GUIDE TO IN-SITU TESTING

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Engineering Units

Multiples

Micro (μ)	$= 10^{-6}$
Milli (m)	$= 10^{-3}$
Kilo (k)	$= 10^{+3}$
Mega (M)	$= 10^{+6}$

Imperial units			**SI units**	
Length	feet	(ft)	meter	(m)
Area	square feet	(ft^2)	square meter	(m^2)
Force	pounds	(p)	Newton	(N)
Pressure/Stress	pounds/foot^2	(psf)	Pascal	(Pa) = (N/m^2)

Multiple units				
Length	inches	(in)	millimeter	(mm)
Area	square feet	(ft2)	square millimeter	(mm^2)
Force	ton	(t)	kilonewton	(kN)
Pressure/Stress	pounds/inch^2	(psi)	kilonewton/meter^2	(kPa)
	tons/foot^2	(tsf)	meganewton/meter^2	(MPa)

Conversion factors

Force:	1 ton	= 9.8 kN	
	1 kg	= 9.8 N	
Pressure/Stress	1kg/cm^2	= 100 kPa	= 100 kN/m^2 = 1 bar
	1 tsf	= 96 kPa	(~100 kPa = 0.1 MPa)
	1 t/m^2	~ 10 kPa	
	14.5 psi	= 100 kPa	
	2.31 foot of water	= 1 psi	1 meter of water = 10 kPa

Derived values from CPT

Friction ratio:	$R_f = (f_s/q_t) \times 100\%$
Corrected cone resistance:	$q_t = q_c + u_2(1\text{-}a)$
Net cone resistance:	$q_n = q_t - \sigma_{vo}$
Excess pore pressure:	$\Delta u = u_2 - u_0$
Pore pressure ratio:	$Bq = \Delta u / q_n$
Normalized excess pore pressure	$U = (u_t - u_0) / (u_i - u_0)$

where: u_t is the pore pressure at time *t* in a dissipation test, and
u_i is the initial pore pressure at the start of the dissipation test

Guide to
In-Situ Testing

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Glossary

This glossary contains the most commonly used terms related to CPT and are presented in alphabetical order.

CPT
: Cone penetration test.

CPTU
: Cone penetration test with pore pressure measurement – piezocone test.

Cone
: The part of the cone penetrometer on which the cone resistance is measured.

Cone penetrometer
: The assembly containing the cone, friction sleeve, and any other sensors and measuring systems, as well as the connections to the push rods.

Cone resistance, q_c
: The force acting on the cone, Q_c, divided by the projected area of the cone, A_c.

$$q_c = Q_c / A_c$$

Corrected cone resistance, q_t
: The cone resistance q_c corrected for pore water effects.

$$q_t = q_c + u_2(1- a_n)$$

Data acquisition system
: The system used to record the measurements made by the cone penetrometer.

Dissipation test
: A test when the decay of the pore pressure is monitored during a pause in penetration.

Filter element
: The porous element inserted into the cone penetrometer to allow transmission of pore water pressure to the pore pressure sensor, while maintaining the correct dimensions of the cone penetrometer.

Friction ratio, R_f
: The ratio, expressed as a percentage, of the sleeve friction, f_s, to the cone resistance, q_t, both measured at the same depth.

$$R_f = (f_s/q_t) \times 100\%$$

Friction reducer

A local enlargement on the push rods, placed a short distance above the cone penetrometer, to reduce the friction on the push rods.

Friction sleeve

The section of the cone penetrometer upon which the sleeve friction is measured.

Normalized cone resistance, Q_t

The cone resistance expressed in a non-dimensional form and taking account of the in-situ vertical stresses.

$$Q_t = (q_t - \sigma_{vo}) / \sigma'_{vo}$$

Net cone resistance, q_n

The corrected cone resistance minus the vertical total stress.

$$q_n = q_t - \sigma_{vo}$$

Excess pore pressure (or net pore pressure), Δu

The measured pore pressure less the in-situ equilibrium pore pressure.

$$\Delta u = u_2 - u_0$$

Pore pressure

The pore pressure generated during cone penetration and measured by a pore pressure sensor:

u_1 when measured on the cone

u_2 when measured just behind the cone, and,

u_3 when measured just behind the friction sleeve.

Pore pressure ratio, B_q

The net pore pressure normalized with respect to the net cone resistance.

$$B_q = \Delta u / q_n$$

Push rods

Thick-walled tubes used to advance the cone penetrometer

Push (thrust) machine

The equipment used to push the cone penetrometer and push rods into the ground.

Sleeve friction, f_s

The frictional force acting on the friction sleeve, F_s, divided by its surface area, A_s.

$$f_s = F_s / A_s$$

Introduction

The purpose of this guide is to provide a concise summary on in-situ testing and its application to geotechnical engineering. The aim of in-situ testing is to define soil stratigraphy and obtain measurements of soil response and geotechnical parameters.

The common in-situ tests include: Standard Penetration Test (SPT), Cone Penetration Test (CPT), Flat Plate Dilatometer (DMT), Field Vane Test (FVT) and Pressuremeter Test (PMT). Each test applies different loading schemes to measure the corresponding soil response in an attempt to evaluate material characteristics such as strength and stiffness. Boreholes are required for the SPT and some versions of the PMT and FVT. For the CPT and DMT no boreholes are needed and the term 'direct-push' is often used. An advantage of direct-push technology is that no cuttings are generated. However, a disadvantage of the direct-push method is that hard cemented layers, bedrock, and some gravel layers can prevent further penetration.

The guide has an emphasis on the Cone Penetration Test (CPT) and the Standard Penetration Test (SPT), since these are the most commonly used in-situ tests in North America. The section on the CPT is a supplement to the book 'CPT in Geotechnical Practice' by Lunne, Robertson and Powell (1997) and is applicable primarily to data obtained using a standard electronic cone with a 60-degree apex angle and a diameter of either 35.7 mm or 43.7 mm (10 cm^2 or 15 cm^2 cross-sectional area). The section on the SPT is applicable to data obtained following ASTM standard D1586-99.

A list of useful references is included at the end of this guide.

Risk Based Site Characterization

Risk and uncertainty are characteristics of the ground and are never fully eliminated. The extent and level of an investigation should be based on the risk of the project. Risk analysis answers three basic questions, namely:

- What can go wrong?
- How likely is it?
- What are the consequences?

Projects can be classified into low, moderate or high risk projects, depending on the probability of the associated hazards occurring and the associated consequences. Low-risk projects could be projects with few hazards, low probability of occurrence, and limited consequences, whereas high risk projects could be projects with many hazards, a high probability of occurrence, and severe consequences. Table 1 shows a generalized flow chart to illustrate the likely geotechnical ground investigation approach associated with low risk, moderate risk and high risk projects.

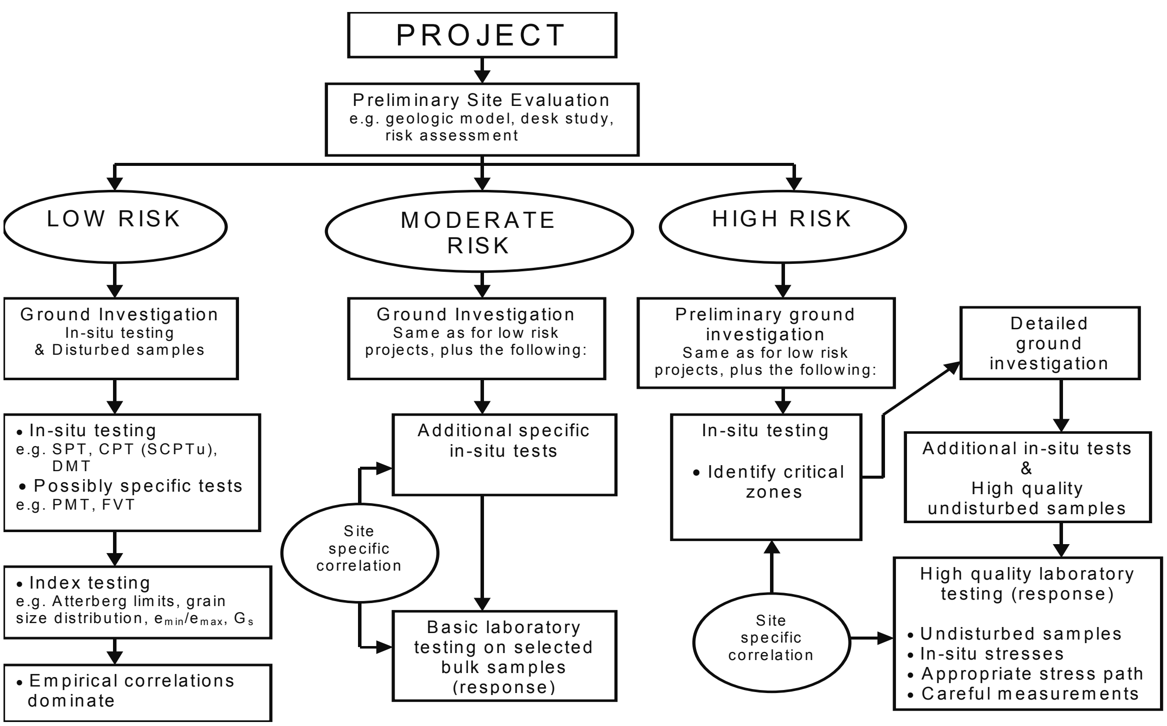


Table 1 Risk-based flowchart for site characterization.

In-Situ Tests

The objectives of any subsurface investigation are to determine the following:

- Nature and sequence of the subsurface strata (geologic regime)
- Groundwater conditions (hydrologic regime)
- Physical and mechanical properties of the subsurface strata

For geo-environmental site investigations where contaminants are possible, the above objectives have the additional requirement to determine:

- Distribution and composition of contaminants

The above requirements are a function of the proposed project and the associated risks. An ideal investigation program should include a mix of field and laboratory tests depending on the risk of the project.

Table 2 presents a partial list of the major in-situ tests and their perceived applicability for use in different ground conditions.

Group	**Device**	**Soil Parameters**														**Ground Type**					
		Soil type	Profile	u	$*\varphi'$	s_u	I_D	m_v	c_v	k	G_O	σ_h	OCR	σ-ε	Hard rock	Soft Rock	Gravel	Sand	Silt	Clay	Peat
Penetrometers	Dynamic	C	B	-	C	C	C	-	-	-	C	-	C	-	-	C	B	A	B	B	B
	Mechanical	B	A/B	-	C	C	B	C	-	-	C	C	C	-	-	C	C	A	A	A	A
	Electric (CPT)	B	A	-	C	B	A/B	C	-	-	B	B/C	B	-	-	C	C	A	A	A	A
	Peizocone (CPT)	A	A	A	B	B	A/B	B	A/B	B	B	B/C	B	C	-	C	-	A	A	A	A
	Seismic (SCPT/SCPTU)	A	A	A	B	A/B	A/B	B	A/B	B	A	B	B	B	-	C	-	A	A	A	A
	Flat dilatometer (DMT)	B	A	C	B	B	C	B	-	-	B	B	B	C	C	C	-	A	A	A	A
	Standard penetration Test (SPT)	A	B	-	C	C	B	-	-	-	C	-	C	-	-	C	B	A	A	A	A
	Resisitivity probe	B	B	-	B	C	A	C	-	-	-	-	-	-	-	C	-	A	A	A	A
Pressuremeters	Pre-bored (PBP)	B	B	-	C	B	C	B	C	-	B	C	C	C	A	A	B	B	B	A	B
	Self boring (SBP)	B	B	A[1]	B	B	B	B	A[1]	B	A[2]	-	B	-	B	B	A	B	B	A	B
	Full displacement (FDP)	B	B	-	C	B	C	C	C	-	A[2]	C	C	C	-	C	B	B	B	A	A
Others	Vane	B	C	-	-	A	-	-	-	-	-	-	B/C	B	-	-	-	-	-	A	B
	Plate load	C	-	-	C	B	B	B	C	C	A	C	B	B	B	A	B	B	B	A	A
	Screw Plate	C	C	-	C	B	B	B	C	C	A	C	B	-	-	-	-	A	A	A	A
	Borehole permeability	C	-	A	-	-	-	-	B	A	-	-	-	-	A	A	A	A	A	A	B
	Hydraulic fracture	-	-	B	-	-	-	-	C	C	-	B	-	-	B	-	-	-	-	A	C
	Crosshole/downhole/surface seismic	C	C	-	-	-	-	-	-	-	A	-	B	-	A	A	A	A	A	A	A

Applicability: A = high, B = moderate, C = low, - = none
$*\varphi'$ = Will depend on soil type; [1] = Only when pore pressure sensor fitted; [2] = Only when displacement sensor fitted.
Soil parameter definitions: u = in situ static pore pressure, φ' = effective internal friction angle; s_u = undrained shear strength; I_D = density index; m_v = constrained modulus; c_v = coefficient of consolidation; K = coefficient of permeability; G_O = shear modulus at small strains; σ_h = horizontal stress; OCR = overconsolidation ratio; σ-ε = stress-strain relationship

Table 2 The applicability and usefulness of in-situ tests
(Lunne, Robertson & Powell, 1997)

Cone Penetration Test (CPT)

The Cone Penetration Test (CPT) and it's enhanced versions (i.e. piezocone-CPTU and seismic-SCPT) have extensive applications in a wide range of soils. Although the CPT is limited primarily to softer soils, with modern larger pushing equipment and more robust cones, the CPT can be performed in stiff to very stiff soils, and in some cases soft rock.

Advantages of CPT:

- Fast and continuous profiling
- Repeatable and reliable data (not operator-dependent)
- Economical and productive
- Strong theoretical basis for interpretation

Disadvantage of CPT:

- High capital investment
- Requires skilled operators
- No soil sample
- Penetration can be restricted in gravel/cemented layers

The Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is used as an indicator of relative density and stiffness of granular soils as well as an indicator of consistency in a wide range of other ground. Methods have been developed to apply SPT results to a wide range of geotechnical applications including shallow and deep foundations and the assessment of liquefaction potential.

Advantages of SPT:

- Simple and rugged
- Low cost
- Obtain a sample
- Can be performed in most soil types
- Available throughout the U.S.

Disadvantages of SPT:

- Disturbed sample (index tests only)
- Crude number (*N* value)
- Not applicable in soft clays and silts
- High variability and uncertainty.

The Field Vane Test (FVT)

The field vane test (FVT) is used to evaluate the undrained shear strength (s_{uv}) of soft to stiff clays and silts. Both peak and remolded strengths can be measured and their ratio is termed soil sensitivity (S_t).

Advantages of FVT:

- Simple test and equipment
- Long history of use in practice

Disadvantages of FVT:

- Limited application to soft to stiff clays and silts
- Slow and time-consuming
- Raw s_{uv} values need (empirical) correction
- Can be affected by sand lenses and seams.

The Flat Plate Dilatometer Test (DMT)

The flat plate dilatometer test (DMT) can be used to estimate a wide range of geotechnical parameters in primarily softer soils.

Advantages of DMT:

- Simple and robust
- Repeatable and reliable data (not operator-dependent)
- Economical

Disadvantage of DMT:

- Difficult to push into dense and hard materials
- Weak theoretical basis for interpretation
- No soil sample
- Penetration can be restricted in gravel/cemented layers

The Pressuremeter Test (PMT)

The pressuremeter test can be used to evaluate the stress-strain response of a wide range of soils and rock. There are three basic types of pressuremeter devices, Pre-bored, Self-bored and Full-displacement, each with different abilities and challenges. In general they have the following advantages and disadvantages:

Advantages of PMT:

- Strong theoretical basis for interpretation
- Tests large volume of ground

Disadvantages of PMT:

- Complicated equipment and procedures
- Requires skilled operator
- Time consuming and expensive
- Equipment can be easily damaged

Cone Penetration Test (CPT)

Introduction

In the Cone Penetration Test (CPT), a cone on the end of a series of rods is pushed into the ground at a constant rate and continuous measurements are made of the resistance to penetration of the cone and of a surface sleeve. Figure 1 illustrates the main terminology regarding cone penetrometers.

The total force acting on the cone, Q_c, divided by the projected area of the cone, A_c, produces the cone resistance, q_c. The total force acting on the friction sleeve, F_s, divided by the surface area of the friction sleeve, A_s, produces the sleeve friction, f_s. In a piezocone, pore pressure is also measured, as shown in Figure 1.

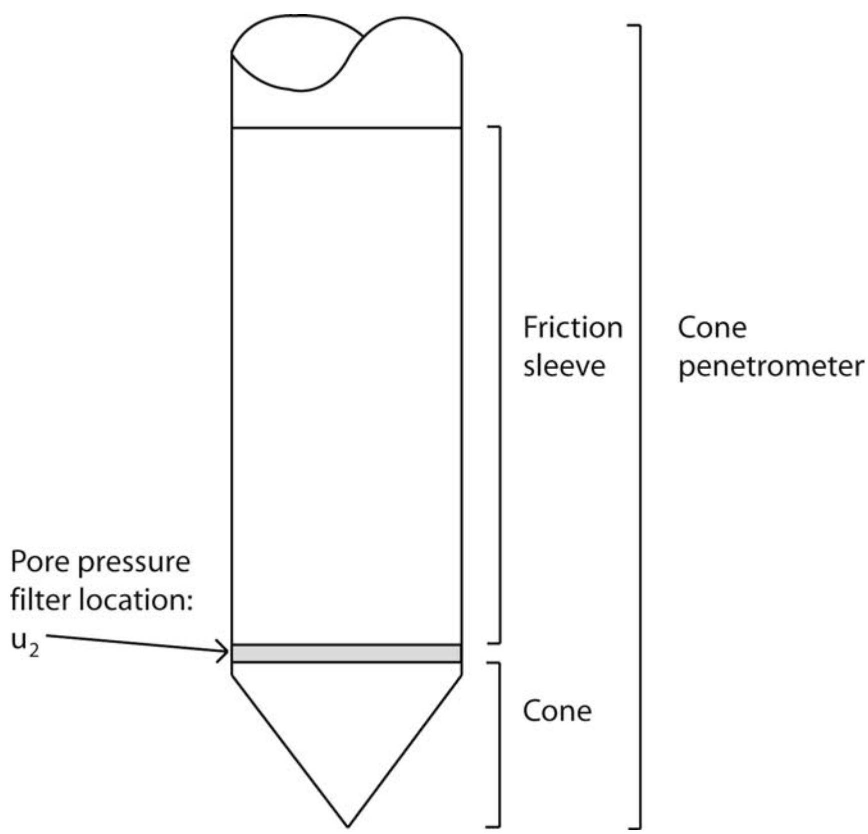


Figure 1 Terminology for cone penetrometers

History

1932

The first cone penetrometer tests were made using a 35 mm outside diameter gas pipe with a 15 mm steel inner push rod. A cone tip with a 10 cm^2 projected area and a 60^o apex angle was attached to the steel inner push rods, as shown in Figure 2.

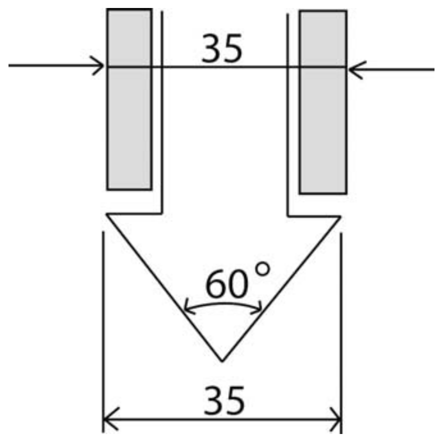


Figure 2 Early Dutch mechanical cone (After Sanglerat, 1972)

1935

Delf Soil Mechanics Laboratory designed the first manually operated 10 ton cone penetration push machine, see Figure 3.

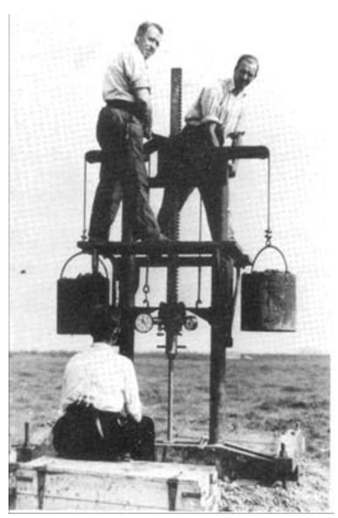


Figure 3 Early Dutch mechanical cone (After Delft Geotechnics)

1948

Improvement of the original Dutch mechanical cone by adding a conical part just above the cone. The purpose of the geometry was to prevent soil from entering the gap between the casing and inner rods. The basic Dutch mechanical cones, shown in Figure 4, are still in use in some parts of the world.

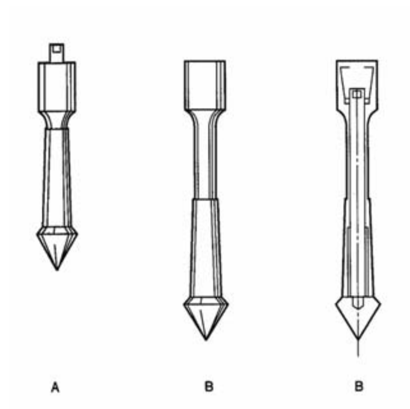


Figure 4 Dutch mechanical cone penetrometer with conical mantle

1953

Addition of a friction sleeve ('adhesion jacket') behind the cone to include measurement of the local sleeve friction (Begemann, 1953), see Figure 5. Measurements were made every 8 inches (20 cm), and for the first time, friction ratio was used to classify soil type (see Figure 6).

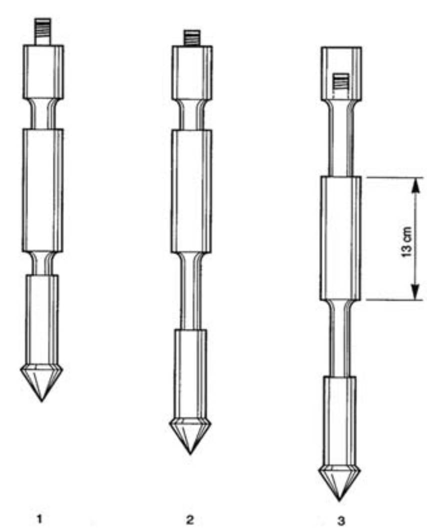


Figure 5 Begemann type cone with friction sleeve

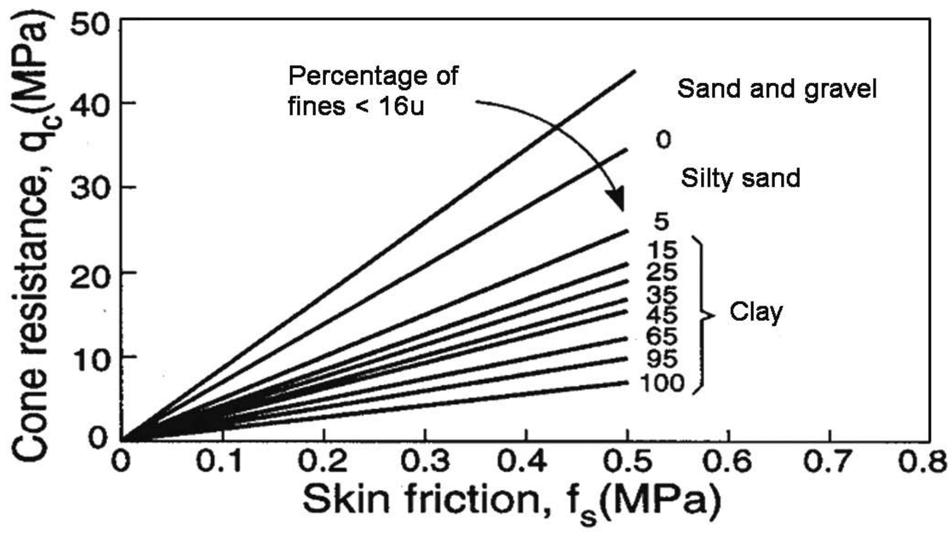


Figure 6 First soil classification for Begemann mechanical cone

1965

Development of an electric cone by Fugro, of which the shape and dimensions formed the basis for the modern cones and the International Reference Test and ASTM procedure. The main improvements relative to the mechanical cone penetrometers are:

- Elimination of incorrect readings due to friction between inner rods and outer rods and weight of inner rods.
- Continuous testing with continuous rate of penetration without the need for alternate movements of different parts of the penetrometer and no undesirable soil movements influencing the cone resistance.
- Simpler and more reliable electrical measurement of cone resistance and sleeve friction.

1974

Introduction of cone penetrometers that could also measure pore pressure (piezocone). Early design had various shapes and pore pressure filter locations. Gradually the practice has become more standardized so that the recommended position of the filter element is close behind the cone at the u_2 location. With the measurement of pore water pressure it became apparent that it was necessary to correct the cone resistance for pore water pressure effects (q_t), especially in soft clay.

Test Equipment and Procedures

Cone Penetrometers

Cone penetrometers come in a range of sizes with the 10 cm^2 and 15 cm^2 probes the most common and specified in most standards. Figure 7 shows a range of cones from a mini-cone at 2 cm^2 to a large cone at 40 cm^2. The mini cones are used for shallow investigations, whereas the large cones can be used in gravely soils.

Figure 7 Range of CPT probes (from left: 2 cm^2, 10 cm^2, 15 cm^2, 40 cm^2)

Additional Sensors/Modules

Since the introduction of the electric cone in the early 1960's, many additional sensors have been added to the cone, such as;

- Temperature
- Geophones (seismic wave velocity)
- Pressuremeter
- Camera (visible light)
- Radioisotope (gamma/neutron)
- Electrical resistivity/conductivity
- Dielectric
- pH
- Oxygen exchange (redox)
- Laser/ultraviolet induced fluorescence
- Membrane interface probe (MIP)

The latter items are primarily for geo-environmental applications.
One of the more common additional sensors is a geophone to allow the measurement of seismic wave velocities. A schematic of the seismic CPT (SCPT) is shown in Figure 8.

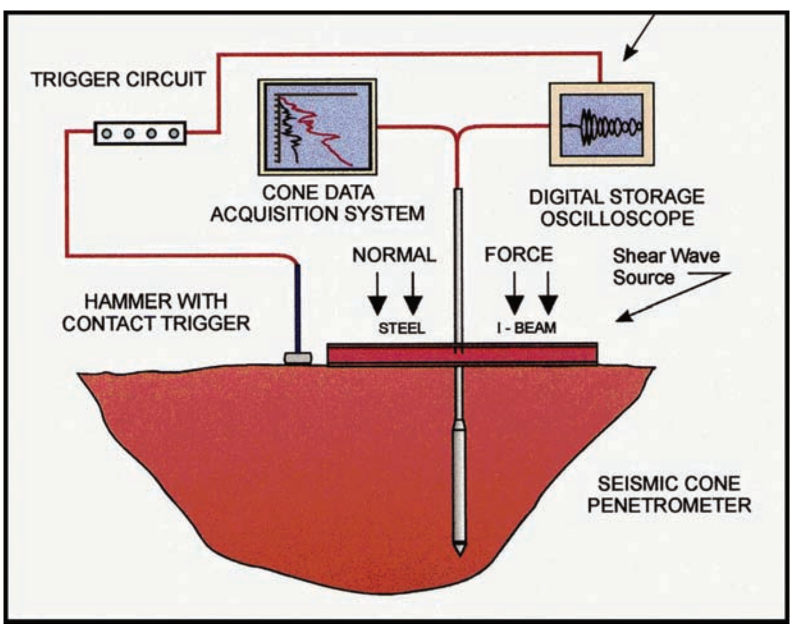


Figure 8 Schematic of Seismic CPT (SCPT)

Pushing Equipment

Pushing equipment consists of push rods, a thrust mechanism and a reaction frame.

On Land

Pushing equipment for on land applications generally consist of specially built units that are either truck or track mounted. CPT's can also be carried out using an anchored drill-rig. Figures 9 to 12 show a range of on land pushing equipment.

Figure 9 Truck mounted 25 ton CPT unit

Figure 10 Track mounted 20 ton CPT unit

Figure 11 Small anchored drill-rig unit

Figure 12 Ramset for CPT inside buildings or limited access

Over water

There is a variety of pushing equipment for over water investigations depending on the depth of water. Floating or Jack-up barges are common in shallow water (depth less than 80 feet), see Figures 13 and 14.

Figure 13 Mid-size jack-up barge

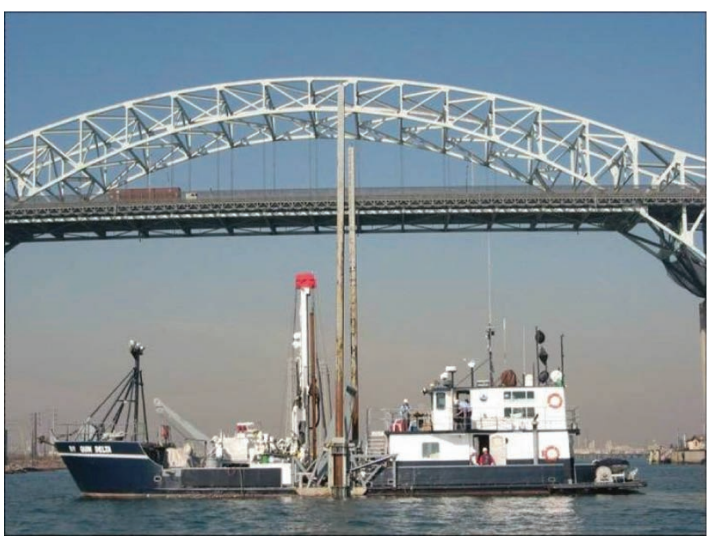


Figure 14 Quinn Delta ship with spuds

Rate of Penetration

The standard rate of penetration is 2 cm per second, which is approximately 1 inch per second. Hence, a 60 foot sounding can be completed (start to finish) in about 30 minutes. The cone results are generally not sensitive to slight variations in the rate of penetration.

Interval of Readings

Electric cones produce continuous analogue data. However, most systems convert the data to digital form at selected intervals. Most standards require the interval to be no more than 8 inches (200mm). In general, most systems collect data at intervals of between 1 to 2 inches (25 - 50mm), with 2 inches the most common.

Dissipation Tests

During a pause in penetration, any excess pore pressure generated around the cone will start to dissipate. The rate of dissipation depends upon the coefficient of consolidation, which in turn, depends on the compressibility and permeability of the soil. The rate of dissipation also depends on the diameter of the probe. A dissipation test can be performed at any required depth by stopping the penetration and measuring the decay of pore pressure with time. If equilibrium pore pressures are required, the dissipation test should continue until no further dissipation is observed. This can occur rapidly in sands, but may take several days in plastic clays. Dissipation is faster for smaller cones.

Calibration and Maintenance

Calibrations should be carried out at regular intervals (approximately every 3 months). For major projects, calibrations should be carried out before and after the field work, with functional checks during the work. Functional checks should include recording and evaluation of zero load measurements.

With careful design, calibration, and maintenance, strain gauge load cells and pressure transducers can have an accuracy and repeatability of better than +/- 0.2% of full scale reading.

Maintenance	*Start of Project*	*Start of Test*	*End of Test*	*End of Day*	*Once a Month*	*Every 3 months*
Wear	x	x			x	
O-ring seals	x			x		
Push-rods		x			x	
Pore pressure-filter	x	x				
Calibration						x
Computer					x	
Cone					x	
Zero-load		x	x			
Cables	x				x	

Table 3 Summary of checks and recalibrations for the CPT

Pore Water Effects

In soft clays and silts and in over water work, the measured q_c must be corrected for pore water pressures acting on the cone geometry, thus obtaining the corrected cone resistance, q_t:

$$q_t = q_c + u_2 (1 - a_n)$$

where a_n is the net area ratio determined from laboratory calibration.

CPT Interpretation

Numerous semi-empirical correlations have been developed to estimate geotechnical parameters from the CPT for a wide range of soil. These correlations vary in their reliability and applicability. Because the CPT has additional sensors (e.g. pore pressure: CPTU and seismic: SCPT), the applicability to estimate soil parameters varies. Since CPT with pore pressure measurements (CPTU) is commonly available, Table 4 shows an estimate of the perceived applicability of the CPTU to estimate soil parameters. If seismic is added, the ability to estimate soil stiffness (E, G & G_o) improves further.

Soil Type	D_r	Ψ	K_o	**OCR**	S_t	s_u	ϕ	**E,G***	**M**	G_0*	**k**	c_h
Sand	2-3	2-3		5			2-3	3-4		2-3	3	3-4
Clay			2	1	2	1-2	4	3-4	4	3-4	2-3	2-3

Table 4 Perceived applicability of CPTU for deriving soil parameters

1 = high; 2 = high to moderate; 3 = moderate; 4 = moderate to low; 5 = low reliability;
Blank = no applicability

Where:

D_r	Relative density	ϕ	Friction angle
Ψ	State Parameter	K_0	In-situ stress ratio
E, G	Young's and Shear moduli	G_0	Small strain shear moduli
OCR	Over consolidation ratio	M (or m_v)	Compressibility
s_u	Undrained shear strength	S_t	Sensitivity
c_H	Coefficient of consolidation	k	Permeability

Soil Profiling and Classification

The major application of the CPT is for *soil profiling and classification*. Typically, the cone resistance, (q_t) is high in sands and low in clays, and the friction ratio ($R_f = f_s/q_t$) is low in sands and high in clays. CPT classification charts cannot be expected to provide accurate predictions of soil type based on grain size distribution but provide a guide to the mechanical characteristics of the soil, or the *soil behavior type* (SBT). CPT data provides a repeatable index of the aggregate behavior of the in-situ soil in the immediate area of the probe. Hence, prediction of soil type based on CPT is referred to as soil behavior type (SBT).

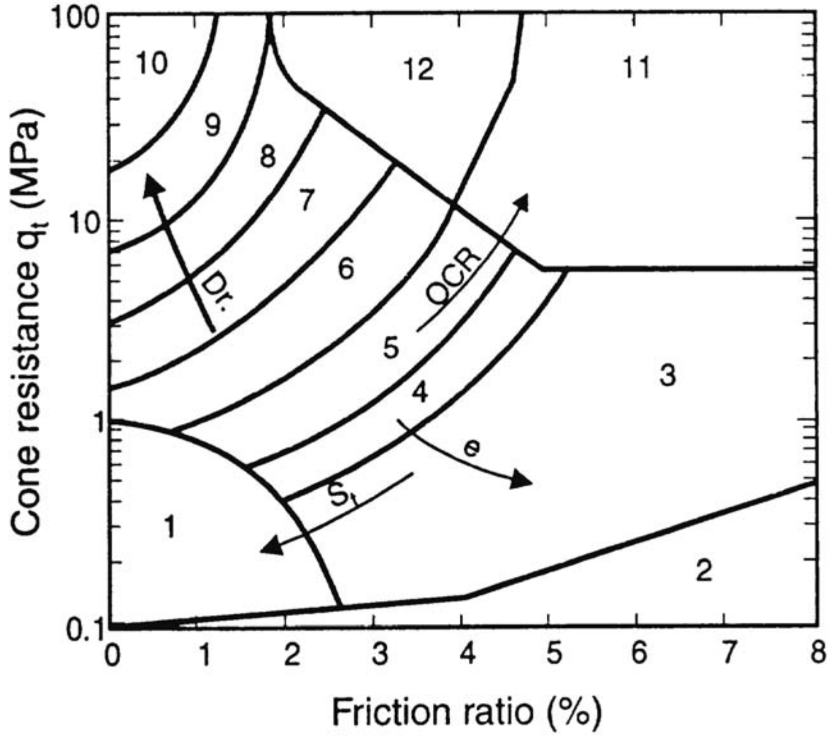
Non-Normalized Charts

The most commonly used CPT soil behavior type method is the chart suggested by Robertson et al. (1986) and shown in Figure 15. This chart uses the basic CPT parameters of cone resistance, q_t and friction ratio, R_f. The chart is global in nature and can provide reasonable predictions of soil behavior type for CPT soundings up to about 60ft (20m) in depth. The chart identifies general trends in ground response such as increasing relative density (D_r) for sandy soils, increasing stress history (OCR), soil sensitivity (S_t) and void ratio (e) for cohesive soils. Overlap in some zones should be expected and the zones should be adjusted somewhat based on local experience.

Normalized Charts

Since both the penetration resistance and sleeve friction increase with depth due to the increase in effective overburden stress, the CPT data requires normalization for overburden stress for very shallow and/or very deep soundings.

A popular CPT soil classification chart based on normalized CPT data is that proposed by Robertson (1990) and shown in Figure 16. A zone has been identified in which the CPT results for most young, un-cemented, insensitive, normally consolidated soils will fall. Again the chart is global in nature and provides only a guide to soil behavior type (SBT). Overlap in some zones should be expected and the zones should be adjusted somewhat based on local experience.

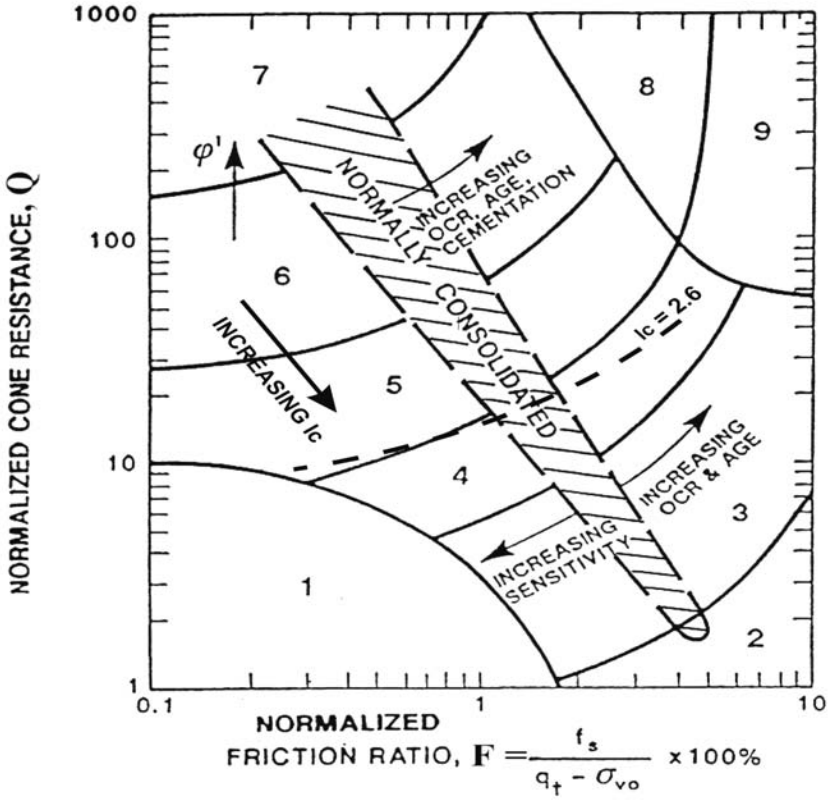


Zone	*Soil Behavior Type*
1	*Sensitive fine grained*
2	*Organic material*
3	*Clay*
4	*Silty Clay to clay*
5	*Clayey silt to silty clay*
6	*Sandy silt to clayey silt*
7	*Silty sand to sandy silt*
8	*Sand to silty sand*
9	*Sand*
10	*Gravelly sand to sand*
11	*Very stiff fine grained**
12	*Sand to clayey sand**

** Overconsolidated or cemented*

1 MPa = 10 tsf

Figure 15 CPT Soil Behavior Type (SBT) chart
(Robertson et al., 1986).



Zone	*Soil Behavior Type*	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – peats*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

** Heavily overconsolidated or cemented*

Figure 16 Normalized CPT Soil Behavior Type (SBT_N) chart
(Robertson, 1990).

If no prior CPT experience exists in a given geologic environment it is advisable to obtain samples from appropriate locations to verify the classification and soil behavior type. If significant CPT experience is available and the charts have been modified based on this experience samples may not be required.

Soil classification can be improved if pore pressure data is also collected. In soft clay the penetration pore pressures can be very large, whereas, in stiff heavily over-consolidated clays or dense silts and silty sands the penetration pore pressures can be small and sometimes negative relative to the equilibrium pore pressures (u_0). Also the rate of pore pressure dissipation during a pause in penetration can guide in the soil type. In sandy soils any excess pore pressures will dissipate very quickly, whereas, in clays the rate of dissipation can be very slow.

To simplify the application of the CPT SBT_N chart shown in Figure 16, the normalized cone parameters Q_t and F_r can be combined into one Soil Behavior Type index, I_c, where I_c is the radius of the essentially concentric circles that represent the boundaries between each SBT zone. I_c can be defined as follows;

$$I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5}$$

where:

Q_t = the normalized cone penetration resistance (dimensionless)

= $(q_t - \sigma_{vo})/\sigma'_{vo}$

F_r = the normalized friction ratio, in %

= $(f_s/(q_t - \sigma_{vo})) \times 100\%$

The boundaries of soil behavior types are then given in terms of the index, I_c, as shown in Figure 16. The soil behavior type index does not apply to zones 1, 8 and 9. Profiles of I_c provide a simple guide to the continuous variation of soil behavior type in a given soil profile based on CPT results. Independent studies have shown that the normalized SBT_N chart shown in Figure 16 typically has greater than 80% reliability when compared to samples.

Equivalent SPT N_{60} Profiles

The Standard Penetration Test (SPT) is one of the most commonly used in-situ tests in many parts of the world, especially North America. Despite continued efforts to standardize the SPT procedure and equipment there are still problems associated with its repeatability and reliability. However, many geotechnical engineers have developed considerable experience with design methods based on local SPT correlations. When these engineers are first introduced to the CPT they initially prefer to see CPT results in the form of equivalent SPT N-values. Hence, there is a need for reliable CPT/SPT correlations so that CPT data can be used in existing SPT-based design approaches.

There are many factors affecting the SPT results, such as, borehole preparation and size, sampler details, rod length and energy efficiency of the hammer-anvil-operator system. One of the most significant factors is the energy efficiency of the SPT system. This is normally expressed in terms of the rod energy ratio (ERr). An energy ratio of about 60% has generally been accepted as the reference value which represents the approximate historical average SPT energy.

A number of studies have been presented over the years to relate the SPT N-value to the CPT cone penetration resistance, q_c. Robertson et al. (1983) reviewed these correlations and presented the relationship shown in Figure 17 relating the ratio $(q_c/p_a)/N_{60}$ with mean grain size, D_{50} (varying between 0.001mm to 1mm). Values of q_t are made dimensionless when dividing by the atmospheric pressure (p_a) in the same units as q_c. It is observed that the ratio increases with increasing grain size.

The values of N used by Robertson et al. correspond to an average energy ratio of about 60%. Hence, the ratio applies to $N_{60,}$ as shown on Figure 17. Other studies have linked the ratio between the CPT and SPT with fines content for sandy soils.

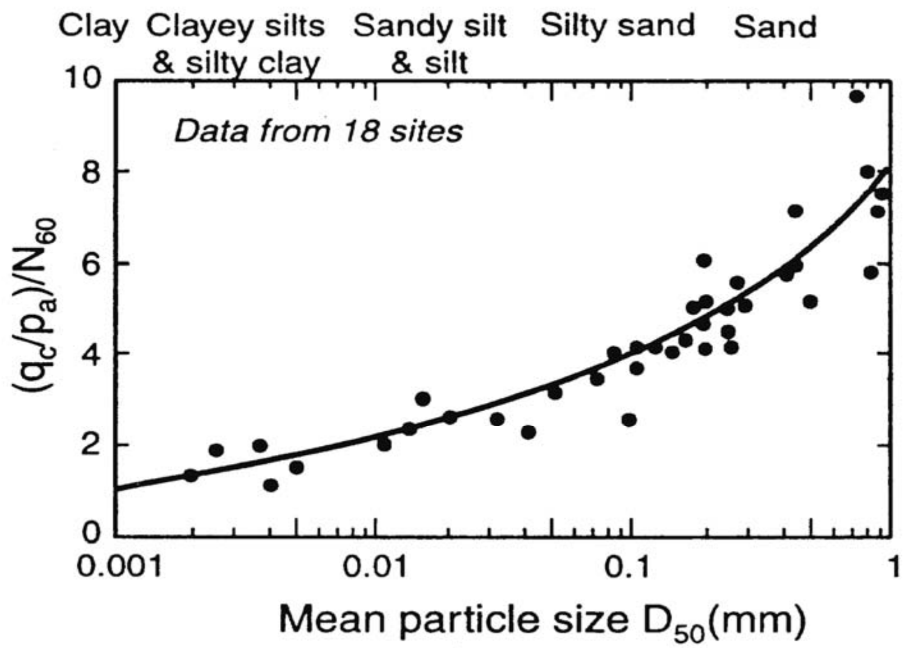


Figure 17 CPT-SPT correlations with mean grain size
(Robertson et al., 1983)

The above correlations need the soil grain size information to determine the mean grain size (or fines content). Grain characteristics can be estimated directly from CPT results using soil classification or soil behavior type (SBT) charts. The CPT SBT charts show a clear trend of increasing friction ratio with increasing fines content and decreasing grain size. Robertson et al. (1986) suggested $(q_c/p_a)/N_{60}$ ratios for each soil behavior type zone using the non-normalized CPT chart. The suggested ratio for each soil behavior type is given in Table 5.

These values provide a reasonable estimate of SPT N_{60} values from CPT data. For simplicity the above correlations are given in terms of q_c. For fine grained soft soils the correlations should be applied to total cone resistance, q_t.

One disadvantage of this simplified approach is the somewhat discontinuous nature of the conversion. Often a soil will have CPT data that crosses different soil behavior type zones and hence produces discontinuous small changes in predicted SPT N_{60} values.

Zone	Soil Behavior Type	$\frac{(q_c/p_a)}{N_{60}}$
1	sensitive fine grained	2
2	organic soils - peats	1
3	clay	1
4	silty clay to clay	1.5
5	clayey silt to silty clay	2
6	sandy silt to clayey silt	2.5
7	silty sand to sandy silt	3
8	sand to silty sand	4
9	sand	5
10	dense sand to gravelly sand	6
11	very stiff fine grained	1

Table 5 Suggested $(q_c/p_a)/N_{60}$ ratios.

Jefferies and Davies (1993) suggested the application of a soil behavior type index, I_c to link with the CPT-SPT correlation. The soil behavior type index, I_c, can be combined with the CPT-SPT ratios to give the following relationship:

$$\frac{(q_c/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

Jefferies and Davies (1993) suggested that the above approach can provide a better estimate of the SPT N-values than the actual SPT test due to the poor repeatability of the SPT.

In very loose soils the weight of the rods and hammer can dominate the SPT penetration resistance and produce very low N-values, which can result in high $(q_c/p_a)/N_{60}$ ratios due to the low SPT N-values measured.

Geotechnical Parameters

Undrained Shear Strength (s_u)

No single value of undrained shear strength exists, since the undrained response of soil depends on the direction of loading, soil anisotropy, strain rate, and stress history. Typically the undrained strength in tri-axial compression is larger than in simple shear which is larger than tri-axial extension ($s_{uTC} > s_{uSS} > s_{uTE}$). The value of s_u to be used in analysis therefore depends on the design problem.

Since anisotropy and strain rate will inevitably influence the results of all in-situ tests, their interpretation will necessarily require some empirical content to account for these factors, as well as possible effects of sample disturbance.

Recent theoretical solutions have provided some valuable insight into the form of the relationship between cone resistance and s_u. All theories result in a relationship between cone resistance and s_u of the form:

$$s_u = \frac{q_t - \sigma_v}{N_{kt}}$$

Typically N_{kt} varies between 10 to 20, with 15 as an average. N_{kt} tends to increase with increasing plasticity and decrease with increasing soil sensitivity.

For deposits where little experience is available, estimate s_u using the total cone resistance (q_t) and preliminary cone factor values (N_{kt}) from 15 - 20. For a more conservative estimate, select a value close to the upper limit.

In very soft clays where there may be some uncertainty with the accuracy in q_t, estimates of s_u can be made from the excess pore pressure (Δu) measured behind the cone (u_2) using the following:

$$s_u = \frac{\Delta u}{N_{\Delta u}}$$

Where $N_{\Delta u}$ varies from 7 to 10. For a more conservative estimate, select a value close to the upper limit.

If previous experience is available in the same deposit, the values suggested above should be adjusted to reflect this experience.

For larger, moderate to high risk projects, where high quality field and laboratory data may be available, site specific correlations should be developed based on appropriate and reliable values of s_u.

Soil Sensitivity

The sensitivity (S_t) of clay is defined as the ratio of undisturbed undrained shear strength to totally remolded undrained shear strength.

$$S_t = \frac{s_u}{s_{u(Remolded)}} = \frac{q_t - \sigma_v}{N_{kt}} \; (1 / f_s)$$

The remolded undrained shear strength can be assumed equal to the sleeve friction stress, f_s. Therefore, the sensitivity of a clay can be estimated by calculating the peak s_u from either site specific or general correlations with q_t or Δu and $s_{u(Remolded)}$ from f_s.

For relatively sensitive clays ($S_t > 5$), the value of f_s can be very low with inherent difficulties in accuracy. Hence, the estimate of sensitivity should be used as a guide only.

Estimation of OCR and K_o – for cohesive soils

Overconsolidation Ratio (OCR)

Overconsolidation ratio (OCR) is defined as the ratio of the maximum past effective consolidation stress and the present effective overburden stress:

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}}$$

For mechanically overconsolidated soils where the only change has been the removal of overburden stress, this definition is appropriate. However, for cemented and/or aged soils the OCR may represent the ratio of the yield stress and the present effective overburden stress. The yield stress will depend on the direction and type of loading. The easiest and generally the most reliable method to estimate OCR in cohesive soils is:

$$OCR = k \left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \right)$$

An average value of k = 0.3 can be assumed, with an expected range of 0.2 to 0.5. Higher values of k are recommended in aged, heavily overconsolidated clays. If previous experience is available in the same deposit, the values of k should be adjusted to reflect this experience and to provide a more reliable profile of OCR.

For larger, moderate to high-risk projects, where additional high quality field and laboratory data may be available, site-specific correlations should be developed based on consistent and relevant values of OCR. The estimated OCR is influenced by soil sensitivity, pre-consolidation mechanism, soil type and local heterogeneity.

In-Situ Stress Ratio (K_o)

There is no reliable method to determine K_o from CPT. However, an estimate can be made based on an estimate of OCR, as shown in Figure 18.

Kulhawy and Mayne (1990) suggested a similar approach, using:

$$K_o = 0.1 \left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \right)$$

These approaches are generally limited to mechanically overconsolidated soils. Considerable scatter exists in the database used for these correlations and therefore they must be considered only as a guide.

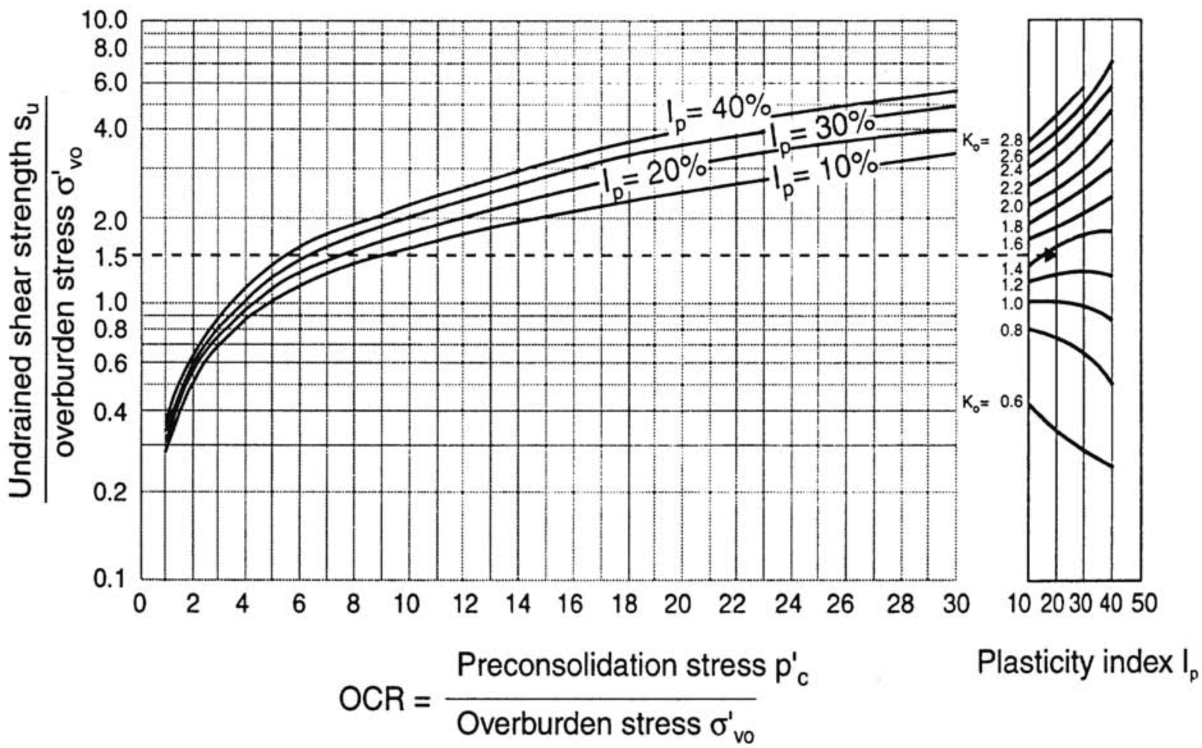


Figure 18 OCR and K_o from s_u/σ'_{vo} and I_p
(after Andresen et al., 1979)

Friction Angle

The shear strength of uncemented, cohesionless soil is usually expressed in terms of a peak secant friction angle, ϕ'.

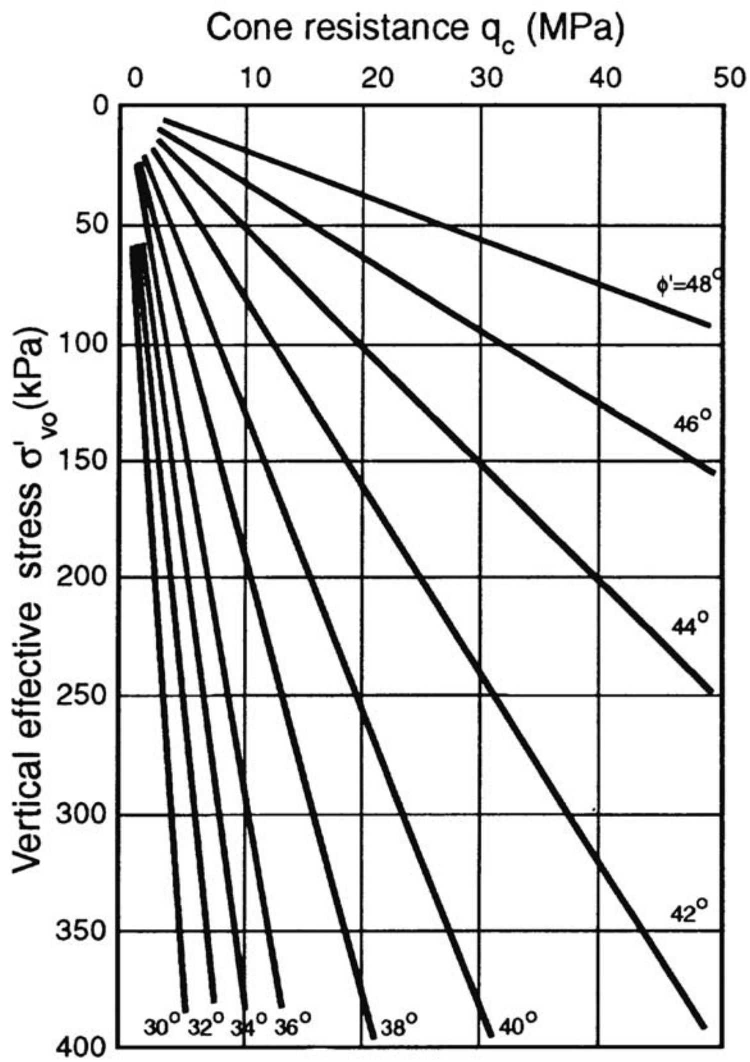
Numerous studies have been published for assessing ϕ' from the CPT in clean sands and basically the methods fall into one of three categories:

- Bearing capacity theory
- Cavity expansion theory
- Empirical, based on calibration chamber tests

Significant advances have been made in the development of theories to model the CPT penetration process in sands (Yu and Mitchell, 1998). Cavity expansion models show the most promise since they are relatively simple and can incorporate many of the important features of soil response. However, empirical correlations based on calibration chamber test results are still the most commonly used.

A review of calibration chamber test results was made by Robertson and Campanella (1983) to compare cone resistance to measured peak secant friction angle. The peak secant friction angle was measured in drained triaxial compression tests performed at the confining stress approximately equal to the horizontal stresses in the calibration chamber before the CPT.

The recommended correlation for uncemented, unaged, moderately compressible, predominately quartz sands proposed by Robertson and Campanella (1983) is shown in Figure 19. For sands of high compressibility (i.e. carbonate sands or sands with high mica content), the chart will tend to predict low friction angles.



Note: 0.1MPa = 100 kPa = 1 bar ≈ 1 tsf ≈ 1 kg/cm²

$$\tan \phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$$

Figure 19 Friction angle, φ', from CPT in uncemented silica sand
(Robertson and Campanella, 1983)

Relative Density (D_r)

For cohesionless soils, the density, or more commonly, the relative density or density index, is often used as an intermediate soil parameter. Relative density, D_r, or density index, I_D, is defined as:

$$I_D = D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where:

> e_{max} and e_{min} are the maximum and minimum void ratios and e is the in-situ void ratio.

The problems associated with the determination of e_{max} and e_{min} are well known. Also, research has shown that the stress strain and strength behavior of cohesionless soils is too complicated to be represented by only the relative density of the soil. However, for many years relative density has been used by engineers as a parameter to describe sand deposits.

Research using large calibration chambers has provided numerous correlations between CPT penetration resistance and relative density for clean, predominantly quartz sands. The calibration chamber studies have shown that the CPT resistance is controlled by sand density, in-situ vertical and horizontal effective stress and sand compressibility. Sand compressibility is controlled by grain characteristics, such as, grain size, shape and mineralogy. Angular sands tend to be more compressible than rounded sands as do sands with high mica and/or carbonate compared with clean quartz sands. More compressible sands give a lower penetration resistance for a given relative density then less compressible sands.

Based on extensive calibration chamber testing on Ticino sand, Baldi et al. (1986) recommended a formula to estimate relative density from q_c. A modified version of this formula, to obtain D_r from q_{c1} is as follows:

$$D_r = \left(\frac{1}{C_2}\right) \ln\left(\frac{q_{c1}}{C_0}\right)$$

where:

C_0 and C_2 are soil constants
σ'_{vo} = effective vertical stress in kPa
q_{c1} = $(q_c / p_a) / (\sigma'_{vo}/p_a)^{0.5}$
= normalized CPT resistance, corrected for overburden pressure
p_a = reference pressure of 1 tsf, in same units as q_c and σ'_{vo}
q_c = cone penetration resistance

For moderately compressible, normally consolidated, unaged and uncemented, predominantly quartz sands the constants are: $C_o = 15.7$ and $C_2 = 2.41$.

Kulhawy and Mayne (1990) suggested a simpler formula for estimating relative density:

$$D_r^2 = \frac{q_{c1}}{305 Q_C Q_{OCR} Q_A}$$

where:

q_{c1} and p_a are as defined above
Q_C = Compressibility factor ranges from 0.91 (low compress.) to 1.09 (high compress.)
Q_{OCR} = Overconsolidation factor = $OCR^{0.18}$
Q_A = Aging factor = $1.2 + 0.05\log(t/100)$

A constant of 350 is more reasonable for medium, clean, uncemented, unaged quartz sands that are about 1,000 years old. The constant is closer to 300 for fine sands and closer to 400 for coarse sands.

The equation can then be simplified to:

$$D_r^2 = q_{c1} / 350$$

Stiffness and Modulus

CPT data can be used to estimate modulus for subsequent use in elastic or semi-empirical settlement prediction methods. However, correlations between q_c and moduli (E) are sensitive to stress and strain history, aging and sand mineralogy.

A useful guide for estimating Young's moduli for uncemented predominantly silica sands is given in Figure 20. The modulus has been defined as that mobilized at 0.1% strain. For more heavily loaded conditions (i.e. larger strain) the modulus would decrease. The results for NC sands are applicable for young recent fills with an age less than 10 years and the results for Aged NC sands are applicable for natural sands with an age greater than 1,000 years.

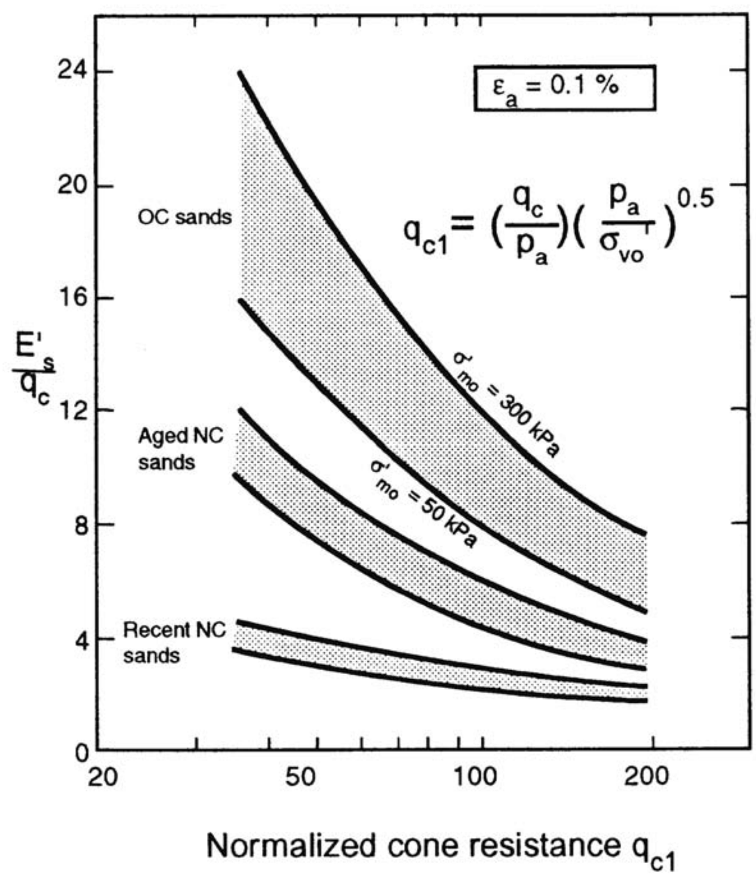


Figure 20 Evaluation of drained Young's modulus from CPT for silica sands

Modulus From Shear Wave Velocity

A major advantage of the seismic CPT is the additional measurement of the shear wave velocity, V_s. The shear wave velocity is measured using a downhole technique during pauses in the CPT resulting in a continuous profile of V_s. Elastic theory states that the small strain shear modulus, G_o can be determined from:

$$G_o = \rho\ V_s^2$$

where ρ is the mass density of the soil ($\rho = \gamma/g$).

Hence, the addition of shear wave velocity during the CPT provides a direct measure of soil stiffness.

The small strain shear modulus represents the elastic stiffness of the soils at shear strains (γ) less than 10^{-4} percent. Elastic theory also states that the small strain Young's modulus, E_o is linked to G_o, as follows:

$$E_o = 2(1 + \upsilon)G_o$$

Where υ is Poisson's ratio, which is often between 0.1 to 0.3 for most soils.

Application to engineering problems requires that the small strain modulus be softened to the appropriate strain level. For most well designed structures the degree of softening is often close to a factor of 3. Hence, for many applications the equivalent Young's modulus (E_s') can be estimated from:

$$E_s' \simeq G_o = \rho\ V_s^2$$

The shear wave velocity can also be used directly for the evaluation of liquefaction potential. Hence, the seismic CPT provides two independent methods to evaluate liquefaction potential.

Identification of Unusual Soils Using the SCPT

Almost all available empirical correlations to interpret in-situ tests assume that the soil is well behaved, i.e. similar to the soils in which the correlation was based. Many of the existing correlations apply to soils such as, unaged, uncemented, silica sands. Application of the existing empirical correlations in sands other than unaged and uncemented can produce incorrect interpretations. Hence, it is important to be able to identify if the soils are ‘well behaved’. The combined measurement of shear wave velocity and cone resistance provides an opportunity to identify these ‘unusual’ soils. The cone resistance (q_c) is a good measure of soil strength, since the cone is inducing very large strains and the soil adjacent to the probe is at failure. The shear wave velocity (V_s) is a direct measure of the small strain soil stiffness (G_o) since the measurement is made at very small strains. Recent research has shown that unaged and uncemented sands have data that falls within a narrow range of combined q_c and G_o, as shown in Figure 21 and the following equations:

Upper bound, unaged & cemented $\quad G_o = 280\ (q_c\ \ \sigma'_{vo}\ \ p_a)^{0.3}$

Lower bound, unaged & cemented $\quad G_o = 110\ (q_c\ \ \sigma'_{vo}\ \ p_a)^{0.3}$

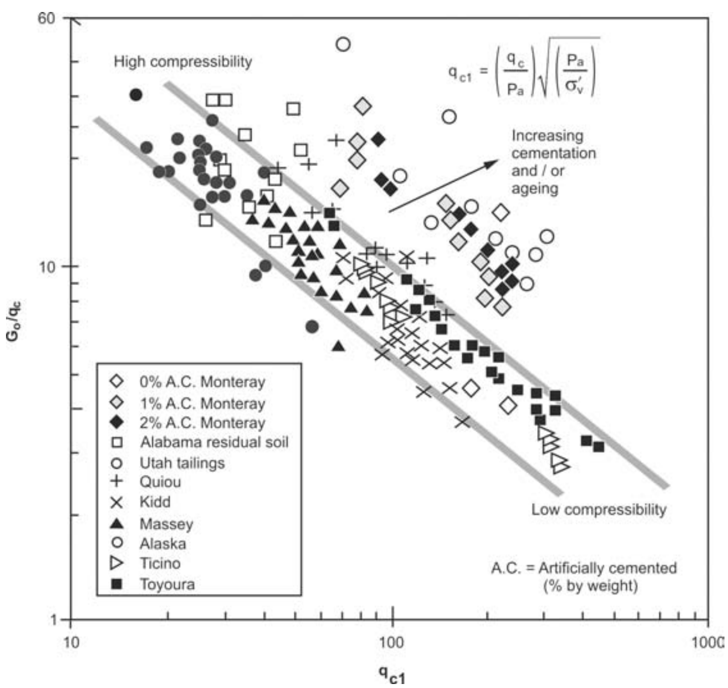


Figure 21 Characterization of uncemented, unaged sands
(after Eslaamizaad and Robertson, 1997)

Hydraulic Conductivity (k)

An approximate estimate of soil hydraulic conductivity or coefficient of permeability, *k*, can be made from an estimate of soil behavior type using the CPT SBT charts. Table 6 provides estimates based on the non-normalized chart shown in Figure 15, while Table 7 provides estimates based on the normalized chart shown in Figure 16. These estimates are approximate at best, but can provide a guide to variations of possible permeability.

Zone	Soil Behavior Type (SBT)	Range of permeability k (m/s)
1	Sensitive fine grained	$3x10^{-9}$ to $3x10^{-8}$
2	Organic soils	$1x10^{-8}$ to $1x10^{-6}$
3	Clay	$1x10^{-10}$ to $1x10^{-9}$
4	Silty clay to clay	$1x10^{-9}$ to $1x10^{-8}$
5	Clayey silt to silty clay	$1x10^{-8}$ to $1x10^{-7}$
6	Sandy silt to clayey silt	$1x10^{-7}$ to $1x10^{-6}$
7	Silty sand to sandy silt	$1x10^{-5}$ to $1x10^{-6}$
8	Sand to silty sand	$1x10^{-5}$ to $1x10^{-4}$
9	Sand	$1x10^{-4}$ to $1x10^{-3}$
10	Gravelly sand to dense sand	$1x10^{-3}$ to 1
11	Very stiff fine-grained soil	$1x10^{-8}$ to $1x10^{-6}$
12	Very stiff sand to clayey sand	$3x10^{-7}$ to $3x10^{-4}$

Table 6 Estimation of soil permeability (k) from the non-normalized CPT SBT chart by Robertson et al. (1986) shown in Figure 15

Baligh and Levadoux (1980) recommended that the horizontal coefficient of permeability can be estimated from the expression:

$$k_h = \left(\frac{\gamma_w}{2.3\,\sigma'_{vo}}\right) RR \;\; c_h$$

where *RR* is the re-compression ratio in the overconsolidated range. It represents the strain per log cycle of effective stress during recompression and can be determined from laboratory consolidation tests. Baligh and Levadoux recommended that *RR* should range from $0.5x10^{-2}$ to $2x10^{-2}$.

Zone	Soil Behavior Type (SBT)	Range of permeability k (m/s)
1	Sensitive fine grained	$3x10^{-9}$ to $3x10^{-8}$
2	Organic soils	$1x10^{-8}$ to $1x10^{-6}$
3	Clay	$1x10^{-10}$ to $1x10^{-9}$
4	Silt mixtures	$3x10^{-9}$ to $1x10^{-7}$
5	Sand mixtures	$1x10^{-7}$ to $1x10^{-5}$
6	Sands	$1x10^{-5}$ to $1x10^{-3}$
7	Gravelly sands to dense sands	$1x10^{-3}$ to 1
8	Very stiff sand to clayey sand	$1x10^{-8}$ to $1x10^{-6}$
9	Very stiff fine-grained soil	$1x10^{-8}$ to $1x10^{-6}$

Table 7 Estimation of soil permeability (k) from the normalized CPT SBT chart by Robertson (1990) shown in Figure 16

Robertson et al. (1992) presented a summary of available data to estimate the horizontal coefficient of permeability from dissipation tests, and is shown in Figure 22. Since the relationship is also a function of the recompression ratio (RR) there is a wide variation of + or – one order of magnitude. Jamiolkowski et al. (1985) suggested a range of possible values of k_h/k_v for soft clays as shown in Table 8.

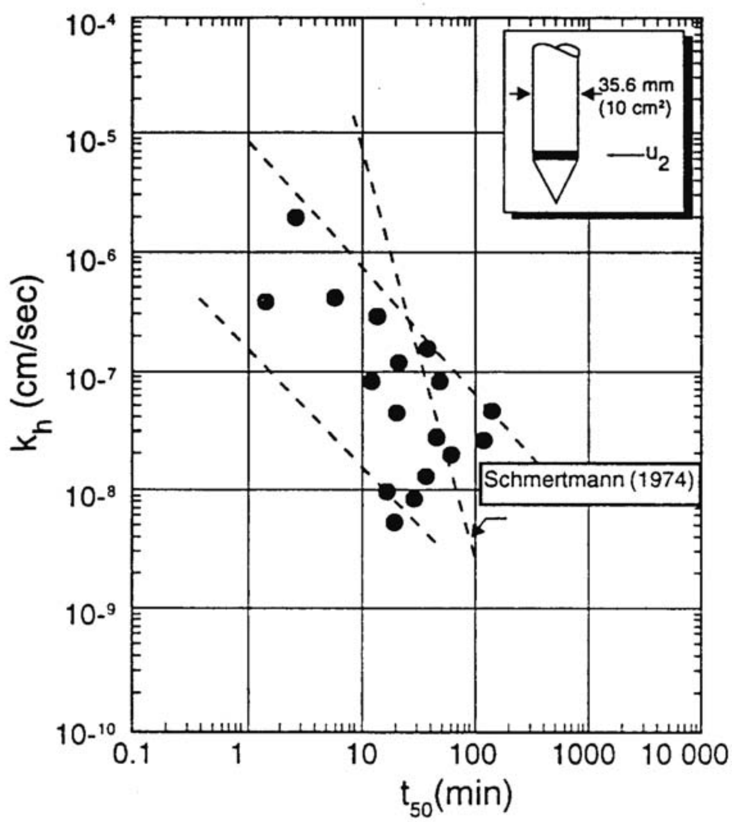


Figure 22 Summary of data for estimating horizontal coefficient of permeability from dissipation tests
(after Robertson et al., 1992)

Nature of Clay	k_h/k_v
No macrofabric, or only slightly developed macrofabric, essentially homogeneous deposits	1 to 1.5
From fairly well-to well-developed macrofabric, e.g. sedimentary clays with discontinuous lenses and layers of more permeable material	2 to 4
Varied clays and other deposits containing embedded and more or less continuous permeable layers	3 to 15

Table 8 Range of possible field values of k_h/k_v for soft clays
(after Jamiolkowski et al., 1985)

Consolidation Characteristics

Flow and consolidation characteristics of a soil are normally expressed in terms of the coefficient of consolidation, *c*, and hydraulic conductivity, *k*. They are inter-linked through the formula:

$$c = \frac{k\ \mathrm{M}}{\gamma_w}$$

where M is the constrained modulus relevant to the problem (i.e. unloading, reloading, virgin loading).

The parameters *c* and *k* vary over many orders of magnitude and are some of the most difficult parameters to measure in geotechnical engineering. It is often considered that accuracy within one order of magnitude is acceptable. Due to soil anisotropy, both *c* and *k* have different values in the horizontal (c_h, k_h) and vertical (c_v, k_v) direction. The relevant design values depend on drainage and loading direction.

Details on how to estimate *k* from CPT soil classification charts are given in another section.

The coefficient of consolidation can be estimated by measuring the dissipation or rate of decay of pore pressure with time after a stop in CPT penetration. Many theoretical solutions have been developed for deriving the coefficient of consolidation from CPT pore pressure dissipation data. The coefficient of consolidation should be interpreted at 50% dissipation, using the following formula:

$$c = \left(\frac{\mathrm{T}_{50}}{\mathrm{t}_{50}}\right) \mathrm{r_o}^2$$

where:

T_{50} = theoretical time factor
t_{50} = measured time for 50% dissipation
r_o = penetrometer radius

It is clear from this formula that the dissipation time is inversely proportional to the radius of the probe. Hence, in soils of very low permeability, the time for dissipation can be decreased by using smaller probes.

Robertson et al. (1992) reviewed dissipation data from around the world and compared the results with the leading theoretical solution by Teh and Houlsby (1991), as shown in Figure 23.

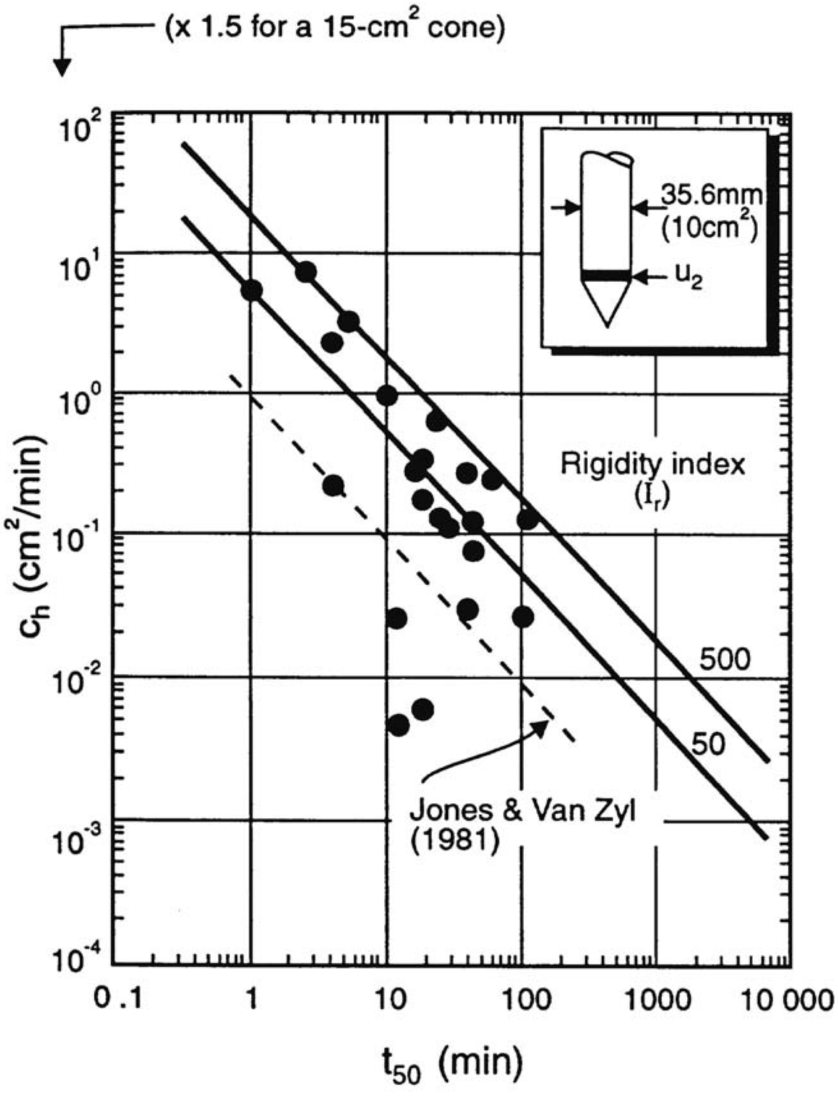


Figure 23 Average laboratory c_h values and CPTU results
(Robertson et al., 1992)

The review showed that the theoretical solution provided reasonable estimates of c_h. The solution shown in Figure 23 applies to pore pressure sensors located just behind the cone tip (i.e. u_2).

The ability to estimate c_h from CPT dissipation results is controlled by soil stress history, sensitivity, anisotropy, rigidity index (relative stiffness), fabric and structure. In overconsolidated soils, the pore pressure behind the cone tip can be low or negative, resulting in dissipation data that can initially rise before a decay to the equilibrium value. In these cases, the pore pressure sensor can be moved to the face of the cone or the t_{50} time can be estimated using the maximum pore pressure as the initial value. Care is required to ensure that the dissipation is continued to the correct equilibrium and not stopped prematurely after the initial rise.

Based on available experience, the CPT dissipation method should provide estimates of c_h to within + or – half an order of magnitude. However, the technique is repeatable and provides an accurate measure of changes in consolidation characteristics within a given soil profile.

An approximate estimate of the coefficient of consolidation in the vertical direction can be obtained using the ratios of permeability in the horizontal and vertical direction given in the section on hydraulic conductivity, since:

$$c_v = c_h \left(\frac{k_v}{k_h} \right)$$

Table 8 can be used to provide an estimate of the ratio of hydraulic conductivities.

CPT Applications

The previous sections have described how CPT results can be used to estimate geotechnical parameters that can be used as input in analyses. An alternate approach is to apply the in-situ test results directly to an engineering problem. A typical example of this approach is the evaluation of pile capacity directly from CPT results without the need for soil parameters.

As a guide, Table 9 shows a summary of the applicability of the CPT for direct design applications. The ratings shown in the table have been assigned based on current experience and represent a qualitative evaluation of the confidence level assessed to each design problem and general soil type. Details of ground conditions and project requirements can influence these ratings.

Type of Soil	Pile Design	Bearing Capacity	Settlement	Compaction Control	Liquefaction
Sand	1 – 2	1 – 2	2 – 3	1 – 2	1 – 2
Clay	1 – 2	1 – 2	3 – 4	3 – 4	1 – 2
Intermediate Soils	1 – 2	2 – 3	3 – 4	2 – 3	1– 2

Reliability rating: 1 = High; 2 = High to moderate; 3 = Moderate; 4 = Moderate to low; 5 = low

Table 9 Perceived applicability of the CPT/CPTU for various direct design problems

Shallow Foundations

The CPT has been used extensively for design of shallow foundations in both granular soils and fine grained soils for both bearing capacity and settlement. This can be done by two approaches:

- Using geotechnical parameters derived from the CPT combined with conventional design methodologies, e.g. undrained shear strength and bearing capacity equations.

- Using direct empirical design approaches using CPT results and design methodologies bases on past published field observations, e.g. direct estimates of bearing capacity from cone resistance.

Settlement, rather than bearing capacity criteria usually control design of shallow foundations with a width greater than about 4 ft. (1.2m). In these cases, settlement calculations can be improved using shear wave velocity measurements from a seismic CPT. In cohesive soils, settlements are often controlled by the stress history of the deposit. Profiles of stress history can be estimated using the CPT.

Deep Foundations

Design of piles was one of the earliest applications of the CPT, due to the similarity of the loading conditions. The most common approach for the design of piled foundations using CPT results is using direct empirical design approaches. In these cases the unit bearing capacity and side friction of the pile is estimated directly from CPT cone resistance.

Liquefaction Assessment

Historically, much of the case history evidence of soil liquefaction was from sites where only SPT data was available. In recent years the number of case histories where CPT data are available has increased and is now larger than the SPT database. Hence, the CPT has become increasingly more popular to estimate the potential for soil liquefaction due to the continuous nature of the results and the increased reliability of the data (Robertson & Wride, 1997).

Methods have also been developed where post-earthquake displacements (settlements and lateral spreads) can also be estimated using CPT results (Zhang et al., 2002 & 2004).

Compaction Control

The CPT has been found to be one of the best methods to monitor and document the effect of deep compaction due to the continuous, reliable and repeatable nature of the data. Most deep compaction techniques involve cyclic shear stresses in the form of vibration to induce an increase in soil density. Vibratory compaction is generally more effective in soil deposits with a friction ratio less than 1%. When the friction ratio exceeds about 1.5% vibratory compaction is usually not effective. These recommendations apply to average values in a soil deposit. Local seams or thin layers with higher friction ratio values are often of little practical importance for the overall performance of a project and their effect should be carefully evaluated when compaction specifications are prepared.

Most compaction techniques besides increasing the density, induce significant changes in the horizontal stresses. The CPT, like any penetration resistance, is also influenced by many factors, the most important being soil density and in-situ stresses.

New Developments

Significant developments have taken place in the past 20 years and these developments are likely to continue as the CPT becomes increasingly popular due to its reliability, repeatability and continuous data.

Standard Penetration Test (SPT)

Introduction

The concept of the test consists of driving a standard 2 inch (50mm) outside diameter thick walled sampler into the ground at the bottom of a borehole using the repeated blows of a 140 pound (63.5kg) hammer falling freely through 30 inches (760mm). The SPT N-value is the number of blows required to achieve a penetration of 12 inches (300mm), after an initial seating drive of 6 inches (150mm).

The purpose of this section is to briefly describe the SPT and to present a summary of the major factors that affect the results and interpretation of the results. A short summary of the main applications of the SPT is also provided.

History

The SPT was introduced in the USA in 1902 by the Raymond Pile Company. The earliest reference to a 'Standard Penetration Test' procedure is in a paper by Terzaghi in 1947. The test was not standardized in the USA until 1958 (ASTM D1586-58T). It is currently covered by ASTM D1586-99, and by many other standards around the world. More recently an International Reference Test Procedure (IRTP) was published under the auspices of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE, 1998).

However, one of the main problems of the SPT remains that the test equipment, drilling techniques, and test procedures have not been fully standardized on an international basis. Hence, there can be large variations in test results, even within one country.

Test Equipment and Procedures

The main standard for the SPT is the American Society for Testing and Materials (ASTM D-1586-99).

There are significant differences between the drilling techniques, SPT equipment and test procedures used in different regions and countries. These differences have developed due mainly to adaptation to local ground conditions, site access and local equipment.

Figure 24 shows the ASTM split-barrel sampler and Figure 25 shows a sketch of the basic SPT set-up using a 'cathead and rope' system with manual release along with a donut hammer.

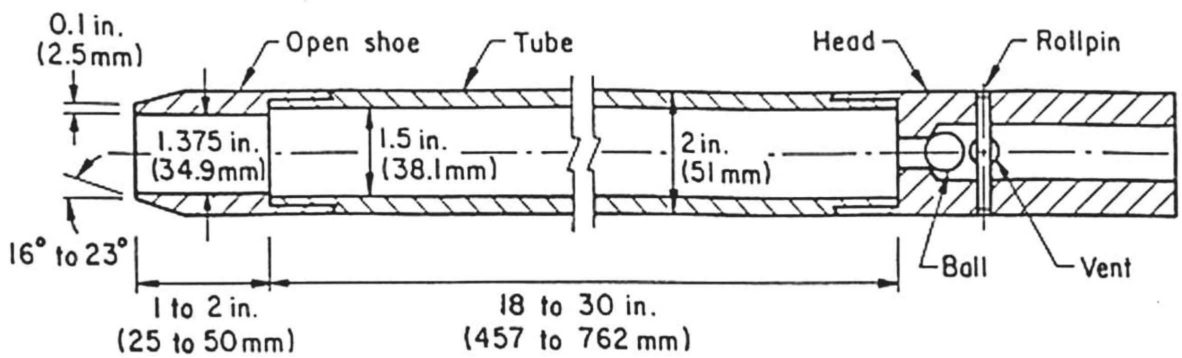


Figure 24 SPT split-barrel sampler (ASTM)

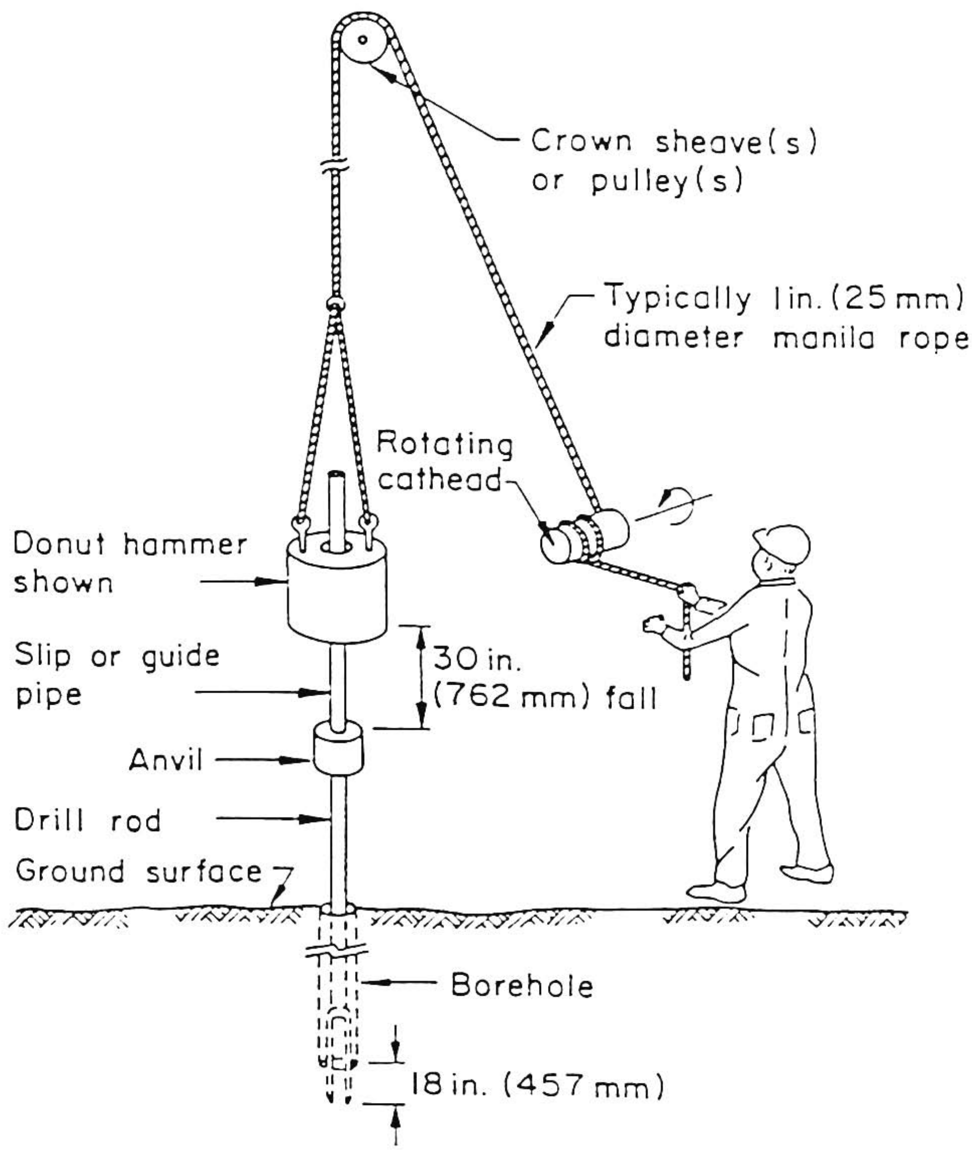


Figure 25 Schematic of SPT set-up using a rope and cathead

Factors Affecting the SPT

The major factors that influence the measurement of the SPT N-value can be divided into the following main areas:

- Drilling and borehole technique
- SPT equipment
- Test procedure

Differences in drilling techniques can have the largest affect on the measured SPT N-values, especially in granular soils below the water table. Differences in test equipment and procedure can also be important.

Drilling and Borehole Techniques

The main factors that may influence the SPT N-value associated with drilling and borehole techniques are:

- Method of borehole advancement
- Method of borehole support
- Size of borehole

The most common drilling methods used around the world are wash-boring, rotary drilling, augering, and light percussion drilling. Rotary drilling and augering are the most common drilling methods in North America.

Drilling methods that add water or a flush fluid to the borehole, such as rotary drilling, have the advantage of keeping the borehole full of fluid and hence, minimize any rapid flow of groundwater into the borehole during drilling and the borehole cleaning process. Methods which tend to remove fluid, such as during central plug removal with hollow-stem augering, can lead to loosening of the soil near the base of the borehole.

Most standards restrict the borehole diameter to minimize the decrease in vertical effective stress at the base of the borehole. The ASTM standard limits the borehole size between 2 to 6 inches (55mm to 150mm). Some countries have no limit to the size of the borehole. Borehole size is particularly important when performing the SPT in granular soils.

SPT Equipment

The main factors that may influence the SPT N-value associated with equipment are:

- Hammer design
- Rod size and type
- Sampler design

There are many different hammer designs used as part of the SPT. The hammer consists of the hammer, anvil and the lift-release mechanism. The only statement in most standards regarding the hammer is that it must weight 140 pounds and must be 'free falling' from a height of 30 inches. The hammer mechanisms can be grouped as follows:

- Manual lift and release
- 'Rope and cathead' lift and release
- Machine lift with hand-controlled trip
- Machine lift with automatic trip
- Automatic lift and release

Manual lifting and release is rarely used except in some developing countries. Historically, the 'rope and cathead' system shown in Figure 25 was common around the world, especially in North America. The efficiency of the system depends on the number of turns of rope on the cathead and the experience of the operator. Figure 26 shows typical 'Safety' and 'Donut' hammers used in North America. A growing trend in North America is to lift and release the hammer using a winch and cable, instead of a 'rope and cathead'. This can reduce the energy efficiency even further.

The automatic trip mechanism is becoming more common, especially in the USA, UK, Japan and Australia. Figure 27 illustrates an automatic trip hammer. The hammer is often raised using a manually controlled winch and the release is automatic.

In North America there is a growing number of automatic lift and release hammers. These hammers raise the hammer using automatic hydraulic or chain drive systems and automatically release the hammer similar to what is shown in Figure 27.

The size of the anvil associated with the hammer mechanism can vary widely. Heavier anvils tend to reduce the energy transmitted from the hammer and hence increase the measured N-values.

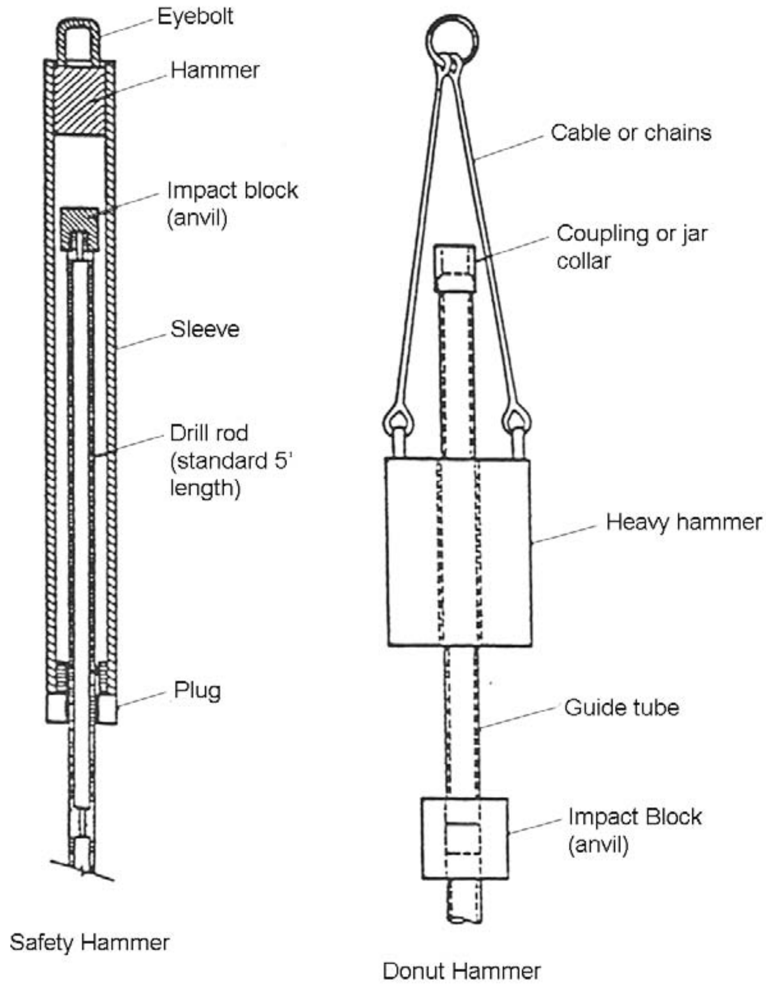


Figure 26 Safety and Donut Hammer

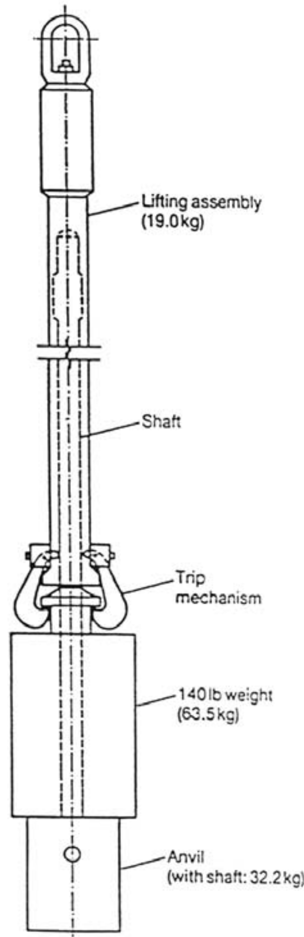


Figure 27 Automatic tripping mechanism

Rod size and stiffness can vary widely even within countries. Long strings or lighter rods tend to produce higher N-values, as a result of energy lost in bending and through the many rod couplings.

In very soft soils the weight of the rods and hammer can overwhelm the penetration resistance and produce very low measured N-values.

The sampler geometry also varies to some extent throughout the world. In North America almost all samplers have enlarged internal diameters to take a sample liner, but they are often used without the liner (Schmertmann, 1979). This inside clearance improves sample recovery, but leads to a reduced SPT N-value.

Test Procedure

The main factors that may influence the SPT N-value associated with test procedures are:

- Seating drive
- Method of measurement
- Rate of hammer blows

The ASTM standard requires a seating drive of 6 inches, and the SPT N-value is taken as the number of blows for the last 12 inches after the seating drive. The relationship between energy delivered from the hammer and the blow count for 12 inches of penetration is not linear when the SPT N-values exceeds about 50. However, it is common to record values in excess of 100 in very stiff soils and soft rocks. The rate of hammer blows can also influence the measured N-value depending on the ground conditions; fast rates can liquefy loose sands below the water table. Fast rates can also cause the hammer to be 'thrown' above the assigned drop height, which can reduce the measure N-values.

The recommended SPT procedure is summarized in Table 10.

Borehole size	2.5 inches < Diameter < 4.5 inches
Borehole support	Casing for full length and/or drilling mud
Drilling	Wash boring; side discharge bit Rotary boring; side or upward discharge bit Clean bottom of borehole*
Drill rods	A or AW for depths of less than 50 feet N or NW for greater depths
Sampler	Standard 2.0 inch O.D. 1.5 inch I.D. > 18 inches length
Penetration Resistance	Record number of blows for each 6 inch penetration N = number of blows from 6 to 18 inches penetration
Blow count rate	30 to 40 blows per minute

*Maximum soil heave within casing < 3 inches (75mm)

Table 10 Recommended SPT Procedure

Factors Affecting Interpretation of the SPT

The factors that influence the interpretation of the SPT are:

- Energy delivered
- Overburden stress
- Ground conditions

Details on the influence of equipment and procedures were given in the previous section. The main way to assess the influence of test equipment on the SPT N-value has been through the measurement of the energy delivered to the SPT rods from the hammer/anvil system. Schmertmann and Palacios (1979) showed that, up to N = 50, the SPT N-value varies inversely with the energy transmitted to the sampler by the rods during the first compressive wave pulse, provided the pulse is of sufficient duration. The energy delivered to the SPT rods is normally expressed in terms of the rod energy ratio (ER). An energy ratio of 60% has generally been accepted as the reference value and represents the approximate historical average SPT energy. The value of the rod energy ratio delivered by a particular SPT set-up depends on the type of hammer/anvil system and the method of hammer lift and release. Values of the correction factor to modify the SPT results to 60% energy (ER/60) can vary from 0.3 to 1.6 corresponding to field values of ER of 20% to 100%.

The potential variation of energy using a rope and cathead system is illustrated in Figure 28. Automatic trip hammers generally produce a narrower range of energy variation with a higher average energy. The automatic lift and trip hammers can vary in efficiency, especially during cold weather when the automatic lift system can be inefficient until warm.

Additional corrections have been developed for rod length, borehole diameter and samplers without liners. Table 11 shows the range of corrections recommended to correct the measured SPT N-value for energy, borehole size, rod length and sampling method.

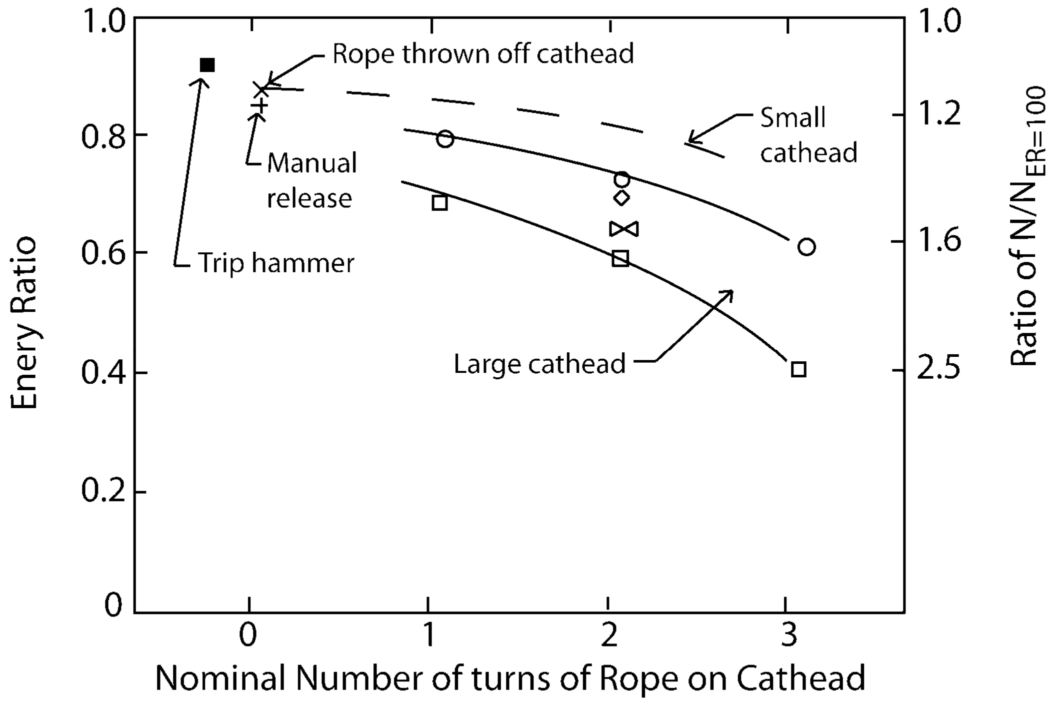


Figure 28 Illustration of variation in SPT energy ratio

Penetration resistance increases with increasing overburden stress. For uncemented, normally consolidated granular soils at a constant relative density, the penetration resistance increases non-linearly with increasing vertical effective stress. It is recommended that the SPT N-value be normalized to its equivalent value at an effective overburden stress of 1 atmosphere (1 tsf or 100 kPa), using a correction factor C_N. A range of C_N values have been recommended over the years, but recently the following simplified expression has been common:

$$C_N = p_a / (\sigma'_{vo})^{0.5}$$

Where p_a is atmospheric pressure (p_a = 1 tsf) in the same units as the vertical effective stress, σ'_{vo}.

Typically the correction factor, C_N, has been limited to maximum values of 2.0 and minimum values of 0.5.

Factor	Equipment Variable	Term	Correction
Overburden Stress		C_N	$(p_a / \sigma'_{vo})^{0.5}$ but < 1.7
Energy Ratio	Donut hammer-rope Safety hammer Automatic hammer	C_E	0.5 to 1.0[1] 0.7 to 1.2[1] 0.8 to 1.3[1]
Borehole Diameter	2.5 to 4.5 inches 6 inches 8 inches	C_B	1.0 1.05 1.15
Rod Length	< 10 feet 10 to 15 feet 15 to 20 feet 20 to 35 feet 35 to 100 feet > 100 feet	C_R	0.75 0.80 0.85 0.95 1.0 <1.0
Sampling Method	Standard sampler Sampler without liner	C_S	1.0 1.1 to 1.3

[1]Values presented are for guidance only; actual ER values should be measured per ASTM D 4633

Table 11 Range of corrections to the SPT
(Youd et al., 2000)

Hence, the SPT N-value corrected for overburden stress, rod length, borehole diameter and sampling method is given by:

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

The ground conditions that can influence the interpretation of the SPT are:

- Granular soils
    - horizontal stress and stress history
    - grain characteristics
    - age and cementation
    - drainage
- Cohesive soils
    - stress history
    - sensitivity
    - soil structure
- Weak and weathered rocks
    - spacing of joints
    - weathering
    - hard inclusions

The in-situ horizontal effective stress has a major effect on the penetration resistance. Therefore, stress (geologic) history of the deposit can have a significant influence on the penetration resistance. Grain characteristics, such as, average grain size, grain size distribution, and particle angularity will also influence the N-values.

The mineralogy of the grains will also influence the N-values, since highly compressible sands, such as carbonate sands or sands with high mica content, will tend to have low penetration resistance. Cementation between particles reduces compressibility and therefore increases the penetration resistance. Cementation is always a possibility, especially in older deposits. Likewise age will tend to make soil deposits stiffer and produce higher penetration resistance, as illustrated in Figure 29.

In clean coarse granular soils the SPT is often carried out in a drained manner, whereas, in silty fine sands the test is close to undrained. When fine sands are dense, this can induce negative pore pressures due to soil dilation resulting in an increase in measured N-values. When fine sands are very loose, the test may induce positive pore pressures which can liquefy the soil around the sampler and significantly reduce the measured N-values.

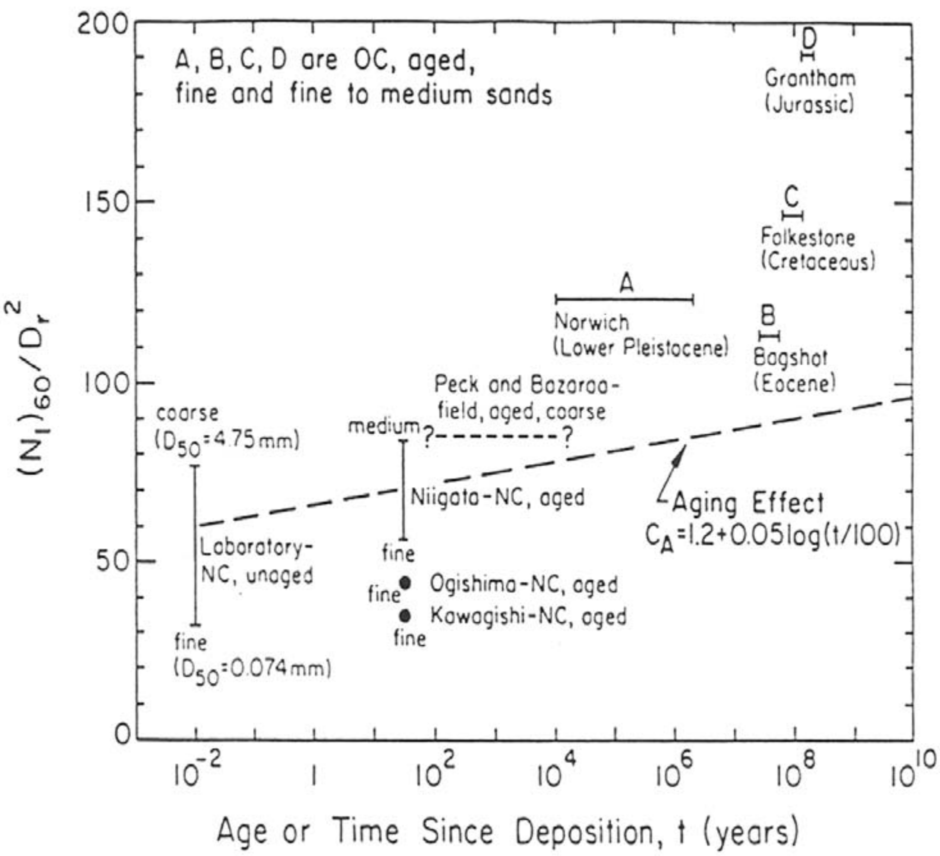


Figure 29 Influence of age on SPT

In cohesive soils, some of the same factors can influence the SPT results. Additional factors such as soil plasticity, sensitivity and structure can also influence the N-values. Schmertmann (1979) showed that up to 70% of the soil resistance in insensitive cohesive soils is derived from side friction. When a cohesive soil is sensitive the SPT N-values can decrease due to the remolding of the soil resulting in a reduction in side friction both inside and outside the SPT sampler. Older cohesive soils can have structures such as cementation and fissures that will influence the penetration resistance.

In weak rocks, the SPT N-value is influenced by the strength of the rock, porosity of the rock, spacing of the joints, aperture and tightness of joints, and the presence of hard intrusions. When joints are widely spaced and tight, such as in less-weathered weak rocks, the resistance is a function of porosity and intact strength. As fractures or joints become more frequent, penetration resistance reduces. In weak and weathered rocks, the SPT is affected by many factors and the interpretation is uncertain.

SPT Interpretation

Numerous empirical correlations have been developed to estimate geotechnical parameters from the SPT N-value for a wide range of soil and weak rock. These correlations vary in their reliability and applicability. Table 12 shows an estimate of the perceived applicability of the SPT to estimate soil parameters.

Because of the dynamic nature of the test, interpretation of the SPT to obtain geotechnical parameters is generally restricted to clean cohesionless soils where penetration takes place under drained conditions. However, correlations have been developed for a wide range of ground.

Soil Type	D_r	Ψ	K_o	OCR	S_t	s_u	ϕ	E,G	M	G_0	k	c_h
Sand	3-4	4		5			3-4	4-5		4-5		
Clay		5	5	4	5	3-4	5	4-5	5	4-5	5	5

1 = high; 2 = high to moderate; 3 = moderate; 4 = moderate to low; 5 = low;
Blank = no applicability

Table 12 Perceived applicability of SPT for deriving soil parameters

Relative Density

Interpretation of SPT data in cohesionless soils has centered on empirical correlations with relative density, D_r. A simple relationship between relative density (Dr) and $(N_1)_{60}$ is:

$$(N_1)_{60} / D_r^{\,2} = \text{constant}$$

Where the constant is 60 for young, normally consolidated sand deposits.

Figure 29 shows the effect of age on the relationship between D_r vs. $(N_1)_{60}$. Age can have a significant influence for deposits older than about 100 years.

Kulhawy and Mayne (1990) suggested combining the factors of particle size, age and overconsolidation into one relationship as follows:

$$D_r^{\,2} = (N_1)_{60} / C_P\, C_A\, C_{OCR}$$

Where the factors for particle size (C_P), age (C_A) and overconsolidation ratio (C_{OCR}) are given by the following:

$$C_P = 60 + 25 \log D_{50} \qquad (D_{50} \text{ in mm})$$

$$C_A = 1.2 + 0.05 \log (t/100) \qquad (t \text{ in years})$$

$$C_{OCR} = OCR^{\,0.18} \qquad (OCR = \sigma'_p / \sigma'_{vo})$$

Almost all the available D_r vs. N_{SPT} correlations have been established for predominantly silica sands. Their use in more crushable and compressible sands, like calcareous sands or silica sands with a large amount of fines can lead to an underestimate of D_r. Because the correlations are based on vertical effective overburden stress (σ'_{vo}), the correlations are only applicable to normally consolidated sands, The empirical correlations between D_r and SPT N-values provide only an estimate of D_r because of uncertainties in drilling technique, energy corrections, compressibility and age of sand, and the general lack of knowledge of in-situ horizontal stresses (σ'_{ho}).

Friction Angle

De Mello (1971) developed an empirical correlation between SPT N-value and ϕ' and σ'_{vo} for cohesionless soils (Figure 30) based on the experimental data from Gibbs and Holtz (1957). However, considering that the correlation was based on data where the energy level during the SPT was unknown but probably low (ER < 60%), the degree of accuracy in prediction of ϕ' must be viewed with caution.

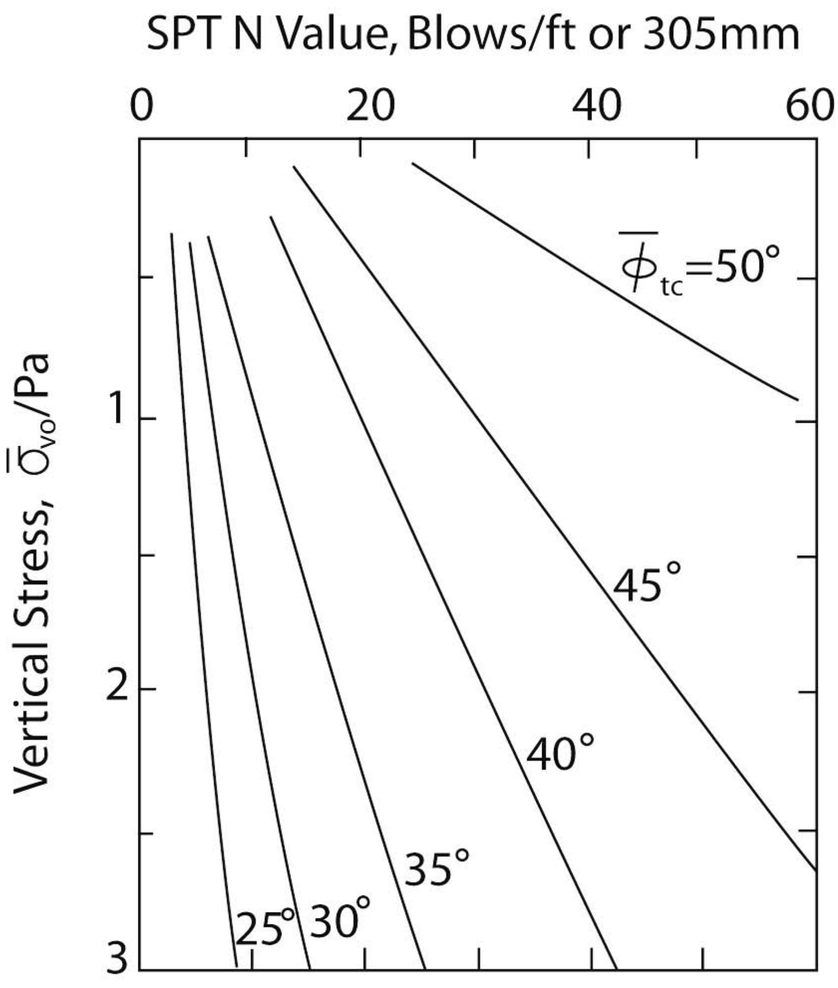


Figure 30 Friction angle from SPT in uncemented, unaged sands

An alternate approach is (Hatanaka & Uchida, 1996);

$$\Phi' = (15.4\ (N_1)_{60})^{0.5} + 20^o$$

Stiffness and Modulus

Predictions of settlement of structures on granular soils make wide use of SPT data. This is generally done by means of direct empirical correlations such as first proposed by Terzaghi and Peck (1948) and improved by Burland and Burbridge (1984).

However, SPT data are also used to estimate modulus for subsequent use in elastic or semi-empirical settlement prediction methods. However, correlations between N-values and moduli (E) are sensitive to stress and strain history, aging and sand mineralogy.

A useful guide for estimating Young's moduli for uncemented predominantly silica sands is given in Figure 31. The modulus has been defined as that mobilized at about 25% of the failure load. For more heavily loaded conditions the modulus would decrease. The results for NC sands are applicable for young recent fills with an age less than 10 years and the results for OC sands are applicable for natural sands with an age greater than 1,000 years.

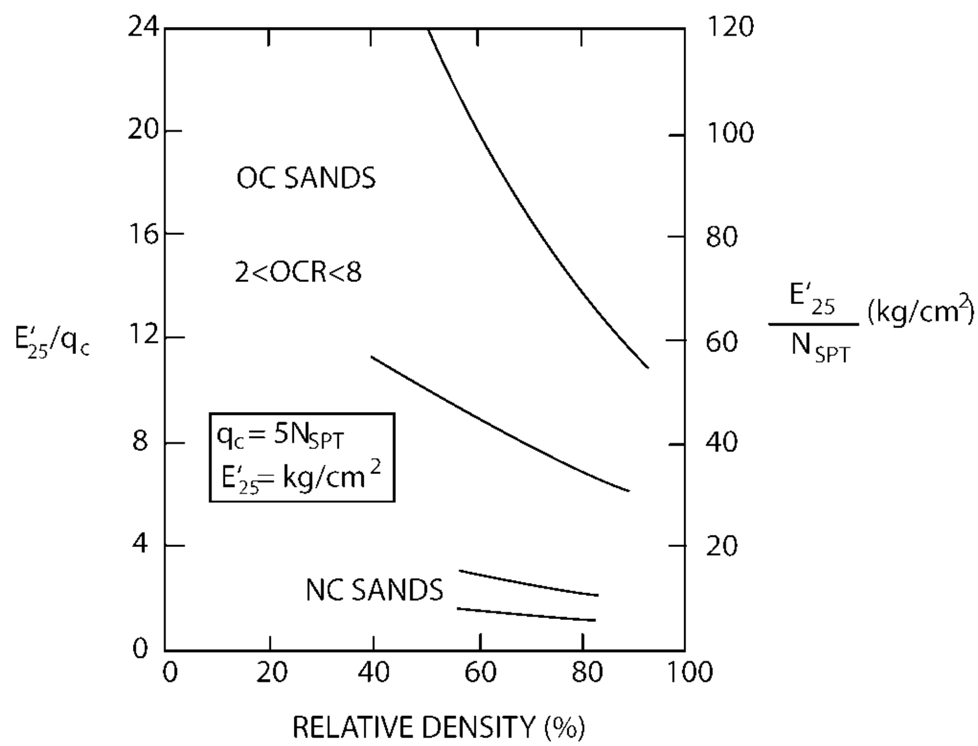


Figure 31 Young’s moduli from SPT in clean, uncemented silca sands

Stroud (1989) recognized the importance of strain level on the mobilized modulus and plotted E'/N_{60} as a function of 'degree of loading' (q_{net}/q_{ult}), as shown in Figure 32. Here the N-value is corrected to 60% rod energy ratio but not for overburden stress level. Values of q_{ult} were estimated using N-values corrected for both energy and stress level (Stroud, 1989). Figures 31 and 32 provide similar estimates of modulus. Both suggest that the stiffness of over-consolidated sands is about 2 to 4 times higher than that of young normally consolidated sands.

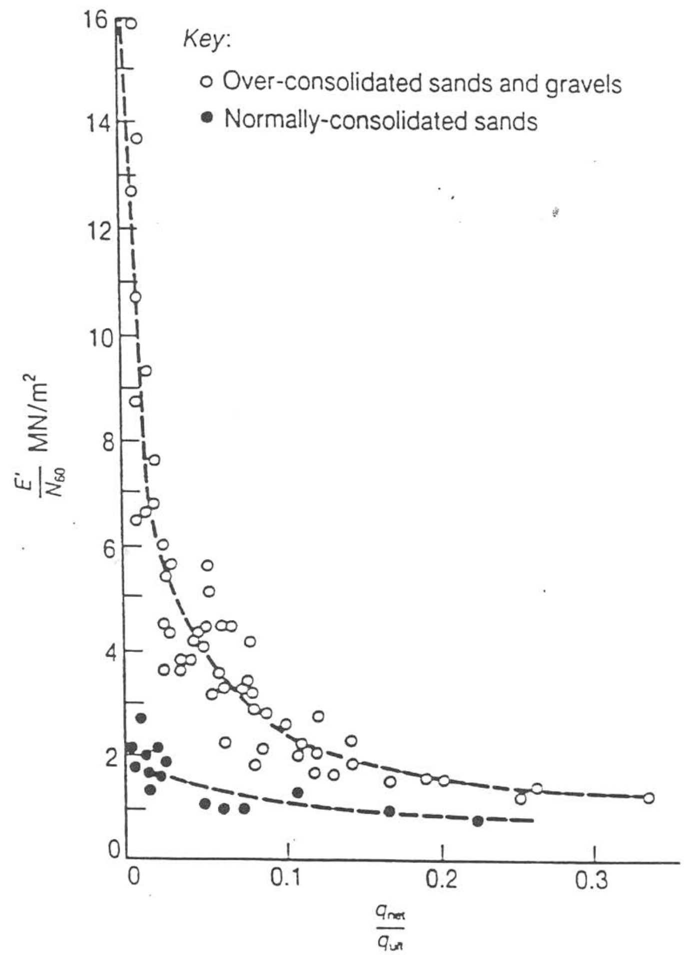


Figure 32 Young's moduli in clean, uncemented silca sands as a function of degree of loading

Undrained Shear Strength

Many correlations have been proposed to estimate s_u from SPT N-values, even though it is known that the link is weak. A summary of the main correlations is shown on Figure 33. Part of the reason for the wide scatter in correlations is the large variation in SPT drilling and test procedures as well as the wide variation in test methods used to measure s_u. Undrained shear strength is not a unique soil parameter, but depends on direction and type of loading. The correlation also depends on soil plasticity, sensitivity and structure.

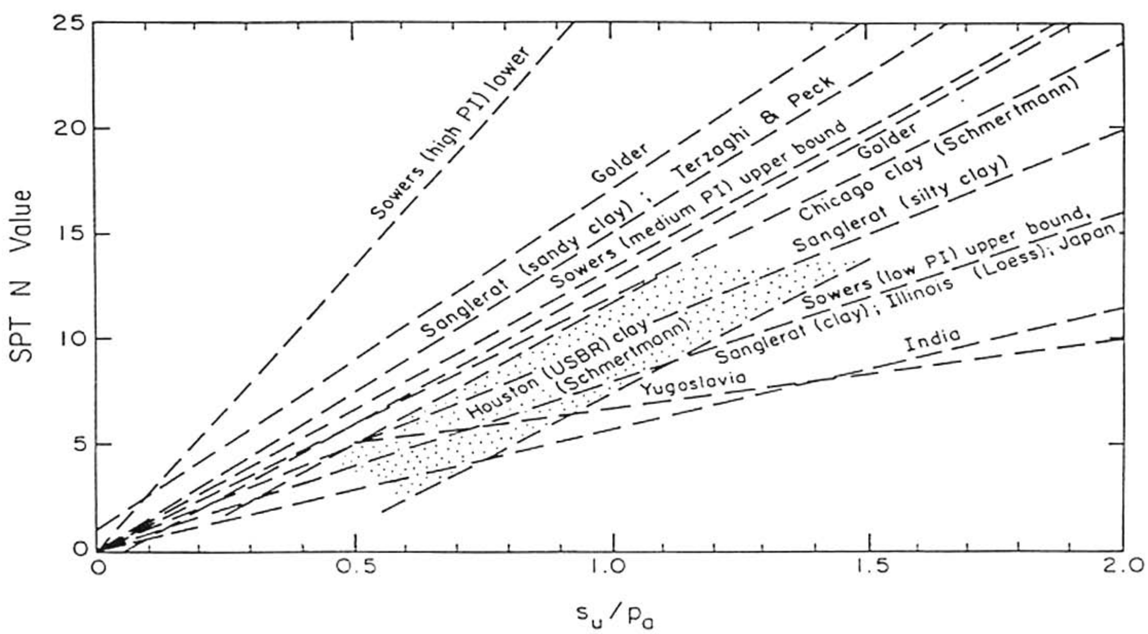


Figure 33 Undrained shear strength of clays from the SPT

Schmertmann (1975) estimated that up to 70% of the SPT resistance comes from side friction for insensitive clays. The very low N-values measured in sensitive clays are thus due to greatly reduced remolded strengths acting along the sides of the SPT sampler. Ladd et al. (1977) suggest that predictions of undrained shear strength (s_u) from SPT N-values are of little value in cohesive soils unless the clay is relatively stiff and insensitive.

Soil structure such as fissuring can also influence the correlation between s_u and SPT N-values. Fissuring can have a large influence on the reference s_u, since the size of the test becomes important.

Stress History (OCR)

Attempts have been made to estimate the stress history of cohesive soils by correlating OCR with SPT N-values. Figure 34 shows a suggested correlation proposed by Kulhawy and Mayne (1990). This relationship is approximate at best and should be used with caution, especially when the measured N-value is less than about 10 and the measurement becomes insensitive.

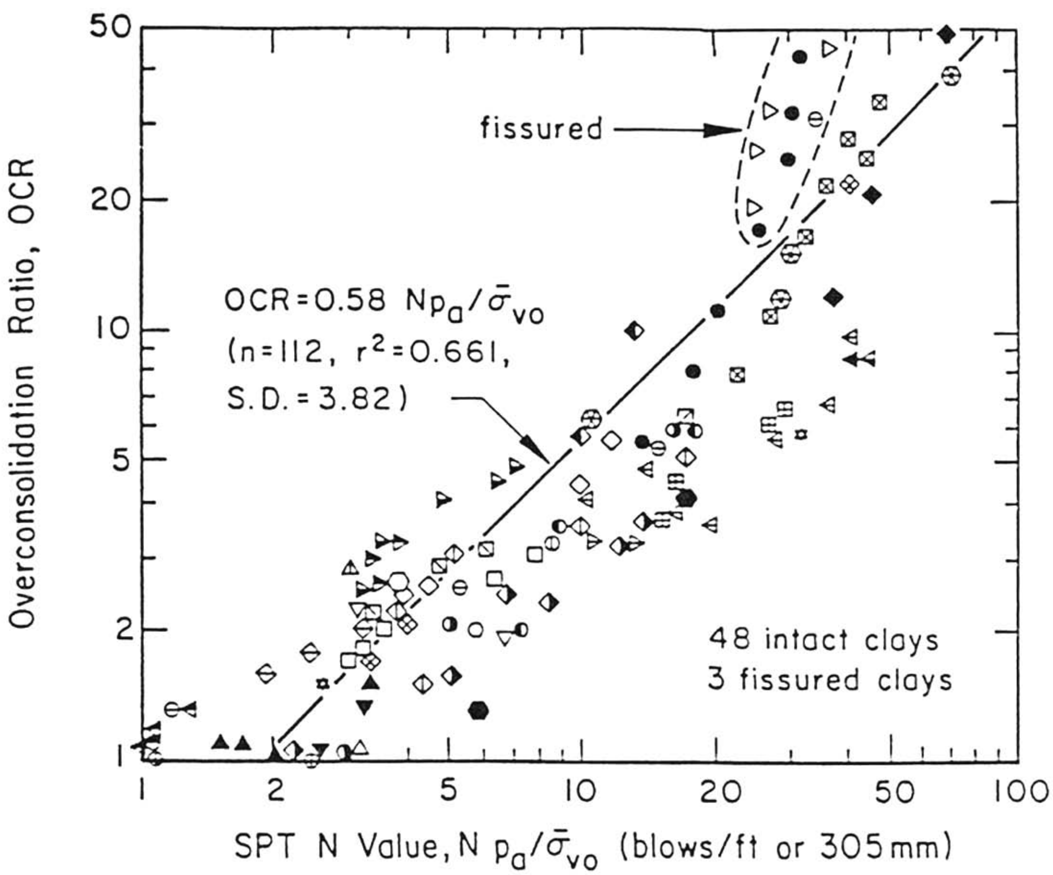


Figure 34 OCR in clays using SPT

Compressibility

The compressibility of cohesive soils is commonly expressed in terms of the compression index (Cc) or the coefficient of volume compressibility (m_v). The predicted compression of cohesive soils is strongly dependent on their stress history (OCR), which can not be reliably estimated from the SPT. Also, for normally to lightly over-consolidated clays, the SPT N-value is likely to be less than 10 and will be an insensitive index of compressibility. Therefore, the use of the SPT to predict settlements of normally to lightly overconsolidated clays is not recommended.

In the UK, the coefficient of volume compressibility of stiff fissured (UK) clays has been correlated with SPT N-values by Stroud and Butler (1975), using:

$$m_v = 400\ N \qquad (m^2/MN)$$

based on oedometer tests on 3-inch (76 mm) diameter samples. However, this correlation is approximate at best, due to problems of laboratory testing with these stiff fissured clays.

SPT Applications

The SPT has maintained a major role in many geotechnical investigations due in part, to its simplicity, low cost and that it provides a sample. The sample, combined with the somewhat crude index of consistency from the blow count, provide a reasonable method for profiling the ground and classifying a wide range of soil types.

As a guide, Table 13 shows a summary of the applicability of the SPT for design applications. The ratings shown in the table have been assigned based on current experience and represent a qualitative evaluation of the confidence level assessed to each design problem and general soil type. Details of ground conditions and project requirements can influence these ratings.

For all these applications, a large amount of SPT results is desirable to capture the natural variability of the deposits and to minimize isolated measurement errors.

Type of Soil	Pile Design	Bearing Capacity	Settlement	Compaction Control	Liquefaction
Sand	2 – 3	1 – 2	2 – 3	2 – 3	1 – 2
Clay	3 – 4	3 – 4	4 – 5	4 – 5	1 – 2
Intermediate Soils	3 – 4	2 – 3	3 – 4	4 – 5	1 – 2

Reliability rating: 1 = High; 2 = High to moderate; 3 = Moderate; 4 = Moderate to low; 5 = low

Table 13 Perceived applicability of the SPT for various direct design problems

Shallow Foundations

The SPT N-value has been used extensively for design of shallow foundations in granular soils for both bearing capacity and settlement. This is generally done by means of direct empirical correlations.

Settlement, rather than bearing capacity criteria usually control design of shallow foundations on sands with a width greater than about 4 ft. (1.2m).

Since predictions of undrained shear strength from SPT N-values are at best crude, the design of shallow foundations in clay using SPT data is therefore rather uncertain.

Deep Foundations

Use is often made of SPT N-values for the design of piles driven into cohesionless soils. Two basic approaches exist:

- Conversion of SPT N-values to CPT q_c and use of CPT pile methods

- Direct use of SPT data for pile design [Reese and O'Neill (1978)]. Because of the large variety of different pile types and installation procedures no single correlation can be expected to provide reliable estimates of pile length or capacity. However, many locally useful correlations have been developed.

Because of the variability in local SPT procedures, foundation design based on SPT data will continue to be based on local experience, until greater standardization of test procedures and energy levels is achieved.

Liquefaction Assessment

After the Niigata earthquake of 1964, use of the SPT to predict the resistance to cyclic loading or "liquefaction potential" developed rapidly. The method proposed by Seed et al. (1985) and updated by Youd et al (2000) is based upon a large amount of field experience, and is widely used and accepted. The main disadvantages are that SPT measurements are discontinuous and are not always reliable and repeatable. The SPT also has very poor resolution in soft fine grained soils, such as silt or sandy silt. The repeatability of the SPT can be somewhat improved if the test is performed according to the conditions suggested by Seed et al. (1985) and Youd et al (2000).

The reliance placed upon the SPT N-value for predicting liquefaction potential or deformation under cyclic loading appears somewhat surprising considering the large variability associated with the test. Seed et al. (1983) have noted that because the empirical approach is founded on such a large body of field data, they believe the method to be the best available at that time. However, they also note that "the SPT cannot be performed conveniently at all depths [say deeper than 30 m (100 ft.) or through large depths of water] or in all soils [such as those containing a significant proportion of gravel particles]". Therefore, it is desirable that the SPT be supplemented by other in-situ test methods which can also be correlated with soil liquefaction, such as the Cone Penetration Test (CPT), seismic wave velocity measurements (Vs), and the Becker Penetration test for gravelly soils.

Compaction Control

SPT data has been used extensively for evaluation of compaction techniques such as virbro-compaction, dynamic compaction and vibratory rollers. Most compaction techniques besides increasing the density, induce significant changes in the horizontal stresses. The SPT N-value, like any penetration resistance, is also influenced by many factors, the most important being soil density and in-situ stresses.

Variability and inconsistency in SPT N-values can cause major problems on large ground improvement projects when different SPT equipment is used.

The SPT is generally not a good indicator of compaction techniques in fine grained soils with appreciable fines content.

SPT in Soft Rocks

Soft rocks are often difficult to sample, and this has lead to attempts to apply the SPT to interpret their engineering characteristics. The group comprises soft and weathered rocks, including chalk, marl, shale, and poorly cemented sandstone.

New Developments

Recently an additional measurement has been added to the SPT. Ranzini, (1988) and Decourt (1991, 1998) have suggested the measurement of the torque at the end of the SPT to estimate the side friction on the sampler. Upon completion of the 18-inches SPT drive, the sampler is turned by means of a torque wrench applied to the drill rods at the ground surface, and the maximum torque, T, is measured in either t/ft or kgf/m units. Decourt (1991) suggested comparing the measured torque with the associated SPT N-value, and that the ratio T/N would be an indirect measure of soil structure.

Summary

The SPT continues to be used world wide because of its simplicity, low cost and because it provides a sample. It is important that engineers specifying, supervising, reporting and using the SPT results understand fully the factors that influence the test and recognize its limitations.

Field Vane Test (FVT)

The field vane test (FVT) consists of inserting a simple four-bladed vane into either clay or silt and rotating the device about a vertical axis and measuring the torque. Limit equilibrium is used to relate the measured torque to the undrained shear strength mobilized. Both peak and remolded strengths can be measured. A selection of vanes is available in terms of size, shape and configuration, depending on the consistency and strength of the soils.

The standard vane (ASTM D 2573) has a rectangular geometry with a blade height to diameter ratio of 2. Figure 35 shows a typical field vane.

Figure 35 Typical field vane showing protective sheath for pushing in soft clays. A standard 10 cm^2 cone penetrometer is shown for scale.

Test Procedure

Test procedures are outlined in ASTM D 2573. The test is often carried out by pushing the vane into the soil from the bottom of a borehole and the vane should be pushed at least four borehole diameters below the base of the borehole to avoid disturbance from drilling. The test can also be carried out using direct-push equipment pushing from the ground surface when there are no hard layers. Within 5 minutes after insertion, rotation should be carried out at a constant rate of 6 degrees per minute (0.1^o/s) with frequent measurements of the mobilized torque. Figure 36 illustrates a typical FVT.

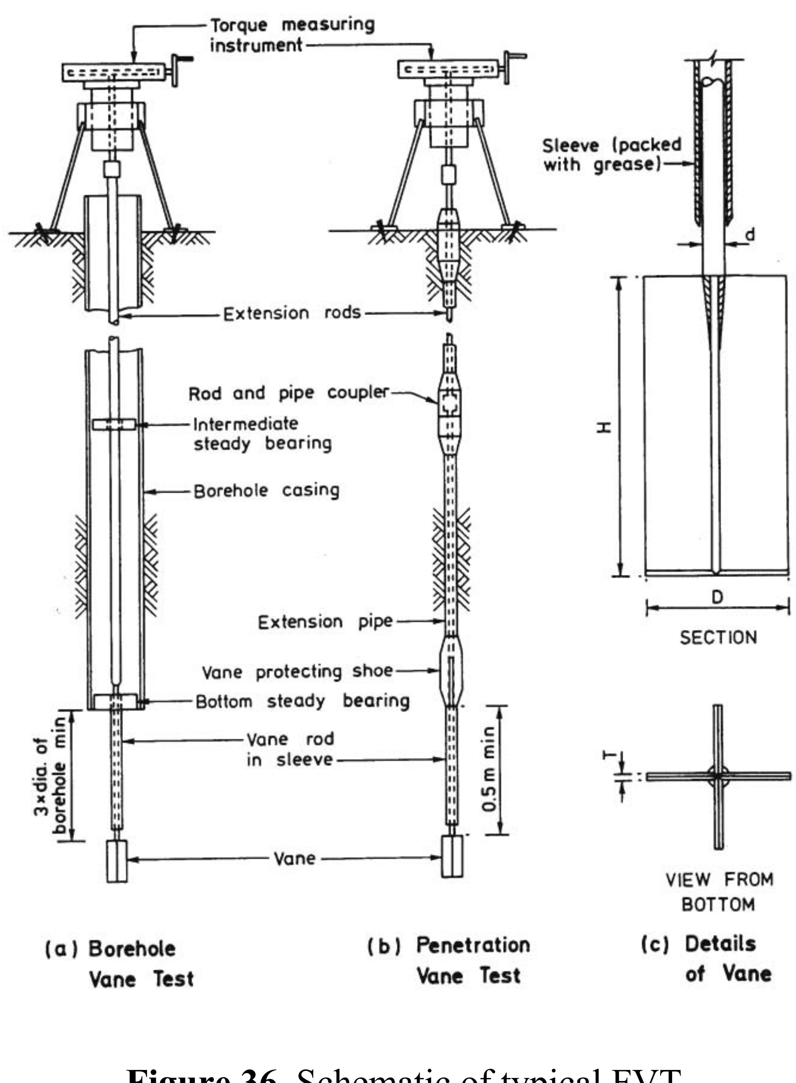


Figure 36 Schematic of typical FVT

Depending on the type of equipment used, there is the potential for friction to develop along the push rods. This friction needs to be either minimized or accounted for in the measurements. Typical methods to minimize or account for rod friction include:

Protected rods within a sheath
Measurement of friction with a slip coupling

Figure 35 shows an example of a protective sheath to remove rod friction.

Undrained Shear Strength and Sensitivity

The conventional interpretation to obtain the FVT undrained shear strength (s_{uv}) from the maximum torque (T_{max}) assumes a uniform distribution of shear stresses both top and bottom and along the blades and a vane with a height-to-width ratio H/D = 2:

$$s_{uv} = 6T_{max} / 7\pi D^3$$

After the peak $s_{uv(peak)}$ is obtained, the vane is rotated quickly through 10 complete revolutions and the test repeated to measure the remolded values ($s_{uv(remolded)}$). The sensitivity, S_t is then:

$$S_t = s_{uv(peak)} / s_{uv(remolded)}$$

Vane Correction Factor

Since there is no unique value for the undrained shear strength of fine grained soils, it is common that the FVT strength is corrected prior to application in stability analyses involving embankments on soft ground, bearing capacity and excavations in soft ground. The mobilized shear strength is given by:

$$\tau_{mobilized} = \mu_R \, s_{uv}$$

Where μ_R is an empirical correction factor that has been related to plasticity index (PI) based on back calculated case histories of full-scale projects.

The following expression has been recommended (Chandler, 1988):

$$\mu R = 1.05 - \boldsymbol{b} \text{ (PI)}0.5$$

Where the parameter b is a rate factor that depends on the time to failure (t_f in minutes) and is given by:

$$b = 0.015 + 0.0075 \log t_f$$

Flat Dilatometer Test (DMT)

The flat dilatometer test (DMT) uses pressure readings from an inserted flat plate to obtain estimates of soil type and various soil parameters. The device consists of a tapered stainless steel blade with an 18^o wedge tip that is pushed vertically into the ground at 8-inch (200mm) intervals at a rate of (0.8-inch/s) (20mm/s). The blade is connected to a readout pressure gauge at the ground surface via a special air-tubing with internal wire pre-threaded through push rods. A 2.4 inch (60mm) diameter flexible steel membrane located on one side of the blade is inflated pneumatically to give two pressure readings:

- A-reading: a lift-off or contact pressure where the membrane becomes flush with the face ($\delta = 0$), and,
- B-reading: an expansion pressure corresponding to $\delta = 1.1$mm outward deflection at the center of the membrane.

A small spring-loaded pin at the membrane center detects the movement and relays to a buzzer at the readout unit. Normally nitrogen gas is used to inflate the membrane. After the B-reading, the membrane is quickly deflated and the blade pushed to the next test depth. Procedures are given in ASTM D 6635.

Two calibrations are taken before each sounding to obtain corrections for membrane stiffness in air. These corrected 'A' and 'B' readings are recorded as p_0 and p_1 respectively using:

- $p_0 = A + \Delta A$
- $p_1 = B - \Delta B$

Where ΔA and ΔB are calibration factors for the membrane stiffness in air.

Figure 37 shows a schematic of the DMT probe and the two measurements.

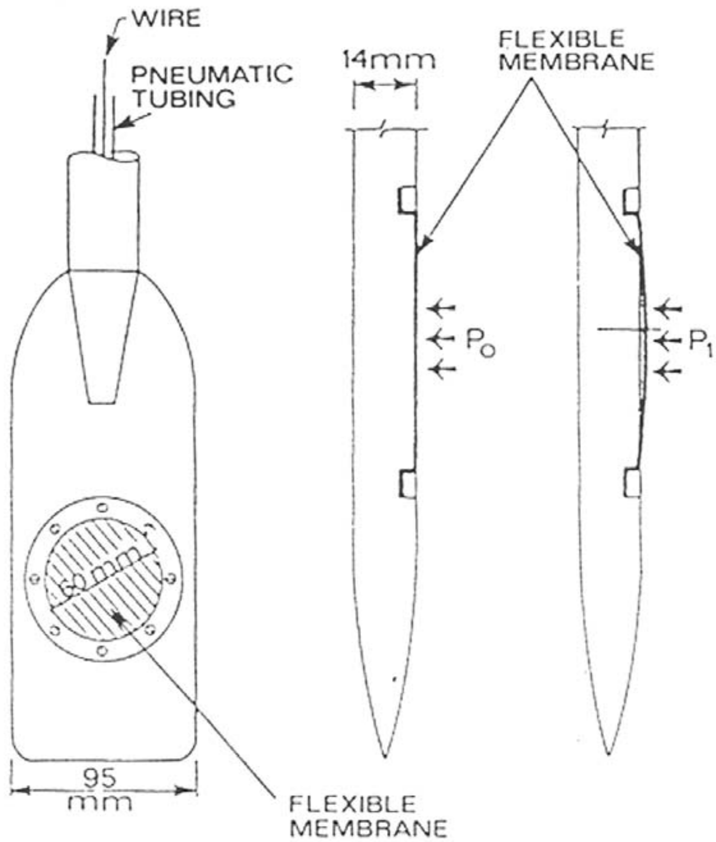


Figure 37 Schematic of DMT probe and the two measurements

The two DMT readings are utilized to provide three indices that can provide estimates of soil type and various soil parameters:

- Material Index: $I_D = (p_1 - p_0)/(p_0 - u_0)$
- Horizontal Stress Index: $K_D = (p_0 - u_0)/ \sigma'_{vo}$
- Dilatometer Modulus $E_D = 34.7(p_1 - p_0)$

Where u_0 = in-situ hydrostatic pore water pressure and σ'_{vo} = in-situ vertical effective stress.

Figure 38 shows an example of a DMT sounding.

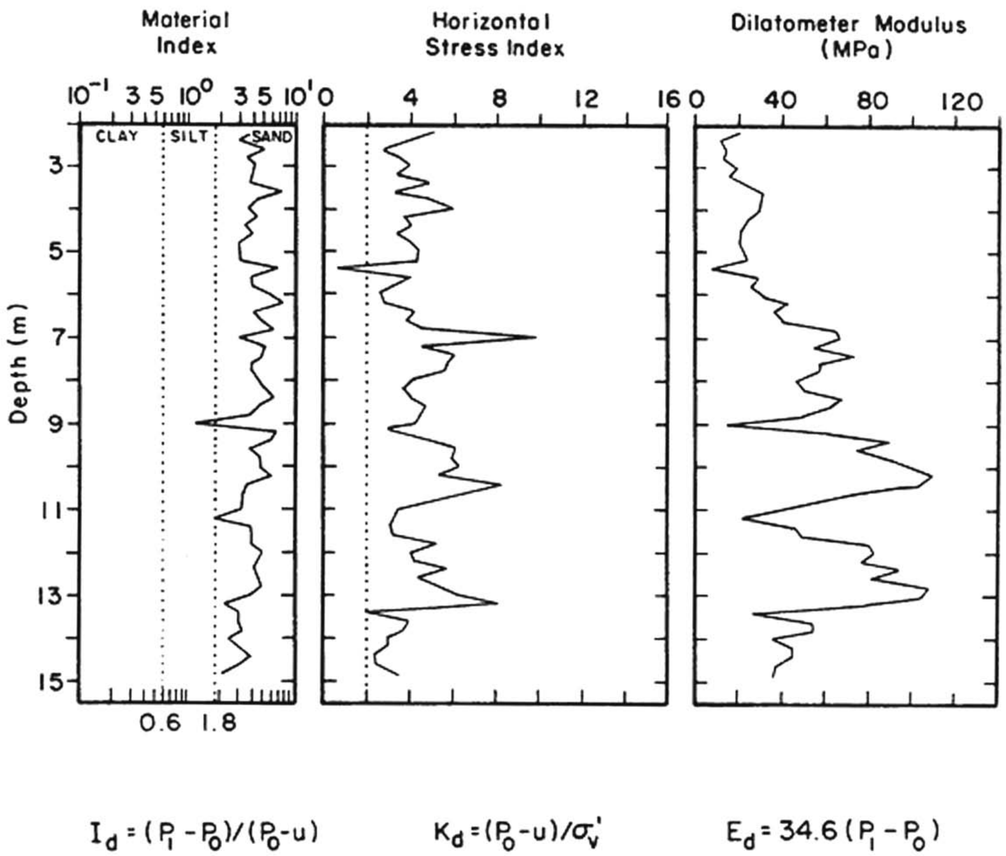


Figure 38 Example of DMT sounding

Some of the challenges for the DMT are:

- Push force is approximately twice that for a standard cone (CPT),
- Membrane is susceptible to damage in hard and gravely soil, and,
- No theoretical basis for interpretation.

Modifications to the basic DMT include:

- C-reading: deflation pressure to where the membrane again becomes flush with the face ($\delta = 0$),
- Thrust-force: force to push blade into the ground,
- Dissipation readings with time,
- Seismic wave velocity measurements.

Pressuremeter Test (PMT)

The pressuremeter consists of a long cylindrical probe that has a flexible membrane that is expanded radially into the surrounding ground. The pressure-expansion is measured in terms of a change in either volume or diameter of the probe and the inflation pressure. The pressure-expansion curve can be interpreted to give an estimate of the stress-strain-strength response of the ground.

The original 'pressiometer' was introduced by the French engineer Louis Menard in 1955. The Menard type pressuremeter has a complex triple-cell design, whereas newer designs are mono-cell with simpler control panels. Standard probes range from 1.5 inch (35mm) to 3-inch (75mm) diameter with length-to-diameter ratios from 4 to 6. Procedures are given in ASTM D 4719.

Equipment and Test procedures

The three basic pressuremeter devices are defined by the method of installation:

- Pre-bored
- Self-bored
- Full-displacement

Pre-bored Pressuremeter Test (PBPMT): This test is carried out in a pre-bored borehole. The Menard type pressuremeter is a pre-bored pressuremeter. A typical pressure-expansion curve is shown in Figure 39. The PBPMT is described in ASTM D 4719.

For the Menard PBPMT, the results are simplified into two key parameters; the pressuremeter modulus (E_M) and the limit pressure (P_l). The pressuremeter modulus (E_M) is derived from the approximately linear portion of the pressure-expansion curve and is a measure of the stiffness of the ground. The limit pressure (P_l) is the pressure when the probe has doubled in volume and is a measure of the strength of the ground. An example of Menard PBPMT results is shown in Figure 40.

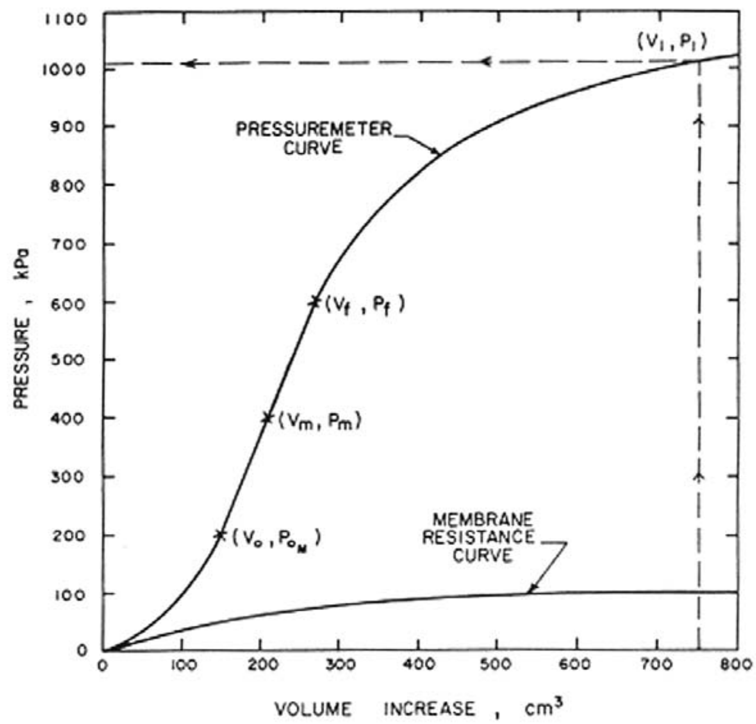


Figure 39 Schematic test result from PBPMT

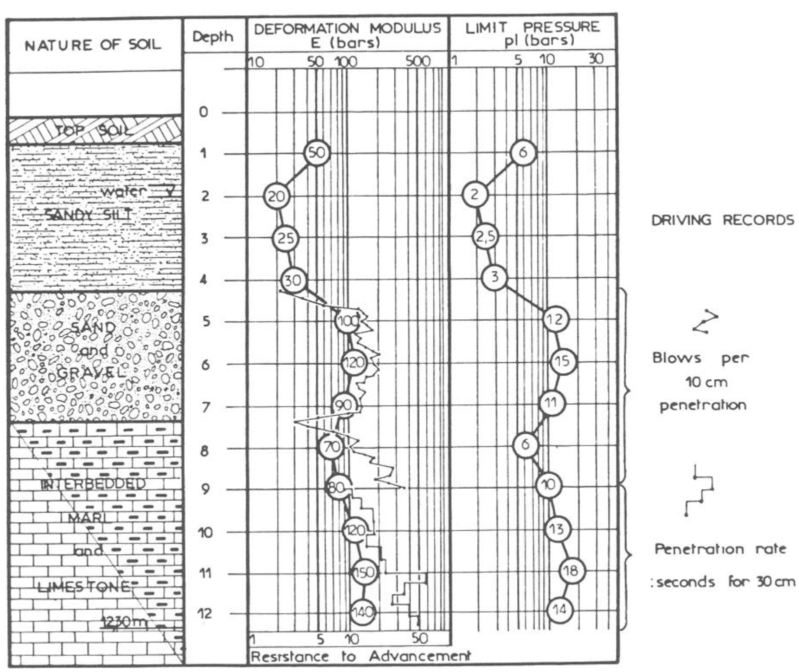


Figure 40 Example of Menard PBPMT results

Soil disturbance due to the pre-boring of the borehole is inevitable in a PBPMT. The type and amount of disturbance depends on the method of borehole preparation and the soil type. Experience shows that soil disturbance has a significant influence on the (E_M) but less influence on (P_l). To reduce the influence of soil disturbance on PBPMT results Menard developed standard test procedures and borehole techniques.

Self-bored Pressuremeter Test (SBPMT): This test is carried out by self-boring the probe into the ground. The self-boring can be done using either an internal rotary cutter or by a jetting device. The cuttings are returned through the hollow center of the probe. A typical SBPMT pressure-expansion curve is shown in Figure 41.

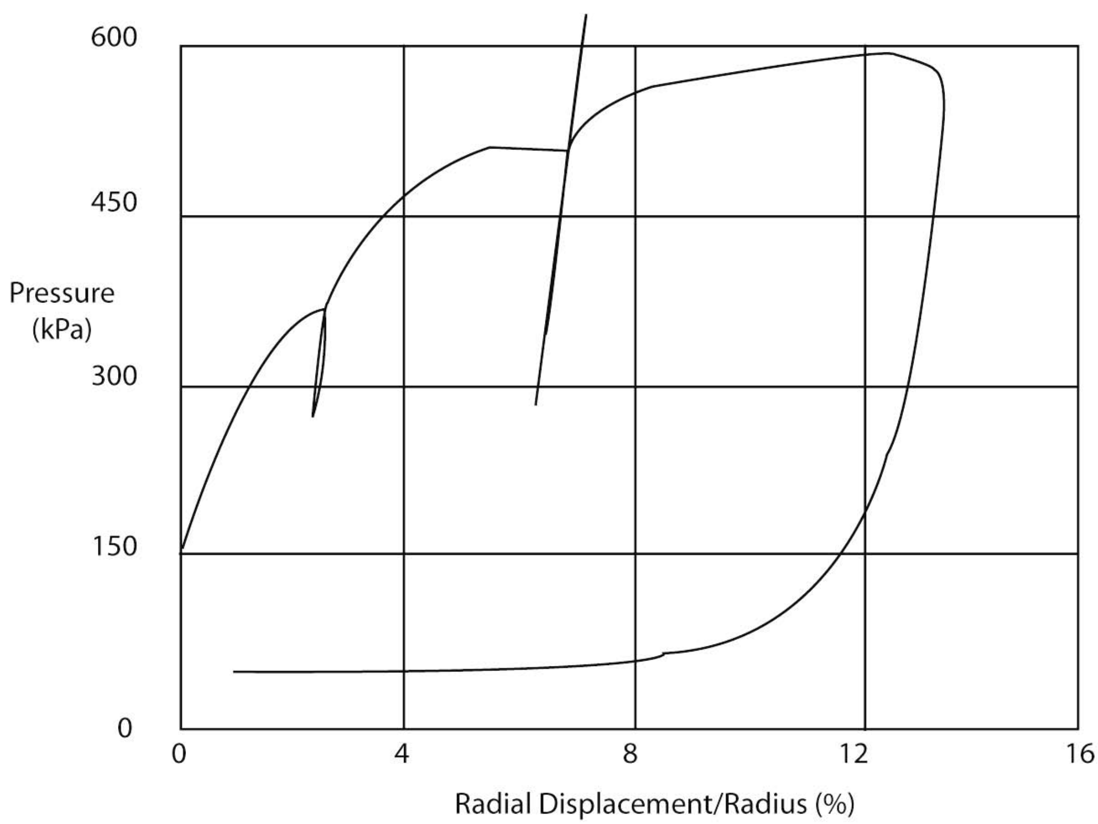


Figure 41 Example of SBPMT in sand, with two unload-reload loops

SBPMT results are often interpreted using cavity expansion theory to derive in-situ horizontal stress and stress-strain-strength characteristics.

Although the goal in a SBPMT is no soil disturbance, some disturbance always occurs. Generally, disturbance is larger in stiffer soils. Hence, SBPMT's are generally limited to softer soils.

Full-displacement Pressuremeter Test (FDPMT): This test is carried out after the probe is pushed into the ground in a full-displacement manner, i.e. as a closed-ended device. Often the probe is located behind a cone to form a cone-pressuremeter. In the case of a FDPMT, soil disturbance is inevitable, but the disturbance is repeatable. Figure 42 shows a schematic of a cone-pressuremeter and the range of measurements.

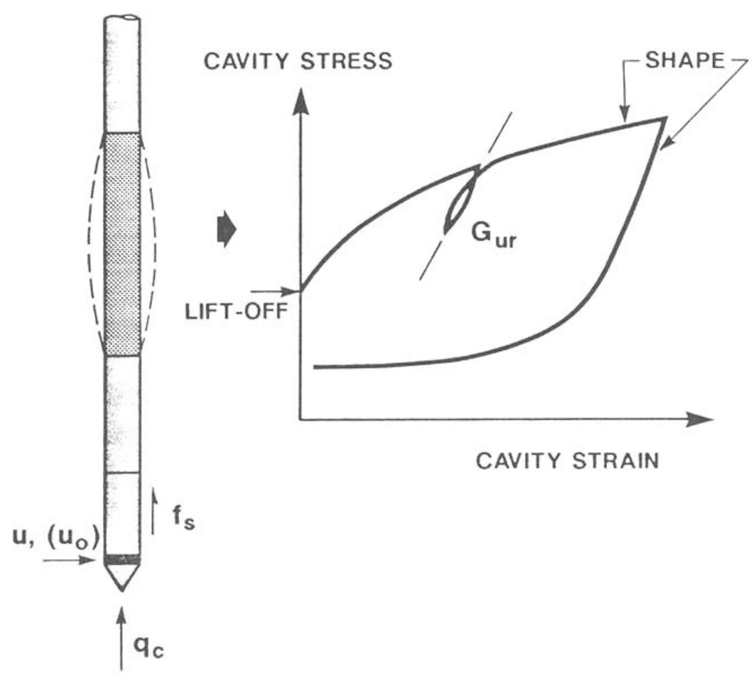


Figure 42 Schematic of a cone-pressuremeter, FDPMT
(Robertson and Hughes, 1986)

General

Probe expansion can be measured using either strain arms to record the change in diameter or fluid volume to measure the change in volume of the probe. Most probes are mono-cell (i.e. one pressure cell).

Calibrations are required to correct the measurements for membrane stiffness and system compliance. The probe is inflated in air to record the membrane stiffness and inflated in a very stiff steel cylinder to record system compliance. It is also common for the flexible membrane to be protected using a steel sheath.

It is common for almost all forms of pressuremeter testing to perform small unload-reload cycles to evaluate the 'elastic' stiffness of the ground. Since the initial pressure expansion loading includes soil disturbance effects, small unload-reload cycles can provide a useful measure of the medium-strain level stiffness of the ground. Figures 41 and 42 show examples of small unload-reload cycles to measure soil stiffness. System compliance can be critical for an accurate measure of the ground stiffness from small unload-reload cycles.

A major advantage of the PBPMT is that the test can be performed in a very wide rage of ground conditions from soft soils through to rock, since the test is carried out in a pre-bored hole. In general, the PBPMT is better suited to stiff soils, since membrane stiffness can often dominate the test results in soft soils. In very stiff ground, such as rock, system compliance is critical and pre-bored pressuremeter probes often have special strain sensors to reduce system compliance effects. Self-boring pressuremeter tests are often limited to soft soils where self-boring is effective and soil disturbance small. Full-displacement (cone-pressuremeter) tests are often limited to deep-water off-shore investigations where the cost of the ship warrants the expense of the equipment and test.

Pre-bored pressuremeter tests are often interpreted using empirical techniques using the extensive Menard published correlations. However, in these cases, the PBPMT should be carried out according to the standard Menard techniques to minimize errors due to variability in soil disturbance and variations in equipment and test procedures.

Self-boring pressuremeter tests are often interpreted using cavity expansion theories, but the data are often complex and it can be difficult to evaluate and incorporate soil disturbance.

Full-displacement pressuremeter tests are often interpreted using semi-empirical techniques.

Computer-aided curve fitting has become increasingly popular for interpretation of all forms of pressuremeter test results, since it can incorporate various stress-strain relationships and account, to some degree, soil disturbance.

In general, pressuremeter testing is slow and expensive.

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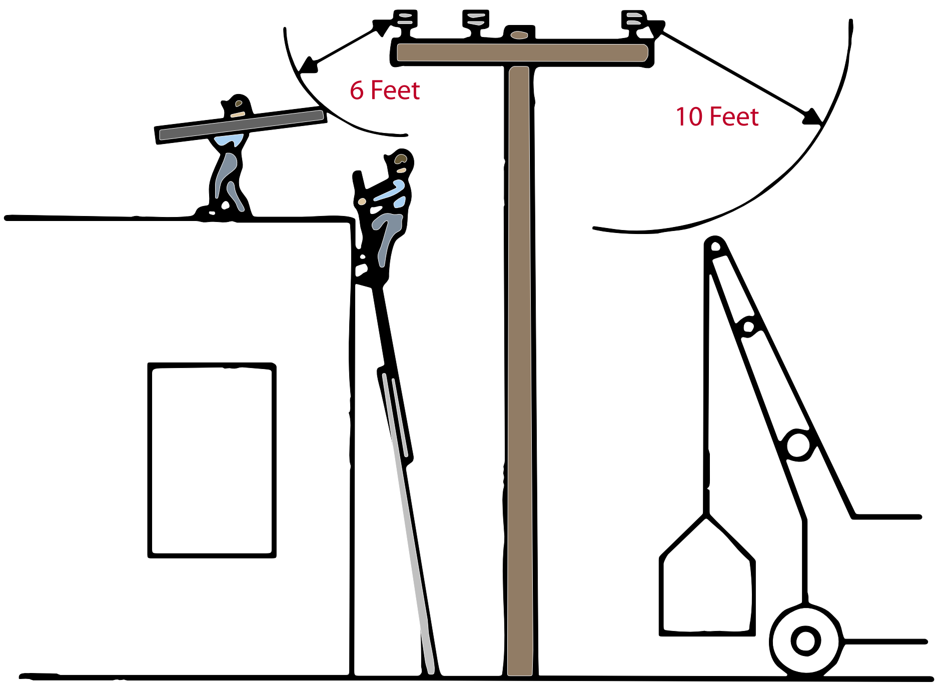

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