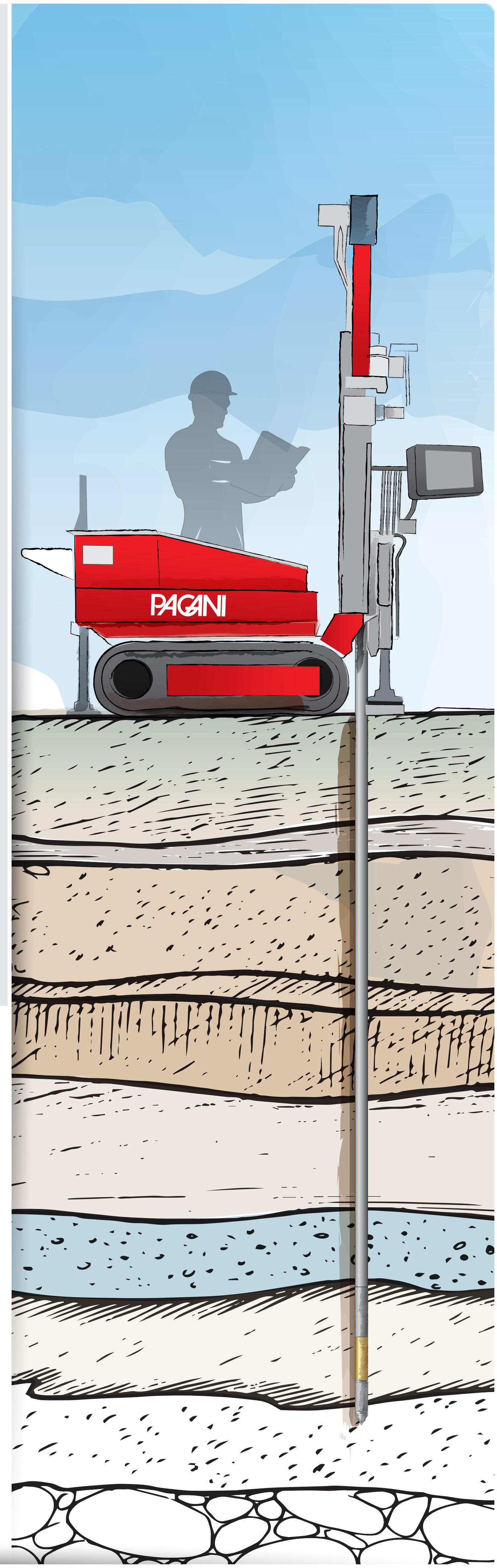
CPT HANDBOOK

"Use of cone penetration tests for soil profiling and design of shallow and deep foundations"



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1. Introduction

Among the vast number of in situ devices, the static cone penetrometer (CPTm, CPTe) and the piezocone (CPTu and SCPTu) represent the most versatile tools currently available for soil exploration. CPTm, CPTe, CPTu and SCPTu have major advantages over traditional methods of field site investigation, such as drilling and sampling, because they are fast, repeatable and economical.

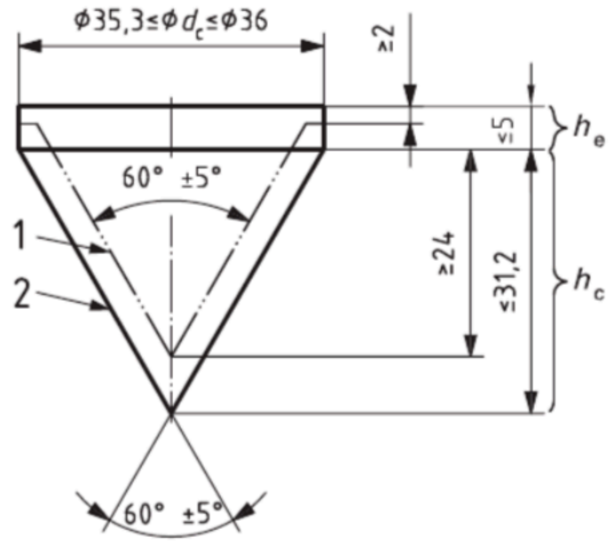
This report reviews the static cone penetration testing current practices. In particular, it provides a background on cone penetrometer equipment (chapter 2), field testing procedures (chapter 3), application (chapter 4) and interpretation (chapter 5) of CPT and CPTu (soil stratigraphy and soil parameter evaluation) and on the direct use of CPT for the design of shallow and deep foundations. Information was gathered by a literature review of international experience (books by Lunne et al, 1997 and by Mayne, 2007; international publications) and by research experiences of the University of Pisa and University of Pavia.

2. Cone penetrometer equipment

A CPT system includes the following components: (1) an electrical or mechanical penetrometer, (2) hydraulic pushing system with rods, (3) cable or transmission device, (4) depth recorder, and (5) data acquisition unit.

2.1.Penetrometers

The mechanical cone penetration test consists of pushing a cone penetrometer, by means of a series of push rods, into the soil at a constant rate of penetration (20 mm/s). During penetration, discontinuous measurements (every 20 cm of penetration) of cone penetration resistance or tip resistance (q_c), total penetration resistance and/or sleeve friction (f_s) can be recorded. Measurements are carried out by using sensors located at ground level. There are other types of mechanical cones that can only measure the cone penetration resistance but their use in practice is very limited as will be shown later on.
The front end consists of a 60° apex conical tip that has a small lip approximately 5mm (0.2 in.) long at the upper portion. The penetrometers are normally available in two standard sizes: (1) a 35.7-mm (1.4-in.) diameter version having a corresponding cross-sectional area A_c = 10 cm^2 and sleeve area A_s = 150 cm^2; and (2) a 44-mm diameter version (1.75-in.) (A_c = 15 cm^2 e A_s = 200 to 300 cm^2) (Fig.1 and Fig.2).

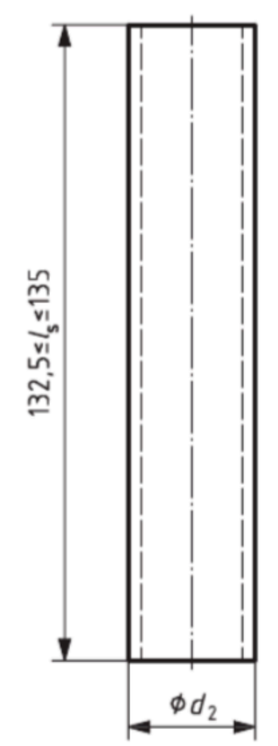


Key

1 minimum shape of cone after wear

2 maximum shape of cone

Figure 1. Tolerance requirements for use of cone penetrometer (ISO 22476-12 (2009) (Dimensions in mm)



$A_s = 15\,000\ \text{mm}^2$

$d_2 \geqslant d_c$

$d_2 < d_c + 0{,}35$

$d_2 < 36{,}1$

Key

A_s surface area of friction sleeve

l_s length of friction sleeve

d_c diameter of cone

d_2 diameter of friction sleeve

Figure 2. Tolerance requirements for friction sleeve (ISO 22476-12 (2009) (Dimensions in mm)

Although the 10-cm^2 size is the original standard size, many commercial firms have found the 15- cm^2 version to be stronger for routine profiling and more easily outfitted with additional sensors in specific needs. As rod sizes are normally 35.7 mm (1.4 in.) in diameter, the 15-cm^2 size cone also tends to open a larger hole and thus reduce side rod friction during pushing.

Within the electrical cone and piezocone penetration test, three subcategories of the cone penetration test are considered (Fig.3):

- the electrical cone penetration test (CPTe) that includes continuous measurement (every 2 cm of penetration) of cone resistance, sleeve friction and inclination;
- the piezocone test (CPTu) that is a cone penetration test with the additional measurement of pore pressure;
- the seismic piezocone test (SCPTu) that is a piezocone with the additional possibility of discontinuous measurement of body wave propagation velocities mainly in a down – hole configuration.

The CPTu is performed like a CPT with the measurement of the pore pressure at one or several locations on the penetrometer surface.
The cone penetrometer has internal load sensors for the measurement of force on the cone (cone resistance), side friction on the friction sleeve (sleeve friction) and if applicable pore pressure at one or several locations on the surface of cone penetrometer. An internal inclinometer is included for measurement of the inclination of the penetrometer.

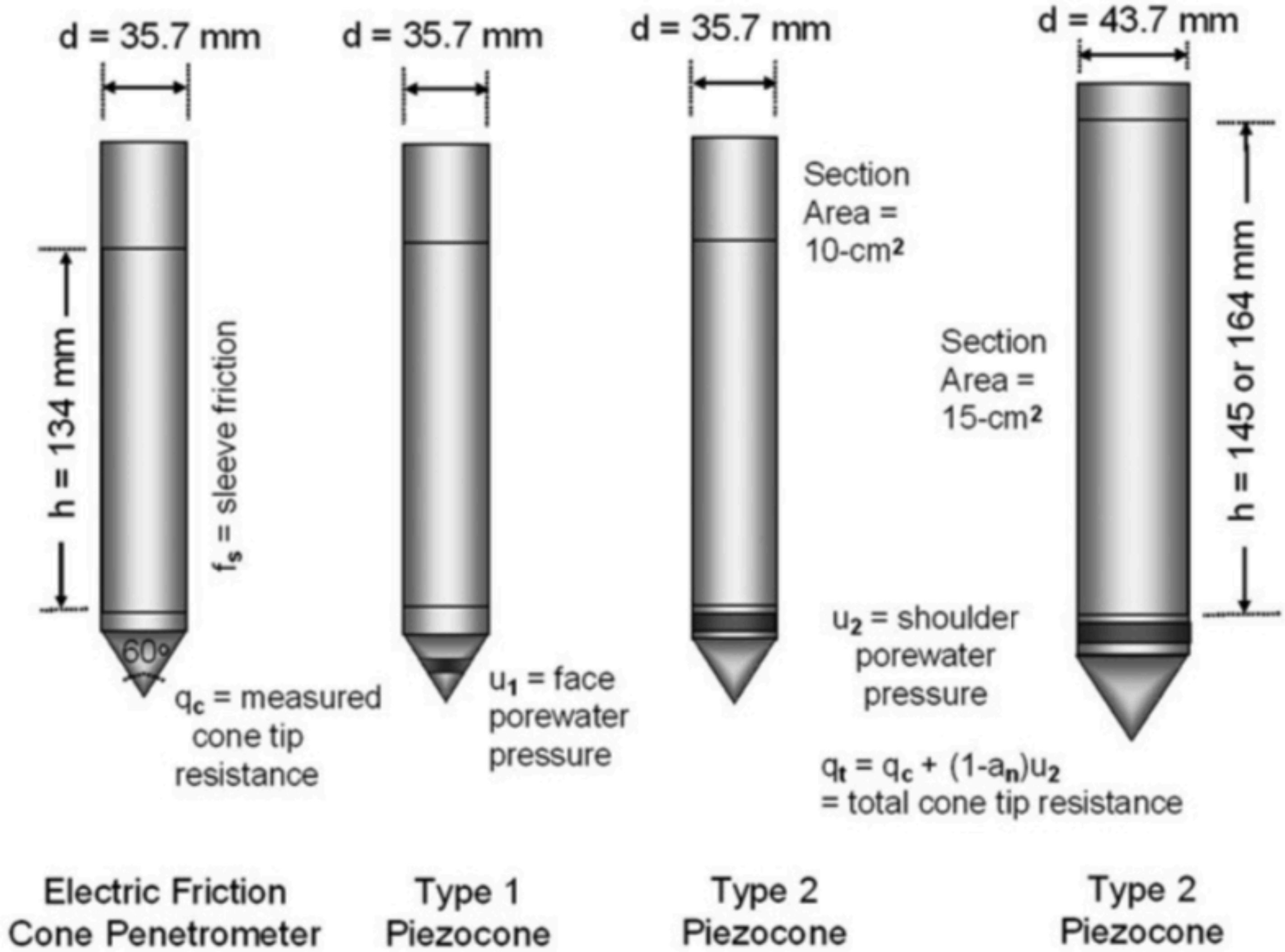


Figure 3. Basic styles of penetrometers in routine use (de Ruiter 1971).

Depending on the types of soils being tested, in the CPTu the porous filter is usually located either at the apex or mid-face (termed Type 1) or at the shoulder (Type 2) just behind the cone tip, or else positioned behind the sleeve (Type 3) (Fig.3). For the proper correction of measured cone tip resistance to total resistance, the Type 2 is required by national and international standards until proven otherwise.

Specifications on the machine tolerances, dimensions, and load cell requirements for electrical CPTe are outlined in the international reference test procedure (ASTM D 5778, 2000; ISO 22476- 12).
Most penetrometers are constructed of tool-grade steel, although a few commercial units are available in stainless steel or brass.

Periodically, the tip and sleeve elements are replaced as a result of wear or damage. It is common to replace the porewater filter after each sounding with either a disposable plastic ring type or else a reusable sintered metal or ceramic type. The reusable types can be cleaned in an ultrasonics bath.

The main differences between mechanical and electric cone are illustrated in Figure 4a-4d. It is evident that in the case of mechanical cone, the force (Q_c) necessary to push (using inner rods) into the soil only the tip is measured. After a strike of about 4 cm the inner rods start to push into the soil both the tip and the mantle therefore measuring the total force (Q_t) for another 4 cm. Therefore the sleeve friction can be inferred as Q_t-Q_c divided by the sleeve area (A_s). Apart the consideration that Q_c and Q_t are measured at different depths, it is possible to have a measurement every 20 cm. Pushing the external rods is necessary to recompact the cone and move the tip to a new position for another measurement.

On the contrary, in the case of the electric cone tip and sleeve resistances are continuously measured and inner rods are no more necessary.

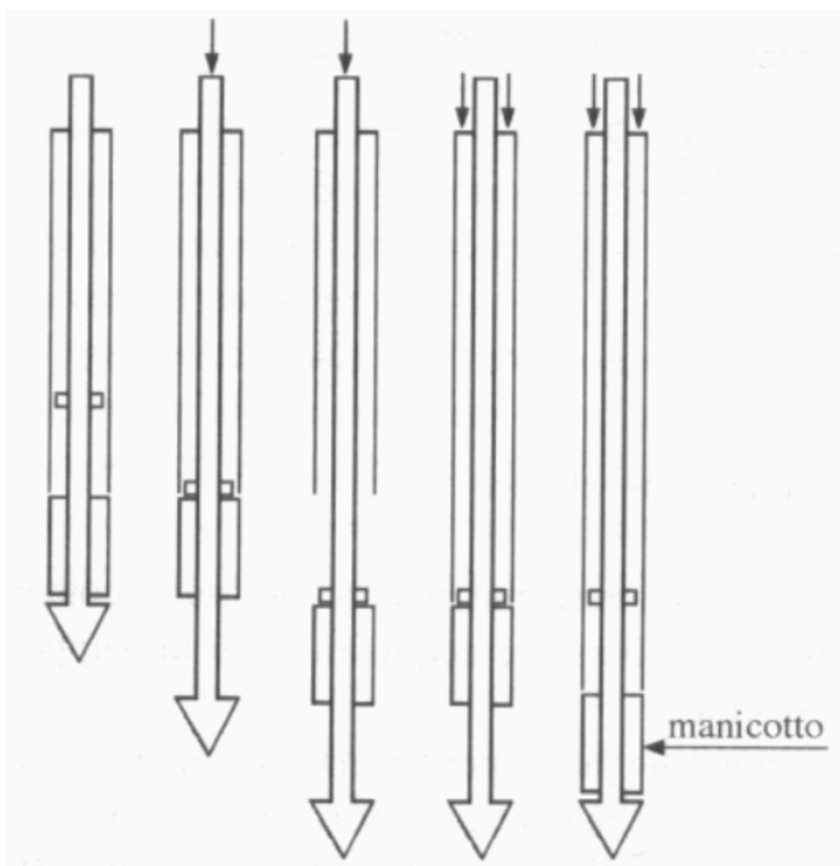


Figure 4a. Scheme of CPTm

Figure 4b. Picture of CPTm tip

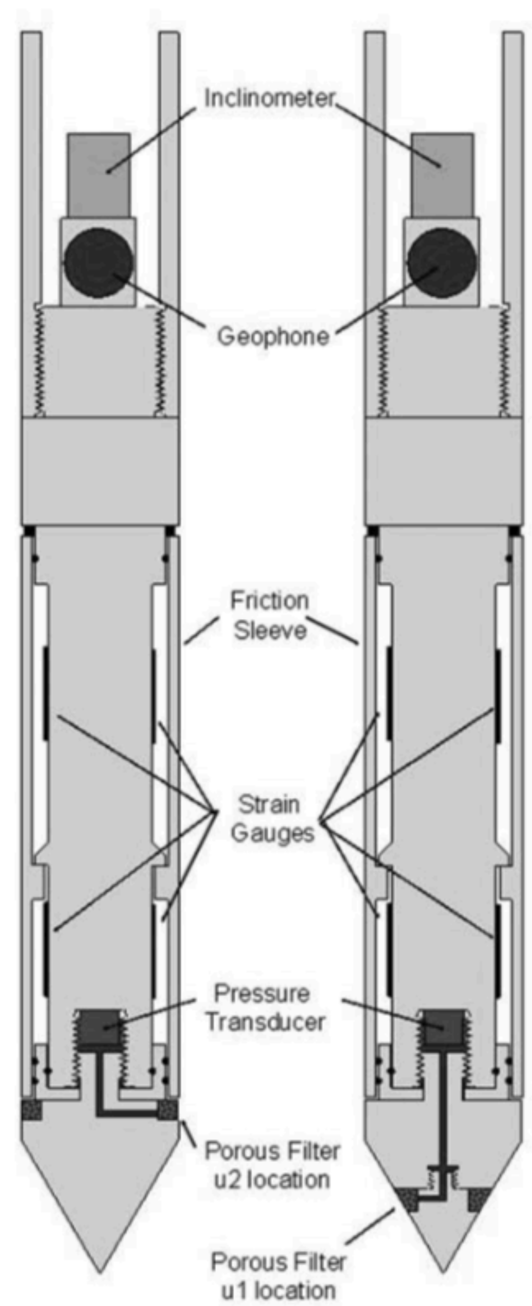


Figure 4c. Scheme of piezocone tip

Figure 4d. Picture of CPTu tip

2.2. Penetration cone resistance and Sleeve Readings

The measured force on the cone (Q_c), divided by the area gives the measured cone resistance, $q_c = Q_c/A_c$. This stress must be corrected for porewater pressures acting on unequal tip areas of the cone, especially important in soft to firm to stiff intact clays and silts (Jamiolkowski et al. 1985; Campanella and Robertson 1988; Lunne et al. 1997). The corrected cone resistance is designated as q_t, and requires two prerequisites: (1) calibration of the particular penetrometer in a triaxial chamber to determine the net area ratio (a_n); and (2) field porewater pressures to be measured at the shoulder position (u_2), as illustrated in Fig. 5.

The corrected cone resistance is determined as:

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

In clean sands and dense granular soils, the value $q_t = q_c$; thus, the correction is not paramount. However, in soft to stiff clayey soils, appreciable porewater pressures are generated and the correction can be very significant, from 20% to 70% in some instances (Lunne et al., 1986; Campanella and Robertson, 1988). Perhaps not appreciated is that, even with standard friction-type cones that do not measure porewater pressures (CPT), the correction is still needed.

The measured axial force over the sleeve (F_s) is divided by the sleeve area to obtain the sleeve friction, $f_s = F_s/A_s$. However, this too requires a correction; two porewater pressure readings are needed, taken at both the top and bottom ends of the sleeve and therefore, at this time, beyond standard practice and not required by the ASTM nor international standards.

For SI units, the depth (z) is presented in meters (m), corrected cone resistance (q_t) in either kilopascals (1 kPa = 1 kN/m^2) or megapascals (1 MPa = 1000 kN/m^2), and sleeve resistance (f_s) and pore water pressures (u) in kPa.
For conversion to English units, a simple conversion is: 1 tsf = 1 bar =100 kPa = 0.1 MPa.

If the depth to the water table is known (z_w), it is convenient to show the hydrostatic porewater pressure (u_0) if the groundwater regime is understood to be an unconfined aquifer (no drawdown and no artesian conditions). In that case, the hydrostatic pressure can be calculated from: $u_0=(z-z_w)\gamma_w$, where γ_w = 9.8 kN/m^3 = 62.4 pcf for freshwater; γ_w* = 10.0 kN/m^3 =64.0 pcf for saltwater. In some CPT presentations, it is common to report the u reading in terms of equivalent height of water, calculated as the ratio of the measured porewater pressure divided by the unit weight of water, or $h_w = u/\gamma_w$.

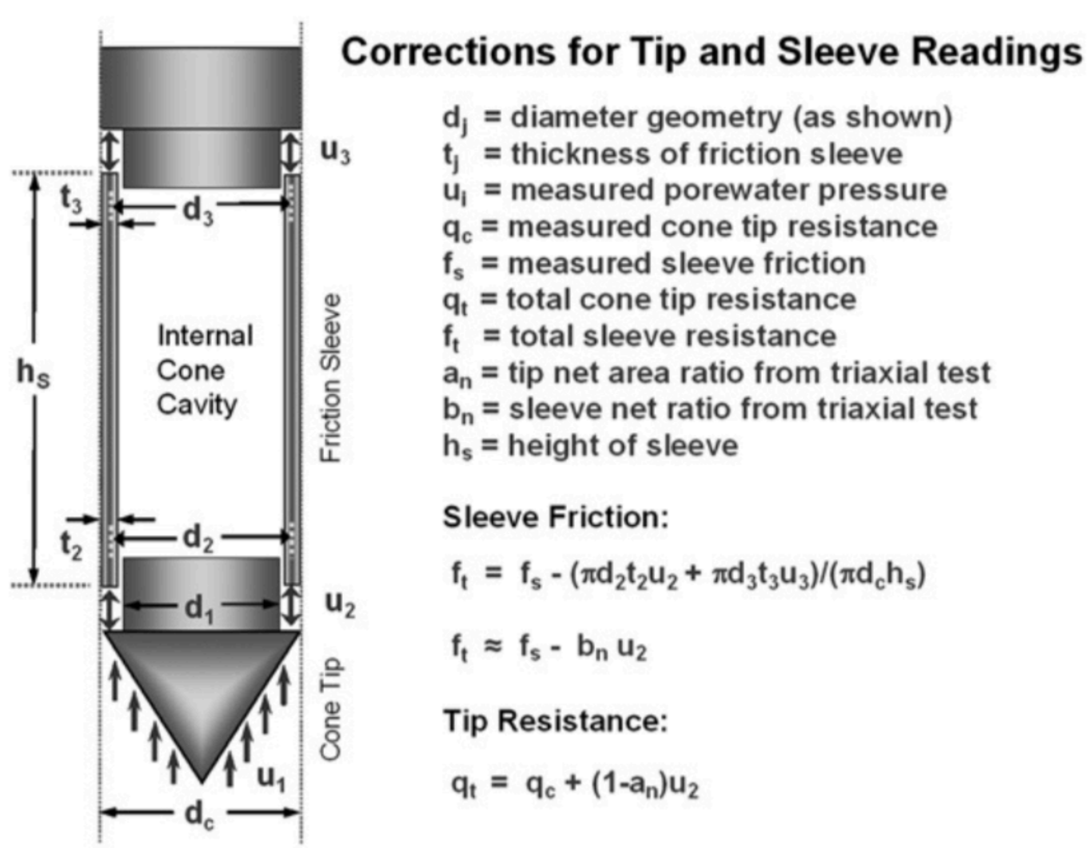


Figure 5. Determination of total cone resistance and total sleeve friction in CPTu (Jamiolkowski et al., 1985)

As a general rule of thumb, the magnitudes of CPT measurements fall into the following order: $q_t > f_s$ and $q_t > u_1 > u_2 > u_3$. The measured cone tip stresses in sands are rather high (q_t > 5 MPa or 50 tsf), reflecting the prevailing drained strength conditions, whereas measured values in clays are low (q_t < 5 MPa or 50 tsf) and indicative of undrained soil response owing to low permeability. Correspondingly, measured porewater pressures depend on the position of the filter element and groundwater level. At test depths above the groundwater table, porewater pressure readings vary with capillarity, moisture, degree of saturation, and other factors. Below the water table, for the standard shoulder element, clean saturated sands show penetration porewater pressures often near hydrostatic ($u_2 = u_0$), whereas intact clays exhibit values considerably higher than hydrostatic ($u_2 > u_0$). Indeed, the ratio u_2/u_0 increases with clay hardness. For soft intact clays, the ratio may be around $u_2/u_0 \sim \pm 3$, which increases to about $u_2/u_0 \sim \pm 10$ for stiff clays, yet as high as 30 or more for very hard clays. However, if the clays are fissured, then zero to negative porewater pressures are observed (e.g., Mayne et al. 1990).

The friction ratio is defined as the ratio of the sleeve friction to cone tip resistance, designated $R_f = f_s/q_t$, and reported as a percentage. The friction ratio has been used as a simple index to identify soil type. In clean quartz sands to siliceous sands (comparable parts of quartz and feldspar), it is observed that friction ratios are low: $R_f<1\%$, whereas in clays and clayey silts of low sensitivity, $R_f > 4\%$. However, in soft sensitive to quick clays, the friction ratio can be quite low, approaching zero in many instances.

2.3.Penetration Pore water Pressures

The measured pore pressure is influenced by soil type, in situ pore pressure and filter location on the surface of the cone penetrometer. The pore pressure consists of two components, the original in situ pore water pressure and the additional or excess pore pressure caused by the penetration of the cone penetrometer into the ground. Usually, porewater pressures are monitored using a saturated filter element connected through a saturated portal cavity that connects to a pressure transducer housed within the penetrometer. The standard location is the shoulder element (just behind the tip, designated u_2), because of the required correction to total tip stress discussed previously. However, in stiff fissured clays and other geologic formations (e.g., residual soils), zero to negative porewater pressures can be recorded. Therefore, in these cases, superior profiling capability is attained using a face porous element, usually located midface, although some apex versions have been used as well. The pore pressure measuring system shall be saturated at the start of the test. The filter should remain saturated even when the cone penetrometer is penetrating an unsaturated layer.

Without due care, the resulting measurements will appear either incorrect or sluggish, not realizing their full magnitude, because of trapped air pockets or gas within the system.

2.4. Hydraulic Pushing System

The hydraulic pushing system can consist of a standard drill rig or a dedicated CPT hydraulic system mounted on a truck, track, trailer, all-terrain vehicle, skid arrangement, or portable unit (Fig.6). A full-capacity hydraulic system for CPT work is considered to be on the order of 200 kN (22 tons). The dedicated CPT systems push near their centroid of mass and usually rely on deadweight reaction of between 100 to 200 kN (11 to 22 tons) for capacity. A few specialized vehicles have been built with add-on weights to provide up to 350 kN (40 tons) reaction. After being positioned at the desired test location, the rig is usually leveled with hydraulic jacks or "outriggers." There are also many small lightweight CPT systems in the 18 to 50 kN range (2 to 6 tons).

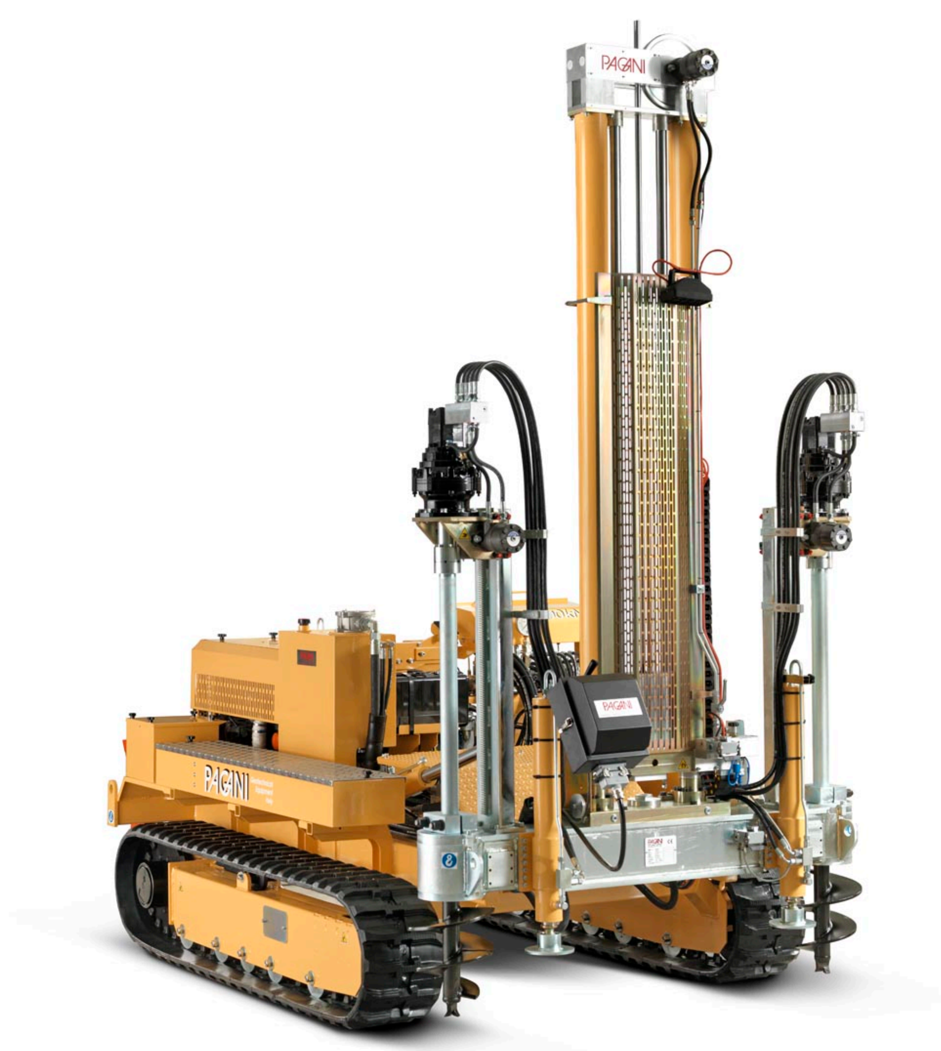


Figure 6. Example of CPT hydraulic system mounted on tracks

As an alternative or in addition to deadweight, the use of earth anchoring is the way to get a 100 to 200 kN capacity (Fig.7). These anchored rigs can obtain significant depths and penetrate rather dense and hard materials, yet are more mobile and portable than the deadweight vehicles.

Figure 7. Example of anchoring system

Typical depths of penetration by CPT rigs depend on the site-specific geologic conditions; however, most commercial systems are set up for up to 30 m (100 ft). In some special cases, onshore CPTs have reached 100 m using direct-push technology from the ground surface. Downhole CPTs can also be conducted step-wise in deep boreholes by alternating off and on with rotary drilling bits, with depths up to 300 m (1,000 ft) or more achievable (e.g., Robertson 1990).

The standard rate of testing is at a constant push of 20 mm/s (0.8 in./s) per ASTM D 5778.
Typical rates of drilling of soil borings by state agencies are between 15 and 30 m/day (50 to 100 ft/day). Therefore, in terms of linear productivity, CPT is two to five times more efficient than conventional rotary drilling. A disadvantage of the CPT rigs is that their basic abilities include only pushing and pulling the probes. Some limited ability exists for occasional soil sampling, if necessary; however, this is not routine.

The advantages of using standard drill rigs for CPT work include the added capabilities to drill and bore through hard cemented or very dense zones or caprock, if encountered, and then continue the soundings to the desired depths, as well as to obtain soil samples on-site, using the same rig. This reduces costs associated with mobilizing a dedicated CPT truck. Major difficulties with CPTs done using standard drill rigs include: (1) the deadweight reaction is only around 50 kN (5.5 tons); (2) during advancement, rods are pushed from the top, thus an escape slot or special subconnector piece must be provided for the electrical cable, as necessary; (3) during withdrawal, rods must be pulled from the top, thus a subconnector piece must be added and removed for each rod break; and (4) care in manual control of hydraulic pressure must be made to achieve a constant 20 mm/s push rate.

As an alternative it is possible to use static/dynamic penetrometer (Fig. 8). This is a penetrometer that, in addition to CPT can carry out continuous dynamic penetration tests. In this case it is possible to pass the hard or coarse layer in dynamic mode by hammering the closed end tip used for dynamic probing.

Figure 8. Example of static/dynamic penetrometer

2.5.Push Rods and inner roads

Push rods shall have the same diameter as the cone for at least 400 mm, measured from the cone base for cones with a base area of 1000 mm2. The inner rods are solid rods sliding inside the push rods and transferring the force from the cone and, optionally, the friction sleeve, to the measuring system. As already stated inner rods are necessary only in the case of mechanical cone.

A friction reducer is often provided to facilitate pushing operations. The friction reducer is merely an enlarged section of rods (e.g., a ring welded to the outside rod) on the subconnector above the penetrometer that opens the pushed hole to a larger diameter, thereby reducing soil contact on all the upper rods (Fig. 9).

Figure 9. Push rods and internal rod (for CPTm)

2.6.Depth logger

There are several methods to record depth during the advancement of the CPT. Some common systems include depth wheel, displacement transducer [either linear variant displacement transducer (LVDT) or direct current displacement transducer (DCDT)], potentiometer (spooled wire), gear box, ultrasonics sensor, and optical reader. All are available from commercial suppliers and some designs are patented for a particular system. In most cases, a cumulative tracking of each one-meter rod increment is made to determine depth. In other cases, the actual total cable length is monitored. Because each of the channel sensors is technically positioned at slightly different elevations, it is standard practice to correct the readings to a common depth, usually taken at the tip of the penetrometer (Fig. 10).

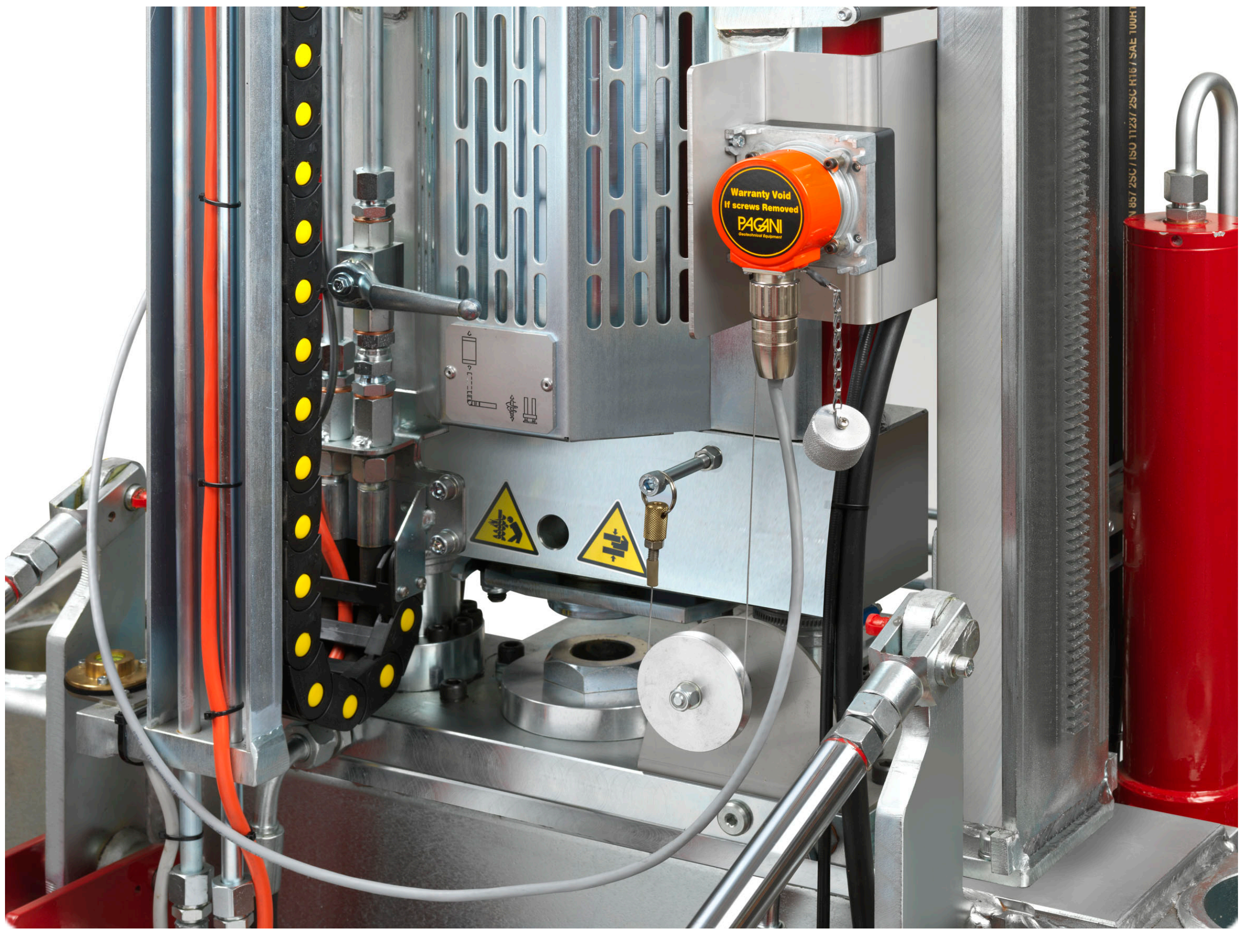


Figure 10. Example of depth logger

2.7.Data Transmission and Cabling

All analog CPT systems and many digital CPT systems use a cable threaded through the rods for transmission of data uphole. The cable is used to provide voltage (or current) to the penetrometer and to transmit data back up to the computer for storage. A power supply is normally used to provide a voltage of between 5 and 20 V, depending on the manufacturer design.

In some of the newest designs, wireless (or cableless) digital CPT systems have been developed. They are particularly favored when CPT is conducted using standard drill rigs and crews (because the cable might easily be damaged) and in offshore site investigations where wireline can deploy the units to great depths. A variety of wireless systems are available based on the following technologies for data transmission or storage:

1. infrared signals conveyed uphole in glass-lined rods;

2. audio-transmitted signals;
3. data stored in a battery powered microchip until the penetrometer is retrieved back at the surface.

With these infrared and acoustic transmissions, a special receiver is required uphole at the top end of the rods to capture the signals and decode them for digital output (Fig.11).

Figure 11. Cable transmission arrangements

2.8.Data acquisition system

A wide variety of data acquisition systems have been developed for electric CPTe, initially starting with simple pen plotters and analog–digital converters to matrix dot printers, and evolving to fully digital systems with ruggedized notebooks and microchip technologies, with memory within the cone penetrometer itself. An advantage of the older analog systems is that they could be adapted to accommodate any type of commercial cone. The disadvantage of some newer digital systems is that proprietary designs restrict the data coding and channel sequences from the output. Therefore, only a matched set of penetrometer, cable, and data acquisition system can be used.

In the case of mechanical CPTm data acquisition is still carried out manually.

Figure 12a. Example of manual DA for mechanical tip

Figure 12b. Example of data acquisition system for CPTu tests

3. Testing procedures

3.1.Calibration and maintenance of penetrometer

The penetrometer requires calibration and maintenance on a regular basis; the frequency of which depends on the amount of use and care taken during storage between soundings. For most CPT operators, it appears that the penetrometers and/or field computers are returned to their respective manufacturers to confirm the equipment is within calibration and tolerances.

However, calibrations can be conducted in-house to check for load cell compliance using a compression machine. A sealed and pressurized triaxial apparatus can be used to check for pressure transducer calibrations, as well as the net area ratio (an). Full details concerning the calibration of cone and piezocone penetrometers are given elsewhere (e.g., Mulabdic´ et al., 1990; Chen and Mayne, 1994; Lunne et al., 1997).

The tip and sleeve should be replaced if damaged or if excessively worn. For a typical CPT rate of 60 m/day, used 4 days/week, an annual production of 12 000 m/year would likely require tips and sleeves being replaced once to twice per annum. The rate will depend on soils tested, as sands are considerably more abrasive than clays.

3.2.Filter elements

The filter elements used for piezocone testing are usually constructed of porous plastic, ceramic, or sintered metal. The plastic versions are common because they are disposable and can be replaced after each sounding to avoid any possible clogging problems particularly those associated with plastic clays. For face elements, a ceramic filter is preferred because it offers better rigidity and is less prone to abrasion when compared with plastic filters. The protocol for environmental soundings recommends that sintered stainless steel filters be used, because polypropylene types are from petroleum based manufacturer and may cross contaminate readings.

Sintered elements are not to be used for face filters however because of smearing problems. The sintered metal and ceramic filters are reusable and can be cleaned using an ultrasonics bath after each sounding.
Saturation of the filter elements should be accomplished using a glycerine bath under vacuum for a period of 24 h. An alternative would

be the use of silicone oil as the saturation fluid. It is also possible to use water or a 50–50 mix of glycerine and water; however, those fluids require much more care during cone assemblage. It is normal practice to presaturate 10 to 15 elements overnight for use on the next day's project.

In the field, the filter elements must be installed so that a continuity of fluid is maintained from the filter face through the ports in the penetrometer and cavity housing the pressure transducer. These ports and cavities must also be fluid-filled at all times. This is best accomplished using a penetrometer having a male plug in the tip section to promote positive fluid displacement when the tip is screwed onto the chassis. The fluid should be 100% glycerine (or silicone oil) that is easily applied using a plastic syringe. Otherwise, if a female plug is provided on the tip unit, the penetrometer must be carefully assembled while submerged in the saturating fluid, usually accomplished with a special cylindrical chamber designed for such purposes. Considerably more effort is expended with this procedure than the aforementioned approach with a positive displacement plug on the tip.

Once assembled, it is common practice to tightly place a prophylactic containing saturation fluid over the front end of the penetrometer. Several rubber bands are used to secure the rubber covering and help maintain the saturated condition.
During the initial push into the ground this light rubber membrane will rupture automatically. In new developments, in lieu of a filter element and saturation procedure, it is possible to use a very thin (0.3 mm) grease-filled slot to record porewater pressures (Elmgren, 1995; Larsson, 1995). This avoids problems associated with vacuum presaturation of elements, assembly difficulties in the field, and desaturation of elements in the unsaturated vadose zone, however, at the expense of a more sluggish transducer response and less detailing in the u profiling.

3.3. Baseline readings

Before each sounding, electronic baselines or "zero readings" of the various channels of the penetrometer are recorded. It is also recommended that a set of baseline readings be secured after the sounding has been completed and the penetrometer withdrawn to the surface. These baselines should be recorded in a field log booklet and checked periodically to forewarn of any mechanical or electronic shifts in their values, as possible damage or calibration errors may occur.

3.4.Advancing the penetrometer

The standard rate of push for CPT/CPTu soundings is 20 mm/s, usually applied in one-meter increments (standard cone rod length). With dedicated CPT rigs, the hydraulic system is automatically established to adjust the pressures accordingly to maintain this constant rate.
When penetrating coarse materials, predrilling may be used in parts of the profile if the penetration stops in dense, coarse or stone-rich layers. Predrilling may be used in coarse top layers sometimes in combination with casings to avoid collapse of the borehole. In soft or loose soil predrilling can be used to penetrate the crust to reach the groundwater table. If the groundwater table is located at large depths, the pore pressure system should be saturated. Predrilling can be done by ramming a dummy-rod of (45 to 50) mm diameter through the dense layer to provide an open hole and reduce the penetration resistance. As already stated, static/dynamic penetrometers offer the possibility of passing a hard or coarse layer in dynamic mode.

The achievable penetration length depends on the soil conditions, the allowable penetration force, the allowable forces on the push rods and push rod connectors, the application of a friction reducer and/or push rod casing and the measuring range of the cone penetrometer.

3.5.Tests at intermittent depths

At each one-meter rod break, there is an opportunity to conduct intermittent testing before the next succession of pushing as the next rod is added. Two common procedures include: (1) dissipation testing, and (2) downhole shear wave velocity measurements.

3.5.1.Porewater Dissipation Tests

Dissipation testing involves the monitoring of pore water pressures as they decay with time. The installation of a full displacement device such as a cone penetrometer results in the generation of excess porewater pressures (Δu) locally around the axis of perturbation. In clean sands, the Δu will dissipate almost immediately because of the high permeability of sands, whereas in clays and silts of low permeability the measured Δu will require a considerable time to equilibrate. Given sufficient time in all soils, the penetrometer pore water channel will eventually record the ambient hydrostatic condition corresponding to u_0. Thus, the measured pore water pressures (u) are a combination of transient and hydrostatic pressures, such that:

$$u = \Delta u + u_0$$

During the temporary stop for a rod addition at one-meter breaks, the rate at which Δu decays with time can be monitored and used to interpret the coefficient of consolidation and hydraulic conductivity of the soil media. Dissipation readings are normally plotted on log scales; therefore, in clays with low permeability it becomes impractical to wait for full equilibrium that corresponds to Δu = 0 e u = u0.

A standard of practice is to record the time to achieve 50% dissipation, designated t_{50}.

3.5.2.Shear Wave Testing

A convenient means to measure the profile of shear wave velocity (V_s) with depth is through the seismic cone penetration test (SCPTu).

At the one-meter rod breaks, a surface shear wave is generated using a horizontal plank or autoseismic unit. The shear wave arrival time can be recorded at the test elevation by incorporating one or more geophones within the penetrometer. The simplest and most common is the use of a single geophone that provides a pseudo-interval downhole V_s (Campanella et al., 1986). This approach is sufficient in accuracy as long as the geophone axis is kept parallel to the source alignment (no rotation of rods or cone) and a repeatable shear wave source is generated at each successive one-meter interval.

A more reliable V_s is achieved by true-interval downhole testing; however, this requires two or more geophones at two elevations in the penetrometer [usually 0.5 or 1.0 m vertically apart (1.5 to 3.0 f_t)]. Provision of a biaxial arrangement of two geophones at each elevation allows correction for possible cone rod rotation, because the resultant wave can be used. For downhole testing, incorporation of a triaxial geophone with vertical component is useful only to measure P wave arrival. There are cases where P wave measurements in saturated soils does not make sense and is in practice not possible. The vertical component could be used in a crosshole test arrangement for SV waves (e.g., Baldi et al. 1988).

3.6.Hole Closure

After the sounding is completed, a number of possible paths may be followed during or after extraction:

- CPT hole is left open.
- Hole is backfilled using native soils or pea gravel or sand.
- Cavity is grouted during withdrawal using a special “loss tip” or retractable portal.
- After withdrawal, hole is reentered using a separate grouting system.

4. Application of CPT and CPTu

Application classes have been developed to give guidance on selecting the type of CPT/CPTu and the required accuracy. The application class specifies the type of cone penetrometer to be used and the suggested use of CPT/CPTu results for given soil profiles. The use of CPT/CPTu results is stated in terms of profiling, material identification and definition of soil parameters (ISO 22476-1; ISO 22476-12).

4.1. Mechanical CPTm

One of the following types of measuring system (type a, b or c) shall be used.

a) Type a: This consists of manometers measuring the hydraulic pressures generated by the force acting on the cone and transferred to the top of the inner rods and, if applicable, by the force on the cone and friction sleeve, and by the total force on the push rods. The use of two significantly different ranges for manometers simultaneously and switching frequently to the appropriate range is recommended for this type of measuring device.

b) Type b: This is comprised of electrical sensors measuring the hydraulic pressures generated by the force acting on the cone and transferred to the top of the inner rods and, if applicable, by the force on the cone and friction sleeve, and by the total force on the push rods.

c) Type c: This type comprises electrical sensors directly measuring the forces on the cone penetrometer. The use of separate devices to measure the forces needed for determining cone penetration resistance, sleeve friction and total penetration resistance is recommended for this type of measuring system.

Table 1 - Types of static cone penetration test

Test type	Measured and derived parameters	Measurement system
TM1	Cone penetration resistance and total penetration resistance or cone penetration resistance and sleeve friction	Electrical sensor (type c) - discontinuous testing
TM2	Cone penetration resistance and total penetration resistance or cone penetration resistance and sleeve friction	Manometers or electrical sensor converting hydraulic pressures (types a and b) – discontinuous testing
TM3	Cone penetration resistance	Manometers or electrical sensor converting hydraulic pressures (types a and b) – discontinuous testing
TM4	Cone penetration resistance	Manometers or electrical sensor converting hydraulic pressures (types a and b) – discontinuous testing

Table 2 - Application classes

Application class	Type of cone	Allowable minimum accuracy (a)			Suggested use	
					Soil type (b)	Interpretation (c)
5	TM1	q_c	500 kPa	O 5%	A	F
		Q_t	1 kN	O 5%	B	G, H*
		f_s	50kPa	O 20%	C	G, H*
		L	0,2 m	O 2%	D	G, H*
6	TM2	q_c	500 kPa	O 5%		
		Q_t	1 kN	O 5%	B	G, H*
		f_s	50kPa	O 20%	C	G, H*
		L	0,2 m	O 2%	D	G, H*
	TM3 TM4	q_c	500 kPa	O 5%		
		Q_t	1 kN	O 5%	B	F*
		f_s	50kPa	O 20%	C	F*
		L	0,2 m	O 2%	D	F*

(a)

The allowable minimum accuracy of the measured parameter is the larger value of the two quoted. The relative accuracy applies to the measured value and not the measured range

b)

A: Homogeneously bedded soils (typically q_c < 2 MPa).

B: Clays, silts and sands (typically 2 MPa <= q_c < 4 MPa).

C: clays, silts, sands and gravels (typically 4 MPa <= q_c <= 10 MPa)
D: clays, silts, sands and gravels (typically qc> 10 MPa).

c)
F: profiling.
F*: profiling possible if extra information is provided.
G: profiling and material identification.
G*: indicative profiling and material identification.
H: interpretation in terms of engineering parameters..
H*: indicative Interpretation in terms of engineering parameters..

Class 5 is intended for the evaluation of mixed bedded soils, soil types A to D. For soil types B to D, profiling, material identification and indicative interpretation in terms or engineering parameters is achievable. For very soft layers (soil type A) only soil profiling is possible. Material identification and interpretation in terms or engineering parameters, especially for very soft layers, is only possible if complementary and relevant geological and geotechnical information is available. Tests are lo be performed with a cone penetration test type TM1.
Class 6 is intended for the evaluation of mixed bedded soils, with soil types B to D, in terms of profiling and material identification. Evaluation of very soft layers is limited to detection of these layers. Tests are to be performed using test type TM2.
Class 7 is intended only for indicative profiling for mixed bedded soils, soil types B to D. No interpretation in terms of material identification and engineering parameters can be given only on the basis of these test results. Tests are to be performed using test type TM3 orTM4.

Although electrical CPTe is preferred to mechanical CPTm, mechanical CPTm can be preferable in case of risk of damage by, for example, debris, cobbles or bedrock.

4.2.Electrical CPTe, CPtu

Application class	Test type (d)	Measured parameters	Allowable minimum accuracy (a)	Maximum length between measurements	Use	
					Soil (b)	Interpretation (c)
1	TE2	Cone resistance	35 kPa o 5%	20 mm	A	G, H
		Sleeve friction	5 kPa o 10%			
		Pore pressure	10 kPa o 2%			
		Inclination	2°			
		Penetration length	0.1 m o 1%			
2	TE1 TE2	Cone resistance	100 kPa o 3%	20 mm	A B C D	G, H* G, H G, H G, H
		Sleeve friction	15 kPa o 15%			
		Pore pressure	25 kPa o 3%			
		Inclination	2°			
		Penetration length	0.1 m o 1%			
3	TE1 TE2	Cone resistance	200 kPa o 5%	20 mm	A B C D	G G, H* G, H G, H
		Sleeve friction	25 kPa o 15%			
		Pore pressure	50 kPa o 5%			
		Inclination	5°			
		Penetration length	0.2 m o 2%			
4	TE1	Cone resistance	500 kPa o 5%	50 mm	A B C D	G* G* G* G*
		Sleeve friction	50 kPa o 5%			
		Penetration length	0.2 m o 1%			

a)

The allowable minimum accuracy of the measured parameter is the larger value of the two quoted. The relative accuracy applies to the measured value and not the measured range

b)

A: homogeneously bedded soils with very soft to stiff clays and silts (typically q_c<3MPa)

B: mixed bedded soils with soft to stiff clays (tipically q_c<=3MPa) and medium dense sands (tipically 5MPa<=q_c<10 MPa)
C: mixed bedded soils with soft to stiff clays (typically 1.5 MPa<=q_c<=3Mpa) and medium dense sands (tipically q_c>20 MPa)
D: mixed bedded soils with stiff clays (typically qc>=3Mpa) and very dense coarse soils (qc>=20 MPa)

c)
G: profiling and material identification with low associated uncertainty level
G*: indicative profiling and material identification with low associated uncertainty level
H: interpretation in terms of design with low associated uncertainty level
H*: indicative interpretation in terms of design with low associated uncertainty level

d)
TE1: cone resistance, sleeve friction are the measured parameters (CPTe)
TE2: cone resistance, sleeve friction and pore pressure are the measured parameters (CPTu)

Application Class 1 is intended for soft to very soft soil deposits. Class 1 penetration test are normally not apt for mixed bedded soil profiles with soft to dense layers (although pre-drilling through stiff layers can overcome the problem). Tests can only be performed with use of the CPTu.

Application Class 2 is intended for precise evaluation for mixed bedded soil profiles with soft to dense layers, in terms of profiling and material identification. Interpretation in terms of engineering properties is also possible, with restriction to indicative use for the soft layers. Penetrometer type to be used depends on project requirements.

Application Class 3 is intended for evaluation of mixed bedded soil profiles with to soft to dense soils, in terms of profiling and material identification. Interpretation in terms of engineering properties in achievable for very stiff to hard and dense to very dense layers. For stiff clays or silts and loose sands only an indicative interpretation can be given. Penetrometer type to be used depends on project requirements.

Application Class 4 is only intended for indicative profiling and material identification for mixed bedded soil profiles with soft to very stiff or loose to dense layers. No appreciation in terms of engineering parameters can be given. Tests are to be performed with a standard electrical cone penetrometer and inclination measurement may be omitted.

5. Interpretation of CPT and CPTu

The CPT and CPTu have three main applications:

- To determine sub-surface stratigraphy and identify materials present;
- To estimate geotechnical parameters;
- To provide results for direct geotechnical design.

5.1.Soil stratigraphy

The engineering geological model can be used in the characterization of a site for engineering purposes. Also it can provide an indication of the potential variations in the properties of the soil and hence possible errors in calculations or assumptions, especially those assuming homogeneity. The engineering geological model can be achieved through the identification of the stratigraphic units and the spatial reconstruction of the lithological variability; generally this can be done through geognostic surveys (boreholes, trench pits, etc...).
Unfortunately, drilling methods have comparatively high costs and the number of boreholes is often largely insufficient for geological engineering mapping purposes. A series of alternative methods, including geophysical tests, can be used as possible complementary tools for stratigraphic investigations. Among these, penetration tests, which are significantly less expensive, are economical methodologies which enable one to get continuous measurements of some soil parameters (tip resistance q_c, sleeve friction f_s, and in the case of CPTu, the pore water pressure during penetration). Measurement repeatability and the possibility of investigating a soil volume, greater than that of laboratory samples, as well as the opportunity to get continuous records, make CPT and CPTu ideal for the identification of lithologic variations and the reconstruction of the stratigraphic profile, which are important tasks for engineering geological model construction. Amorosi and Marchi (1999), based on a detailed comparison of piezocone test data and boreholes, show that piezocone testing can be used for sedimentological purposes, including detailed facies characterization, subsurface stratigraphic correlations, and identification of the key surfaces for sequence-stratigraphic interpretation. Lafuerza et al. (2005) constructed a 3D model from cone penetration tests (CPT) and piezocone tests (CPTu) in order to establish the architectural stacking

pattern of deltaic sediment bodies for sedimentological and stratigraphical purposes.
Measurement repetitivity and the possibility of investigating a soil volume, greater than that of laboratory samples, as well as the opportunity to get continuous records, make CPT and CPTu ideal for the reconstruction of the stratigraphic profile and the determination of the mechanical properties of the mechanical parameters.
Therefore, CPT and CPTu are complementary tools for stratigraphic investigations because they allow:

- lithotype identification
- identification of stratigraphic boundaries
- lithological variations
- reconstruction of the stratigraphic profile
- stratigraphic correlations
- providing a high-resolution data set suitable for 3D modeling.

More importantly, CPT and CPTu can provide soil parameters (mainly strength parameters in drained or undrained conditions). Therefore the capability of discriminating the Soil Behavior Type using the CPT or CPTu results is essential for a correct assessment of soil parameters.

5.1.1. Soil classification charts

A soil classification system provides a means of grouping soils according to their engineering behavior. The conventional method for determining a soil type is by laboratory classification of samples retrieved from a borehole. If a continuous, or nearly continuous, subsurface profile is desired, the cone penetration test (CPT) provides time and cost savings over traditional methods of sampling and testing.
A number of classification methods are reported to predict soil type from either CPT or/both CPTu data (Begemann, 1965; Schmertmann, 1978; Searle, 1979; Douglas and Olsen, 1981; Senneset and Janbu, 1985; Robertson et al., 1986; Campanella and Robertson, 1988; Robertson, 1990, 2009, 2010; Jefferies and Davies, 1991; Eslami and Fellenius, 1997; Fellenius and Eslami, 2000; Jung et al., 2008; Cetin and Ozan, 2009).

The stratigraphic profile for mechanical CPT is usually obtained using one of the following approaches:

a) BEGEMANN (1965): the classification chart for mechanical cone penetration tests is based on 250 different data, relating to Dutch soils (Fig.13). The q_c is on the y-axis and the sleeve friction f_s on the x-axis. The lines (passing through the origin), which subdivide the map in fields, allowing us to identify the soil, were obtained on the basis of the weight percentage of particles with a diameter less than 16 µm.

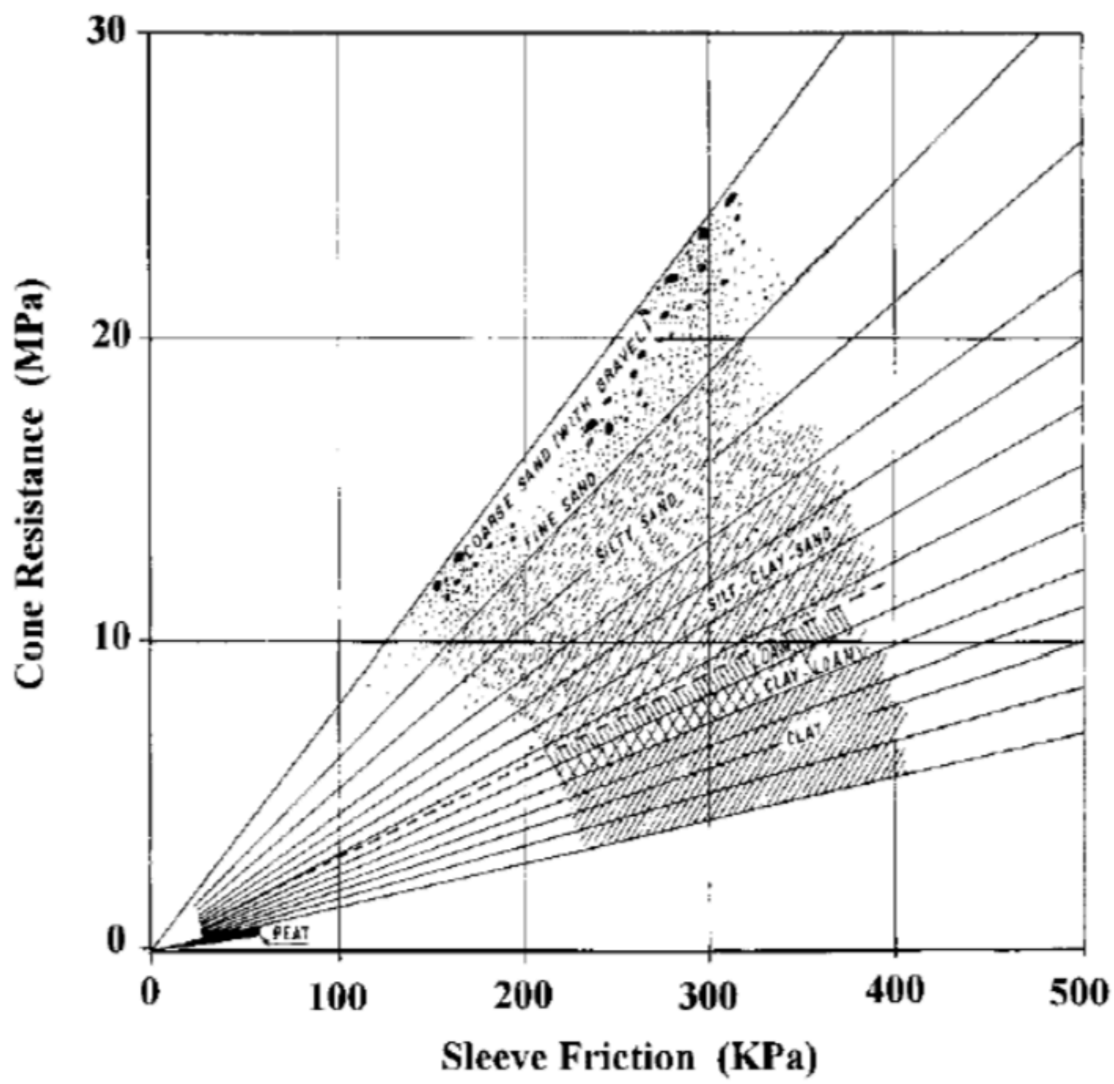


Figure 13. Soil classification chart (Begemann, 1965)

b) SCHMERTMANN (1978): the graph uses the Begemann database and a series of mechanical cone penetration tests carried out in Florida (Fig. 14). On the y-axis q_c is plotted on a logarithmic scale, whereas the friction ratio $R_f = (f_s/q_c)*100$ is plotted on the x-axis on a linear scale. Qualitative indications about density of sands (increasing with q_c) and stiffness of clays (increasing with f_s) are also given. The most important differences with respect to the Begemann chart concern the limits for the different lithologies and the non- linearity between q_c and f_s. The method is not so accurate for low q_c values

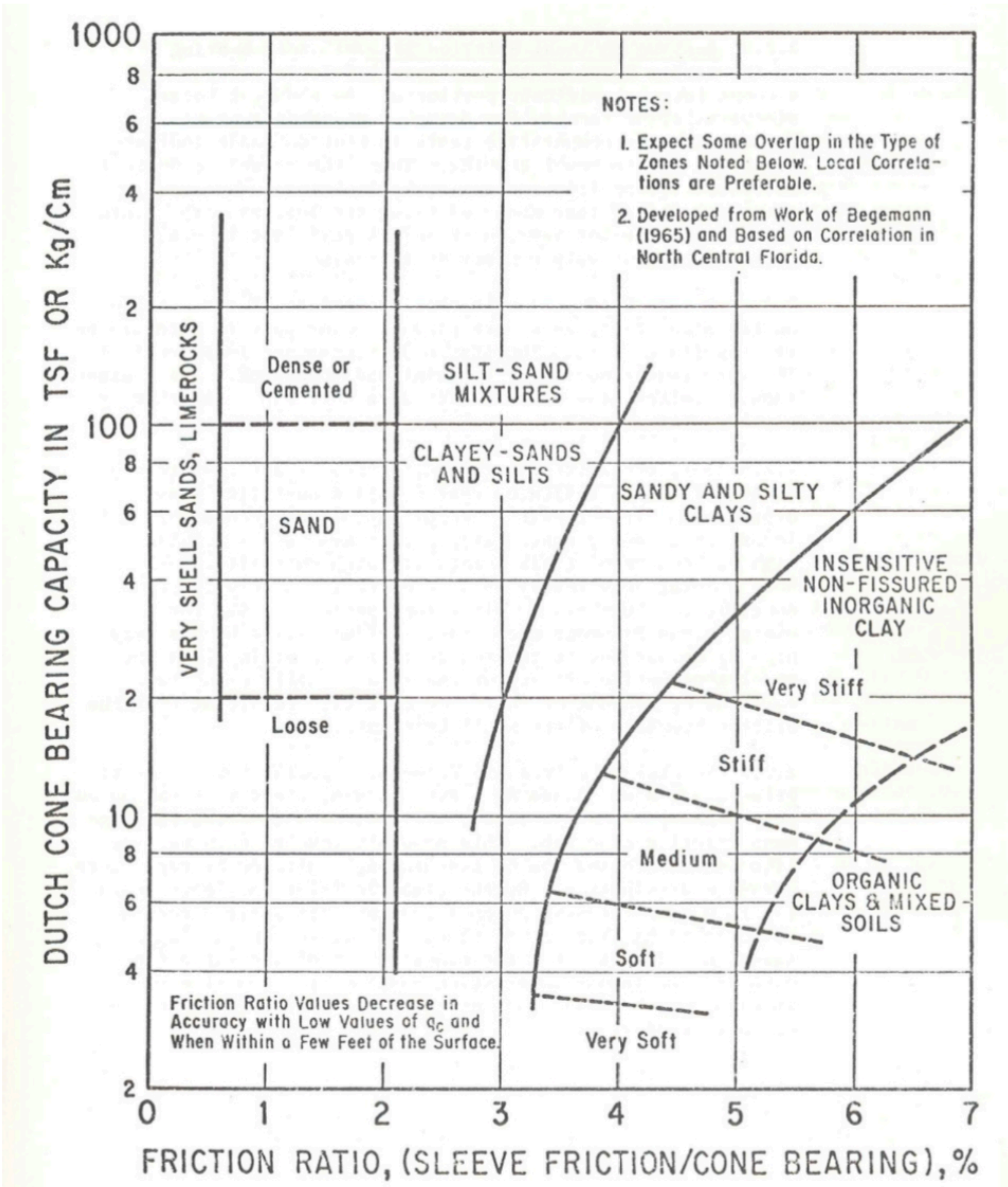


Figure 14. Soil classification chart (Schmertmann, 1978)

c) SEARLE (1979): the classification chart represents the cone resistance q_c (MPa) on the y axis Rf in the same scale (Fig.15). The Searle method, like the Schmertmann method, provides additional indications, such as the density of sands and stiffness of fine soils.

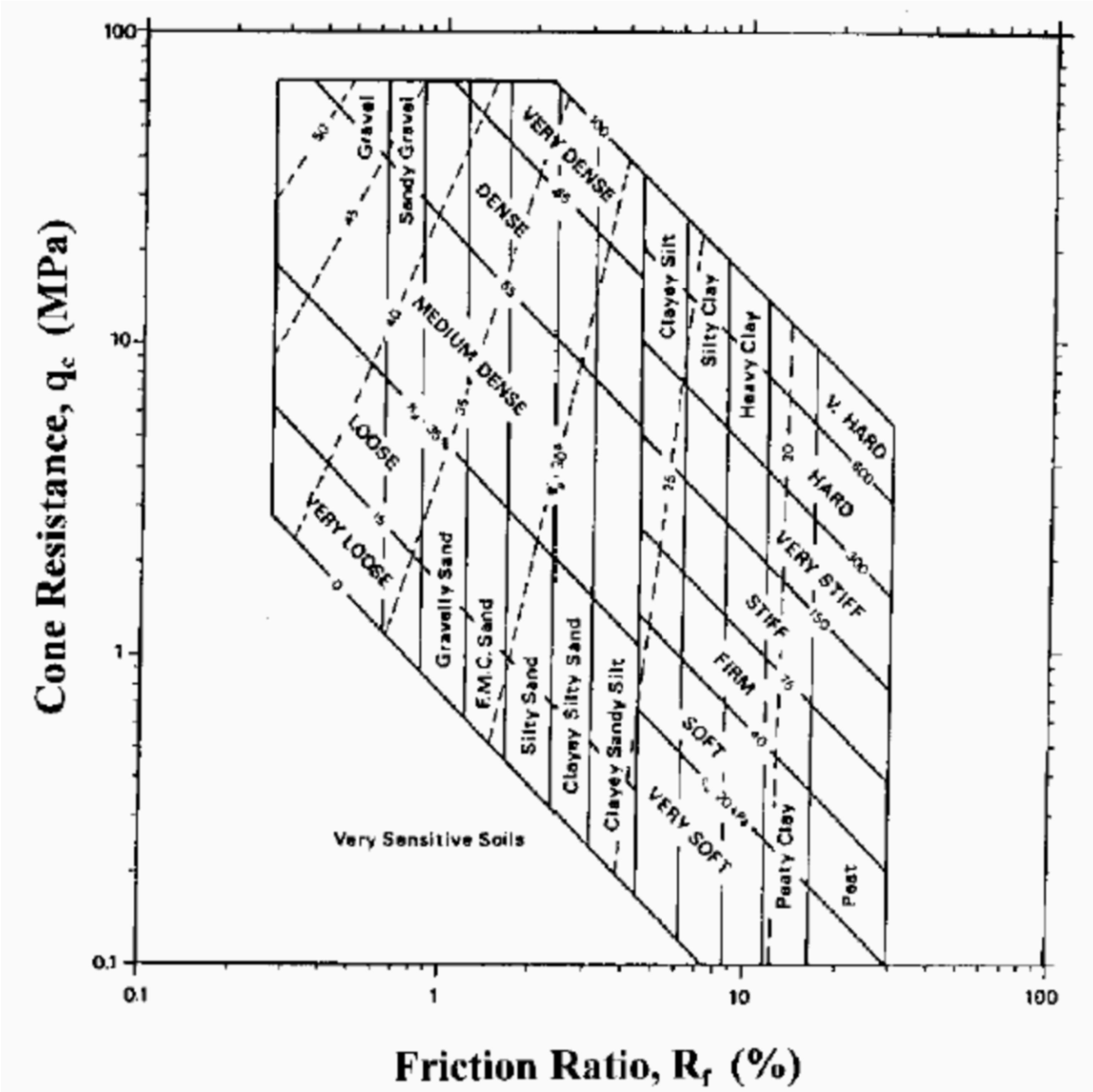


Figure 15. Soil classification chart (Searle, 1979)

d) DOUGLAS and OLSEN (1981) developed a chart for cone penetration tests with electrical tip .

For CPTu the mostly used classification charts are:

a) ROBERTSON et al. (1986): Robertson and Campanella set up two classification charts using the parameter (qt) for the y-axis, but two different parameters for the x-axis (R_f e B_q) (Fig.16). q_t is the total cone resistance corrected on the basis of the u measured during penetration and the ratio of the shoulder area (A_n) unaffected by the pore water pressure divided by the total shoulder area (A_c).

$q_t = q_c + u_2*(1-A_n/A_c)$

$R_f = 100*f_s\ q_t$

B_q, the pore pressure ratio

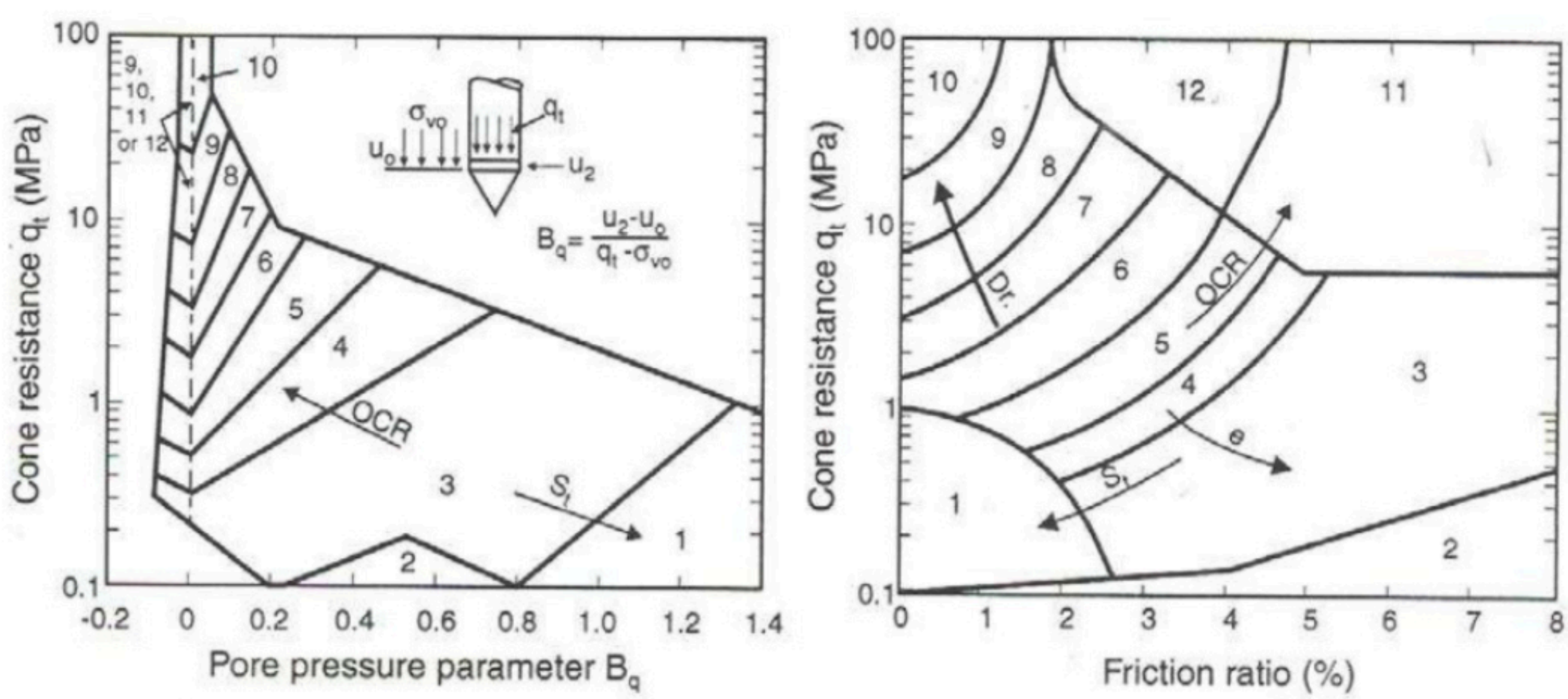
$B_q = (u_2 – u_0) / (q_t – \sigma_{v0})$

Where:

u_2 = pore pressure measured with a porous filter placed immediately after the base of the cone during penetration

u_0= idrostatic pore pressure
q_t= cone resistance corrected depending on u value

σ_{v0} = total vertical geostatic stress



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

Figure 16. Soil classification chart ROBERTSON et al. (1986).

The authors suggest using both graphs, because the influential factors are numerous. Obviously the use of both charts can lead to different indications. In such circumstances it is necessary to refer to the expertise and judgement of the operator.
Just as an example: if during the test we get the following values: q_t=1MPa; R_f = 4%; B_q = 0.1 the soil being examined might be classified

clay on the q_t/R_f chart and clayey silt on the other. It is possible to resolve the doubts by measuring the dissipation. If dissipation is somewhat rapid (t_{50}<60s), the soil belongs to the second category.

The chart by Robertson et al. (1986) has 12 soil types (SBT) and it could be used in real-time to evaluate soil type during and immediately after the CPTu, since it only requires the basic CPTU measurements. Robertson (2009, 2010) provides an update of the chart in terms of dimensionless cone resistance, (q_c/pa), where pa=atmospheric pressure (pa = 1 bar = 100 kPa = 0.1 MPa) and R_f (in percent), both on log scales to expand the portion where R_f < 1%. The number of soil behaviour types has also been reduced to 9 to match the Robertson (1990) chart (Fig.17).

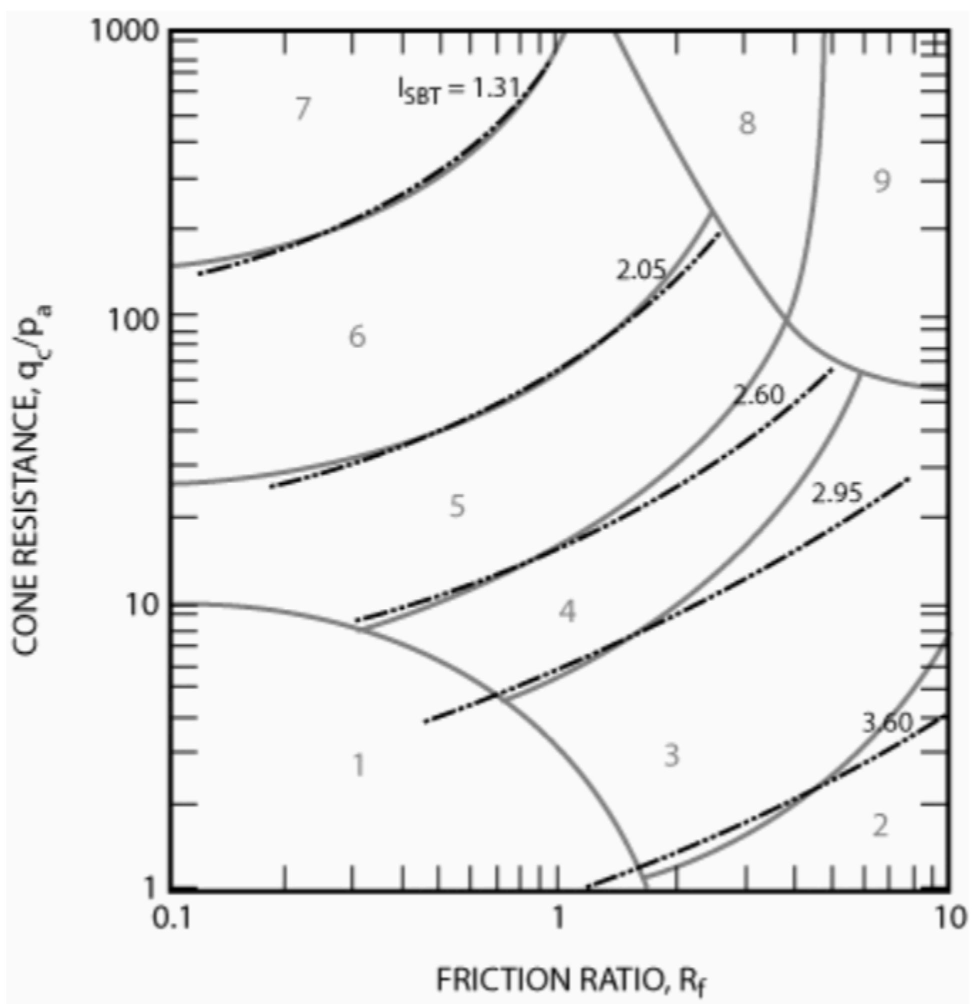


Figure 17. Zone Soil Behaviour Type (SBT): 1 Sensitive fine-grained; 2 Clay - organic soil; 3 Clays: clay to silty clay; 4 Silt mixtures: clayey silt & silty clay; 5 Sand mixtures: silty sand to sandy silt; 6 Sands: clean sands to silty sands; 7 Dense sand to gravelly sand; 8 Stiff sand to clayey sand*; 9 Stiff fine-grained*; * Overconsolidated or cemented (Robertson, 2009, 2010).

b) ROBERTSON (1990): the author has introduced two new parameters, to take into account the influence that the lithostatic pressure may exert at great depths (Fig.18).

Q_t normalized = $(q_t - \sigma_{vo}) / \sigma'_{vo}$; value used for the ordinates of both graphs

R_f normalized = $f_s / (q_t - \sigma_{vo})$

B_q normalized $B_q = \Delta U / (qt - \sigma_{vo})$

Where

$\Delta U = U_2 - U_0$

The chart has 9 soil types (SBTn) and it is only applicable where the lithostatic stress is very

high, such as to significantly modify qc. For this reason the authors suggest the use of the above chart for depths of more than 30 m from ground level. The normalization of the parameters requires also some input of soil unit weight and groundwater conditions (use of the chart during post- processing).

Jefferies and Davies (1993) identified that a Soils Behaviour Tipe Index Ic could represent the SBTn zones in the Robertson (1990) chart where Ic is the radius of concentric circles that define the boundary of soil type. The authors suggest that the SBT Index Ic could also be used to modify empirical correlations that vary with soil type.

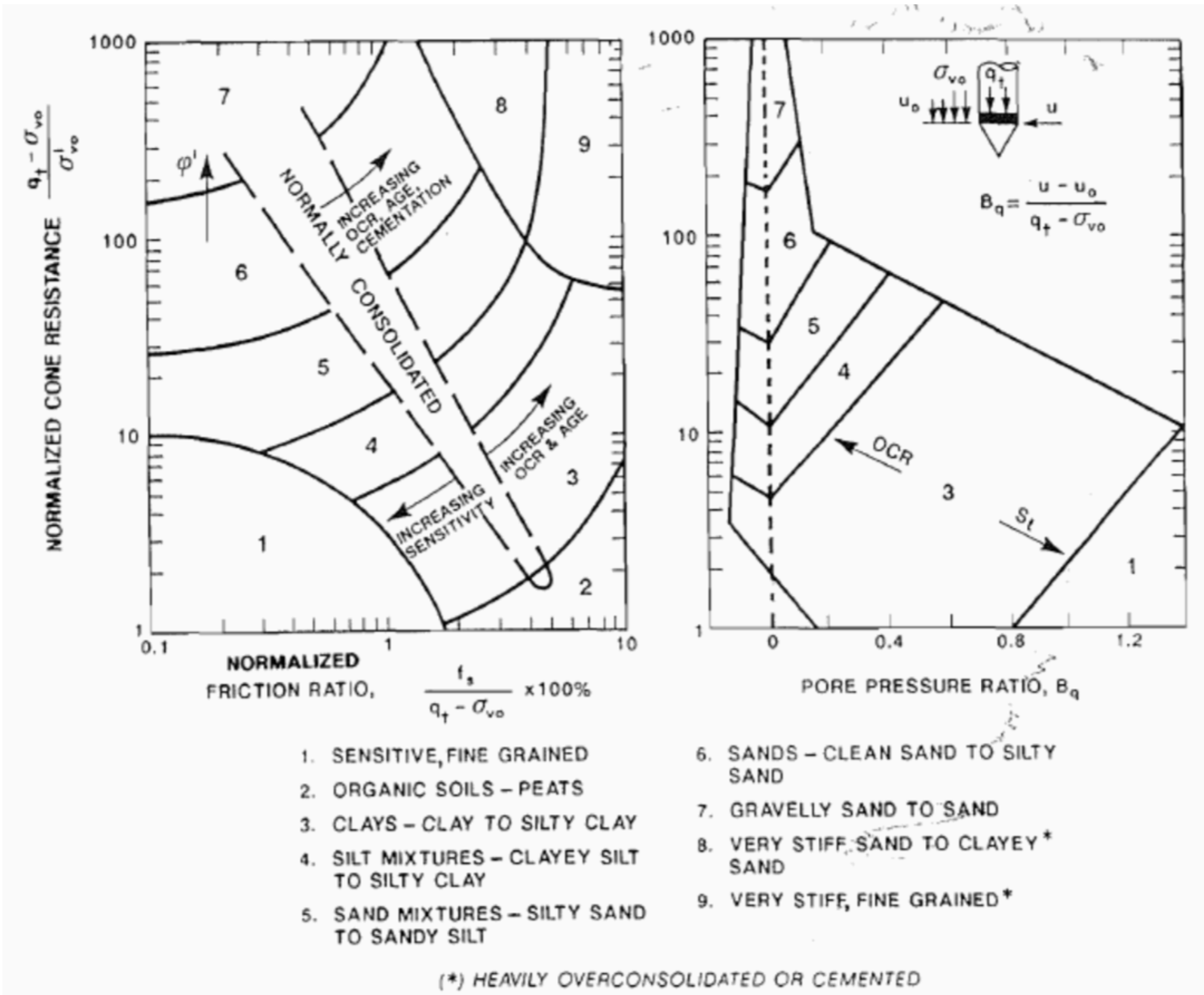


Figure 18. Soil classification chart (ROBERTSON et al. (1990)

c) ESLAMI e FELLENIUS (1997): the classification chart set up by Eslami and Fellenius is based on a database containing CPT and CPTu, associated to laboratory test for soils taken from 20 sites from various parts of the world (Fig.19).
The database does not include cases of cemented soils or very stiff clays and a consequence these lithotypes are not reported on the chart. The x-axis gives f_s, whereas on the y-axis we have a new parameter q_e (effective cone resistance = (q_t-u_2). In dense sandy soils q_e only differs marginally from qt; whereas in the case of the fined grained soils q_t and q_e could assume very different values. The authors divided the classification chart into a series of fields, corresponding to the various lithotypes of the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1985).

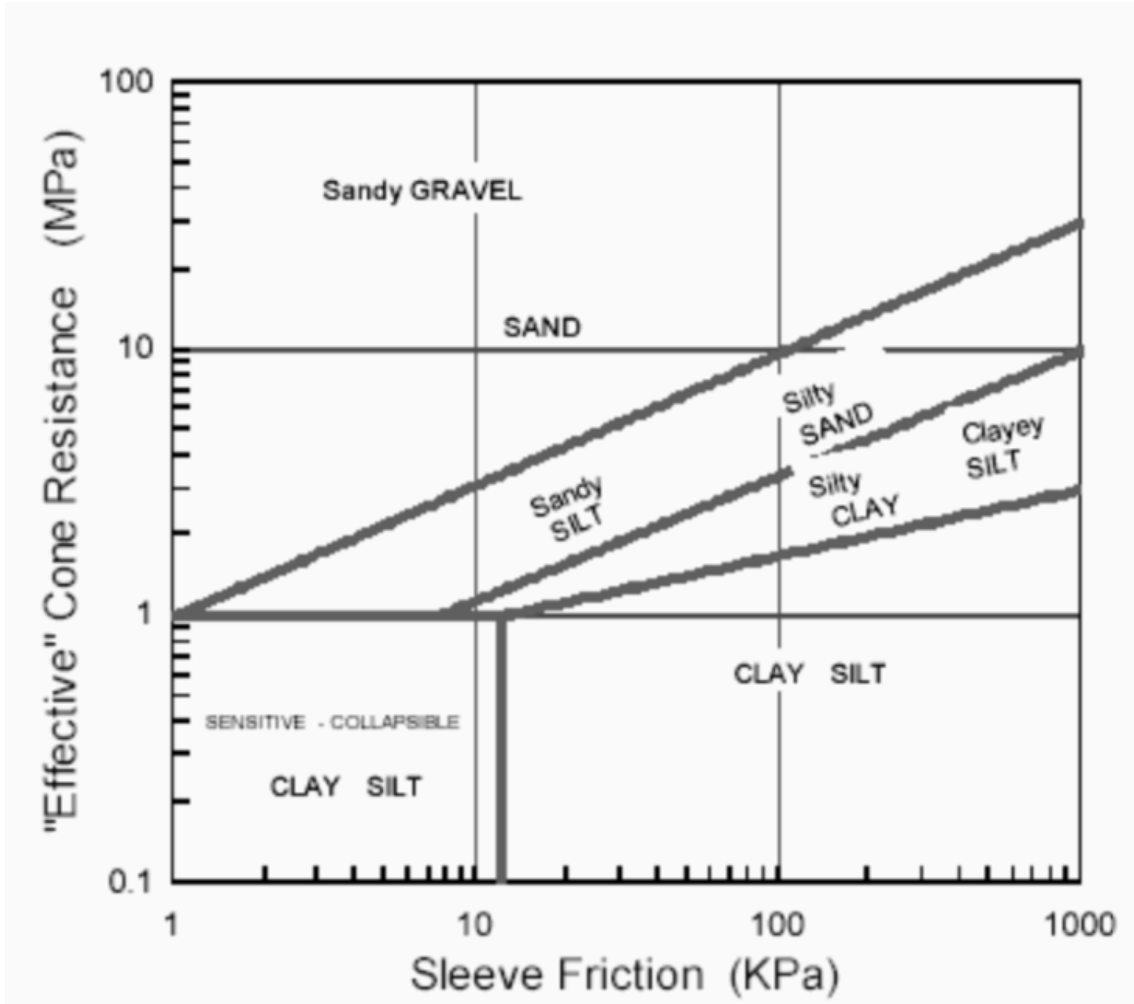


Figure 19. Soil classification chart (Eslami e Fellenius, 1997)

When using the classification charts there are however some important aspects to be considered (Lo Presti et al, 2009):

- The correlations were established on soils coming from geological contexts that might be different than the soils being examined. The geological-geotechnical conditions (lithotype, degree of alteration, cementation, consolidation, etc..) of soil used to find the correlations should be carefully analysed to verify their applicability to the soil studied.
- The soil classification boundaries, defining soil classification zones, were largely subjectively determined (Cai et al., 2011).
- The classifications methods have some limits: the application of Begemann (1965) classification chart is difficult for values where q_c<5 MPa and f_s <50 kPa, in that the lines that distinguish one class from another end up very close; the Schmertmann method (1978) is not so accurate for low qc values; with the Robertson (1990) method the normalisation of cone resistance and the sleeve friction subject to the overburden stress tends, at shallow depths, to overestimate the grain size of the soil.
- The type of penetrometer (mechanical or electrical tip or piezocone) used is also an important factor (Cestari, 1990). Reduction of the

diameter of the tip above the cone, in the mechanical penetrometer, gives (especially relevant in very dense sands) lower q_c measurements than ones obtained with an electrical bit. On the contrary, the soil friction along the protective sleeve above the cone is responsible for a greater q_c than the one measured with the electrical bit (especially relevant in loose sand and soft clay). In the case of Begemann cone with sleeve not only do we measure the friction but, because of the union at the lower end of the sleeve, also a part of resistance at the base (return flow material after the cone has passed). For this reason f_s measured with the mechanical bit is always greater than the one measured with the electrical bit (the difference is practically negligible for clay).

- The CPT and CPTu-based charts were predictive of soil behaviour type (SBT), since the cone responds to the in-situ mechanical behaviour of the soil and not directly to soil classification criteria based on grain-size distribution and soil plasticity (Robertson, 2009) (e.g. Unified Soil Classification System, USCS). Factors such as stress history, in situ stresses, macro fabric, void ration and water content will also influence CPT/CPTu response and resulting SBT. The USCS classification system is also based on remolded soil conditions rather than in situ conditions. Fortunately, soil classification criteria based on grain-size distribution and plasticity often relate reasonably well to in-situ soil behaviour and hence, there is often good agreement between USCS-based classification and CPTu- based SBT, except for mixed soils (i.e. sand mixtures and silt mixtures)

- The classification charts are also shown to be sensitive to penetration rate, they are not appropriate for penetration rates other than the penetration rates they are created for (Jaeger et al., 2010).

A research was carried out by Lo Presti et al. (2009) in order to verify the application of CPT and CPTu for soil profiling, i.e. the identification of the lithotype and the stratigraphic boundaries.
CPT, CPTu data, approximately 6-23 m deep, from 11 different Italian sites, belonging to different geological contexts (lacustrine organic soils, very heterogeneous alluvial lacustrine soils, terraced alluvial soils, recent alluvial soils, alluvial fan soils, estuarine - marine soils) were collected from published reports or obtained from tests carried out with a Pagani penetrometer (TG 63-100, TG 63-200, TG 73-200) (Pagani, 2009). The test equipment consists of 60° cone (piezocone for CPTu and Begemann mechanical cone for CPTm), with a 10 cm^2 base area and a

150 cm^2 friction sleeve located above the cone. The filter position for pore pressure measurements is behind the cone tip (u_2). CPTu were carried out at constant speed of 2 cm/s. The pushing equipment consists of hydraulic jacking and reaction system mounted on a heavy lorry with screw anchors. The thrust capacity is of 100 to 200 kN. The field data acquisition system includes analogue to digital converters. The piezocone provides values of cone resistance, sleeve friction and pore pressure every 1 cm.
Soil profiles have been established through borehole-logs. In addition, laboratory investigation helped in the geotechnical characterization of the soils. Laboratory tests included classification tests, oedometer, triaxial and direct shear tests. In some sites, penetration tests were repeated in different periods of the year (dry and wet period) and with the use of different fluids for the filter saturation (silicon oils and glycerin).
A percentage of success was calculated as the ratio between the numbers of interval correctly classified in a soil category/total number of intervals of the soil category.
From the analysis the following main conclusions were drawn (Lo Presti et al., 2009):

- the success rates are different in relation to the different classification charts;
- CPT interpretation charts (Begemann, Schmertmann and Searle) usually identify peaty soils (78% of rate of success) and homogeneous saturated deposits but they show unsatisfying results for mixed silty soils (silts, clayey and sandy silts and fine sands with silt) (0-28%) and for soils made up of very different grain size (e.g. gravelly clay);
- For the CPTs, the Begemann method and in particular the Schmertmann method gave good success rates in the case of soft clays, organic clays or sands. The Searle method has lower success rate. However, the lithotypes are in general classified as “adjacent” or similar and so the misinterpretation observed for such a method are, in practice, acceptable. The interesting aspect of the Searle method is that it is based on a significantly greater number of classes. All the considered methods correctly identified the stratigraphic boundaries.
- CPTu gave a better estimation of the soil profile with respect to CPT. For some interpretation methods, data filtering greatly enhanced the

ability to accurately predict soil profile. In some case it seems that there are problems with detecting thin layers even when using CPTU. All the considered methods correctly identify the stratigraphic boundaries. the Begemann method and in particular the Schmertmann method gave good success rates in the case of soft clays, organic clays or sands.

- Robertson et al. (1986) chart correctly identify 100% of organic soils, clays and sands, whereas most of intermediate soils (such as clayey silt and sandy silt) are not recognized, with percentages of success that range from 50% to 0%;
- Robertson chart (1990) shows results comparable to the previous chart;
- Eslami and Fellenius chart (1997) does not present high rates of success for mixed soils, while clay and sand shows satisfying results (rate of success up to 100%).
- The presence of a shallow partially saturated crust (especially in the case of fine - grained soils) led to over estimation of the soil grain size. Such misinterpretation is emphasized when using the Robertson (1990) method.
- The results underline that the considered interpretation methodologies depend very closely on the geological conditions of the soils, on which these classifications were established, and hence cannot be regarded as totally reliable. Moreover penetration tests always need a calibration by means of stratigraphic logs from boreholes.
- The stratigraphic logging and classification based on CPT and CPTu data requires knowledge about the geological history and soil genesis to allow for a proper interpretation. Nevertheless, the CPT and CPTu can be used with confidence when supported by all the other tests and information at our disposal from the site investigation.
- CPT/CPTu tests can be used for subsurface stratigraphic correlations and they can significantly help in the identification of engineering geological units and in the construction of the engineering geological model of a site. They can define local situations which require detailed studies.

The use of CPT and CPTu for the identification of lithotypes and stratigraphical boundaries is sometimes complicated by several constraints, namely:

- the minimum layer thickness that can be detected by penetration resistance,
- the presence of partially saturated soils,
- the presence of soils made up of different grain size (e.g. gravelly clay),
- the presence of mixed soils (i.e. sand mixtures, silt mixtures)
- the repeatability of the tests in different climatic conditions.

It is widely accepted that the q_c measurements represent the local soil response near the tip.
However we still have the problem about which portion of soil influences these measurements, in order to be able to establish the minimum layer thickness that can be detected by the penetration resistance. On the basis of numerical analyses and calibration chamber tests Vreugdenhil et al. (1994), Ahmadi and Robertson (2005) have tried to answer the above question. The mentioned works unanimously indicate that such a thickness depends on the relative stiffness of two contiguous layers. In particular, the penetration resistance of a soft layer (clay) below a rigid layer (dense sand) is fully mobilized even for thicknesses of 1-2 diameters, whereas a thickness of 10-20 diameters is needed to fully mobilize the resistance of a rigid layer underneath a soft one.

One of limitations of CPTu in fine grained soils containing granular inclusions, such as coarse- gravel and cobbles, is that these inclusions can distort the soil interpretation by causing sharp reductions in pore-water pressure that temporarily impair the performance of the cone sensor, when the cone sensor is located on the cone shoulder. Furthermore, the classes of soils proposed by the various authors indicate a gradual transition from fine to coarse - grained soils. Soil made up of very different grain size (e.g. gravelly clay) can not be interpreted correctly.

Another constraint is the difficulty to apply classification charts in partially saturated soils (especially fine soils) because of the soil suction which modify the effective stress state. Application of the classification charts under these conditions leads generally to an overestimate of the soil grain size (Lo Presti et al. 2009).

The CPT and CPTu test typically shear fine-grained materials in an undrained manner and coarse- grained materials in a drained manner. No current method to estimate drained or undrained material properties from CPT/CPTu can be applied reliably to intermediate soils whose response to conventional CPT/CPTu in partially drained (Jaeger et al, 2010). The success rates are good for saturated homogeneous soils, particularly for soft clay or organic soils. For silty clays or soft silty sands the classification charts mis-classify the soil type. Intermediate soils tend to be much more difficult to differentiate (Ramsey, 2010; Lo Presti et al., 2010).

Another problem is the repeatability of CPT and CPTu tests. Q_c and f_s depend on the in situ conditions, which are related to the climatic conditions of the period when the tests are carried out. An example is represented by the CPTU carried out in different periods in the alluvial fan of the Scuropasso Stream (an Apennine right tributary of the River Po), in the province of Pavia (Northern Italy). The soil is made up of silty clays and clayey silts, which are very heterogeneous (CH, CL) until a depth of 19 m, overhanging sandy deposits containing a semi-confined aquifer (Fig.20). A perched water table is present at a depth of about 1.8 m. The CPTu tests were repeated, in the same site, in different periods of the year i.e. wet period (13/06/2001) and dry period (28/09/2001); in both cases the piezocone was saturated with grease.

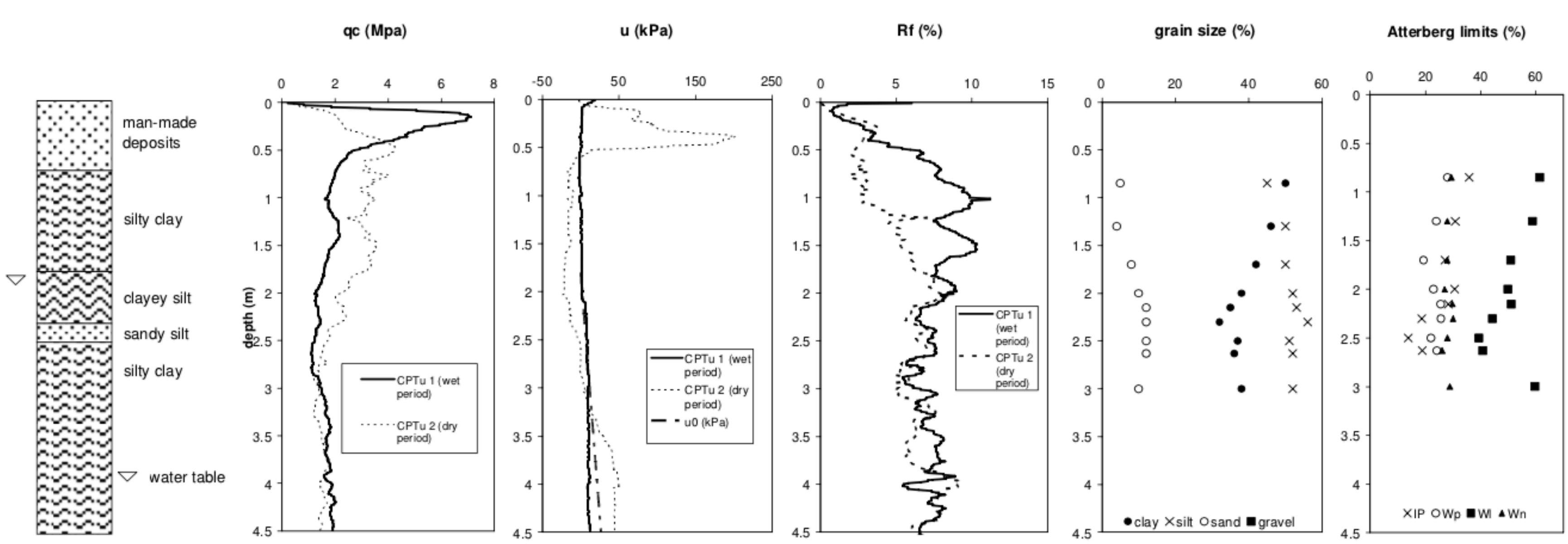


Figure 20. Scuropasso Stream alluvial fan. Soil profile and geotechnical characteristics. qc: cone resistance; u_0: in-situ pore pressure; u_2: pore pressure measured at cone base; fs: sleeve friction; Rf: friction ratio (fs/qc*100) IP: plastic index; Wp: plastic limit; Wl: liquid limit.

From the results obtained from the CPTu tests we can observe that qc reaches values that are close to 7 MPa (wet period) and 4 MPa (dry period). At greater depths it decreases maintaining different values in the two tests until about 2.8 m. From this point on the values of qc are similar (2 MPa).

The higher values of q_c in the man-made deposits are probably due to the presence of coarse- grained material, usually met during the CPTu1 test. The higher penetrometric resistance values observed in the dry period down to a depth of almost 3.0 m, are however to be attributed to higher values of the effective stresses as an effect of the partial saturation in the dry period. The different trend of q_c in the two periods, also confirmed by the Rf friction ratio, seems to show the thickness of the soil, which is sensitive to the variations of moisture content as a result of the climate ("active zone"). As for the pore pressure u_2, considering CPTu1 test (wet period) we can observe reduced values close to ground level ($u < 25$ kPa) and a very contained increase in depth. On the other hand in the CPTu2 test (dry period) there are negative values until 2.5 m (probably linked to the partial saturation) and an increase in u at a greater depth. Anyway, the saturation with grease of the filter does not seem to give good measurements of pore pressure in most cases.

The above considerations have the obvious consequence of overestimating soil grading when using data carried out during dry periods.

More recently probabilistic soil classification methods to assess the percentages of clay, silt, and sand have been developed. In these methods, based on statistical approaches, the uncertainties are attributed to both the soil mechanical behavior and the soil composition. Zhang and Tumay (1999) explored the accuracy of CPT classification through the use of conformal mapping of two independent indices: soil classification index and soil in situ state index; a fuzzy subset approach was later introduced. The method is termed "P-Class" and uses the cone tip resistance and sleeve friction to evaluate probability of soil type. It is fully automated by computer software and available as a free download from the Louisiana Transportation Research Center (LTRC) website (http://www.coe/ su.edu/cpt/).

Kurup and Griffin (2006) explored the capabilities of regression-based artificial neural network (ANN) model in predicting soil composition from CPT data.

5.2. Soil parameter evaluation

Soils are very complex materials because they can be comprised of a wide and diverse assemblage of different particle sizes, mineralogies, packing arrangements, and fabric. Moreover, they can be created from various geologic origins (marine, lacustrine, glacial, residual, aeolian, deltaic, alluvial, estuarine, fluvial, biochemical, etc.) that have undergone long periods of environmental, seasonal, hydrological, and thermal processes. These facets have imparted complexities of soil behavior that relate to their initial geostatic stress state, natural prestressing, nonlinear stress–strain–strength response, and drainage and flow characteristics, as well as rheological and time–rate effects. As such, a rather large number of different geotechnical parameters have been identified to quantify soil behavior in engineering terms.

Selected relationships utilized in the strength parameters derived from the CPT and CPTu are presented in the subsequent subsections.

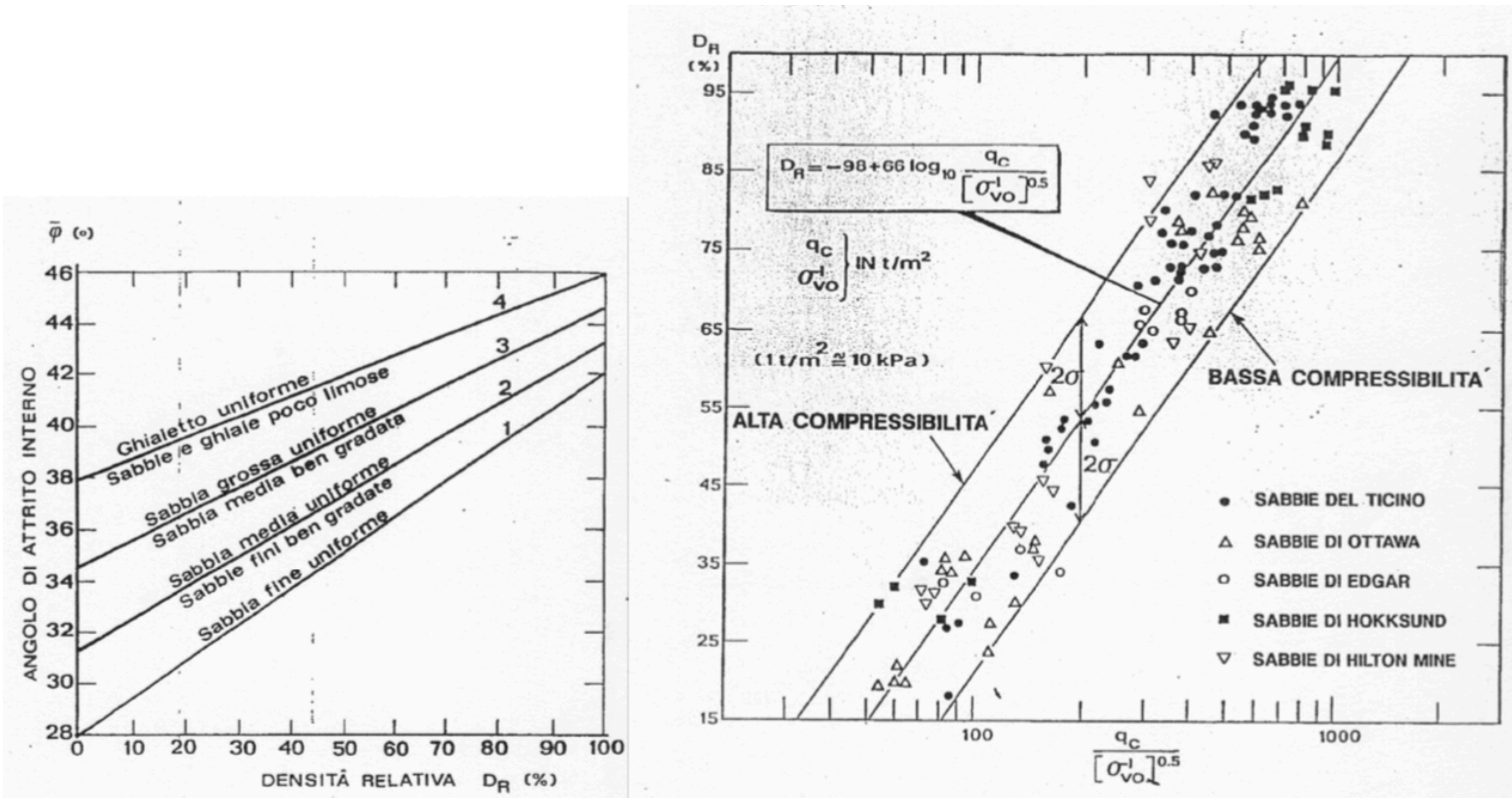
5.2.1.Strength Characteristics

5.2.1.1. Sands

Cone penetration testing in coarse-grained soils, such a sandy soils, is generally drained. Under drained conditions there should be no excess pore pressures generated as a result of cone penetration, that is, the in situ static pore pressure is measured.
The strength of soils is controlled by the effective stress frictional envelope, often represented in terms of the Mohr– Coulomb parameters: φ’ = effective friction angle and c’ = effective cohesion intercept. Numerous methods for assessing φ’ from cone resistance have been published. Basically the methods fall into one of the following categories:

- Empirical or semi-empirical correlations (Kulhawy and Mayne, 1990; Jamiolkowski et al. 2001: Mayne, 2007)
- Bearing capacity theory (Robertson and Campanella, 1983)
- Cavity expansion theory
- Laboratory testing on undisturbed sand/gravel samples (see as an example Wride and Robertson 1999, 2000; Mimura, 2003; Lunne et al., 2003; Lee et al., 1999; Ghionna and Porcino, 2006)

Anyway, the most consolidated approach consists of firstly inferring the relative density of the granular soil deposit from the penetration resistance (Jamiolkowski et al., 1985) and after that to determine the peak angle of shear resistance from the obtained relative density according to Schmertmann (1978) (Fig.21).



1. $\varphi' = 28 + 0.14 \cdot D_R$ (uniform fine sand)
2. $\varphi' = 31.5 + 0.115 \cdot D_R$ (well graded fine sand – uniform medium sand)
3. $\varphi' = 34.5 + 0.10 \cdot D_R$ (well graded medium sand – corse uniform sand)
4. $\varphi' = 38 + 0.08 \cdot D_R$ (uniform fine gravel – silty sand and gravel)

Figure 21. Relative density and angle of shear resistance of granular soil deposit from penetration resistance

5.2.1.2.Undrained shear strength of clays

For geotechnical applications involving short-term loading of clays and clayey silts, the undrained shear strength ($su=c_u$) of the soil (formerly termed c = cohesion) is commonly sought for stability and BC analyses. The classical approach to evaluating su from CPT readings is through the net cone resistance:

$su = (q_t - \sigma_{vo})/N_{kt}$

where N_{kt} is a bearing factor.

More papers and research programs have focused on the assessment of relevant value of N_{kt} for an interpretation of su than for any other single parameter (e.g., Keaveny and Mitchell, 1986; Konrad and Law, 1987), without any consensus reached. This is because, in part, the value of su is not unique, but depends on the direction of loading, strain rate, boundary conditions, stress level, sample disturbance effects, and other factors (Ladd, 1991). Indeed, a suite of different undrained shear strengths are available for a given clay soil. For the basic laboratory shear modes, there are many available apparatuses, including CIUC, PSC, CK0UC, direct shear simple (DSS), DS, PSE, CK0UE, UU, UC, as well as hollow cylinder, true triaxial, and torsional shear (Jamiolkowski et al., 1985; Kulhawy and Mayne, 1990).

Anyway it is generally accepted to select the values of N_{kt} in the range below reported. On the other hand the use of CPT in fissured clays is strongly discouraged.

Soft clay : $N_{kt} = 14 \pm 4$
Overconsolidated clay : $N_{kt} = 17 \pm 5$
Fissured clay : $N_{kt} = 10 \div 30$

In lieu of the classical approach, an alternate and rational approach can be presented that focuses on the assessment of (σp’) from the CPT. The magnitude of preconsolidation stress (σp’) is uniquely defined as the yield point from the e-logσv’ plot obtained from a consolidation test. The influence of OCR in governing the undrained shear strength of clays is very well established (e.g., Trak et al. 1980; Leroueil and Hight, 2003).

From considerations of critical state soil mechanics(CSSM), this simple shear mode can be expressed in normalized form (Mayne, 2007):

$su/\sigma_{vo'DDS} = 1/2 \sin\varphi' OCR^{\Lambda}$
where

$\Lambda = 1 - Cs/Cc$ = plastic volumetric strain potential,

Cs = swelling index

Cc = virgin compression index of the material.

For many clays of low to medium sensitivity, $0,7 <= \Lambda <= 0,8$, whereas for sensitive and structured clays, a higher range between $0,9 <= \Lambda <= 1,0$ can be observed.

5.3. General factor affecting interpretation

Before analyzing any electric CPT data, it is important to realize and account for the potential errors that each element of data may contain (equipment design, in situ stresses, compressibility, cementation and particle size, stratigraphy, rate of penetration, pore pressure element location; Lunne et al., 1997)

5.3.1.Equipment design

The three major areas of cone design that influence interpretation are:

1. Unequal area effects.

2. Piezometer location, size and saturation.

3. Accuracy of measurements.

The errors associated with equipment design are usually most significant for penetration in soft, normally consolidated, fine-grained soils. Test results in sand are little influenced by the above factors, except possibly variations in friction sleeve stress f_s.

5.3.2. In situ stresses

Theoretical models and chamber test studies have shown that the in situ horizontal effective stress, σ'_{ho}, has a dominant effect on the cone resistance, and the friction sleeve stress. Therefore, the stress (geologic) history of the deposit is of great importance in CPT interpretation. Unfortunately, there is often only qualitative data concerning geological history and the techniques for measuring in situ horizontal stresses are still not very reliable, especially for sands.

An excavation will reduce the horizontal stress in adjacent soils. Even an open borehole, if closer than 10-20 hole diameters may reduce the horizontal stress, depending on the soil conditions. Both static and vibratory compaction or the installation of piles can change the horizontal stress.
Applied surface loads (such as from surface fill or CPT equipment) can also increase the effective stress. The interpretation of CPT data should, at least qualitatively, account for such effects that may influence the horizontal stress.

5.3.3.Compressibility, cementation and particle size

The compressibility of soils can significantly influence q_c and f_s. Highly compressible sands tend to have low cone resistance and in some cases, high friction ratio values.
Some carbonate sands have friction ratios as high as 3%, whereas typical incompressible quartz sands have friction ratios of about 0.5%. The compressibility of sand during cone penetration is also influenced by grain crushing.

Cementation between particles reduces compressibility and thereby increases the cone resistance. Cementation is always a possibility in situ and is more likely in older soil deposits. When the particle size of a soil penetrated becomes a significant fraction of the cone diameter, then the cone resistance can increase abruptly because of the decreased compressibility of the soil due to the need for the cone to displace these particles as rigid units. This effect tends to produce sharp peaks in the cone resistance profile when encountering gravel-sized particles.

Intersecting very large particles can abruptly stop penetration or cause a sudden deflection. Penetration through gravelly soils often produces a distinct sound up the cone rods.

5.3.4. Stratigraphy

Even if the CPT measures the correct mechanical characteristics in uniformly soft or strong materials, the transition from one layer to another will not necessarily be registered as a sharp change. Cavity expansion and strain path theories as well as laboratory studies (Schmertmann, 1978; Treadwell, 1976) show that the cone resistance is influenced by the material ahead and behind the penetrating cone. Hence the cone will start to sense a change in material type before it reaches the new material and will continue to sense a material even when it has entered a new material. Therefore, the CPT will not always measure the correct mechanical properties in thinly interbedded materials.

The distance over which the cone senses an interface increases with material stiffness. In soft materials the diameter of the sphere of influence can be as small as two or three cone diameters, whereas in stiff materials the sphere of influence can be up to 10 or 20 cone diameters. Hence, the cone resistance can respond fully (that is, reach full value within the layer) in thin soft layers better than in thin stiff layers. Soft layers thinner than 100 mm can be fully detected by the cone resistance, whereas stiff layers may need to be as thick as 750mm or more for the cone resistance to reach its full value.

The CPT will detect thin stiff layers but the strength of stiff layers could be underestimated if the layer is less than about 750 mm. It is possible to detect the presence of soft layers as thin as 75 to 100 mm using the cone resistance. Therefore, care should be taken when interpreting cone resistance in a thin sand layer located within a soft clay deposit.

5.3.5. Rate of penetration

Rate effects can be caused to some extent by creep and particle crushing. In general, however, the pore pressure effects predominate and are of most interest, especially when using the piezocone in fine-grained soil. Normally a tenfold increase in rate causes 10-20% increase in cone resistance in stiff c1ays and 5-10% in soft c1ays.

5.3.6.Pore pressure element location

Piezocone testing it is recommended to measure the pore pressure just behind the cone (u2) for the following reasons:

- good protection from damage
- easy saturation
- generally good stratigraphic detail
- generally good dissipation data
- correct location to determine qt.

5.4.Direct use of CPT for the design of shallow and deep foundations

5.4.1.Settlement of shallow foundation on granular soils

Settlement computation is the way to verify the Serviceability Limit State (SLS) of any type of foundation. The Schmertmann approach (Schmertmann et al., 1978) is a semi – empirical method based on the comparison between CPT and Plate Load Tests carried out in a Calibration Chamber on reconstituted sand samples. According to Eurocodes and Italian Technical Standards the characteristic value of soil stiffness is considered (i.e. a value not corrected by any safety factor).

As for the serviceability loads, in case of settlements quickly developing, permanent loads are combined with a large fraction (about 70%) of variable loads (rare combination). For long term settlements (secondary settlement) the combination considers a smaller amount of variable loads, about 30% (quasi permanent combination).

The tip resistance profile gives the possibility of evaluating the soil deformability within an elastic approach. Operational Young modulus is obtained as $E = 2{,}5 \cdot q_c$ for circular or square foundations and as $E=3{,}5 \cdot q_c$ for strip foundations $L \geq 10 \cdot B$ (B = foundation width). The influence depth is equal to 2B in case of circular or square foundations and to 4B in case of strip foundations. The compressible layer is then divided into "n" sub – layers and the following expression is used for settlement computation:

$$w = c_1 * c_2 * q * \sum_{i=1}^{n} \frac{I_{z,i}}{E_i} \Delta z_i$$

q = net load (i.e. the total load minus the effective vertical geostatic stress σ'_{v0} at the depth of the foundation base)

Δz_i = thickness of i – layer

Ei = average Young modulus for the i – layer

$I_{z,i}$ = strain factor (see Fig. 22) depends on the foundation shape and has been inferred from the CC experiments. The maximum value of the strain factor is at a depth of B/2 for square or circular foundations and at B for strip foundations $L \geq 10 \cdot B$. The maximum value is computed as $0{,}5 + 0{,}1 \cdot \sqrt{(q/\sigma'_{vp})}$, where σ'_{vp} is the vertical effective geostatic stress at B/2 or B depending on the foundation shape.

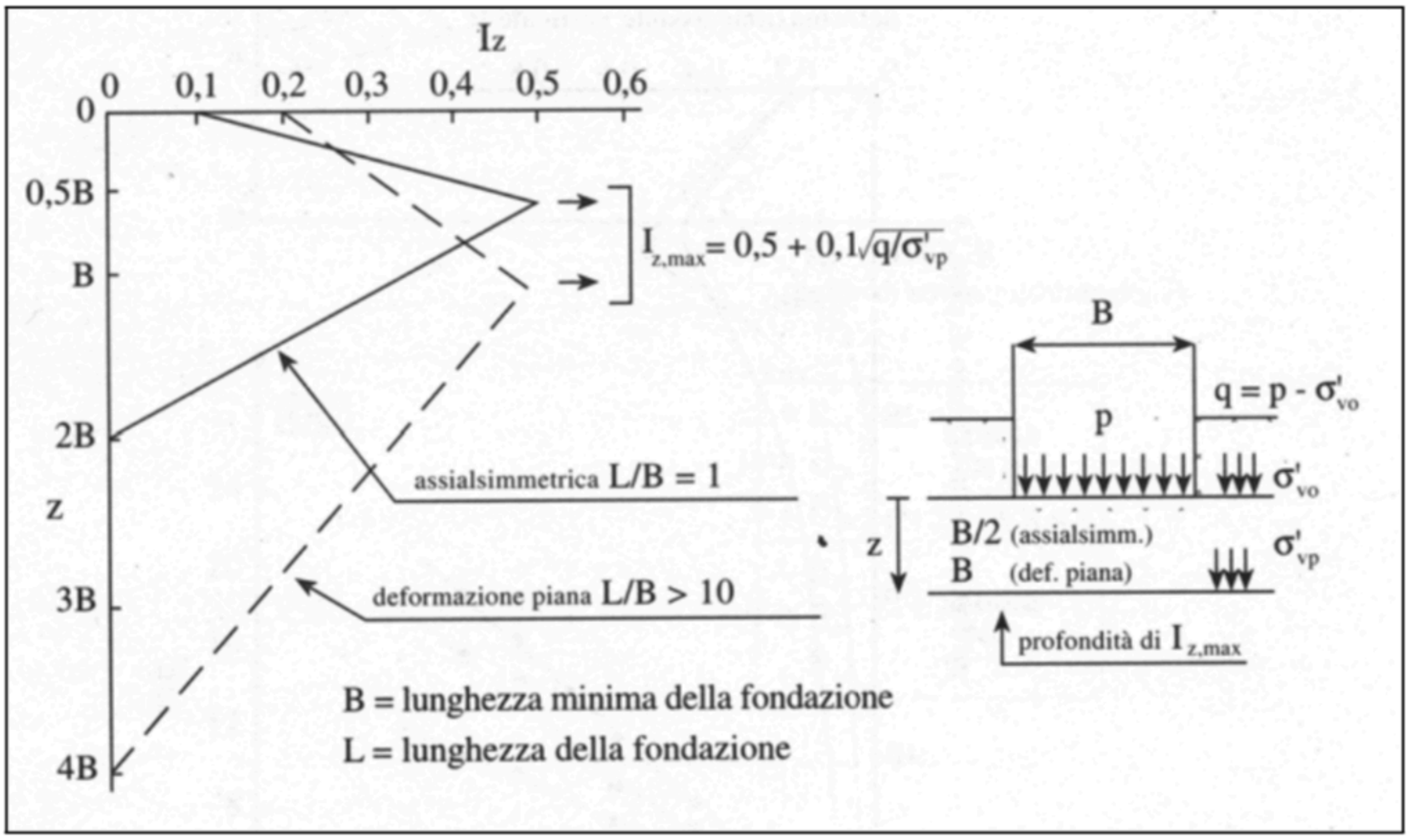


Figure 22. Strain Factor(Schmertmann et al., 1978)

The coefficient c1 takes into account the foundation embeddement. It takes the value of $1\text{-}0,5\cdot(\sigma'_{v0}/q)$ and must be equal or greater than 0,5. The coefficient c_2 takes into account secondary settlement (creep) and is computed as $1 + 0,2\cdot\log(10\cdot t)$, where t is the time expressed in year. The Schmertmann method assumes that consolidation time for shallow foundations on granular soil deposits is generally about 0.1 year. According to Eurocodes and Italian Technical Standard the time t should represent the life time of the construction (considering only ordinary maintenance works).

As already stated, the net pressure for the computation of the long term settlement (creep) can be different than that used for the consolidation settlement on granular deposits.
According to the suggestions given by Schmertmann et al. (1978) and based on the personal experience of the authors of the present handbook, the computed settlement overestimates the real settlement at least by a factor between 2 and 4. Anyway, for fully compensated shallow foundations, the net load is practically equal to zero and therefore, in this case, the computed settlement is negligible.

5.4.2.Bearing capacity of axially loaded piles in granular soils

The best way for assessing the bearing capacity of vertically loaded piles is to use prototype pile load testing. Anyway, CPT can be considered, at a reduced scale and with some other differences, as a steel driven pile. On the other hand the tip resistance and sleeve friction can be considered respectively as the ultimate unit base resistance and the ultimate unit shaft resistance.

The bearing capacity of axially loaded piles is usually computer by means of the following equations:

Bearing capacity (compression): $Q_u^c + W_p = Q_{su} + Q_{bu} \quad [F]$

Bearing capacity (tension): $Q_u^T - W_p = Q_{su} \quad [F]$

where

$Q_{su} = \pi D \int_0^L \tau_{su} dz \quad [F]$ and $Q_{bu} = \frac{\pi D^2}{4} q_{bu} \quad [F]$

q_{bu} = ultimate unit base resistance [FL^{-2}],

τ_{su} = ultimate shaft base resistance [FL^{-2}]

W_p = pile dead weight [F].

The implicit assumptions for the above equations are the following:

- mobilization of base and shaft resistances occurs for the same displacements of the pile head. This is only true in the case of driven piles;
- the pile is cylindrical in shape;
- the same unit shaft resistance is mobilized in compression and tension (there are not enough evidences on this aspect).

Driven piles

In case f driven piles the ultimate unit base resistance can be computed according to Meyerhof (1976):

$q_{bu} = q_t$

while the ultimate unit shaft resistance can be computed according to De Beer (1985):

$\tau_{su} = q_t/200$ when $q_t \geq 20MPa$

$\tau_{su} = q_t/150$ when $q_t \leq 10MPa$

Drilled piles

In case of drilled piles the ultimate unit base resistance is mobilized only for an extremely large displacement of the pile head, therefore the computations rely on a critical value corresponding to a relative displacement (s/D) equal to 0.05. According to Jamiolkowski and Lancellotta (1988) the base resistance can be computed using the following equation:

$q_{bu} = \alpha \cdot q_t$

The parameter α decreases as the pile diameter (D) increases and can be inferred from Fig. 23.

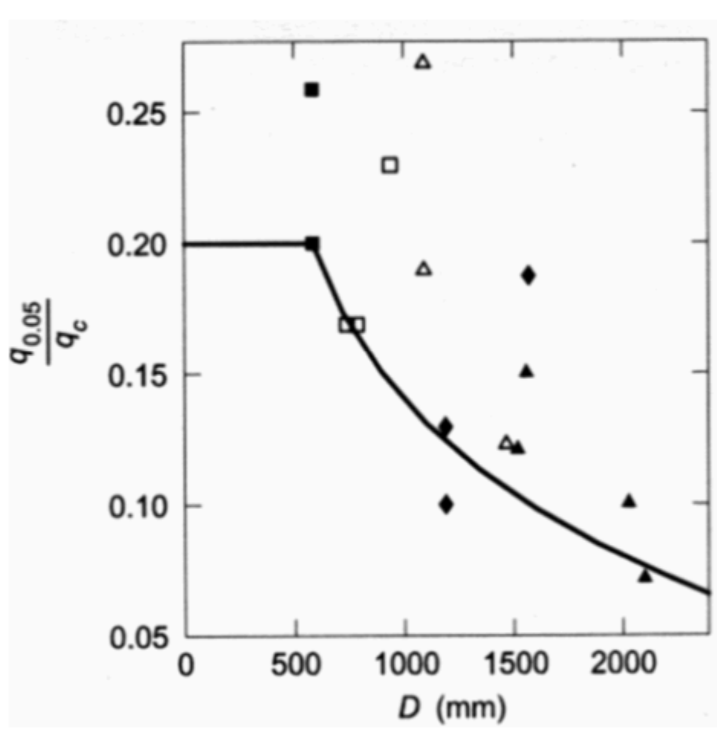


Figure 23. α parameter (Jamiolkowski and Lancellotta, 1988)

The ultimate unit shaft resistance can be computed according to the following equation:

$q_{bu} = \beta \cdot q_t$

Values of the β parameter can be inferred from Fig. 24.

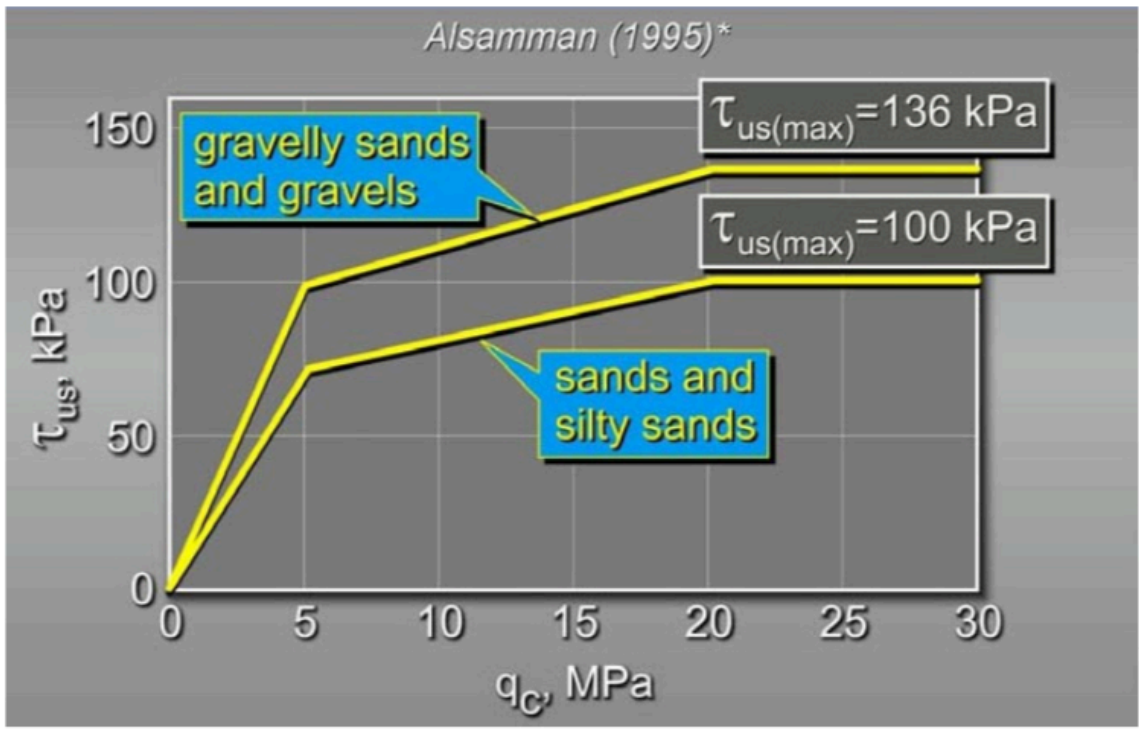


Figure 24. Values of the β parameter Alsamman (1995)

According to Eurocodes and Italian Technical Standards the characteristic value of the base and shaft pile resistance is obtained reducing the values, which have been computed by means of the above equations, by a given factor which in turn depends on the degree knowledge of the subsoil. More specifically, the reduction of the computed values significantly decreases with the number of investigations. In practice, a small increase of the investigation costs can greatly reduce the construction costs.

The design value of the base and shaft resistance is obtained by the application of partial safety factors. The actions, to be compared to the pile bearing capacity in order to evaluate the Ultimate Limit State are amplified by means of appropriate partial safety factors and represent an appropriate combination of permanent and variable loads.

6. Some innovative - research topic

This chapter mainly describes two research activities of the authors of the present handbook (Lo Presti et al, 2010; Squeglia and Lo Presti, 2010).

6.1. Use of CPT and CPTu for soil profiling of "intermediate" soils: a new approach

The problem of misclassification of intermediate soils (silt, clayey silt, sandy silt,...) generally can be solved with dissipation tests (Robertson, 1990) which can significantly aid in identifying soil behavior type, but dissipation tests are often time-consuming.

Lo Presti et al, 2010 proposed a new and faster experimental approach, based and the execution of two contiguous CPTu probes with different penetration rates. The first probe will be carried out at the standard penetration rate (2 cm/s), while the other at a slower speed (1cm/s).

Even though a partially drained penetration will continue to occur at the lower penetration rate, based on the results shown in the previous section we expect that:

- in "intermediate" soils both sleeve friction and tip resistance will increase, while the pore pressure will decrease;
- in "clay" and "sand" the above effects will not appear.

Therefore it will be possible to identify the "intermediate" soils by comparing, at an appropriate scale, the results of the two tests.
In practice it seems more useful to consider the comparisons of sleeve friction and pore pressure than that concerning the tip resistance. In fact, the tip resistance increase is less evident. This is because while the tip resistance increases approaching the drained conditions it decreases with the pore pressure.

CPTu were carried out at two different sites in order to verify the proposed method and more specifically to observe the influence of different penetration rates on the test results in different soil types. Therefore, two contiguous CPTu were carried out at each site using two different penetration rates (2 cm/s and 1 cm/s). The distance in plan between the standard CPTu and that carried out using a reduced penetration rate was about 1 m. A borehole was also available for each site. The distance in plan between CPTu and borehole locations is 4 m.

Figures 25 and 26 compare the stratigraphy inferred from the interpretation of standard CPTu by means of Robertson (1990) chart and that obtained from borehole. The results obtained in each site with standard and reduced penetration rates are compared in Figures 27 and 30.

The site 1 corresponds to alluvial – lacustrine deposits of the Serchio River in Paganico (Lucca Tuscany). The upper layer, of variable thickness, is mainly an alluvial deposit consisting of silty sands or sandy silts and overlying the lacustrine deposit (clay and silty clay) (Fig. 25).

The compared CPTu tests reached a maximum depth of about 6 m.
The interpretation of the standard CPTu probe (Fig. 25) with Robertson chart (1990) correctly identifies "sandy silts", "clayey silts" and "sand and gravel"; only "clayey-sandy silts" (from 2m to 3m) are misclassified.
The site 2 corresponds to the continental-marine sediments of the Livorno coastal plain (Tuscany), which deposited during multiple cycles of sea ingression and regression (Fig. 26).
The compared CPTu tests reached a maximum depth of about 21 m.
The CPTu was interpreted with Robertson chart (1990); we obtain a proper classification of most of the tested soils, and only intermediate soils (5.1-7.5 m and 10-10.5 m) shows unsatisfying results.

It is possible to consider the following working hypotheses:

- for any type of soil and for the considered penetration rates (2 cm/s and 1 cm/s) it is possible to assume partial drainage conditions;
- for the standard penetration rate (2 cm/s) it is possible to simplify assuming "undrained conditions" for clays, "drained conditions" for sands and "partially drained conditions " for intermediate soils;
- the reduced penetration rate should cause a reduction of sleeve friction and tip resistance because of creep effects. Moreover, creep effects could be responsible for an increase of pore pressure;
- the reduced penetration rate should produce an increase of tip resistance and sleeve friction when approaching the drained conditions. For the same reason it is possible to expect a reduction of pore pressure in the case of a reduced penetration rate;
- in conclusion, using a reduced penetration rate, we expect an increase of tip resistance and sleeve friction in intermediate soils, if the effects related to the drainage conditions prevails over creep

effects. We also expect a reduction of pore pressure. Therefore we expect to observe effects of reduced penetration rate in sandy – clayey silts and almost negligible effects in clay and sands.

Figures 27 and 30 show the variation of q_c, f_s and u_2 experimentally observed in the case of CPTu carried out at reduced penetration rate in the two sites. Variations are expressed as percentages and are computed after the application of a moving point average based on 10 values. Some extreme values are probably due to local soil heterogeneities. Some systematic increase or decrease of the measured values enable us to draw preliminary conclusions.

More specifically the slower probe carried out at Paganico shows (Figures 27 and 28):

- from 0 to 2 m a decrease of pore pressure (-138 %) and an increase of tip resistance (30 %) and sleeve friction (40 %), as expected in sandy silts;
- from 2 to 4.5 m negligible variations of resistances (7-9 %) and pore pressures (3%), probably related to the presence of clayey-silty soils (Fig.25);
- from 4.5 to the end of the probe strong differences of q_c, f_s and u_2 between the standard probe and the slower probe. Those changes are not due to the different penetration rate but to local lithological heterogeneities.

In the Livorno site the comparison between the two penetrometric tests highlights the following intervals (Figures 29 and 30):

- from 5 to 10 m (borehole log: intermediate soils) there is an increase of tip resistances (43%) and sleeve frictions (46%) whereas pore pressures decrease (-129%);
- from 10 to 15 m (borehole log: clayey soils) the variation of q_c (2%), f_s (-4%) and u2 (18%) seems negligible, as expected;
- from 15 m to the end of the probe (borehole log: sandy soils) the differences between probes are probably due to local lithological differences.

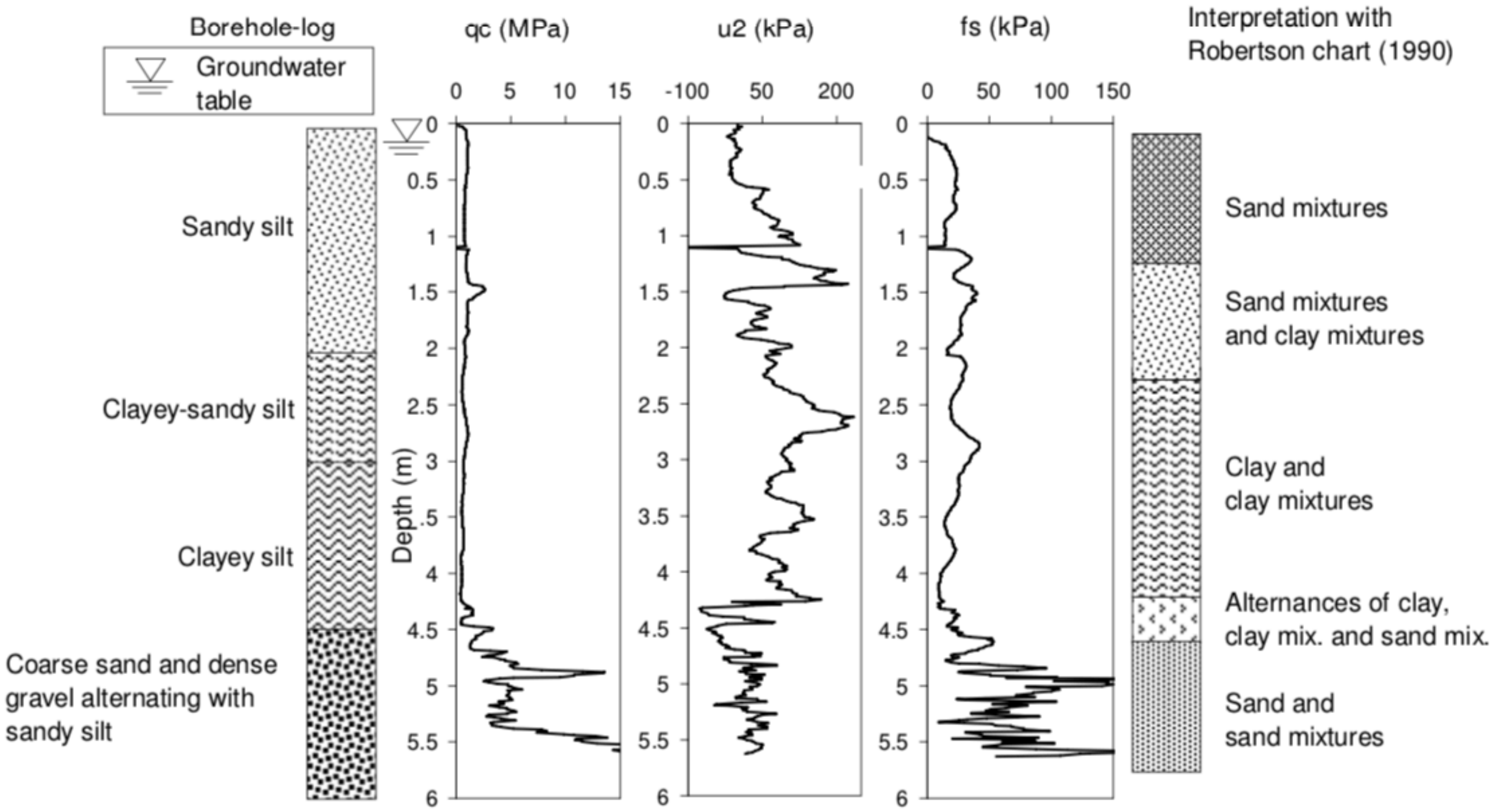


Figure 25. Site 1 (Paganico). Standard CPTu (2 cm/s) compared to a near borehole-log.

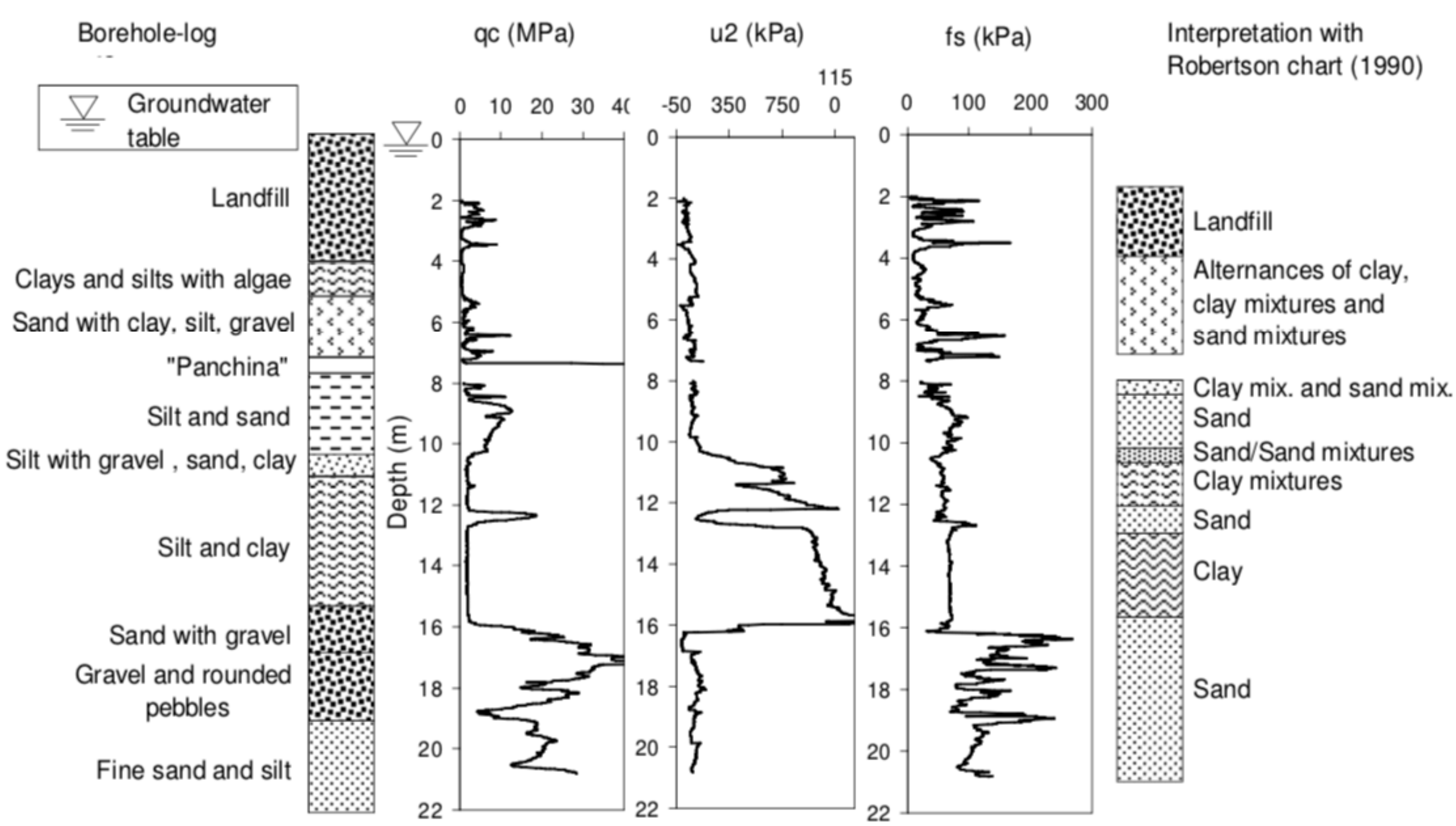


Figure 26. Site 2 (Livorno). Comparison between the CPTu carried out with the standard penetration rate (2 cm/s) and a nearby borehole-log.

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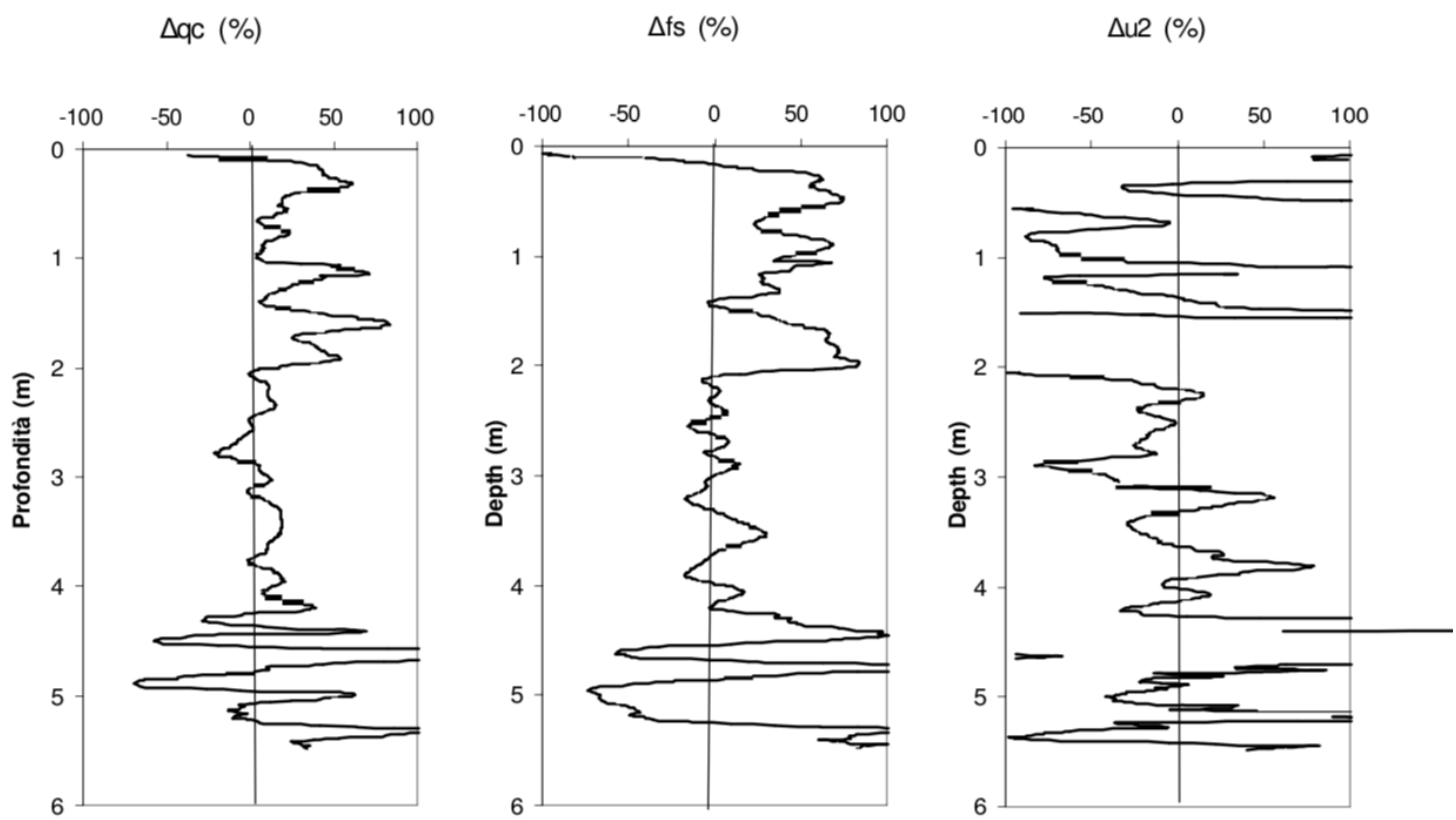

Figure 27. Site 1 (Paganico). Variations of q_c, f_s and u_2 between 2 cm/s and 1 cm/s expressed as percentages

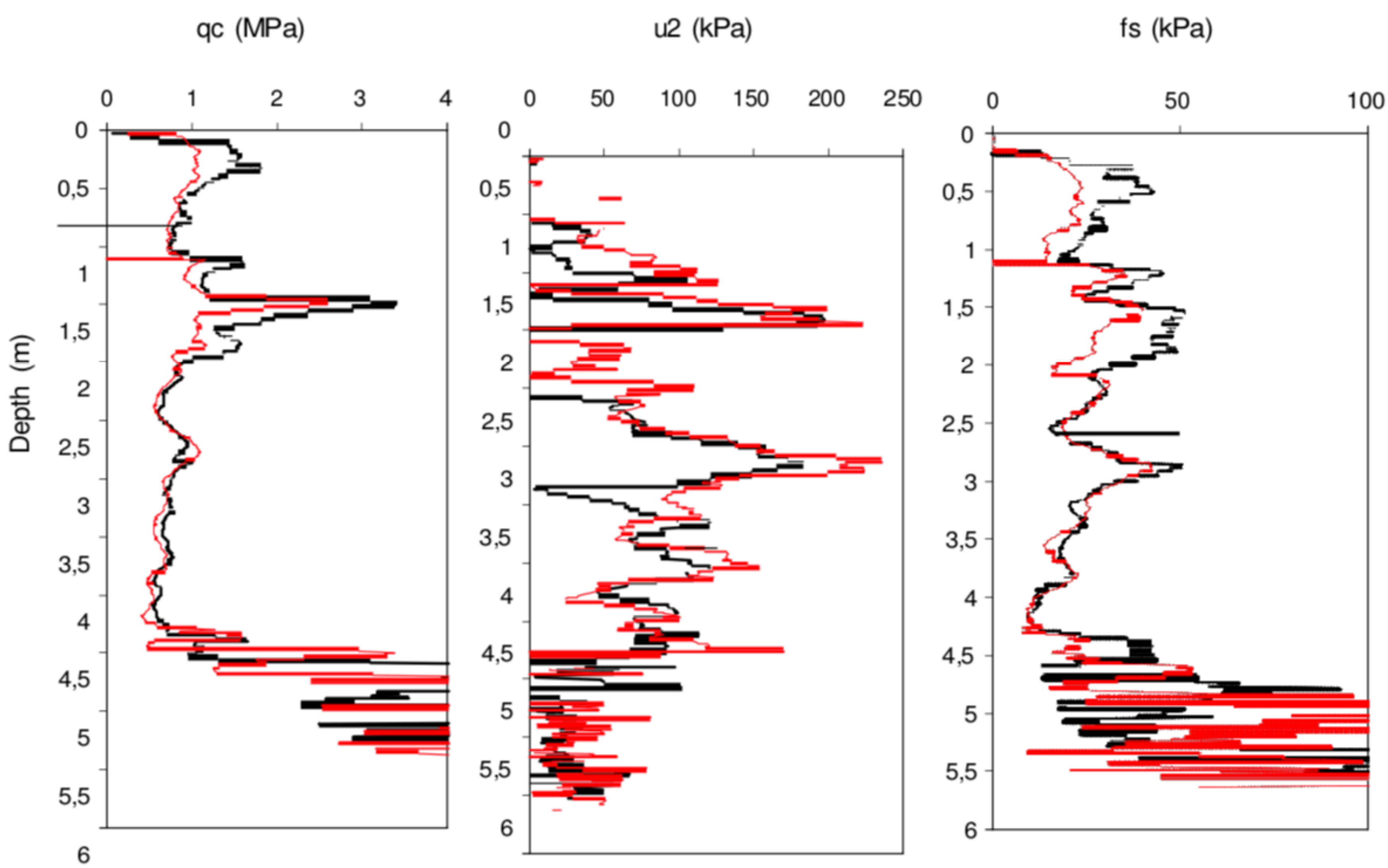

Figure 28. Paganico. Comparison of the standard rate (red line) and the slower rate (black line)

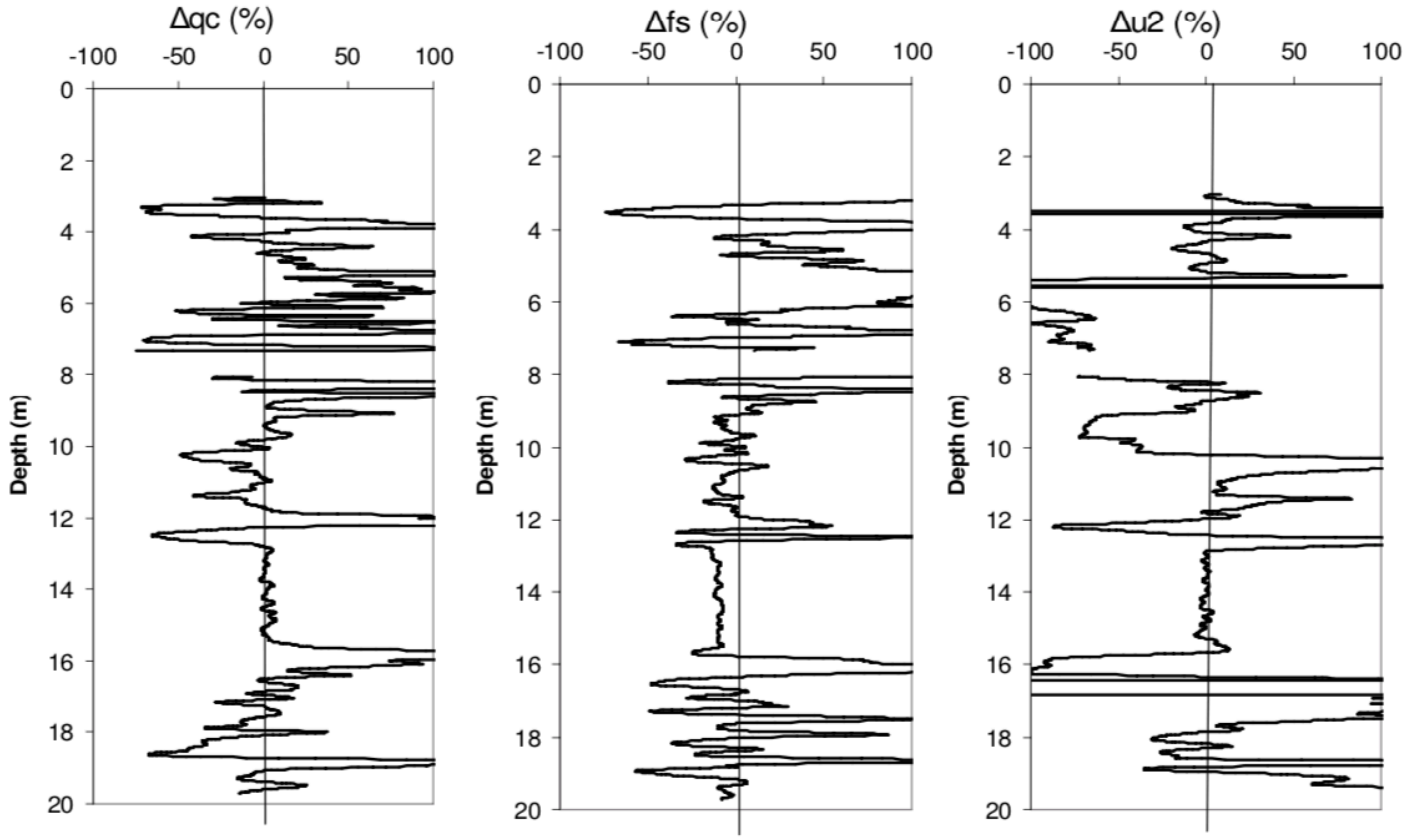

Figure 29. Site 2 (Livorno). Percentage variation of q_c, f_s and u_2 between 2 cm/s and 1cm/s

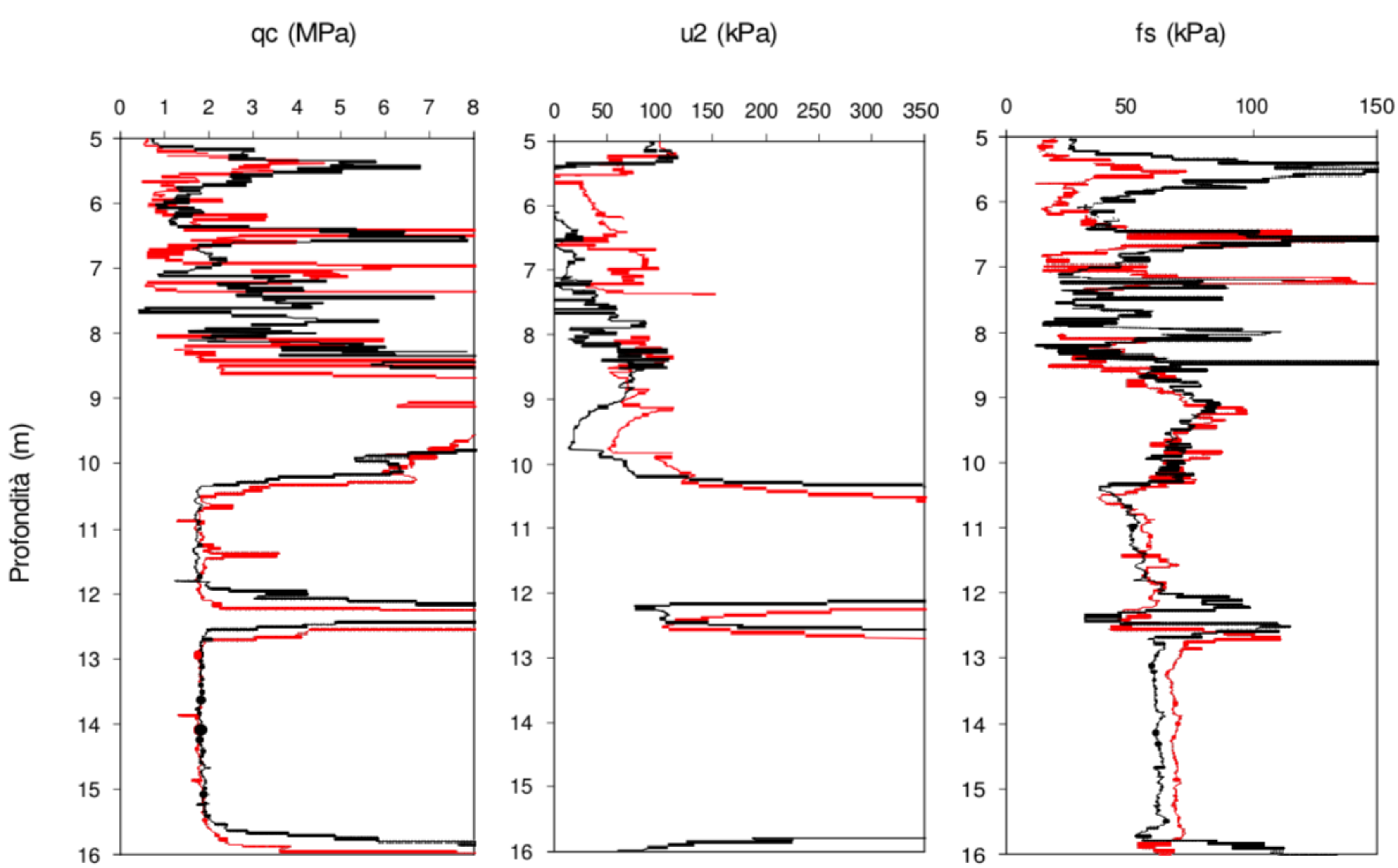

Figure 30. Livorno. Comparison between the slower rate (black line) and the standard rate (red line).

6.2. Use of CPT for assessing the compaction degree of earth works of fine-grained soils

An innovative procedure for assessing the compaction degree of earth works of fine-grained soils has been developed and verified. The proposed method is mainly based on the following hypotheses:

- the tip resistance (q_c) is not affected by the tip diameter, therefore we expect to measure the same q_c (in the same soil under the same conditions) using a standard cone having a diameter of 35.7 mm and a mini – penetrometer with a diameter of 8 mm;
- the tip resistance measured in situ using a standard cone [$q_{c(standard)}$] and that measured in a mini – Calibration Chamber (mini – CC) using a mini-penetrometer [$q_{c(mini)}$] under the condition of no lateral strain are the same for the same soil, with the same density and vertical effective stress;

If the above indicated assumptions are true, it is possible to measure the $q_{c(mini)}$ in the laboratory using specimens reconstituted at the prescribed density in a Proctor – mold and consolidated at different vertical pressures. We expect that $q_{c(standard)}$ measured in situ is the same as $q_{c(mini)}$ for the same soil with the same density and vertical effective stress. Therefore, we can (a – priori) establish which is the expected q_c corresponding to a prescribed density.
The assumptions have been experimentally verified and the proposed method has been applied in a real case consisting in the construction of a river – embankment 4m high. The construction material was classified as A4 to A6 according to AASHTO M 145 (1991).
The practical application of the method gave a further and final verification of the correctness of the previously indicated hypotheses. In fact "undisturbed" samples were retrieved with different methods at the river embankment, giving a direct evaluation of the soil density in situ.
A similar procedure is described by XP P 94-063 (1997) and XP P 94-105 (2000). This procedure is applied to coarse grained soils and requires the construction of a trial embankment and the performance of dynamic penetration tests. This procedure is applied to the control of the compaction degree of trenches (Setra – Lcpc 1994, 2007).
It is worthwhile to remark that there are few specific studies concerning the performance criteria of river embankments. As a consequence, in most cases are adopted the same design prescriptions as for road embankments or for earth-dams, which control type of material and compaction degree.

6.2.1. Equipment

The equipment consists of the following items:

- two end platens connected by three tie rods;
- an air piston fixed onto the lower end platen;
- Proctor - mold, which represents the mini – CC. The mold contains the test soil compacted to the desired density and is located between the air piston and the upper end platen. The air piston can apply a vertical pressure to the soil in the Proctor – mold through a rigid platen. The contrast is given by the upper end platen. The mold has an inner diameter of 152.4mm and a height of 116mm;
- the upper platen has a bush to allow the passage of the mini – penetrometer. The electric motor to push the mini – penetrometer at a constant rate of 2 cm/s is fixed onto the upper platen. The system has a couple of sensors to control the stroke of the mini – penetrometer (upper and lower position);
- the mini - penetrometer has a tip of 8 mm in diameter with an apex angle of 60°. Therefore the tip area is equal to 50.27 mm^2. The mini – penetrometer has an external casing to avoid measurement of side friction. The tip resistance is continuously measured by means of a load cell (max 5kN, accuracy 5N), located just above the mini – penetrometer.

The details of the described equipment are shown in Fig. 31.

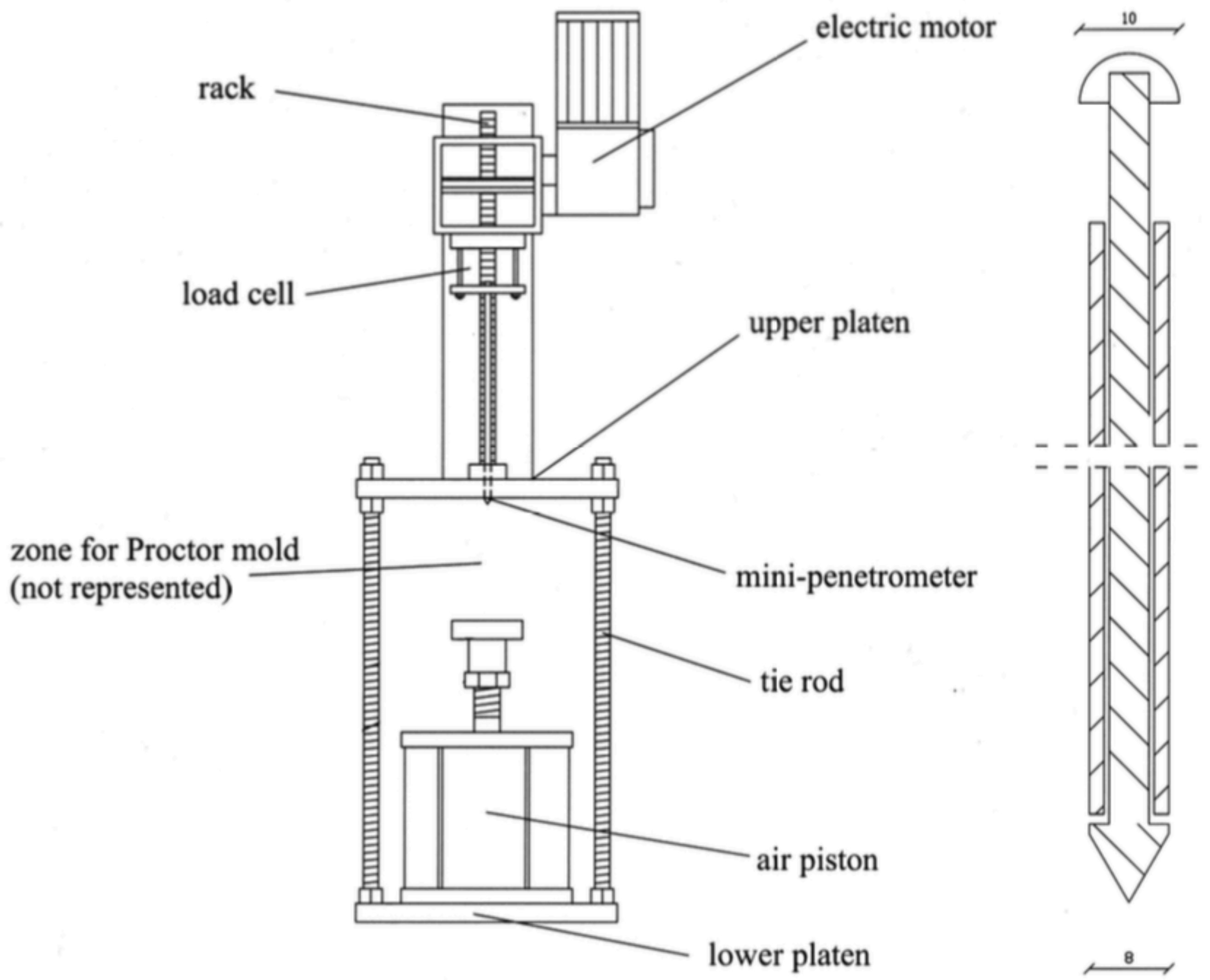

Figure 31. Scheme of equipment. On right see the detail of mini penetrometer.

Obviously, for each tip resistance it is necessary to apply a minimum vertical pressure to the test soil in order to guarantee the force equilibrium.
Additional information on the equipment is available in Carelli (2009) and Vuodo (2009).

6.2.2.Experimental assessment of the working hypotheses

Two different types of experimental activities have been run to verify the working hypotheses. The first step was to verify that the $q_{c(mini)}$ is equal to $q_{c(standard)}$ under the same site conditions. To this purpose in situ penetration tests using a standard cone and a small diameter cone in the same site at close distances have been carried out. In a second step the hypothesis that the effects of the mini – chamber sizes can be considered negligible has been verified. For this purpose some tests in the mini – CC with the mini – penetrometer using as test soil the Ticino sand (TS) were performed. There is a huge literature concerning the cone penetration tests (CPTe) run in CC on TS samples to which refer the results of tests carried out in the present research (e.g. Baldi et al. 1986, Jamiolkowski et al. 2000, 2001).

6.2.3.In situ assessment

Four tests with a standard cone and four tests with a mini – penetrometer were run in Calendasco (PC – Italy) on dry sandy silts. The tests were performed at close distance from each other (about 1m). The upper and lower envelopes obtained with the two different cones are shown in Fig.32. There is no systematic discrepancy between the results obtained with the two different cones.

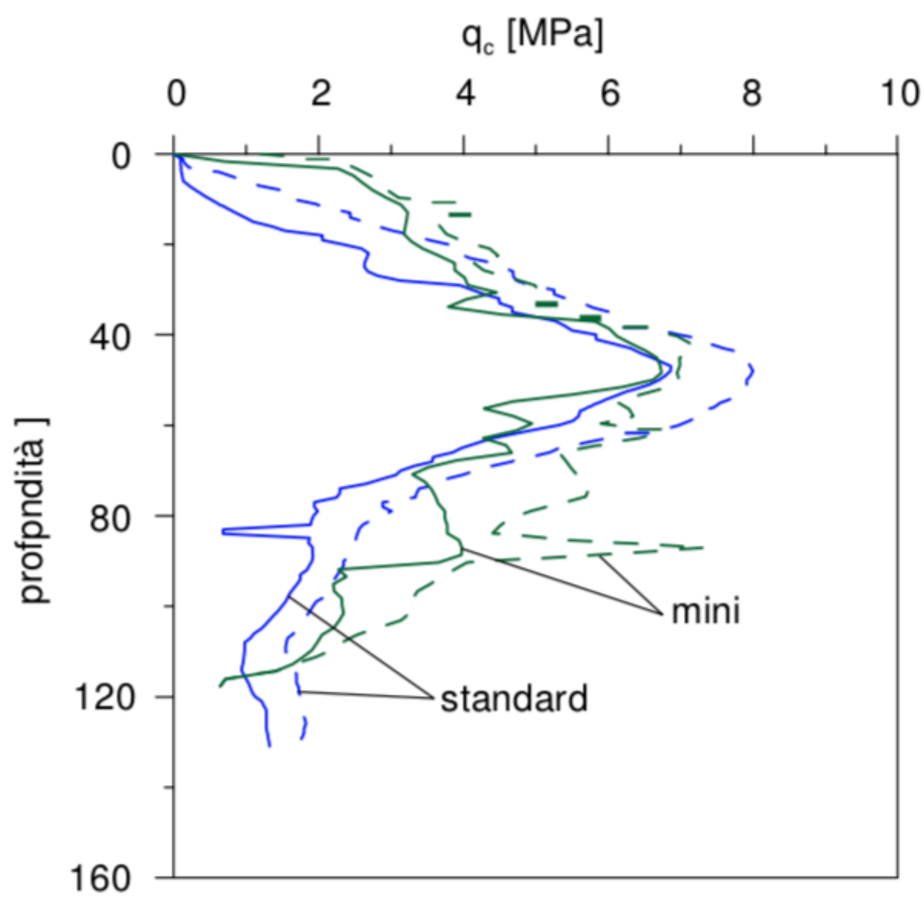

Figure 32. Results from in situ tests

6.2.4. Laboratory assessment

Penetration tests, using the mini – penetrometer, were run on TS samples reconstituted to a given relative density (about 50 %) in the mini – CC and consolidated under BC3 conditions (no lateral strain) under a given vertical pressure (100 ÷ 400 kPa). More specifically the so – called TS4 (γ_{dmin} = 13.91 kN/m3; γ_{dmax} = 17.00 kN/m3) was used for the tests. For these tests the ratio between the mini - CC diameter and the cone diameter (D_c/d_c) is equal to 19.5. The results were compared to those obtained in a large CC using standard cone under BC1 conditions (constant vertical and horizontal stresses). For these tests Dc/dc = 33.6.

Figura 33 shows typical examples of tip resistance with depth in the mini - CC. In one case the tip resistance attains a constant value, in the other case the tip resistance is continuously increasing. According to a well-established practice, the tip resistance at mid – height of CC was selected as reference value (Garizio, 1997).

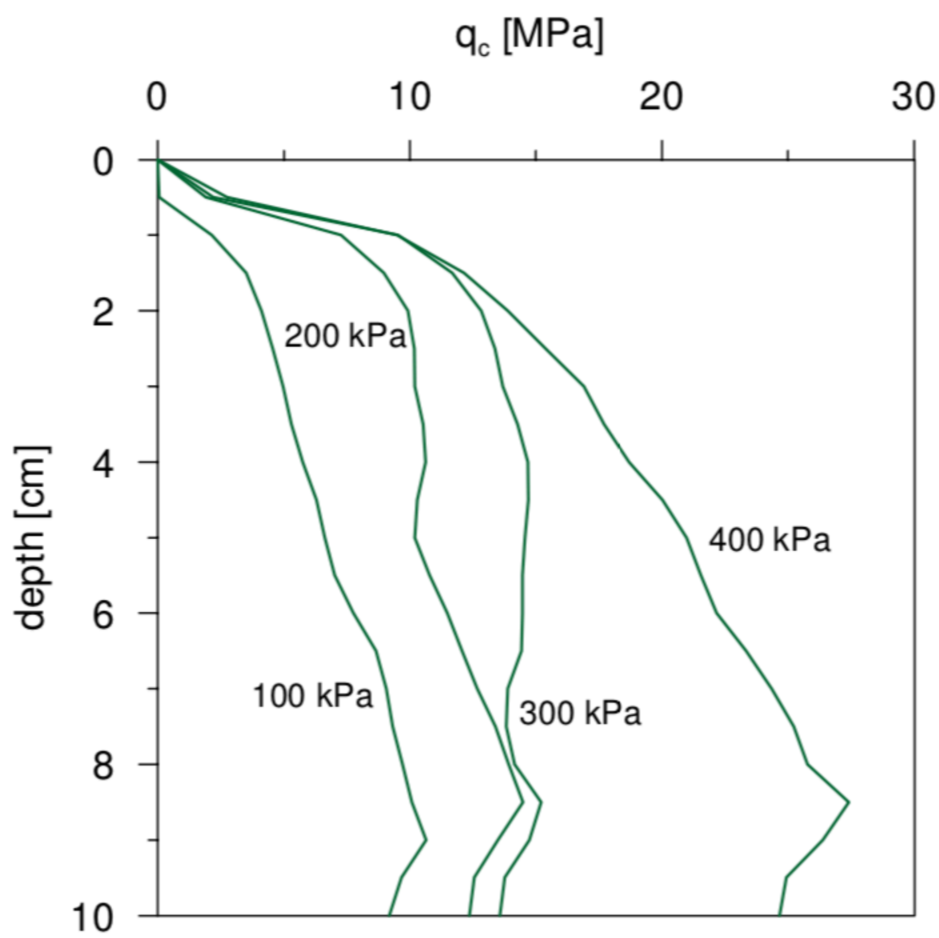

Figure 33. Some measurements in mini-CC

The correlations given for the TS by Baldi et al. (1986) and Jamiolkowski et al. (1988) were used for comparison. More specifically the tip resistance in the large CC with a standard cone was determined according to the following Equation 1:

$$q_c = C_0 \ (\sigma'_{v0})^{C1} e^{(DrC2)}$$

where C_0, C_1 and C_2 are empirical coefficients respectively equal to 172, 0.51 e 2.73. The term σ'_{v0} is the applied vertical pressure and Dr is the relative density of the sample in the mini - CC. The comparison is shown in Fig.34 .

There are several reasons to suppose that $q_{c(mini)}$ in the mini – CC must be different than $q_{c(CC)}$ in the large CC. More specifically:

- The Dc/dc ratio is different. For this reason the $q_{c(mini)}$ is expected lower than $q_{c(CC)}$. Several relationships have been suggested to take into account this aspect of phenomenon (Mayne & Kulhawy, 1991, Tanizawa, 1992, Garizio, 1997), but without a shared point of view;
- Tests performed with the mini – penetrometer in the mini – CC were run under BC3 condition. On the contrary the tests in the large CC were run under BC1 condition which is more representative of the conditions in an embankment. For this reason we also expect $q_{c(mini)} > q_{c(CC)}$. Unfortunately there are not enough experimental evidences to quantify this effect;
- The large CC has flexible boundaries (i.e. about nil fiction). On the contrary the mini – CC has rigid boundaries and therefore very high friction. For this reason we expect that $q_{c(mini)} < q_{c(CC)}$.

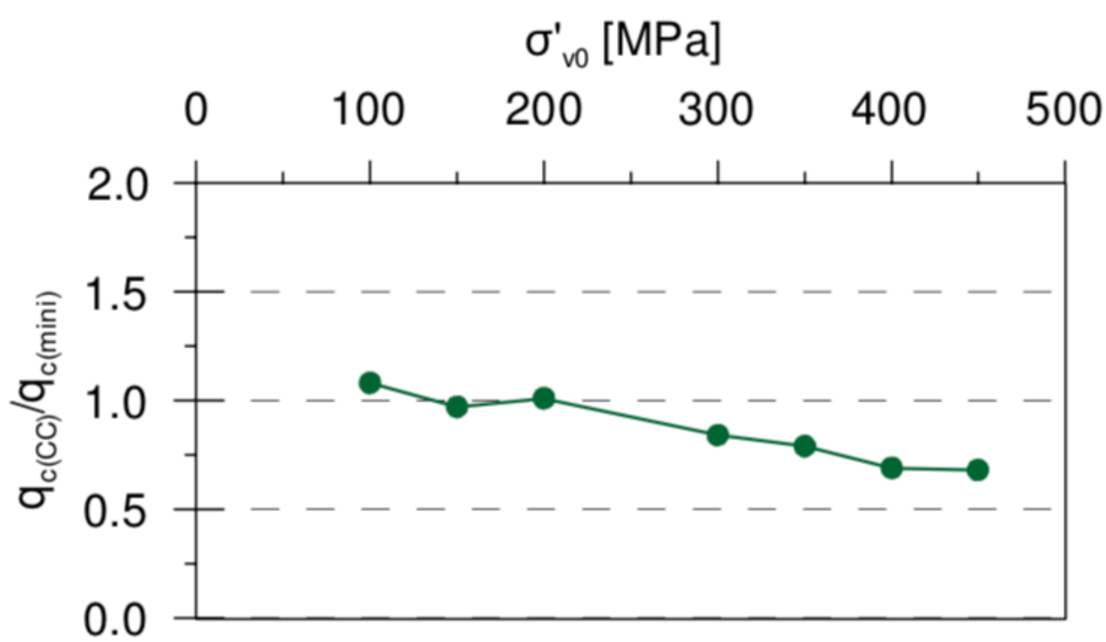

Figure 34. Comparison of results obtained in mini-CC with Equation 1

In the light of the above considerations, it is possible to assume that for the selected relative density there is a sort of effect compensation of the recalled phenomena within the pressure interval 100 – 300 kPa, so that $q_{c(CC)}/q_{c(mini)}$ is about equal to 1. It is worthwhile to remark that the indicated interval contains the stress level involved in the real case discussed in the next paragraph.

6.2.5. Application of the method to a real case

6.2.5.1. Design prescriptions

The main prescriptions for the contractor can be summarized as follows:

- a material classified as A4 to A6, according to AASHTO M 145 (1991), should be used for the embankment construction;
- lift of 30 cm of compacted material should be realized;
- the minimum compaction degree should correspond to a dry volume weight not less than 90% of the optimum density, without any specification of which optimum should be considered (Standard Proctor or Modified Proctor).

It is worthwhile to point out that, as a consequence of an especially wet season with intense and continuous raining, when the embankment construction was initiated the water table was at the embankment bottom. There was no specific prescription in the contract, which considered this adverse condition and possible countermeasures.

6.2.5.2.Soil type and classification

The following tests were performed on several samples of the construction material in order to control its quality:

- Standard Proctor (Fig.35)
- Modified Proctor (Fig.35)
- Grain size distribution (Fig.36)
- Index properties (Atterberg Limits) (Table 4)

Table 4. Atterberg Limits

Samples	WL [%]	WL [%]	PI %]
C1	31	24	6,5
C2	28,7	20	8,7
C3	28,7	18,6	10
C4	29	19,6	9,4
C5	26,1	19,1	7
C6	25,8	18,2	7,6

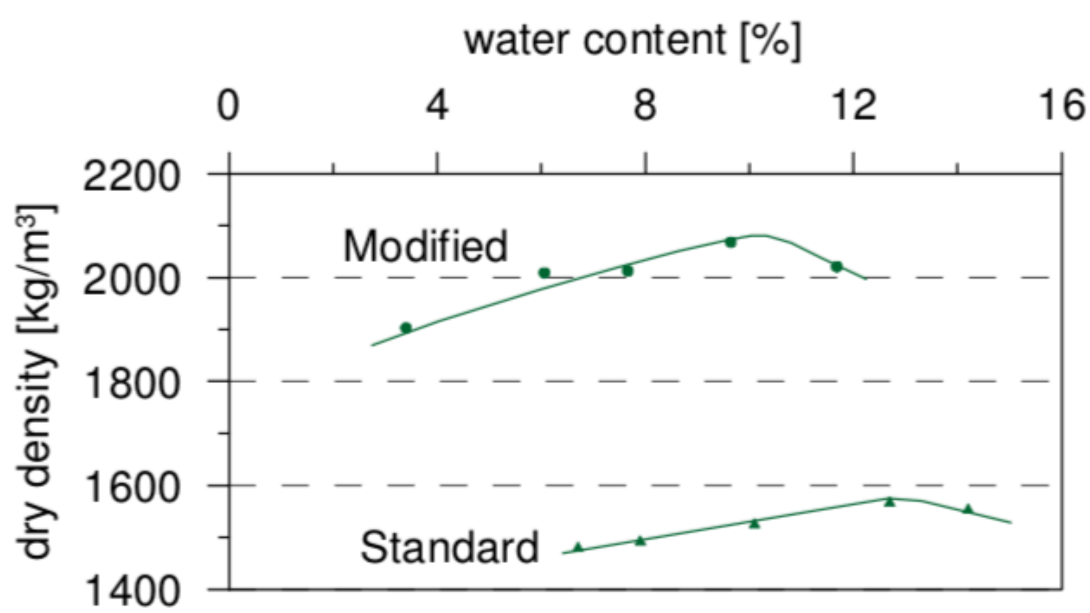

Figure 35. Results of compaction tests

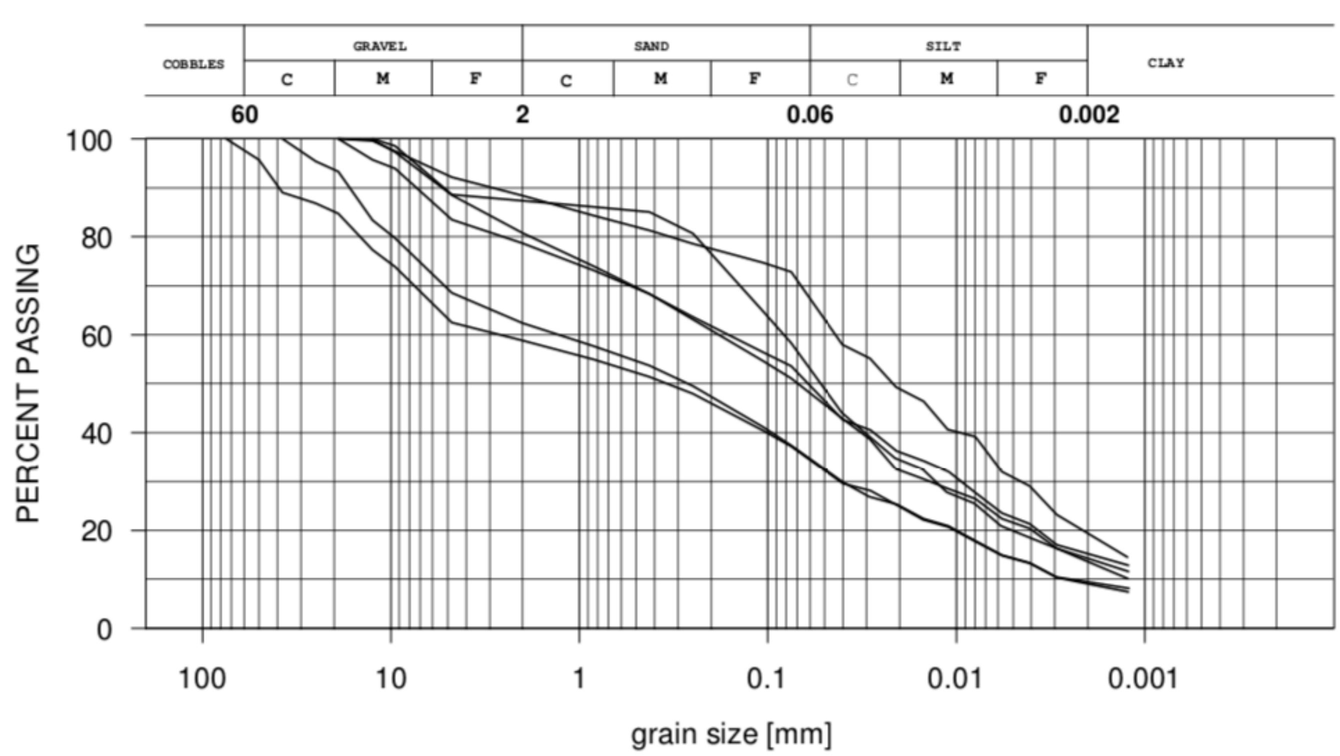

Figure 36. Grain size distribution of samples

6.2.5.3. Control of the degree of compaction

The compaction degree was controlled in the following way:

- CPTe's were performed in the lab on specimen reconstituted at two different densities corresponding to 80 and 90% of the optimum (Modified Proctor). For each density several specimens were reconsolidated at different vertical pressures. Obviously these tests were performed with the mini – penetrometer. For each density data have been interpolated using the equation $q_c=C_0\ (z)^{c1}$;
- CPTe's with the standard cone were performed in situ at three different locations in the embankment. For this purpose we used a TG63-200 static/dynamic penetrometer by Pagani Geotechnical Equipment (Pagani 2009);
- Undisturbed (or partially disturbed) samples were retrieved at the same locations of the in situ CPTe's. More specifically three very shallow block samples were retrieved. In addition, at two locations a specially devised sampler (AF shallow coring system) was used (Principe et al. 1997, 2007). The first sample extended down to a depth of 340 cm. The second only reached a depth of 90 cm, because of a failure of the equipment.

The AF shallow coring system is a very light equipment (handly transportable) which enables one to obtain up to 10 m long, continuous and partially disturbed micro-cores. For the case under consideration 38 mm in diameter cores were retrieved. The penetration of the equipment and elevation of the top of the sample were frequently monitored in order to account for a sample compaction during coring operations. The core diameter after extraction was also measured.

Fig. 37 shows the in situ CPTe profile (at a given location), interpolation curves of the laboratory experimental tip resistance, dry unit weight from "undisturbed" samples as a percentage of the optimum density (Modified Proctor) and end of the embankment.

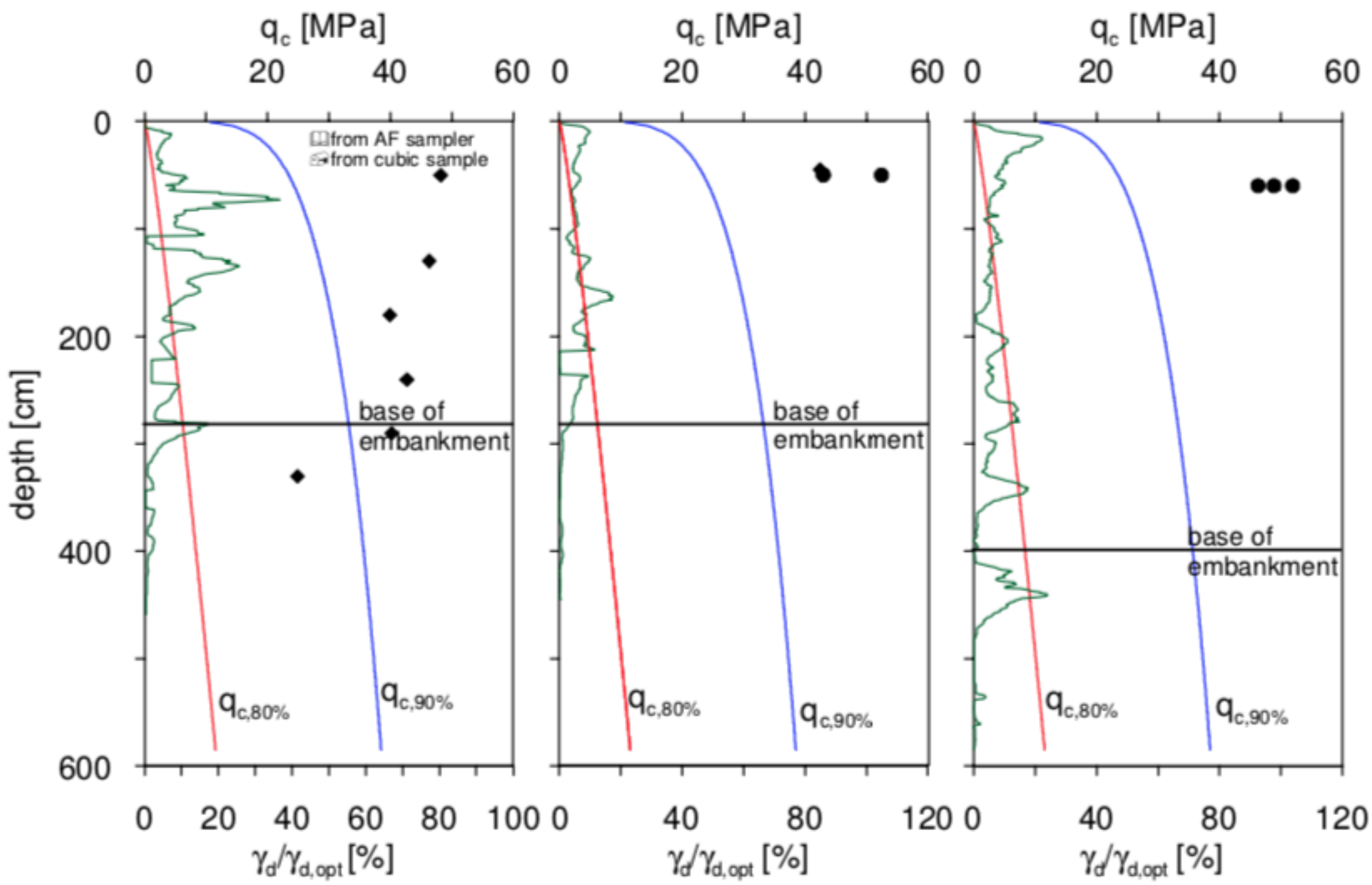

Figure 37. Results of in situ tests. Dots are referred to lower axis.

It is possible to observe that the measured laboratory and in situ values of tip resistances are consistent each other considering the in situ determined dry density.

The very low value of tip resistance at the bottom of the embankment is also quite evident, as expected as a consequence of the very high water content of the first layer, which probably was higher than 30 cm.

6.2.5.4. Conclusions

The proposed method was successfully applied as a quick control tool of the density of a river embankment.

The authors believe that specific researches are necessary to clearly define the design criteria of river embankments. Probably the construction details and type of soil are more relevant than the compaction degree.

Anyway, the proposed method seems applicable to any earthwork using fine soils.

7. References

AASHTO M 145 1991. Standard Specification for Classification of Soils and Soil-Aggregate AASHTO M 145 1991. Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes

Ahmadi M.M. E Robertson P.K. 2005. Thin layer effects on the CPT q_c measurement. Can. Geotech. J., 42, 1302-1317.

Alsamman O.M. (1995) The use of CPT for calculating the axial capacity of drilled shafts. Ph. D. Thesis UIUC IL, 128 – 174.

Amorosi A. and Marchi N. 1999. High-resolution sequence stratigraphy from piezocone tests: an example from the Late Quaternary deposits of the southeastern Po Plain Sedimentary Geology 128, 67–81.

ASTM. 2000. Standard test method for performing electronic friction cone and piezocone penetration testing of soils. ASTM standard D5778–95. ASTM International, West Conshohocken, Pa.

Baldi, G., Bellotti, R., Ghionna, V., Jamiolkowski, M. & Pasqualini, E. 1986. Interpretation of CPT's and CPTu's. 2nd Part: Drained Penetration. Proceeding 4th International Geotechnical Seminar, Singapore, pp.143-156

Baldi, G., D. Bruzzi, S. Superbo, M. Battaglio, and Jamiolkowski M. 1988. Seismic Cone in Po River Sand. Penetration Testing 1988, Vol. 2 (Proc. ISOPT-1, Orlando, Fla.), Balkema, Rotterdam, The Netherlands, pp. 643–650.

Begemann, H. K. S. 1965. The Friction Jacket Cone as an Aid in Determining the Soil Profile, Proc. 6th ICSMFE, 1, 17-20.

Cai G., Liu S., Puppala A. J. 2011. Comparison of CPT charts for soil classification using PCPT data: Example from clay deposits in Jiangsu Province, China. Engineering Geology 121, 89–96.

Campanella, R.G. and Robertson P.K. 1988. Current Status of the Piezocone Test. Proceedings of the First International Symposium on Penetration Testing, Vol. 1, Orlando, Fla. (Penetration Testing 1988), Balkema, Rotterdam, The Netherlands, Mar. 20–24, 93–116.

Campanella, R.G., Robertson P.K., and Gillespie D. 1986. Seismic Cone Penetration Test," Use of In- Situ Tests in Geotechnical Engineering (GSP 6), American Society of Civil Engineers, Reston, Va., pp. 116–130.

Canadian Geotechnical Society. 1985. Canadian Foundation Engineering Manual. Part 1 Fundamentals, Canadian Geotechnical Society, BiTech Publishers, Vancouver, BC, 456 p.

Carelli I. 2009. Metodi di controllo tradizionali ed innovativi di costruzioni in materiali sciolti. B. Thesis - Department of Civil Engineering - University of Pisa. In Italian.

Cestari F., Prove geotecniche in sito. Ed. Geo-graph s.n.c, Segrate, 1990.

Chen, B.S.-Y. and Mayne P.W. 1994. Profiling the Overconsolidation Ratio of Clays by Piezocone Tests, Report No. GIT-CEEGEO-94-1 to National Science Foundation by Geosystems Engineering Group, Georgia Institute of Technology, Atlanta, 1994, 280 pp. [Online]. Available: http://www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html .

De Beer E. E. (1985) Belgian Jubilee Volume, XI ICSMFE, S. Francisco

DeRuiter, J., "Electric Penetrometer for Site Investigations,"Journal of the Soil Mechanics and Foundations Division, Vol. 97, No. SM2, 1971, pp. 457–472.

Douglas, J. B. and Olsen R. S. 1981. Soil Classification using Electric Cone Penetrometer. Symposium on Cone Penetration Testing and Experience, Geotechnical Engineering Division, ASCE, St. Louis, pp. 209-227.

Elmgren, K. 1995. Slot-Type Pore Pressure CPTu Filters. Proceedings, International Symposium on Cone Penetration Testing, Vol. 2, Swedish Geotechnical Society Report 3:95, Oct. 4–5, 1995, Linköping, The Netherlands, pp. 9–12.

Eslami A., Fellenius B.H. 1997. Pile capacity by direst CPT and CPTu methods applied to 102 case histories. Can. Geotech. J., 34, 886-904.

Fellenius H.B., Eslami A. 2000. Soil Profile Interpreted from CPTu Data. "Year 2000 Geotechnics",

Geotechnical Engineering Conference, Asian Institute of Technology, Bangkok.

Garizio G.M. 1997. Determinazione dei parametri geotecnici e in particolare di K0 da prove penetrometriche M.Sc. Department of Structural Engineering, Politecnico di Torino. In Italian.

Ghionna, V.N. and D. Porcino. 2006. Liquefaction Resistance of Undisturbed and Reconstituted Samples of a Natural Coarse Sand from Undrained Cyclic Triaxial Tests. Journal of Geotechnical and Geoenvironmental Engineering, 132, 2, 194–202.

ISO 22476-1. 2005. Geotechnical investigation and testing. Field testing, part 1: Electrical cone and piezocone penetration tests.

ISO 22476-12. 2009. Geotechnical investigation and testing. Field testing, part 12: Mechanical cone penetration test.

Jaeger R.A., DeJong J.T., Boulanger R.W., Low H.E., Randolph M.F. 2010. Variable penetration rate CPT in an intermediate soil. CPT'10, 2nd International Symposium on Cone Penetration Testing, May 9-11 2010, Huntington Beach, California.

Jamiolkowski M., Ghionna V.N., Lancellotta R. & Pasqualini E. 1988. New correlations of penetration tests for design practice. Proc., Penetration Testing 1988, ISOPT 1, Orlando, Florida, J. De Ruiter ed., Vol. 1 pp: 263-296

Jamiolkowski M., Lo Presti D.C.F. & Garizio G.M. 2000. Correlation between Relative Density and Cone Resistance for Silica Sands. 75th Anniversary of Karl Terzagni's ERDBAU

Jamiolkowski M., Lo Presti D.C.F. & Manassero M. 2001. Evaluation of Relative Density and Shear Strength of Sands from CPT and DMT, Invited Lecture Ladd Symposium, GSP No. 119, ASCE, pp. 201-238.

Jamiolkowski, M., C.C. Ladd, J.T. Germaine, and Lancellotta R. 1985. New Developments in Field and Lab Testing of Soils. Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, San Francisco, Calif., Aug. 12–16, 1985, pp. 57–154.

Jamiolkowski, M., D.C.F. LoPresti, and Manassero M. 2001. Evaluation of Relative Density and Shear Strength of Sands from Cone Penetration Test and Flat Dilatometer Test," Soil Behavior and Soft Ground Construction (GSP 119), American Society of Civil Engineers, Reston, Va., pp. 201– 238.

Jefferies M.G. and Davies M.P. 1993. Use of CPTu to estimate equivalent SPT N60. Geotechnical Testing Journal, ASTM, 16, 458-468.

Jung B.C., Gardoni P., Biscontin G. 2008. Probabilistic soil identification based on cone penetration tests. Géotechnique, 58, 591-603.

Keaveny, J.M. and Mitchell J.K. 1986. Strength of Fine-Grained Soils Using the Piezocone. Use of In- Situ Tests in Geotechnical Engineering (GSP 6), American Society of Civil Engineers, Reston, Va., pp. 668–699.

Konrad, J.-M. and Law K.T. 1987. Undrained Shear Strength from Piezocone Tests. Canadian Geotechnical Journal, 24, 3, 392–405.

Kulhawy, F.H. and Mayne P.W. 1990. Manual on Estimating Soil Properties for Foundation Design, Report EPRI EL-6800, Electric Power Research Institute, Palo Alto, Calif., 306 pp.

Kurup, U. & Griffin, E.P. 2006. Prediction of soil composition from CPT data using general regression neural network. Journal of Computing in Civil Engineering, 20, 281-289.

Ladd, C.C.. 1991. Stability Evaluation During Staged Construction," The 22nd Terzaghi Lecture, Journal of Geotechnical Engineering, 117, 4, 540–615.

Lafuerza S., Canals M., Casamor a J.L., Devincenzi J.M. 2005. Characterization of deltaic sediment bodies based on in situ CPT/CPTu profiles: A case study on the Llobregat delta plain, Barcelona, Spain. Marine Geology , 222–223, 497–510.

Larsson, R. 1995. Use of a Thin Slot as Filter in Piezocone Tests. Proceedings, International Symposium on Cone Penetration Testing, Vol. 2, Swedish Geotechnical Society Report 3:95, Oct. 4–5, 1995, Linköping, Sweden, pp. 35–40.

Lee, J.H. and R. Salgado. 1999. Determination of Pile Base Resistance in Sands," Journal of Geotechnical and Geoenvironmental Engineering, 125, 8, 673–683.

Leroueil, S. and D. Hight. 2003. Behavior and Properties of Natural Soils and Soft Rocks. Characterization and Engineering Properties of Natural Soils, Vol. 1, Swets and Zeitlinger, Lisse, The Netherlands, pp. 29–254.

Lo Presti D, Squeglia N., Meisina C. & Visconti L. 2010 Use of CPT and CPTu for soil profiling of "intermediate soils": a new approach. CPT'10, 2nd International Symposium on Cone Penetration Testing, May 9-11 2010, Huntington Beach, California.

Lo Presti, D., Meisina, C., Squeglia, N. 2009. Use of cone penetration tests for soil profiling. Rivista Italiana di Geotecnica, 2, 9-33.

Lunne T., Robertson P.K., and Powell J.J.M. 1997. Cone penetration testing in geotechnical practice. Blackie Academic, EF Spon Routledge Publ., New York, 312 pp.

Lunne, T., M. Long, and Forsberg C.F. 2003. Characterization and Engineering Properties of Holmen, Drammen Sand," Characterisation and Engineering Properties of Natural Soils, Vol. 2 (Proc. Singapore), Swets and Zeitlinger, Lisse, The Netherlands, pp. 1121–1148.

Lunne, T., T. Eidsmoen, D. Gillespie, and Howland J.D. 1986. Laboratory and Field Evaluation of Cone Penetrometers. Use of In-Situ Tests in Geotechnical Engineering (GSP 6), American Society of Civil Engineers, Reston, Va., 1986, pp. 714–729.

Maine, P. W., Kulhawy, F. H. 1991. Calibration chamber database and boundary effects correction for CPT data. In Calibration chamber Testing. New York: Elsevier, pp. 257-264.

Mayne, P.W. 2007.Cone penetration testing: A synthesis of highway practice. Project 20-5. Transportation Research Board, Washington, D.C. NCHRP synthesis 368.

Mayne, P.W., F.H. Kulhawy, and Kay J.N. 1990. Observations on the Development of Porewater Pressures During Piezocone Testing in Clays. Canadian Geotechnical Journal, 27, 4, 418–428.

Meyerhof G.G. (1976) Bearing capacity and settlement of pile foundations, JGED, ASCE, GT3, 197 - 228

Mimura, M. 2003. Characteristics of Some Japanese Natural Sands—Data from Undisturbed Frozen Samples," Characterisation and Engineering Properties of Natural Soils, 94 Vol. 2 (Proc. Singapore), Swets and Zeitlinger, Lisse, The Netherlands, pp. 1149–1168.

Mulabdic´, M., S. Eskilson, and Larsson R. 1990. Calibration of Piezocones for Investigations in Soft Soils and Demands for Accuracy of the Equipment. Report Varia 270, Swedish Geotechnical Institute, Linköping, Sweden, 1990, 62 pp.

Pagani Geotechnical Equipment. 2009. http://www.pagani-geotechnical.com.

Principe C. Malfatti A., Rosi M., Ambrosio M. Fagioli M.T. 1997. Metodologia innovativa di carotaggio microstratigrafico: esempio di applicazione alla tefrostratigrafia di prodotti vulcanici distali. Geologia Tecnica Ambientale Vol 39, 4/97. In Italian.

Principe C., Malfatti A.,. Ambrosio M., Fagioli M.T., Rosi M.,. Ceccanti B., Arrighi S., Innamorati D. 2007. Finding distal Vesuvius tephra at the borders of Lago Grande di Monticchio, in AF SHALLOW CORING SYSTEM micro-cores. Atti Soc. tosc. Sci. nat., Mem., Serie A, 112 (2007). pagg. 189-197, figg. 4, tabb. 4

Ramsey N. 2010. Some issues related to applications of the CPT. CPT'10, 2nd International Symposium on Cone Penetration Testing, May 9-11 2010, Huntington Beach, California.

Robertson P. K., Campanella R. G., Gillespie D., Grieg J. 1986. Use of Piezometer Cone Data. Proceedings of American Society of Civil Engineers, ASCE, "In Situ 86: Specialty Conference", edited by S. Clemence, Blacksburg, June 23 – 25, Geotechnical Special Publication GSP No. 6, pp. 1263-1280.

Robertson P.K. 1990. Soil Classification Using the Cone Penetration Test. Can. Geotech. J., vol. 27, pp. 151-158.

Robertson P.K. 2010. Soil behaviour type from the CPT: an update. CPT'10, 2nd International Symposium on Cone Penetration Testing, May 9-11 2010, Huntington Beach, California.

Robertson, P.K. 2009. Interpretation of cone penetration tests – a unified approach. Can. Geotech. J. 46, 11, 1337-1355.

Robertson, P.K. and Campanella R.G. 1983. Interpretation of Cone Penetration Tests: Sands. Canadian Geotechnical Journal, 20, 4, 719–733.

Schmertmann J.H., Hartman J.D. and Brown P.R. (1978) Improved strain influence factor diagrams, JGED, ASCE, Technical Note, 104, GT8, 1131 – 1135.

Schmertmann, J.H., 1978. Guidelines for Cone Penetration Test, Performance and Design. Report No. FHWA-TS-78-209, U.S. Department of Transportation, Washington, D.C., pp. 145, 1978.

Searle, I.W. 1979. The interpretation of Begemann Friction Jacket Cone Results to Give Soil Types and Design Parameters. Design Parameters in Geotechnical Engineering, BCS London 2: 265-270.

Sennest K., Sandven R., Jambu N. 1989. Evaluation of Soil parameters from piezocone test. In – Situ Testing of Soil Properties for Transportation, Transportation Research Record No. 1235, Washington D.C., pp. 24 – 37.

Setra – Lcpc. 1994. Remblelayage des tranchées et réfection des chaussées – Guide Technique. Setra/LCPC, réf. D9441

Setra – Lcpc. 2007. Remblelayage des tranchées et réfection des chaussées – Compléments. Setra/LCPC, no. 117

Squeglia N., Lo Presti D. C. 2010. Use of mini CPT to evaluate degree of compaction in fine grained soils, 1st Europe-China Workshop on Capability of Penetration Tests in geotechnical Research and Practice, pp 65-72, Pisa,vol. 1,2010.

Tanizawa F. 1992. Correlations between cone resistance and mechanical properties of uniform clean sand, Internal Report ENEL – CRIS, Milan.

Trak, B., P. LaRochelle, F. Tavenas, S. Leroueil, and M. Roy. 1980. A New Approach to the Stability Analysis of Embankments on Sensitive Clays. Canadian Geotechnical Journal, 17, 4, 526–544.

Vreugdenhil R., Davis R., Berril J. 1994. Interpretation of cone penetration results in multilayered soils. International Journal for numerical and analytical methods in geomechanics, vol. 18, pp. 585- 599.

Vuodo C. 2009. Uso delle prove CPT per il controllo della qualità dei rilevati B. Thesis - Department of Civil Engineering - University of Pisa. In Italian.

Wride, C.E. and Robertson P.K. 1999. CANLEX: The Canadian Liquefaction Experiment: Data Review Report (Five Volumes), BiTech Publishers Ltd., Richmond, BC, Canada, 1,081 pp.

Wride, C.E., Robertson, P.K., Biggar, K.W., Campanella, R.G., Hofmann, B.A., Hughes, J.M.O., Ku ̈pper, A., and Woeller, D.J. 2000. Interpretation of in situ test results from the CANLEX sites. Canadian Geotechnical Journal, 37(3): 505–529. doi:10.1139/T00-044.

XP P 94-063. 1997. Controle de la qualité du compactage-methode au penetrometre dynamique a energie constante. AFNOR

XP P 94-105. 2000. Controle de la qualité du compactage-methode au penetrometre dynamique a energie variable. AFNOR

Zhang, Z. & Tumay, M.T. 1999. Statistical to fuzzy approach toward CPT soil classification. J. Geotech. Geoenviron. Eng. 125, 179-186.

8. Terms and Definitions

Average surface roughness
Ra
Average deviation between the real surface of the probe and a medium reference plane placed along the surface of the probe

Cone
Conically shaped bottom part of the cone penetrometer
NOTE When the penetrometer is pushed into the ground the cone penetration resistance is transferred through the cone by inner rods to the measuring device at ground level.

Cone penetration test (CPT)
Pushing of a cone penetrometer at the end of a series of cylindrical push rods into the ground at a constant rate of penetration

electrical CPTe
Cone penetration test in which forces are measured electrically in the cone penetrometer

Mechanical CPTm
CPT where forces are measured mechanicaliy or electrically at ground level

Piezocone penetration test (CPTu)
Electrical CPT with measurement of the pore pressures at or close to the cone

Cone penetrometer
Assembly containing cone, friction sleeve (optional), connection to the push rods and measuring devices for the determination of the cone penetration resistance and, if applicable, the total resistance and/or local side friction

Cone penetration resistance
Cone resistance
resistance to the penetration of the cone

Continuous penetration testing
Test method in which cone penetration resistance is measured while cone and push rods are moving continuously until stopped for the addition of a push rod

Corrected cone resistance (q_t)
Measured cone resistance qc corrected for pore pressure effects

Corrected friction ratio (Rn)
Ratio of the sleeve friction to the corrected cone resistance measured at the same depth

Corrected sleeve friction (f_t)
Measured sleeve friction fc corrected for pore pressure effects

Discontinuous penetration testing
Test method in which cone penetration resistance and, optionally, sleeve friction are measured during a penetration stop of the push rods

Dissipation test
Measure of the pore pressure change recording the values of the pore pressures in time during a pause in pushing while holding the cone penetrometer stationary

Excess pore pressure Δu_1, Δu_2, Δu_3
$\Delta u_1 = u_1 - u_0$

$\Delta u_2 = u_2 - u_0$

$\Delta u_3 = u_3 - u_0$

Additional pore pressure at the at the level of the filter caused by the penetration of the cone penetrometer into the ground.

Filter element
Porous element in the cone penetrometer that transmits the pore pressure to the pore pressure sensor, maintaining the geometry of the cone penetrometer,

Force acting on the friction sleeve (F_s)
Force that will be obtained by subtracting the measured force on the cone from the measured force on the cone and friction sleeve

Friction ratio (R_f)
Ratio of sleeve friction to cone penetration resistance measured at the same depth, expressed as a percentage:

$R_f = f_s/q_c \times 100\%$

NOTE: In some cases the inverse of the friction ratio, called the *friction index*, is used.

Friction reducer
Local and symmetrical enlargement of the diameter of a push rod to reduce the friction along the push rods

Friction sleeve
Section of the cone penetrometer where sleeve friction is determined

In situ equilibrium pore pressure (u_o)
Original in situ pore pressure at filter depth

Inclination
Deviation of the cone penetrometer from the vertical

Initial pore pressure (u_1)
Measured pore pressure at the start of the dissipation test

Inner rods
Solid rods sliding inside the push rods and transferring the forces from the cone and, optionally, the friction sleeve, to the measuring system.

Measured pore pressure at the start of the dissipation test inner rods
Solid rods sliding inside the push rods and transferring the forces from the cone and, optionally, the friction sleeve, to the measuring system

measured cone penetration resistance (q_c)
division of the measured force , Qc on the cone by the cross-sectional area, Ac: $q_c = Q_c/A_c$

NOTE The measured cone penetration resistance obtained from a mechanical CPTm can differ from that obtained from an electrical CPTe.

Measured sleeve friction f_s
Force, F_s, acting on the friction sleeve divided by the area of the sleeve, A_s:

$f_s = F_s/A_s$

NOTE The measured sleeve friction obtained from a mechanical CPTm test can be different from the value obtained from an electrical CPTe test.

Measured total penetration force (Q_t)
Force needed to push cone and rods together into the soil

Measuring system
All sensors and auxiliary parts used to transfer and/or store the signals

generated during the cone penetration test
NOTE: The force on the cone and, if applicable, the total penetration resistance and/or sleeve friction are measured with manometers or with electrical load sensors.

Net area ratio (a_n)
Ratio of the cross-sectional area of the load cell or shaft of the cone penetrometer above the cone at the location of the gap or groove where pore pressure can act (A_n), to the nominal cross-sectional area of the base of the cone (A_c)

net cone resistance (q_n)
Measured cone resistance corrected fot the total overburden soil pressure

Net friction ratio (R_{fn})
Ratio of the sleeve friction to the net cone resistance

Normalized excess pore pressure (U)
Excess pore pressure during a dissipation test compared to the initial excess pore pressure

Penetration depth (z)
Depth of the base of the cone, relative lo a fixed horizontal plane

NOTE: With mechanical CPT, penetration depth cannot be determined, as there is no inclinometer measurement for depth correction.

Penetration length (l)
Sum of the lengths of the push rods and the cone penetrometer, reduced by the height of the conical part, relative to a fixed horizontal plane
NOTE: The fixed horizontal plane usually corresponds with a horizontal plane through the ground surface at the location of the test.

Pore pressure at time t during dissipation test (U_t)
Pore pressure at time t during dissipation test

Pore pressure ratio (B_q)
Ratio of the excess pore pressure at the U_2 filter position to the net cone resistance

Push rod
Part of a string of rods for the transfer of forces to the cone penetrometer

Reference reading
Reading of a sensor just before the penetrometer penetrates the ground or just after the penetrometer leaves the ground

Seismic piezocone test (SCPTu)
Piezocone with the additional possibility of discontinuous measurement of body wave propagation velocities mainly in a down – hole configuration

Thrust machine
Equipment that pushes the cone penetrometer and rods into the ground at a constant rate of penetration
NOTE: The required reaction for the thrust machine can be supplied by dead weights and/or soil anchors.

Total overburden stress (σ_{v0})
Stress due to the total weight of the soil layers at the depth of the base of the cone

Total side friction force (Q_{st})
Force needed to overcome the side friction on the push rods, when these are pushed into the ground NOTE: The total side friction force is obtained by subtracting the force on the cone (Q_c) from the measured total penetration force (Q_t):
$Q_{st} = Q_t - Q_c$

Zero drift
Absolute difference between the zero readings of a measuring system at the start and after completion of a cone penetration test

Symbol	Name
A_c	Cross sectional projected area of the cone
a_n	Net area ratio
A_n	Area of load cell or shaft
A_s	Area of friction sleeve
A_{sb}	Cross sectional area of the bottom of the friction sleeve
A_{st}	Cross sectional area of the top of the friction sleeve
B_q	Pore pressure ratio
c'	Effective cohesion
C_c	Vergin compression index
C_{inc}	Correction factor for the inclination of the cone penetrometer relative to the vertical axis
C_s	Swelling index
c_u	The same of su = undrained shear strength
d_2	Diameter of the friction sleeve
d_c	Diameter of the cylindrical part of the cone
D_{cone}	Diameter of the cone at a specified height
d_{fil}	Diameter of the filter
DR	Relative density of sand
F_s	Measured force on the friction sleeve
f_s	Measured sleeve friction
f_t	Corrected sleeve friction
h_c	Height of the conical section of the cone
h_e	Length of the cylindrical extension of the cone
L	Penetration length
l_s	Lenght of the friction sleeve
N_{kt}	Cone bearing factor for evaluating undrained shear strength
Q_c	Measured fore on the cone
q_c	Measured cone resistance
q_e	Effective cone resistance
q_n	Net cone resistance
q_t	Corrected cone resistance
R_a	Average surfaces roughness

R_f	Friction ratio
R_{fn}	Net friction ratio
R_{ft}	Corrected friction ratio
t	Time
t_{50}	Time needed for 50 % pore pressure dissipation
U	Normalised excess pore pressure
u	Pore pressure
u_1	Pore pressure in the face of the cone
u_2	Pore pressure in the gap between the cone and the sleeve
u_3	Pore pressure measured above the friction sleeve
u_0	In situ, initial pore pressure
u_t	Pore pressure at time t in a dissipation test
Vs	Shear wave velocity
z	Penetration depth
z_w	Depth to ground water table
Δ_1, Δ_2, Δ_3	Excess pore pressure at filter locations 1, 2 and 3
α	Measured total angle between the vertical axis and the axis of the cone penetrometer
αx, y	Angle between the vertical axis and the projection of the cone penetrometer on a fixed vertical piane
β	Angle between the vertical axis and the projection of the cone penetrometer on a vertical plane that is perpendicular to the plane of angle α
φ'	Effective friction angle
γ_d	Dry unit weight
γ_w	Unit weight of water
σ'_{ho}	Effective geostatic lateral stress
σ'_p	Efective preconsolidation test
σ_{vo}	Total overburden stress
σ'_{vo}	Effective vertical overburden stress

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