Characterization of the recent soft silty clay deposit in the Ravenna port area (Italy)

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ABSTRACT: To plan upgrading works of several quays, an extensive geotechnical survey has been carried out including both in-situ testing as well as laboratory testing. The unusual extent of the investigated area, ranging from coastline to 4 km inland, allowed to appreciate the variation of the stratigraphic arrangement of the deposit related to its depositional history. This paper presents the results of such investigation focusing on the geotechnical properties of the thick, recent, soft silty clay deposit that governs the design of the quay walls. The wide available database and the choices to concentrate in a restricted number of sites several investigations allows to compare properly the results coming from laboratory and in-situ tests. In particular, due to their implication on fundamental design decision, the attention has been focused on the identification of the soil and on the undrained shear strength. These two aspects, related to the time to develop settlements and short term response to the loads, have been analysed in depth by comparing the results of laboratory tests on undisturbed samples with the estimation given from CPTu and DMT tests. The results shows that in-situ tests are powerful tools but require care when used to evaluate strength parameters and the association with a limited number of laboratory tests allows to obtain a reliable and cost-effective geotechnical characterization.

Keywords: in-situ testing; DMT; CPTu; laboratory tests; case study; clays; undrained strength

1. Introduction

The Port of Ravenna, one of the largest Italian harbour, is located along a channel connected to the Northern Adriatic Sea. By virtue of its strategic geographic position, this port is the natural gate for the trade between Italy and the markets of the Eastern Mediterranean and the Black Sea, also playing an important role towards the trade routes to the Middle and Far East.

The port was established after World War II, when a large petrochemical complex was built along an old channel. From the 1970's, the port expanded its commercial operations and, after the year 2000, there was growth in the container handling and passenger traffic as well.

In the Italian experience, the port of Ravenna is one of the first in which sheet piles have been used extensively to build the quays, thanks to the poor geotechnical properties of the subsoil that made easy and cost-effective the use of such class of structural solutions.

Over the last few years, the extraordinary development of the maritime shipping related to the rapid increase in the size of container vessels and the strong competitiveness among the ports have led towards the upgrading of the existing port facilities (Ruggeri et al., 2019 [1]). It is known, in fact, that large ships require deep seabed, large cranes and capability to handle and store heavy loads. So, to meet market needs, several coordinated actions have been recently planned at the Ravenna harbour, including the deepening of the seabed from 10.00 m to approximately 12.50 m (locally up to 14.50 m below sea level) and the strengthening of a number of quays. To design such interventions, an extensive geological-geotechnical campaign of investigation has been planned and carried out. Considering the large number of past surveys, it has been chosen to collect the available data in a single database and concentrate the new investigations on a limited number of sites in which several kind of tests have been carried out. This procedure allowed both to compare the results of different tests on the same soil stratigraphy and to have available a homogeneous survey to check the reliability of the old campaigns carried out over a period of tens of years.

An overview of the site in which is located the Port of Ravenna indicates clearly that the area is part of the Po Plain. This means that the main features of the subsoil are related to the evolution of the Po river which in turn depends on the eustatic movements of the sea level. In fact the variation in the level of the sea produced always different phases of erosion and sedimentation of the river plain but, in the present case study, the eustatic movements have been so strong in the recent geologic time, that the area of the Ravenna has experimented a transgressive-regressive marine cycle, with deposition of sediments in marine environment. This recent geologic evolution promoted the formation of three main layers that characterize the entire subsoil of the port area. From the engineering perspective, the most significant stratum is the intermediate one, a soft silty clay deposit of marine origin, due to its low shear strength and high compressibility. Moreover, the low permeability and the saturated conditions of this layer implies an undrained response of the deposit to the variation of the loads as well as the development of consolidation settlement over a long-lasting period of time.

This paper, after a brief overview of the geologic and stratigraphic characteristics of the area, focuses on the geotechnical characterization of the soft silty clay layer based on the results of the different investigations that have been carried out during a recent geotechnical survey. In particular, results obtained by piezocone penetration testing (CPTu), flat dilatometer testing (DMT) and laboratory tests on undisturbed samples will be considered and compared.

2. Geological-stratigraphic setting

The zone of interest belongs to the South-Eastern coastal part of the Po Plain, an area enclosed to the north by the Alps and to the west by the Apennines. The deposits that form the plain are mainly of Plio-Pleistocene origin, while the shallow soils of the coastal area belong to the Holocene epoch. It is therefore clear that the stratigraphic sequence of the area is strictly related to the sediment transported by the Po river in relation to the sea level fluctuation. This point of view results of great help in the comprehension of the soil layers included in the first 30 m, that have been deposited during a transgressive-regressive marine cycle. These sediments are of major importance for the geotechnical works because they are always included in the zone of influence of the quay structures. In particular, approximately 18.000 years ago, the sea level reached the lowest elevation of the last glacial episode, more than 100 m below the current level. In that time the delta of the Po river was located several hundreds of kilometres on the South in respect of its current location and the entire Nord Adriatic Sea was part of the extended alluvial plain of the river.

With the end of the glacial episode the sea level rose quickly determining a flooding of the coastal area with a shoreline movement toward higher groundland (marine transgression). The maximum marine transgression was reached approximately 5.500 years ago. At that time the area of Ravenna was a shallow marine environment. Then, due to sediment supply by the Po river the shoreline moves again seaward, till to reach the actual configuration of the coastline.

The described process, well documented in Amorosi & Marchi (1999) [2], has determined the stratal architecture of the soil deposit in the Ravenna area that is showed in the cross-section of Figure 1. The section represents approximately the stratigraphic sequence of approximately 15 km from the current coastline towards inland. Beneath a thick sediments layer (20 - 30 m) of Holocene epoch it is found the continental deposit of the old alluvial plain (lithofacies A) that have been deposited by the Po river during the Pleistocene. A layer of 1-3 m, constituted by silty sands to fine sands, lays on the Pleistocene deposit. This layer, named association facies T, reflects the landward migration of the sandy barrier during the Holocene transgression.

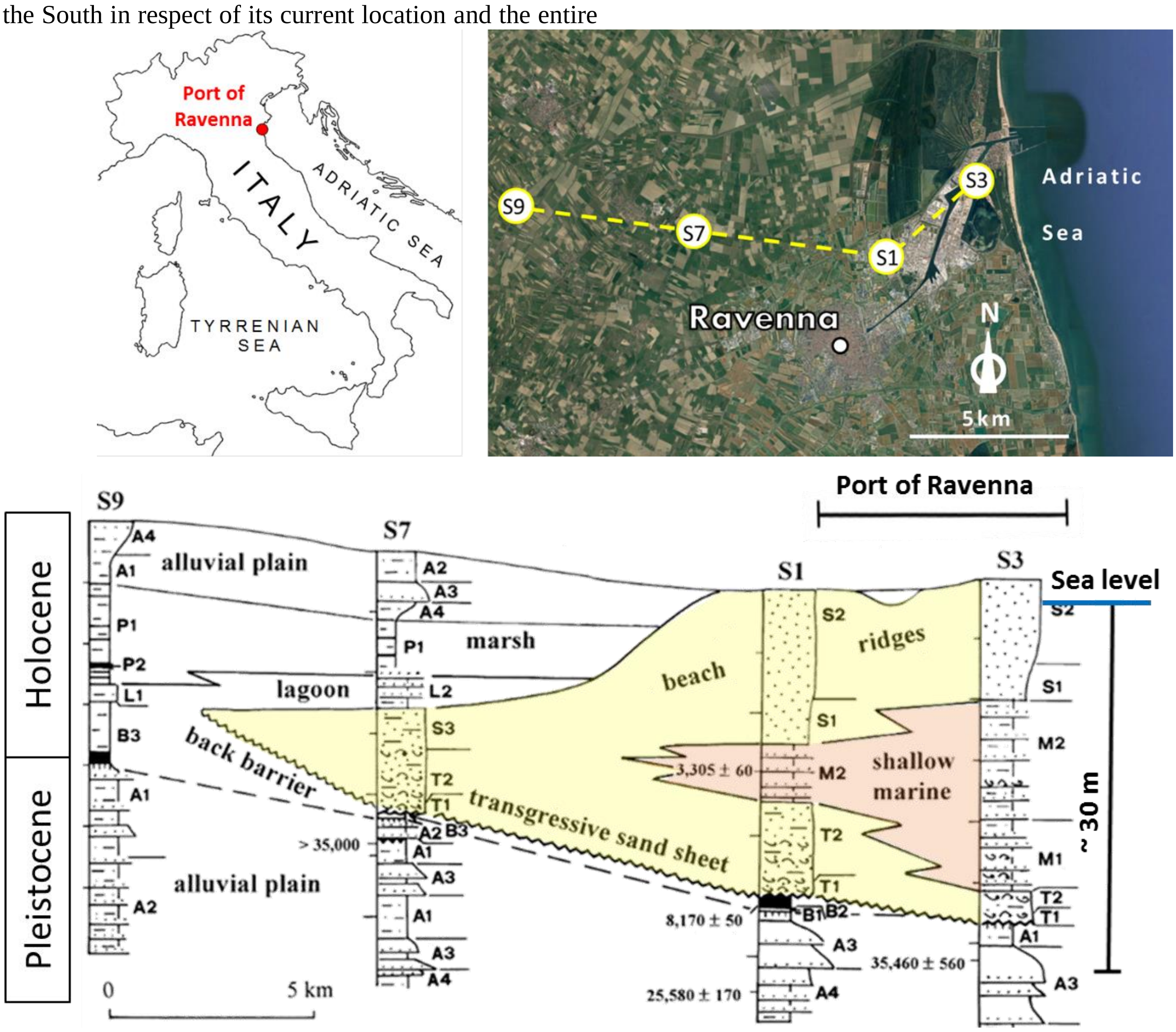


Figura 1. Map of Italy with the location of the Ravenna port; Aerial view of the Ravenna area with the location of some significant boreholes; Stratal architecture of the uppermost Pleistocene–Holocene deposits along the line S9-S7-S1-S3 (modified from Amorosi et al., 1999 [2])

As the shoreline moves inland, a depositional environment of shallow marine water takes place, allowing the sedimentation of fine grained soils that is indicated as facies association M in the Figure 1. This deposit has a banded appearance produced by the rhythmical alternate sequence of grey clay-silty clays and very fine sand layers. Radiocarbon dating from the middle part of this facies association provided an age of 3.300 years before present.

When the shoreline moves back towards the sea (marine regression), a retrogradation of the sandy barrier is observed. This deposit, identified as facies association S in Figure 1, is interpreted as laterally continuous beach ridges occurred in response to progradation of a wave-dominated delta system of the Po river.

So, in the first 30 m of depth at the current shoreline, we can currently observe a first sand layer (S) followed by a thick soft fine-grained deposit (M) that laid on a thin sand layer (T) that in turn overlay the Pleistocene soils of the old alluvial plain. Moreover it is relevant to note that the migration of the sandy barrier determined the onset of consolidation settlement of the soft deposit, with the formation of a back-barrier lagoon. This means that overlying the sand layer S it is often encountered organic clay deposit of marshy environment (P) whose thickness is related to the speed rate of the barrier migration.

3. Port area: typical stratigraphic models

As pointed out by Amorosi & Marchi [2] in 1999 the piezocone penetration test (CPTu) is a powerful tool to identify the stratigraphic sequence of the soft deposit of the south-eastern Po Plain, both in term of accuracy and in term of timely and cost-effectiveness. In this work, by taking up the same methodological approach, we consider the results of the extensive surveys carried out in 2014 to define the reference stratigraphic models of the port area.

The typical readings of a CPTu carried out in the port area are showed in Figure 2, where the correct cone resistance (q_t), sleeve friction (f_s) and penetration pore water pressure ($u_{2)}$ compared with hydrostatic pressure (u_0), are represented. The penetrometer profiles clearly show the influence of the soil stratigraphy, allowing to identify the main geotechnical units. In particular, 4 geotechnical units are clearly apparent and described in the following.

Fine Sands. (Unit S) – The layer spans between -3 m to -15 m a.m.s.l., even if at the depth of 6 m from sea level a stratum of organic clay, 1 m thick, breaks the continuity of the deposit. The values of q_t increase with depth, from 5 MPa at the top to 8 MPa at the base. The excess pore water pressure ($\Delta u = u_2 - u_0$) up to a depth of about 11 m from sea level is negligible, indicating the presence of a highly permeable soils.

Silty clay (Unit M) – This layer falls between -15 m and -26 m a.m.s.l.; it presents a very low cone resistance, linearly increasing with depth from 1.0 to 1.7 MPa. The development of a positive excess pore pressure (Δu), ranging from 200 to 300 kPa, is observed during the penetration of the cone in this layer, indicating a general low permeability of the deposit.

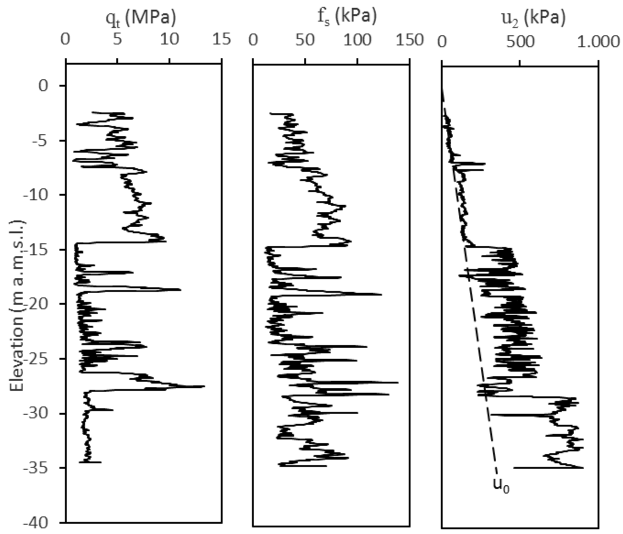


Figure 2. Typical CPT profile in the port area

These properties are consistent with the predominantly fine grained composition of the facies association M. However, several thin sandy layers are detected in the fine grained mass as demonstrated by the spikes of the q_t resistance in the profile.

Transgressive sandy layer (Unit T) – A sandy stratum is encountered at -25 m a.m.s.l., as highlighted by the sharp increment of the q_t values, that reach 7-10 MPa, and by the drop off of the excess pore water pressure. This layer indicates the end of the Holocene sequence.

Deposits of alluvial plain (Unit A, Pleistocene) – Below the Unit T the alluvial sequence of the old Po Plain is encountered. The sequence begins with fine grained lithotype as demonstrated by the low values of q_t, that increase with the depth. The plot of the excess pore pressure denotes positive values ranging from 400 to 500 kPa in this layer with some fluctuations related to the presence of clay of varying amounts. In other words, the positive values of the pore pressure indicate the absence of an evident dilatant behaviour of the clay during the penetration of the probe [3]. However, Unit A can contain sandy strata also, related to the floods of the Po river.

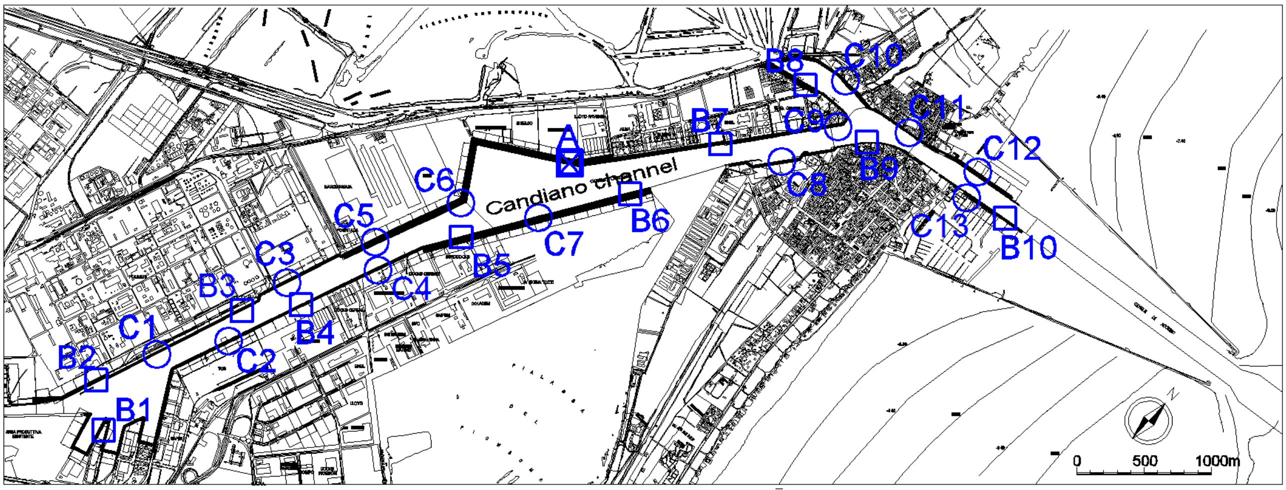
3.1. Recent geotechnical site investigation

In 2014 a new site investigation plan has been defined and carried out. The planning of such investigation has been made by taking into account the large number of surveys that had already been executed over tens of years at the Port of Ravenna. Specifically, considering the in-situ tests, 80 borehole logs, 217 CPT, 6 DMT and some seismic investigations have been collected and organized.

Accordingly, the new general survey has been planned by considering that existing data were generally of good quality, that some locations were not properly covered, that geophysical data were scarce and that it was necessary to distribute such new testing over the entire area of interest to obtain a uniform database for comparison with the existing data. Following these lines of action, it has been chosen to concentrate the new investigations in 24 test sites with several different probing carried out at close distance from each other's. Three levels of test sites were planned:

- Site A: the most complete investigation,
- Site B: extensive investigation
- Site C: limited investigation.

In Figure 3 it is shown the map of the port of Ravenna, the locations of the test sites and the summary of the tests included for each. Beyond several boreholes with core drilling and taking of undisturbed samples, the geotechnical characterization was based on in-situ tests and specifically seismic down-hole, (DH), seismic cross-hole (CH), piezocone penetration testing (CPTu) and flat dilatometer testing (DMT).



	Borehole	CPTu	DMT	Seism. down-hole	Seism. cross-hole
☒ Inv. site A	×	×	×		×
□ Inv. site B	×	×	×	×	
○ Inv. site C	×	×	×		

Figure 3. Map of the Ravenna port with the location of the geotechnical test sites; Type of tests executed in each site.

3.2. Stratigraphic variability along the Channel

The large number of investigation allowed to define a detailed stratigraphic profile of the subsoil along the sides of the Candiano channel. In Figure 4 the geotechnical cross-sections obtained along the left and right banks of the channel are shown. This representation, mainly based on CPTu results, is useful to appreciate the variability in the thickness of the different geotechnical units. From a stratigraphic perspective, it is possible to distinguish 3 consistent stratigraphic sectors, South-Western, Central and North-Eastern, moving from the shoreline to inland. The main characteristics of these sectors are:

- South-Western sector: it is characterized by a great thickness of the fine sands (Unit S), the absence of the marsh (Unit P) and the relevant presence of thin sandy layer in the Silty clay (Unit M).
- Central sector: it presents a significant presence of the clay deposit of marshy environment (P) at the expense of the Unit S that has a little thickness. It is also meaningful to note the scarce presence of thin sandy layer in the fine grained deposit (Unit M).
- North-Eastern sector: it corresponds to the strip closest to the current shoreline, is characterized by the large thickness of the Unit M that exceeds 15 m and the practical lack of thin sand layer in the deposit. The Unit P is not present in this sector consistently with the geological evolution of the area.

From Figure 4 it emerges that over the entire port area the first meters below the ground level were altered by some anthropic activities. Moreover, it can be observed that the Pleistocene deposits (Unit A) are everywhere encountered at about 25-27 m from sea level, consistently to the very low sloping angle of the old alluvial plain. Such unit presents a large variability in composition that is sometimes fine-grained and sometimes coarse-grained, as demonstrated by the very different values of cone resistance in this deposit.

In Figure 5 these findings are summarised in a schematic stratigraphic model of the port area in which the typical cone profiles of each sector superimposed to the stratigraphic representation offers an immediate idea of the related mechanical properties of the encountered geotechnical units.

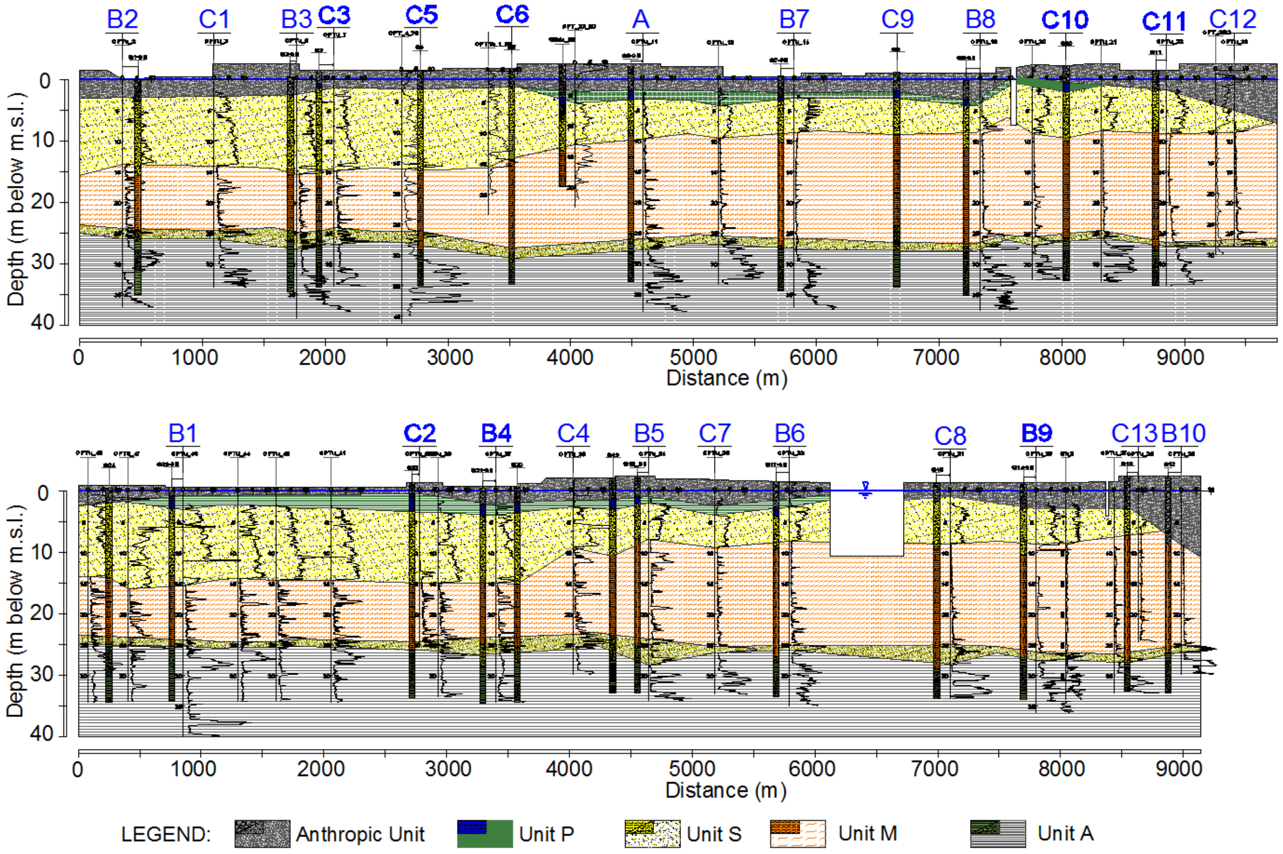


Figure 4. Geotechnical cross-section along the left and the right bank of the "Candiano" channel

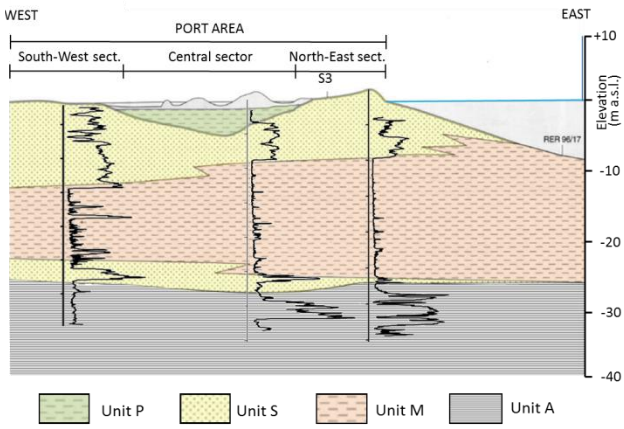


Figure 5. Schematic stratigraphic model of the port area (modified from Calieri et al., 2002 [4]) with superimposed the typical CPT profile of the3 main sector (distance not to scale).

3.3. Identification of the soils from CPTu, DMT and laboratory tests

One of the major applications of the CPT is the identification of soil type. Robertson (1990) [5], considering that CPT profile is related to the mechanical behaviour of the soil and not directly to soil classification (i.e. grain-size distribution), introduced the term "soil behaviour type" (SBT) to distinguish between the possible soils. This requires the assessment of the normalized dimensionless cone parameters, Q_t, F_r and B_q, where:

$$Q_t = (q_t - \sigma_{v0}) / \sigma'_{v0} \tag{1}$$

$$F_r = [(f_s/(q_t - \sigma_{v0})] \, 100\% \tag{2}$$

$$B_q = (u_2 - u_0)/(q_t - \sigma_{v0}) = \Delta u/(q_t - \sigma_{v0}) \tag{3}$$

Where:

σ_{v0} is the in-situ total vertical stress;
σ'_{v0} is the in-situ effective vertical stress
u_0 is the in-situ equilibrium water pressure
$\Delta u = (u_2 - u_0)$ is the excess penetration pore pressure

Robertson suggested two charts for soil identification based on either $Q_t - F_r$ and $Q_t - B_q$ but recommended to consider the $Q_t - F_r$ chart, (also called SBTn chart) was generally more reliable. These parameters are used to classify a soil in one of the nine zones that form the SBTn chart (see Table 1).

Table 1. Proposed description of the SBTn zones

Zone	Soil behaviour type
1	Sensitive fine-grained
2	Organic soils: peats
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	Dense sand to gravelly sand
8	Stiff sand to clayey sand (OC or cemented)
9	Stiff fine-grained (OC or cemented)

To simplify the application of the SBTn chart, the normalized cone parameters Q_t and F_r can be combined into one Soil Behavior Type index, I_c, where I_c is the radius of the quasi concentric circles that represent the boundaries between each SBTn zone. According to Robertson and Wride (1998) [6], I_c can be defined as follows:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5} \tag{4}$$

In Table 2 the values of I_c that bound the Robertson SBT zones are given. Note that soils falling in zones 1, 8 and 9 of Table 1 cannot be properly identify by any I_c values.

Table 2. Boundaries of soil behaviour type

I_c	Zone	Soil behaviour type
$I_c < 1.31$	7	Gravelly sand to dense sand
1.31 < Ic < 2.05	6	Sands: clean sand to silty sand
2.05 < Ic < 2.60	5	Sand mixtures: silty sand to sandy silt
2.60 < Ic < 2.95	4	Silt mixtures: clayey silt to silty clay
2.95 < Ic < 3.60	3	Clays: silty clay to clay
Ic > 3.60	2	Organic soils: peats

Robertson (2009) [7] suggested that the normalized SBTn chart should be used with the normalized cone resistance (Q_{tn}) calculated using a stress exponent (n), that varies with soil type via I_c. So:

$$Q_{tn} = [(q_t - \sigma_{v0})/p_a]\cdot(p_a/\sigma'_{v0})^n \tag{5}$$

where:
$(q_t - \sigma_{v0})/p_a$ is the dimensionless net cone resistance
$(p_a/\sigma'_{v0})^n$ is the stress normalization factor
n is the stress exponent that varies with SBTn
p_a is the atmospheric pressure in same units as q_t and σ_v

The stress exponent n can be estimated by the following relationship:

$$n = 0.381\ (I_c) + 0.05\ (\sigma'_{v0} / p_a) - 0.15 \tag{6}$$

where $n \leq 1$

The flat dilatometer test (DMT), developed in Italy by prof. Silvano Marchetti in the 1980's, is nowadays a popular in-situ tests in several parts of the world. The test involves two pressure readings A (the pressure required to just move the membrane of the probe) and B (the pressure required to expand the membrane center of 1.1 mm into the soil), to be corrected for membrane stiffness, gage zero offset and feeler pin elevation to derive pressures p_0 and p_1 used for interpretation. Among the several soil properties that can be estimated by this test, there is also the soil identification index, I_D, defined as:

$$I_D = (p_1 - p_0)/\ (p_0 - u_0) \tag{7}$$

where u_0 is the pre-insertion in situ pore pressure.

According to Marchetti (1980) [8], the soil type can be identified on the base of Material Index as shown in Table 3. As for CPT, for DMT I_D reflects the mechanical behaviour of the soil that is only indirectly related to the particle size distribution. More detailed information on the DMT equipment, test procedure and all the interpretation formulae may be found in the comprehensive report by ISSMGE Technical Committee TC16 (Marchetti et al., 2001, [9]).

Table 3. Soil type identification by using the Material Index, I_D

I_D	Soil behaviour type
$0.1 < I_D < 0.33$	Clay
$0.33 < I_D < 0.6$	Silty Clay
$0.6 < I_D < 0.8$	Clayey Silt
$0.8 < I_D < 1.2$	Silt
$1.2 < I_D < 1.8$	Sandy silt
$1.8 < I_D < 3.3$	Silty sand
$3.3 < I_D < (10)$	Sand

The comparison between the soil stratigraphies obtained by CPTu and DMT testings is shown in Figure 6 for the 3 relevant sectors of interest, South-West, Central and North-Est, respectively. It can be noted that both CPTu and DMT identify the stratigraphic sequence very accurately. The Unit S is identified as Sand mixtures (zone 5) to Sand (zone 6) through CPTu and, consistently, between Sand and Silty sand by DMT.

The unit M is interpreted as clay, both according to CPT interpretation, where the points fall mainly in zone 3 (silty clay to clay), and DMT results.

The unit S is identified as silty sand according to CPT and from sandy silt to silty sand according to DMT results.

The identification obtained for the unit S and M are shown in Figure 7, in term of I_D versus I_c, together with the results collected by Robertson (2009) [10] from several investigations all around the world. In the same graph it is plotted the trend line between the two mentioned indexes of equation:

$$I_D = 10^{(1.67 - 0.67\ Ic)} \tag{8}$$

It is worth to note that the results of the present study matches very well the equation (8).

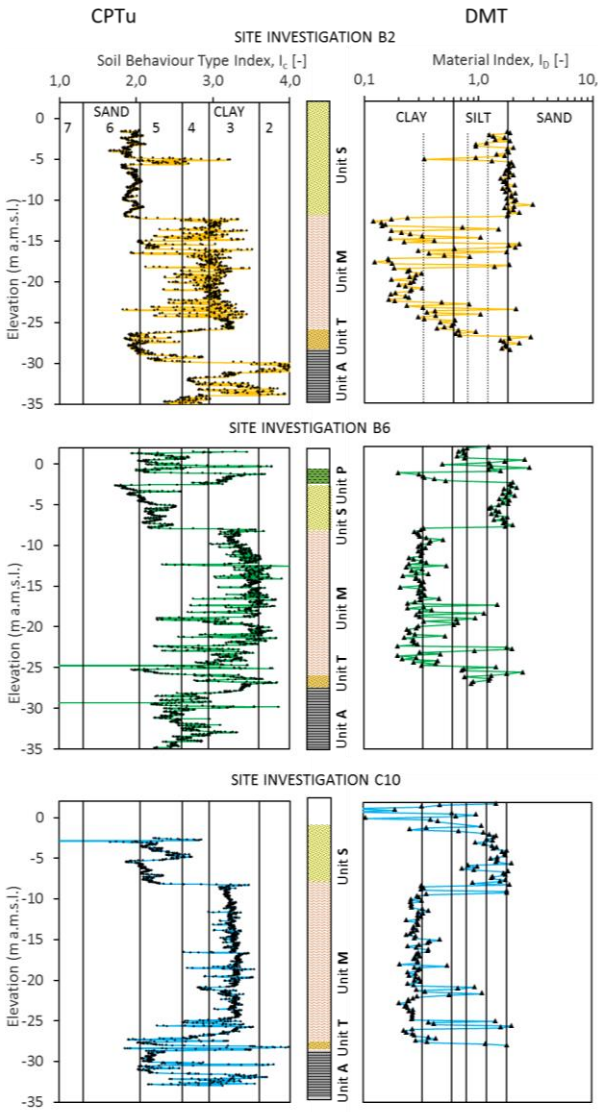


Figure 6. Comparison of stratigraphic identifications of the soil obtained by CPTu and DMT for 3 relevant site investigation of the port area

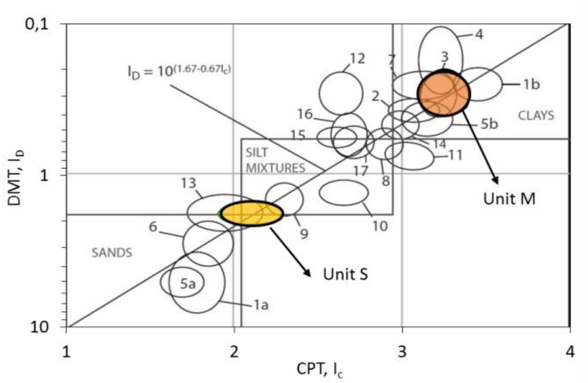


Figure 7. I_D versus I_c of unit M and unit S compared with I_D versus I_c average values from adjacent CPT and DMT profiles obtained all around the world (modified from Robertson, 2009 [10])

Moreover, the recent geotechnical survey at the Ravenna harbour offers the opportunity to compare the soil identification based on in-situ tests with the real particle curve distribution of the soils, thanks to the large number of samples that have been tested. In Figure 8 the typical particle distribution curves of samples belonging to Unit P, Unit S and Unit M are presented. It can be observed that in Unit M the amount of silt is always greater larger than the amount of clay. The most clayey soil results to be the Unit P, that are deposit formed in marsh environment. The Unit S presents a uniform granular part made of fine sandy grains.

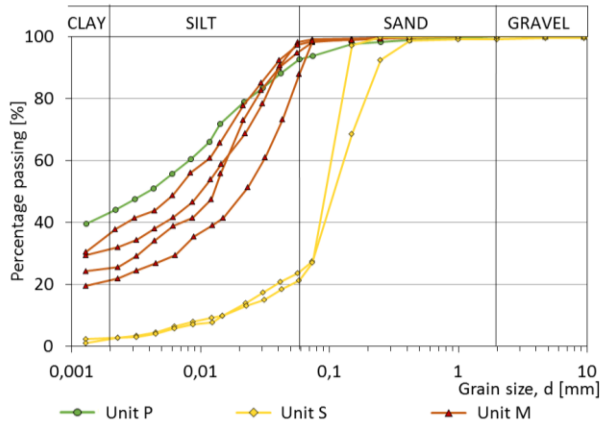


Figure 8. Particle size distribution curves of some samples belonging to Unit P, Unit S and Unit M

It appears interesting to focus the attention on the results of the tests carried out on the site investigation A where, besides the execution of CPTu and DMT, more than 30 samples have been taken and tested in laboratory to determine particle size distribution and Atterberg limits. In Figure 9 it is shown the comparison between the grading of the samples and the soil identification from CPTu and DMT. It can be pointed out that the sands are correctly identified (note that at the site A the Unit S has been largely removed and altered by previous works). However, the Units M is identified by the in-situ test as more clayey than its real granulometric composition. This fact was already be pointed out in the past because the cone of the CPTu as well as the DMT probe respond to the in-situ mechanical behaviour of the soil (e.g. strength, stiffness and compressibility) and not directly to soil classification criteria. This fact is well known in technical literature with reference to the mechanical behaviour of the soil mixtures where, depending on the grading of the granular part, even limited amount of silt and clay is sufficient to cause the decreasing of the mechanical properties of the mixture (e.g. shear strength) to values corresponding to that of a fine-grained soil (see Ruggeri et al., 2016 [11]). Precisely for these reasons, when the soil identification by in-situ testing does not match the real particle size distribution indicates the prevailing influence of the finer component on its general mechanical response.

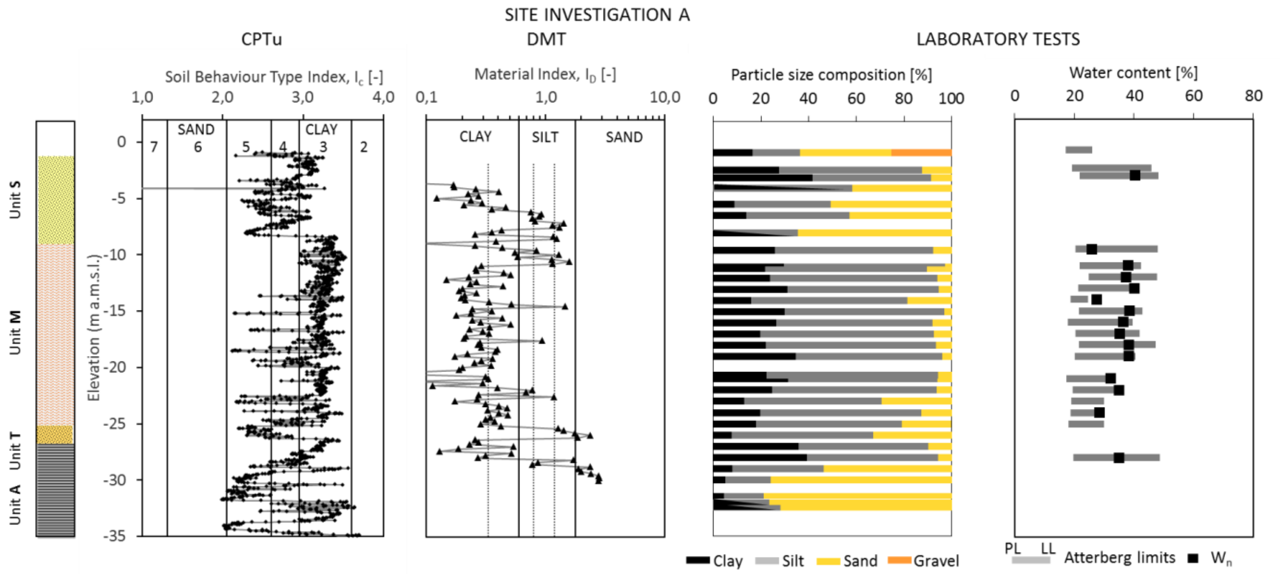


Figure 9. Site investigation A: comparison of stratigraphic identifications from CPTu and DMT with particle size composition, Atterberg limits and natural water content obtained from laboratory tests of rotary core samples

4. Main mechanical parameters

Besides to focus on the mechanical properties of the unit M, appears useful to summarize the main geotechnical parameters of all the Holocene deposits typically used for design purposes.

UNIT S. The silty sand has a unit weight of 17.5-19.5 kN/m^3, an effective friction angle between 33 and 38° and a secant Young modulus in the operative strain range from 20 to 50 MPa. The initial shear modulus (G_0) increases with depth from 30 to 60 MPa.

UNIT P. The silty clay of marsh has a unit weight of 17-18 kN/m^3, Atterberg limits LL = 50 and PL = 20, a low effective friction angle of 24-26° and very low undrained strength (5-20 kPa). The G_0 is equal to 25 MPa and the constrained modulus varies between 1 and 3 MPa.

UNIT M. The soft silty clay presents a unit weight of 17,5-18,5 kN/m^3, Atterberg limits LL = 30-55 and PL = 10-25, an effective friction angle of 28°-32° and an undrained strength ranging from 20 to 60 kPa, linearly increasing with depth. The G_0 varies from 35 to 80 MPa and the constrained modulus ranges between 2 and 6 MPa.

UNIT T. This sand deposit can be characterized with the same geotechnical parameters presented for unit S.

More details on the geotechnical characterization of the subsoil in port area can be found in Segato et al. (2010) [12].

5. Focus on soft Silty clay (Unit M)

The unit M is essentially a fine-grained soil sedimented in marine environment. The study on the evolution of the coastline pointed out that its age ranges from about 7,000 to 2,000 years old, moving from the base to the top of the stratum, respectively.

It has been observed that the geology of the deposit explains the variation of both the thickness, from about 10 m to 20 m moving from the south-western to the north-eastern sector of the port area, and of the number of thin sand strata in the soil mass. As a consequence, the coefficient of consolidation (c_v) at the scale of the deposit changes dramatically from a site where sand layers are present to another in which they are not.

From a geotechnical perspective, unit M is undoubtedly a normally consolidated, saturated, deposit. This is confirmed by 1D consolidation test results from which plots of void ratio (e) versus logarithm of vertical effective stress (σ'_v) were obtained as shown in Figure 10: the in-situ state of the sample falls at the right of the laboratory consolidation curve. The NC state of the samples is fully compatible with the geotechnical properties obtained for the unit M in terms of undrained shear strength and compressibility.

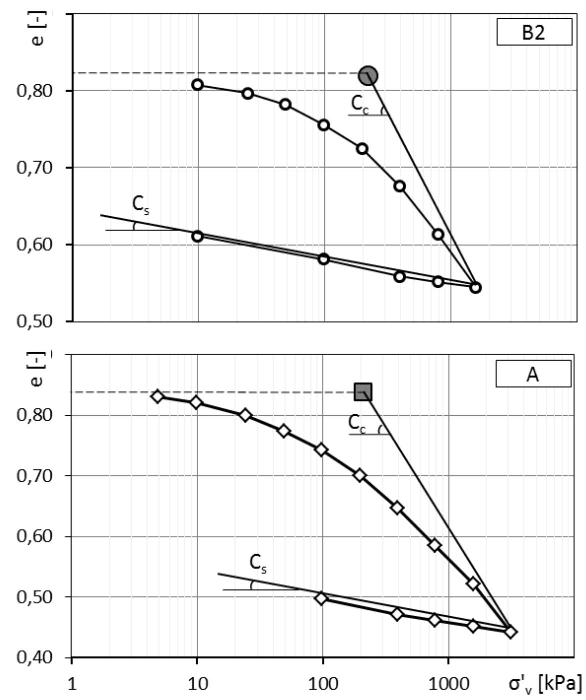


Figure 10. Unit M: plot of e against log σ'_v of two oedometer tests on undisturbed samples

Oedometer tests indicate a compression index (C_c) from 0.2 to 0.4 and a swelling index (C_s) between 0.02 and 0.05.

The determination of the mechanical parameters of unit M has been based on laboratory tests on undisturbed samples and on in-situ tests. Given the redundancy of available tests and the importance of this deposit for design purposes, the geotechnical characterization was obtained through a comparison between results of many different tests.

First of all the grading of the soil as shown in Figure 8 indicate that it can be classified as a Clayey Silt with 60-70% of silt, 20-40% of clay and 0-10% fine sand. Moreover, Figure 11, with the values of liquid limit between 30 and 55 and of the plasticity index between 10 and 25, indicates that according to the Casagrande plasticity chart unit M classifies as a clay of low plasticity (CL).

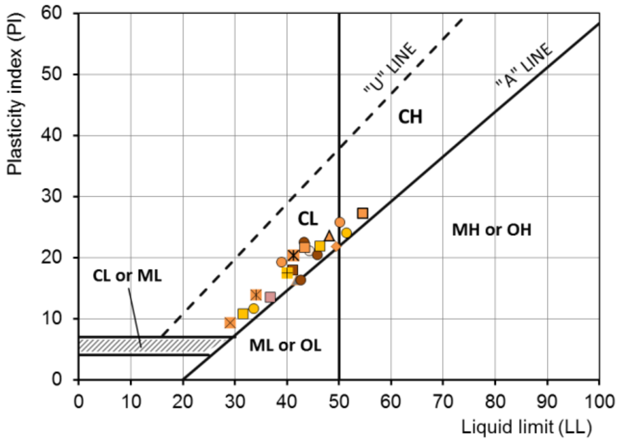


Figure 11. Unit M: Casagrande plasticity chart

5.1. Effective shear strength parameters

Effective shear strength parameters were obtained by means of isotropically consolidated drained (CD) and undrained (CU) triaxial compression tests on undisturbed samples.

The results of several triaxial tests are presented in Figure 12 in the stress plane t : s′, where t = $(\sigma_1-\sigma_3)/2$ is the maximum shear stress and s′ = $(\sigma'_1+\sigma'_3)/2$ is the mean effective stress, respectively. Results indicate that the effective strength envelope is linear and crossing the origin of the axes. It is convenient to define two linear strength envelopes to represent the plotted data: an average and a lower strength envelope with corresponding values of the effective cohesion, c′ and effective friction angle, φ′:

average envelope

$$\varphi' = \text{sen}^{-1}(\tan\beta) \cong 32° \qquad c' = 0 \tag{9}$$

lower envelope

$$\varphi' = \text{sen}^{-1}(\tan\beta) \cong 28° \qquad c' = 0 \tag{10}$$

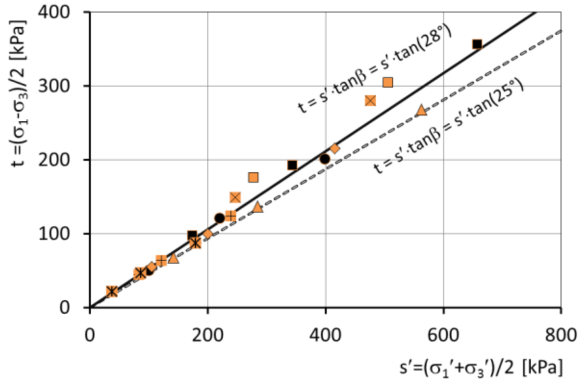


Figure 12. Unit M: results of triaxial tests on the stress plane t : s′

It is interesting to place some of the obtained results on the Kenney's chart, in which experimental data of several soils, in term of effective friction angle at constant volume (φ'_{cv}) versus Plasticity Index, are compared. As shown in Figure 13 it can be observed that the results of the unit M are right on the regression line proposed by Kenney.

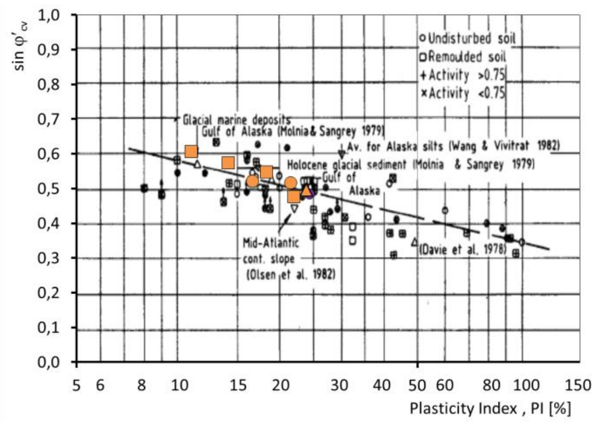


Figure 13. Unit M: relationship between friction angle at constant volume (φ'_{cv}) and Plasticity Index (modified from Kenney's chart, 1959)

5.2. Undrained shear strength

The undrained shear strength (s_u) plays a very important role in the design of geotechnical works in soft fine-grained deposit, because it represents the short-term capacity of the soil, very often the more critical one when a new quay is built. It is known that the mode of shearing affects the magnitude of undrained shear strength in clays (Mayne, 2016 [13]), so that the s_u value to be used in analysis depends on the design problem.

The undrained strength is often evaluated from correlations with in-situ testing results. In the present case study, the availability of several investigation sites allows to compare different estimates of s_u. So, even if several authors suggest to calibrate the correlations by means of laboratory testing, in this particular case it is proposed to proceed other way round, that is to estimate the undrained strength directly from in-situ probing profiles and then compare the results with those obtained from laboratory tests.

5.2.1. Undrained strength from CPTu

The undrained shear strength from CPTu test is often estimated from the net cone tip resistance by the following equation:

$$s_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{v0}}{N_{kt}} \quad (11)$$

where N_{kt} is a bearing factor. According to the study of Rad and Lunne (1988) [14], N_{kt} varies from 8 to 29 with OCR being the principal variable. More recently, for soils ranging from soft to firm clays, Lunne et al. (2005) [15] recommend $N_{kt} = 12$. Based on the piezocone data of 3 onshore and 11 offshore clays, Low et al. (2010) [16] found the range $8.6 \leq N_{kt} \leq 15.3$, with a mean value of $N_{kt} = 11.9$. These authors recommended for preliminary studies to assume $N_{kt} = 10$ for sensitive clays, $N_{kt} = 12$ for normally consolidated soft clays, $N_{kt} = 14$ for over consolidated intact clays and $N_{kt} = 25$ for over consolidated fissured clays.

According to Robertson (2012) [17], N_{kt} is largely influenced by soil sensitivity and can be linked to that via the normalized friction ratio, F_r, through the following approximate relationship:

$$N_{kt} = 10.5 + 7 \cdot \log (F_r) \quad (12)$$

Differently, taking into account that N_{kt} decreases with B_q (Mayne et al., 2015 [18]), it is possible to use the following equation to estimate the value of N_{kt}:

$$N_{kt} = 10.5 - 4.6 \cdot \ln (B_q + 0.1) \quad (13)$$

which only applies when $B_q > -0.1$.

In Figure 14, the profile of undrained shear strength evaluated from CPTu tests by using the bearing factor N_{kt} estimated by Eq. (12) and Eq. (13), is showed, respectively. Note that, to reduce the scattering of the results due to the presence of thin sand layers in the deposit M, the undrained cohesion has been evaluated only for $I_c > 2.9$, where Ic is the Soil Behavior Type index.

As shown in Table 4, under this hypothesis, the average value of N_{kt} from F_r ranges from 13.0 to 14.3 whereas the one from B_q ranges from 13.1 to 15.5 for the five considered profiles. Note that the standard deviation (S.D.) of N_{kt} value on each profile generally decreases when the thin sand layers in the unit M became less frequent, as happens moving from B2 to B10. The average N_{kt} values are slighter higher than the above suggested values .

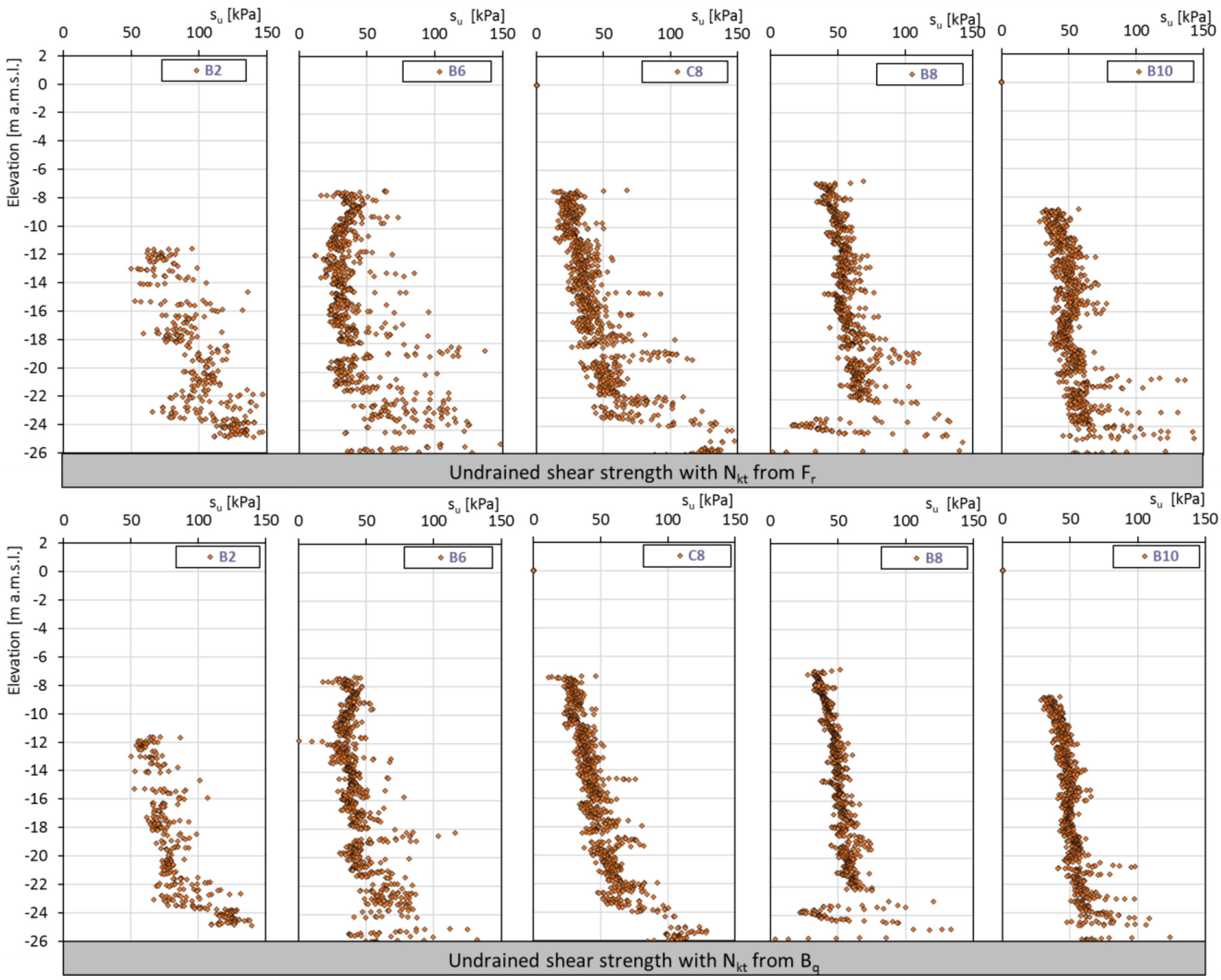


Figure 14. Profile of undrained shear strength with N_{kt} evaluated from F_r by Eq.(13), above, and from B_q by Eq. (14), below

Table 4. Statistics on N_{kt} for the considered investigation sites

Site	No. of data	N_{kt} from F_r		N_{kt} from B_q	
		$N_{kt,avg}$	S.D.	$N_{kt,avg}$	S.D.
B2	322	13.4	1.6	15.5	1.3
B6	813	14.3	1.2	13.1	2.2
C8	730	14.3	0.8	13.1	1.3
B8	736	13.0	0.6	14.8	1.1
B10	765	13.5	0.5	14.0	0.9

By using this procedure, it has been obtained a profile of s_u ranging from 50 kPa to 110 kPa for sites B2 and C6 and from 30 kPa to 80 kPa for other sites. All profiles show the typical s_u increase with depth of NC fine grained deposit. It is worth to note that B_q values are generally low, ranging between 0.2 and 0.6 in most of the cases and between 0 and 0.4 for site B2 and C6.

5.2.2. Undrained strength from DMT

Marchetti (1980) [7] proposed the following equation to estimate the undrained strength from DMT results:

$$s_u = 0.22 \cdot \sigma'_{v0} \cdot (0.5 K_D)^{1.25} \quad (14)$$

where K_D is the horizontal stress index evaluated from basic DMT data as:

$$K_D = (p_1 - p_0) / \cdot \sigma'_{v0} \quad (15)$$

Equation (14) can be used for $I_D < 1.2$ that is when the soil is fine grained. It is known that Eq. (14) is based on several hypotheses (Marchetti, 2015 [19]) and, even if a number of alternative formulations have been proposed, it still represents a reference for estimating s_u.
In Figure 15 s_u profiles evaluated from DMT for the 6 considered investigation sites are shown. It can be observed a linear trend for s_u progressively increasing with depth, with values from 30 to 50 kPa.

5.2.3. Undrained strength from laboratory tests

Some Unconsolidated Undrained triaxial compression tests (TRX-UU) have been carried out on undisturbed samples taken from continuous cored boreholes. To obtain high quality samples, the Osterberg infixion sampler has been adopted. The results of these laboratory tests have been used as reference to compare the interpretation of the in-situ tests. As example, in Figure 16 it is shown the result of the test carried out on the sample taken in B10, represented in the plane t - ε_a. The tests have been performed with a cell pressure equal to 200 kPa. It can be observed a s_u value of 32-36 kPa, a ductile response of the soil and a good agreement between the stress-strain responses of the three samples.

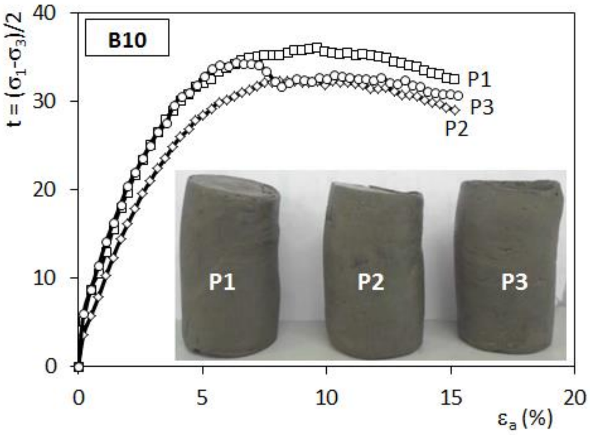


Figure 16. Unit M: example of triaxial compression test for s_u

In Figure 17 the results of the laboratory tests are represented together with the results of several pocket vane tests carried out immediately after boring. These last tests give a lower bound estimation of the real strength due to the possible disturbance caused to the soil. Measured values of the undrained shear strength range from 20 to 35 kPa.

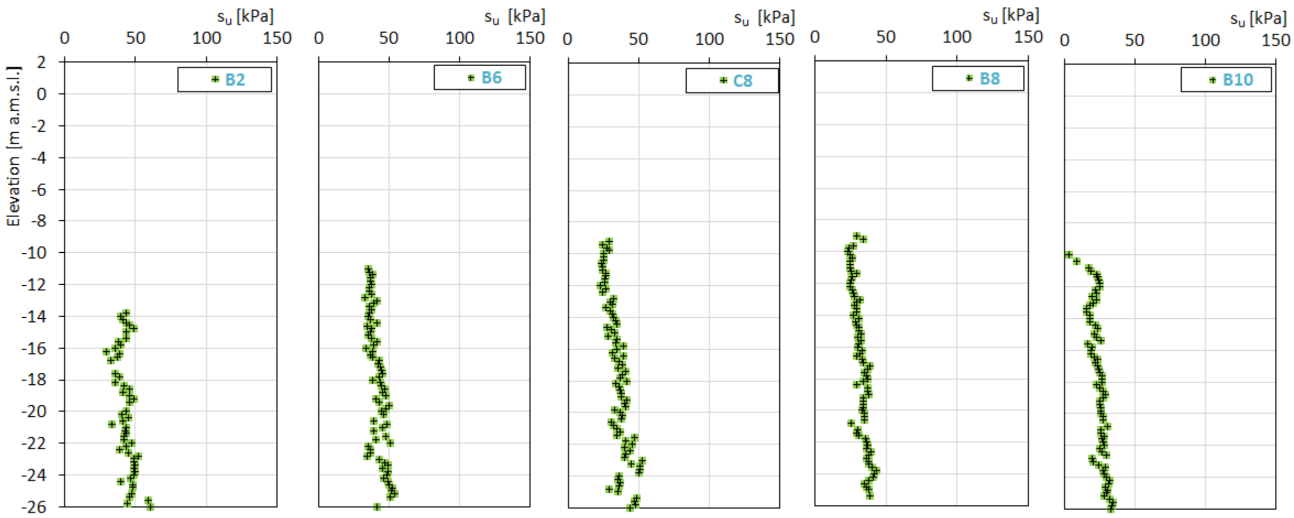


Figure 15. Profile of undrained shear strength from DMT

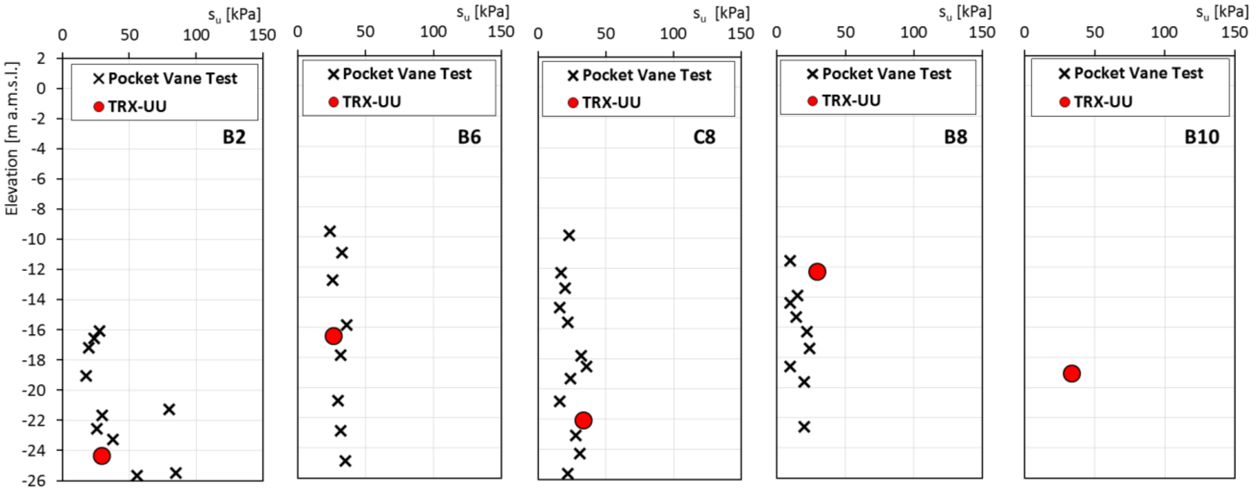


Figure 17. Profile of undrained strength from Pocket Vane Test on cored samples and Unconsolidated Undrained Triaxial test on undisturbed samples

5.2.4. Comparison of the obtained s_u values

In Figure 18 the results of the all considered tests for the five profiles take into account are shown together. For s_u from CPTu two interpretations are considered: the first one (Figure 18a) with variable N_{kt} obtained from Bq and the second one (Figure 18b) with a constant value of N_{kt} equal to 20. Assuming the results of the compressive triaxial tests as the best evaluation of the undrained shear strength, it can be observed that the best agreement is obtained by DMT test results. CPTu interpretation generally captures the order of magnitude of the s_u, even if it seems to overestimate slightly the values.

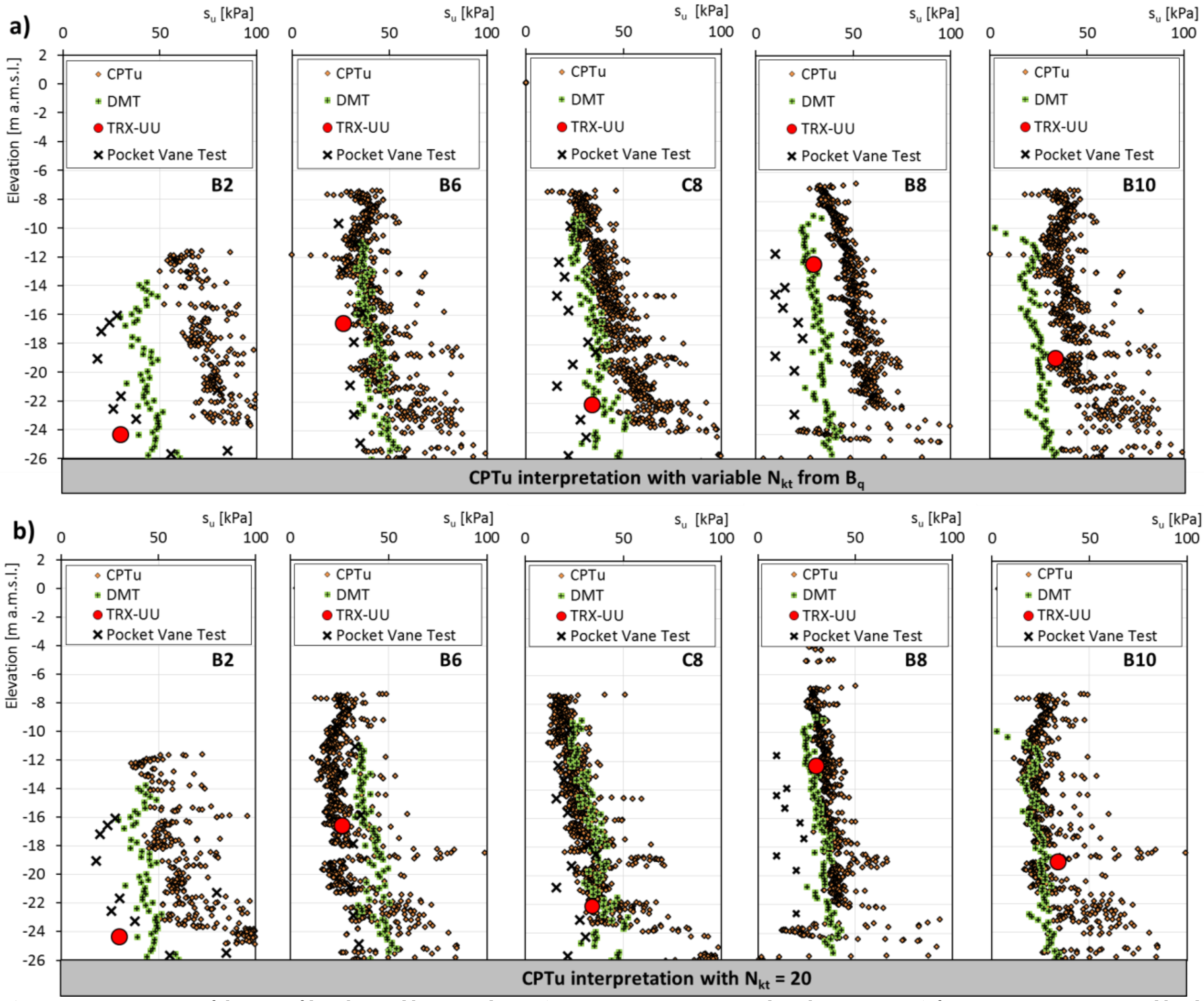


Figure 18. Comparison of the s_u profiles obtained by several tests (CPTu, DMT, TRX-UU and Pocket Vane tests) for 6 investigation sites and by different N_{kt} values for CPTu interpretation: a) variable N_{kt} from Bq; b) constant N_{kt}

An exception is represented by site B2 where, probably due to the dense alternance of cohesive and sandy layers (note that B2 is located on the south-west sector of the port), all the tests give erratic results. By using a constant values of cone factor and looking for the best overlapping of CPTu trends with DMT profiles, it has been obtained a N_{kt} = 20 (see Figure 18, below). This value is larger than expected but it allows to obtain a very good agreement between s_u estimated by CPTu and triaxial compression tests on every considered sites with the exception of B2. The reason of the observed behaviour can probably be referred to the partial drainage that can take place in mainly silty soil, when CPTu is performed at the standard rate (20 mm/s). Indeed, by studying similar deposits of the Pol Plain, several authors observed that a partial drainage for standard CPTu tests may be important and suggest careful in the use of empirical correlation (Tonni and Gottardi, 2011 [20]; Martinez et al., 2018 [21]).

6. Conclusions

The comprehensive geotechnical survey that has been recently carried out to the port area of Ravenna (Italy) to plan an upgrading of several quays has made available a large database of geotechnical information. Two aspects of the survey appear of interest:

- the possibility to link geological and geotechnical aspects of the recent deposit that constitute the first 30 m of the port area.
- the concentration in a limited number of sites of several kind of tests

The recent geologic evolution explains clearly the arrangement of all the strata that are encountered in the significative volume of interest (30 m). The attention has been mainly focused on the thick soft silty clay deposit, called unit M, due to its poor mechanical properties that govern the geotechnical design of every works in the area. In particular it appears logical the observed variation in thickness, from 10 to 20 m, when it is linked to the origin of the deposit that is related to the transgression-regression marine cycle that happened after the last glacial episode. Again, such an explanation gives a perspective to account for the variation in the number of the thin sand layers that are frequently encountered on land but disappear toward the north-eastern sector of the port, an aspect apparently of minor importance but with strong consequences on the resulting consolidation properties at the scale of the deposit.

The availability of several kind of tests on a same site allows to compare the resulting picture by the different tests of the same stratigraphy. Moreover, the large number of laboratory tests on undisturbed and disturbed samples, allows to compare parameters estimated by correlation with measured ones. Two aspects have been analysed in depth, due to their implication on the geotechnical design:

- the identification of the soil;
- the undrained shear strength.

The identification of the soils has been done for CPTu and DMT profiles by using the most common correlations proposed in technical literature and the results have been compared with particle size distribution curves obtained in laboratory. The comparison indicates a good agreement between CPTu and DMT tests. However, the unit M is identified by the in-situ test as more clayey than its real granulometric composition. This happens when the general mechanical response of the soil is controlled by its finer components.

The undrained shear strength of the unit M has been estimated from CPTu and DMT tests and compared with values from unconsolidated undrained triaxial testing on undisturbed samples. The comparison indicates that the DMT gives a good estimation of s_u while CPTu tend to overestimate the values from laboratory testing. To investigate this outcome it has been evaluated the cone factor N_{kt} that allows to obtain the best fit of the s_u profile from CPTu with laboratory results. Such a value, equal to 20, is higher than the values suggested in the literature for the cone factor to interpret the CPTu. The reason could be probably ascribed to the silty nature of the deposit that determines a partial drainage during the cone penetration at the standard speed rate. This explanation is supported by other studies on similar deposits of the Po plain in which a partial drainage has been observed.

In conclusion, the concentration of several kind of tests in a limited number a sites appeared a powerful tool to plan extensive geotechnical investigations. The in-situ tests are able to collect a huge number of information, but a limited number of targeted laboratory tests are necessary to guarantee the reliability of the estimated geotechnical parameters from CPTu and DMT.

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