

Guide to Cone Penetration Testing for Geo-Environmental Engineering



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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

Length	feet	(ft)
Area	square feet	(ft ²)
Force	pounds	(p)
Pressure/Stress	pounds/foot ²	(psf)

SI Units

meter	(m)
square meter	(m ²)
Newton	(N)
Pascal	(Pa) = (N/m ²)

Multiple Units

Length	inches	(in)	millimeter	(mm)
Area	square feet	(ft ²)	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:	$q_t = q_c + u_2(1-a)$
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

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By

**P. K. Robertson
and
K.L. Cabal (Robertson)**

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Gregg Drilling & Testing, Inc.
Corporate Headquarters
2726 Walnut Avenue
Signal Hill, California 90755

Telephone: (562) 427-6899
Fax: (562) 427-3314
E-mail: info@greggdrilling.com
Website: www.greggdrilling.com

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Basic CPT 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.

$$q_t = q_c + u_2(1 - a_n) \quad (a_n = \text{net area ratio, typically } 0.7 \text{ to } 0.8)$$

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.

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

Normalized cone resistance, Q_{tn}

The cone resistance expressed in a non-dimensional form taking account of the in-situ vertical stresses and where the stress exponent (n) varies with soil type. When $n = 1$, $Q_{tn} = Q_t$.

$$Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{P_{a2}} \right) \left(\frac{P_a}{\sigma'_{vo}} \right)^n$$

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 face

u_2 when measured just behind the cone.

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

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

In recent years there has been a steady increase in geo-environmental projects where geotechnical/geological engineering has been combined with environmental concerns. Many of these projects involve some form of ground contaminant which can take the form of vapors, liquids and solids. Hence, there has been a change in site characterization techniques to accommodate these environmental issues related to contaminants. Traditional drilling techniques generally produce considerable disturbance to the materials surrounding the drill hole, which can have a significant effect on subsequent sample quality. With increasing application of data quality management, drilling and sampling techniques are becoming less acceptable. In addition, drilling methods produce cuttings of the material removed from the drill hole which may require special handling and costly disposal methods.

The most rapidly developing site characterization techniques for geo-environmental purposes involve direct push technology, specifically cone penetration tests. Direct push devices generate essentially no cuttings, produce little disturbance and reduce contact between field personnel and contaminants, since the penetrometer push rods can be decontaminated during retrieval. The most popular direct push logging test for geo-environmental investigations in soil is the Cone Penetration Test (CPT). The CPT provides a rapid, repeatable, reliable and cost effective method of continuous measurement.

The purpose of this guide is to provide a concise resource for the application of the CPT for geo-environmental projects. This guide is a supplement to the book ‘CPT in Geotechnical Practice’ by Lunne, Robertson and Powell (1997). This guide is applicable primarily to data obtained using a standard electronic cone with a 60-degree apex angle and either a diameter of 35.7 mm or 43.7 mm (i.e. either 10 or 15 cm² cross-sectional area).

Recommendations are provided on applications of CPT data for soil profiling, material identification, evaluation of geotechnical parameters and application of CPT technology to geo-environmental projects. This guide provides only the basic recommendations for the application of the CPT for geo-environmental projects.

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?

In general, projects can be classified as either, 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/or severe consequences. The level of sophistication in a site investigation is also a function of the project objectives and the potential for cost savings.

Expedited Site Assessment

Expedited site assessment (ESA) is a framework for rapidly characterizing site conditions for input into corrective active decisions. It involves field generated data and on-site interpretation, a flexible sampling and analytical program and experienced staff in the field who are authorized to make sampling and analytical decisions. A summary of the ESA process is shown in Figure 1.

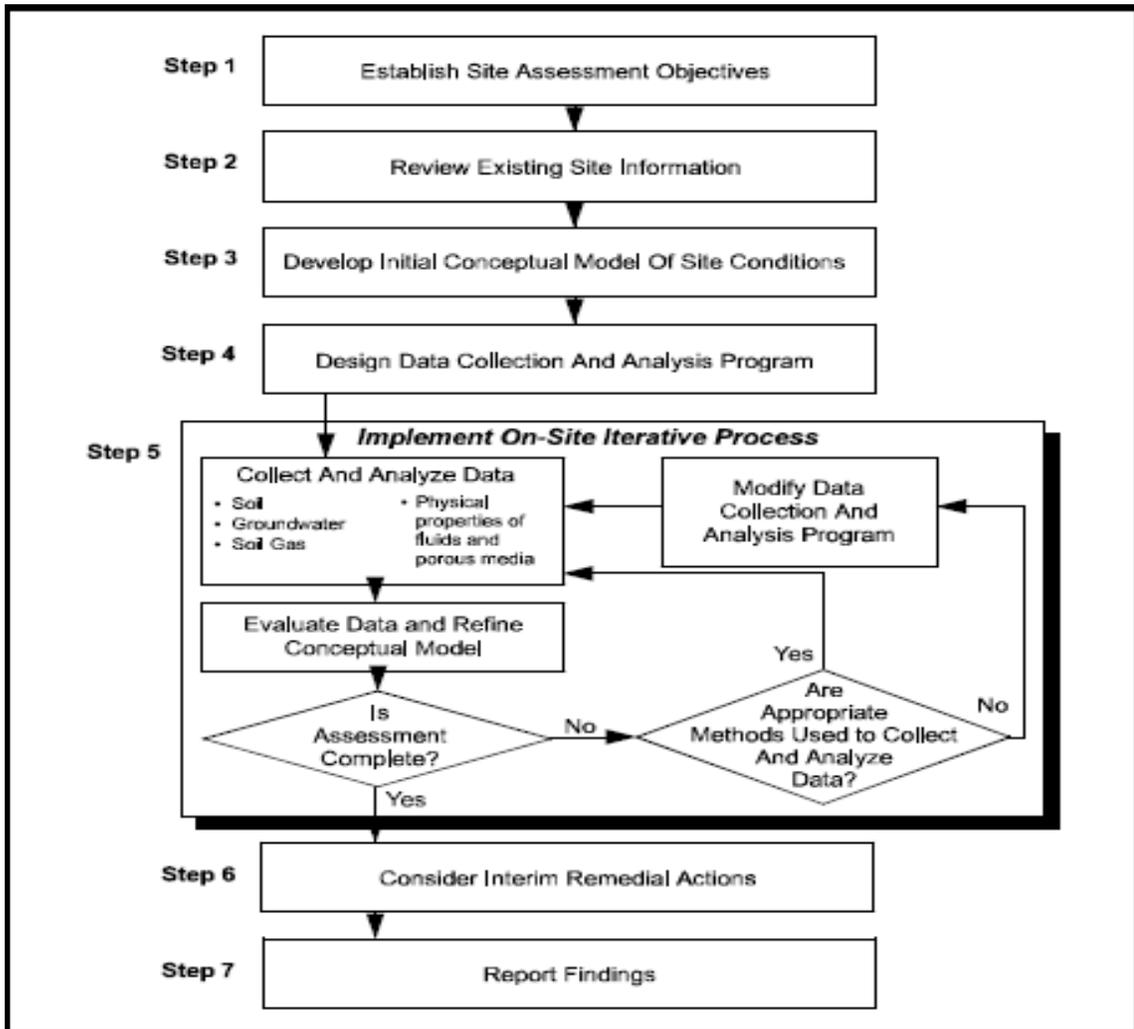


Figure 1 Summary of the Expedited Site Assessment Process

A comparison between ESA and conventional site assessment is shown in Table 1.

Process Component	Conventional Site Assessments	Expedited Site Assessments
Number of phases of investigation	Multiple	Single
Technical Strategy	Focus on plan view map; sampling location based on limited information. Sampling locations are pre-determined	Use of multiple complementary technologies; sampling locations depend on existing data; minimal well installation
Work plan	Rigid plan	Flexible plan
Data analysis	Interpretation of data is weeks or months later	Regular (hourly/daily) interpretation of data
Innovative technologies (i.e. direct push and field analytical methods)	May or may not be used; not integrated into process	Standard practice, on-site interactive process

Table 1 Comparison between ESA and Conventional Site Assessment

The CPT and associated sampling equipment are ideal to collect and evaluate on-site data in an ESA program.

Role of the CPT

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 contaminants can exist in vapor, liquid and solid forms. The above requirements are a function of the proposed project and the associated risks. The variety in geological conditions and range in project requirements make the subject complex. There are many techniques available to meet the objectives of a site investigation and these include both field and laboratory testing. An ideal investigation program would likely include a mix of field and laboratory tests depending on the risk of the project.

For geo-environmental projects where potential contaminants are identified, long term monitoring and sampling maybe required for both design and either remediation or containment. Hence, the objectives for geo-environmental site characterization can be quite different from those for a more traditional geotechnical site characterization.

The Cone Penetration Test (CPT) and its enhanced versions with additional sensors have extensive applications in a wide range of soils. Although the CPT is limited primarily to softer soils, with modern large 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 (operator-independent)
- Economical and productive
- Strong theoretical basis for interpretation
- Additional sensors

Disadvantage of CPT:

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

Sensors have been developed to identify certain contaminants. Although a disadvantage is that no soil sample is obtained during a CPT, it is possible to obtain soil samples using CPT direct-push equipment. Samples can be obtained of solids, liquid and vapor.

The continuous nature of CPT results provides detailed stratigraphic profiles to guide in selective sampling appropriate for the project. The recommended approach is to first perform several CPT soundings to define the stratigraphic profile and to provide initial estimates of soil and groundwater conditions and then follow with selective sampling, appropriate to the project. The type and amount of sampling will depend on the project requirements and risk as well as the stratigraphic profile. Typically sampling will be focused in the critical zones as defined by the CPT. Several soil, fluid and vapor samplers are available that can be pushed in to the ground using CPT pushing equipment. Details on various sampling methods are provided in later sections.

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 2 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* ($CPTu$), pore pressure is also measured, typically behind the cone in the u_2 location, as shown in Figure 2.

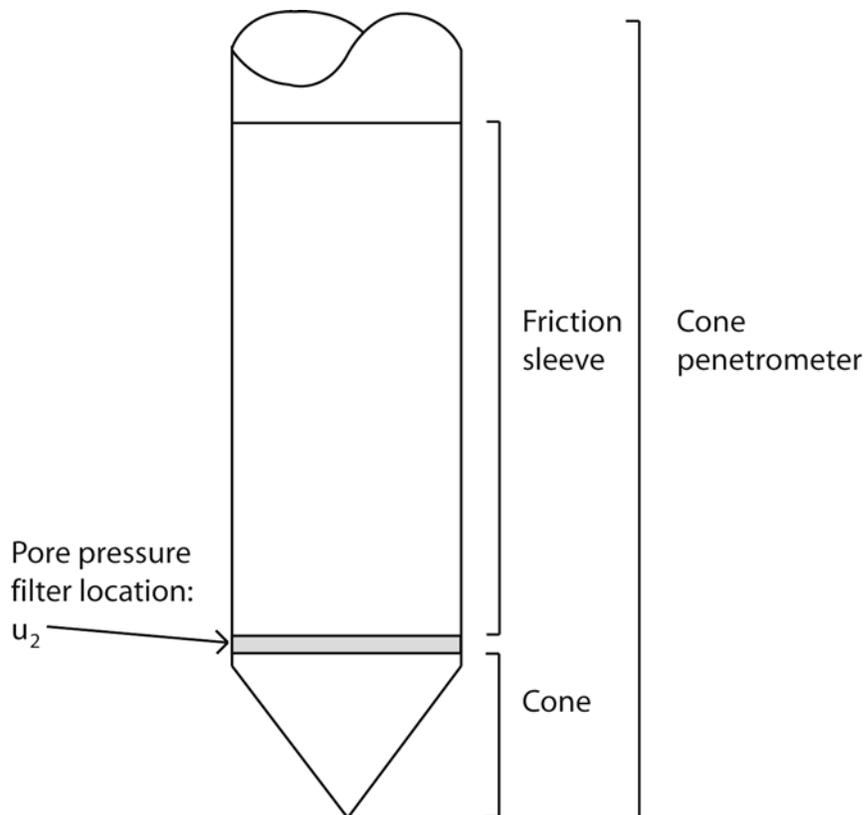


Figure 2 Terminology for cone penetrometers

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 cm² projected area and a 60° apex angle was attached to the steel inner push rods, as shown in Figure 3.

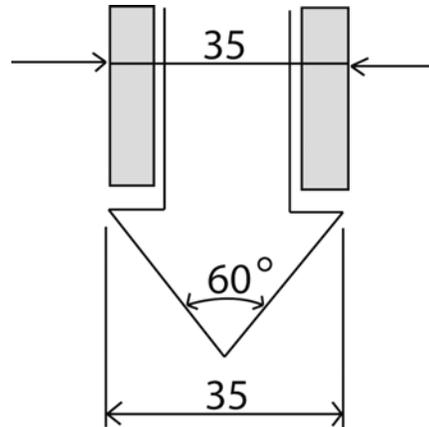


Figure 3 Early Dutch mechanical cone (After Sanglerat, 1972)

1935

Delft Soil Mechanics Laboratory designed the first manually operated 10 ton (100 kN) cone penetration push machine, see Figure 4.

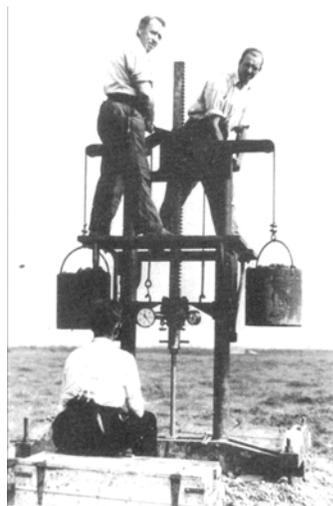


Figure 4 Early Dutch mechanical cone (After Delft Geotechnics)

1948

The original Dutch mechanical cone was improved 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 5, are still in use in some parts of the world.

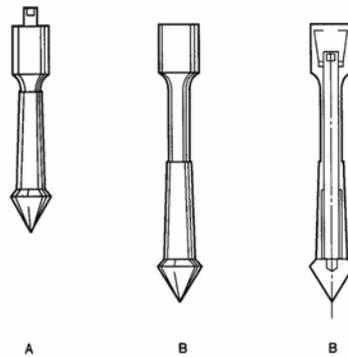


Figure 5 Dutch mechanical cone penetrometer with conical mantle

1953

A friction sleeve ('adhesion jacket') was added behind the cone to include measurement of the local sleeve friction (Begemann, 1953), see Figure 6. Measurements were made every 8 inches (20 cm), and for the first time, friction ratio was used to classify soil type (see Figure 7).

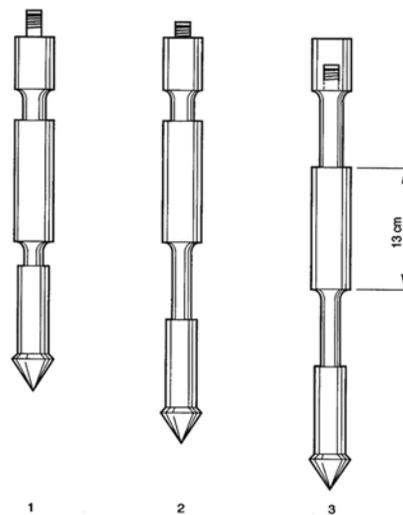


Figure 6 Begemann type cone with friction sleeve

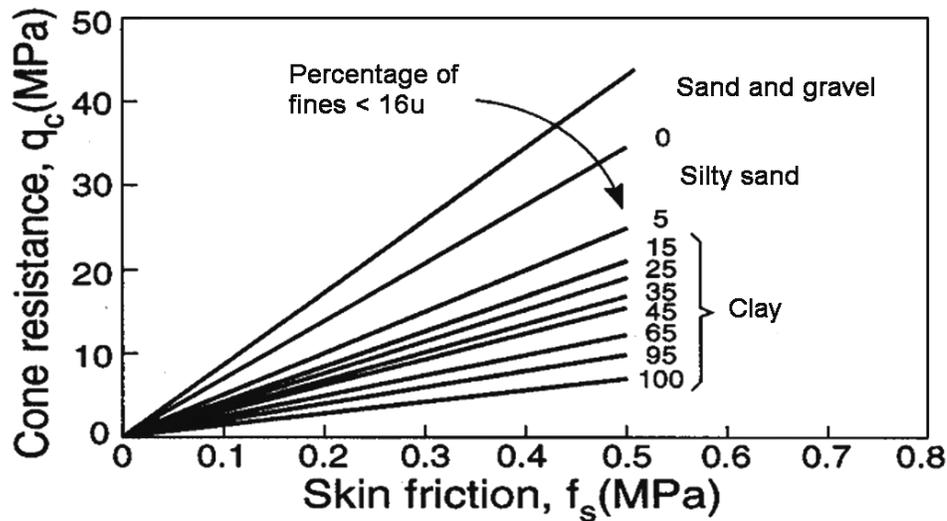


Figure 7 First soil classification for Begemann mechanical cone

1965

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

- 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

Cone penetrometers that could also measure pore pressure (piezocone) were introduced. 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 cm² and 15 cm² probes the most common and specified in most standards. Figure 8 shows a range of cones from a mini-cone at 2 cm² to a large cone at 40 cm². The mini cones are used for shallow investigations, whereas the large cones can be used in gravelly soils.



Figure 8 Range of CPT probes (from left: 2 cm², 10 cm², 15 cm², 40 cm²)

Pushing Equipment

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

On Land

Pushing equipment for land applications generally consist of specialty 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 Portable 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 100 feet/30m), see Figures 13 and 14.



Figure 13 Mid-size jack-up boat



Figure 14 Gregg Quinn Delta ship with spuds

Depth of Penetration

CPT's can be performed to depths exceeding 300 feet (100m) in soft soils and with large capacity pushing equipment. To improve the depth of penetration, the friction along the push rods should be reduced. This is normally done by placing an expanded coupling (friction reducer) a short distance (typically 3 feet, ~1m) behind the cone. Penetration will be limited if either very hard soils, gravel layers or rock are encountered. It is common to use 15 cm² cones to increase penetration depth, since 15 cm² cones are more robust and have a slightly larger diameter than the 10 cm² push rods and do not require a friction reducer.

Test Procedures

Pre-drilling

For penetration in fills or hard soils it may be necessary to pre-drill in order to avoid damaging the cone. Pre-drilling, in certain cases, may be replaced by first pre-punching a hole through the upper problem material with a solid steel dummy probe with a diameter slightly larger than the cone. It is also common to hand auger the first 5 feet (1.5m) in urban areas to avoid underground utilities.

Verticality

The thrust machine should be set up so as to obtain a thrust direction as near as possible to vertical. The deviation of the initial thrust direction from vertical should not exceed 2 degrees and push rods should be checked for straightness. Modern cones have simple slope sensors incorporated to enable a measure of the non-verticality of the sounding. This is useful to avoid damage to equipment and breaking of push rods. For depths less than 50 feet (15m), significant non-verticality is unusual, provided the initial thrust direction is vertical.

Reference Measurements

Modern cones have the potential for a high degree of accuracy and repeatability (0.1% of full-scale output). Tests have shown that the zero load

output of the sensors can be sensitive to changes, such as temperature. It is common practice to record zero load readings of all sensors to track these changes. Zero load readings should be recorded at the start and end of each CPT.

Rate of Penetration

The standard rate of penetration is 2 cm/sec (approximately 1 inch per second). Hence, a 60 foot (20m) 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 typically produce continuous analogue data. Most systems convert the data to digital form at selected intervals. Newer cones convert the signal to digital form downhole in the cone for improved transmission quality. Most standards require the data collection 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 (50 mm) being 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 both 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 many hours in plastic clays. Dissipation rate increases as probe size decreases.

Calibration and Maintenance

Calibration checks should be carried out at regular intervals (approximately every 6 months). For major projects, check calibrations can be carried out before and after the field work, with functional checks during the work.

Functional checks should include recording and evaluating the zero load measurements (baseline) before and after every sounding.

With careful design, calibration, and maintenance, strain gauge load cells and pressure transducers can have an accuracy and repeatability of better than +/- 0.1% of full scale reading.

Table 2 shows a summary of checks and recalibrations for the CPT

<i>Maintenance</i>	<i>Start of Project</i>	<i>Start of Test</i>	<i>End of Test</i>	<i>End of Day</i>	<i>Once a Month</i>	<i>Every 6* months</i>
<i>Wear</i>	<i>x</i>	<i>x</i>			<i>x</i>	
<i>O-ring seals</i>	<i>x</i>			<i>x</i>		
<i>Push-rods</i>		<i>x</i>			<i>x</i>	
<i>Pore pressure-filter</i>	<i>x</i>	<i>x</i>				
<i>Calibration</i>						<i>x</i>
<i>Computer</i>					<i>x</i>	
<i>Cone</i>					<i>x</i>	
<i>Zero-load</i>		<i>x</i>	<i>x</i>			
<i>Cables</i>	<i>x</i>				<i>x</i>	

Table 2 Summary of checks and recalibrations for the CPT

* depending on stability of zero-load readings

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 with a typical value of between 0.70 to 0.85. In sandy soils $q_c = q_t$.

Basic Soil Mechanics

Introduction

A soil can be visualized as a skeleton of solid particles enclosing continuous voids containing water and/or air. For the range of stresses usually encountered in practice the individual solid particles and water can be considered incompressible. The volume of the soil skeleton as a whole can change due to rearrangement of the solid particles into new positions, mainly by rolling and sliding, with the corresponding change in the forces acting between particles. In a fully saturated soil, a reduction in volume is possible only if some of the water can escape from the voids.

Groundwater

Groundwater is the water occurring beneath the earth's surface that fills (saturates) the void space of sediments. In a soil environment, coarse soils such as sand and gravel deposits usually provide good flow of water and are referred to as aquifers; whereas, the finer-grained clay and silt deposits provide relatively poor flow of water and are referred to as aquitards.

Groundwater and surface water are connected physically in the hydrologic cycle. The two most important physical properties for groundwater flow are porosity, n , (void ratio is more commonly used in geotechnical engineering) and hydraulic conductivity, k , (which is often expressed as the coefficient of permeability).

Porosity (n) is the ratio of the volume of the voids in sediments to the total volume of the material, whereas void ratio (e) is the ratio of the volume of the voids to the volume of the solids. The two are linked by:

$$n = e / (1+e)$$

Porosity is usually expressed as a percentage, e.g. when $n = 50\%$, $e = 1.0$. The term effective porosity refers to the degree of interconnectedness of the pore spaces. For coarse sediments, such as, sand and gravel, the effective porosity can be equal to the overall porosity. However, in fine-grained sediments, such as clay, the effective porosity can be very low due to water

that is tightly held in the small pore spaces and the bound water held by the surface chemistry of clay particles.

Hydraulic conductivity (k), is a measure of a sediments ability to transmit water. The size, shape, and interconnectedness of pore spaces affect hydraulic conductivity. The presence of a small percentage of fines in a coarse-grained soil results in a value of permeability significantly lower than for the same soil without the fines. If a soil is stratified the permeability for flow parallel to the direction of stratification is higher than that for flow perpendicular to the direction of stratification. Hydraulic conductivity is expressed in units of length/time: feet/day, meters/day, cm/sec, etc. The values of k for different types of soil are typically within the range of 1 to 10^{-10} m/s (i.e. ten orders of magnitude).

In most depositional environments, coarse-grained deposits are interbedded with finer-grained deposits creating a series of aquifers and aquitards. When a saturated aquifer is bounded on top by an aquitard (also known as a confining layer), the aquifer is called a confined aquifer. Under these conditions, the water is under pressure so that it will rise above the top of the aquifer if the aquitard is penetrated by a well. The elevation to which the water rises is known as the potentiometric (or piezometric) surface. When an aquifer is not bounded on top by an aquitard, the aquifer is said to be unconfined. In an unconfined aquifer, the pressure on the top surface of the groundwater is equal to that of the atmosphere (i.e. the pore water pressure is zero). This surface is known as the water table or phreatic surface. The arrangement of aquifers and aquitards in the subsurface is referred to as hydrostratigraphy.

When a well penetrates a confined aquifer with a potentiometric (piezometric) surface that is higher than the land surface, water will flow naturally to the surface. This is known as artesian flow, and results from pressure within the aquifer. The pressure results when the recharge area for the aquifer is at a higher elevation than the point at which discharge is occurring. The confining layer prevents the groundwater from returning to the surface until that layer is penetrated by a well. When a CPT penetrates a confined aquifer, equilibrium pore pressure measured by the CPT will be higher than hydrostatic pressure below the water table, due to the elevated piezometric surface. Hence, the equilibrium pore pressure measured during a pause in a CPT defines the piezometric surface at that depth. Figure 15 is an example of variable

piezometric surfaces as defined from three CPT dissipation tests that show a downward flow.

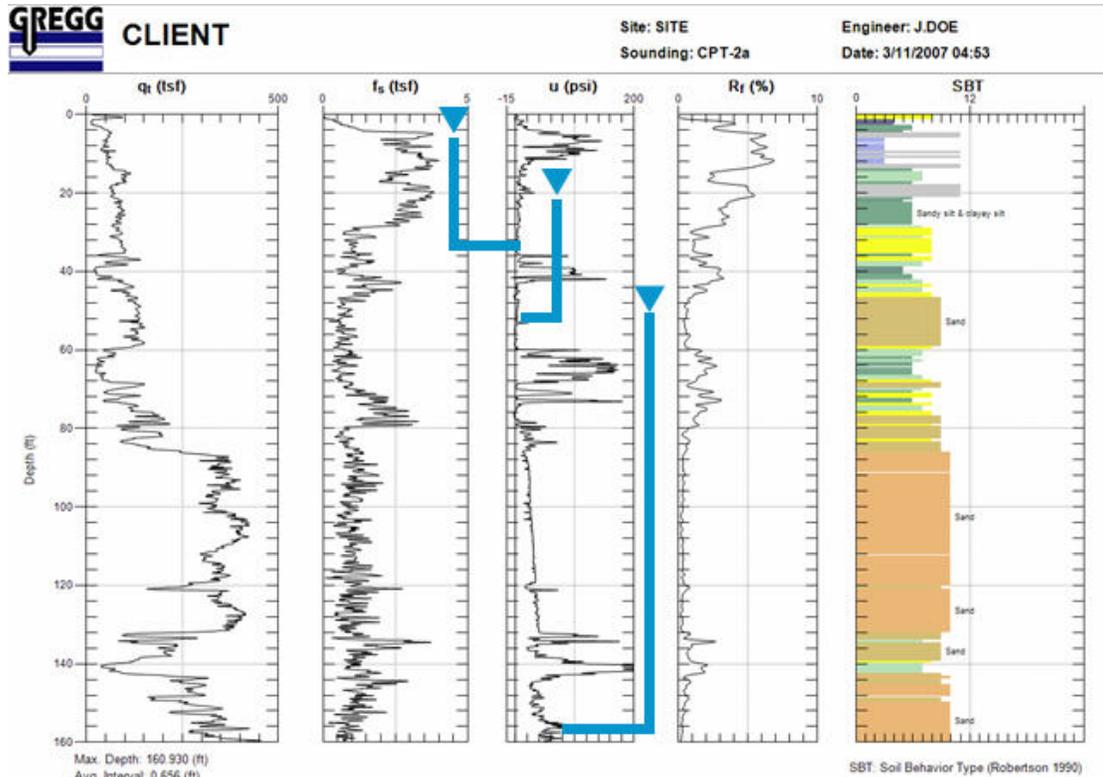


Figure 15 Example of different piezometric surfaces defined from CPT
(Note: 1 psi = 2.31 feet of water)

Below the water table the soil is assumed to be fully saturated although it is likely that, due to the presence of small volumes of entrapped gas, the degree of saturation will be slightly less than 100%, especially near to the water table. The level of the water table changes according to climatic conditions and human activity. A *perched* water table can occur locally, contained by soil of low permeability, above the normal water table level. The region immediately above the water table can be essentially saturated due to capillary tension in the small pore spaces. The zone of saturated soil above the water table can be large in fine-grained soils and small in coarse-grained soils. Above the saturated zone the soil is unsaturated and is referred to as the *vadose* zone, due to the vapor (gas) in the interconnected pore spaces.

Movement of Groundwater - Seepage

Movement of groundwater is controlled by the difference in total hydraulic head, which is the combination of elevation and pressure head. In an unconfined aquifer, the water table elevation represents the hydraulic head, while in a confined aquifer the potentiometric (piezometric) surface represents the hydraulic head. Water moves in response to the difference in hydraulic head from the point of highest energy (head) towards the lowest. On a regional scale this results in flow of groundwater from recharge areas to discharge areas. The rate of groundwater flow depends on the hydraulic conductivity (k) and the rate of change of hydraulic head over some distance.

Below the water table the pore water may be static, with the hydrostatic pressure increasing with depth below the water table, or the pore water could be seeping through the soil under a hydraulic gradient. When the water is moving (flowing) Bernoulli's theorem applies to the pore water. Since seepage velocities in soil are normally small, the velocity head can be neglected. Thus:

$$h = u/\gamma_w + z$$

where: h = total hydraulic head, u = pore water pressure, γ_w = unit weight of water (9.81 kN/m³ or 62.4 lb/ft³), and z = elevation head above a chosen datum.

In one dimension, water flows through a fully saturated soil in accordance with Darcy's empirical law:

$$q = A k i$$

or

$$v = q/A = k i$$

where: q = volume of water flowing per unit time, A = cross-sectional area of soil corresponding to the flow q , k = coefficient of permeability, i = hydraulic gradient, and v = discharge velocity. The units of the coefficient of permeability are those of velocity (m/s or ft/s). The hydraulic gradient is given by:

$$i = \Delta h / L$$

where: Δh = total hydraulic head loss, and L = the length of flow over which the hydraulic head loss occurs. The hydraulic head loss is the difference in total hydraulic head over the zone of flow ($\Delta h = h_2 - h_1$).

Note that under hydrostatic conditions, where no flow is occurring, the total hydraulic head is constant (i.e. the hydraulic head loss is zero, $\Delta h = 0$) but the pore pressure increases with depth (i.e. elevation).

The Principle of Effective Stress

Shear stresses can only be resisted by the skeleton of solid particles, by means of forces developed at the interparticle contacts. Normal stress can be resisted by the soil skeleton through an increase in the interparticle forces. If the soil is fully saturated, the water filling the voids (pores) can also withstand normal stress by an increase in (pore) pressure.

Soil behavior is controlled by the effective interparticle stress defined as:

$$\sigma' = \sigma - u$$

where: σ' = the effective normal stress, representing the stress transmitted through the soil skeleton only

σ = the total normal stress, representing the force per unit area transmitted in a normal direction across the soil imagined as a solid material

u = the pore pressure, the pressure of the water filling the void space between the solid particles. The pore pressure acts equally in every direction.

Figure 16 illustrates the concept of effective stress.

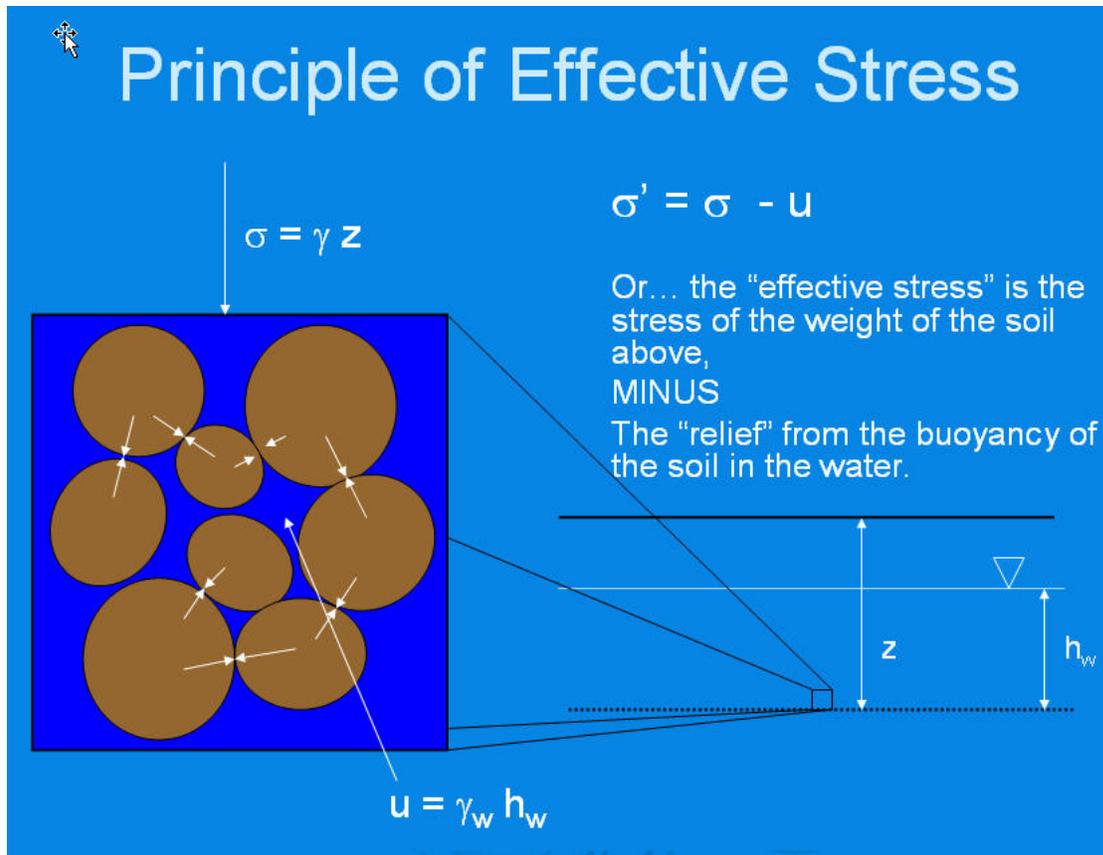


Figure 16 Principle of effective stress in soil

Consider a soil mass having a horizontal surface and with the water table at the surface level. The total vertical stress (i.e. the total normal stress on a horizontal plane) at depth z is equal to the weight of all material (solids + water) per unit area above that depth, i.e.

$$\sigma_v = \gamma_{sat} z$$

The pore water pressure at any depth will be hydrostatic since the void space between the solid particles is continuous, therefore at depth z :

$$u = \gamma_w z$$

Hence, the vertical effective stress at depth z will be:

$$\sigma'_v = \sigma_v - u$$

$$\sigma'_v = (\gamma_{sat} - \gamma_w) z = \gamma' z$$

where γ' is the buoyant unit weight of the soil.

When the water table is not at the ground surface, the vertical effective stress is the difference between the total vertical stress and the pore pressure. The total vertical stress is the weight of all the material (solid + water) per unit area above the depth in question. The pore pressure is the equilibrium water pressure corresponding to the depth below the water table. Typically this is the hydrostatic pressure, however, depending on groundwater conditions, the equilibrium water pressure may not be hydrostatic.

The principle of effective stress is very important, since all soil behavior is controlled by the effective stress. The CPT responds to the mechanical characteristics of the soil, which in turn, is controlled by the effective stress. Hence, CPT data are often normalized using the vertical effective stress.

CPT Interpretation

Numerous semi-empirical correlations have been developed to estimate geotechnical parameters from the CPT for a wide range of soils. 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 3 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_0) improves further.

Soil Type	D_r	Ψ	K_0	OCR	S_t	s_u	ϕ'	E,G*	M	G_0^*	k	c_h
Sand	2-3	2-3		5			2-3	2-3	2-3	2-3	3	3-4
Clay			2	1	2	1-2	4	2-4	2-3	2-4	2-3	2-3

Table 3 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,
* improved with SCPT

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	Compressibility
s_u	Undrained shear strength	S_t	Sensitivity
c_h	Coefficient of consolidation	k	Permeability

A summary of methods for the interpretation of geotechnical soil parameters is given in the Appendix. Details for soil profiling and estimating soil permeability are provided in the next sections.

Soil Profiling and Soil Type

The major application of the CPT is for *soil profiling and soil type*. 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. The CPT cannot be expected to provide accurate predictions of soil type based on physical characteristics, such as, grain size distribution but provide a guide to the mechanical characteristics (i.e. strength and stiffness) 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 often referred to as Soil Behavior Type (SBT).

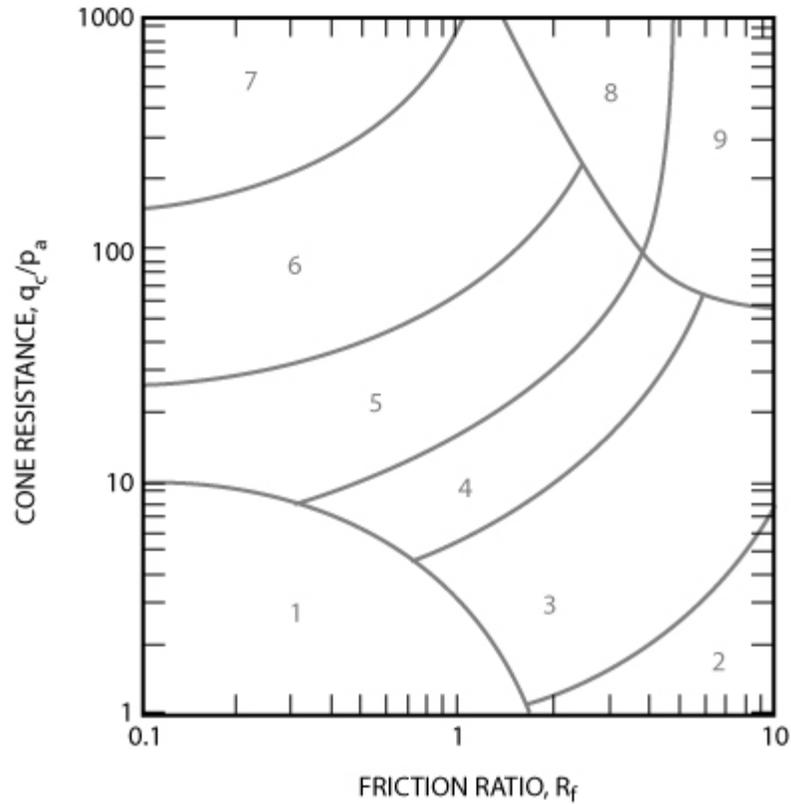
Non-Normalized Charts

The most commonly used CPT soil behavior type method is the chart suggested by Robertson et al. (1986) and an updated, dimensionless version (Robertson, 2010) is shown in Figure 17. 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. 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 (σ'_v), 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 18. 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.

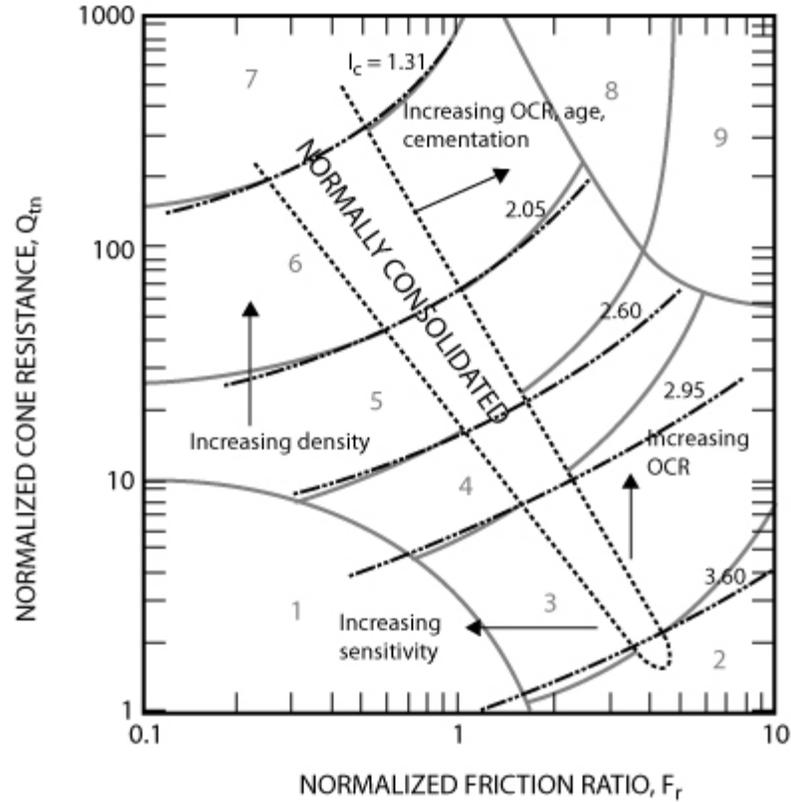


Zone	Soil Behavior Type
1	<i>Sensitive, fine grained</i>
2	<i>Organic soils - clay</i>
3	<i>Clay – silty clay to clay</i>
4	<i>Silt mixtures – clayey silt to silty clay</i>
5	<i>Sand mixtures – silty sand to sandy silt</i>
6	<i>Sands – clean sand to silty sand</i>
7	<i>Gravelly sand to dense sand</i>
8	<i>Very stiff sand to clayey sand*</i>
9	<i>Very stiff fine grained*</i>

** Heavily overconsolidated or cemented*

$P_a = \text{atmospheric pressure} = 100 \text{ kPa} \sim 1 \text{ tsf}$

Figure 17 CPT Soil Behavior Type (SBT) chart (Robertson et al., 1986, updated by Robertson, 2010).



Zone	Soil Behavior Type	I_c
1	Sensitive, fine grained	N/A
2	Organic soils – clay	> 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 18 Normalized CPT Soil Behavior Type (SBT_N) chart, $Q_t - F$ (Robertson, 1990).

Note: Soil Behavior Type Index, I_c is given by:

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

The full normalized SBT_N charts suggested by Robertson (1990) also included an additional chart based on normalized pore pressure parameter, B_q , as shown on Figure 19, where;

$$B_q = \Delta u / q_n$$

and; excess pore pressure, $\Delta u = u_2 - u_0$
 net cone resistance, $q_n = q_t - \sigma_{vo}$

The $Q_t - B_q$ chart can aid in the identification of saturated fine grained soils where the excess CPT penetration pore pressures can be large. In general, the $Q_t - B_q$ chart is not commonly used due to the lack of repeatability of the pore pressure results (e.g. poor saturation or loss of saturation of the filter element, etc.)

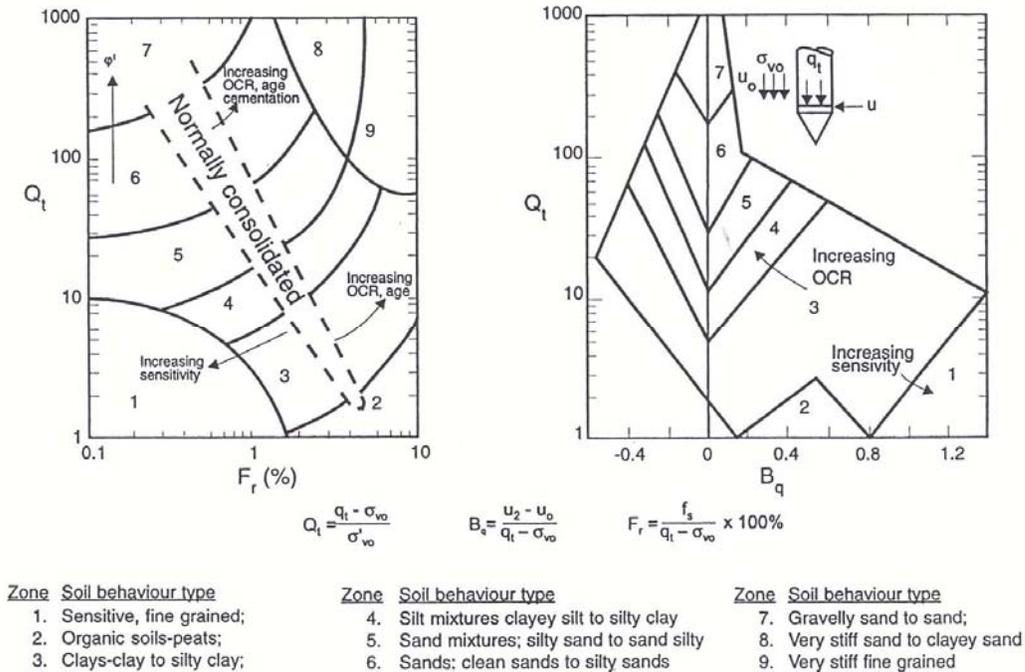


Figure 19 Normalized CPT Soil Behavior Type (SBT_N) charts $Q_t - F_r$ and $Q_t - B_q$ (Robertson, 1990).

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 type can be improved if pore pressure data is also collected, as shown on Figure 19. 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). The rate of pore pressure dissipation during a pause in penetration can also 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 18, 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:

$$\begin{aligned} Q_t &= \text{the normalized cone penetration resistance (dimensionless)} \\ &= (q_t - \sigma_{vo}) / \sigma'_{vo} \\ F_r &= \text{the normalized friction ratio, in \%} \\ &= (f_s / (q_t - \sigma_{vo})) \times 100\% \end{aligned}$$

The term Q_t represents the simple stress normalization with a stress exponent (n) of 1.0, which applies well to clay-like soils. Recently Robertson (2009) suggested that the normalized SBT_N charts shown in Figures 18 and 19 should be used with the normalized cone resistance calculated by using a stress exponent that varies with soil type via I_c (i.e. Q_{tn} , see Glossary).

The boundaries of soil behavior types are then given in terms of the index, I_c , as shown in Figure 18. 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 18 typically has greater than 80% reliability when compared with samples.

In recent years, the SBT charts have been color coded to aid in the visual presentation of SBT on a CPT profile. An example CPT profile is shown in Figure 20, the location of soil and groundwater samples are also shown.

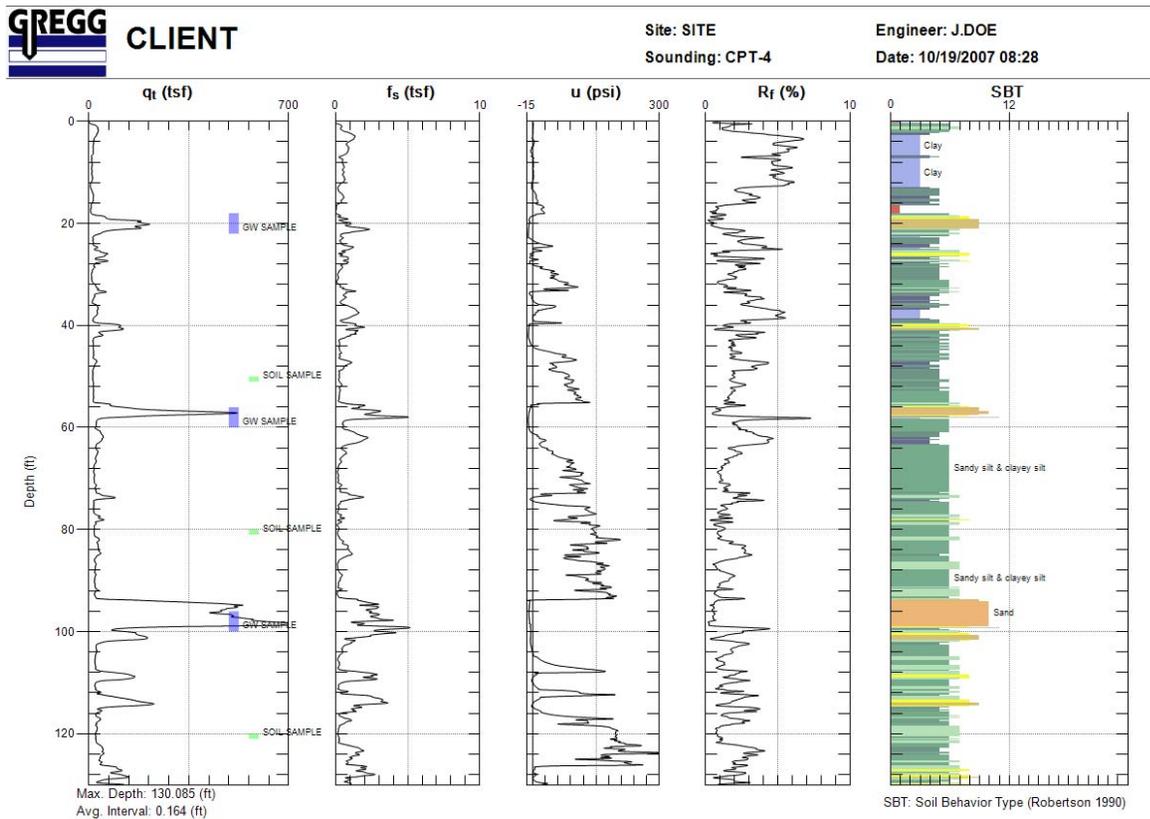


Figure 20 Example CPTU profile with SBT
 (1 tsf ~ 0.1 MPa, 14.7 psi = 100kPa)

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 4 provides estimates based on the CPT-based SBT charts shown in Figures 17 and 18. These estimates are approximate at best, but can provide a guide to variations of possible permeability.

SBT Zone	SBT	Range of k (m/s)	SBT I_c
1	Sensitive fine-grained	3×10^{-10} to 3×10^{-8}	NA
2	Organic soils - clay	1×10^{-10} to 1×10^{-8}	$I_c > 3.60$
3	Clay	1×10^{-10} to 1×10^{-9}	$2.95 < I_c < 3.60$
4	Silt mixture	3×10^{-9} to 1×10^{-7}	$2.60 < I_c < 2.95$
5	Sand mixture	1×10^{-7} to 1×10^{-5}	$2.05 < I_c < 2.60$
6	Sand	1×10^{-5} to 1×10^{-3}	$1.31 < I_c < 2.05$
7	Dense sand to gravelly sand	1×10^{-3} to 1	$I_c < 1.31$
8	*Very dense/ stiff soil	1×10^{-8} to 1×10^{-3}	NA
9	*Very stiff fine-grained soil	1×10^{-9} to 1×10^{-7}	NA

*Overconsolidated and/or cemented

Table 5 Estimated soil permeability (k) based on the CPT SBT chart by Robertson (2010) and shown in Figures 17 and 18

The average relationship between soil permeability (k) and SBT I_c , shown in Table 5, can be represented by:

$$\text{When } 1.0 < I_c \leq 3.27 \quad k = 10^{(0.952 - 3.04 I_c)} \quad \text{m/s}$$

$$\text{When } 3.27 < I_c < 4.0 \quad k = 10^{(-4.52 - 1.37 I_c)} \quad \text{m/s}$$

The above relationships can be used to provide an approximate estimate of soil permeability (k) and to show the likely variation of soil permeability with depth from a CPT sounding. Since the normalized CPT parameters (Q_m and F_r) respond to the mechanical behavior of the soil and depend on many soil variables, the suggested relationship between k and I_c is approximate and should only be used as a guide.

The rate of dissipation of pore pressure during a CPT dissipation test can also be used to estimate the horizontal coefficient of permeability (k_h), typically using the time to reach 50% dissipation (t_{50}). The method to determine the value of t_{50} from a dissipation test is illustrated in Figure 21.

Robertson et al. (1992) presented a summary of available data to estimate the horizontal coefficient of permeability (k_h) from dissipation tests. Since the relationship is also a function of the soil stiffness, Robertson (2010) updated the relationship as shown in Figure 22.

Jamiolkowski et al. (1985) suggested a range of possible values of k_h/k_v for soft clays as shown in Table 6.

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
Varved clays and other deposits containing embedded and more or less continuous permeable layers	3 to 15

Table 6 Range of possible field values of k_h/k_v for soft clays
(Jamiolkowski et al., 1985)

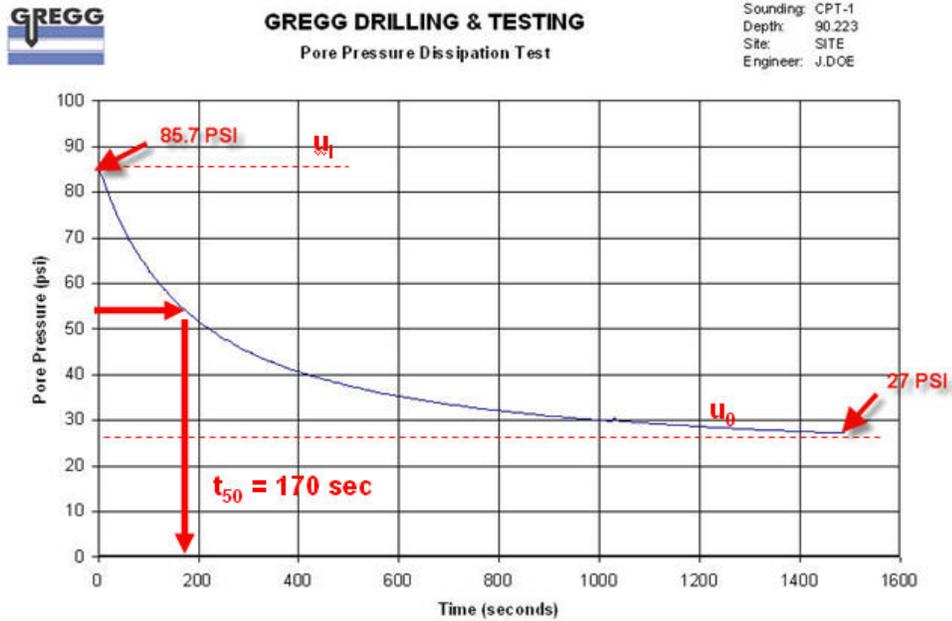


Figure 21 Example dissipation test to determine t_{50}

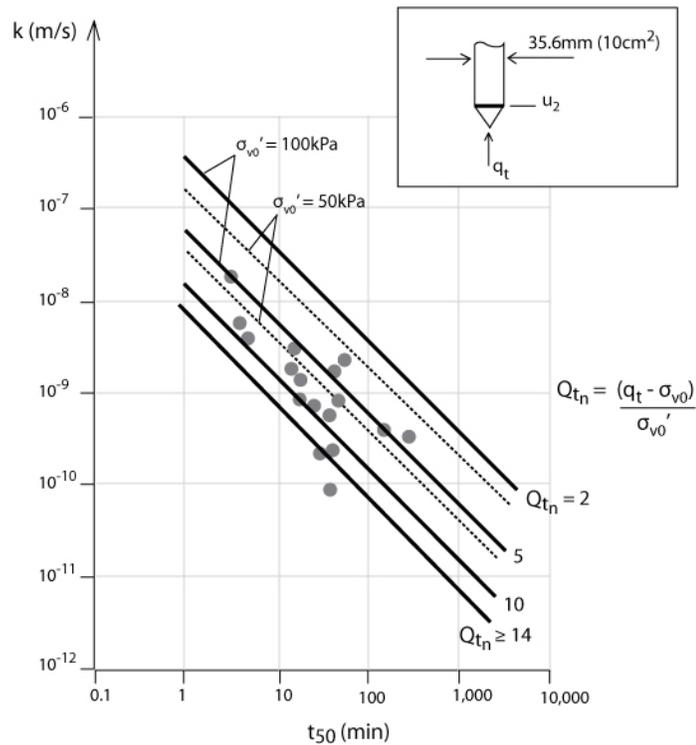


Figure 22 Relationship between CPTu t_{50} (in minutes) and soil permeability (k_h) and normalized cone resistance, Q_{tn} . (After Robertson 2010).

(Note: increase values of k_h by 1.5 when using a 15 cm² cone)

Sensors to Screen for Contaminants

The CPT is an excellent logging device that provides near continuous profiles of mechanical soil parameters. Screening sensors can be added to the CPT to provide valuable qualitative data concerning the location and magnitude of contamination in the ground in addition to the data provided by the CPT. Field screening data can be used to;

- Determine direct sampling locations
- Locate permanent and /or temporary well installations
- Enhance design of remediation programs.

Significant cost saving is possible by combining the CPT with various sensors to detect contaminants.

Temperature

One of the earliest sensors added to a cone was a temperature sensor. Initially temperature sensors were used to aid in either calibration corrections or to locate zones of different ground temperature, such as frozen soil. Recently temperature sensors have been used to aid in the identification of contaminants that generate heat due to biological and/or chemical activity.

Electrical Conductivity/Resistivity

One method of detecting changes in groundwater and soil chemistry is by measuring changes in the electrical bulk resistivity or conductivity. The conductivity is the inverse of resistivity, with the following as a useful guide;

$$\text{Conductivity } (\mu\text{S/cm}) = 10,000 \div \text{Resistivity } (\Omega\text{-m})$$

The rationale for making electrical measurements is that some contaminants change the electrical properties of the soil. Therefore, by measuring soil resistivity, the lateral and vertical extent of soil contamination can be evaluated when the contaminate influences the electrical properties of the ground and/or groundwater. Unsaturated soils

and saturated soils with many non-aqueous-phase-liquid (NAPL) compounds exhibit high electrical resistivity (low conductivity). Dissolved inorganic compounds, such as those contained in brines and landfill leachates, can significantly decrease soil resistivity.

The resistivity CPT (RCPT) works on the principle that the measured voltage drop across two electrodes in the soil, at a given excitation current, is proportional to the electrical resistivity of the soil. The resistivity electrodes are typically steel rings from 5 mm to 15 mm wide that are set apart by distances that vary from 10 mm to 150 mm. The larger the spacing, the greater the depth of penetration for the electrical field into the surrounding soil. Some probes have several electrode spacings so that lateral penetration varies. Some devices use small circular electrodes mounted around the circumference of the probe. The electrodes are designed to be wear resistant and have high electrical conductivity. A non-conducting plastic or other material is used as the insulator separating the electrodes. Figure 23 shows a range of resistivity cones.



Figure 23 Example of some resistivity cones (courtesy P.Mayne)

Electrical resistance is not a material property but a function of the electrode spacing and size. To convert from resistance to resistivity, which is a material property, a laboratory calibration of the probe geometry is necessary. The resistivity of soil is for the most part influenced by the resistivity of the pore liquid, which in turn is a measure of the pore fluid chemical composition (e.g. hydrocarbons are typically non-conductive and will therefore exhibit high resistivity).

Electrical resistivity/conductivity is also strongly influenced by clay content. Low resistivity (high conductivity) is typically linked with high clay content, all other factors constant.

The main disadvantage with electrical measurements is that the bulk resistivity or conductivity is not directly controlled by the chemical properties of the surrounding material. The measurements are strongly influenced by the background soil and pore liquid. Hence, it is important to obtain measurements of the background uncontaminated ground for comparison. For relatively uniform soil conditions it is possible to develop local site specific correlations between the bulk resistivity and selected contaminants. Electrical resistivity can also be very effective in locating the groundwater table, since there is a large change in bulk resistivity between dry and saturated soil.

The primary advantage of the resistivity CPT is that it provides continuous profiles of bulk resistivity along with the full CPT data in a rapid cost effective manner. Profiles of resistivity measurements can then be used to identify potentially critical zones where detailed sampling and/or monitoring can be carried out.

An example resistivity CPT profile is shown in Figure 24 at a site where there was a variation in water quality from fresh to salt water, the salt water has a lower resistivity than fresh water.

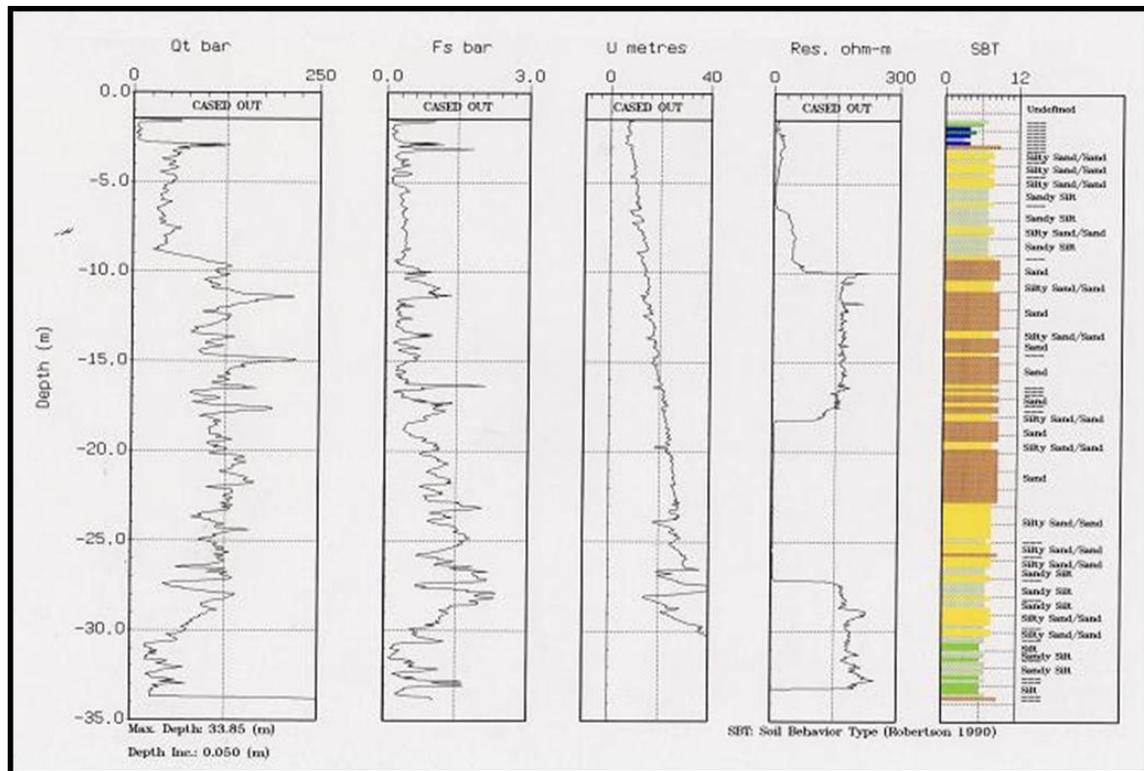


Figure 24 Example electrical resistivity CPT (RCPT) used to identify salt water zone

Dielectric Measurements

The electrical measurements discussed above relate to resistivity. However, electrical measurements can also be made to measure the dielectric constant of the material surrounding a penetrometer. The resistivity can be somewhat insensitive to contaminant type whereas, the dielectric constant can be sensitive to contaminant type. The dielectric constant is frequency dependent. However, above about 50 MHz the dielectric constant is essentially constant. Contaminants such as light and dense NAPL zones can be detected by low values of dielectric constant and conductivity in relation to the background surrounding water saturated soil.

pH

The acidity of a material can be measured using pH sensors mounted either inside a cone or on the surface of the probe. The disadvantage of sensors mounted inside the cone is that a sample of pore liquid must be drawn into the measuring cell and then expelled. This process can be difficult in low permeability soils such as clay and the cell and sensor must be cleaned after each reading. More details about sampling techniques will be given in a later section. Sensors mounted on the outside of the cone have the advantage of direct exposure to the surrounding material, however, abrasion and damage to the sensor can be a major problem. A pH sensor can be located in a small recess a short distance behind the friction sleeve. This recess is designed to produce a small vortex for pore liquid to enter during cone penetration. The sensor is then well protected from abrasion and damage from the surrounding solid material and can measure continuous variations of the pH of the pore liquid. The pH measurements are sensitive to temperature changes and generally a temperature sensor is mounted adjacent to the pH sensor to allow for automatic correction for temperature effects. The continuous measurement of pH can be a useful guide for detecting certain contaminants with significantly different pH values from that of the background soil. A pH sensor can be used to identify either acidic or basic materials.

Oxygen Exchange Capacity

The oxygen exchange capacity of a material can be measured with a sensor for redox (reduction oxidation) potential. CPT probes have been designed to measure redox potential, pH and conductivity. These three parameters are major variables of chemical equilibrium for inorganic substances. The sensors are typically mounted inside the cone and a sample is drawn into a measuring cell located a short distance behind the cone. The measurements are made under a nitrogen atmosphere to reduce the exchange of atmospheric oxygen with dissolved gases from ground water. A slight excess pressure of nitrogen is used during penetration of the probe to stop the flow of liquid into a small (approximately 15ml) measuring cell. The nitrogen is supplied from a gas cylinder at the surface. A stainless steel porous filter is cleaned by de-mineralized water, which is also used to clean the sensors and check the calibration. The

water is pushed out by an excess nitrogen pressure. A pressure sensor is also included to monitor the flow of liquid into the measuring cell and to estimate the hydraulic conductivity of the surrounding soil. Olie et al. (1992) showed that the measurement of redox potential, pH and electrical conductivity enabled the monitoring of in-situ sanitation, designed to dissolve a floating layer of versatic acid with infiltration of hydroxide solution beneath a storage tank at a petrochemical plant in the Netherlands. Redox potential can also be used to monitor conditions during bio-remediation. Further details of devices that sample the surrounding pore liquid or vapor are given in a later section.

Radio-isotope

Gamma and neutron sources and sensors have been added to cone penetrometers in the past in an effort to measure in-situ density and moisture content. However, these devices have not become popular because they contained active radiation sources which can present significant problems if the probes should become lost in the ground and require expensive recovery. Passive gamma-ray sensors have been added to the CPT in an effort to detect radioactive contaminants. A variety of different passive sensors are available, the selection of the appropriate sensor is based on efficiency, range of gamma-ray energies expected, temperature dependence of the sensor and sensor ruggedness. The application of gamma-ray sensors is clearly limited to environments where specific gamma-ray emitting contaminants are present.

Fluorescence

Fluorescence is a property of many aromatic hydrocarbons where absorbed light stimulates the release of photons (light) of a longer wavelength. Fluorescence can be used to detect small amounts of poly-aromatic hydrocarbons (PAH) in soil.

The first CPT laser induced fluorescence (LIF) system was developed by the U.S. Military and was called a Site Characterization and Analysis Penetrometer System (SCAPS). This system was later licensed to the private sector, modified and called a Rapid Optical Screening Tool (ROST). Updated versions of fluorescence devices have been developed that can be added to standard CPT equipment (e.g. UVOST). All CPT

fluorescence devices operate in the ultra violet range of wavelengths. Some use a surface laser for the excitation light and deliver the light through a fibre optic cable (ROST and UVOST), whereas others use a small downhole ultra violet light source (UVIF).

The fluorescence of PAHs has both a spectral (wave length) and temporal (time) component. Environmental samples typically contain several different PAHs along with other fluorophores, and the PAH fluorescence spectra overlap to form broad featureless spectral and temporal emission (compared to pure PAH spectra). If the temporal decay waveforms (intensity versus time) are recorded across the entire spectrum the resulting wavelength-time-intensity matrix can describe the fluorescence emission completely.

Typically, the ultra violet light exits through a small (sapphire) window in the side of the probe. As the probe is advanced, the soil is exposed to the excitation light. If fluorescent compounds exist (i.e. PAH's), fluorescence light is emitted and the intensity is measured. Some devices measure the fluorescence by transmitting the 'signal' light via a fiber back up the probe to a data acquisition system at the surface (e.g. ROST, UVOST). Others measure the intensity via a small photomultiplier tube in the probe and avoid the use of fiber optic cables (e.g. UVIF).

Some systems measure only the intensity of fluorescence at one wavelength of excitation and another for emission (e.g. UVIF), whereas others measure the intensity of fluorescence over a range of emission wavelengths (e.g. UVOST). Results are typically displayed as some form of intensity versus depth profile. With a system that has the capability to measure fluorescence emission over a range of wavelengths, a waveform can also be displayed at selected depths that can assist in identifying the contaminant present at that depth. A schematic of the LIF screening concept is shown in Figure 25.

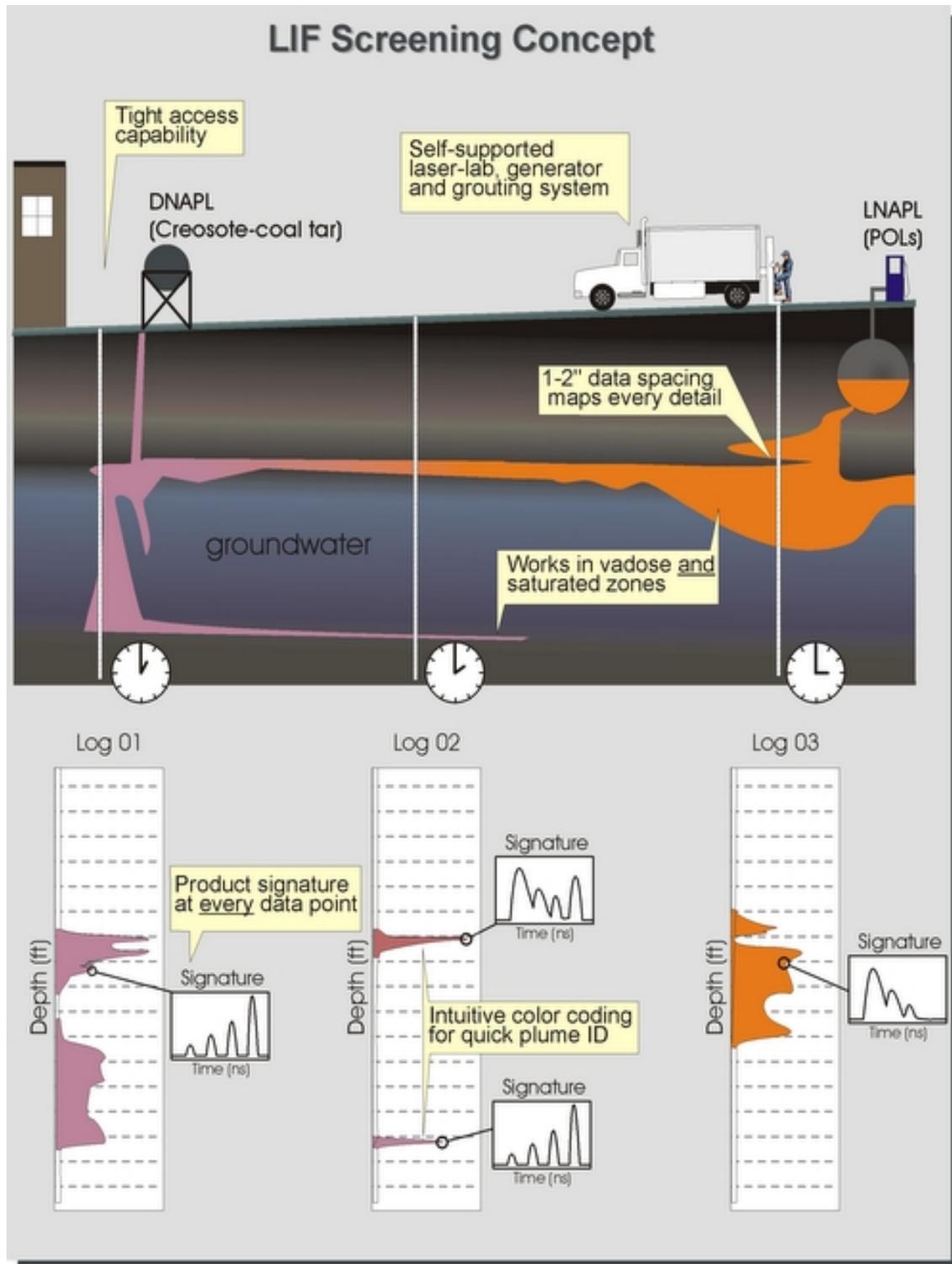


Figure 25 Schematic of LIF screening concept
(courtesy of Dakota Technologies Inc)

Common fuel contaminants, such as heating oil, jet fuels, gasoline and diesel fuel can exhibit strong fluorescence signatures with the degree of fluorescence depending on the excitation wavelength. However, common chemical contaminants such as chlorinated hydrocarbons (e.g. TCE, PCE) do not fluoresce and are generally not suitable for the fluorescence technique.

The intensity of fluorescence is strongly dependent on soil type, with sands having a stronger signal for a given concentration than clayey soils. The calibration and detection limits of fluorescence in soils are complicated by soil type, soil grain size effects, natural organic compounds and the influence of time on the contaminant degradation. Concerns also exist over long term durability and maintenance of the fiber optic cable when used to transmit the signals between the probe and data acquisition system.

Research (Van Ree and Olie, 1993) has shown that the soil effectively cleans the window on the surface of the cone and that the displacement (smearing) of the detected layer does not exceed 5 cm (2 inches). An example UVIF CPT profile is shown in Figure 26 and a schematic of a UVIF CPT system is shown in Figure 27.

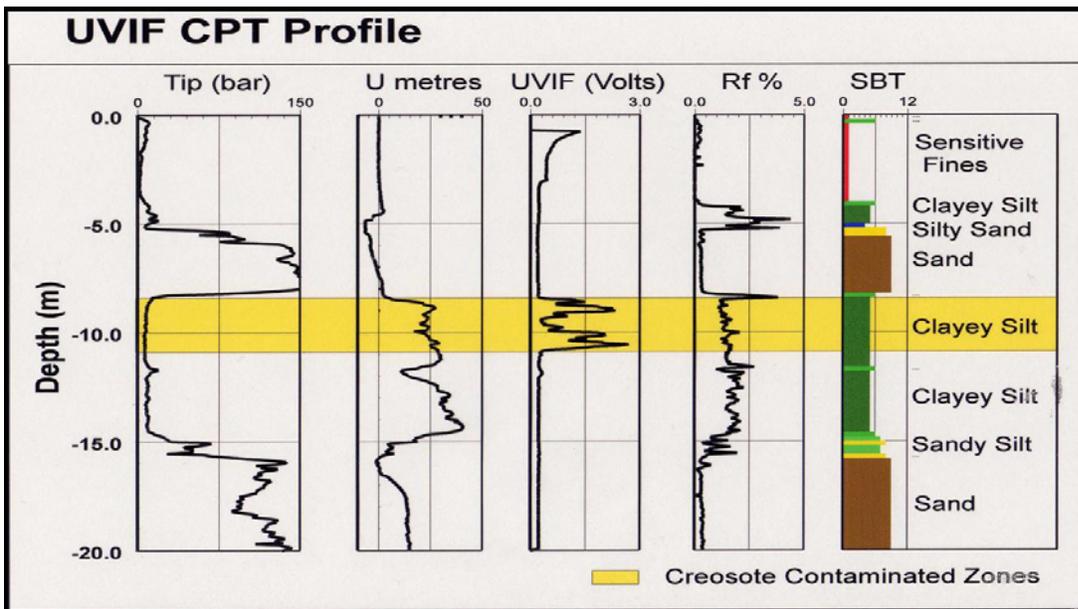


Figure 26 Example UVIF CPT profile at a creosote contaminated site

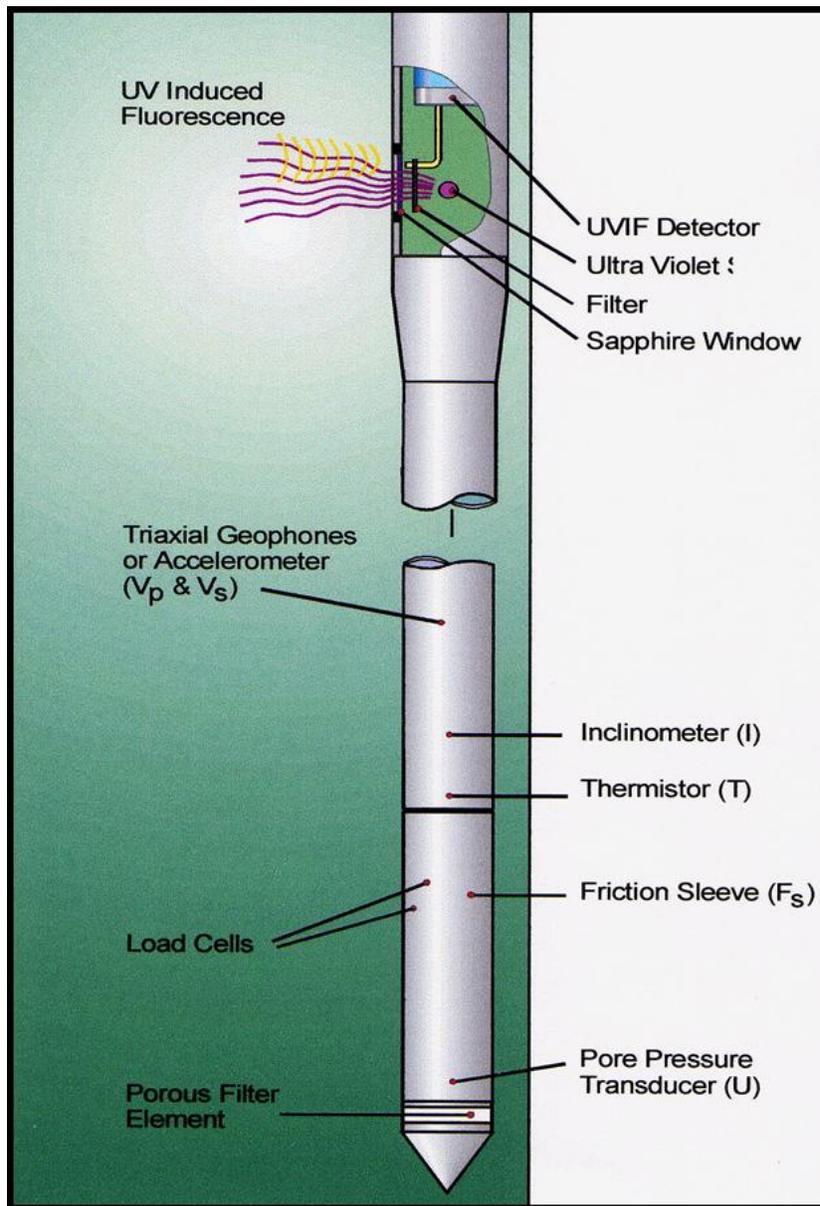


Figure 27 Schematic of CPT UVIF system

Membrane Interface Probe (MIP)

A relatively recent advance in the detection of volatile organic compounds (VOCs) in soil is the membrane interface probe (MIP). The MIP can be pushed into soil using standard direct push equipment or attached behind a CPT. A schematic of the MIP probe is shown in Figure 28. The probe has a small diameter (6.35mm) semi-permeable membrane comprised of a thin film polymer impregnated onto a stainless steel screen for support. The membrane is placed in a heater block attached to the probe. The heater block is thicker along the leading edge to protect the membrane during penetration. The block is heated to about 100 to 120 degrees C to accelerate diffusion of VOCs through the membrane. Diffusion occurs because of a concentration gradient between the contaminated soil and the clean carrier gas (nitrogen) behind the membrane. A constant flow rate of about 35 to 45 mL/min sweeps behind the membrane and carries contaminants to the gas phase detectors in the data acquisition system at the surface. Travel time from the membrane to the surface detectors depends on the depth of probe and flow rate but is typically around 30 to 40 seconds. The probe is pushed into the ground at a rate of about 1 foot (30cm) per minute and collects about 20 data points every 1 foot (30cm). After each 1 foot (30cm), penetration is paused 30 to 60 seconds to re-heat the membrane, since heat is lost during penetration.

A range of gas detectors can be used with the most common being photo-ionization detector (PID), electron capture detector (ECD) and a flame ionization detector (FID). Each detector is designed for sensitivity to a group or type of contaminate. The ECD is used for chlorinated (TCE, PCE) contaminate detection, PID is best for aromatic hydrocarbons (BTEX compounds) and the FID is best for straight chained hydrocarbons (methane, butane). These detectors may be used in series with the least destructive first. The detectors must be low dead volume gas chromatography detectors and must be heated to avoid condensation of any water vapor that crosses the membrane. These detectors provide a qualitative evaluation of contaminants. The MIP can also be used with a field portable Gas Chromatograph Mass Spectrometer (GC-MS) to obtain a more quantitative analysis. Typically the detector signals are recorded with time and the signals presented versus depth.

When the MIP is pushed using standard (Geoprobe-type) direct push equipment, it is often combined with electrical conductivity measurements

to provide a qualitative profile of soil stratigraphy. When the MIP is combined with the CPT it provides more superior profile of soil stratigraphy along with the MIP data.

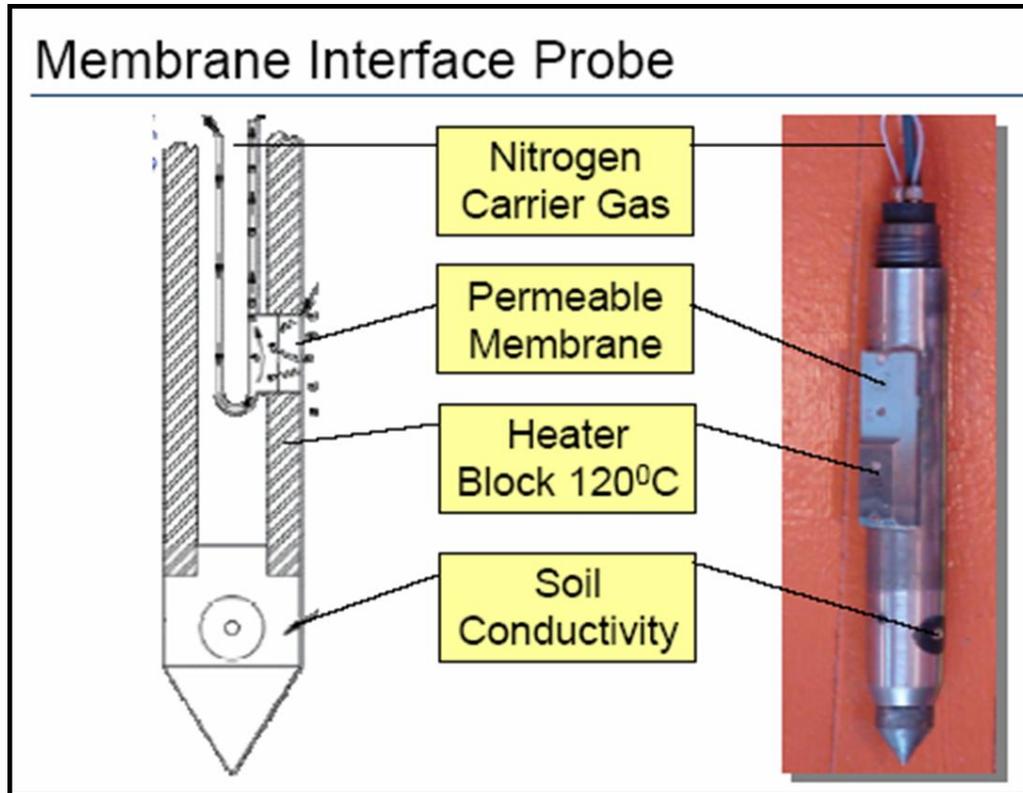


Figure 28 Schematic of MIP system (courtesy Geoprobe Inc.)

Hydraulic Profiling Test (HPT)

As previously discussed, the CPT includes the measurement of pore water pressure during penetration and during pauses in penetration (dissipation test). However, the accuracy and usefulness of the pore water pressure measurement is dependent on the soil and saturation of the pore pressure sensor. Below the water table, where the soil is saturated, the CPT pore pressure measurements provide useful information regarding the behavior of the soil as well as equilibrium water pressure. CPT pore pressure measurements above the water table have limited usefulness because the soil is typically unsaturated.

Recently, the concept of injecting water out of a small porous element on the side of a probe has resulted in a Hydraulic Profiling Test (HPT). Water is injected at a slow rate of about 250mL/min and the flow pressure recorded with depth. The flow pressure is related to the capacity of the soil around the cone to accept the flow of water, i.e. related to the hydraulic conductivity of the soil adjacent to the probe. In fine-grained soils the flow pressure is large, whereas in coarse-grained soils the flow pressure is low. The advantage of the HPT is that the resulting flow pressure profile can be obtained both above and below the water table, resulting in a continuous profile that can provide an indication of hydraulic conductivity with depth.

The HPT is often combined with electrical conductivity (EC) measurements to provide a continuous profile of both EC and flow pressure. Since EC can be influenced by clay content, the combined profiles can provide an indication of grain-size and hydraulic conductivity above and below the water table. Figure 29 shows an example of a HPT flow pressure profile with EC.

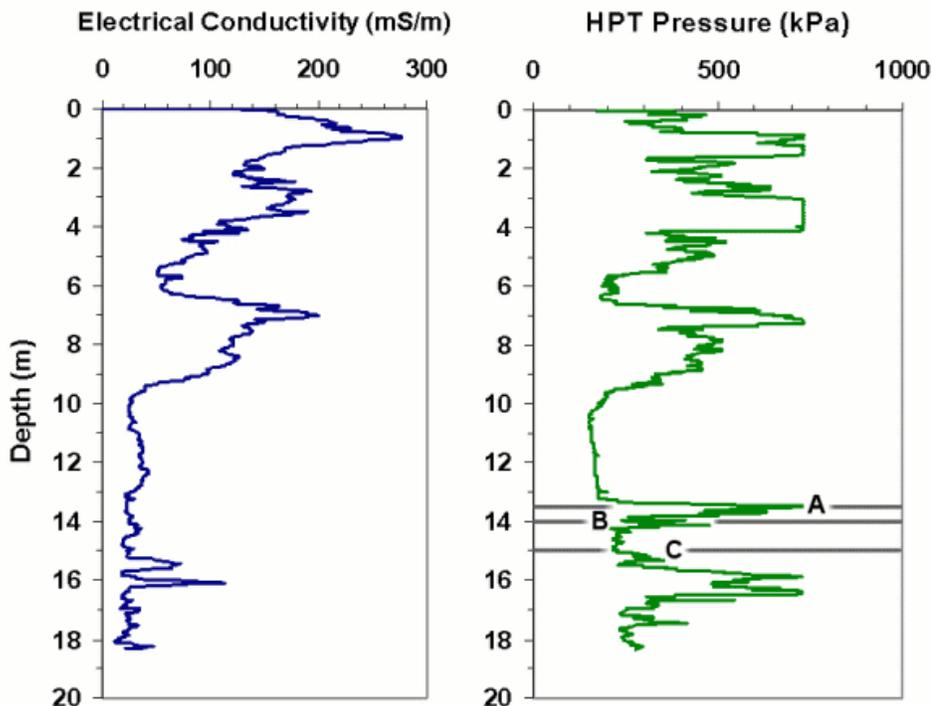


Figure 29 Example HPT profile (courtesy Geoprobe Inc.)

Vision Cone

Several downhole video imaging systems have been developed for CPT probes. These systems use miniature video cameras with magnification and focusing systems integrated into the probe to obtain images of soil. Light-emitting diodes provide illumination. The signal from the camera is sent to the surface where it can be viewed in real-time on a video monitor, recorded or digitized for further analysis. With 100x magnification, objects as small as 20 micrometers can be resolved on a standard monitor. Algorithms have been developed to classify soils electronically from the video image. Figures 30 and 31 show vision cone equipment and an image of coal tar contamination, respectively.

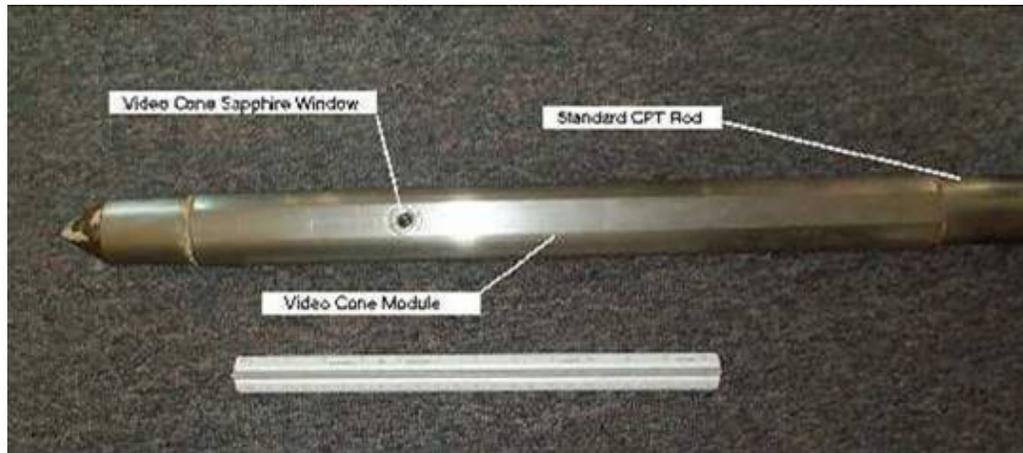


Figure 30 Example Vision Cone (courtesy P.Mayne.)

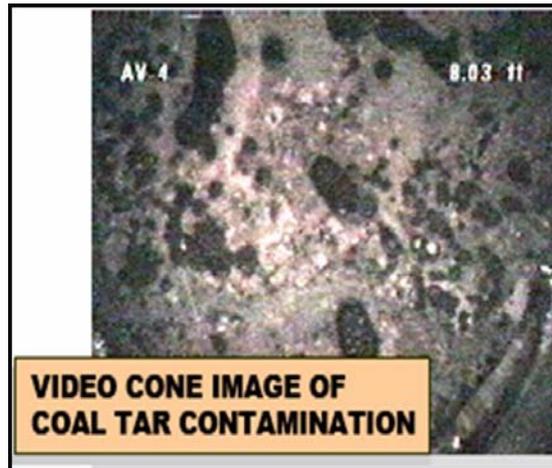


Figure 31 Example image from vision cone of coal tar contamination

Sampling Devices

The cone penetrometers described in the previous section are primarily screening devices that log the stratigraphic profile for geotechnical and chemical measurements. Based on these measurements it is often possible to identify potentially critical zones or regions that may require more selective testing to measure or monitor specific contaminants. Typically, the results from a CPT screening investigation are used to develop a specific sampling and/or monitoring program. A wide range of samplers have been developed for use with direct push techniques for the sampling of vapors, liquids and solids.

Samplers are designed for either one-time samples or as monitoring wells. One-time samplers are pushed to a desired depth and a sample obtained, then the probe is either retracted or pushed to a greater depth for further sampling before removal. Monitoring well samplers are installed and samples retrieved over a period of time before the well is ultimately removed. Small diameter monitoring wells are described in a later section.

Liquid (water) Samplers

The most common direct-push, discrete depth, in-situ water sampler is the Hydropunch. To a lesser extent, the BAT Enviroprobe, Simulprobe and Waterloo Profiler are also used.

The Hydropunch and its variations is a simple sampling tool that is pushed to the desired depth and the push rods withdrawn to expose the filter screen. Figure 32 shows a schematic of a typical Hydropunch system. The filter screen can be made from a range of materials (PVC, stainless steel) although PVC is the most common. Screened intervals can vary in length from 4 inches to 5 feet (100mm to 1500mm) depending on ground conditions, required sample depth, contaminant type and hydraulic conductivity of surrounding material. A small diameter sampler (bailer) is lowered through the hollow push rods and body of the sampler to collect a liquid sample. A peristaltic pump can also be used to pump larger volumes of non-volatile liquids to the surface. This type of push-in liquid sampler is very common, simple to use and can produce large sample volumes. However, samples are considered less discrete in depth than

other methods, due to the longer Hydropunch screen intervals typically used. The hydropunch-type method can also be modified to install more permanent monitoring well installations. Prepacked well screens and grout seals can be used with hydropunch-type wells for construction of monitoring wells.

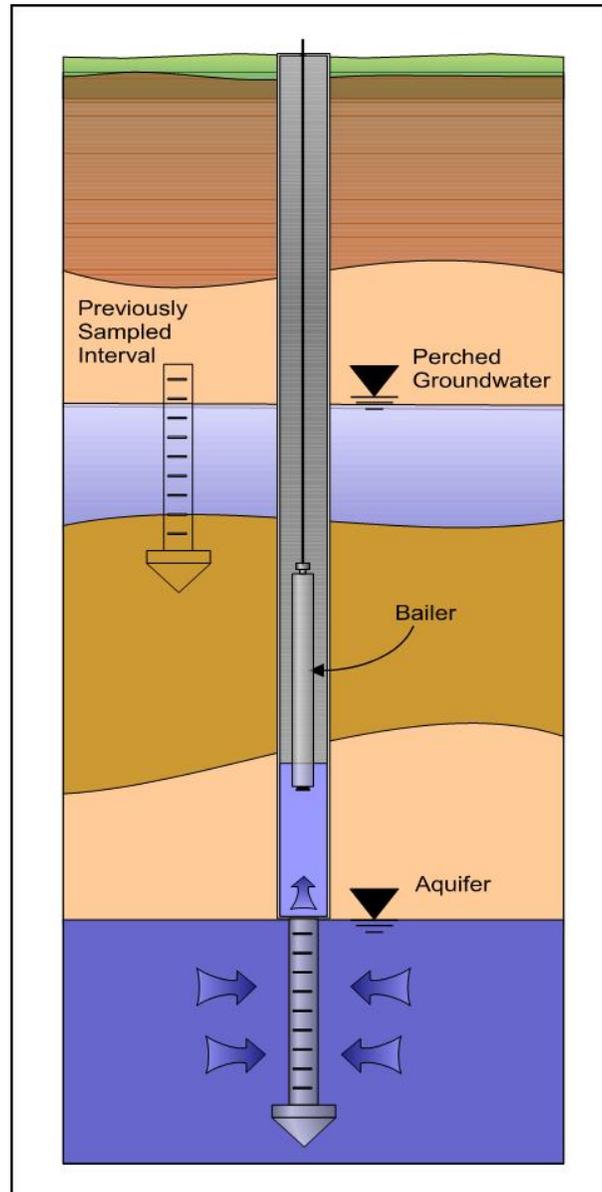


Figure 32 Schematic of a Hydropunch system

The BAT Enviroprobe consists of three basic components, as shown in Figure 33:

1. A sealed filter tip with retractable sleeve attached to the push rods.
2. Evacuated and sterilized glass sample vials, enclosed in a housing and lowered to the filter tip via a wireline system.
3. A disposable, double-ended hypodermic needle which makes a hydraulic connection with the pore fluid by puncturing the self-sealing flexible septum in the filter tip.

The filling rate of the sample vials can be monitored using a pore pressure transducer attached to the vial. The monitoring shows when the fluid infiltration is complete, assuring that pressure inside the vial is equal to the in-situ fluid pressure. These measurements can also be used to estimate the hydraulic conductivity of the surrounding material.

Various modified versions of this concept are now in use for ground water sampling. The system is most applicable for retrieving water samples where limited volumes are sufficient (< 150 mL). The small filter size allows for discrete sampling intervals, but can create longer sampling times in less permeable materials. The protective sleeve and fine filter screen produces water samples with low turbidity, however, sometimes the needle can become blocked with fine material passing through the filter resulting in incomplete filling of the vial.

Samplers have also been developed that allow simultaneous sampling of either soil/water or soil/vapor (e.g. *SimulProbe*). Either before or after a soil sample is obtained by pushing the sampler into the soil, the probe is retracted about 2 inches (50mm) to expose a screened port for either water or vapor sampling. A nitrogen or helium pressurized canister is used and the back-pressure released to allow water/vapor sampling. The fluid sample rises hydrostatically through a longitudinal pathway around the soil core chamber. The pressurized canister maintains any VOC's in solution. Figure 34 shows a soil-water *SimulProbe*.

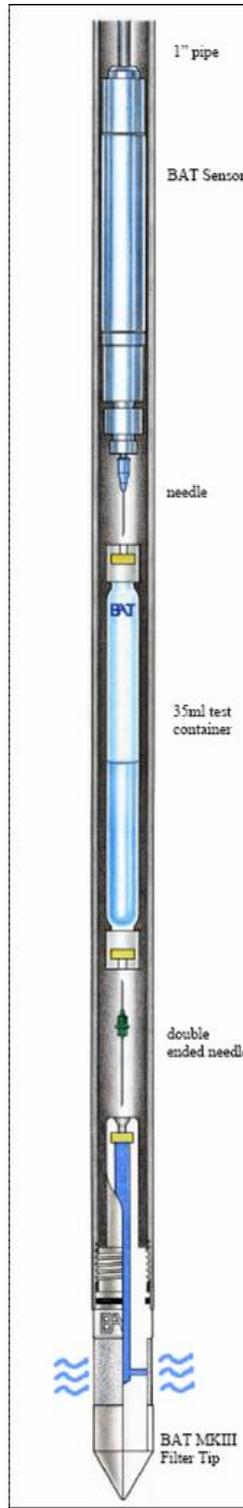


Figure 33 Schematic of BAT sampler with pressure transducer to monitor sampling

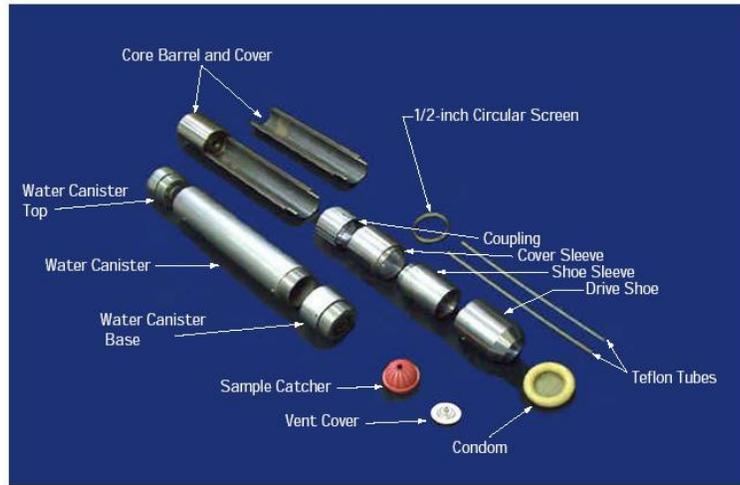


Figure 34 Soil-water *SimulProbe*

(courtesy of BESST Inc.)

The Waterloo Profiler allows multiple groundwater samples in a single push. Deionized water is injected out the sampling ports during penetration to prevent clogging and can be monitored to estimate hydraulic conductivity. Liquid samples are taken at discreet depths using a peristaltic pump. The use of the peristaltic pump limits the total depth of sampling. Figure 35 shows a photograph of a Waterloo Profiler probe with several sampling ports.



Figure 35 Waterloo profiler probe showing several sampling ports

Vapor Samplers

Vapor (gas) samples can be obtained in a manner similar to that described above for liquid samples. The most common direct-push, discrete depth, in-situ gas sampler is the hydropunch-type sampler. The sampler is pushed to the required depth, the filter element exposed and a vacuum applied to draw a vapor sample to the surface. The volume of gas can be monitored using special monitoring equipment and typical sample containers are gas tight syringes and glass or steel sampling vessels. Special disposable plastic tubing is used to draw the sample to the surface.

Vapor sampling modules have also been added to cone penetrometers to allow sampling during a CPT. The sampling module is typically located a short distance behind the cone. Samples can be taken during short pauses in the penetration. Minimizing cross-contamination for subsequent samples within one vertical sounding requires either a two line sampling design for purging of the vapor collection lines between samples or a single line system with nitrogen or argon gas purging. A small positive internal gas pressure can stop the inflow of gases during the penetration process. Figure 35 illustrates typical post-run (hydropunch-type) and in-line vapor sampling devices. Special care is required to purge the sample tubes and store the samples.

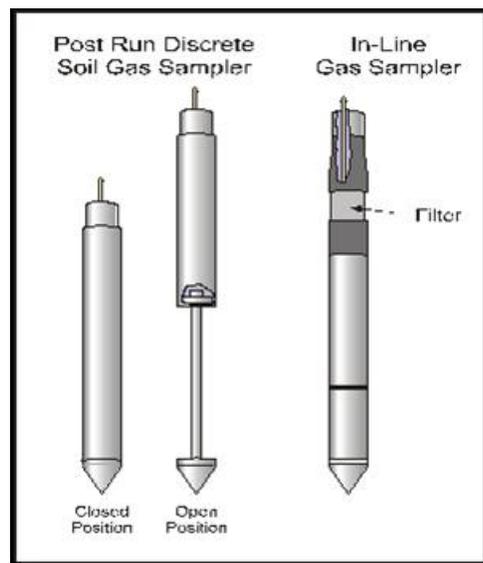


Figure 35 Typical (one-time) Vapor sampling devices

Solid Samplers

A variety of push-in discrete depth soil (solid) samplers are available. Most are based on designs similar to the original Gouda or MOSTAP soil samplers from the Netherlands. The samplers are pushed to the required depth in a closed position. The Gouda type samplers have an inner cone tip that is retracted to the locked position leaving a hollow sampler with small diameter (25mm/1 inch) stainless steel or brass sample tubes. The hollow sampler is then pushed to collect a sample. The filled sampler and push rods are then retrieved to the ground surface. The MOSTAP type samplers contain a wire to fix the position of the inner cone tip before pushing to obtain a sample. Modifications have also been made to include a wireline system so that soil samples can be retrieved at multiple depths rather than retrieving and re-deploying the sampler and rods at each interval. The wireline systems tend to work better in soft soils. Figure 36 shows a schematic of typical (Gouda-type) CPT-based soil sampler.

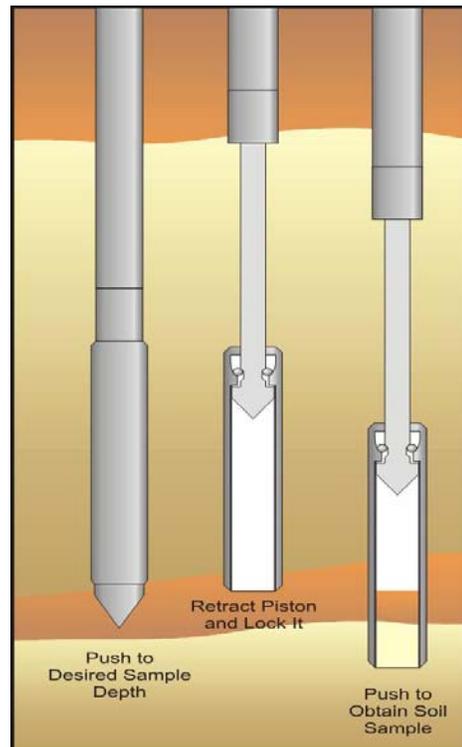


Figure 36 Schematic of CPT-based soil sampler

Continuous soil samplers are also available that can be installed using either CPT equipment or other direct push equipment (e.g. Geoprobe). Single and dual tube continuous soil samplers are available. The dual tube configuration is more rapid because the outer tube remains in place while the inner tube is removed to extract the sample. Sample size is typically 2 inch (50mm) diameter and up to 5ft (1.5m) in length. Figure 37 shows a schematic of a single-tube continuous soil sampler.

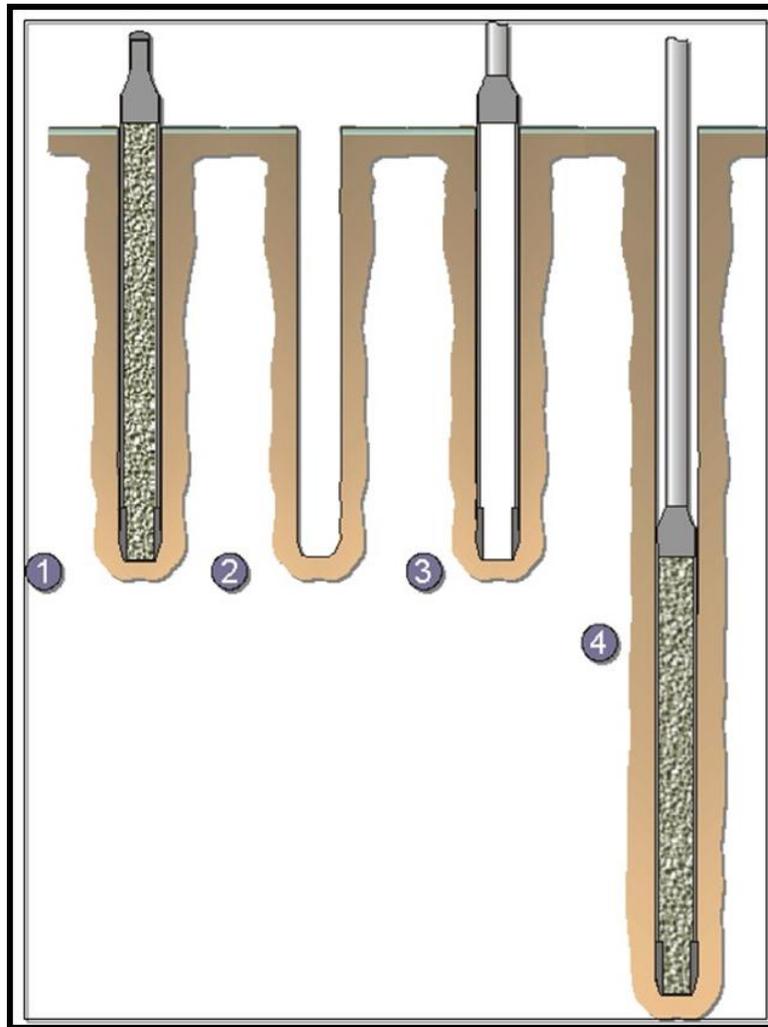


Figure 37 Schematic of single-tube continuous soil sampler
(courtesy of Geoprobe Inc.)

Small Diameter Wells & Piezometers

CPT and direct-push equipment can be an effective means of installing small diameter wells, stand-pipe piezometers and electric piezometers. These techniques generate no soil cuttings and the continuous nature of the CPT data, combined with associated screening sensors, provide valuable information to determine screen intervals and depths for small diameter wells. Wells are typically installed in small 2 inch (50 mm) diameter holes with CPT or other direct-push equipment. The small annular space necessitates the use of pre-packed screens and seals, as illustrated in Figure 38.

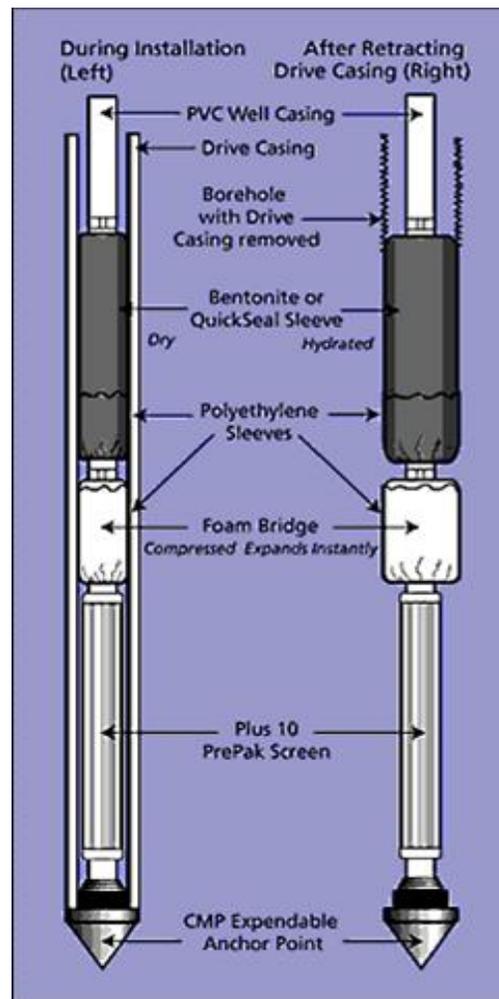


Figure 38 Schematic of direct-push small diameter temporary well with pre-packed screen and seal

Similar soil gas sampling wells can be installed. Soil gas wells typically have double woven stainless steel wire screens that can be inserted down the push-rods and anchored at a desired depth. The screen and sampling tube can be installed and rotated to attach to the anchor point used during driving and then the push-rods removed. Screens are available in a variety of lengths.

Continuous Multichannel Tubing (CMT) can provide groundwater sampling and measurement of hydraulic head at multiple depths within a single installation. High density polyethylene (HDPE) tubing can be installed to multiple depths using direct push techniques in soft ground or through boreholes in stiff ground. The tubing has an outside diameter of about 1.7 inches (43 mm) and can come in either 3 or 7 port configurations. Pre-packed screens and seals make installation possible using CPT direct-push techniques. Figure 39 shows a schematic of a CMT well.

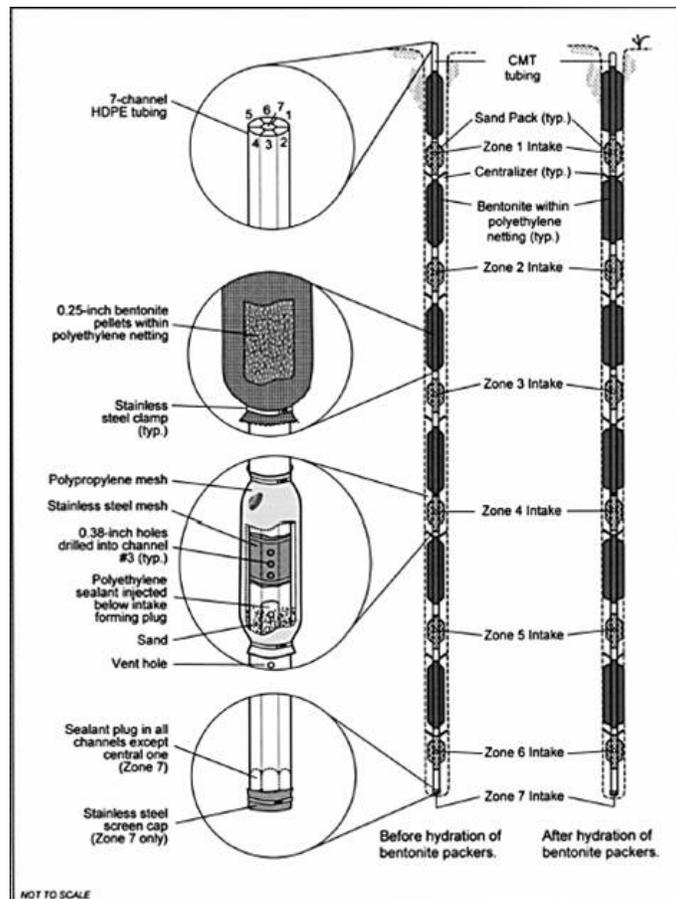


Figure 39 Schematic of a CMT well (courtesy of Solinst Inc.)

FLUTE Liners

Flexible Liner Underground Technologies Inc. (FLUTE) produce a Ribbon NAPL System (commonly called FLUTE liners) for detecting NAPL contaminants. A hydrophobic liner is impregnated with a powdered oil dye which dissolves on contact with a NAPL and leaves a stain on the liner. The liner is placed within the push rods and inflated against the soil using water as the push rods are removed. After a wait of about 1 hour to allow reaction between any NAPL contaminants and the liner, the liner is then retrieved (peeled inside out) from the hole using an internal tubing attached to the bottom of the liner. FLUTE liners provide a qualitative evaluation for NAPL contamination and are usually used in combination with CPT and other sensors. Figure 40 shows a schematic of a FLUTE liner installation.

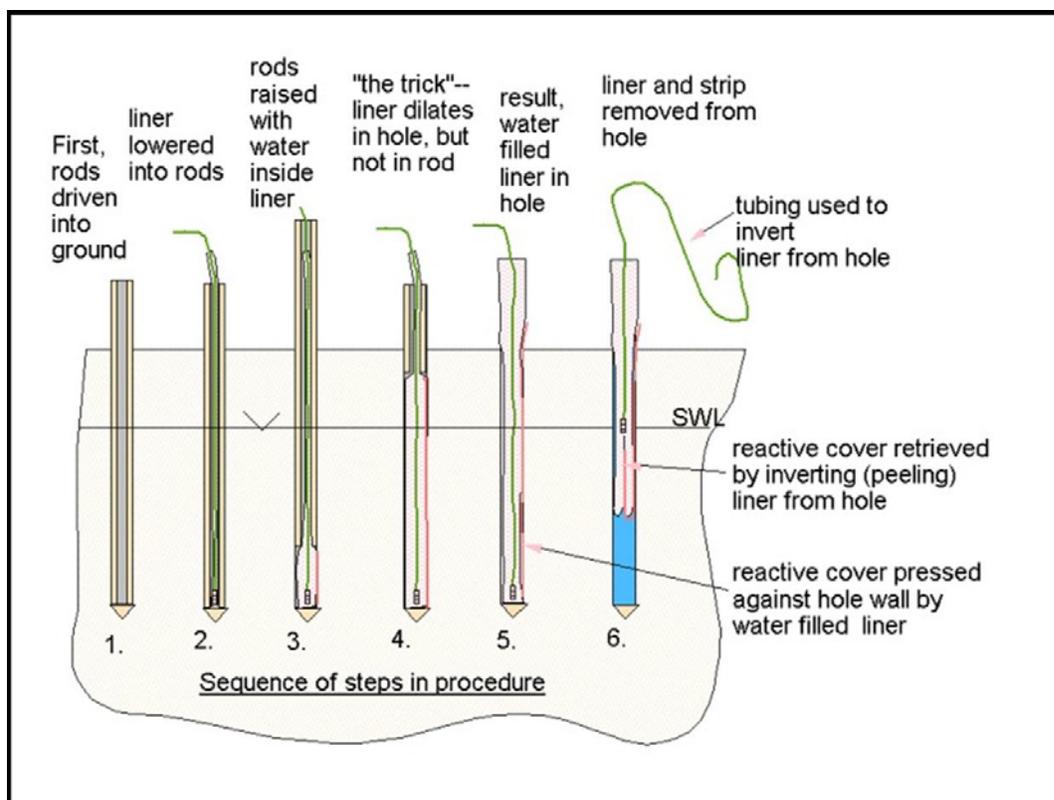


Figure 42 Schematic of FLUTE liner installation for NAPL detection
(courtesy of FLUTE Inc.)

Sealing and Decontamination

The hole produced by a push-in penetrometer may require sealing, usually with a special grout. The grouting can be carried out either after the push rods are removed using re-entry techniques (post-grouting) or during the removal process (retraction-grouting) or during the penetration process. For post-grouting, the hole can be grouted by either surface pouring or using either a flexible or rigid tremie tube. Post-grouting using surface pouring is typically only possible in stiff clays and moist dense sand above the water table where the small diameter hole will stay open after retraction of the push-rods. Post-grouting with a rigid tube is carried out after rod removal using special grout push rods with a disposable tip. The hole is grouted from the bottom up as the grout rod/tremie tube is pulled from the ground, as illustrated in Figure 43. The grout rods generally follow the previous push hole since the rods follow the path of least resistance. Post-grouting is typically the least expensive and easiest technique.

Retraction-grouting allows for grouting during push-rod retraction. The grout can be delivered to the probe through a small diameter grout tube pre-threaded through the push rods. The grout can exit the probe through either a sacrificial cone tip or through ports above the cone located on the friction reducer. A special high torque pump is used to pump the grout. Figure 44 illustrates a retraction-grouting technique. Retraction-grouting is usually expensive and often requires some probe re-design.

Grouting during the advance of the push-rods can provide a seal during penetration of the cone, i.e. while the test proceeds. Grout is contained in an over-size pipe installed and sealed in the ground around the cone hole (typically using a single flight of hollow-stem auger). The grout container can be either open or pressurized to aid in the grout flow, as illustrated in Figure 45. Grouting during probe advance is often simple, inexpensive and allows greater control.

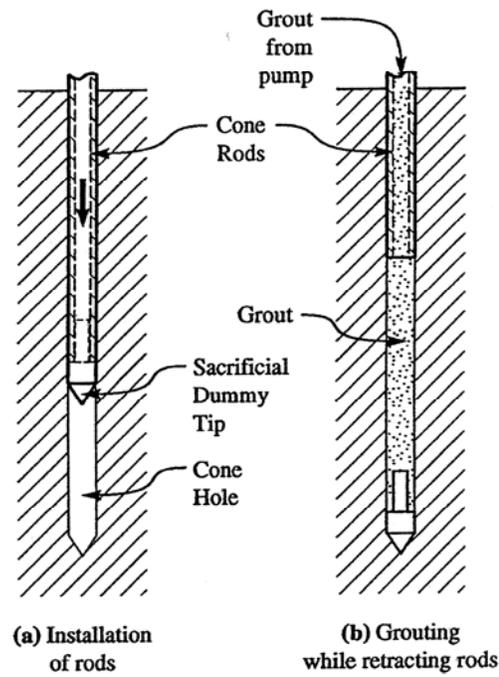


Figure 43 Post-grouting using re-entry with rigid rods and sacrificial tip (Lutenegger and DeGroot, 1995)

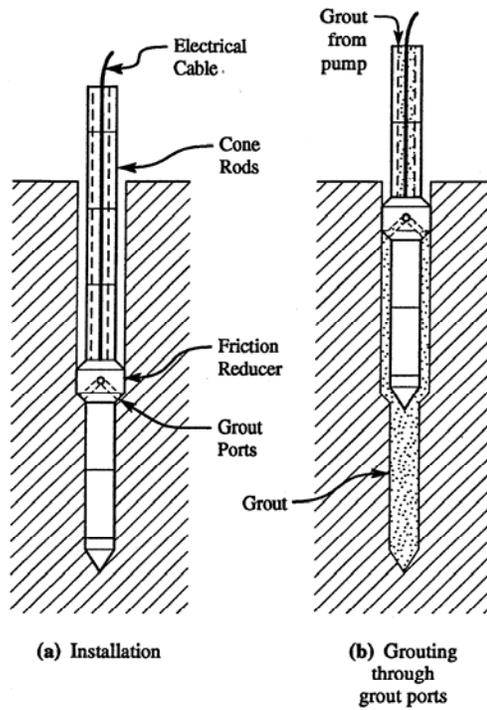


Figure 44 Example of a retraction-grouting technique (Lutenegger and DeGroot, 1995)

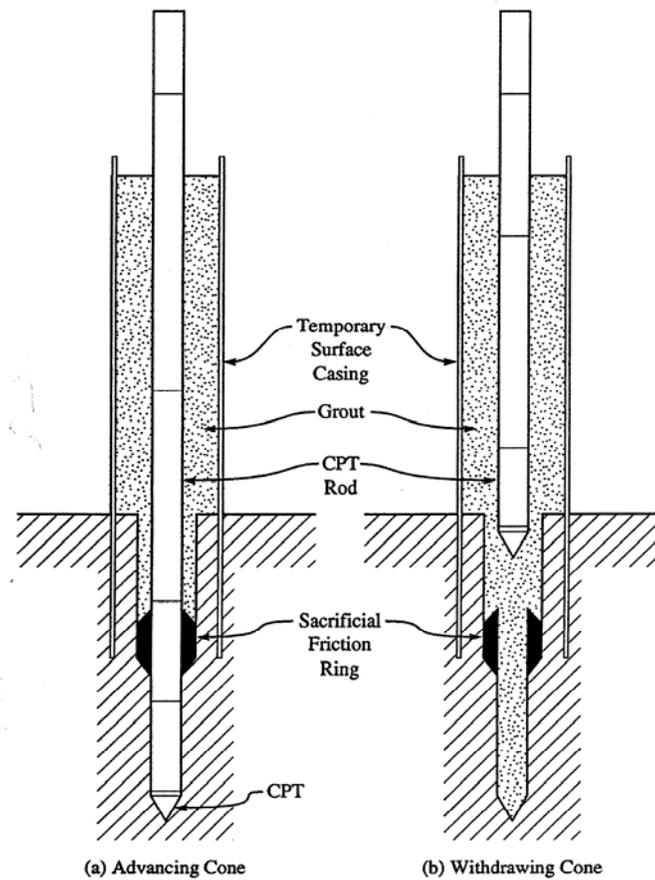


Figure 45 Example of grouting during CPT using gravity feed casing
(Lutenegger and DeGroot, 1995)

To clean the push rods and equipment after a sounding all downhole equipment can be pulled through a rod-washing, decontamination chamber. The chamber can be mounted at the base of the CPT hydraulic thrust cylinders. The push rods are then steam cleaned as they pass through the chamber, before being handled by field personnel. Rubber wipers at the entry and exit of the chamber control water leakage. The waste water is collected for later disposal. Many of the special CPT truck mounted systems also include stainless steel sinks for decontamination of samplers and penetrometers.

Summary

To better characterize potentially environmentally hazardous sites, improved investigation devices and methods have been developed using CPT technology. The CPT gathers high quality in-situ geotechnical information in a rapid cost effective manner. Sensors have been adapted to CPT probes to enable mapping of subsurface contamination in sufficient detail to reduce the quantity of more costly subsurface sampling and monitoring points. Traditional drilling and sampling methods, when compared to CPT methods, have high waste management costs from handling and disposal of contaminated materials. Also, CPT methods minimize exposure of field personnel to hazardous substances.

Significant developments have been made in recent years to improve geo-environmental site characterization using CPT technology. Sensors have been added to the cone penetrometer to enhance the logging and screening capabilities for both mechanical and chemical measurements.

Recent advances in 3-D computer visualization techniques have significantly improved data presentation with additional sensors and discrete sampling. Figure 46 illustrates a 3-D computer visualization technique.

A synergistic approach to 3-D site characterization can also be achieved by utilizing a combination of surface geophysical technology and direct-push CPT technology. By taking advantage of the synergy between the two, a significant improvement can be achieved in the interpretation of the geophysical data.

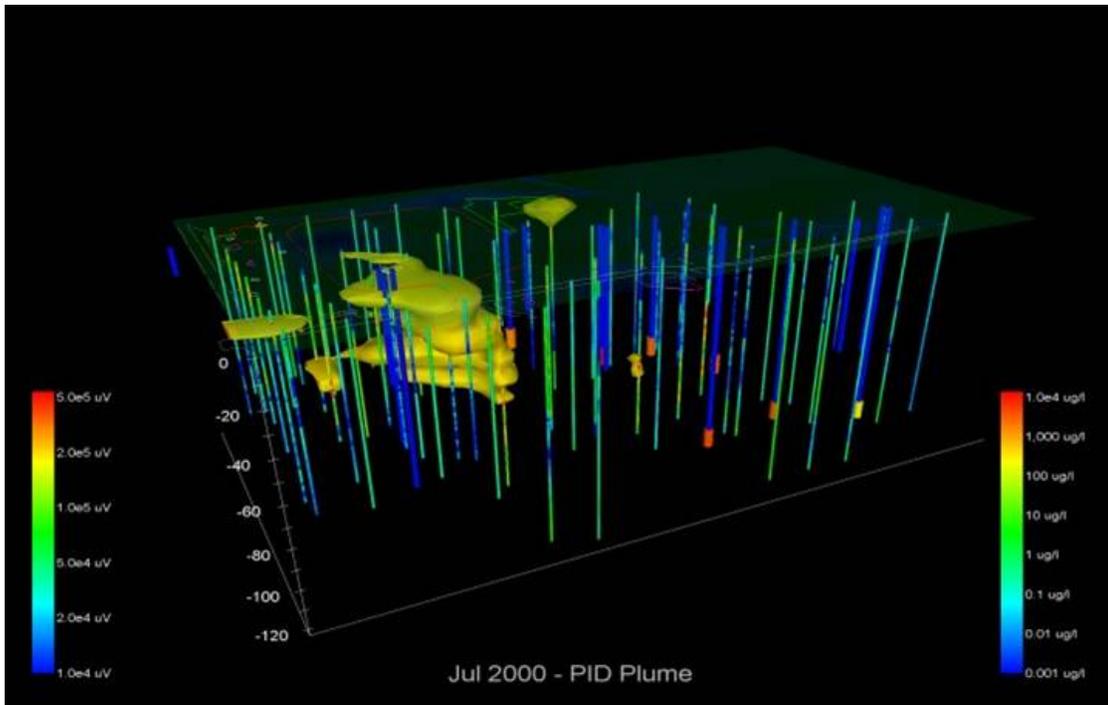


Figure 46 3-D visualization techniques using CPT and MIP results
(courtesy of Columbia Technologies Inc.)

APPENDIX A

Estimating Geotechnical Soil Parameters

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 A1 relating the ratio $(q_c/p_a)/N_{60}$ with mean grain size, D_{50} (varying between 0.001mm to 1mm). Values of q_c 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 A1. Other studies have linked the ratio between the CPT and SPT with fines content for sandy soils.

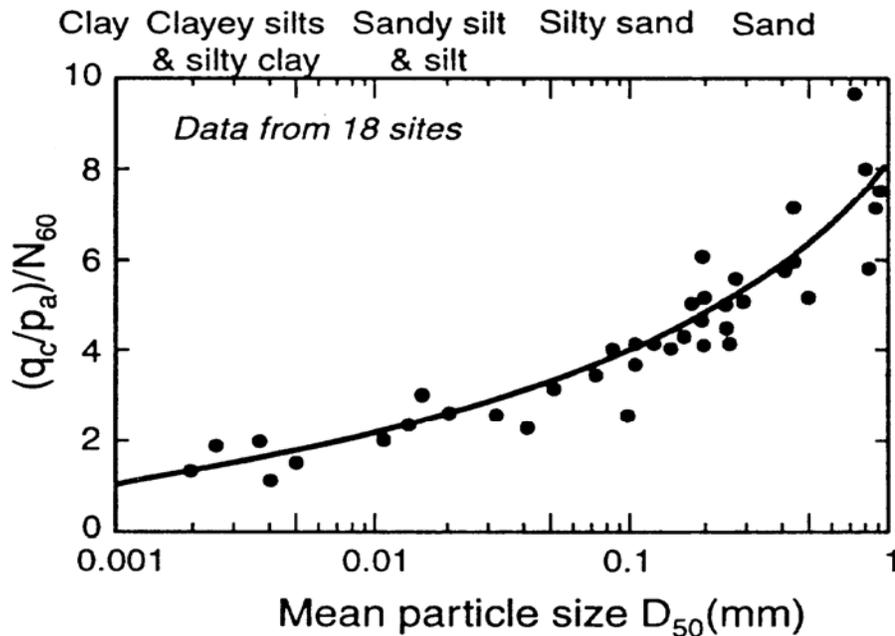


Figure A1 CPT-SPT correlations with mean grain size
(Robertson et al., 1983)

The above correlations require 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 A1.

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 . Note that in sandy soils $q_c = q_t$.

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 changes in predicted SPT N_{60} values.

Zone	Soil Behavior Type	$\frac{(q_c/p_a)}{N_{60}}$
1	<i>Sensitive fine grained</i>	2.0
2	<i>Organic soils – clay</i>	1.0
3	<i>Clays: clay to silty clay</i>	1.5
4	<i>Silt mixtures: clayey silt & silty clay</i>	2.0
5	<i>Sand mixtures: silty sand to sandy silt</i>	3.0
6	<i>Sands: clean sands to silty sands</i>	5.0
7	<i>Dense sand to gravelly sand</i>	6.0
8	<i>Very stiff sand to clayey sand*</i>	5.0
9	<i>Very stiff fine-grained*</i>	1.0

Table A1 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:

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

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. The above relationship applies to $I_c < 4.06$. The above relationship based on I_c does not work well in stiff clays where I_c maybe small due to the high OCR.

In very loose soils ($(N_1)_{60} < 10$) 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.

Soil Unit Weight (γ)

Soil total unit weights (γ) are best obtained by obtaining relatively undisturbed samples (e.g., thin-walled Shelby tubes; piston samples) and weighing a known volume of soil. When this is not feasible, the total unit weight can be estimated from CPT results, such as Figure A2 and the follow relationship (Robertson, 2010):

$$\gamma/\gamma_w = 0.27 [\log R_f] + 0.36 [\log(q_t/p_a)] + 1.236$$

where R_f = friction ratio = $(f_s/q_t)100\%$
 γ_w = unit weight of water in same units as γ
 p_a = atmospheric pressure in same units as q_t

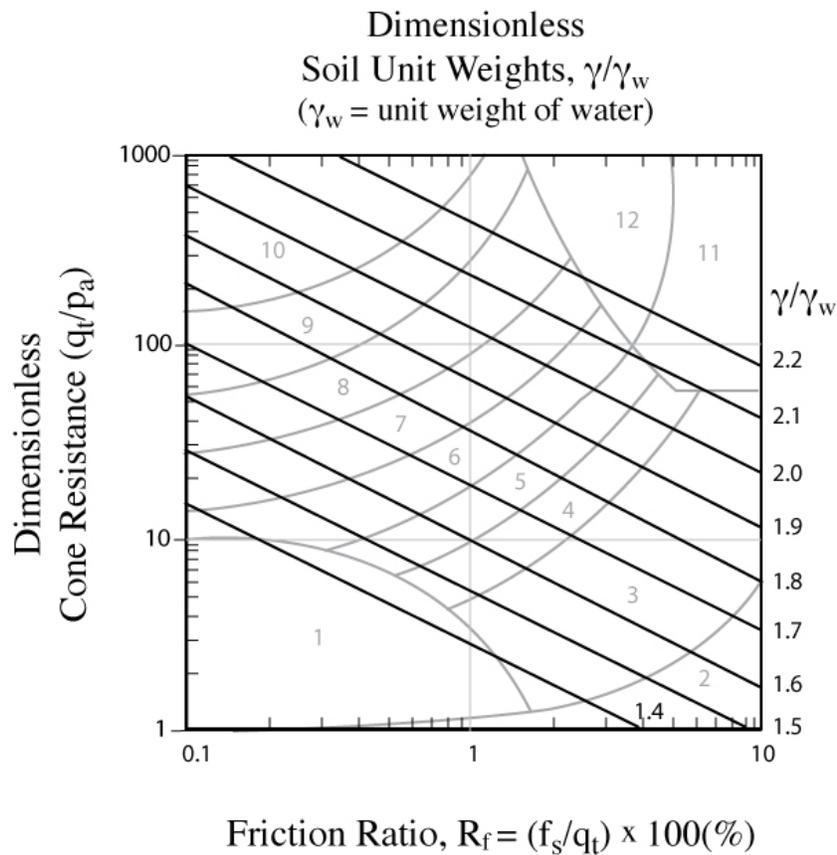


Figure A2 Dimensionless soil unit weight, γ/γ_w based on CPT

Undrained Shear Strength (s_u) - Clays

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. In general, the simple shear direction of loading often represents the average undrained strength.

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 from 10 to 18, with 14 as an average. N_{kt} tends to increase with increasing plasticity and decrease with increasing soil sensitivity. Lunne et al., 1997 showed that N_{kt} varies with B_q , where N_{kt} decreases as B_q increases, when $B_q \sim 1.0$ (i.e. sensitive clay), N_{kt} can be as low as 6.

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 14 - 16. 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 4 to 10. For a more conservative estimate, select a value close to the upper limit. Note that $N_{\Delta u}$ is linked to N_{kt} , via B_q , where:

$$N_{\Delta u} = B_q N_{kt}$$

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.

The remolded undrained shear strength, $s_{u(Rem)}$, 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 $s_{u(Rem)}$ from f_s .

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

For relatively sensitive clays ($S_t > 10$), 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.

Stress History - Overconsolidation Ratio (OCR)

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

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

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.

For overconsolidated clays:

$$(s_u/\sigma'_{vo})_{OC} = (s_u/\sigma'_{vo})_{NC} (\text{OCR})^{0.8}$$

Based on this, Robertson (2009) suggested:

$$\text{OCR} = 0.25 (Q_t)^{1.25}$$

Kulhawy and Mayne (1990) suggested a simpler method:

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

or $\sigma'_p = k (q_t - \sigma_{vo})$

An average value of $k = 0.33$ 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. The simpler Kulhawy and Mayne approach is valid for $Q_t < 20$.

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) - Clays

There is no reliable method to determine K_o from CPT. However, an estimate can be made in fine-grained soils based on an estimate of OCR, as shown in Figure A3.

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.

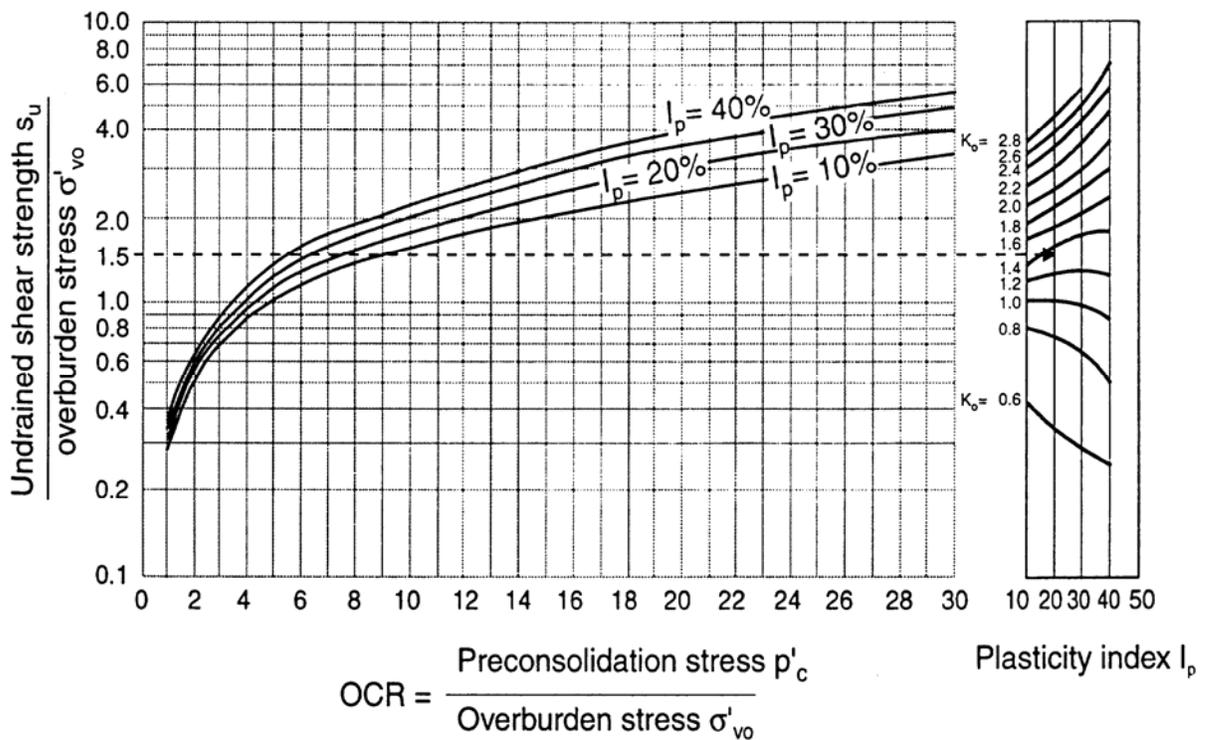


Figure A3 OCR and K_o from s_u/σ'_{vo} and Plasticity Index, I_p
 (after Andresen et al., 1979)

Friction Angle - Sands

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 cone 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.

Robertson and Campanella (1983) suggested a correlation to estimate the peak friction angle (ϕ') for uncemented, unaged, moderately compressible, predominately quartz sands based on calibration chamber test results. For sands of higher compressibility (i.e. carbonate sands or sands with high mica content), the method will tend to predict low friction angles.

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

Kulhawy and Mayne (1990) suggested an alternate relationship for clean, rounded, uncemented quartz sands, and evaluated the relationship using high quality field data, see Figure A4:

$$\phi' = 17.6 + 11 \log (Q_{tn})$$

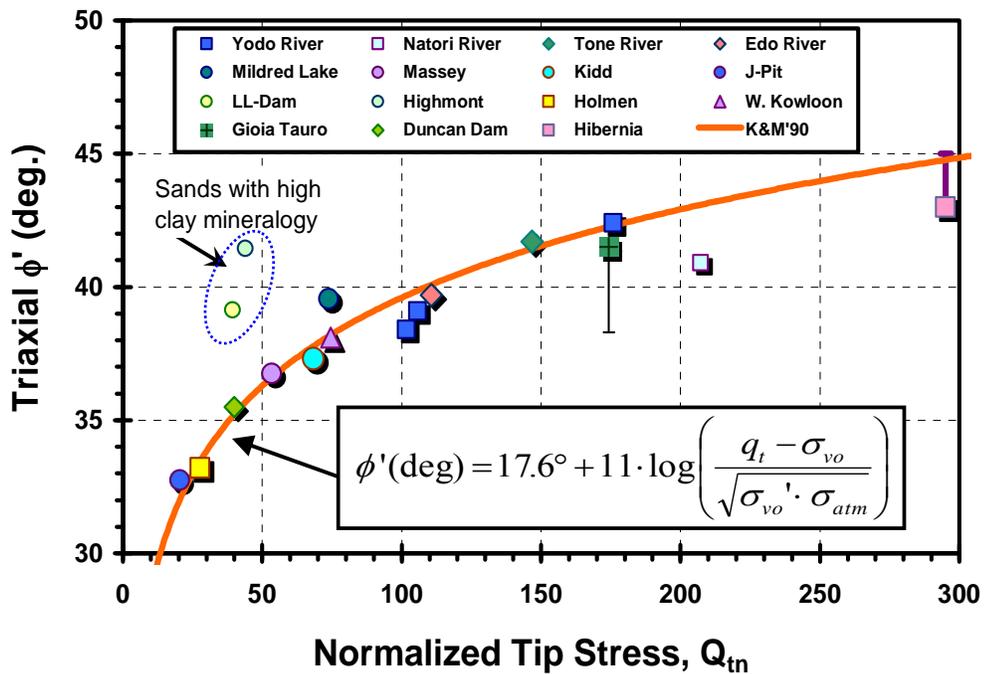


Figure A4 Friction angle, ϕ' , from CPT for unaged, uncemented, clean quartz to siliceous sand (After Mayne, 2006)

For fine-grained soils, the best means for defining the effective stress friction angle is from consolidated triaxial tests on high quality samples. An assumed value of ϕ' of 28° for clays and 32° for silts is often sufficient for many small to medium projects. Alternatively, an effective stress limit plasticity solution for undrained cone penetration developed at the Norwegian Institute of Technology (NTH: Senneset et al., 1989) allows the approximate evaluation of effective stress parameters (c' and ϕ') from piezocone measurements. In a simplified approach for normally- to lightly-overconsolidated clays and silts ($c' = 0$), the NTH solution can be approximated for the following ranges of parameters: $20^\circ \leq \phi' \leq 45^\circ$ and $0.1 \leq B_q \leq 1.0$ (Mayne 2006):

$$\phi' \text{ (deg)} = 29.5^\circ \cdot B_q^{0.121} [0.256 + 0.336 \cdot B_q + \log Q_t]$$

For heavily overconsolidated soils, fissured geomaterials, and highly cemented or structured clays, the above will not provide reliable results and should be verified by laboratory testing.

Relative density (D_r) - Sands

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 than 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:

$$D_r = \left(\frac{1}{C_2} \right) \ln \left(\frac{Q_{cn}}{C_0} \right)$$

where:

- C_0 and C_2 are soil constants
- σ'_{vo} = effective vertical stress
- Q_{cn} = $(q_c / p_a) / (\sigma'_{vo} / p_a)^{0.5}$
 = normalized CPT resistance, corrected for overburden pressure (more recently defined as Q_{tn} , using net cone resistance, q_n)
- p_a = reference pressure of 1 tsf (100kPa), in same units as q_c and σ'_{vo}
- q_c = cone penetration resistance (more correctly, q_t)

For moderately compressible, normally consolidated, unaged and uncemented, predominantly quartz sands the constants are: $C_0 = 15.7$ and $C_2 = 2.41$.

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

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

where:

- Q_{cn} and p_a are as defined above
- Q_C = Compressibility factor ranges from 0.91 (low compress.) to 1.10 (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 constant increases with age and increases significantly when age exceeds 10,000 years.

The relationship can then be simplified for most young, uncemented silica sands to:

$$D_r^2 = Q_{tn} / 350$$

Stiffness and Modulus - Sands

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

A useful guide for estimating Young's moduli for young, uncemented predominantly silica sands is given in Figure A5. The modulus has been defined as that mobilized at about 0.1% strain. For more heavily loaded conditions (i.e. larger strain) the modulus would decrease.

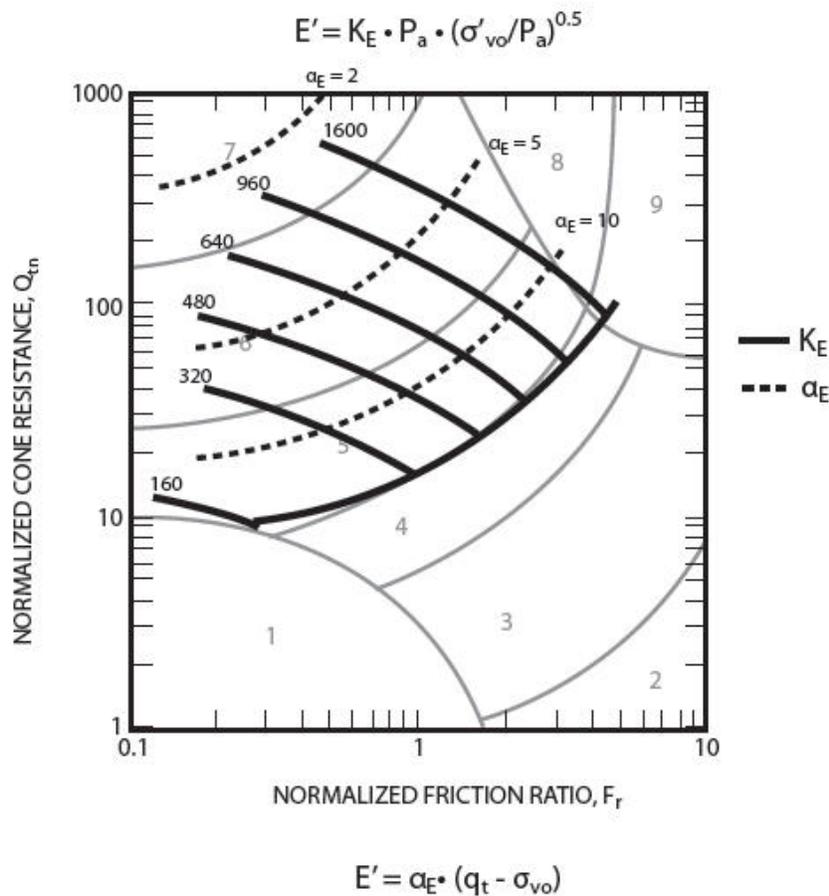


Figure A5 Evaluation of drained Young's modulus from CPT for young, uncemented silica sands, $E = \alpha_E (q_t - \sigma_{vo})$ where: $\alpha_E = 0.015 [10^{(0.55I_c + 1.68)}]$

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:

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

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 and dissipation tests are given in an earlier 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:

$$c = \left(\frac{T_{50}}{t_{50}} \right) r_o^2$$

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 Houslby (1991), as shown in Figure A6 (Teh and Houslby theory shown as solid lines for $I_r = 50$ and 500).

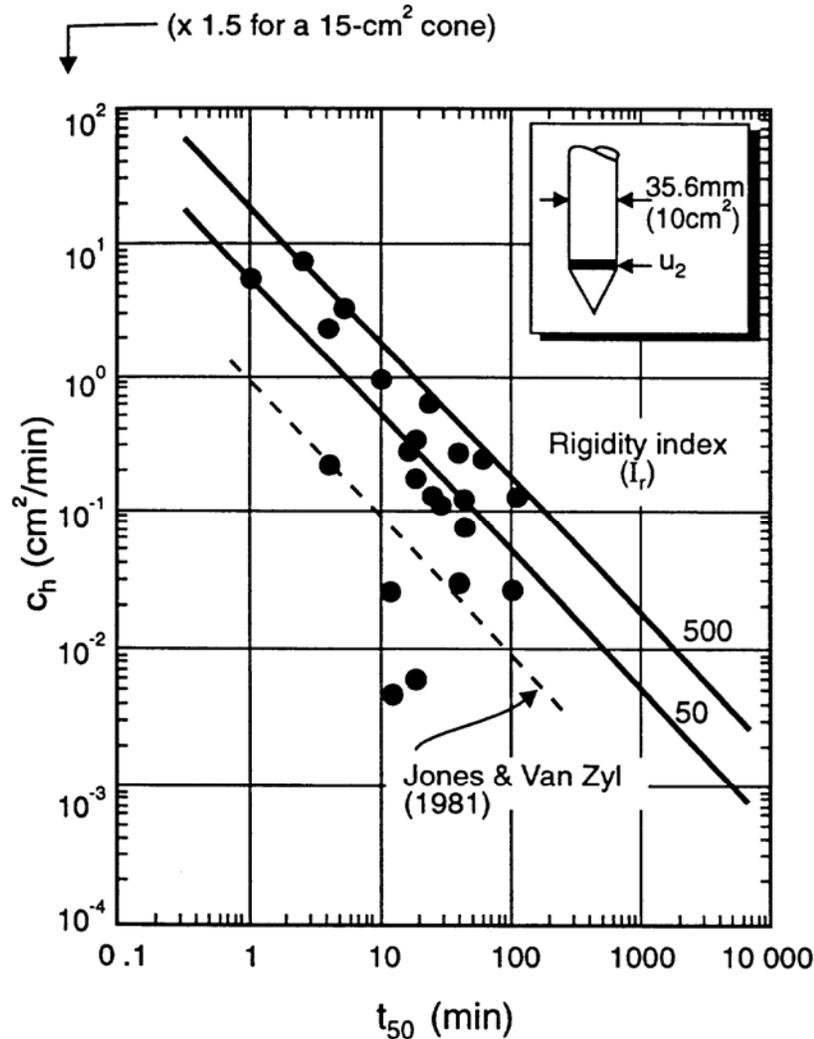


Figure A6 Average laboratory c_h values and CPTU results (Robertson et al., 1992)

The review showed that the theoretical solution provided reasonable estimates of c_h . The solution and data shown in Figure A6 apply 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. Care is required to ensure that the dissipation is continued to the correct equilibrium and not stopped prematurely after the initial rise. 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.

Based on available experience, the CPT dissipation method should provide estimates of c_h to within \pm 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 6 can be used to provide an estimate of the ratio of hydraulic conductivities.

For short dissipations in sandy soils, the dissipation results can be plotted on a square-root time scale. The gradient of the initial straight line is m , where;

$$c_h = (m/M_T)^2 r^2 (I_r)^{0.5}$$

$M_T = 1.15$ for u_2 position and 10 cm^2 cone (i.e. $r = 1.78 \text{ cm}$).

Constrained Modulus

Consolidation settlements can be estimated using the 1-D Constrained Modulus, M , where;

$$M = 1/ m_v = \delta\sigma_v / \delta\varepsilon = 2.3 (1+e_0) \sigma'_{vo} / C_c$$

Where m_v = equivalent oedometer coefficient of compressibility.

Constrained modulus can be estimated from CPT results using the following empirical relationship;

$$M = \alpha_M (q_t - \sigma_{vo})$$

Sangrelet (1970) suggested that α_M varies with soil plasticity and natural water content for a wide range of fine grained soils and organic soils, although the data were based on q_c . Meigh (1987) suggested that α_M lies in the range 2 – 8, whereas Mayne (2001) suggested a general value of 8. Robertson (2009) suggested that α_M varies with Q_t , such that;

When $I_c > 2.2$ use:

$$\alpha_M = Q_t \quad \text{when } Q_t < 14$$

$$\alpha_M = 14 \quad \text{when } Q_t > 14$$

When $I_c < 2.2$ use:

$$\alpha_M = 0.0188 [10^{(0.55I_c + 1.68)}]$$

Estimates of drained 1-D constrained modulus from undrained cone penetration will be approximate. Estimates can be improved with additional information about the soil, such as plasticity index and natural moisture content, where α_M can be lower in organic soils.

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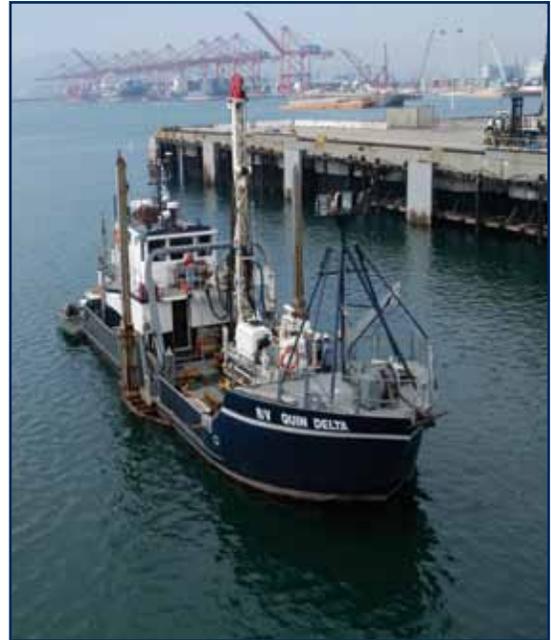
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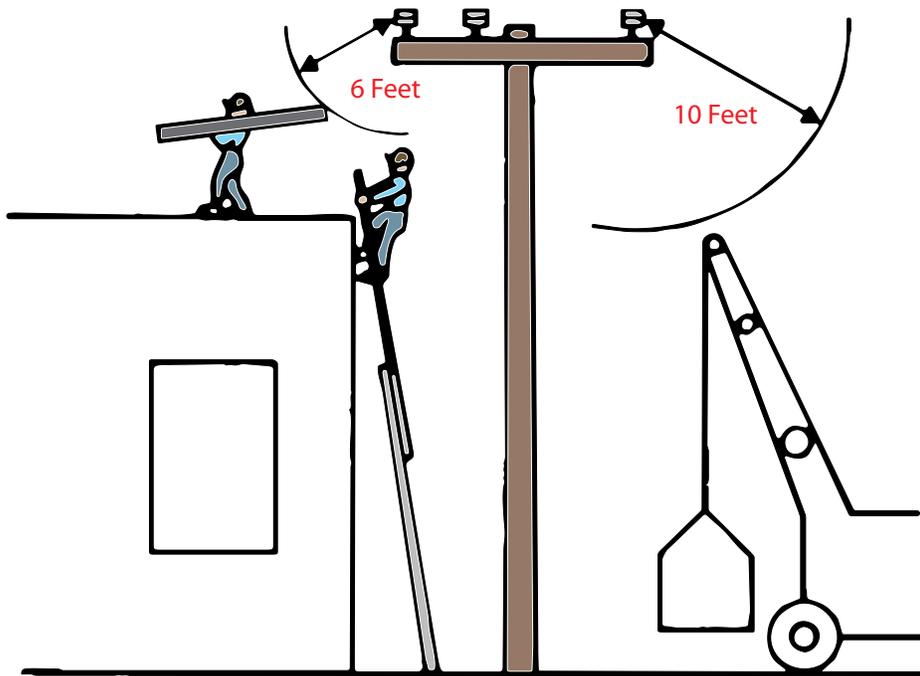


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