation

Upon the arrest of steady penetration, excess pore pressures generated during cone penetration immediately start to dissipate. The rate of dissipation depends upon the coefficient of consolidation of the soil for a homogeneous deposit. Tavenas et al. (1982) have shown that for Canadian clays, the rate of pore pressure dissipation is mainly governed by the consolidation characteristics of the intact clay away from the probe. By monitoring the rate of dissipation of the excess pore pressure, an estimate of the coefficient of consolidation of the soil may be obtained. Several theoretical solutions are available to obtain the coefficient from dissipation of excess pore pressures generated by cavity expansion.

A summary of these solutions is shown on Fig. 21, which highlights the major differences between solutions. In order to compare results of the different solutions, they have been non-dimensionalized and shown in Fig. 21. Figure 21 shows the decay of excess pore pressure, Δu , plotted against a non-dimensional time factor, $T = cht/r^2$. Use of the time factor, T , allows a quick calculation for the coefficient of consolidation, cht/r^2 . The solution by Baligh and Levadoux (1980) and the cylindrical solution by Torstensson (1977) yield essentially the same result. The solution by Randolph and Wroth (1979) is not shown because of its similarity to that of Torstensson (1977).

(a)

Author	Cavity Type	Material Model	Initial Pore Pressure Distribution	Proposed Applications	Remarks
Baligh & Levadoux 1980	combined radial and spherical	non-linear Boston Blue clay	from F.E. studies using strain path method	consolidation characteristics	shows very small influence of spherical component of dissipation
Randolph & Wroth 1979	cylindrical	elastic-plastic	$\Delta u_i = 2 cu \ln\left(\frac{R}{r}\right)$ $\frac{R}{r_0} = (G/cu)^{1/2}$	consolidation around piles pressuremeter analysis	analytical solution
Soderberg 1962	cylindrical	elastic-plastic	$\frac{u}{r} = \frac{r_i}{r}$	consolidation around piles	
Torstensson 1977	cylindrical	elastic-plastic	$\Delta u_i = 2 cu \ln\left(\frac{R}{r}\right)$ $\frac{R}{r_0} = (G/cu)^{1/2}$	consolidation characteristics	proposes average of two results
Torstensson 1977	spherical	elastic-plastic	$\Delta u_i = 4 cu \ln\left(\frac{R}{r}\right)$ $\frac{R}{r_0} = (G/cu)^{1/3}$	consolidation characteristics vertical drains	

(b)

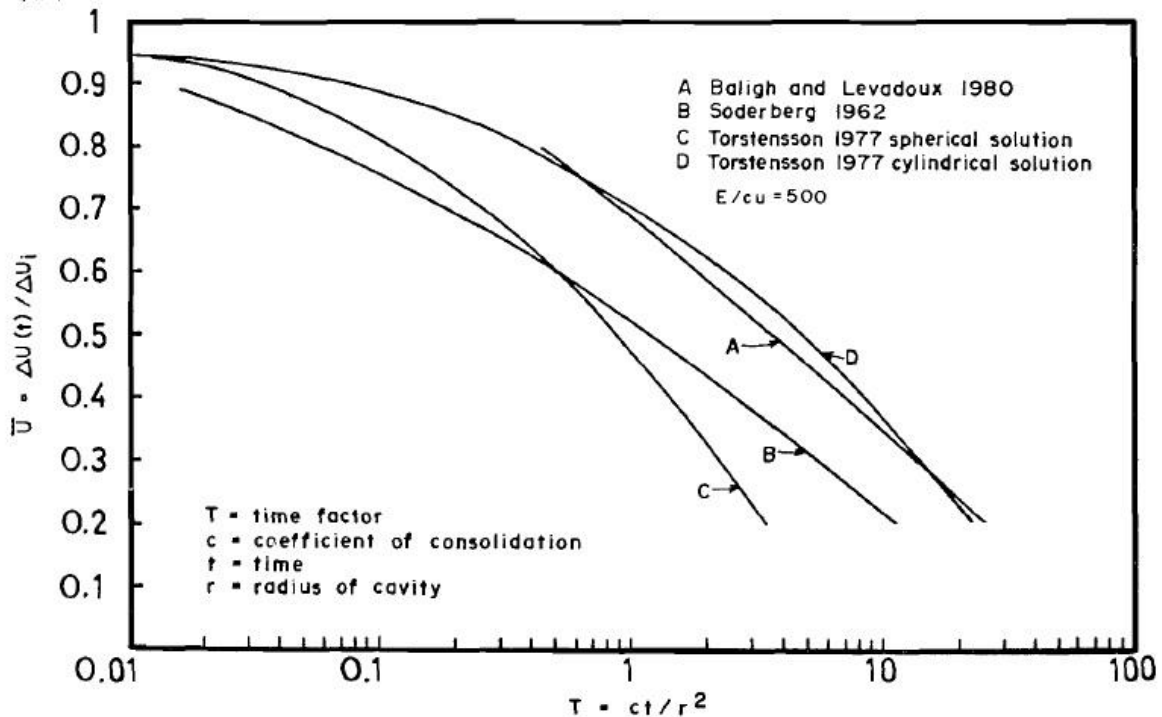


FIG. 21. Summary of existing solutions for pore pressure dissipation (adapted from Gillespie 1981).

4.7.4 Groundwater conditions

The addition of pore pressure measurements during cone testing provides a direct measure of groundwater conditions. The equilibrium piezometric profile can be measured directly during a stop in the penetration. Experience gained by the writers has shown this to be an extremely important feature for the piezometer cone for penetration in both drained and undrained soils. It has been common practice to obtain the height of water in a borehole but rarely are the groundwater conditions hydrostatic. Often there is a slight upward or downward gradient of water pressures resulting from overall regional groundwater conditions. The ability to measure equilibrium piezometric pressures during a stop in the penetration is useful for evaluating consolidation conditions or unusual hydraulic gradients. Identifying the actual groundwater conditions can be extremely valuable for investigations of dams, embankments, tailings disposal areas, slopes, and tidal areas.

The time required to reach full equilibrium pore pressure during a stop in penetration will depend on the soil permeability. For many investigations, it is sufficient to take equilibrium measurements at the end of the profile before pulling the rods and during rod breaks in any sand layers.

The cone penetration test has traditionally produced excellent continuous profiles of undrained shear strength. The major problem, however, with the determination of undrained shear strength (c_u) is the evaluation of the *in-situ* c , appropriate to the particular design problem, since the c , depends on the stress path followed during shear. The recommended general empirical correlation using an N_k value of 15 is related to the field vane strength. With local experience individual correlations for N_k can, and have been, developed for specific clays. With a measure of c , from cone data, it is possible to estimate overconsolidation ratio (OCR) with information on σ and PI.

The authors feel that the present state of the art is such that interpretation of CPT data has not yet reached a stage to allow reliable estimates of drained shear strength parameters from undrained cone penetration data. The concept of effective stress interpretation of CPT data is currently a topic of intense research. The recommended predictions of volume change characteristics, such as compression index, which are based on undrained cone resistance may be in large error. However, with local experience significantly improved correlations can, and have been, developed for specific soil types.

The addition of pore pressure measurements during cone penetration testing has significantly improved the interpretation of CPT data to obtain geotechnical parameters. The continuous measurement of pore pressures has also enhanced the electric penetrometer for stratigraphic logging of soil deposits.

Procedures are recommended to estimate the horizontal coefficient of consolidation from pore pressure dissipation curves. The soil permeability can also be estimated after the soil compressibility is estimated from cone bearing.

The major problem facing the quantitative interpretation of the pore pressure measurements during cone penetration is the location of the porous element. A secondary problem is the saturation of the pore pressure measuring system. A very convincing argument can be made to standardize the location to provide a wide range of applications but yet maintain a practical location for saturation and protection. The authors believe that the pore pressure element should be located immediately behind the cone tip to meet these requirements (campanella et al. 1983). It would appear logical that the overall cone design should also be made such that the porous element location can be changed in the field to allow for possible special soundings to be carried out to obtain specific pore pressure data. Although, in the

authors' experience, this is rarely necessary if the porous element is located immediately behind the tip.

If new pore pressure related correlations are to be developed and applied in engineering practice, a consensus is required as to the cone design and pore pressure element location. As a minimum, all pore pressure measurements from cone testing must clearly identify the location and size of the sensing element.

The authors also feel that the cone designs should be further standardized to include equal end area friction sleeves. Cone penetration resistance data should also be corrected, where possible, to total stress, $q_{t, i}$ is only when all cone data is unaffected by water pressure effects that significant improvements can be made in the interpretation of CPT data. All cones, especially piezometer cones, should be fully calibrated for cross-talk effects, i.e., all channels should be recorded during each calibration of load, all around pore pressure, and friction and that these should be reported for a given cone and its CPT data.

CHAPTER 5

SPT-CPT CORRELATIONS

5.1 General

A method for estimating equivalent Standard Penetration Test (SPT) N-values from Cone Penetration Test (CPT) data has been presented accounting for variations in soil grain size and SPT energy input. A discussion is given regarding the problems associated with the SPT and how these relate to SPT-CPT correlations. A historical review of SPT-CPT correlations is presented with additional recent data that include energy measurements during the SPT. An example is given to illustrate how the SPT data can be rationalized using energy corrections.

The Standard Penetration Test (SPT) is the most commonly used in situ test in North America. It is estimated that up to 90% of conventional foundation design in North America is accomplished by using the SPT N value (Kovacs et al 1981). Despite continued efforts to standardize the SPT procedure there are still continued problems associated with its repeatability and reliability. The Cone Penetration Test (CPT) is becoming increasingly more popular as an in-situ test for site investigation and geotechnical design. The authors believe this test is unequalled with respect to the delineation of stratigraphy and the continuous rapid measurement of parameters like cone bearing, q_c , and sleeve friction, f_s . The procedure and equipment of the quasi-static electric Cone Penetration Test (CPT) are easily standardized (1). The most significant advantages of the CPT are its simplicity, repeatability, accuracy and continuous record (Schmertmann, 1975, Schaap and Zuidberg, 1982). Significant improvements have been made recently in the interpretation of CPT data (Robertson and Campella, 1983). However, many geotechnical engineers have developed considerable experience with design based on local SPT correlations. With time, direct CPT design correlations will also be developed based on local experience and field observation (Thomburn, 1982, Goel, 1982). However, with the initial introduction of CPT data there is a need for reliable SPT-CPT correlations so that CPT data can be used in existing SPT data based design correlations. This chapter presents a review of available SPT-CPT correlations and reviews how these correlations could be applied to geotechnical design.

5.2 SPT – CPT Correlations

The SPT is still the most commonly used in situ test in North America. However, despite continued efforts to standardize the SPT procedure, there are still continued problems associated with its repeatability and reliability. However, many geotechnical engineers have developed considerable experience with design methods based on local SPT correlations. With time, direct CPT design correlations will also be developed based on local experience and field observation. However, with the initial introduction of CPT data, there is a need for better SPT-CPT correlations so that CPT data can be used in existing SPT data based design correlations.

A considerable number of studies (see Fig. 22) have taken place over the years to quantify the relationship between SPT N values and CPT cone bearing resistance, q_c . A wide range of q_c/N ratios have been published leading to much confusion. The variations in published q_c/N ratio can be rationalized somewhat by reviewing the derived q_c/N ratios as a function of mean grain size (D_{50}), as shown in Fig. 22. It is clear that the q_c/N ratio increases with increasing grain size. The scatter in results appears to increase with increasing grain size. This is not surprising since penetration in gravelly sand ($D_{50} = 1.0$ mm) is significantly

influenced by the larger gravel sized particles, not to mention the variability of delivered energy in the SPT data. Also sand deposits in general are usually stratified or non-homogeneous causing rapid variations in CPT tip resistance. There was also some difficulty in defining the D50 from some of the references.

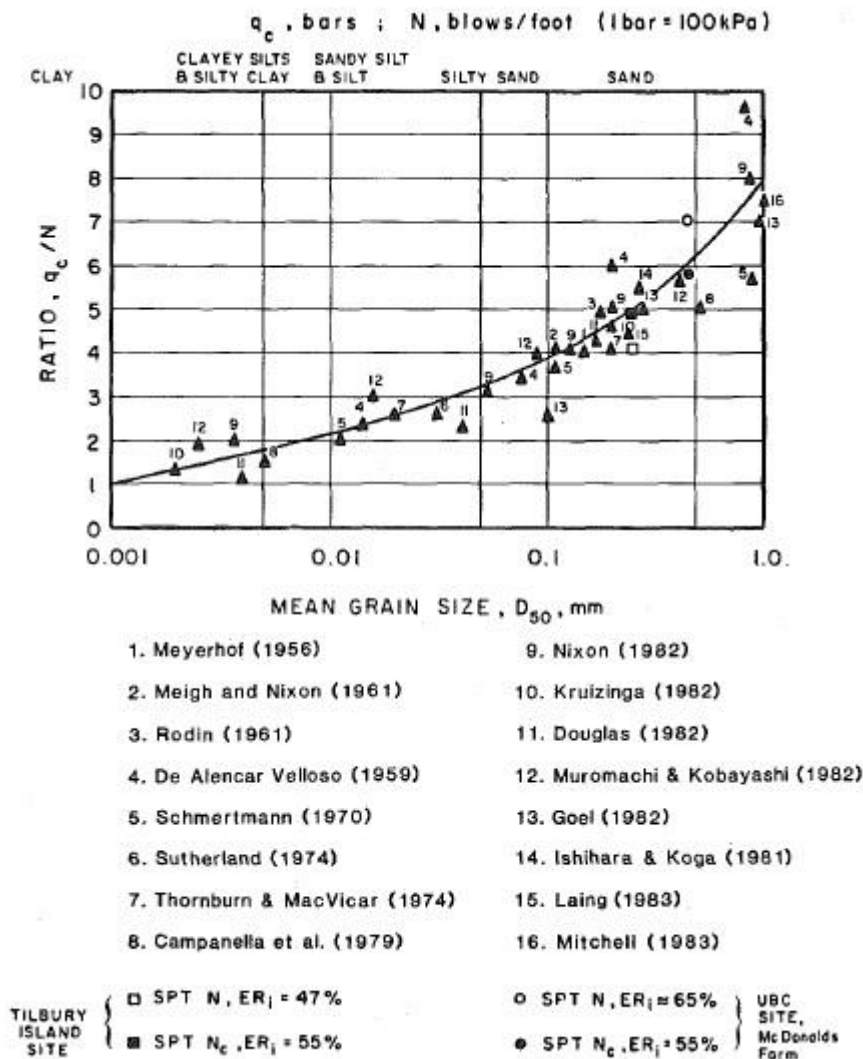


FIG. 22 Variation of q_c/N Ratio with Mean Grain Size

Work by Martins and Furtado (1963) and Douglas (1982) has shown that the q_c/N relationship also varies with SPT hammer type and to some extent with soil density. The q_c/N ratio is significantly affected by SPT hammer type since this effects the energy transmitted to the rods. Most of the data shown in Fig. 22 was obtained using the standard donut type hammer with a rope and cathead system. Schmertmann (1976) has also shown that q_c/N increases in sensitive clays.

Several sites in the Fraser River Delta area, near Vancouver, have provided additional SPT and CPT data. A site on Tilbury Island in the Fraser River Delta also provided energy measurements for the SPT data. The energy measurements were made using an SPT Force Calibrator produced by Binary Instruments Inc. (1982). The SPT calibrator is an analog computer with some digital control that measures the energy (ER_i) in the drill rods below the anvil, by means of a load cell, for each blow of the hammer. The device provides the means to measure the percent of 475 J (4,200 in.-lb) transmitted through the rods.

5.3 Energy Variations

The variation of the average measured energy ratio (ER_j) with SPT N value using the donut hammer for the sand is shown in Fig. 23. The average energy was 47% of the theoretical maximum. The theoretical maximum was calculated using the assumed 0.76 m fall of the hammer. Each data point represents one SPT test and is the average of individual energy measurements for each hammer blow. A total of over 1,000 individual energy measurements were made to compile Fig. 23. A considerable range in energy measurements was recorded with maximum and minimum values of 89% and 19%, respectively.

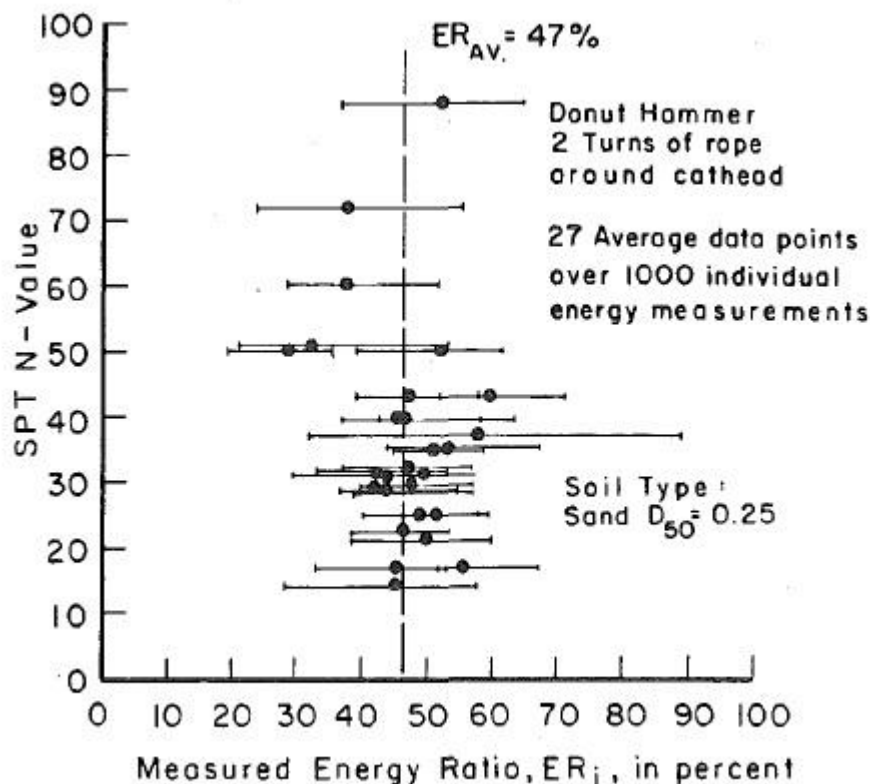


FIG. 23.—Measured Average Energy Ratio for SPT N Values Using Donut Hammer, Tilbury Island

The mean value of the qc/N ratio for the sand deposit was 4.2. Considerable scatter was found in the qc/N ratio for the overlying silt clay deposit due to the interbedded nature of the deposit. Therefore, only the data for the sand deposit are presented here.

The SPT N values for the donut hammer were corrected to an energy ratio level of 55%. The correction was carried out assuming a linear variation of N value with energy as suggested by Schmertmann and Palacios (1974). The resulting qc/N_c ratio, using corrected N values (N_c), was 4.9. The two ratios are also shown on Fig. 21.

5.4 SPT Hammer Comparison

Fig. 3 shows SPT results from one borehole (8217) where a safety and donut hammer were used for alternate tests at about 0.5 m spacing. Energy measurements were made for each hammer blow. The average overall energy ratio levels for the hammers were 43% and 62% for the donut and safety hammers, respectively. Fig. 3 shows clearly that the donut hammer SPT N values are consistently higher than the safety hammer N values. When both hammer N values are corrected to an energy ratio level of 55%, the apparent variation in SPT is rationalized and a consistent SPT N value profile is obtained. This is illustrated by the SPT profiles shown in Fig. 3. Using the corrected N_c values, considerably less scatter in the SPT results was observed.

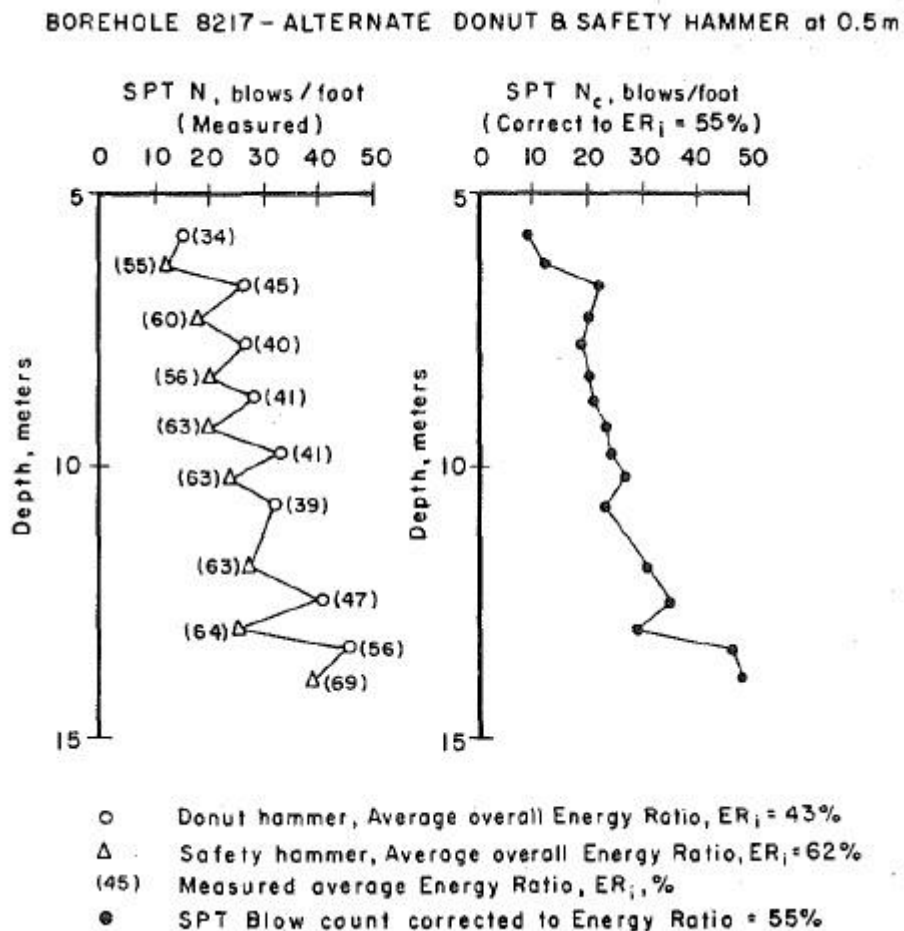


FIG. 24. Comparison of SPT N Values Using Alternate Donut and Safety Hammer with Energy Corrected N_c Values

Based on the studies shown by Kovacs and Salomone (1982) it would appear that the average energy level of the SPT data using the donut hammer from the Tilbury Island site was slightly on the low side for the 2 turns of the rope procedure. This is confirmed by the qc/N ratio of 4.2, which appears to be slightly lower than the historical average (Fig. 22). However, when the SPT N values are corrected to an energy level of 55% the qc/N_c ratio agrees remarkably well with the historical data.

5.5 Other Comparisons

Additional SPT and CPT data were obtained from the UBC research site for in-situ testing, which is located on an abandoned farm (McDonald's Farm) near the Vancouver International Airport. The site is located between the North Arm and Middle Arm of the Fraser River on the north side of the main Fraser River Delta, approximately 8 km west of the Tilbury Island site. Full details of the site and ground conditions are given by Campanella and Robertson (1981). The upper 2 m of the site consists of soft, compressible clays and silts. A sand deposit exists from 2 m to 13 m below ground surface. The sand has a medium to coarse grain size ($D_{50} = 0.45$ mm) with thin layers of medium to fine sand. Groundwater is approximately 1 m below existing ground surface.

SPT N values were obtained using a BBS-37A rotary drilling supplied and operated by the B.C. Ministry of Highways and Transportation. One turn of the rope around the cathead was used to operate the standard safety hammer. The hole was drilled using drilling mud and casing. Unfortunately, no energy measurements were made since the SPT calibrator was not available at that time. This rig and operator will be calibrated in the future. In the meantime, the work by Kovacs and Salomone (1982) would indicate that using the one turn of the rope around the cathead procedure and a safety hammer produces an energy level of 20% larger than using two turns of the rope. Therefore, the energy level can be assumed to be about 65%-70%.

The average qc/N ratio for the sand deposit ($D_{50} = 0.45$ mm) was 7, as shown on Fig. 22. If the N values are corrected to an energy level of 55% the qc/N_c ratio becomes 5.7, as shown on Fig. 22.

The high energy SPT data using 1 turn of the rope around the cathead produced a qc/N ratio significantly higher than the historical average. However, when the SPT N values were corrected to an energy level of 55%, the qc/N_c ratio again agrees remarkably well with the historical data.

Recent additional SPT and CPT data has been obtained from another site located in the Fraser River Delta area. The site will be referred to as the Jacombs Road site and consists of 4.5 m of organic sandy silts overlying about 15 m of medium to medium-fine sand. The average energy for the SPT tests was 56% with a mean value of qc/N in the sand deposit (average $D_{50} = 0.23$ mm) of 4.4. Correcting the N -values to an ER, = 55% gave no significant change in mean qc/N value. The values of 4.4 for q_j/N and 0.23 mm for D_{50} lie only slightly below the curve in Fig. 22 (Laing, 1983), therefore providing additional energy evidence supporting the historical data.

The above examples when seen in the context of Fig. 22 appear to indicate that the average energy ratio of 55% suggested by Schmertmann may represent the energy level associated with many of the SPT based empirical design correlations.

Fig. 22 can therefore be used to convert CPT data to equivalent SPT N values for 55% efficiency. To estimate the mean grain size from CPT data use can be made of the classification chart shown in Fig. 25. The classification chart in Fig. 25 has been adapted from the chart proposed by Douglas and Olsen (1981) for standard electric cones. The classification chart in Fig. 25 should be used as a guide to grain size. The author considers that the addition of pore pressure measurements during cone penetration can significantly improve the soil classification. For mechanical cone data use can be made of classification charts by Schmertmann (1978), Searle (1979) or Muromachi and Atsuta (1980). The friction ratio using the UBC electric cone for the sand deposits at Tilbury Island and McDonald's Farm was about 0.5% with an average qc of around 135 bars (13.5 MPa).

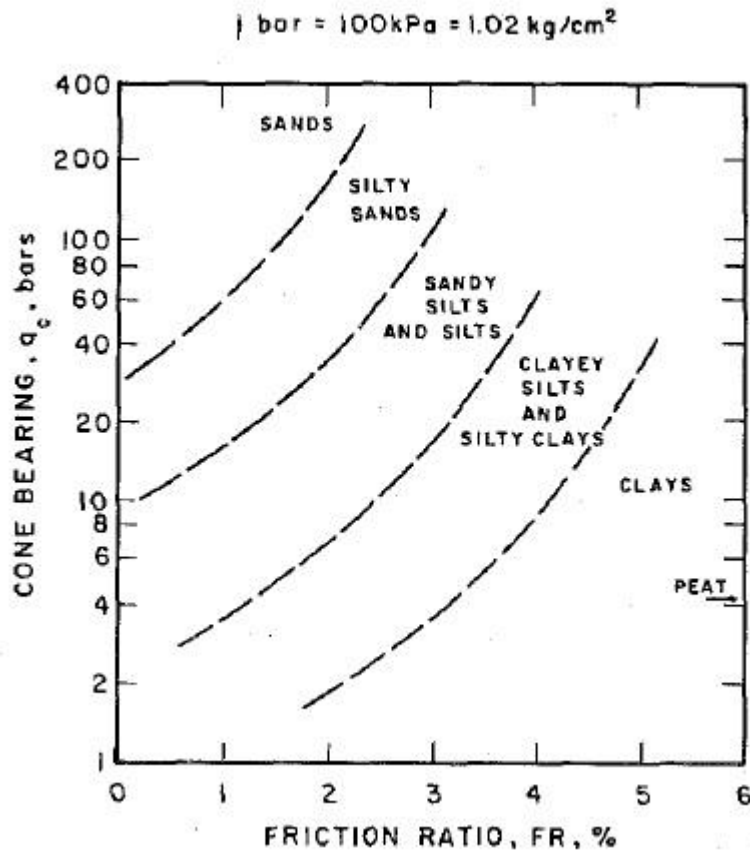


FIG. 25 Simplified Soil Classification Chart for Standard Electric Friction Cone.
(Adapted from Douglas and Olsen, 1981)

If local design correlations have been developed based on SPT data obtained using alternative procedures such as a trip hammer or procedures other than the rope and cathead technique, the q_c/N ratios shown in Fig. 22 may be slightly in error. If a trip hammer was used it is likely that the energy level would be higher than the average 55% level by a factor of about 1.4 (Douglas, 1981). Therefore, q_c/N ratios would be slightly higher than those shown in Fig. 22.

It is important that geotechnical engineers are aware of the average energy level on which local SPT design correlations have been based. It is hoped that this paper will provide some guidelines as to how to correlate CPT data to equivalent SPT N-value based on an understanding of SPT energy level.

CHAPTER 6

CONCLUSIONS

Foundation engineering is a complex art that encompasses many areas of subsurface engineering. The correct application of *in situ* test methods to this "art" is also complex. There are many different *in situ* test methods, each with its own advantages and disadvantages. The correct selection of the appropriate *in situ* test method is governed by the geological conditions, project requirements, type of construction, and method of analysis intended for design.

The two main methods for the application of any *in situ* test data to foundation design involve either use of the physical properties interpreted from the *in situ* test in some form of analysis or use of some direct or empirical correlation from the *in situ* test measurement to the design value. The successful application of the physical properties depends on the degree of reliability in both the interpretation of the *in situ* test results and the method of analysis used. Therefore, no generalized degree of reliability can be stated for an application for a particular *in situ* test method.

Both SPT and CPT test can elicit incorrect data under particular subsurface conditions that are often overlooked, depending on the experience of field personnel operating or logging the tests.

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. Under most conditions, the joint employment of SPT and CPT together has the greatest potential for characterizing sites correctly.

The SPT allows a firsthand look at subsurface materials (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.

The static cone test is most successful in soft or loose soils like silty sands, loose sands, layered deposits of sands, silts and clays as well as in clayey deposits. In areas where some information regarding the foundation strata is already available, the use of test piles and loading tests thereof can be avoided by conducting static cone penetration tests. Experience indicates that a complete static cone penetration test up to depths of 15 to 20 m can be completed in a day with manual operations of the equipment, making it one of the inexpensive and fast methods of sounding available for investigation.

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