Politecnico di Torino

Master of Science in Civil Engineering

Thesis of Master's Degree

Using CPT for hydrogeological characterization in geoenvironmental engineering applications

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Notations

a	net area ratio for the cone ($= A_n/A_c$)
A_c	projected area of the cone
A_n	cross-sectional area of the load cell or shaft
c_h	horizontal consolidation coefficient
f_s	unit sleeve friction resistance
F_r	normalized friction ratio ($= f_s/(q_t - \sigma_{vo})$)
G_0	small strain shear modulus, shear stiffness
I_c	soil behavior type index
k	hydraulic conductivity of the soil (coefficient of permeability)
k_h	hydraulic conductivity in the horizontal direction
M	constrained deformation modulus
M_0	reference constrained modulus corresponding to the in situ vertical effective stress
p_a	reference stress (= 100 kPa)
q_c	tip resistance measured during the test
q_t	corrected tip resistance
Q_{tn}	normalized cone resistance
R	radius of the pushing rod
R_f	friction ratio ($= \frac{f_s}{q_t} \cdot 100\%$)
S_r	degree of saturation
t	time
t_{50}	time for 50% dissipation of excess pore water pressure
T	time factor
u_1	pore water pressure filter element located at the tip of the cone
u_2	pore water pressure filter element located at the base of the cone
u_3	pore water pressure filter element located at the end of the sleeve

v_p	compression wave velocity
v_s	shear wave velocity
γ_w	unit weight of water
$\sigma_{v0}, \sigma'_{v0}$	total vertical geostatic stress
ρ_t	bulk soil mass density
Δh	change in water level

Abbreviations

$ASTM$	American Society for Testing and Materials
CPT	Cone Penetration Test
$CPTU$	Cone Penetration Test with Pore Pressure Measurement (Piezocone Test)
CH	Cross-Hole testing mode
DH	Down-Hole testing mode
NC	Normally Consolidated
OC	Overconsolidated
$SCPTU$	Seismic CPTU

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Abstract

This research aims to estimate the hydraulic conductivity of layered soils using the CPTU test. The site investigations were conducted close to the city of Ravenna (Italy) and the experimental SCPTU-1 and SCPTU-2 data collected were used for analysis.

Mainly three methods described and implemented for several SCPTU test results and classification of soil layers have been obtained. A unique model is created with soil layers and for each layer, the estimation of hydraulic conductivity in horizontal and vertical direction have been obtained.

The focus of this work is to define the permeability of the subsurface in horizontal and vertical directions using the Robertson approach giving a real case study of test data obtained in Ravenna.

These obtained results can further be used for geoenvironmental applications or to make a model for monitoring groundwater contamination.

Introduction

This paper presents and demonstrates the use of CPT (Cone Penetration Test) for hydrogeological characterization in geo-environmental engineering applications.

This document focuses on comparing the methods and estimating hydraulic conductivity from the SCPTU tests and comparing the results for layered soils.

Nowadays the Cone Penetration Test (CPT) is a widely used testing technique to provide fast and accurate subsurface soil stratigraphy data. However, the test can be modified using electric piezocone and adding auxiliary modules and sensors to obtain the required parameters on top of basic ones.

All structures including buildings, bridges, walls, embankments, and pavements are ultimately supported by the existing ground. For safety and economy, the design of these structures is based on the properties of building materials and the properties of the foundation materials in the natural ground. Therefore, the designer of these structures must know the surface and subsurface soil conditions to create safe and efficient designs.

Taking deep boring is an expensive, time-consuming process, obtaining samples for the design of deep foundations involves drilling into the earth to retrieve the soil found at various depths to determine its strength properties. To recover undisturbed samples and perform standard penetration soundings wet rotary drilling equipment is needed. Wet drilling is always messy and sometimes hazardous work. Therefore, Cone Penetration Test (CPT) is introduced to provide subsurface stratigraphy.

Compared to conventional boring, sampling and laboratory testing methods CPT technology provides a reliable, economical, and time-saving tool for site characterization.

The ability of soil to allow liquids to pass through its connecting cavities is one of the most important properties of soil. The study of soil permeability is an important part of soil mechanics. In geotechnical engineering, drainage and water movement in fine-grained soils are very important.

In Chapter 2 various methods to obtain hydraulic conductivity are introduced and in Chapter 3 obtained SCPTU results are used to estimate the hydraulic conductivity of layers by implementing the Robertson approach.

Chapter 4 introduces 3D geological modeling which can collect all parametric data related to geological surveys. These 3D models then can be used to monitor groundwater pollution by knowing permeability in different directions and taking preventive measures against spreading the pollutants.

Chapter 1

1.1. Test equipment

A CPT equipment consists of an electrical or mechanical cone penetrometer, pushing equipment with rods, cable for mechanical probe or transmission device for electrical probe, cone data acquisition system, and depth encoder. A tilt sensor is usually used to control the verticality of the sounding.

The piezocone (CPTU) test is a cone sounding which provides the measure of cone tip resistance, friction sleeve resistance, excess porewater pressure, and the inclination of the probe.

There can be found probes with porous filters mounted at one of the three positions of the probe (Fig. 1). The most common filter positions comprise cone tip (u_1) or cone shoulder/ (between the base of the cone and the friction sleeve) (u_2) and the scarce position is at the end of the sleeve (u_3).

The CPT manual recommends using the cone with the pore pressure measuring unit located just behind the cone (u_2) for these reasons:

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

Cones come in various sizes, with 10 cm^2 and 15 cm^2 being the most common and standardized. The 15 cm^2 CPT probes are very similar to the 10 cm^2 probes but larger which means stronger probes that can reach higher depths.

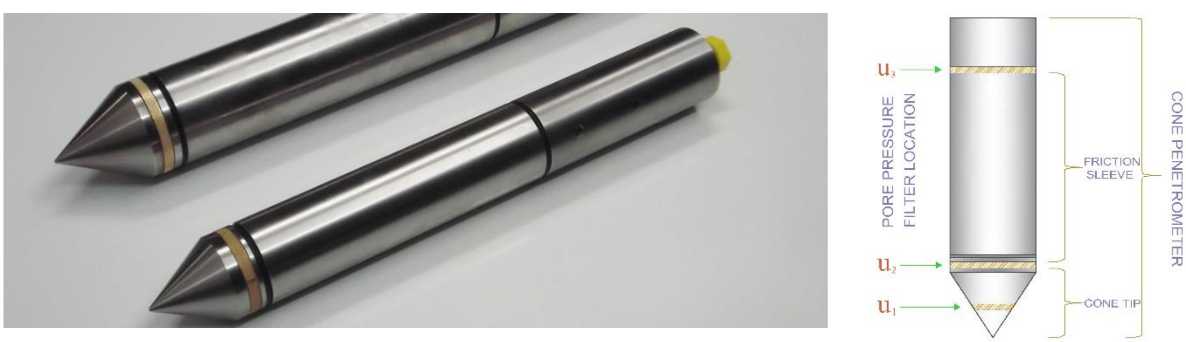


Figure 1 – CPTU probe [i]

The static probing equipment is mounted on a truck (Figure 2) or track (Figure 3) that can operate on land. Usually, the vehicles are very heavy to provide high static forces to perform cone penetration tests.

Figure 2 – CPT truck operated by the Gregg

Figure 3 – CPT on track operated by ConeTec

However, there is over water pushing equipment mounted on the mid-size jack-up boat (Figure 4) or ship with spuds (Figure 5) for shallow depth investigations and crane ship (Figure 6a) underwater CPT equipment (Figure 6b) for deep water investigations.

Figure 4 – Jack-up boat (source: www.greggdrilling.com)

Figure 5 – Gregg Quinn Delta ship with spuds

Figure 6a – Craneship CPT

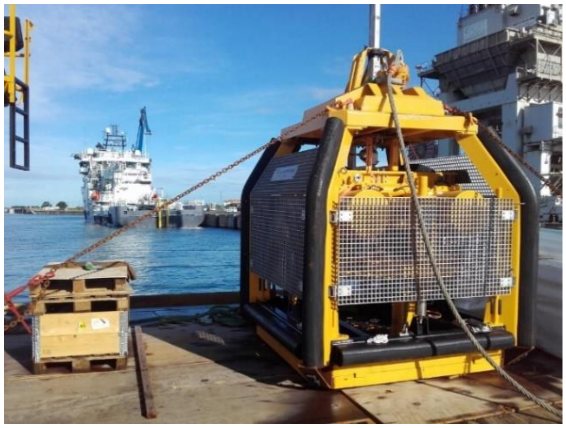


Figure 6b – Underwater penetrating equipment

1.2. CPT Test: Survey methodology and procedure

CPT essentially consists of pushing an electronic probe into the soil at a rate of 2 cm per second. The device is equipped with a load cell of a tip to measure resistance offered by the soil during the intrusion. It is also equipped with a friction sleeve to measure local friction between the surrounding soil and the shaft of the probe. The CPT tip resistance and sleeve friction data can be used to determine soil types along with a variety of engineering soil properties.

Large cone trucks with reaction capacities of 20 tons operate to perform tests. These trucks are used to gather soil information as deep as 60 meters. Besides the 10 cm, cone a larger 15 cm cone other cones are available, to measure pore pressure and electrical resistivity. A cable that is threaded to the push rods transmits the information, hydraulic ram pushes the probe into the ground at an average speed of about 2 cm/s. Usually, up to 60 meters of penetration soundings can easily be made with this equipment and it is much faster than conventional drilling. In addition to that, the thrust machine is placed so that the verticality of sounding can be kept during penetration, and the inclination of the rod during testing is controlled with the slope sensor integrated cones. If the verticality of thrust and straightness of rods are retained, it is unlikely to see the deviation of the rod from the vertical trajectory, except by contact with hard objects. Most electric cone penetrometers have slope sensors, and the use of such cones mainly in stratified soils avoids damage to equipment due to deviation and reduces the errors in deep soundings. The effect of verticality can be seen in Figure 7.

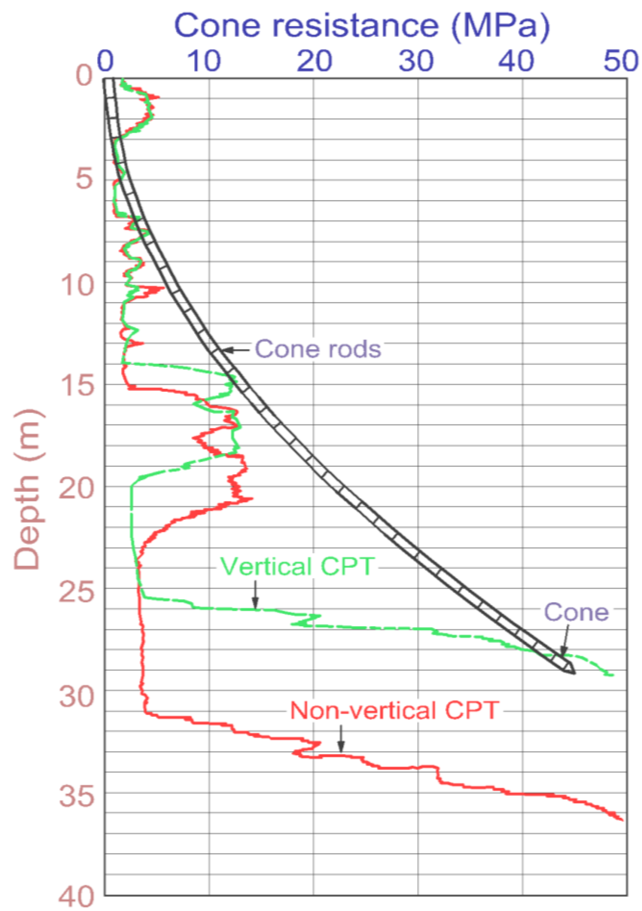


Figure 7 – Effect of verticality on measured depth (from Bruzzi and Battaglio, 1987). [11]

Readings taken every 2 cm are viewed by the operator in real-time on the computer screen. The results are available immediately and can be stored on a computer for use at a later time by design engineers.

Utilization of a CPT during the design phase can drastically decrease the number of soil borings and reduce the cost and time required for subsurface characterization. It is well documented that CPT data can be obtained faster with less cost than conventional soil boring data. CPT soundings taken at closer intervals than traditional borings will provide more details on soil stratification and therefore less potential error.

A continuous intrusion cone penetrometer has been implemented in a four-wheel drive vehicle, the vehicle provides for the leveling and gross weight needed to react during cone penetration. A novel feature of this new in situ testing vehicle is a caterpillar-type continuous push device. It is powered by a hydraulic motor for advancing the cone penetrometer. This one-man operation greatly increases productivity and serviceability. The system may be used to test the natural ground beneath existing pavement through an access hole and even on side slopes. To provide greater mobility and side accessibility the system can also be mounted on smaller four-wheel drive or truck vehicles, a laptop computer is used for data acquisition, processing, and analysis. The computer is equipped with an interface modulus that converts analog input signals to engineering units. As with the larger cones the mini cone can be used to provide a variety of soil properties.

The use of piezocone gives us one more parameter and it measures the pore water pressure.

The use of piezocone remarkably improves the data obtained in static sounding both in terms of measurement precision and data sampling frequency. *"Louisiana Transportation Research Center"* [vi]

Chapter 2

2.1. Geo-environmental applications of penetration testing

The hydraulic conductivity characterization (k) of soil deposits is one of the most important aspects of geo-environmental engineering because it determines the rate of groundwater flow through the subsoil, which controls the advective transport of chemical pollutants. There are many high-quality methods for quantifying the hydraulic conductivity of soil deposits, both in the laboratory and in the field. However, since quantifying in situ hydraulic conductivity using field tests can be time-consuming and costly, an alternative method for determining in situ hydraulic conductivity is proposed based on the results of seismic piezocone tests (SCPTu). [4]

Studies show that in recent years there has been a gradual increase in the number of geo-environmental engineering projects that combine geotechnical engineering with environmental concerns. Most of these projects involve some form of contaminants in the subsoil, which may be in the form of vapors, liquids, and solids. As a result, site characterization procedures have changed in recent years to address these environmental issues related to contaminants.

According to the ASTM standard, most environmental site characterization projects require data on subsurface stratigraphy and hydraulic parameters related to groundwater flow rate and direction. Soil stratigraphy is often determined by various drilling methods and interpretation of data collected in borehole logs. [1]

Drilling methods have been modified to reflect potential ground contamination. However, drilling methods tend to cause significant disturbance to the surrounding materials of the borehole, which can have a significant impact on the subsequent quality of the sample. Nowadays, drilling and sampling techniques are becoming less acceptable due to the increased application of data quality management. In addition, drilling and sampling techniques require material to be cut and removed from the borehole. If these sections are contaminated, special handling and disposal methods may be required.

The direct push technology, which is a penetration test, is the most rapidly developing geo-environmental site characterization technique with the advantages of no cuttings, producing little disturbance, and reduced contact between operator and contaminants because penetrometer push rods can be decontaminated during extraction. [11]

2.2. Goal of a geo-environmental site investigation

The purpose of the geotechnical site investigation is to estimate:

- The nature and sequence of the subsurface layers (geologic regime).
- State of groundwater (hydrogeologic regime).
- The physical and mechanical properties of the subsoil.

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

- The distribution, composition, and concentration of the contaminants.

Depending on the needs of the project, the above studies should be performed in sufficient detail. For geotechnical projects, this usually depends on the proposed structure and the risks involved. The engineer often has control over the risk process and thus the selection of detailed investigations required on site. For geo-environmental projects, the level of detail required to obtain the distribution and composition of contaminants may be controlled by different regulatory authorities, where engineers have limited influence.

When conducting geotechnical investigations, information is often obtained at one point in time, and forecasts are made to determine if conditions will change due to seasonal precipitation. In addition, geo-environmental projects where potential contaminants have been identified may require long-term monitoring and sampling for design and either remediation or containment. Therefore, the goals for geo-environmental site characterization may differ greatly from those of traditional geotechnical site characterization. [11]

2.3. Data interpretation

Wide use of the interpreted data is stratigraphy based on soil behavior types and various charts are available and only normalized CPT Soil Behavior Type (SBT_n) (Robertson) is presented in this document (Figures 8 and 9). The chart in Figure 9, employs tip and friction sleeve resistance data normalized to the estimated in-situ ground stresses. Firstly, the subsurface stratigraphy needs to be defined to estimate the extent and the motion of contaminants. Because contaminants migrate mainly through more permeable layers (e.g., sand), it is impossible to characterize an environmental site without valid stratigraphy. Generally, cone penetrometer data is used as a stratigraphic tool and a pore pressure cone can be used to detect the hydraulic head of the groundwater or to locate perched water zones.

2.4. Soil profiling

Calculations of CPT are based on tip resistance, sleeve friction, and pore pressure considered at each data point. The recorded tip resistance (q_c) is corrected to q_t considering the effects of pore pressure and u_2 pore pressure filter location values are used:

$$q_t = q_c + (1 - a) \cdot u_2 \qquad (1)$$

a – Net Area Ratio for the cone ($a = A_n/A_c$, where A_n – cross-sectional area of load cell or shaft; A_c – projected area of the cone; typical CPT a_n value is 0.7 to 0.8 according to ASTM).

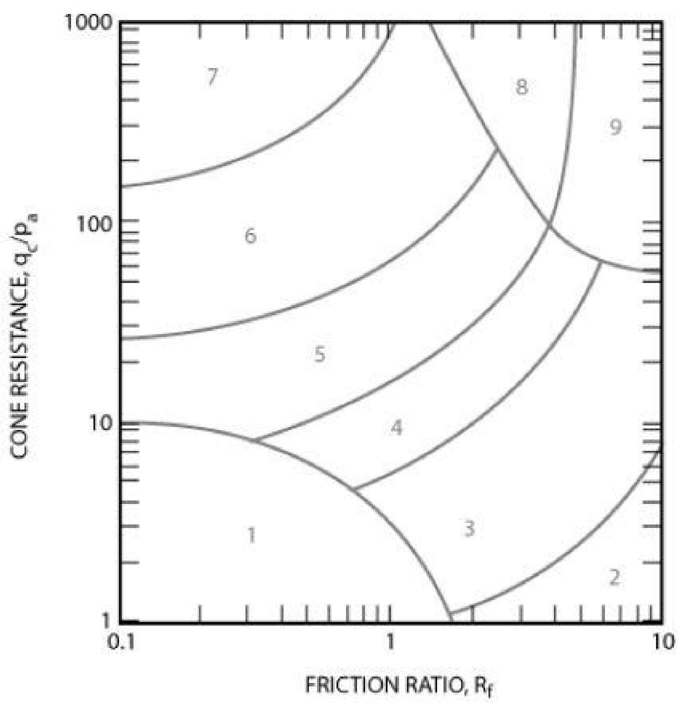
For the cones with equal end area friction sleeves, pore pressure corrections to sleeve friction (f_s), are not required.

The prime use of the CPT is to deliver detailed data about soil profiling and soil type. Generally, sands have high tip resistance compared to clays and a lower friction ratio ($R_f = f_s/q_t$)) than clays.

In the author's opinion, the CPT cannot provide accurate predictions of soil type based on physical characteristics such as grain size distribution but provide a guide to mechanical characteristics (i.e., strength and stiffness) of the soil or the ***soil behavior type*** (SBT). The CPT data come up with a repeatable index of the soil behavior in the immediate zone of the probe during testing. Therefore, soil type predictions based on CPT are usually referred to as Soil Behavior Type (SBT).

2.5. Method of Robertson

The soil classification system groups the soil by its engineering behavior.
There are several classification methods to predict soil type from CPT/CPTu data and one of the most used CPT soil behavior type is the chart by Robertson, which is reported in this document. The updated and dimensionless version of the chart (Robertson, 2010) is shown in Figure 9. The normalized cone resistance Q_{tn} and friction ratio F_r which are the basic CPT parameters plotted on this chart. This chart can give reliable predictions of soil behavior type for cone penetrations of up to 20 meters in depth. In some zones, the overlap is to be expected and zone adjustments should be made based on local experience. [2]



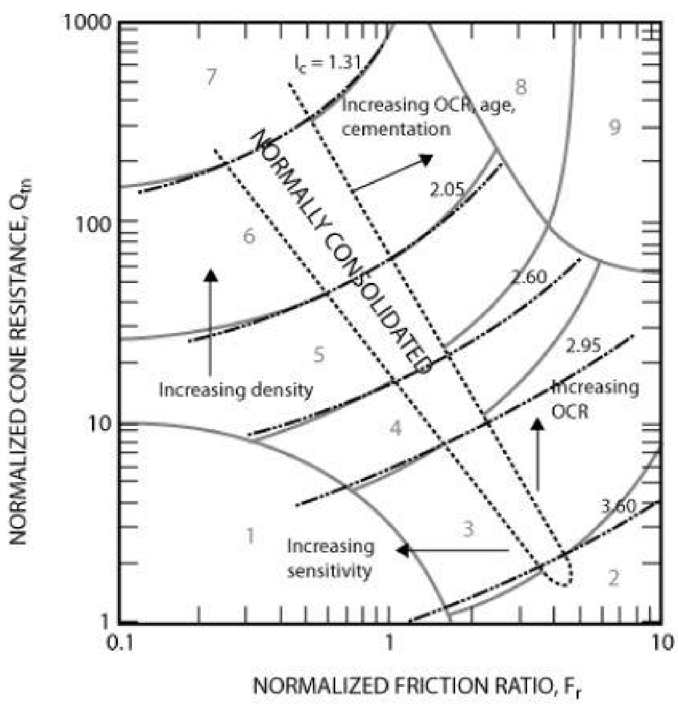
Zone	Soil Behavior Type
1	*Sensitive, fine grained*
2	*Organic soils – clay*
3	*Clay – silty clay to clay*
4	*Silt mixtures – clayey silt to silty clay*
5	*Sand mixtures – silty sand to sandy silt*
6	*Sands – clean sand to silty sand*
7	*Gravelly sand to dense sand*
8	*Very stiff sand to clayey sand**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

*Heavily overconsolidated or cemented

p_a = atmospheric pressure =100 kPa

Figure 8 – CPT Soil Behavior Type (SBT) chart

(Robertson et al., 1986, updated by Robertson, 2010).



Zone	Soil Behavior Type	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6

6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

*Heavily overconsolidated or cemented

Figure 9 – Normalized CPT Soil Behavior Type (SBT_N) chart, Q_t – F (Robertson, 1990). [2]

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

$$I_c = ((3.47 - logQ_{tn})^2 + (logF_r + 1.22)^2)^{0.5} \qquad (2)$$

Normalized cone resistance,

$$Q_{tn} = \left(\frac{q_c - \sigma_{v0}}{p_a}\right)\left(\frac{p_a}{\sigma'_{v0}}\right)^n \qquad (3)$$

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

Normalized friction ratio in %,

$$F_r = \left(\frac{f_s}{q_t - \sigma_{v0}}\right) \cdot 100\% \qquad (4)$$

Hydraulic conductivity (k)

The use of the CPT Soil Behavior Type chart can provide an approximate estimate of the parameter of hydraulic conductivity (k). Table 1 gives the estimates based on Figure 8 and Figure 9, the SBT charts.

SBT Zone	***Soil Behavior Type***	***Range of k (m/s)***	I_c
1	*Sensitive, fine–grained*	$3 \cdot 10^{-10}$ to $3 \cdot 10^{-8}$	N/A
2	*Organic soils – clay*	$1 \cdot 10^{-10}$ to $1 \cdot 10^{-8}$	$I_c > 3.6$
3	*Clay*	$1 \cdot 10^{-10}$ to $1 \cdot 10^{-9}$	$2.95 < I_c < 3.6$
4	*Silt mixture*	$3 \cdot 10^{-9}$ to $1 \cdot 10^{-7}$	$2.60 < I_c < 2.95$
5	*Sand mixture*	$1 \cdot 10^{-7}$ to $1 \cdot 10^{-5}$	$2.05 < I_c < 2.6$
6	*Sand*	$1 \cdot 10^{-5}$ to $1 \cdot 10^{-3}$	$1.31 < I_c < 2.05$
7	*Dense and gravelly sand*	$1 \cdot 10^{-3}$ to 1	$I_c < 1.31$
8	**Very dense / stiff soil*	$1 \cdot 10^{-8}$ to $1 \cdot 10^{-3}$	N/A
9	**Very stiff, fine–grained soil*	$1 \cdot 10^{-9}$ to $1 \cdot 10^{-7}$	N/A

**Overconsolidated and/or cemented*

Table 1 – Estimated soil permeability (k) based on the CPT SBT_N chart by Robertson (2010).

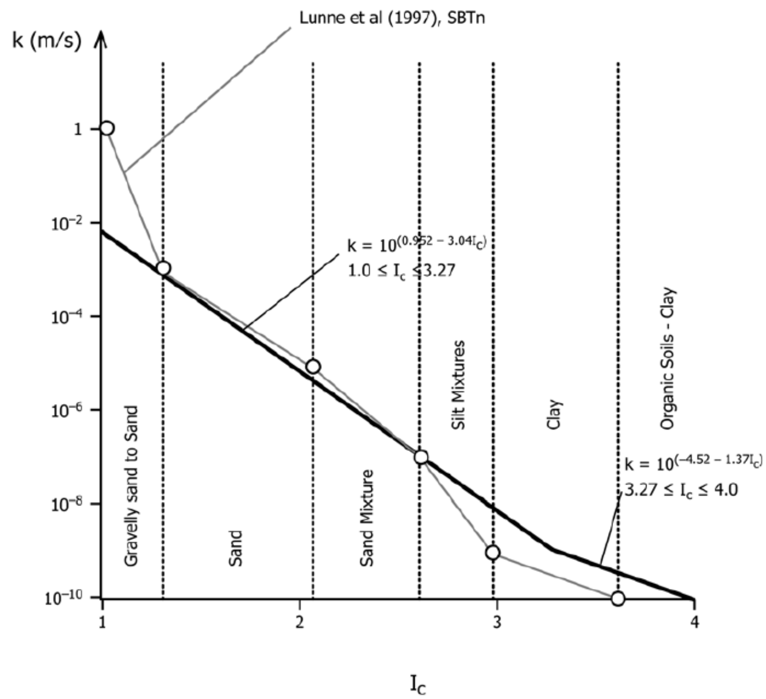


Figure 10 – Proposed Relationship Between *Ic* and Normalized Soil Behavior Type and Estimated Soil Permeability, *k*

The above chart shows suggested variation of soil permeability (k) as a function of SBT I_c

The average relationship between soil permeability (k) and SBT_n I_c shown in Table 1 can be represented as follows:

When $1.0 < I_c \leq 3.27$ then $k = 10^{(0.952-3.04I_c)}$ m/s (5)

When $3.27 < I_c \leq 4.0$ then $k = 10^{(-4.52-1.37I_c)}$ m/s (6)

The above relationships can be used to obtain an approximate estimate of soil permeability (k) and to show the likely variation of soil permeability with depth from a CPT sounding. Because the normalized CPT parameters (Q_{tn} and F_r) are sensitive to the mechanical behavior of the soil and depend on many soil variables, the proposed relationship between k and I_c is an approximation and should be used only as a guide.

2.6. Porewater dissipation tests (using CPT pore pressure dissipation tests to characterize groundwater conditions)

Pore pressure data can also be used to estimate the depth of groundwater, as well as the direction and velocity of groundwater flow. This data is useful for site characterization as well as geo-environmental and remediation applications. [iv]

The dissipation test is the test that measures the decay of porewater pressure in a defined time frame. The rate of dissipation is a function of the consolidation coefficient which consecutively depends on the compressibility and permeability of the soil. Moreover, the rate of dissipation depends on the probe diameter too, meaning that as the probe size increases the dissipation rate decreases.

When penetrating the CPT probe into the ground the excess pore water pressure accumulates around the tip of the cone and it is measured with the porous filter mounted on the probe. The dissipation test results can be obtained at any required depth by pausing the penetration and recording the decay of pore water pressure as a function of time. There is no need for additional setup. Mostly the test is run and the time (t_{50}), to reach 50% of the dissipation is recorded (Figure 11), and it allows for an estimate of geotechnical parameters to assess the hydraulic conductivity to give an idea about the settlement properties of the material. If the request is to reach equilibrium pore pressure, the dissipation test is performed until no further dissipation is detected. Usually, it takes place quickly in sand materials but slowly in plastic clays which may take a lot of hours. However, the results are immediate and allow engineers to make on-site decisions. [14] [i]

With the dissipation of pore water pressure, it is possible to understand the soil behavior under heavy surface loads. [iv]

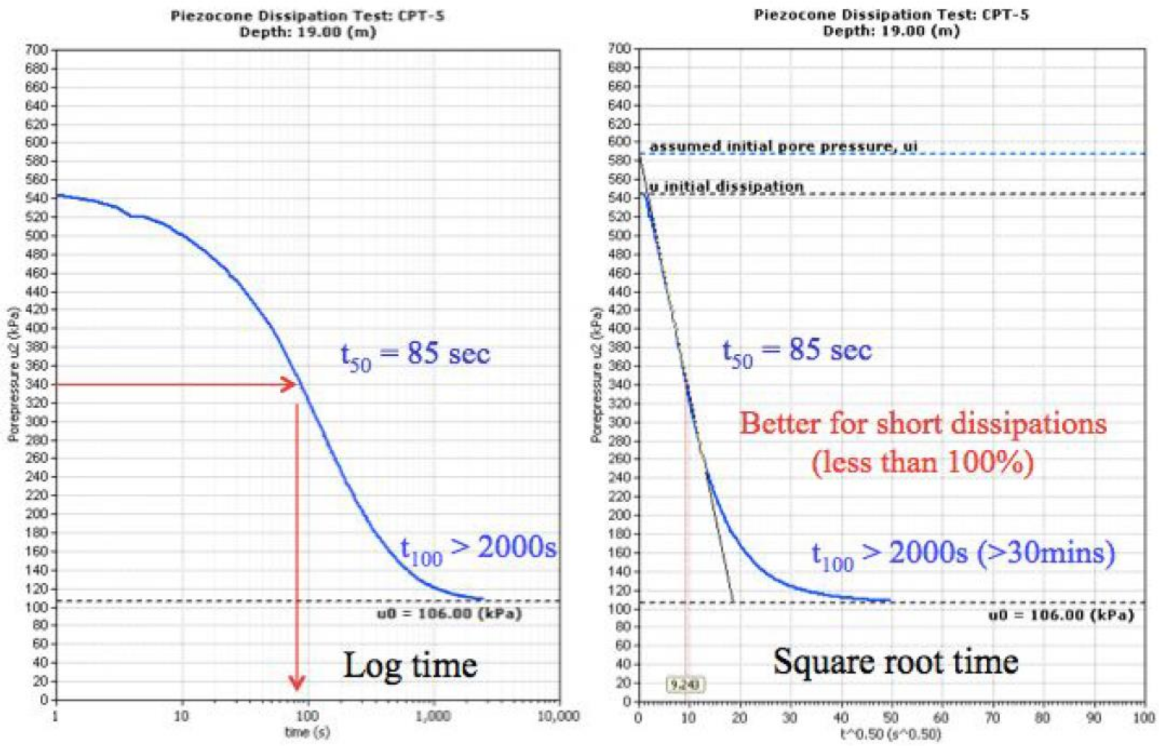


Figure 11 – Example dissipation test to determine t_{50}

The first and simplified approach for the interpretation of dissipation records has been introduced by Torstenson (1975). The soil was considered to be an elastically perfectly plastic material subjected to isotropic initial stress. The initial overpressure was evaluated using the cavity expansion theory (Vesic, 1972), and the consolidation process is pursued using a linear uncoupled one-dimensional theory. [5]

The most recent comprehensive study of this problem was done by Baligh and Levadoux (Figure 12), and suggest computing the consolidation coefficient from:

$$c_h = \frac{TR^2}{t} \qquad (7)$$

where:

R – radius of the pushing rod;

T – time factor, depending on the cone geometry and the location of the filter stone;

The c_h values for 50% excess pore pressure dissipation are representative of horizontal flow in the OC range. To obtain the consolidation ratio in the NC range, the following rule is proposed:

$$c_h(NC) = c_h(OC)\frac{C_r}{C_c} \qquad (8)$$

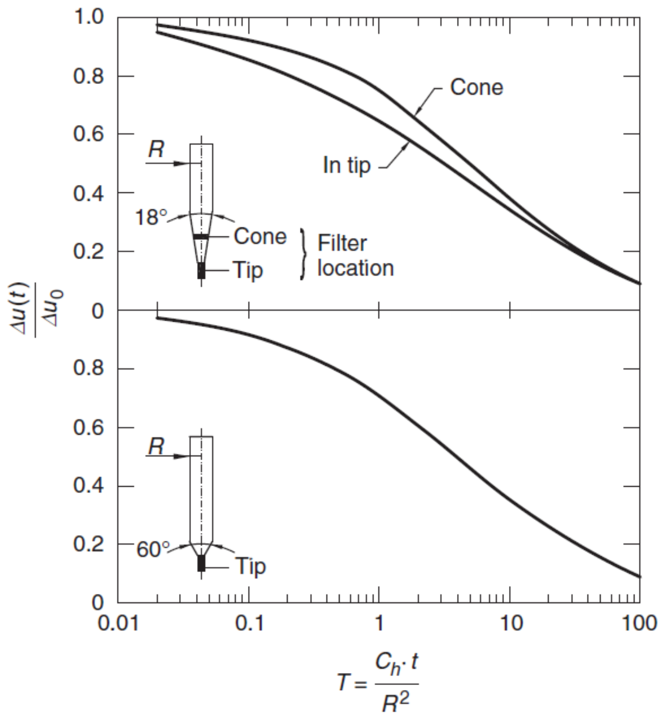


Figure 12 – Pore pressure decay versus time factor (Baligh and Levadoux)

[source: Lancellotta, Renato. *Geotechnical engineering*. CRC Press, 2008]

Generally, soil permeability estimates based on soil type will be approximate but within the correct range. To improve estimates, pore pressure dissipation tests should be conducted in soil layers defined by the CPTu. The pore pressure dissipation is controlled by consolidation coefficient (c_h), which is influenced by the hydraulic conductivity (k_h) and compressibility of soil (M), defined as follows:

$$k_h = (c_h \times \gamma_w)/M \qquad (9)$$

where: M is the one-dimensional constrained modulus;

γ_w – the unit weight of water ($\gamma_w = 9.81\ kN/m^3$);

Using the time for 50% dissipation (t_{50}) from a CPTu dissipation test, Schmertmann (1978), Parez and Fauriel (1988) and Robertson et al (1992) proposed methods for estimating soil permeability (k). As shown in the equation above, these simplified relationships are approximate, since they also depend on soil compressibility (M). An improved estimate of hydraulic conductivity (k) can be obtained by combining an estimate of soil compressibility (M) with the consolidation coefficient which is obtained from the dissipation test.

According to Robertson et al (1992), the coefficient of consolidation in the horizontal direction (c_h) can be approximated by using the following relationship as a function of 50% dissipation time (t_{50}) for a 10 cm^2 cone:

$$c_h = (1.67 \times 10^{-6})10^{(1-logt_{50})}\ m^2/s \qquad (10)$$

An increase of 1.5 is applied to the values of c_h for a 15cm^2 cone.

In a recent review, Robertson (2009) updated the correlation to estimate one-dimensional constrained modulus (M) using the following methods:

$$M = \alpha_M(q_t - \sigma_{v0}) \qquad (11)$$

when $I_c > 2.2$:

$$\alpha_M = Q_{tn} \qquad \text{when } Q_{tn} \leq 14$$

$$\alpha_M = 14 \qquad \text{when } Q_{tn} > 14$$

Note that, in fine-grained soils, where $n = 1.0$, $Q_{tn} = Q_t = (q_t - \sigma_{v0})/\sigma'_{v0}$

By using equations 9,10 and 11 it was obtained the relationship between CPTU t_{50} (min) and hydraulic conductivity of soil (k) for the different values of Q_{tn} and σ'_{v0}, as described in Figure 13.

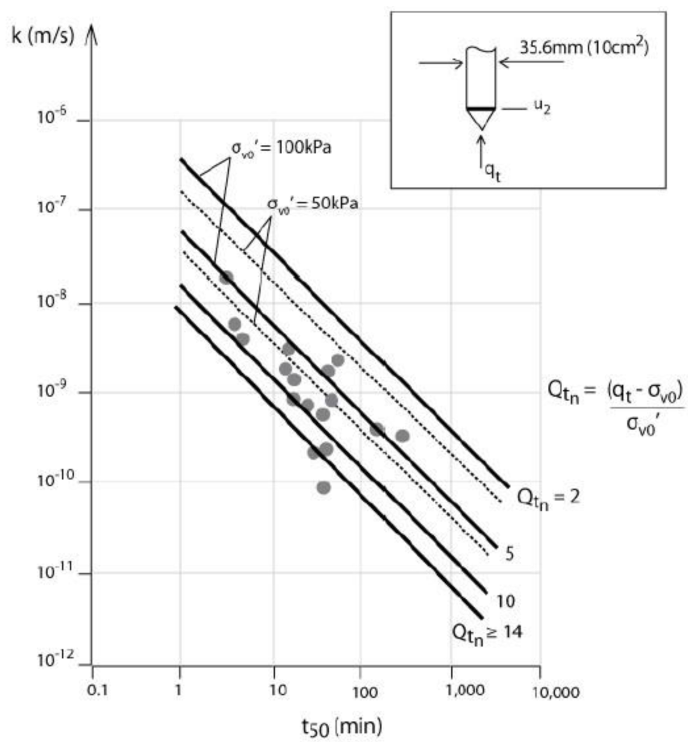


Figure 13 – Proposed relationship between CPTU t_{50} (in minutes) and hydraulic conductivity of the soil (k) and normalized cone resistance (Q_{tn}). [Source: Estimating in-situ soil permeability from CPT and CPTU]

2.7. SCPTU – Seismic Cone Penetration Testing

Besides the advantages of using the CPTU test, we have the possibility of carrying out seismic tests in conjunction with the CPTU. These tests allow simple, low-cost measurement of the shear wave velocity v_s together with tip resistance (q_c), sleeve friction (f_s), and pore pressure (u) which is evident to be reliable appliances to obtain geotechnical parameters. These results can be used to estimate the deformation parameters of soil.

The test uses a geophone integrated probe to measure waves, generated by an impact hammer on a steel plate on the ground surface as a down-hole test.

When the seismic pulse is generated, the time is recorded for the shear wave to reach the geophone at a known distance in the borehole. The distance between the point where the hammer hits and the sounding hole is fixed and usually it is 1.4 meters (Figure 14), and the depth of the geophone is known from sounding.

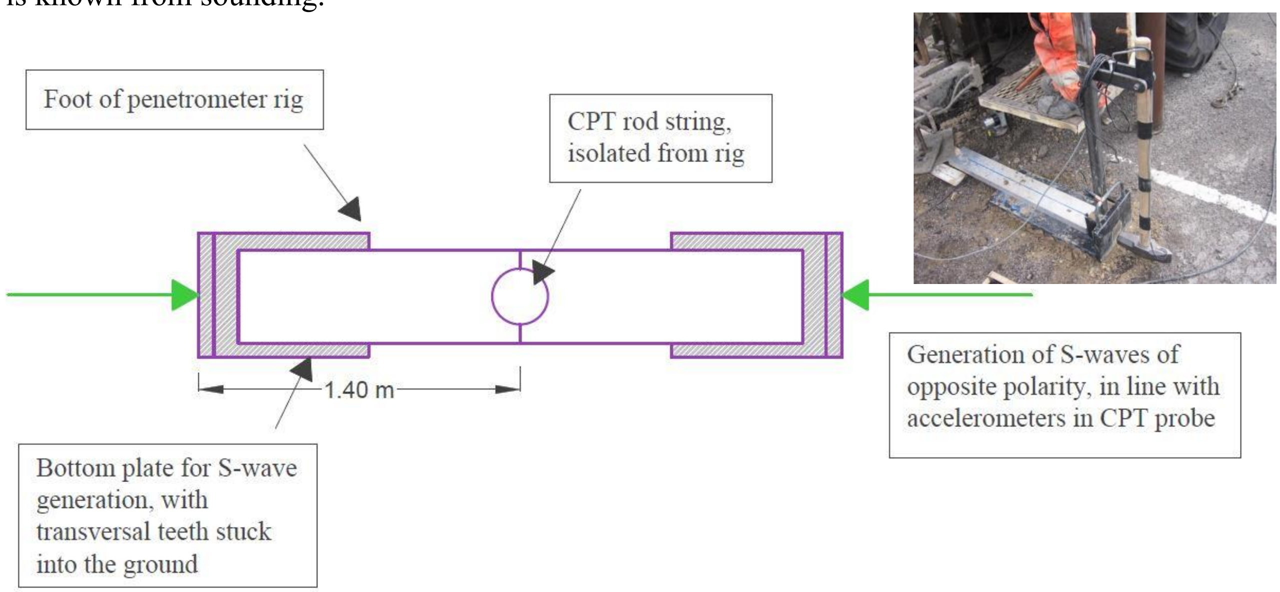


Figure 14 – SCPTU field set up top view and L plate and sledgehammer for S wave generation

To generate S-waves, two plates on the right and left sides of the sounding hole are fixed taking into consideration that the right and the left segment of the S-wave testing are aligned and hit on them with a sledgehammer, respectively. (Figure 15). The sledgehammer and one part of the plate are shown on the right part of Figure 14 above. The plate is L-shaped and has transversal teeth to penetrate the ground and provide better contact.

The seismic test is carried out every meter of the penetration. When the probe reaches the desired depth, the penetration is paused to proceed with the seismic test. The reason to pause the penetration of the probe is, that the SCPTU sounding is noise sensitive. For the quality assessment, the shear velocity can simply be checked on site, once the seismic part of the test is finished the CPT can continue. [13]

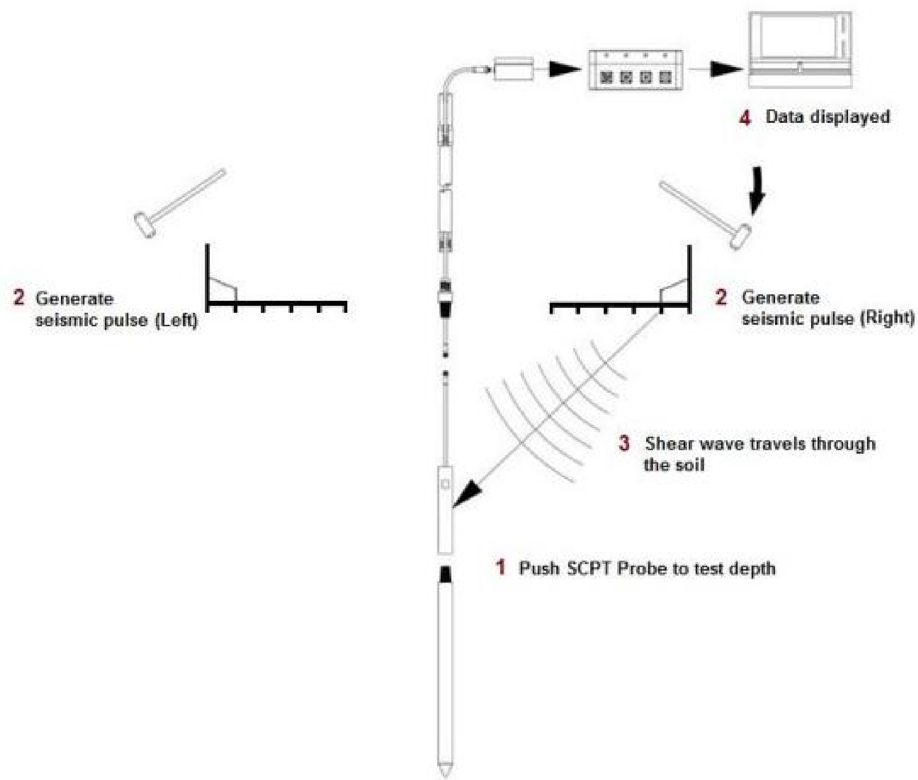


Figure 15 – SCPTU layout of the source-trigger receiver [13]

The cone penetration test with the measurement of pore water pressure (u) and seismic wave velocity (v_s, v_p) is nowadays recognized as the most extensive, and extremely reliable, in situ subsoil test to obtain soil stratigraphy (Figure 16).

As stated by the standard of the test, an electric piezocone is pushed into the subsoil at a constant rate of 2 cm per second, which continuously and every 2 cm penetration depth records these test parameters:

- measured cone resistance q_c,
- sleeve friction resistance f_s, and
- dynamic pore water pressure u_c.

The obtained parameters, q_c, f_s and u are plotted versus the depth of penetration and follow the recommended scales depending on the site and purpose. [11]

If the location of the pore water pressure measuring filter is just above the cone tip, then the general designation u_c can be considered as u_2 (Figure 1).

When the probe penetration stops the dissipation of pore water pressure starts and as time passes the pore water pressure drops.

After pausing the sounding of the probe, the seismic signals are measured at every 1 m of penetration depth and provide the parameters:

- shear wave velocity v_s, and
- compression wave velocity v_p.

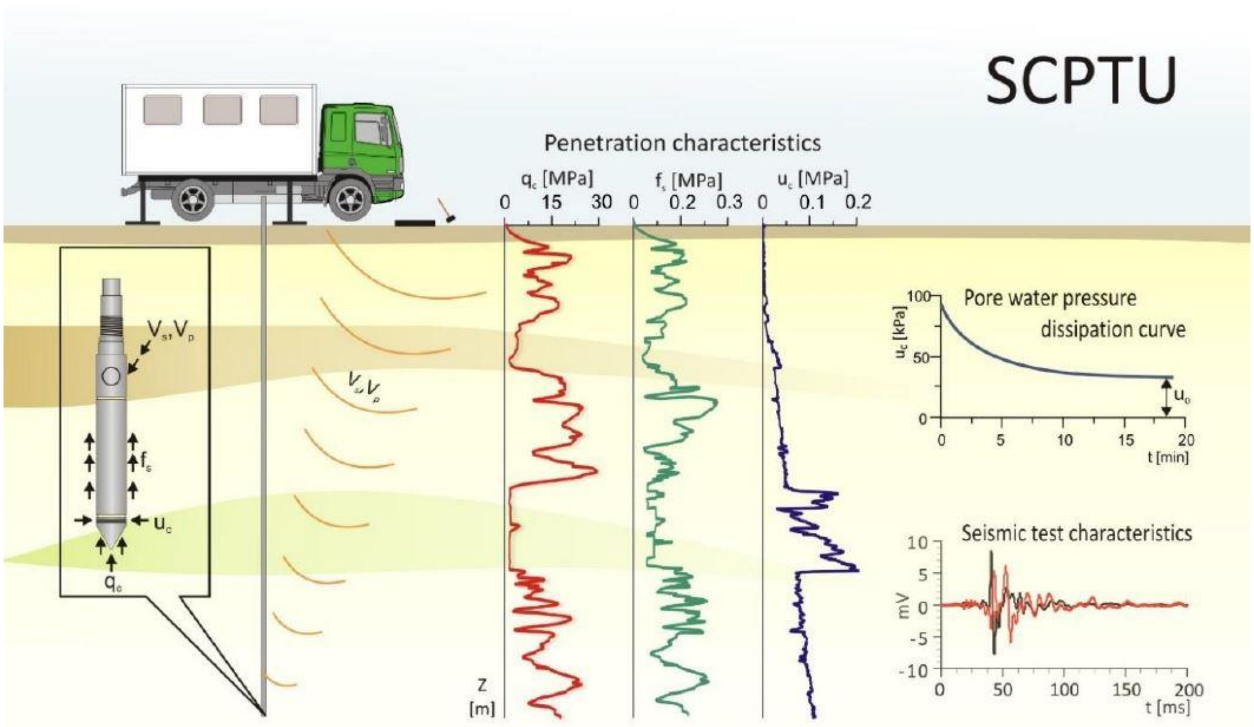


Figure 16 – Principles of seismic piezocone testing. [12]

According to the document by Jamiolkowski, reliable values for compression (V_p) and shear (V_s) wave velocities can be obtained if in-hole techniques such as cross-hole (CH) and down-hole (DH) tests are properly instrumented and performed (Figure 17). [9]

In typical piezocone seismic testing, wave sources are generated at the surface and downhole tests are performed. (Campanella 1994). [4]

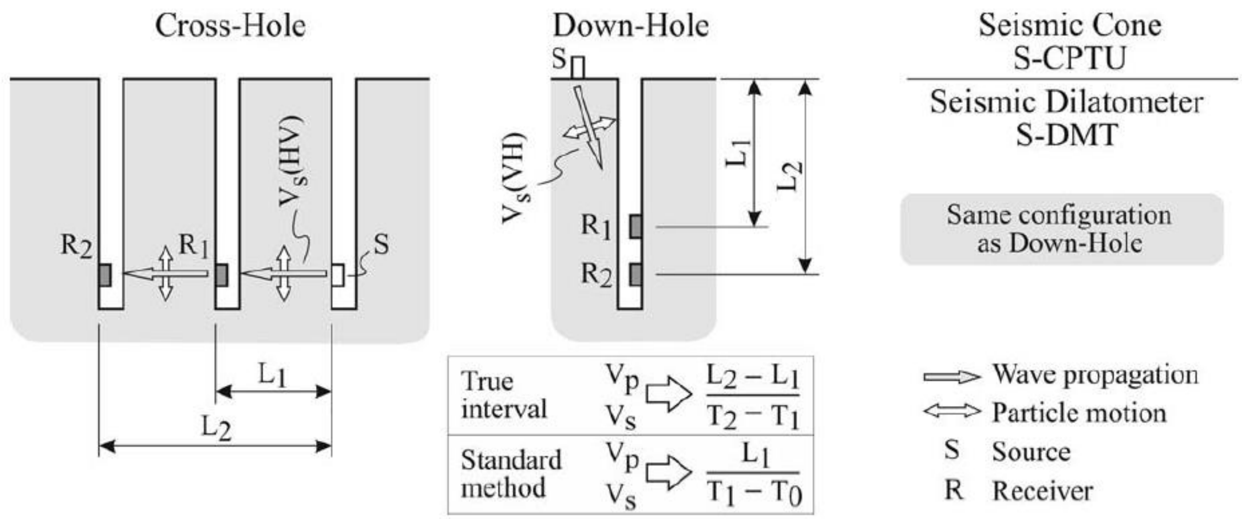


Figure 17 – In-hole geophysical tests. (Extracted from the lecture of Jamiolkowski_De_Mello) [9]

*"When it is only requested the knowledge of V_s, reference will also be made to seismic cone penetration tests (SCPTU) and to seismic Marchetti's dilatometer (S-DMT), equipped to provide a reliable measure of V_s in DH-mode." (*M. Jamiolkowski*)*

The theory of elasticity relates the shear modulus at very small strain (G_0) and constrained modulus (M_0) using:

$$G_0 = \rho_t V_s^2 \qquad (12)$$

$$M_0 = \rho_t V_p^2 \qquad (13)$$

where, ρ_t – bulk soil mass density.

Figure 17 illustrates the main differences and features of CH and DH tests while Figure 18, points out in situ seismic wave propagation and laboratory using bender elements.

The generated shear waves can be characterized by the propagation direction (first capital letter) and the polarization plane (second capital letter).

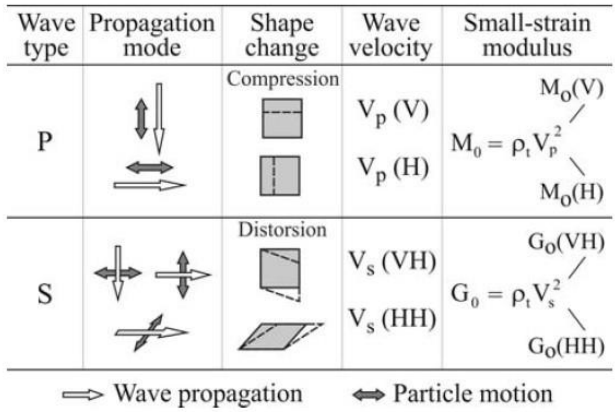


Figure 18 – Compression (P) and shear (S) waves generated in situ and laboratory tests. (Extracted from the lecture of Jamiolkowski_De_Mello) [9]

The research of Jamiolkowski showed that the measurement of compression wave velocity (V_p) in CH tests allowed a simple and reliable distinction between fully saturated and near-saturated tailings. In CH tests compression wave (V_p) measurements enables to determine the saturation line. Since the compression wave velocity in water is known ($V_p \approx 1450\ m/s$), it is easy to accurately indicate if the material is fully or near-saturated and the saturation degree (S_r) plotted against compression wave velocity (V_p) for two types of sand on Figure 19.[7]

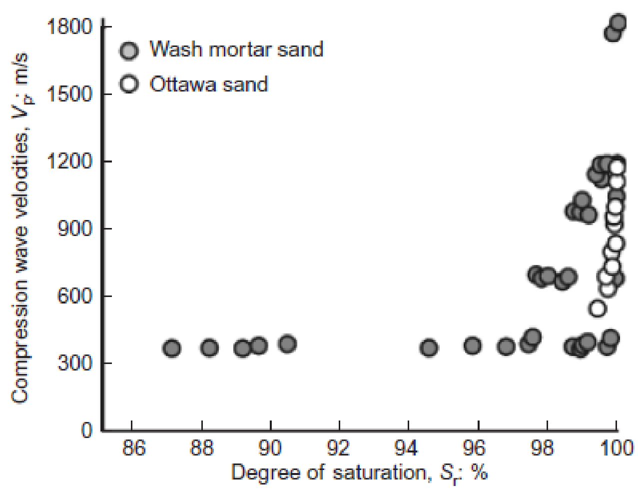


Figure 19 – Saturation degree plotted against compression wave velocity. (Extracted from the lecture of Jamiolkowski, M. (2014). Ge´otechnique 64) [7]

The determination of subsurface water conditions based on CPTU results is a point-by-point evaluation as it relates to the profile depths where the pore water pressure dissipation test was performed. The correct solution to this problem is to move from point identification to zone identification. As part of the cone penetration test, the SCPTU will measure the down-hole seismic wave velocity. The seismic signal is generated from the ground surface, then recorded with a geophone mounted in the seismic cone at the current depth. By using such a measurement method, seismic signals are recorded along a profiling zone limited by successive depths. The compression wave velocity can be used to determine the full and non-full saturation zones of the ground using the elasticity theory. If the compression wave velocity in the subsurface exceeds 1600 m/s, this indicates a two-phase medium in which solid particles and liquid phases are present. There is a substantial reduction in compression wave velocity due to the presence of the third phase indicating air. Figure 20 shows the SCPTU results for the depth profile and the corresponding identified water conditions. Compression wave velocity distributions and pore water pressure dissipation tests indicate that water pressure distributions are similarly based on profile depths.

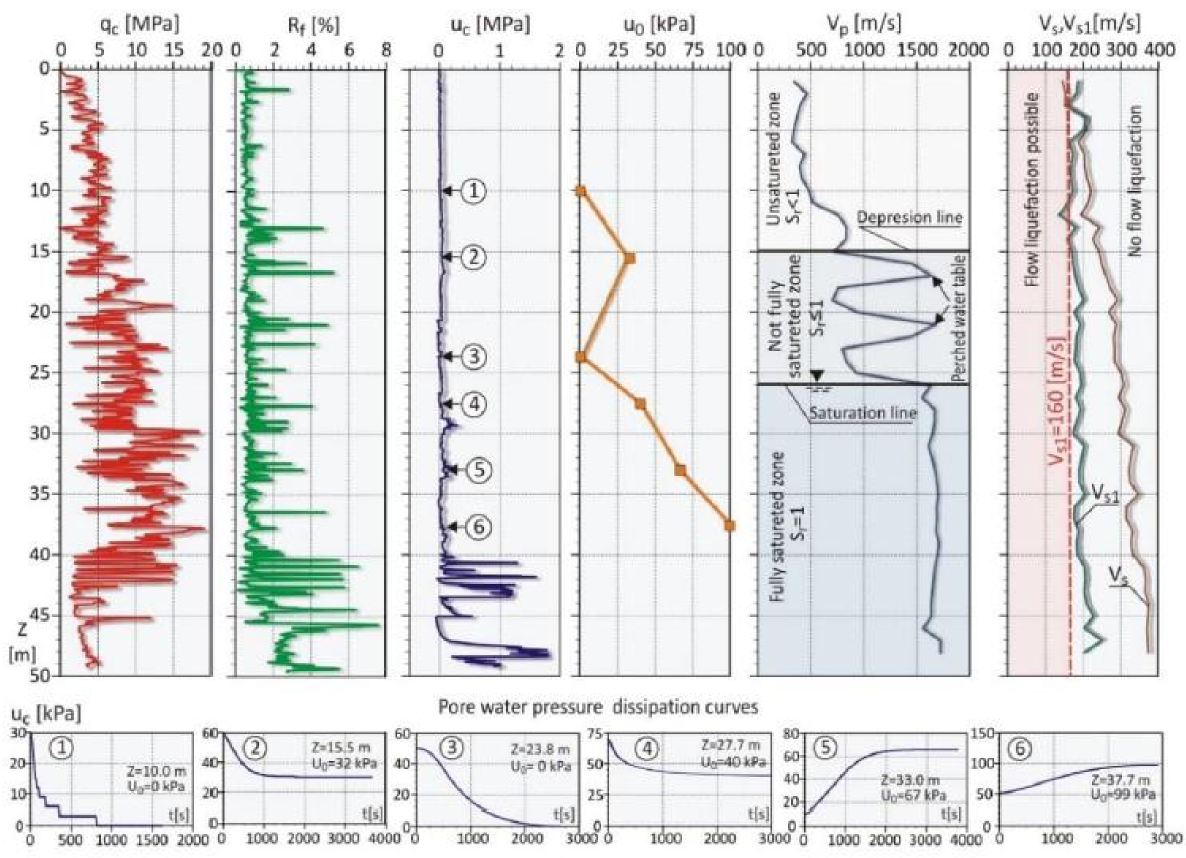


Figure 20 – The complete assessment of water in sediments with the seismic cone penetration test (SCPTU). [12]

According to the sediment profile, three main zones can be identified:

- Unsaturated zone: above the depression line (from the surface to a depth of 15 m), with a saturation degree $S_r < 1$.
- Not fully saturated zone: between the depression and saturation lines (from 15 m to 26 m), with a saturation degree $S_r \leq 1$.
- Fully saturated zone: below the saturation line (below the depth of 26 m), with a saturation degree, $S_r = 1$. [12]

2.8. Methodology of interpretation of seismic piezocone test results to estimate hydraulic conductivity in clays

According to Burns and Mane, by using shear wave velocity, it is possible to develop predictive relationships between V_s, depth, and soil mass density. The hydraulic conductivity of soil deposits can be evaluated with the help of a single seismic piezocone sounding. Hydraulic conductivity for one-dimensional radial drainage can be determined by the following relationship based on a downhole seismic test of the shear wave velocity and a pore water pressure dissipation test of the consolidation coefficient:

$$k_h = \frac{c_h \cdot \gamma_w}{M_0} \qquad (14)$$

According to equation (14), it is necessary to assess the consolidation coefficient and the constrained modulus. In seismic piezocone testing, the coefficient of consolidation can be determined from the dissipation curve of pore water pressure, and the constrained modulus can be calculated from the shear wave velocity. This methodology follows a four-step process:

- The Burns and Mayne (1998) method is used to estimate the coefficient of consolidation based on pore pressure dissipation tests.
- With a downhole seismic piezocone test, assess the velocity of the shear waves.
- A correlation between the shear wave velocity and the constrained modulus can be used to determine the constrained modulus.
- The one-dimensional hydraulic conductivity can be determined using equation (14).

An excess pore water pressure will decay during a dissipation test depending on several factors, such as the consolidation coefficient (c_h) of the soil deposit. There have been a variety of models proposed to evaluate c_h from piezocone dissipation tests. (The models that have been developed using cavity expansion theory (Torstensson 1977; Battaglio et al. 1981), strain path method (Baligh and Levadoux 1986; Houlsby and Teh 1988), empirical methods (Sully and Campanella 1994), and other approaches (Tumay et al. 1982)

Burns and Mayne (1998) developed a model which uses the hybrid concept of cavity expansion theory and critical state soil mechanics.

Using cavity expansion, the normal stress-induced pore water pressure is determined, and the shear stress-induced pore water pressure is determined using critical state soil mechanics.

The constrained modulus is evaluated through the measurements of shear wave velocity using the following equation:

$$M_0 = 0.011 \cdot V_s^{2.43} \qquad (15)$$

When equation (15) is substituted for equation (14), hydraulic conductivity is derived as follows:

$$k_h = 90.9 \frac{c_h \cdot \gamma_w}{V_s^{2.43}} \qquad (16)$$

Using this equation, the hydraulic conductivity was calculated for ten clay deposits with available reference values. Table 2 presents the soil deposits with the obtained hydraulic conductivities.

Test Site	Depth (m)	u_o (kPa)	σ_{vo}' (kPa)	OCR	ϕ'	q_t (kPa)	u_2 (kPa)	c_h[1] (mm^2/s)	V_s (m/s)	Predicted k (cm/s)	[2]Average Lab k_h (cm/s)	[2]Range Lab k_h (cm/s)	References
Amherst, MA (Noncrust)	12.2	120	73.2	1.8	33	690	449	0.83	145	4.1×10^{-7}	6.0×10^{-7}	3×10^{-7} to 1×10^{-6}	(DeGroot and Lutenegger 1994; Lally 1993; Lutenegger 1995; Martin and Mayne 1997)
Bothkennar, U.K.	12.0	107.8	96.2	1.4	33	898	499	0.2	130	1.3×10^{-7}	1.8×10^{-7}	4.8×10^{-8} to 3.3×10^{-7}	(Jacobs and Coutts 1992; Nash et al. 1992)
Drammen, Norway	19.5	179.0	121.0	1.1	34	1000	400	0.2	200	4.6×10^{-8}	1.0×10^{-8}	-	(Lacasse and Lunne 1982)
McDonald Farm, B.C.	20.0	181.5	178.5	1.1	35	1036	600	1.9	180	5.6×10^{-7}	4.0×10^{-7}	-	(Robertson et al. 1988; Sully 1991)
Onsøy, Norway	18.5	159.4	114.5	1.4	34	754	450	0.05	130	3.3×10^{-8}	1.3×10^{-7}	3×10^{-8} to 5×10^{-7}	(Lacasse and Lunne 1982)
St. Alban, Quebec	4.6	40.2	42.6	1.2	27	300	160	0.6	100	7.4×10^{-7}	3.0×10^{-7}	2×10^{-7} to 4×10^{-7}	(Roy et al. 1981; Roy et al. 1982)
Amherst, MA (Crust)	3.0	16.7	38.2	7.0	30.5	1369	80	0.4	215	7.7×10^{-8}	6.0×10^{-7}	3×10^{-7} to 1×10^{-6}	(DeGroot and Lutenegger 1994; Lally 1993; Lutenegger 1995; Martin and Mayne 1997)
Brent Cross, U.K.	12	115	117.8	31	20	2200	100	0.0067	285	6.5×10^{-10}	-	3×10^{-11} to 1×10^{-9}	(Burland and Hancock 1977; Lunne et al. 1986; Lunne et al. 1985)
Cowden, U.K.	17.2	95.0	283.4	3.4	24	898	575	0.2	186	5.4×10^{-8}	2.9×10^{-8}	1.93×10^{-8} to 4.44×10^{-8}	(Lehane and Jardine 1994; Lunne et al. 1985)
Madingley, U.K.[3]	5.8	37.2	72.8	35.0	26	2000	200	0.05	256	6.3×10^{-9}	1.4×10^{-9}	7.61×10^{-10} to 1.74×10^{-9}	(Coop and Wroth 1989; Lunne et al. 1986)

[1] from (Burns and Mayne 1998; Burns and Mayne in review)
[2] (Robertson et al. 1992)
[3] 80 mm Pile

Table 2 – Database of clay sites for seismic piezocone evaluation of hydraulic conductivity.

2.9. Using probes with additional sensors and devices when testing soils by the static probing method

Static soil probing (CPT) using electric probes, which is the main field method of testing soils in the modern practice of engineering and geological surveys, is performed by static continuous or intermittent immersion of a probe into the soil, consisting of a rod and a special electric (usually strain gauge) tip, located in the lower part of the probe and used to measure the resistivity of the soil under the cone q_c [MPa], and on the side surface area (friction sleeve) f_s [MPa], of the probe tip.

In recent years, special electrical probes have become widespread, which differ from standard probes in that they have additional devices and sensors (pore pressure, temperature, radioactive logging, electrical resistance, a seismic sensor, inclinometer, etc.), which allow measuring additional soil characteristics or controlling the sounding process. Several publications

[15,16,17] provide more than twenty types of additional sensors and devices. Special probes can be classified according to:

• purpose:

• geotechnical applications,

• geo-environmental application,

• multifunctional application,

• constructive principle,

• with a single-module tip (additional sensors and devices are located inside the main tip);

• with two or more modular tips (additional sensors and devices are located in a separate module or modules located above the main tip).

Depending on the project, additional data is needed in addition to the information from the CPT and CPTU. Modern electronics, sensor technology, and data acquisition systems enable this to be accomplished. [11]

Geotechnical applications		
Sensors (devices)	Measurements	Applications
Accelerometer/Geophones	Velocities of longitudinal and transverse elastic waves	• Modulus of soil deformation at small deformations • Soil shear modulus at small strains
Acoustic device	Acoustic emission	• Soil type • Compressibility of soil • Soil structure
Lateral stress	Normal pressure on the lateral/side surface of the probe	• Assessment of the natural stress state of the soil
Vibratory module	Soil resistance to probing when the probe is crushed by vibration	• Sand liquefaction possibilities
Video	Video image of the soil in the process of sounding	• Soil particle size • Soil stratigraphy

Gamma radiation	The intensity of natural gamma-radiation	• Natural soil radioactivity • Clay content of dispersed rocks
Gamma – gamma radiation	The intensity of secondary gamma radiation	• Soil density
Inclinometer	Probe verticality	• Probe damage prevention • Correction of immersion depth of the probe
Vane	Torque	• Soil shear resistance • Sensitivity (structural strength) of the soil
Neutron – neutron radiation	Loss of neutron energy during the process of their scattering in soil	• Soil moisture
Pore water pressure	Pore water pressure	• Porewater pressure • Soil type • Consolidation coefficient of the soil • Soil filtration coefficient (and etc.)
Pressuremeter module	Radial deformations	• Soil deformation modulus • Soil shear modulus • Horizontal stresses in the soil • Soil shear resistance • Strength characteristics of soil
Time domain reflectometer	The dielectric constant of a pulsating electromagnetic wave	• Correlation with soil moisture
Temperature	Probe temperature when moving and stopping	• Natural temperature of the soil • Soil type assessment • determination of the state of the soil (thawed, frozen) • Thermal properties of soil
Temperature and heat	The temperature of the probe when it is heated	• Thermal properties of soil
	Heating and changing the resistance of thawed soil to sounding	• Assessment of the mechanical properties of frozen soils during their thawing

Electrical resistivity	The current strength in the soil between insulated electrodes	• Electrical conductivity of soil • Soil type • Soil corrosiveness • Soil porosity • Determination of the groundwater level

Table 3 – Additional sensors and devices used in CPT for geotechnical purposes.

Geoecological applications		
Sensors (devices)	Measurements	Applications
1	2	3
Hydrogen indicator sensor (pH)	Hydrogen ion concentration	• Acid spills • Origin of the straits
Gamma – radiation	The intensity of natural gamma – radiation	• Zones of radioactive contamination
Dielectric constant sensor (HIM – probe)	Dielectric permeability of the soil in an alternating electric field	• Contaminants in the form of organic liquids
Integrated optoelectronics	Chemical concentration using wave interference	• Ammonia detection • Determination of pH
Laser-induced fluorescence (LIF)	Fluorescence of hydrocarbon contaminants	• Contamination of petroleum products (fuel, gasoline, oil, lubricants) that can fluoresce
Redox Potential	Redox potential	• Monitoring the situation during the period of biorecovery of hazardous waste
Raman Spectroscopy	Argon ion concentrations induced by laser fluorescence	• Contamination in the form of organic liquids • Detection of chlorinated hydrocarbons

Temperature	Probe temperature	• Endothermic/exothermic activity resulting from chemical reactions • Identification of the zones of influence of thermal waters • Identification of zones of violation of the groundwater regime due to leaks from water-bearing communications
Electrical Resistivity	Current strengths in the soil between insulated electrodes	• Saltwater penetration • Acid spills • Mineralization of groundwater

Table 4 – Additional sensors and devices used in static sounding for geoecological purposes.

Special probes are used to study special soil conditions and specific soils. Nowadays, probes with additional pore pressure sensors (CPTU) and seismic sensors (SCPT, SCPTU) are most often used.

Installation type	The ultimate force of penetration and extraction of the probe	Soil resistance ranges		
		qc, MPa	fs, kPa	Qs, kN
Easy/soft	Up to 50 kN inclusive	0.5 – 10	2 – 100	0.5 – 10
Medium	Over 50 to 100 kN inclusive	1 – 30	5 – 200	1 – 30
Hard	Over 100 kN	1 – 50	10 – 500	2 – 60

Table 5 – The probe penetration and resistance ranges of soil.

2.9.1. Advantages of static probing compared to alternative methods

- Time-saving and continuous profiling
- Economical and productive
- No errors depending on the operator
- Analysis of liquefaction
- Best for soft soil
- Quiet, no vibrations, minimal site disturbance [i]
- Detailed, high precision, repeatable results [i]

In addition to the advantages of the cone penetration testing for stratigraphic investigations these capabilities of the testing can be added: [10]

- 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

2.9.2. Disadvantages and limits of static probing

- No soil samples for laboratory tests
- Inappropriate for gravely and rocky materials
- Knowledgeable operator to perform sounding [ii]

The application of CPT for the identification of lithotypes and stratigraphical boundaries has complications due to some restraints: [10]

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

Chapter 3

3.1. Objective/scope

This chapter aims to give estimates on the soil behavior type index, undrained shear strength, and hydraulic conductivity for hydrogeological characterization from the given SCPTU test results. The horizontal and vertical parameters of the hydraulic conductivity of the subsoil layer were calculated using various test results with the above methods.

3.2. Soil investigation data

A site investigation was conducted near the city of Ravenna (Italy) and the experimental data obtained are available.

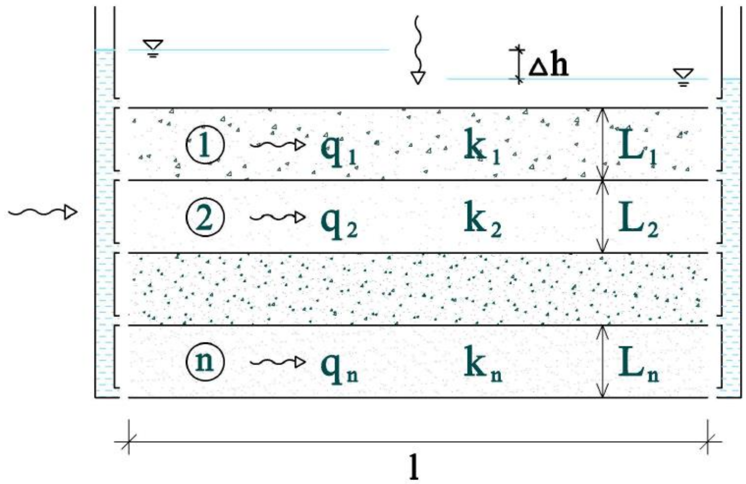
2 seismic cone penetration tests with piezocone (SCPTU-01 and SCPTU-02). SPCTU-01 was located near BH-01, whereas SCPT-02 was located at a distance of 200 m from BH-01.
The groundwater table was located at a depth of 1 m from ground level. The ground surface is flat and topographic level changes are negligible. The average unit weight of the soil, γ, was found to be equal to 19 kN/m^3 from laboratory tests.

3.3. Method to assess the hydraulic conductivity in the horizontal and vertical directions

The coefficient of permeability is calculated by Darcy's law.

Permeability in the horizontal direction:

$$q = -k\left(\frac{\partial h}{\partial x}\right)$$



$$q = k_{eq}\frac{\Delta h}{l}$$

$$q \cdot L = q_1 \cdot L_1 + q_2 \cdot L_2 + \ldots + q_n \cdot L_n = k_1\frac{\Delta h}{l}L_1 + k_2\frac{\Delta h}{l}L_2 + \ldots + k_n\frac{\Delta h}{l}L_n$$

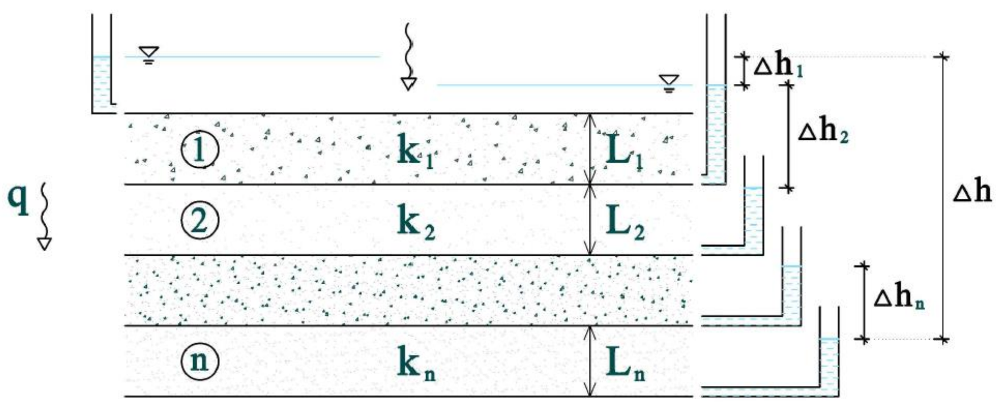
An equivalent hydraulic conductivity in the horizontal direction:

$$k_{eq} = \frac{\sum_{i=1}^{n} k_i L_i}{L}$$

L_i – is the thickness of the small sublayer related to the cone penetration test data

k_i – is the corresponding hydraulic conductivity obtained using the Robertson method.

Permeability in the vertical direction:



$$q = q_1 = q_2 = \ldots = q_n$$

$$q = k_{eq}\frac{\Delta h}{L}$$

$$L = \sum L_i$$

$$q_1 = k_1\frac{\Delta h_1}{L_1} \qquad \frac{L_1}{k_1}q_1 = \Delta h_1$$

$$q_2 = k_2\frac{\Delta h_2}{L_2} \qquad \frac{L_2}{k_2}q_2 = \Delta h_2$$

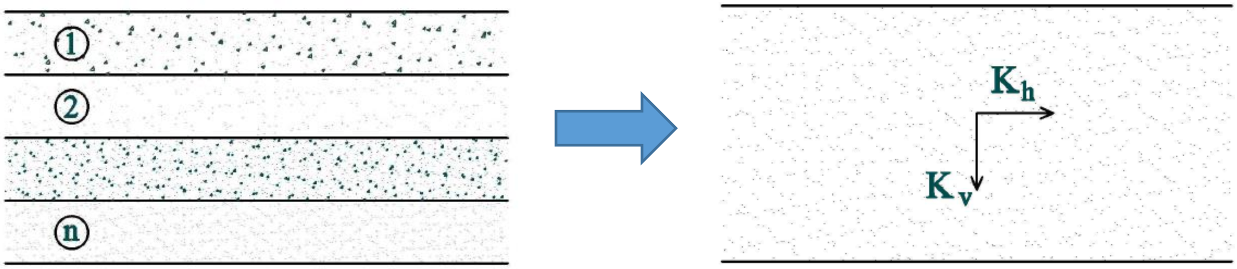
$$\ldots$$

$$q_n = k_n\frac{\Delta h_n}{L_n} \qquad \frac{L_n}{k_n}q_n = \Delta h_n$$

$$\left(\sum_{i=1}^{n}\frac{L_i}{k_i}\right)\cdot q = \Delta h \quad \Longrightarrow \quad q = \frac{1}{\sum_{i=1}^{n}\frac{L_i}{k_i}}\cdot \Delta h$$

An equivalent hydraulic conductivity in the horizontal direction:

$$k_{eq} = \frac{L}{\sum_{i=1}^{n}\frac{L_i}{k_i}}$$



Thus, if we have a stratified soil (with small horizontal layers), we can obtain an equivalent homogeneous soil from a single layer, whose hydraulic conductivity in the horizontal and vertical directions is given by:

$$k_h = \frac{\sum_i k_i L_i}{L} \qquad k_v = \frac{L}{\sum_i \frac{L_i}{k_i}} \qquad k_h \neq k_v$$

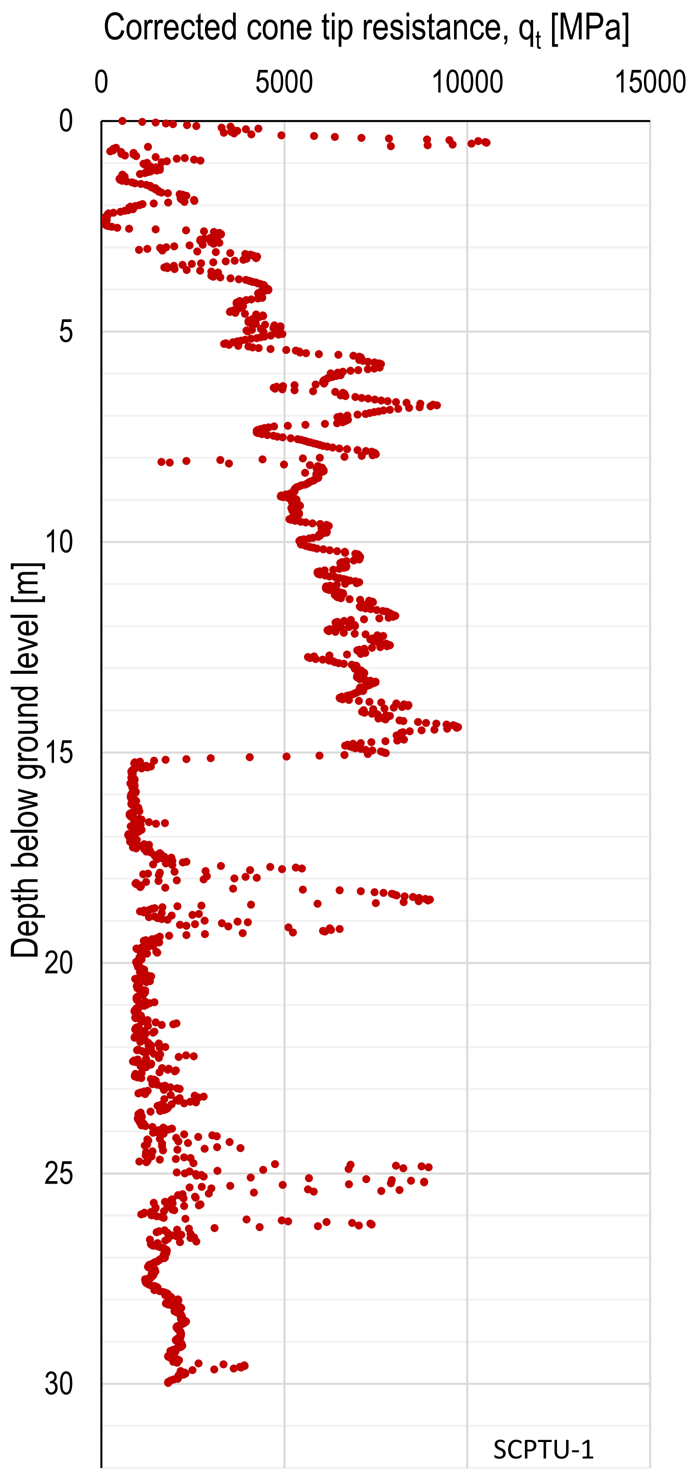
3.4. Method applied to SCPTU test results

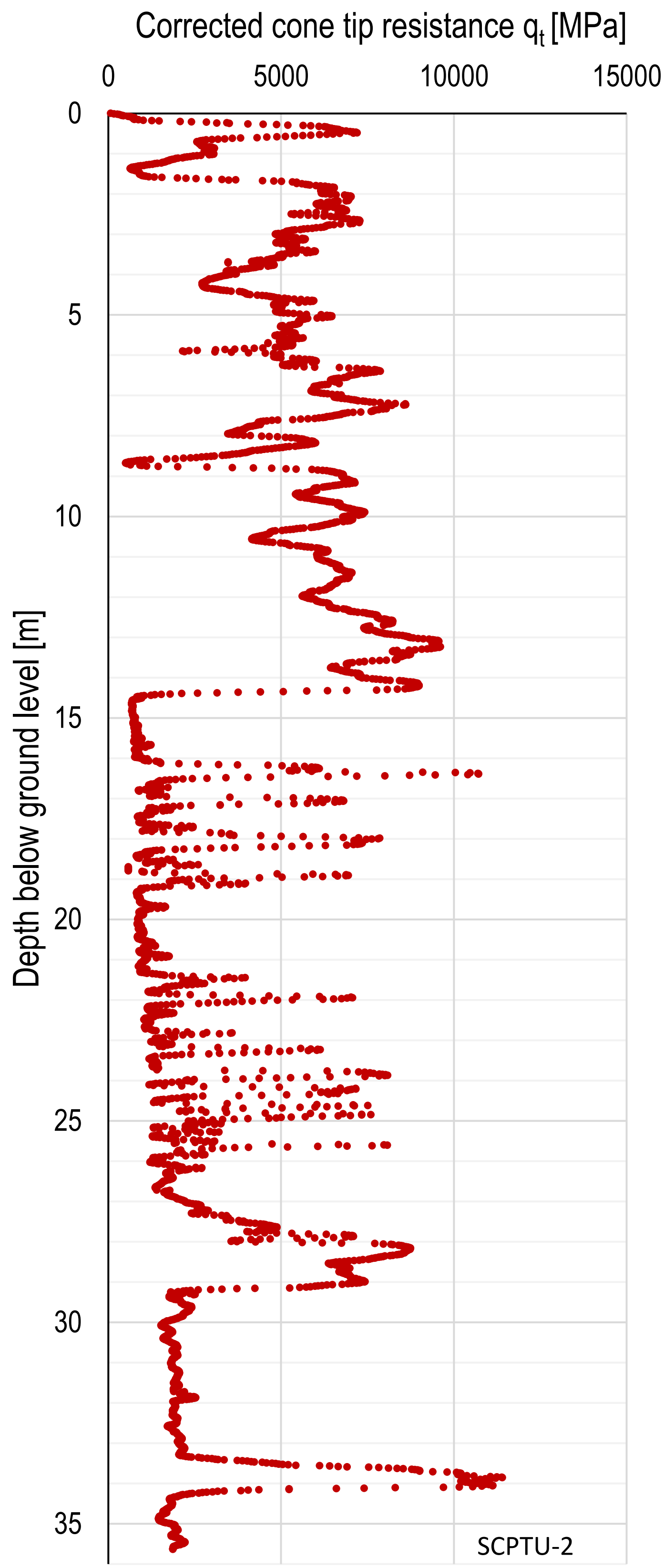
Soil profiling and stratigraphy based on field tests and determination of Soil Behaviour Type Index, Ic, from SCPTU data.

For the geotechnical analysis, some tests will be carried out that will provide relevant information to classify the soil and determine the profile in which the structures will be built.

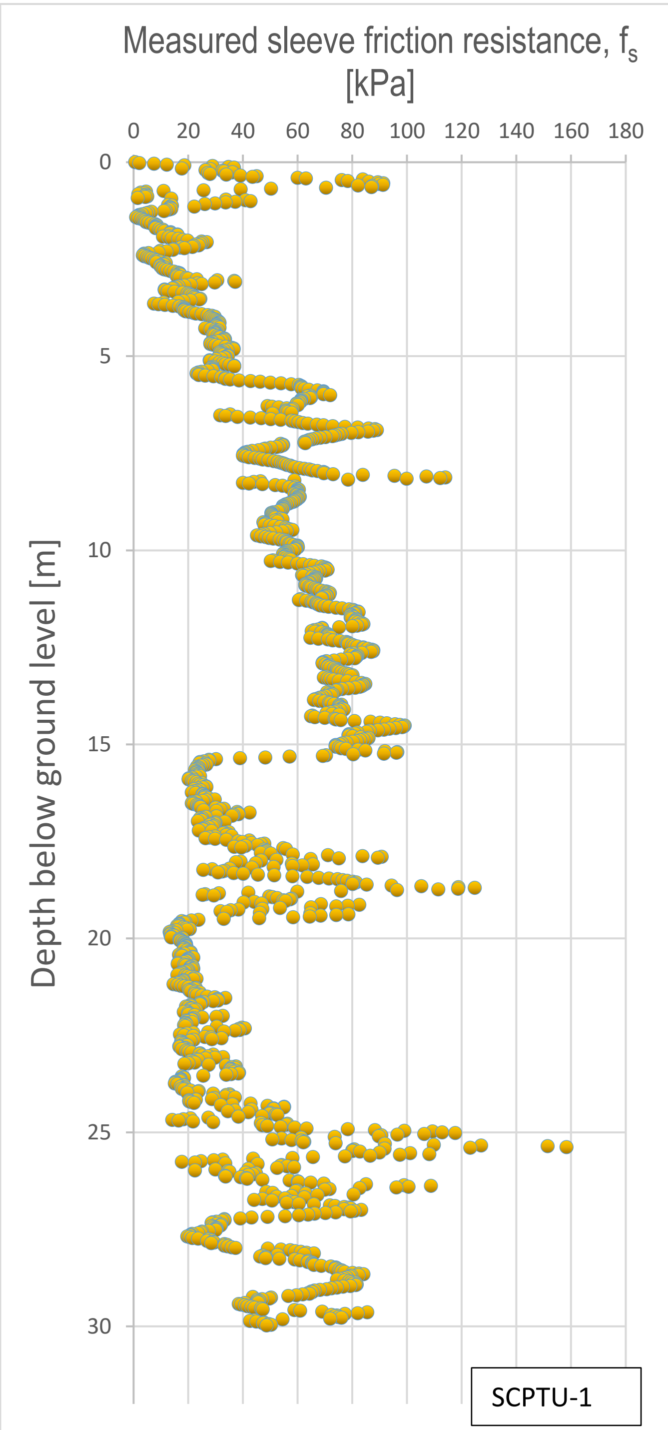
For this point, an in-situ test is used to estimate the stratigraphy of the soil, namely the cone penetration test, which gives an overview of the stratigraphy of the soil to be analyzed.

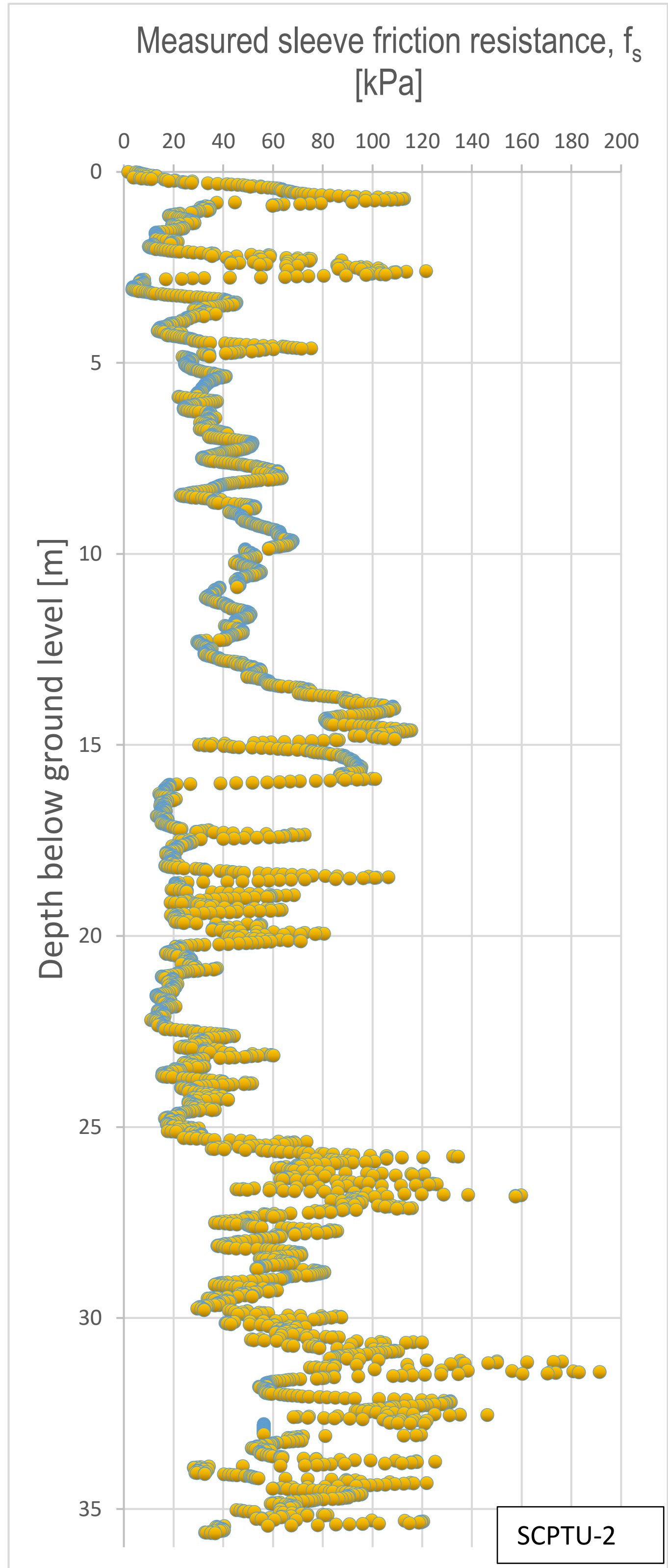
Corrected tip resistance (q_t) is obtained using equation (1): SCPTU-1 (left figure) and SCPTU-2 (right figure)


Corrected cone tip resistance, q_t [MPa]
0
5000
10000
15000
Depth below ground level [m]
0
5
10
15
20
25
30
SCPTU-1

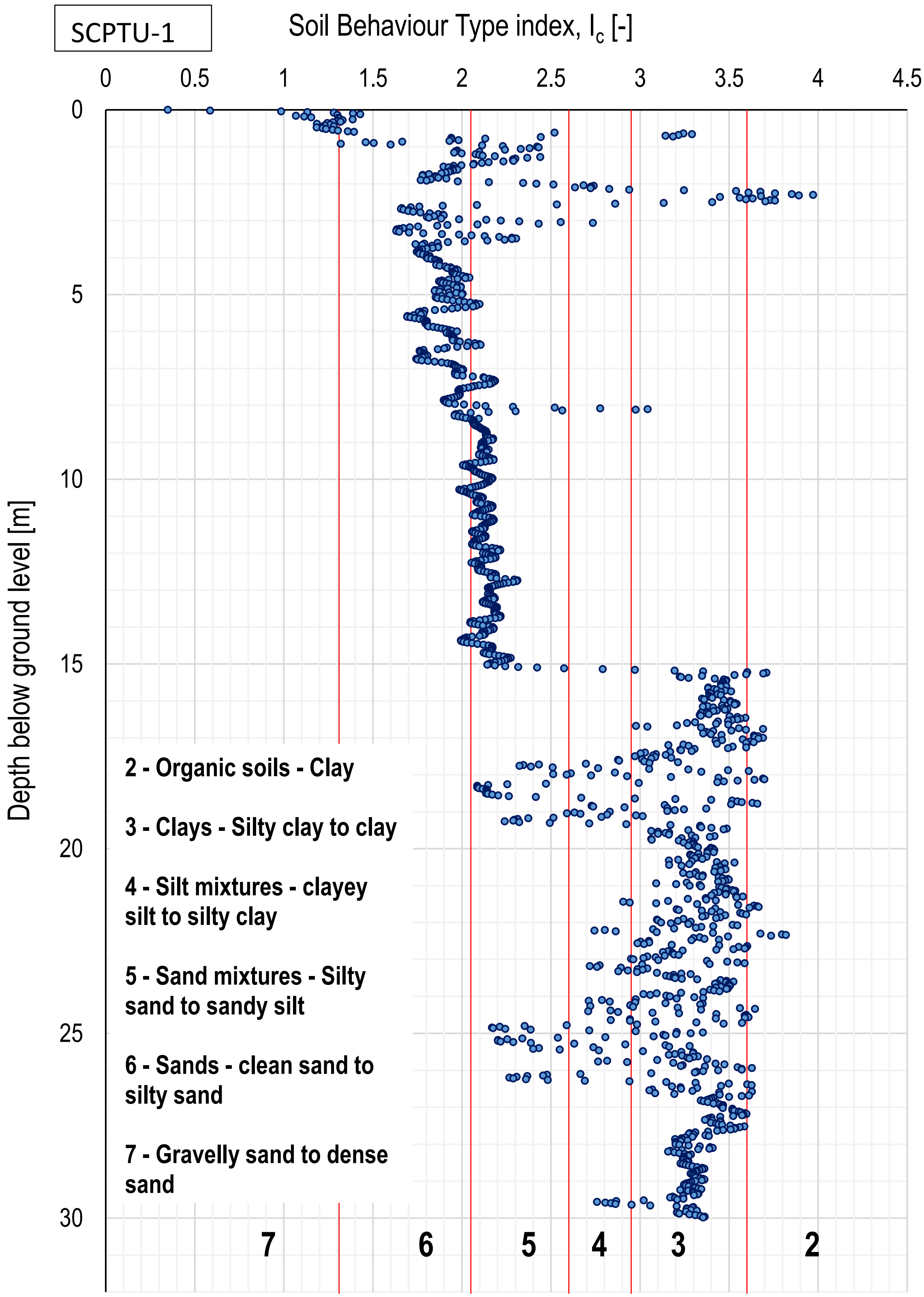

Corrected cone tip resistance q_t [MPa]
0
5000
10000
15000
Depth below ground level [m]
0
5
10
15
20
25
30
35
SCPTU-2

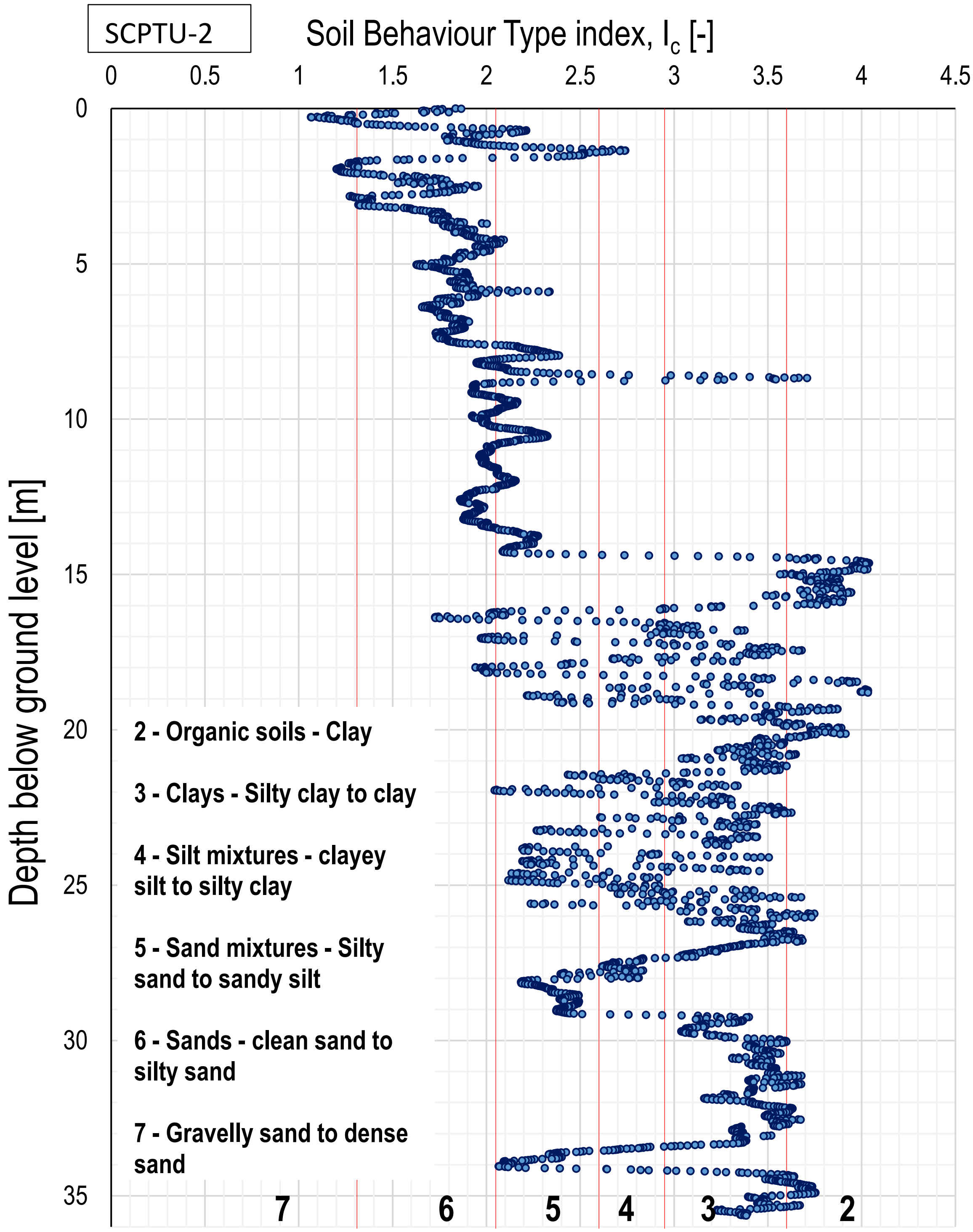
Measured sleeve friction resistance (f_s) for both tests are plotted: SCPTU-1 (left figure) and SCPTU-2 (right figure)



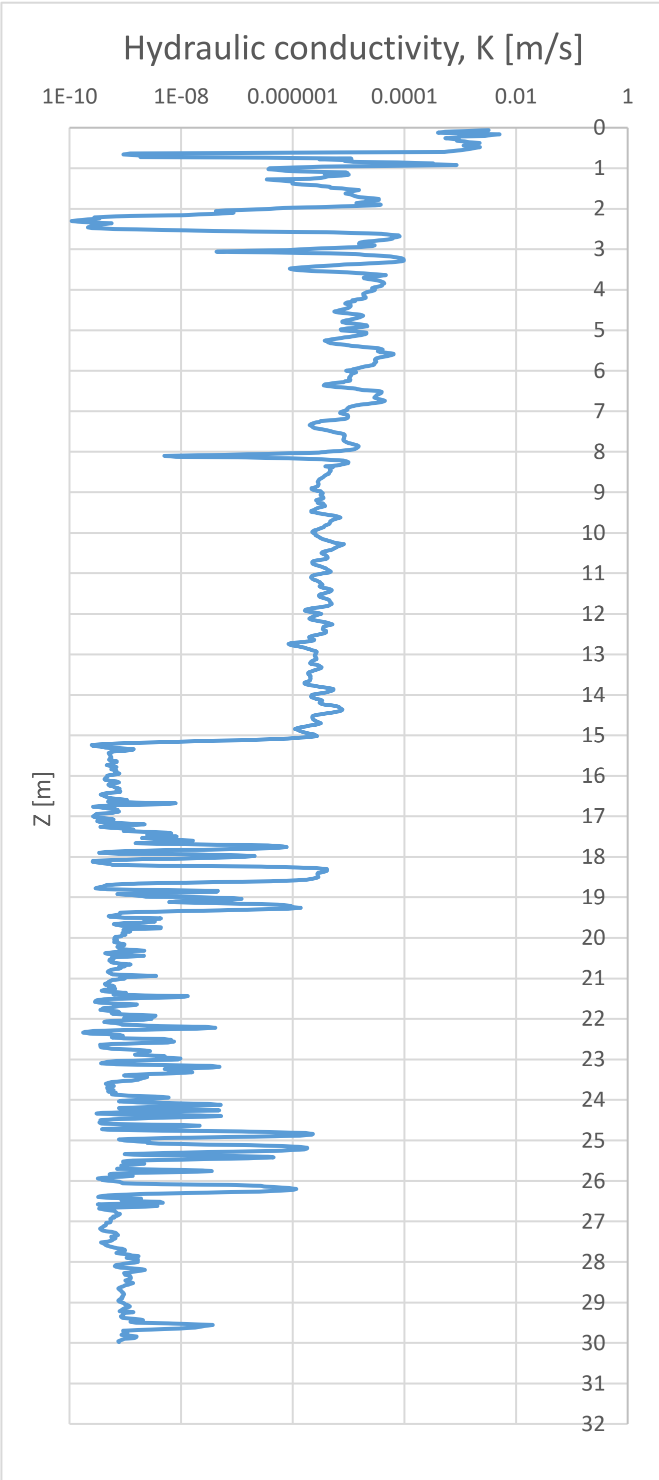


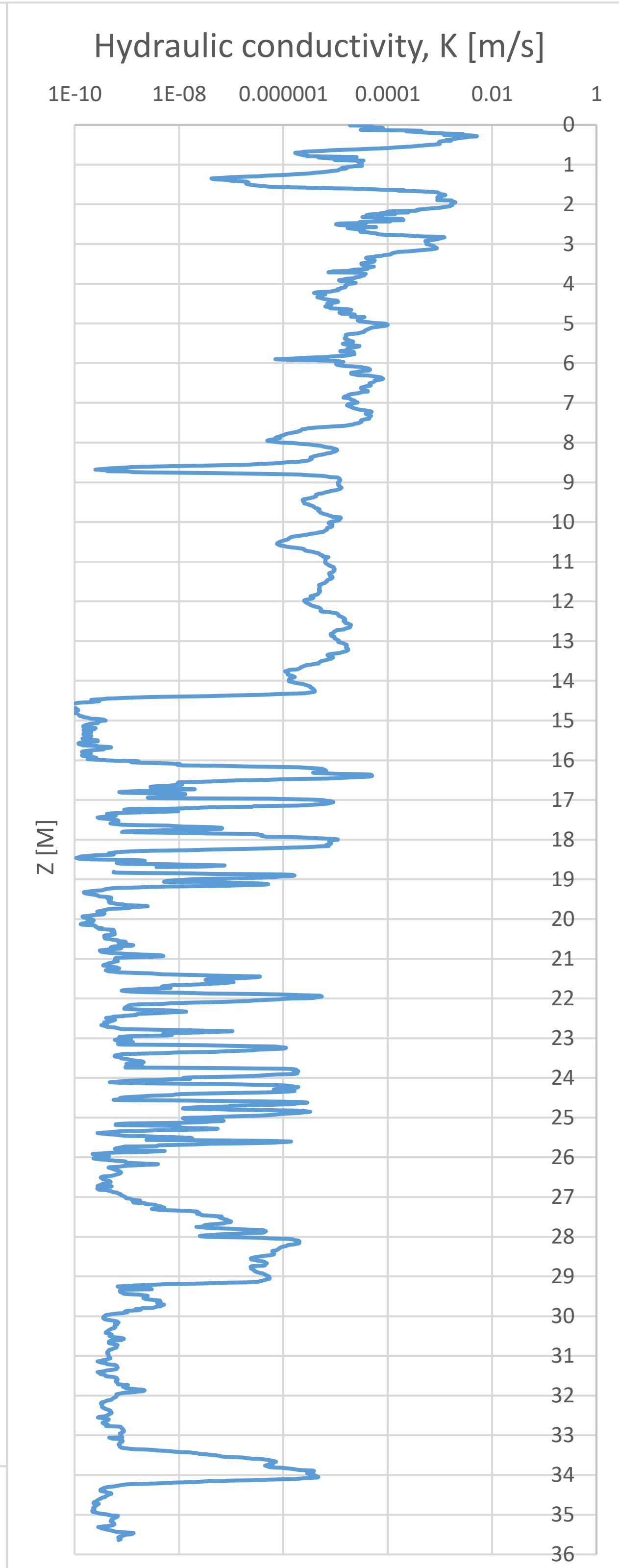
For classification of the Soil Behavior Type Index, I_c the plot is obtained:




SCPTU-2
Soil Behaviour Type index, I_c [-]
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
Depth below ground level [m]
0
5
10
15
20
25
30
35
2 - Organic soils - Clay
3 - Clays - Silty clay to clay
4 - Silt mixtures - clayey silt to silty clay
5 - Sand mixtures - Silty sand to sandy silt
6 - Sands - clean sand to silty sand
7 - Gravelly sand to dense sand
7
6
5
4
3
2

The log-scale plot of estimated values of soil permeability (k) based on the CPT SBT chart by Robertson: SCPTU-1 (left figure) and SCPTU-2 (right figure)





Based on the site stratigraphy these three layers have been considered:

1. G.L. to 5 m: filling material consisting of a mixture of sand and silt
2. 5 m to 15 m: sand
3. 15 m to end of available data: silty clay

For each layer hydraulic conductivity in a horizontal and vertical direction was obtained:

SCPTU-1	K(h)	K(v)
Layer 1	9.55E-03	3.61E-09
Layer 2	3.64E-04	9.89E-07
Layer 3	6.27E-06	8.74E-10

Table 6 – Hydraulic conductivity obtained fromSCPTU-1.

SCPTU-2	K(h)	K(v)
Layer 1	3.23E-02	1.88E-06
Layer 2	1.22E-03	3.14E-09
Layer 3	5.50E-05	7.09E-10

Table 7 – Hydraulic conductivity obtained fromSCPTU-2.

Chapter 4

On the need to transition in engineering-geological and engineering-geotechnical surveys to three-dimensional (3D) modeling and representation of materials

To date, an unstable situation has developed in engineering and geological surveys. On the one hand, chronic underfunding, obvious difficulties with updating old regulatory documents, and the emergence of various companies with different standards and approaches. On the other hand, the increasingly urgent need to switch to the digital transmission of information, in particular, engineering survey data, the use of modern software systems (allowing to solving problems in a three-dimensional formulation) for the design of increasingly complex buildings and structures, the emergence of new methods for studying engineering and geological conditions (and cheaper and faster) and soil behavior models. Under such conditions, the well-established approach to modeling engineering-geological conditions, based on a table with calculated and standard indicators and two-dimensional sections with numbers indicating the numbers of engineering-geological elements, often fails. Design decisions made based on such models are full of errors, and errors that are laid down at the earliest stages and, as a result, are extremely difficult to correct. There is no need to look for examples - these are springboards, a media center, and pop-up gas and oil pipelines.

The author sees a way out of this situation in the use of three-dimensional engineering-geological models - digital systems in a spatial setting, the study of which serves as a means to obtain information about engineering-geological conditions in general and their components in particular, while meeting the needs of economic activity. Of course, we are not talking about a complete replacement of the existing approach, and a rather long transition period is required, and 3D modeling itself is not always required, but only when building complex and unique structures, where we have a sufficient amount of engineering and geological information.

In general terms, the algorithm for constructing a three-dimensional engineering geological model is shown in Figure 21. It contains 4 explicit stages and implicit stages of model calibration, performed after each stage of the life cycle of the structure.

At the same time, modeling begins at the very beginning - when planning construction.

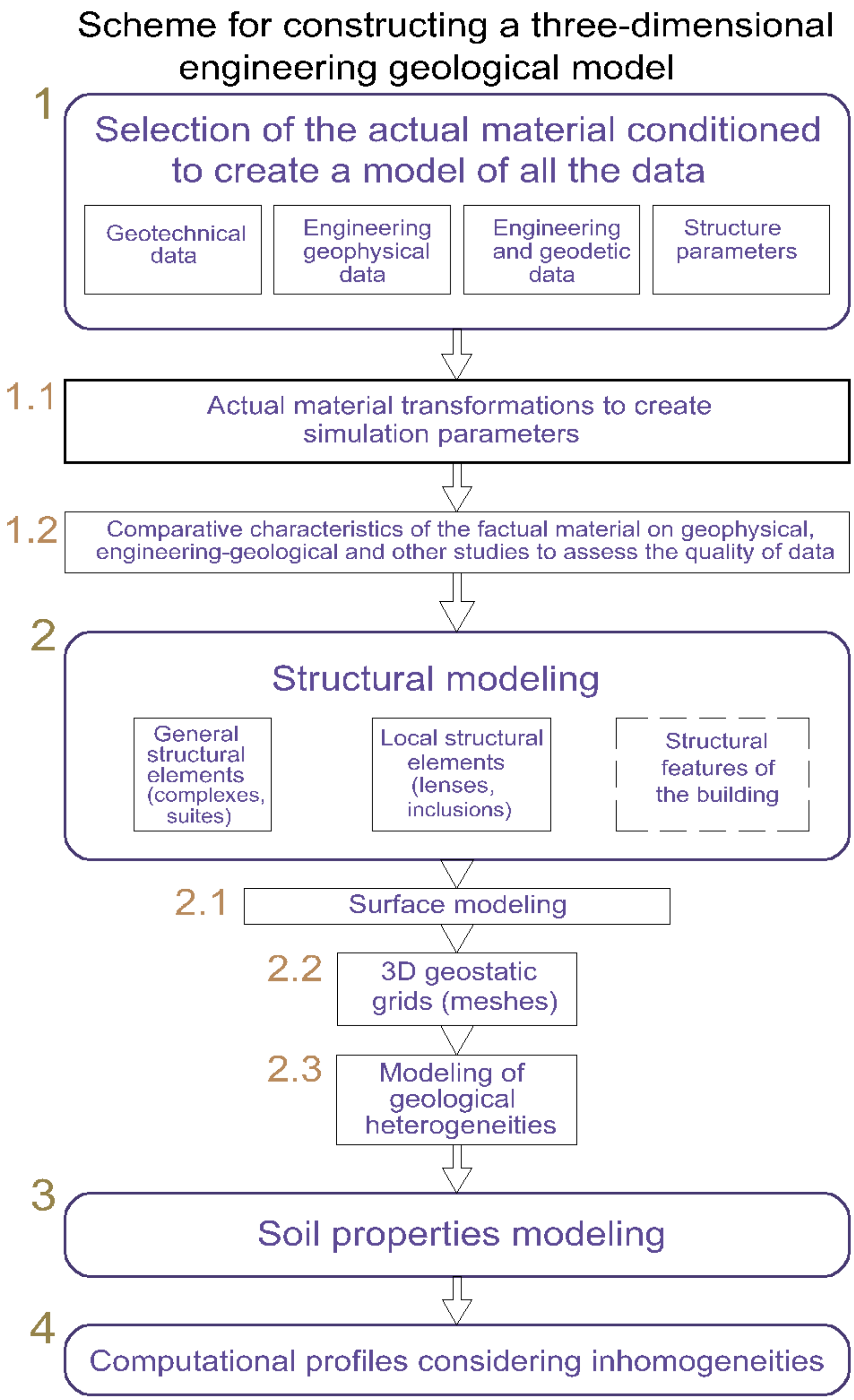


Figure 21 – Scheme for constructing a 3D engineering-geological model

Stage 1. Collection of engineering-geological, geodetic, geophysical, design, and other information (for example, on pre-design and design stages). Bringing all data to the same format and spatial coordinates. Entering data into a single software package (Figure 22).

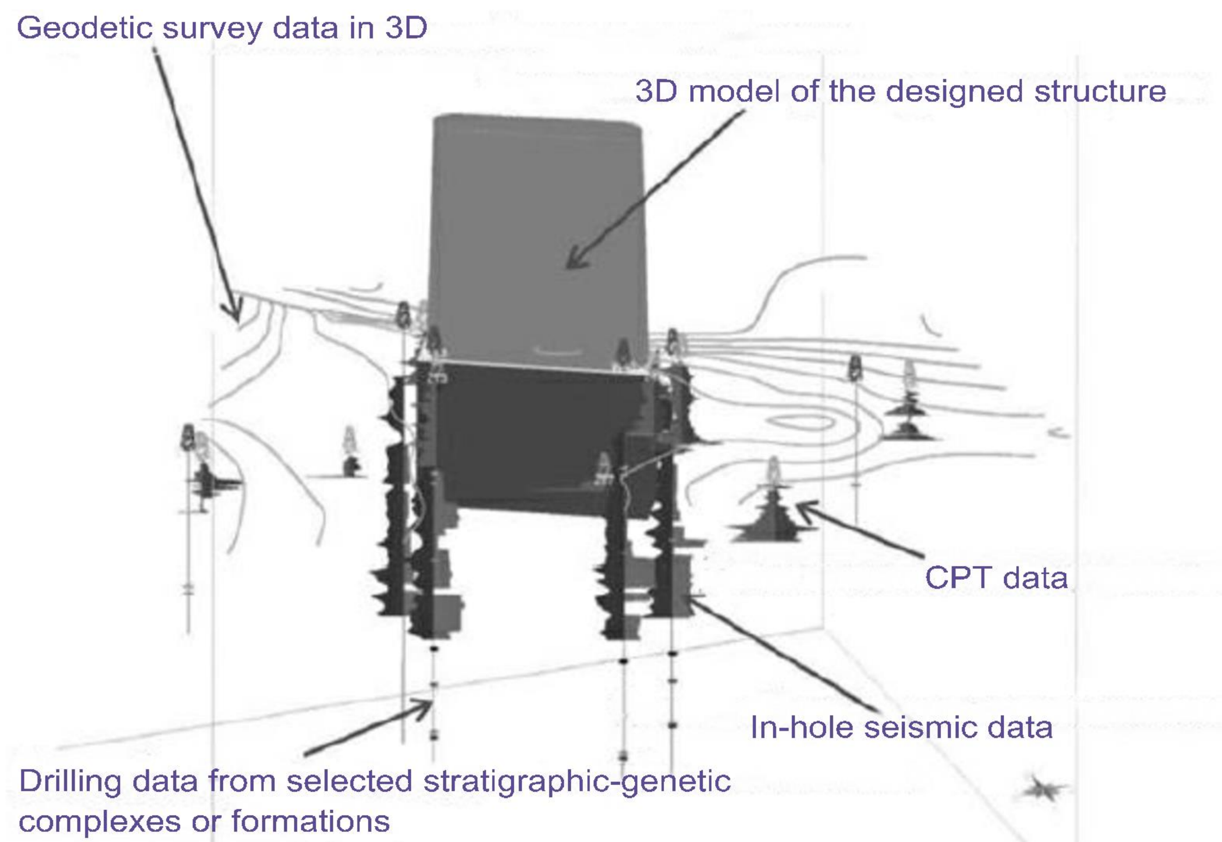


Figure 22 – An example of data integrated into a single 3D space

Stage 2.1. Construction of a structural model – continuous surfaces, reflecting the position in space of the roof and soles of extended geological bodies of the same genesis (Figure 23). With such constructions, the important points are the following: we cannot build local bodies (lenses, voids, etc.), and the most important point is the stratigraphic scheme and ideas about the history of the development of the territory. Otherwise, surface modeling can turn into a mechanistic connection of bodies with numbers, however, such “models” lead to disastrous results.

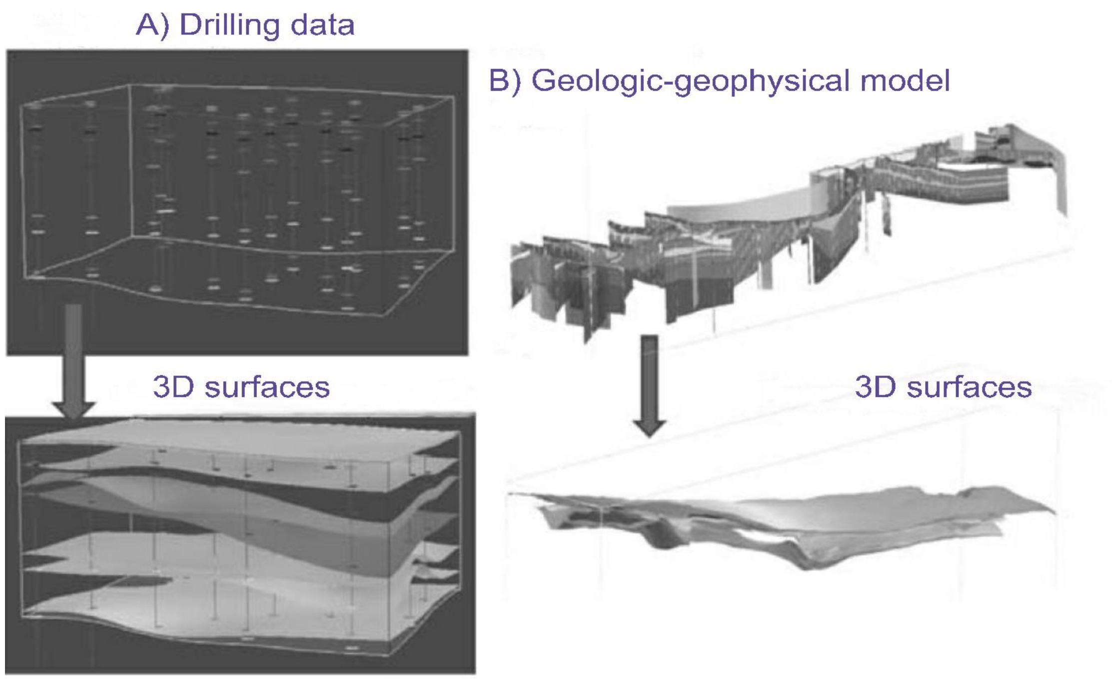


Figure 23 – An example of roof surfaces modeled according to drilling data (a) and geological and geophysical data (b), identified geological unified genesis

Stage 2.2. Discretization of the volume of the simulated array and obtaining finite elements. It should be noted that the constructed grids (their conformality and stratigraphic or their absence) have a significant impact on the subsequent geostatistical modeling based on which the state and properties of the soils that make up the massif and its geological heterogeneity are assessed.

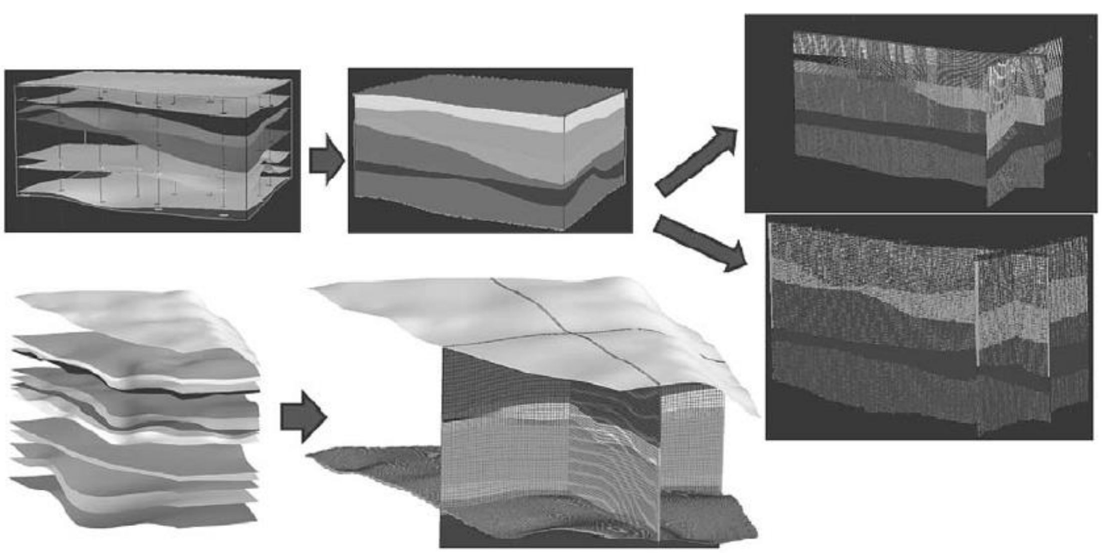


Figure 24 – An example of constructing homogeneous and inhomogeneous finite elements (grids)

Stage 2.3. Identification (probabilistic) of the most dangerous regions in a simulated 3D engineering-geological massif: karst cavities (Figure 25), lenses of soft soils, pockets of specific soils - eluvium, swelling, and subsidence differences.

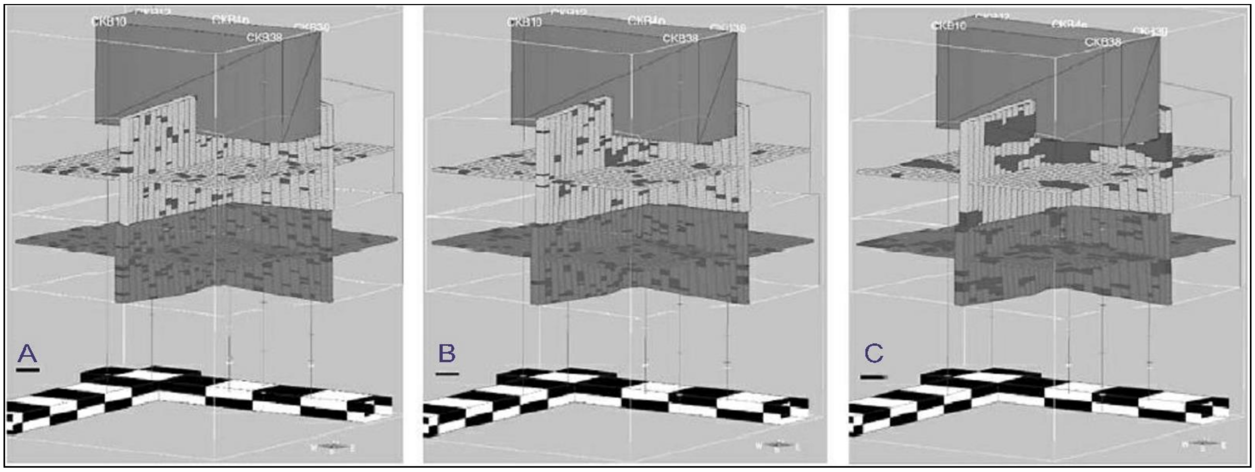


Figure 25 – An example of modeling karst voids from testing data and site descriptions

Stage 3. Modeling in space the required set of values of indicators of soil properties. In the volume under study, presented as a set of tetrahedra/hexahedra, geostatistical modeling is carried out according to actual data, and for each selected geological body separately. In this case, physical and physicomechanical properties can be confined either to the tops of the cells or to their geometric center. Continuous values of soil properties are modeled by the Gaussian random field method (Gaussian Simulation), based on Simple Kriging, with a non-stationary shape and direction of the variogram obtained from the actual data of laboratory and field studies of soils with the revealed size of variations and/or considering the expert judgment.

Stage 4. Providing designers with an engineering-geologically substantiated 3D model of the soil foundation of the designed structures in the form of a 3D mesh of finite elements with specified properties, the set of which is determined by the further purpose of the model. For example, if we are talking about geomechanical modeling, then the set of properties can be limited by the soil behavior model used - MC - soil density, internal friction angle, cohesion, deformation modulus, Poisson's ratio, dilatancy angle.

In the future, at the next stages of the life cycle of structures, obtaining additional engineering-geological information (for example, after the stage of AD-approved document) and/or geotechnical monitoring leads to calibration, a built three-dimensional engineering-geological model, and clarification of the spatial position of hazardous regions and the distribution of soil properties.

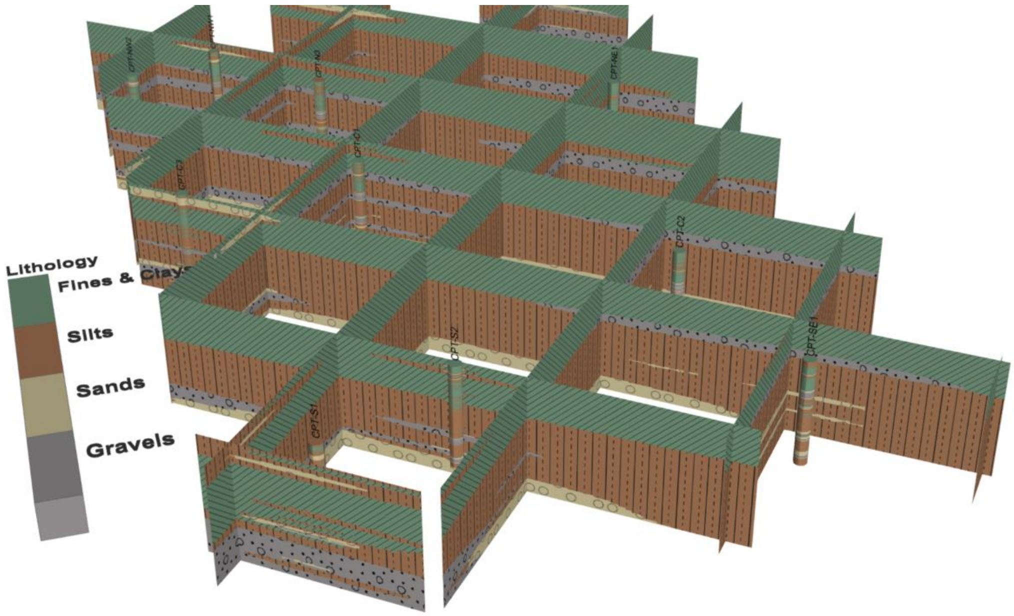


Figure 26 – 3D model of subsurface lithology (Source: ctech.com/3d-geologic-modeling/)

Capabilities of the approach based on 3D engineering-geological modeling:

- The proposed approach to the description of engineering-geological conditions makes it possible to solve a wide range of engineering-geological problems, including the identification of spatial patterns in the distribution of the structure, condition, and properties of soils, spatial localization of attenuated differences, etc., declared in many regulatory documents.
- 3D modeling makes it possible to obtain unambiguous, consistent, easily visualized engineering-geological 3D models containing the necessary information about the properties of the soils that make them up each finite element (division figure). In particular a section along any section of interest within the model, and not along wells.

- Analysis and statistical modeling (simulation) of the constructed models allow for finding the worst implementations, localizing spatial inhomogeneities, and performing calculations based on the worst (best) conditions.
- The constructed models are used for any calculations, including those based on the finite element method: the stress-strain state of the soil massif-structure system, the settlement of the structure and the thickness of the compressible stratum, the stability of slopes, the possibility of forming karst sinkholes, filtration problems, problems of heat and mass transfer, etc.

Benefits of an approach based on 3D geotechnical modeling:

- The resulting models are of higher quality than standard 2D tabular sections. The very principle of constructing computer three-dimensional models involves the resolution of ambiguities and contradictions in the position of the surfaces of the roof and the sole of the identified geological bodies at the stage of their creation.
- The resulting models are easily visualized and, as a result, allow viewing of modeling objects from any angle, implement automatic and semi-automatic drawing up of sections of any complexity and directions for the needs of designers.
- They are easily calibrated and updated upon receipt of new engineering and geological information and do not require large expenditures for new surveys in the event of a change in the landing of the structure.
- Allow proceeding to the assessment of engineering and geological risks at the local (object) level, since they are initially based on a probabilistic analysis of geological uncertainties.
- They provide designers with an engineering-geologically substantiated 3D model of the soil foundation of the designed structures in the form of a three-dimensional mesh of finite elements with specified properties.
- They allow to carry out multi-variant designs on their basis and find a reliable and optimal solution. [1]

[1] Барвашов В.А. Неопределенность данных инженерно-геологических изысканий и поведение системы ≪основание-фундамент-сооружение≫ // Инженерные изыскания. – *2014.* – Page 16–23.

- Due to their accuracy and high integration of data from various sources, they make it possible to avoid expensive design solutions with large safety factors for geological heterogeneity.
- They are the basis for geotechnical monitoring during the construction and operation of structures. [2]

[2] *Бершов А.*, *Томс Л.* Концепция геотехнического мониторинга на территориях объектов, расположенных на хребтах Псехако и Аибга // Инженерные изыскания. – 2013. – Page 48–52.

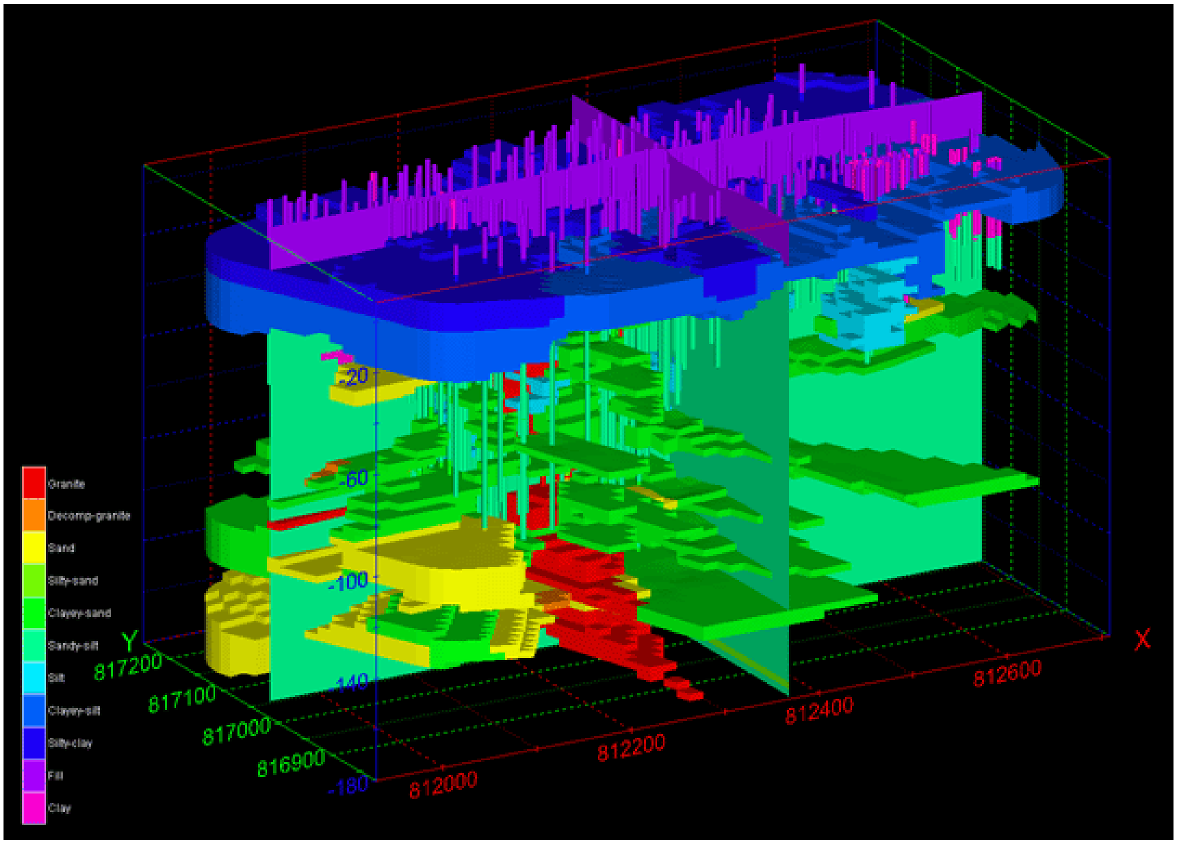


Figure 27 – Geological survey – investigation of Karst geology (Source: ctech.com/3d-geologic-modeling/)

The stratigraphic modeling can usually be modeled by using test and site data. Nowadays, many modeling tools such as Earth Volumetric Studio, Georeka, DionisosFlow, etc. are available.

The graphical user interface is integrated with modular analysis and graphics routines that can be customized and combined to meet the analysis and visualization needs of any application. They can be used to analyze all types of analytical and geophysical data in any environment (e.g., soil groundwater-surface water air, noise, resistivity, etc.). Some of them combine advanced volumetric gridding, geostatistical analysis, and 4D visualization tools into a software system designed to meet the needs of all geoscience disciplines.

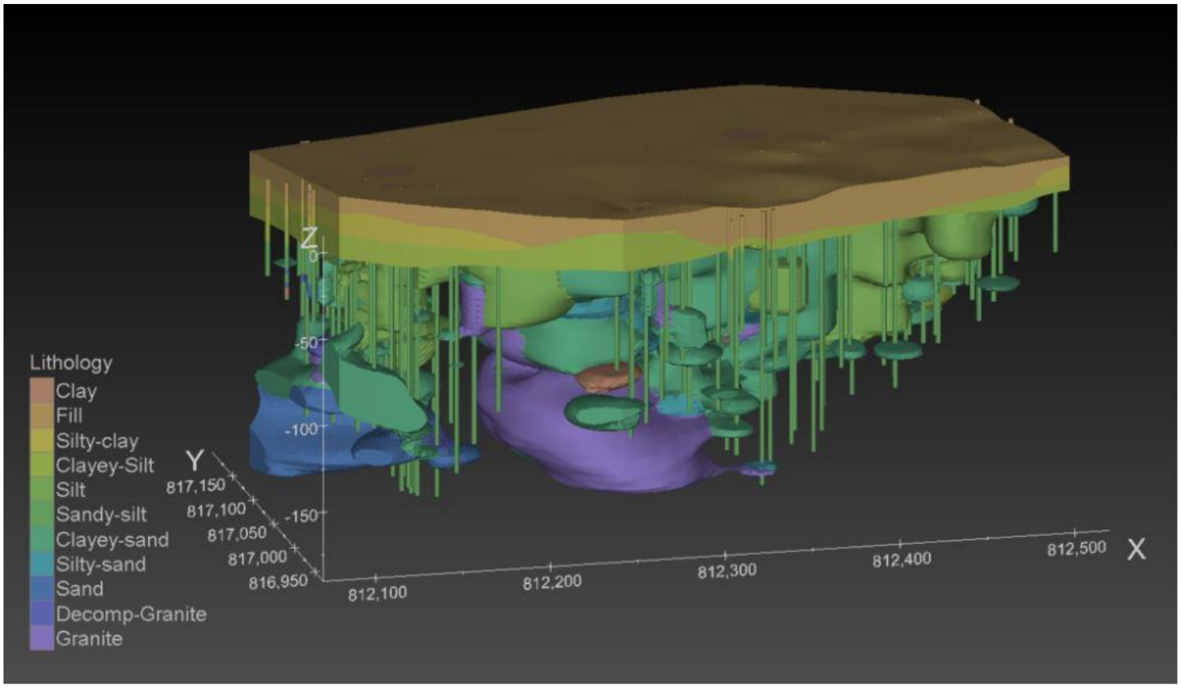


Figure 28 – Geological survey – an example of a subsurface lithology model created by Earth Volumetric Studio (Source: ctech.com/3d-geologic-modeling/)

Conclusion and Future Research

In this work the use of CPT test for the hydrogeologic characterization of soil stratigraphy have been discussed with the practical application of the test results for the described approach.

Based on site stratigraphy the 3 layers of soil have been obtained and hydraulic conductivity in horizontal and vertical direction for each layer is evaluated. It is observed that permeability in horizontal direction is greater than in vertical direction. Hence the hydraulic conductivity of soil in a layered system must be considered as dependent upon flow direction, relative position, and thickness of the layer.

Other different methods to obtain hydraulic conductivity have been left for the future due to lack of time (i.e. the experiments with real data are usually very time-consuming).

For the future interested studies, the present topic in Chapter 4 provides an introduction model. The novelty of 3D geological model is to collect all data from various tests and can be a great tool to control and monitor groundwater pollution and to obtain other required parameters.

References

1. ASTM Designation: D6067/D6067M – 17 Standard practice for using the electronic piezocone penetrometer tests for environmental site characterization and estimation of hydraulic conductivity.

2. Robertson, P. K., and K. L. Cabal. "Guide to Cone Penetration Testing for Geo-Environmental Engineering." *Prepared for Gregg Drilling & Testing Inc.,* (2008).

3. Robertson, Peter K. "Estimating in-situ soil permeability from CPT & CPTu." In *Memorias del 2nd International Symposium on Cone Penetration Testing, California State Polytechnic University Pomona, CA. http://www. cpt10. com/PDF_Files/2-51Robehc. pdf*. 2010.

4. Burns, Susan E., and Paul W. Mayne. "Interpretation of seismic piezocone results for the estimation of hydraulic conductivity in clays." *Geotechnical Testing Journal* 25.3 (2002): 334-341.

5. Lancellotta, Renato. *Geotechnical engineering*. CRC Press, 2008.

6. Manassero, M. "Hydraulic conductivity assessment of slurry wall using piezocone test." *Journal of geotechnical engineering* 120.10 (1994): 1725-1746.

7. Jamiolkowski, M. "Soil mechanics and the observational method: challenges at the Zelazny Most copper tailings disposal facility." *Géotechnique* 64.8 (2014): 590-618.

8. Elsworth, D., and D. S. Lee. "Limits in determining permeability from on-the-fly uCPT sounding." *Geotechnique* 57.8 (2007): 679-685.

9. Jamiolkowski, M. "Role of geophysical testing in geotechnical site characterization." *Soils and Rocks* 35.2 (2012): 117-137.

10. Lo Presti, D., Meisina, C. “Use of cone penetration tests for soil profiling and design of shallow and deep foundations” *CPT handbook.*

11. Lunne, Tom, Peter K. Robertson, and John J.M. Powell. Cone Penetration Testing in Geotechnical Practice. London: E & FN Spon, 1997. Print.

12. Tschuschke, Wojciech, Sławomir Gogolik, Magdalena Wróżyńska, Maciej Kroll, and Paweł Stefanek. "The application of the seismic cone penetration test (SCPTU) in tailings water conditions monitoring." *Water* 12, no. 3 (2020): 737.

13. Firouzianbandpey, Sarah, Benjaminn Nordahl Nielsenl, Lars Vabbersgaard Andersen, and Lars Bo Ibsen. "Geotechnical Site Assessment by Seismic Piezocone Test in North of Denmark." In *International Conference on Case Stories in Geotechnical Engineering and Symposium in Honor of Clyde Baker*, pp. Paper-no. Missouri University of Science and Technology, 2013.

14. Robertson, Peter K., and K. L. Cabal. "Guide to cone penetration testing for geotechnical engineering." *Gregg Drilling & Testing, Inc* 6 (2015).

15. Рыжков *И.Б., Исаев О.Н.* Статическое зондирование грунтов. – М.: АСВ, 2010. – 496 с.

16. *Lunne T., Robertson P.K.* and Powell J.J.,M. 2004. Cone penetration testing in geotechnical practice. London and New York: Spon Press.

17. Burns, Susan E., and Paul W. Mayne. *Penetrometers for soil permeability and chemical detection*. Geosystems Engineering Group, School of Civil & Environmental Engineering, Georgia Institute of Technology, 1998.

i. https://www.lankelma.co.uk/cpt-services/testing-services/piezocone/
ii. https://www.geotech.hr/en/cone-penetration-test-with-pore-water-pressure-measurement-cptu/
iii. https://www.ctech.com/3d-geologic-modeling/
iv. https://www.vertekcpt.com/using-cpt-pore-pressure-dissipation-tests-to-characterize-groundwater-conditions/
v. https://www.ltrc.lsu.edu/