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Research article

Assessing compressibility characteristics of silty soils from CPTU: lessons learnt from the Treporti Test Site, Venetian Lagoon (Italy)

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Abstract: This paper explores the capability of cone penetration testing as an effective tool to estimate the compressibility characteristics of silts, silt mixtures and sand mixtures. The study uses part of the large amount of piezocone (CPTU) data and subsoil deformation measurements assembled over approximately 6 years at the Treporti Test Site (Venice, Italy), within an extensive research project aimed at thoroughly analyzing the stress-strain-time response of the complex assortment of sandy and silty sediments forming the Venetian lagoon subsoil. The combined analysis of piezocone data and subsoil deformations measured beneath a full-scale test embankment primarily showed that the existing interpretation approaches developed for 'standard' sands or clays - and thus based on the idealized assumption of fully drained or fully undrained testing conditions - generally result in invalid estimates of soil compressibility parameters. Therefore, site-specific correlations have been developed in order to correctly predict the compressibility of Venetian sediments, both primary and secondary, and consequently obtain the best fit of the measured settlements. By taking this very well documented case study as a base, the proposed paper aims at discussing some key issues on the use of piezocone tests for the geotechnical characterization of silts and other sedimentary soils with very scattered grain size distributions, thus falling in the so-called intermediate permeability range. The idea is basically to provide a critical appraisal of the CPTU-based approaches currently available for the mechanical characterization of natural soils, and explore potential advances in the interpretation methods, which might take account of possible partial drainage phenomena around the advancing cone during the test.

Keywords: piezocone; silt, sand mixtures; compressibility; partial drainage; Treporti Test Site; Venetian lagoon

1. Introduction

The Treporti Test Site (TTS) was set up in the early 2000s, within a major joint research project aimed at better understanding the stress-strain-time response of the heterogeneous, predominantly silty sediments underlying the historic city of Venice and the surrounding lagoon. This project came after a series of valuable studies on the shallow Pleistocene sediments of the Venetian lagoon basin, carried out in the previous 30 years: first in the ’70s [1,2], in relation to the regional land subsidence, and then in the ’90s, in consequence of the comprehensive site investigation program devised for the design of the submersible gates (Mo.S.E.) intended to protect Venice from recurrent flooding [3,4].

Following the experience gained from the above projects, in 2001 a new extensive research program—involving the Italian Universities of Bologna, Padova and L’Aquila—was launched, with the financial support of the Italian Ministry of Education. The aim was to identify a representative test site of the typical Venetian lagoon subsoil and carry out an accurate geotechnical characterization by means of in situ-tests (boreholes, piezocone and flat dilatometer tests) and high-quality laboratory tests, the attention being mainly focused on the compressibility characteristics of Venetian sediments. To this purpose, the research project also included the gradual construction of a full-scale vertical-walled cylindrical test bank, along with a very detailed monitoring system, which provided measurements of pore water pressures, surface settlements and subsoil deformations beneath the bank during the following 6 years. The gradual removal of the bank started in June 2007 and was fully completed at the end of March 2008.

As a consequence of the exhaustive experimental program carried out in this relatively small area, coupled with continuous monitoring of subsoil deformations, a complete database of field measurements has become available for interpretation in terms of compressibility characteristics of Venetian soils. Taking this well-documented site as a base, this paper explores the capability of cone penetration as an effective tool to estimate the compressibility characteristics of silts, silt mixtures and sand mixtures and discusses the effectiveness of the available interpretation approaches, most of them developed for “standard” sands or clays, in predicting reliable values of the compression moduli when applied to such intermediate sediments. Empirical, site-specific correlations for the estimate of primary and secondary compression characteristics are proposed. According to the experience gained within the Treporti Test Site project, the paper is meant to draw attention to some key issues on the interpretation of piezocone test results in silts and other sedimentary soils having very scattered grain size distributions and therefore potentially affected by partial drainage effects during the test.

2. Outline of the geological setting and basic features of Venetian Lagoon sediments

The Venetian Lagoon is underlain by about 800 m of Quaternary deposits, which originated from alternating continental and marine sedimentation in consequence of marine regressions and transgressions occurred over the last 2 million years. In particular, the upper 100 m of the Venetian lagoon basin appear as a complex system of interbedded continental deposits, such as sands, silts and silty clays, chaotically accumulated during the Würm glaciation. Only the shallowest sediments, from ground level to 5–10 m in depth, are indeed referable to the Holocene Epoch. Thin layers of peat can be also found, due to the occasional presence of lacustrine depositional environments [5]. According to the Unified Soil Classification System, 95% of the Venetian sediments can be grouped into the following three main classes: medium-fine sands (SP-SM) with subangular grains, silts (ML), and very silty clays (CL). The remaining 5% may be classified as organic clay and peat.

Sediments are generally normally consolidated or slightly overconsolidated due to aging, with the exception of the so-called *Caranto*, a highly overconsolidated silty clay forming the top Würmian layer, which was subjected to desiccation during a sedimentation gap between 18,000 and 7000 yr BP. Most of the Venetian historic buildings are supported on driven wooden compaction piles, sometimes reaching such very stiff formation. However, the *Caranto* is not present all over the lagoon area due to subsequent erosion phenomena and may be from a few centimeters to a few meters thick, with a tendency to become deeper from the mainland towards the shoreline [3].

Despite the observed grain size heterogeneity, research has shown that Venetian sediments have a common mineralogical composition. Sands are predominantly calcareous and siliceous, with feldspar as other significant component and minor quantities of mica sporadically detected. Furthermore, quartz and feldspar grains are generally angular, thus suggesting a limited transportation process. Silts and silty clays, which originated from mechanical degradation of sands, have a content of non-active clay minerals (illite with minor quantities of chlorite, kaolinite and smectite) typically lower than 20% in weight. Clay minerals appear generally highly crystallized, on account of their detrital origin [3].

As a direct consequence of the common mineralogical origin and depositional environment, it was shown that the different soil classes exhibit a similar mechanical behavior, mainly controlled by inter-granular friction [6].

3. The Treporti Test Site

The Treporti Test Site is a 15,000 m^2 test site area, located just outside the old fishermen village of Treporti, facing the North-Eastern Venetian Lagoon (Figure 1). The site was in use from 2001 to 2008, within the context of a major collaborative research project aimed at better understanding the mechanical behavior of the highly heterogeneous, predominantly silty soils forming the upper 100 m of the Venetian Lagoon basin. The Treporti subsoil had been indeed recognized as truly representative of the relevant typical stratigraphic conditions, especially of the portion surrounding the three inlets connecting the lagoon to the Adriatic Sea, and thus considered as appropriate for the accurate investigation of compressibility and consolidation characteristics of Venetian sediments. Such piece of information appeared to be of particular interest for the prediction of both short-term and long-term settlements of the submersible mobile barriers across the three inlets for the protection of Venice against flooding, currently nearing completion, as well as of the new offshore breakwaters and the additional protection structures devised as part of the Mo.S.E. project [7].

In this area, a very detailed in situ testing campaign was carried out and a full-scale 6.7 m high, 40 m diameter, vertical-walled cylindrical test bank (Figure 2) was progressively built and continuously monitored during approximately 6 years [8]. Details on the whole site investigation program and the test-bank monitoring are provided below.

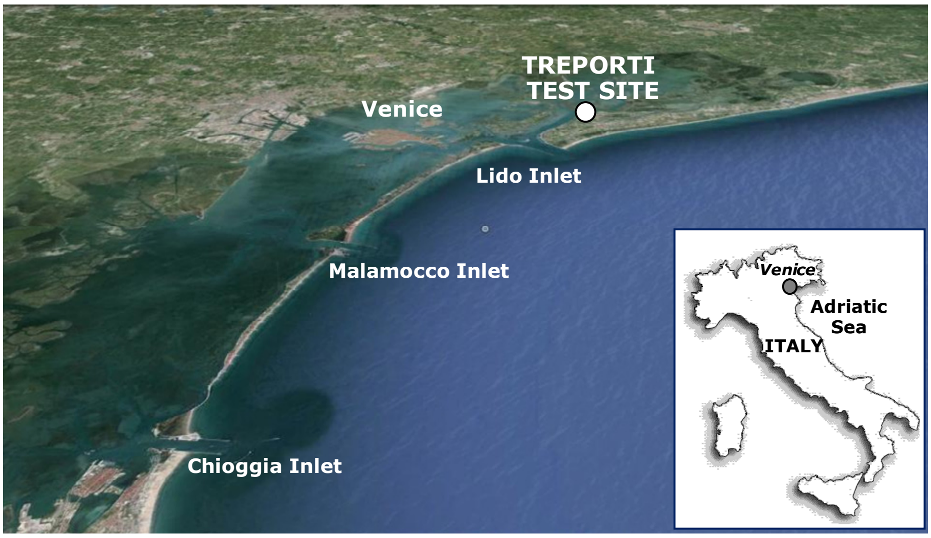

Figure 1. Satellite view of the Venetian lagoon and location of the Treporti Test Site**.**

Figure 2. View of the cylindrical test bank.

3.1. Site investigation campaigns

The accurate geotechnical characterization of the Treporti subsoil was mainly based on *in-situ* testing: boreholes with undisturbed soil sampling, a large number of piezocone tests (CPTU) and dilatometer tests (DMT) together with a few seismic piezocone (SCPTU) and seismic dilatometer (SDMT) tests. The CPTU testing program, in particular, was implemented by the Mobile Geotechnical Laboratory of the University of Bologna, using a Delft Geotechnics piezocone equipment, whereas seismic piezocone and part of the seismic dilatometer tests were performed with the equipment available at the Georgia Tech University. The University of L'Aquila was in charge of the whole "standard" DMT testing campaign, together with a few SDMT. Such extensive experimental program was carried out in four different phases, basically corresponding to the different phases of the loading history applied by the test bank.

Initially, No. 7 piezocone tests (CPTUs from 1 to 7 in Figure 3) were carried out in order to develop a preliminary stratigraphic model of the whole 15,000 m^2 area and thus gain some insight into the horizontal spatial variability of the Treporti subsoil [5]. The basic idea behind this *First Phase Testing Campaign* was essentially to identify the most appropriate location for the loading bank to be built, which had to lie on the most compressible soil layers. Piezocone tests from No. 1 to No. 4 were pushed to a 44–45 m depth, the remaining CPTUs were stopped at approximately 30 m. No dissipation tests were performed in this first phase. The water table was found to be very close to the ground level, consistent with the nearby canal level, and subjected to local tidal excursions, about ±0.5 m twice a day.

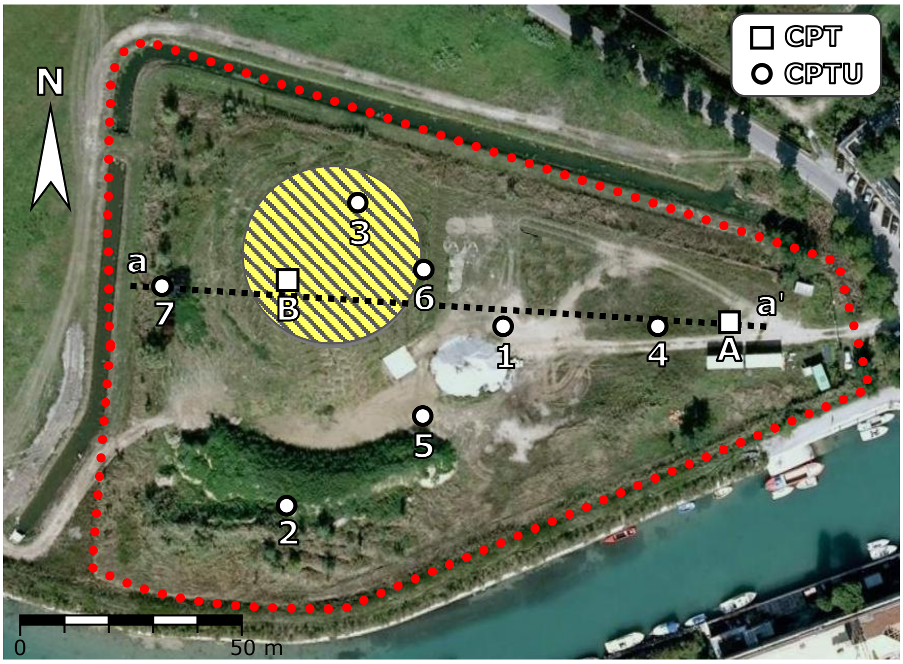

Figure 3. Satellite view of the test site area and location of the first-phase site investigations. The dashed circular area indicates the position finally selected for the test bank.

The main site investigation campaign, also referred to as *Second Phase Testing Campaign*, was devised to provide an accurate geotechnical characterization of the subsoil beneath the loading bank, prior to its construction. It consisted of No. 10 CPTUs, also including dissipation tests [5,9], No. 10 DMTs [10,11], No. 2 standard mechanical CPTs and No. 4 boreholes, the latter pushed to 60 m in depth. All the tests were located within a 45 m diameter circular area. The CPTUs and DMTs were located in adjacent positions and numbered from 11 to 20. They were lined up along three diameters, splitting the circular area in three 120° identical slices, and placed along two concentric circumferences (inner diameter, D = 30 m, and outer diameter, D = 45 m) with respect to the loading bank (D = 40 m). All tests were pushed as deep as possible, depending on the maximum available thrust, typically to about 42 m. Such main testing program was shortly after integrated with a few supplementary tests, i.e. No. 3 pairs of SCPTU and SDMT [12,13], which were carried out close to three existing pairs of CPTU/DMT.

The so-called *Third Phase Testing Campaign*, performed at the end of the loading bank construction, consisted of No. 2 CPTUs (CPTU 29 and CPTU 30) and No. 2 DMTs. All these soundings were carried out from the top of the bank and located as close as possible to the tests of the *Second Phase*, the idea being to investigate the overburden stress effect on the experimental soil response by comparing two adjacent vertical alignments.

In 2008, a final *Fourth Phase* testing campaign was launched at the end of the loading bank removal, with the aim of examining the stress-history effect on the soil mechanical properties along a number of verticals which had been already investigated in both the pre-bank and post-bank construction phases. Hence, No. 3 CPTUs and No. 4 SDMT were carried out: a couple of soundings CPTU-SDMT was located in the center of the loaded area, the others were located on the 30 m diameter circumference of the loaded area. Furthermore, the final part of the experimental program was complemented by two additional piezocone tests (CPTU 34_{min} and CPTU 34_{max}), both located in the center of the loaded area and performed at non-standard penetration rates, i.e. at v_{min} = 0.15 cm/s and v_{max} = 4.0 cm/s. These tests were intended to identify potential partial drainage effects [14,15] which are very likely to occur during cone penetration in intermediate soils [16–18].

A sketch of the test locations for the second, third and fourth phases is provided in Figure 4.

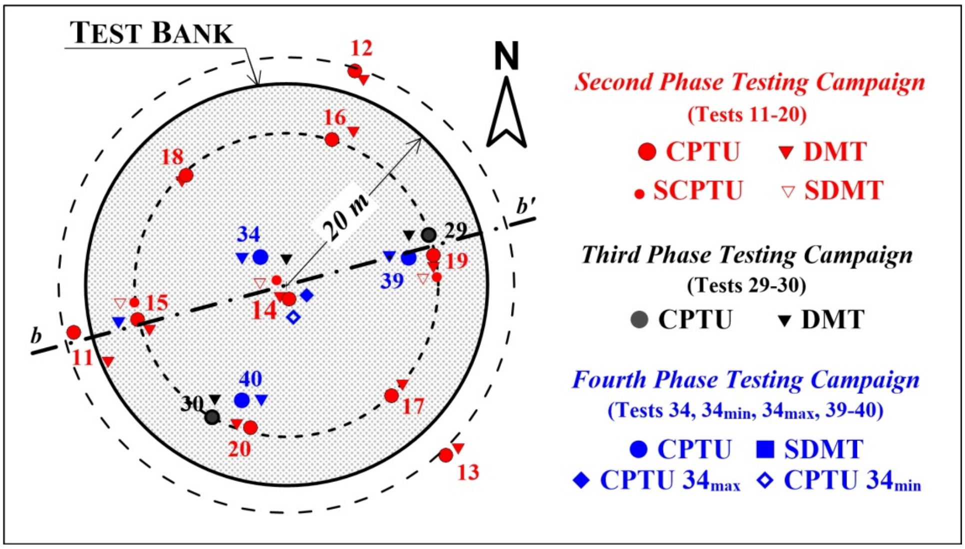

Figure 4. Location of tests carried out within the loaded area, in the second, third and fourth phases of the testing campaign.

3.2. The loading bank construction and monitoring system

The construction of the 40 m diameter, geogrid-reinforced vertical-walled cylindrical sand bank was carried out in 13 steps, 0.5 m in height each, from September 2002 to March 2003. Short, prefabricated vertical drains were first installed, both to accelerate drainage of the top soft clay layer and to prevent possible lateral squeezing during construction. When completed, the sand bank was covered with 0.2 m of gravel, thus reaching a total height of 6.7 m above the original ground level. At the end of construction, a uniform load of about 106 kPa was estimated to be applied on the circular area of the loading bank. The test bank remained in place until April 2007 and then gradually removed during the following year.

Prior to the bank construction, a very precise and varied monitoring system was installed [8] in order to track surface vertical displacements, horizontal displacements with depths, induced total vertical stress beneath the bank, pore water pressure and, finally, subsoil vertical strains, the latter representing a crucial piece of information for the analysis of Venetian sediment compressibility discussed herein.

Measurements of subsoil vertical strains were indeed obtained by means of a device known as sliding deformeter, able to monitor soil axial displacements at 1 m intervals, with a precision of ±30 μm. The equipment consists in a telescopic guide casing installed in a pre-drilled borehole and suitably grouted to the surrounding soil. The casing is therefore free to slide according to soil deformation and a measuring probe can be inserted to measure the actual distance between reference points located along the axis. Four sliding deformeters were installed beneath the bank, one (labelled as SL3) in the center of the circular loaded area and the remaining three (SL1, SL2, SL4) symmetrically located along a 30 m diameter concentric circumference. They provided measurements of soil vertical strain from the ground level to 57 m in depth, for approximately 6 years, i.e. from the start of the loading bank construction to the end of its removal.

A scheme of the whole construction history of the test bank (thus also including its removal), together with settlements obtained from integration of the vertical strains measured by the sliding deformeters, is provided in Figure 5. For a useful comparison, the curve of surface vertical displacements obtained from a GPS receiver located in the center of the bank is also reported in the figure.

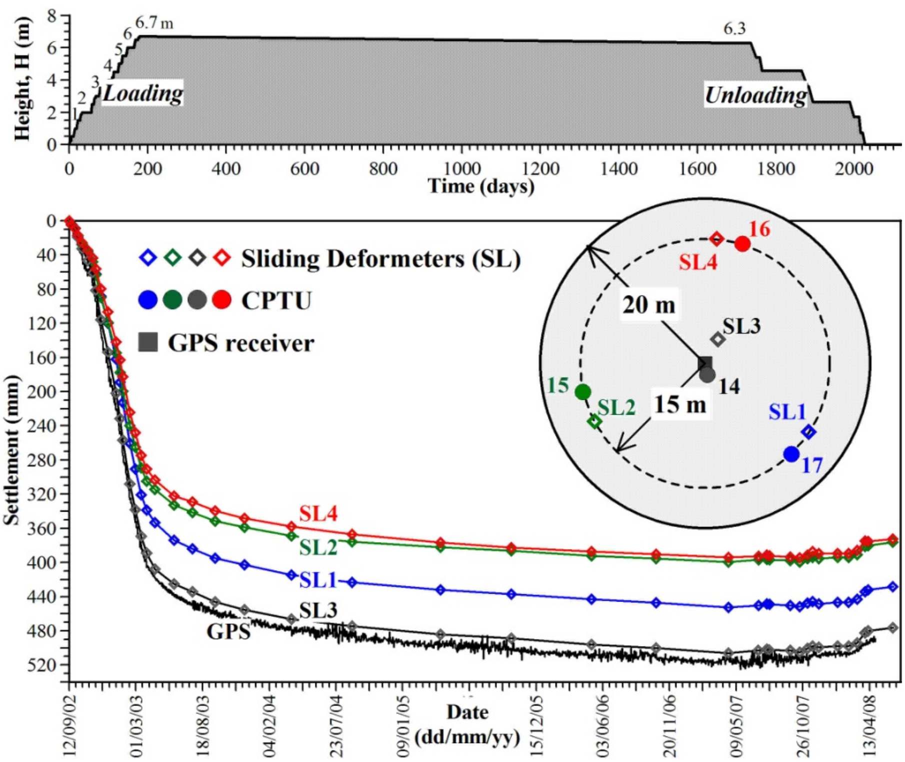

Figure 5. Construction history of the test bank and settlements measured beneath the loaded area, together with location of the sliding deformeters and of relevant piezocone tests.

4. The piezocone database

In this paper, attention will be focused on the large amount of piezocone data collected during the different phases of the research project, with the attention particularly focused on the evaluation of soil compressibility and consolidation characteristics.

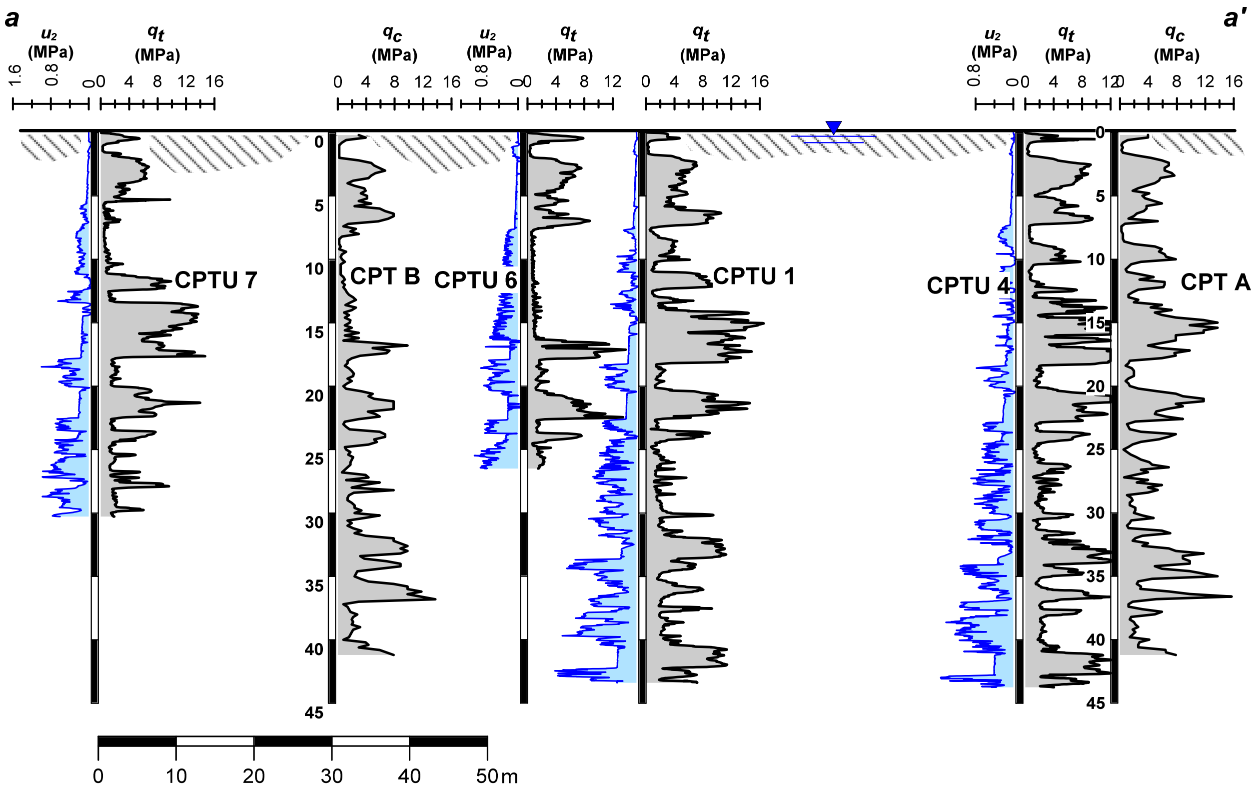

Figure 6. Cross-section *a-a'* (WE) of the test site area.

In order to provide a preliminary description of the stratigraphic conditions of the whole Treporti Test Site area, Figure 6 shows the corrected cone resistance (q_t) and pore water pressure (u_2) profiles of a number of CPTUs carried out in the *First Phase Testing Campaign*, located along a WE alignment (cross-section *a-a'* drawn in Figure 3). Measurements of cone tip resistance q_c, obtained from a couple of additional mechanical tests (CPTA and CPTB), have been also plotted.

The whole set of q_t and u_2 profiles provides clear evidence of a highly stratified subsoil, with a well-defined top layer of sand, 6–7 m thick, followed by a dense alternation of thin layers of sandy and clayey silts. The pore water pressure profiles, in particular, rarely follow up the hydrostatic level, often fall below it, but never exhibit high values of excess pore water pressure Δu_2, as typical in silts. Occasionally, but consistently throughout the test site area, thin layers of peat and organic soil are detected, being typically revealed by high values of the friction ratio (f_s/q_t).

A potentially more compressible fine-grained unit, mainly composed of silts, can be identified in all the soundings from 7–8 m to 20 m in depth, though with different thicknesses within the site area. This silty unit is indeed often interbedded with a clean sand layer which tends to progressively reduce its thickness moving eastwards, and then disappears. Only tests CPTU 2 and CPTU 3 (not plotted in Figure 6) indicated the occurrence of a continuous silty layer from 8 to 20 m. As a result, the northwest corner of the area was eventually selected as the most appropriate location of the loading bank.

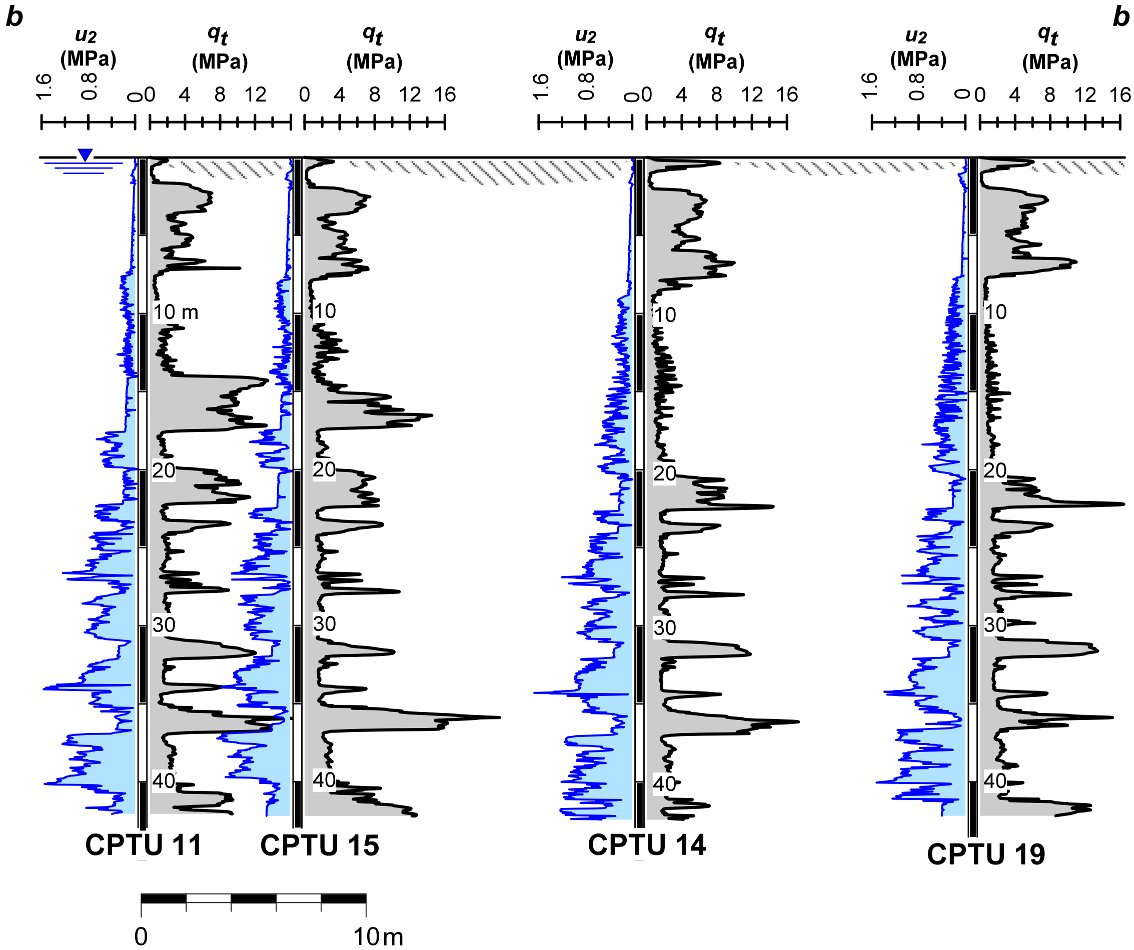

Figure 7. Cross section along the test-bank diameter direction *b-b'*.

A very detailed description of the stratigraphic conditions solely beneath the loading area is provided in Figure 7 with reference to the cross-section *b-b'*, drawn along the diameter connecting the 2nd phase tests CPTU11–CPTU19 (see Figure 4). Furthermore, in order to provide a better insight into the stratigraphic complexity of the Treporti subsoil, results from the application of two different soil classification methods to the central test CPTU 14 are plotted in Figure 8, together with the relevant profiles of the corrected cone resistance q_t, pore water pressure u_2, sleeve friction f_s and shear wave velocity V_s, this being obtained from an adjacent seismic piezocone test, SCPT14 [12]. The dotted line in plot (c) indicates the equilibrium pore pressure u_0, the phreatic surface being located at 0.7 m in depth. The figure also shows the grading characteristics of a number of soil samples extracted in the upper 45 m of a borehole drilled in the center of the test bank area.

The interpretation of piezocone data in terms of the Schneider et al. [19] classification approach (column f) reveals a general predominance of intermediate sediments, with most of the experimental points from 8 to 20 m falling in the domain of silts (1a) or in *transitional soils* (3), the latter including a wide variety of soil mixtures (e.g. clayey sands, silty sands, silty sands with clay, clayey sands with silt). A complex assortment of fine (1b) to coarse (2) sediments, without any evident specific trend, can be observed below 22 m. Compared to the well-known *Soil Behaviour Type* (SBT_n) classification framework developed by Robertson [20], also reported in Figure 8, this procedure provides a somewhat more accurate description of the general intermediate nature of the Venetian sediments.

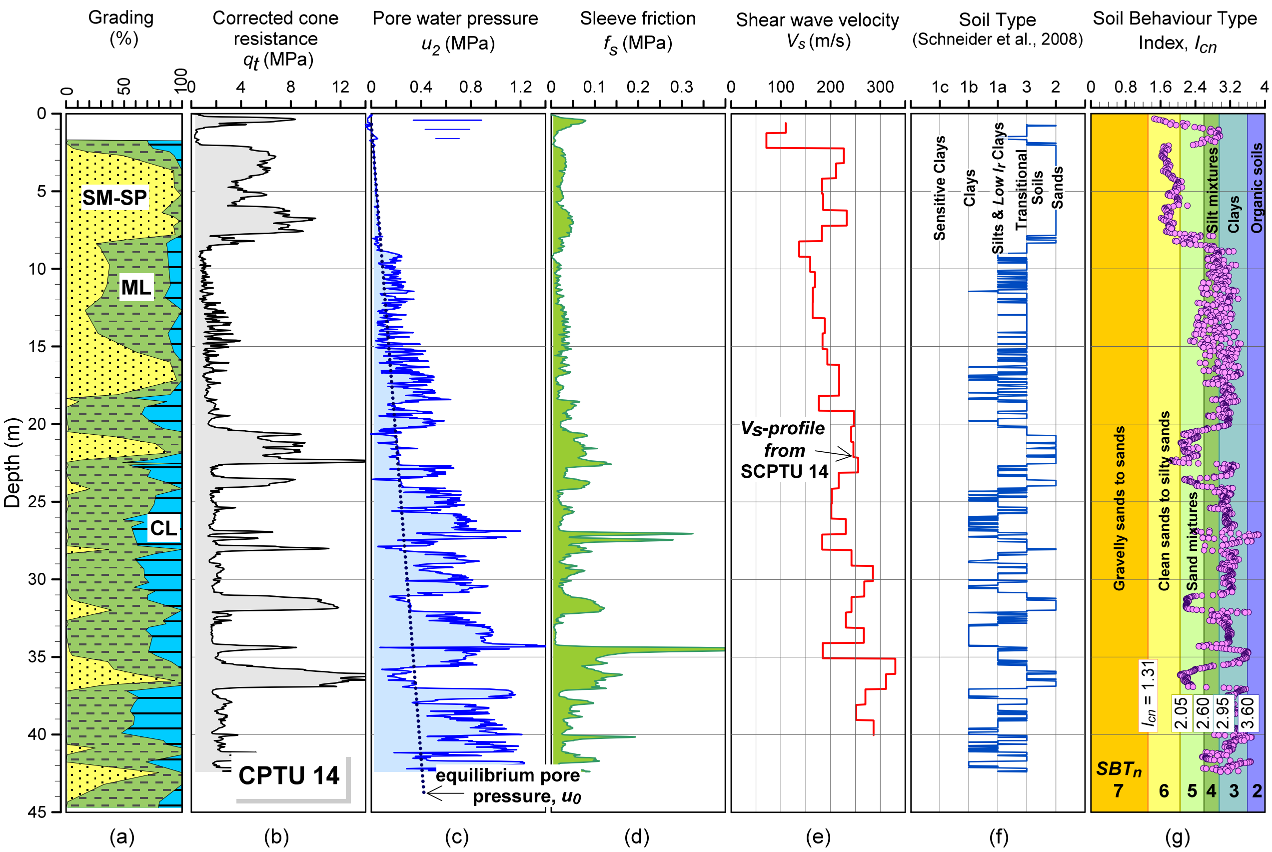

Figure 8. CPTU 14 log profiles and classification results.

Such observation seems to be confirmed also in relation to the updated *Soil Behaviour Type Chart* lately proposed by Robertson [21], consisting in a modification of the classical SBT_n domains by using a modified Soil Behaviour Type Index I_B in conjunction with a contractive-dilative boundary *CD*. Figure 9 shows indeed that only a limited part of the Venetian sediments from 8 to 20 m in depth fall in the region of *transitional soils*, obeying the condition $22 < I_B < 32$, whilst a large number of data points are still classified as contractive clay-like soils, at times sensitive soils. The index properties of a selection of soil samples extracted from boreholes drilled within the loaded area in the *Second Phase Testing Campaign* are provided in Table 1. Additional data can be found in Tonni and Gottardi [5].

It is worth observing here that in the stratigraphic conditions described above, with a prevalence of silts and *transitional soils*, cone penetration is often occurring in partial drainage conditions. Partial drainage in Venetian silty sediments is also confirmed by the application to CPTU 14 of the simple interpretation procedure described by Schnaid et al. [16], which relies on the combined analysis of the pore pressure parameter B_q, the normalized cone resistance Q_t and the computed undrained shear strength ratio s_u/σ'_{v0}. As shown in Figure 10, the method assumes that in normally consolidated silty soils partial drainage prevails when B_q < 0.3, as in the case of most of the experimental data collected between 8 and 20 m in depth. In such case, the resultant estimated undrained shear strength ratio turns out to be significantly higher than values commonly accepted for NC or slightly OC soils, when a cone factor N_{kt} = 15 [22] is assumed to convert q_t to the undrained shear strength s_u.

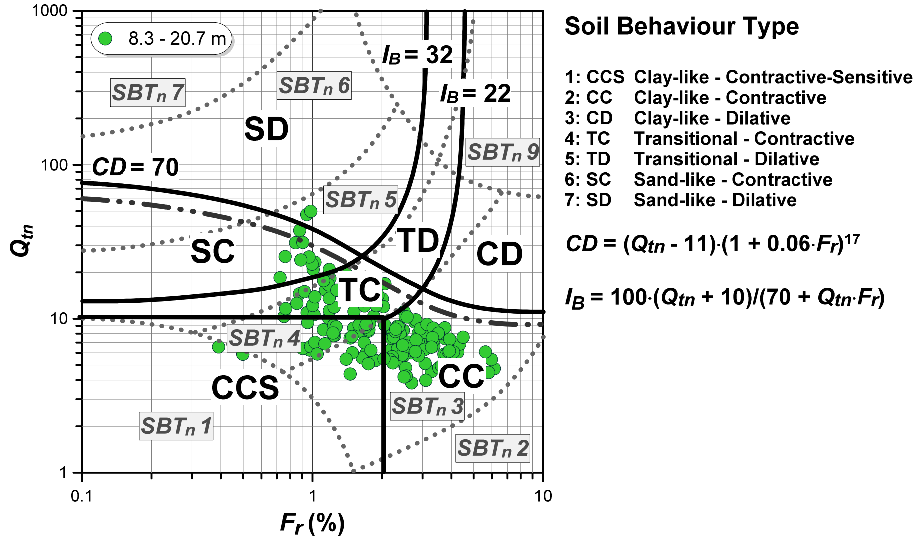

Figure 9. CPTU-based classification of Venetian soils from 8.3 to 20.7 m in depth, according to the updated SBT_n chart [21]. In the grey boxes, the reference to the former SBT_n zones [20].

Table 1. Index properties of the Treporti Test Site sediments, at various depths.

Depth (m)	G (-)	γ_n (kN/m^3)	e (-)	w (%)	w_L (%)	w_P (%)
4.1	2.79	20.0	0.81	29.43	-	-
5.1	2.80	19.4	0.85	28.22	-	-
7.0	2.79	19.6	0.81	27.45	-	-
10.1	2.80	19.9	0.82	29.51	-	-
12.1	2.77	18.9	0.94	32.28	-	-
14.5	2.80	20.0	0.78	27.07	-	-
15.9	2.75	19.3	0.91	31.98	32.58	28.25
17.0	2.75	20.6	0.69	26.89	33.13	22.74
19.1	2.79	19.5	0.87	30.94	38.09	23.43
21.9	2.75	18.3	0.95	29.91	-	-
23.3	2.78	18.7	0.90	27.56	29.32	18.51
24.8	2.72	17.9	1.09	37.60	43.30	23.40

G, grain specific gravity; γ_n, in situ unit weight; e, in situ void ratio; w, in situ water content; w_L, liquid limit; w_P, plastic limit.

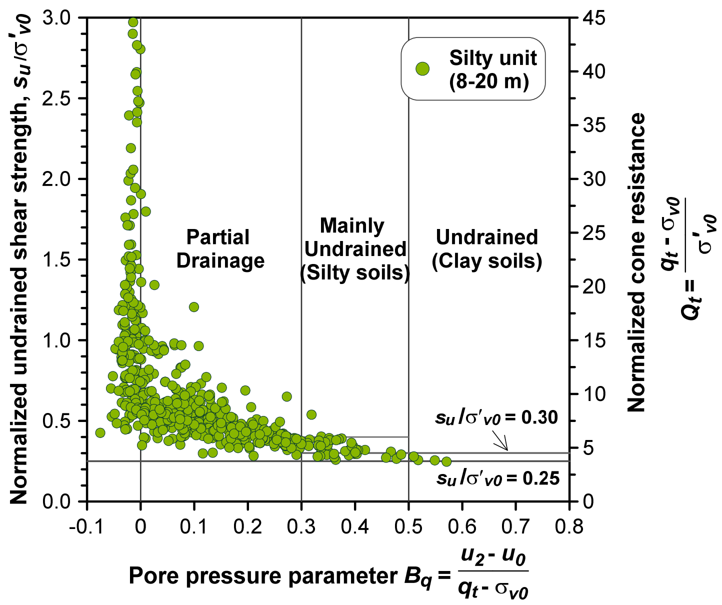

Figure 10. Assessment of drainage conditions on CPTU 14 data, according to the simplified procedure described by Schnaid et al. [16].

Further evidence of potential partial consolidation effects is provided by results of dissipation tests in the silty layer between 8–20 m. Several dissipation tests were indeed carried out in the second and third phases of the testing campaign, with the aim of estimating the consolidation characteristics of sediments detected between 8 and 30 m in depth. A selection of representative dissipation curves, showing data collected both in silts and in the finer deep layers, is provided in Figure 11. The decay of the excess pore pressures was found to be in general rather rapid in the whole set of tests, taking approximately 1000 s in the silty unit for a complete dissipation, a little longer in silty clays.

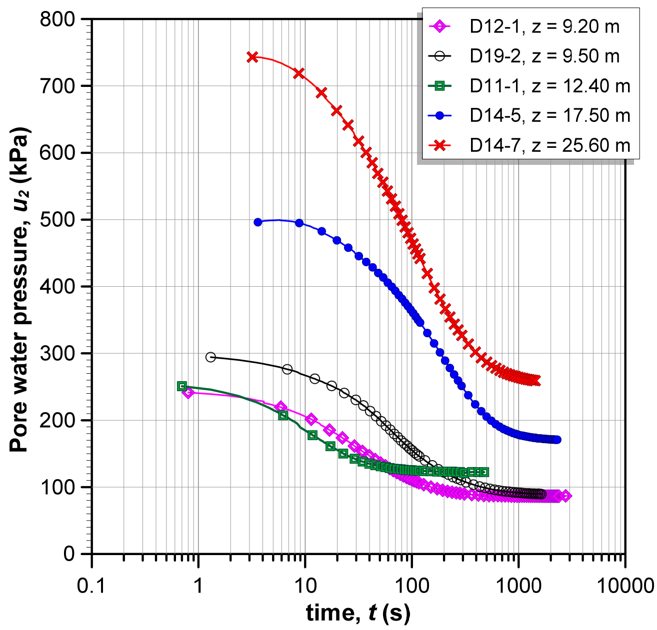

Figure 11. Pore pressure decay with time during a few representative dissipation tests.

It is important to observe, in particular, that the time required for 50% dissipation (t_{50}) was less than 100 seconds in most of the dissipation tests. According to DeJong and Randolph [23], in such case the effects of partial drainage on the interpretation of pore water pressure dissipation should be taken in careful consideration and inaccuracy in results due to application of conventional procedures, based on the assumption of initial fully undrained conditions around the advancing cone, may be greater than 20%.

As a result, the values of the horizontal coefficient of consolidation c_h provided by the well-established method of Teh and Houlsby [24]—typically in the range 5×10^{-5}–10^{-4} m^2/s for silts, around 10^{-5} m^2/s for the clayey silts below 18 m in depth—should be considered with great care. At the same time, the values of the vertical coefficient of consolidation c_v, as obtained from a number of laboratory oedometer tests, turned out to be on average 2×10^{-5} m^2/s for silts (between 8 and 16 m in depth) and 8×10^{-6} m^2/s for the clayey silts at 18–20 m. On the other hand, such values cannot be directly compared with c_h, because of soil anisotropy, especially significant for hydraulic properties of highly stratified deposits as in the case of the Venetian subsoil.

As an example of the 3rd phase tests and of the effect induced by the loading bank on the in-situ soil response, Figure 12 shows a comparison, in terms of corrected cone resistance q_t and normalized cone resistances Q_t and Q_{tn}, between the 2nd phase test CPTU20 and the adjacent CPTU30, the latter being carried out from the bank crest. Both tests were located at a distance of 15 m from the center of the loaded area. The plot is limited to the upper 20 m, where the presence of the loading bank seems to have a more pronounced effect on field measurements.

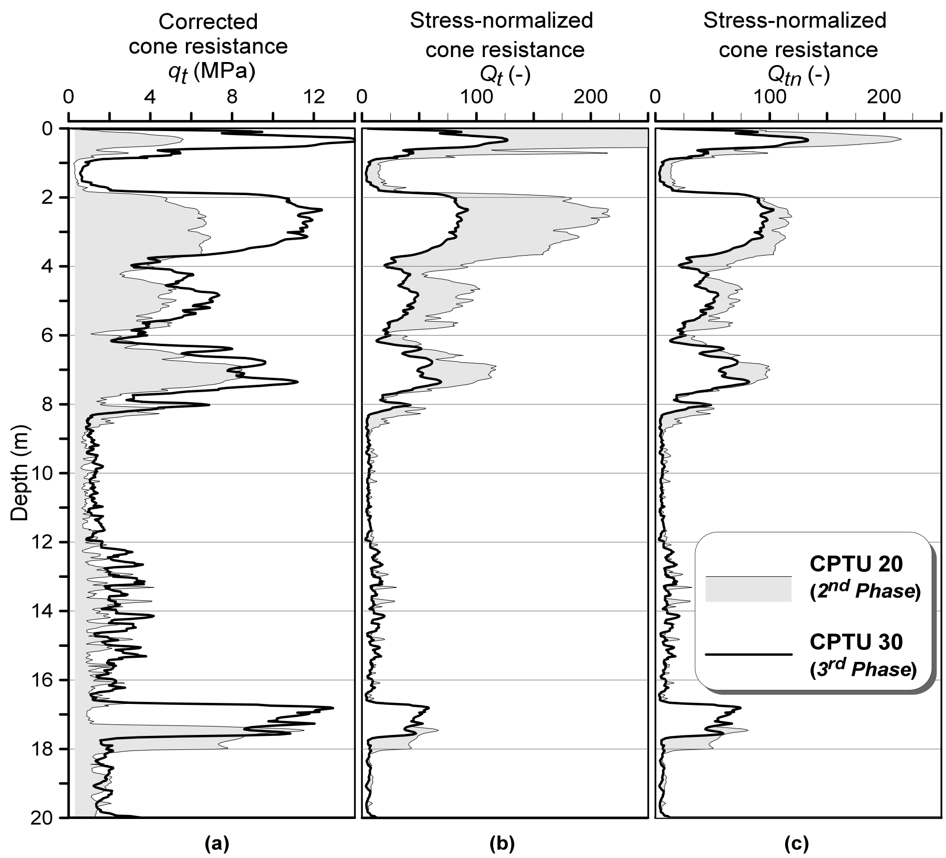

Figure 12. Comparison between cone resistance profiles of 2nd and 3rd phase tests.

The influence of the applied stress increment on the corrected cone resistance measurements is particularly evident in the upper sandy layer, from 2 m to approximately 8 m in depth. Furthermore, an expected relative shift of the cone resistance profiles can be noticed, as a consequence of the vertical strains induced by the loading bank. It is worth observing that, despite the overburden stress normalization, a significant difference in the values of Q_t can be still detected within the sandy layer, although such a response cannot be explained either by potential horizontal spatial variability of the subsoil nor by the estimated changes of the void ratio due to loading. The effect of overburden stress on CPTU measurements, especially those in sands, seems indeed to be more effectively captured by Q_{tn}, which is based on the iterative, non-linear stress normalization procedure recently described by Robertson [20].

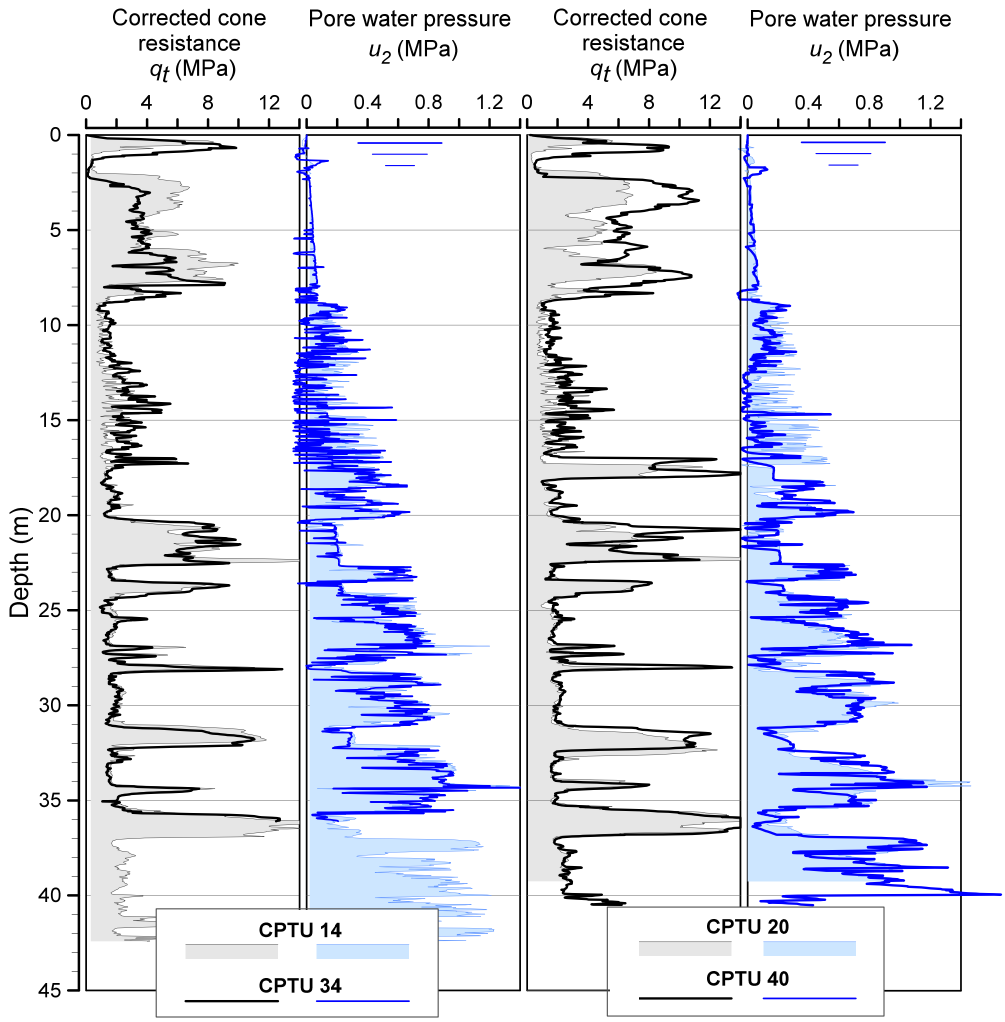

Figure 13. Comparison between 2nd and 4th phase piezocone tests.

Finally, in order to capture the influence of overconsolidation on the experimental data, Figure 13 shows the q_t and u_2 profiles of two representative tests carried out in the *Fourth Phase Testing Campaign*, i.e. CPTU 40 and the central CPTU 34, versus profiles of the 2nd phase adjacent tests CPTU14 and CPTU 20. A general increase in the corrected cone resistance, q_t, in conjunction with a more pronounced dilative behavior with very low or even negative excess pore pressures, can be clearly detected within the silty unit from 8 to 20 m in depth. An unexpected general decrease of q_t, possibly related to the high horizontal spatial variability of the Treporti subsoil, can be observed in the 4th phase central test CPTU34, only within the shallow sandy unit. On the other hand, such anomalous response is not detected in test CPTU40, as shown by the relevant profile, nor in test CPTU 39 (not reported in the figure).

As regards non-standard-rate piezocone tests (CPTU 34_{min} and 34_{max}), it is important to emphasize that this set of tests was obtained only at the end of the research project and that the comparative analysis with data from adjacent "standard" tests was undoubtedly made harder by the overconsolidation induced by former loading history. However, the tests provided clear evidence that a decrease of the penetration rate to 0.15 cm/s resulted in a general increase of the corrected cone resistance q_t and in a decrease of pore water pressures u_2, especially from 9 to 17 m in depth, whereas minor to negligible rate effects were detected in the clay-silt mixtures at 18–20 m and in sands [14,15,17].

Unlike results from CPTU centrifuge physical models in NC kaolin clay or silty clay [25], these "pioneering" experimental data from Venetian natural silts are difficult to interpret in terms of a well-defined consolidation trend line [23], which is crucial for assessing the degree of partial drainage during standard tests. Such preliminary attempt to investigate drainage conditions in Venetian silts would have undoubtedly required additional tests, covering a wider cone velocity interval to better identify a potential consolidation trend line of these sediments. Unfortunately, technical limitations of the equipment in use did not allow to perform tests outside the reported velocity range.

5. Vertical strain analysis

Plots of the local vertical displacements measured by the sliding deformeters SL1, SL2, SL3 and SL4, collected at the end of bank construction (March 2003) and immediately before its gradual removal (April 2007), are shown in Figure 14. To support the analysis of the observed soil deformations, the corrected cone resistance profiles of four piezocone tests located very close to the sliding deformeters have been also included in the figure. Field data show that larger vertical strains have occurred in the shallow silty clay layer and within the silty unit, from 8 m to 20 m depth. On the other hand, the contribution to the total settlement of the soil layers below 25–30 m from the ground surface appears negligible, also in relation to the rapid reduction of the induced stress increment with depth.

As expected, the largest vertical displacements were measured by the sliding deformeter SL3, located very close to the centerline. Here, the integral displacement at the end of the bank construction turned out to be 389 mm, being 250 mm the amount of vertical displacement occurred in the only silty unit, from 8 m to 20 m depth. Four years later, just before the gradual removal of the bank, a total settlement of approximately 506 mm was measured, basically resulting from the major contribution of the silty unit ($\approx$ 311 mm) and, secondarily, from the shallow clayey layer ($\approx$ 69 mm) and the sands (42 mm). Furthermore, the comparative analysis of the sliding deformeter profiles shows that the settlement beneath the bank did not develop axial-symmetrically about the centerline, mainly due to the uneven amount of deformation of the silty unit caused by the occasional presence of sand intercalations.

It is worth observing that the maximum horizontal displacements measured by the inclinometers located just outside the bank, close to SL1, SL2 and SL4, turned out to be one order of magnitude smaller than the total vertical displacement recorded by the adjacent sliding deformeters, thus revealing that the deformation process beneath the test bank prevalently developed in the vertical direction. Moreover, according to the piezometer readings, it is reasonable to assume that primary compression took place almost entirely during the loading bank construction and that 100% consolidation was therefore attained shortly after the end of March 2003. Hence, deformations occurred after the bank completion are most likely to be ascribed to secondary compression.

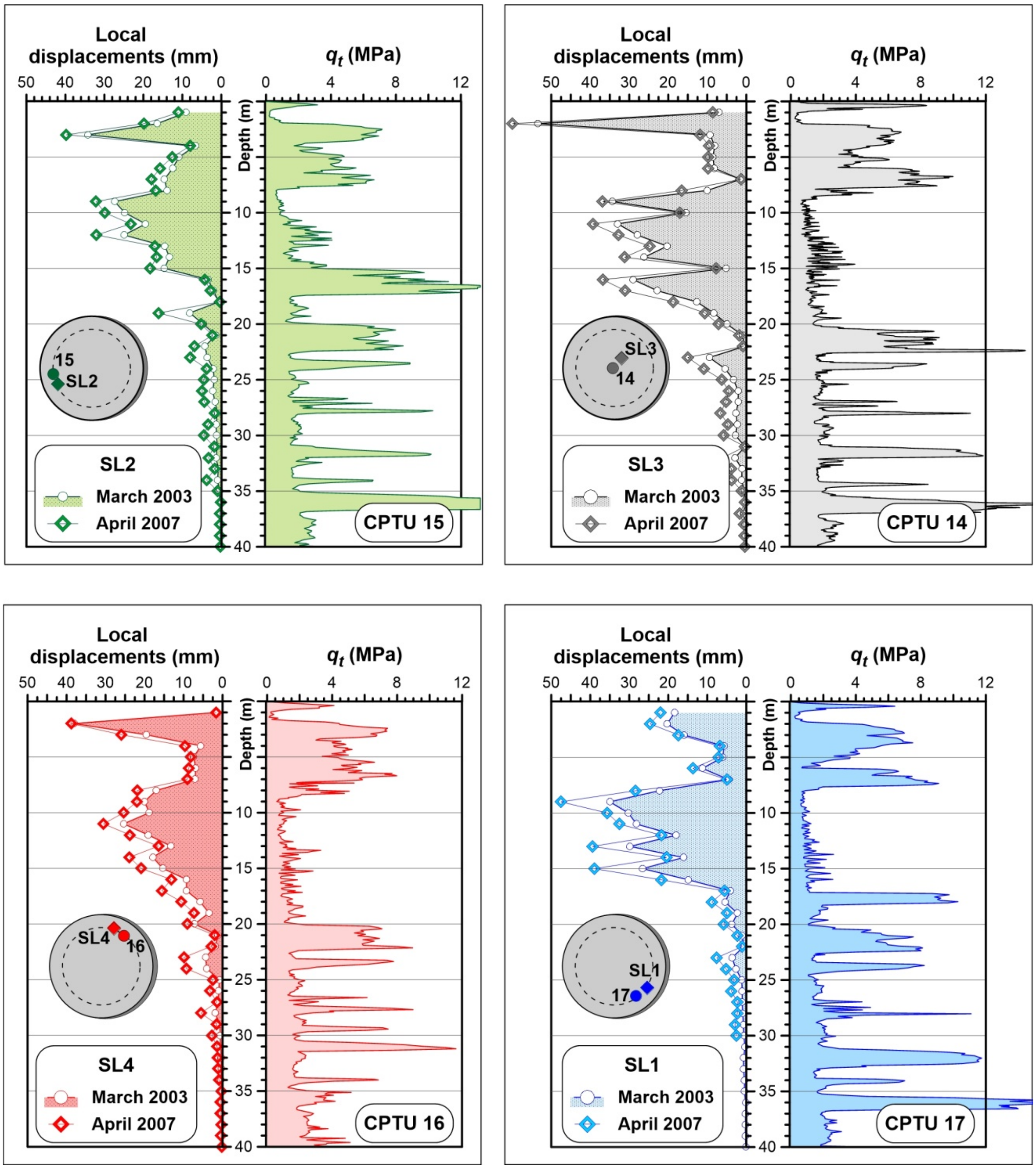

Figure 14. Local vertical displacements recorded at different times by the sliding deformeters SL1, SL2, SL3, SL4 and cone resistance profiles of adjacent piezocone tests.

An effective description of the subsoil deformation process during both loading and stationary stages is provided by plotting the differential displacements at 1 m interval (ε_v) versus the logarithm of time (Log*t*), as shown in Figure 15(a) for the strain measurements along the centerline (SL3) between 6 and 23 m depth. Both sandy and silty layer responses are thus depicted in the plot. The resulting curves, all having the same shape, basically recall the plots of the deformation dial readings versus the logarithm of time, for a given load increment, in an oedometer test. The latter part of each curve, referring to deformations occurred after the bank construction, is usually found to be approximately linear. On the basis of a simple analysis of this semi-logarithmic plot, the time required for completion of the primary compression settlements (418 mm) can be identified at t_{100} = 218 days since the start of the loading-bank construction. Similar trends of the subsoil deformations were obtained also from measurements by the out-of-centerline sliding deformeters SL1, SL2 and SL4.

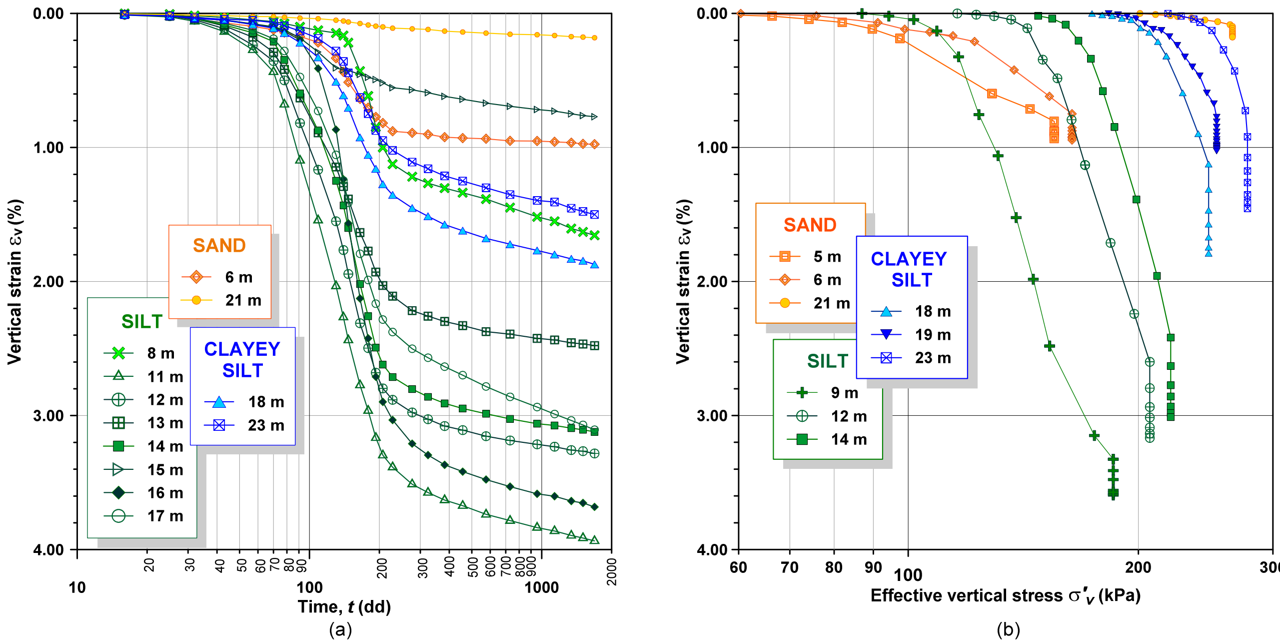

Figure 15. (a) Field axial strains beneath the center of the loaded area, from the beginning of the bank construction until its removal; (b) Field axial strains versus stress beneath the center of the loaded area.

The subsoil deformation measurements were also analyzed as function of the effective stress. A selection of the ε_v-Logσ_v' curves obtained from SL3 has been reported in Figure 15(b). It is worth observing that these curves typically exhibit a sharp change in the slope after a few stress increments. This transition may be interpreted as a yielding point, which can in turn be assumed as an approximation of the preconsolidation pressure σ'_p [8]. The estimates of σ'_p confirmed that Venetian sediments are generally normally consolidated or slightly overconsolidated, with *OCR* values in the range 1.1–2.

As a result of such detailed dataset of subsoil deformations, the on-site compressibility of Venetian sediments, in both primary and secondary compression, can be easily determined. Attention has been paid in particular to the interpretation of vertical strain data provided by the sliding deformeter located beneath the center of the loading bank (SL3), since in this case the assumption of 1D conditions can be reasonably accepted.

Compressibility in primary compression was first expressed in terms of the drained constrained modulus $M = \Delta\sigma'_v/\Delta\varepsilon_v$, taking the differential displacement measured at t_{100}. Similarly to oedometer tests, a compression ratio $C_{c\varepsilon}$ was also obtained from field data plotted on the semi-log plane ε_v – Logσ_v', by calculating the slope of the straight line describing soil compressibility in the normally-consolidated domain.

The secondary compression coefficient $C_{\alpha\varepsilon}$ was determined from the slope of the approximately straight line portion of the ε_v - Log$t$ curves depicted in Figure 15(a). In this way, sets of M, $C_{c\varepsilon}$ and $C_{\alpha\varepsilon}$ were obtained at 1 m intervals, from 5 to 25 m in depth. It is worth noting here that, based on the straight to concave-upward shape of the ε_v-Log$t$ curves, $C_{\alpha\varepsilon}$ seems to slightly change with time, thus confirming the non-linear response reported by Leroueil et al. [26] for normally consolidated specimens subjected to long-term creep tests. Accordingly, $C_{\alpha\varepsilon}$ values were eventually computed from field measurements in the early stages of the secondary compression process.

6. CPTU-based interpretation of primary and secondary compression

6.1. Primary compression

The evaluation of soil compressibility from CPT/CPTU measurements is traditionally expressed in terms of a linear relationship between the cone resistance q_t and the constrained modulus M. The correlations proposed in the geotechnical literature by various authors [27–30] have been typically based on a clear distinction between fine-grained and coarse-grained soils, thus between drained and undrained conditions. Only in recent years, attempts have been made to develop unified interpretation approaches for the estimate of M, irrespective of the different grain-size distribution of soils, i.e. from clays to silts and sand mixtures [20]. At the same time, there is a substantial lack of CPTU-based correlations for the estimate of the dimensionless coefficient $C_{c\varepsilon}$.

Plots (a) and (b) of Figure 16 show a comparison between subsoil deformations and computed settlements beneath the center of the loading bank, due to primary compression. The analysis was limited to the upper 20 m, where larger vertical displacements occurred. Assuming 1D-conditions, settlements were calculated via the following formula:

$$\Delta H = \sum_n \frac{(\Delta\sigma'_v)_n}{M_n} \cdot \Delta z_n \tag{1}$$

where n is the number of homogeneous layers considered in the calculation, $(\Delta\sigma_v')_n$ is the computed effective stress increment at the n^{th} layer, according to standard elastic stress distribution theory, Δz_n is the thickness of each layer, typically equal to 1 m, whereas M_n is the constrained modulus for the n^{th} soil layer, as estimated from CPTU. A number of well-known CPTU-based correlations developed by various Authors (Lunne and Christophersen [28], Senneset et al. [29], Kulhawy and Mayne [30]), each devised for a specific soil class, were applied to the central test CPTU 14, the idea being to assess the predictive capability of these alternative approaches with respect to the intermediate and highly heterogeneous sediments of the Venetian Lagoon. Being based on the idealized assumptions of drained or undrained testing conditions, the existing approaches are likely to be unsuitable for interpretation of CPT/CPTU in intermediate soils, where partial drainage effects occur.

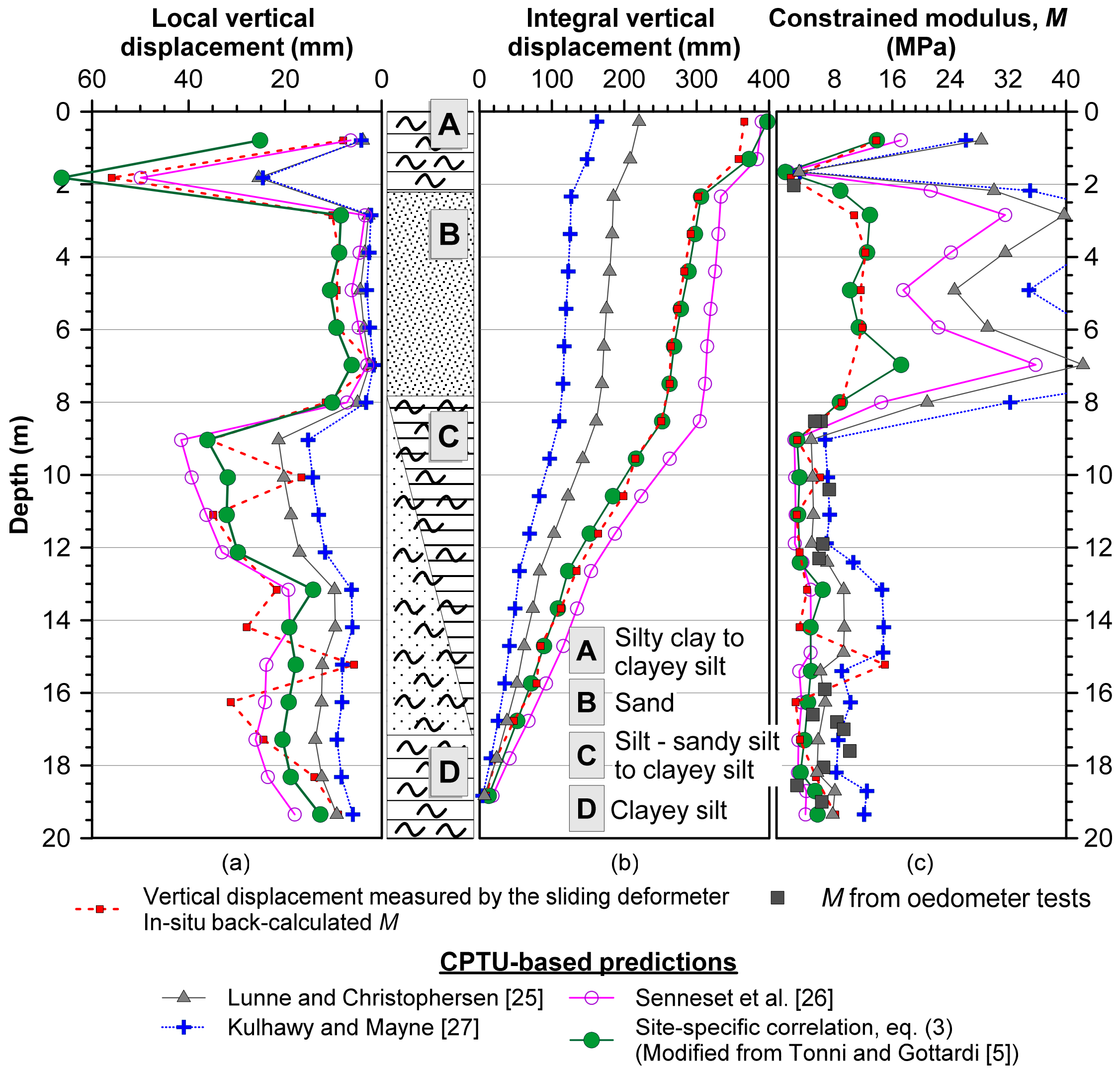

Figure 16. (a)-(b) Comparison between measured and computed displacements of the subsoil from 0 to 20 m in depth, beneath the center of the loading bank; (c) Comparison between in-situ back-calculated values of the constrained modulus, oedometer test results and CPTU-based estimates of M.

The best-fit of the measured subsoil deformations was eventually obtained using the Senneset et al. [29] correlation, originally calibrated on CPTU in silty soils of the North Sea. The method is given by:

$$M = M_0 \cdot \sqrt{\frac{\sigma'_{v0}+\Delta\sigma_v/2}{\sigma'_{v0}}} = 2q_t \cdot \sqrt{\frac{\sigma'_{v0}+\Delta\sigma_v/2}{\sigma'_{v0}}} \text{ , for } q_t < 2.5 \text{ MPa} \tag{2a}$$

$$M = M_0 \cdot \sqrt{\frac{\sigma'_{v0}+\Delta\sigma_v/2}{\sigma'_{v0}}} = (4q_t - 5) \cdot \sqrt{\frac{\sigma'_{v0}+\Delta\sigma_v/2}{\sigma'_{v0}}} \text{ , for } 2.5 < q_t < 5 \text{ MPa} \tag{2b}$$

being $q_t < 5$ MPa the most common upper limit of the corrected cone resistance in silty soils. In this study, the settlement profiles plotted in Figure 16 (local and integral) were obtained from the application of Eqs 2a,b to all data, including the medium-fine sands from 2.5 to 7.5 m. In this layer, computed settlements are generally smaller than measurements provided by the sliding deformeter, whilst agreement with actual vertical strains in clayey silts or silty clays is significantly greater. By contrast, the correlation of Lunne and Christophersen [28] for sands and that of Kulhawy and Mayne [30] for fine-grained soils were both found to considerably overestimate the constrained modulus, thus resulting in underestimates of the vertical displacements.

It is worth observing that the more recent approach developed by Robertson [20] in the context of a SBT_n/I_{cn}-based framework, provided estimates of M close to those predicted by the correlation of Lunne and Christophersen, irrespective of the soil class. However, following Robertson, in this study attempts were made to fit the back-calculated values of M within a unified formulation, in order to overcome difficulties caused by the typical, already mentioned, features of Venetian subsoil.

By assuming a simple relationship of the form:

$$M = \alpha \cdot I_{cn} \cdot (q_t - \sigma_{v0}) \tag{3}$$

the best-fit was obtained for α = 1.22, with a coefficient of determination R^2 = 0.93. The above correlation is a revised version of the relationship originally calibrated by Tonni and Gottardi [5] on the same data, though using the Soil Behaviour Type index I_c, based on a linear stress-normalization of q_t, in place of the latest I_{cn} [20]. In this way, indeed, a slightly better fit to the back-calculated M is obtained. As shown in Figure 16, the relevant profiles (green lines) of the predicted local and integral vertical displacements turn out to be the closest to field measurements (red colored profiles).

The CPTU-based estimates of M, provided by the different approaches mentioned above, are reported in Figure 16(c) and compared with the values calculated from subsoil deformations. The plot also includes a number of experimental points obtained from interpretation of oedometer tests on undisturbed samples of silts and silty clay/clayey silts, taking into account the relevant stress interval of the compression curve. It can be noticed that the oedometer-based values of M turn out to be in general fairly close to field values, especially in clayey silts.

Possible correlations between the primary compression coefficient $C_{c\varepsilon}$ and piezocone measurements were also investigated. However, regression analyses based on the sole normalized cone resistance, expressed either as q_t/p_a or Q_{tn}, provided moderate values of the coefficient of determination R^2 (see Figure 17). Only a very slight improvement was attained when multiple regression analyses were considered, including I_{cn} or a factor accounting for the excess-pore pressure Δu_2 (e.g. $\Delta u_2/\sigma'_{v0}$ or B_q) as additional independent variable. After exploring various combinations of the different variables, the following correlations provided the best fit of field $C_{c\varepsilon}$ calculated along the centerline:

$$C_{c\varepsilon} = 1.32 \cdot (q_t/p_a)^{-1.20} \cdot (I_{cn})^{1.081} \ , R^2 = 0.63 \tag{4}$$

$$C_{c\varepsilon} = 12.30 \cdot (q_t/p_a)^{-1.53} \cdot (1 + \Delta u_2/\sigma'_{v0})^{-0.33} \ , R^2 = 0.63 \tag{5}$$

with identical coefficients R^2. It is worth mentioning that the use of the stress-normalized cone resistance Q_{tn} resulted in slightly less strong correlations. On the other hand, no evidence of improvement was observed when using the net cone resistance ($q_t - \sigma_{v0}$) instead of q_t [31]. In the light of such results, correlations between cone resistance and M should be preferred.

The analysis proposed herein has clearly shown that interpretation of Venetian soil compressibility within a CPTU-based framework is rather complex, due to the interaction of multiple factors such as the predominantly silty nature of the sediments, intrinsic heterogeneity, dense assortment of coarse to fine grained soils. Although the approaches described by Eqs 2a,b and 3 lead to a better description of the observed subsoil response, it is worth emphasizing that none of them accounts for the complex mechanics of cone penetration in such intermediate sediments and for the actual degree of drainage during the test. At this stage, thus, such formulations should be considered as a preliminary tool for the estimate of the constrained modulus of Venetian soils (or other similar sediments in analogous depositional environments).

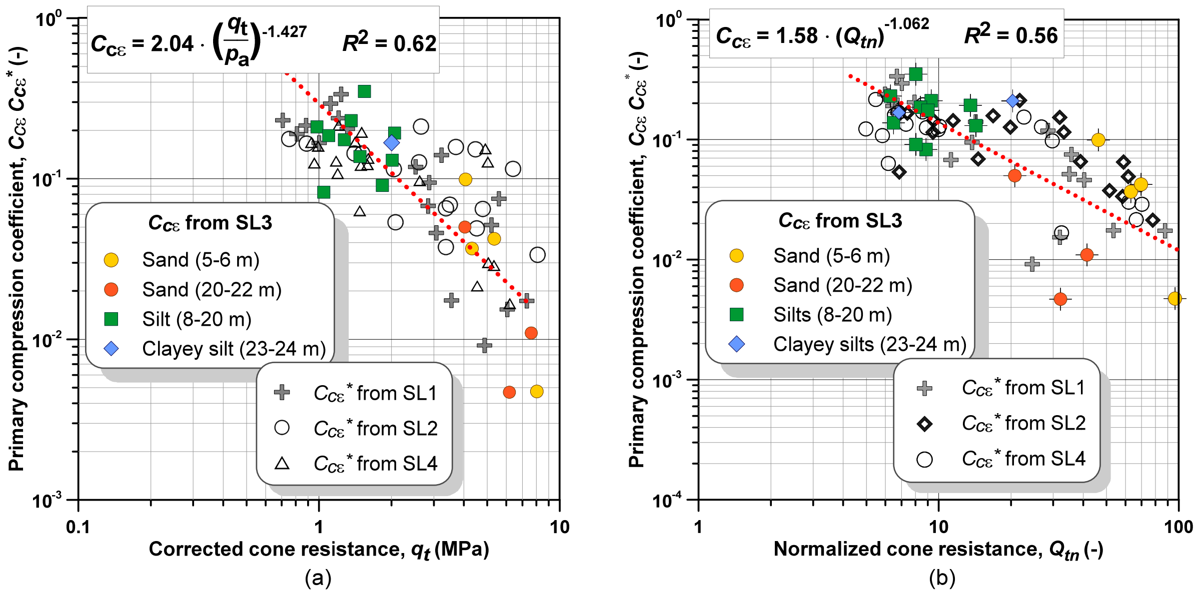

Figure 17. In situ $C_{c\varepsilon}$ as function of the corrected cone resistance q_t (a) and of the normalized cone resistance Q_{tn} (b). $C_{c\varepsilon}^*$ indicates the values obtained from the out-of-centerline sliding deformeters.

6.2. Secondary compression

Although creep phenomena are more significant in clays, it is now widely accepted that sands show considerable amounts of time-dependent deformations as well. Augustesen et al. [32] observed that the time-dependent behavior of granular materials can essentially assume two alternative features, in relation to the confining stress level: at low confining stresses, the deformations are caused by rearrangement with time due to sliding and rolling among sand particles, whilst at high confining pressures the deformations are associated to continuous crushing and deformation of grains.

The subsoil deformations measured at the Treporti Test Site showed that secondary compression phenomena are not negligible in both Venetian silts and sands, hence a proper evaluation of the relevant parameters is of crucial importance for settlement predictions. The secondary compression coefficient in silts, as back-calculated from the sliding deformeter SL3, was found to generally vary between 0.0026 and 0.0054, with a mean value of approximately 0.0046, whereas representative values of $C_{\alpha\varepsilon}$ in sands turned out to fall in the range 0.00058–0.00095. Similar values were also obtained from the slope of the ε_v-Log$t$ curves provided by the out-of-centerline sliding deformeters SL1, SL2 and SL4, herein referred to as $C_{\alpha\varepsilon}^*$.

Compared with oedometer test results on undisturbed samples, the in situ secondary compression coefficient $C_{\alpha\varepsilon}$ was found to assume higher values, thus confirming the observations reported by Leroueil [33] with respect to a number of very well documented embankments on clays.

Differences between laboratory test results and field evidence are generally recognized to be mainly due to strain rate effects, rather than sampling disturbance: laboratory tests provide indeed compression curves corresponding to strain rates larger than 3×10^{-8} s^{-1}, whereas strain rates under embankments are generally lower than 10^{-9} s^{-1}. In the case of the Treporti test bank, the calculated strain rates turned out to vary in the range 10^{-11} s^{-1}–10^{-9} s^{-1}.

The field-based values of $C_{\alpha\varepsilon}$ and $C_{\alpha\varepsilon}^{*}$ are plotted in Figure 18(a) as function of the corresponding values of the field primary compression coefficient, $C_{c\varepsilon}$ and $C_{c\varepsilon}^{*}$, the latter obtained from the primary compression deformations measured by SL1, SL2, SL4. In this way, the relationship $C_{\alpha\varepsilon}/C_{c\varepsilon}$ [34] can be analyzed. The values of $C_{\alpha\varepsilon}/C_{c\varepsilon}$ turn out to be in the interval 0.02–0.04, with the only exception of the coarse sediments from 20 to 22 m depth, which significantly exceed the upper boundary value. Furthermore, the ratio $C_{\alpha\varepsilon}/C_{c\varepsilon}$ seems to be independent of the soil type.

It is worth observing that also the laboratory $C_{\alpha\varepsilon}/C_{c\varepsilon}$ values generally fall in the range 0.02–0.04. As shown in Figure 18(b), typical values for sands ($\cong$ 0.031 on average) turn out to be close to the upper limit, whilst intermediate values of $C_{\alpha\varepsilon}/C_{c\varepsilon}$ ($\cong$ 0.0281) have been obtained for silts. The range for clays is somewhat lower ($\cong$ 0.0267), although no significant differences can be appreciated between the three Venetian soil classes.

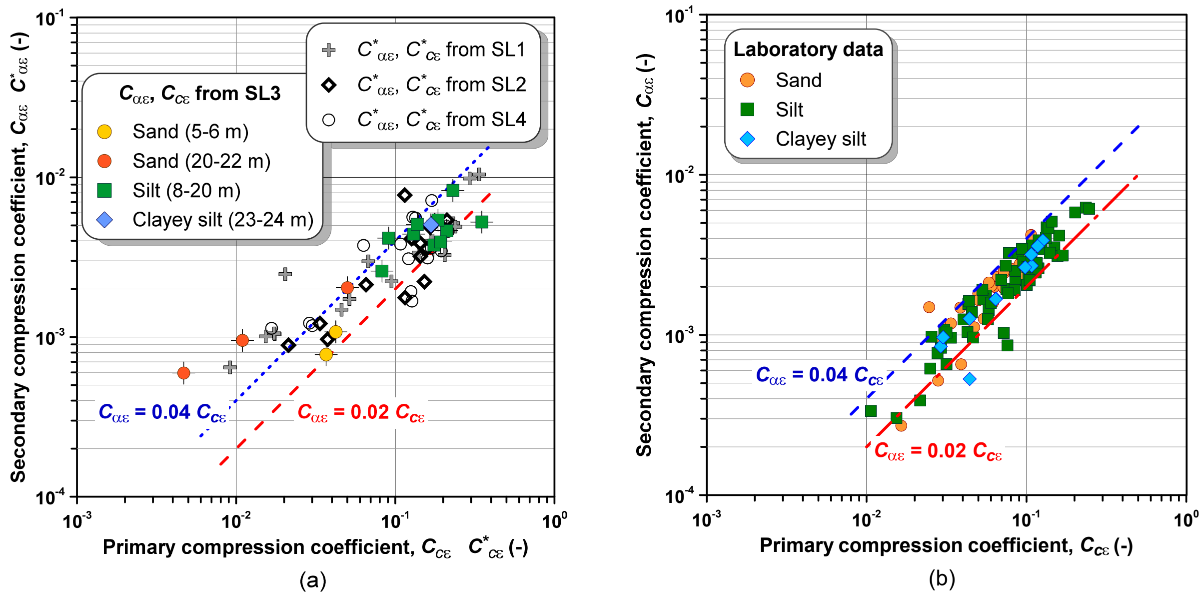

Figure 18. Secondary versus primary compression coefficient: (a) from field data, (b) from oedometer tests.

Despite the application of cone penetration to the estimate of soil compressibility has traditionally mainly dealt with parameters governing primary consolidation, it is worth observing that the time-dependent response of granular soils, here described by $C_{\alpha\varepsilon}$, and the cone penetration resistance q_t are both mainly controlled by inter-particle friction, hence it makes sense to investigate a potential correlation among them.

The interpretation of the back-calculated values of the secondary compression coefficient in terms of the corrected cone resistance allowed identifying a trend between the field $C_{\alpha\varepsilon}$ and q_t, as clearly shown in Figure 19a. The graph includes about 70 paired observations of $C_{\alpha\varepsilon}$ and q_t plotted on a log-log plane, obtained from adjacent pairs of sliding deformeters and piezocone tests (i.e. S3-CPTU14, S1-CPTU15, S2-CPTU16 and S4-CPTU17). The regression analysis on the only experimental data recorded along the centerline (colored symbols in Figure 19a) resulted in the following correlation [35], based on a power-function relationship:

$$C_{\alpha\varepsilon} = k \cdot (q_t/p_a)^h = 0.066 \cdot (q_t/p_a)^{-1.03} \;, R^2 = 0.74 \tag{6}$$

where p_a is the atmospheric pressure, expressed in the same units as q_t. At the same time, the best-fit regression curve on the whole set of data points gave $k = 0.033$ and $h = -0.80$, with a coefficient of determination $R^2 = 0.64$.

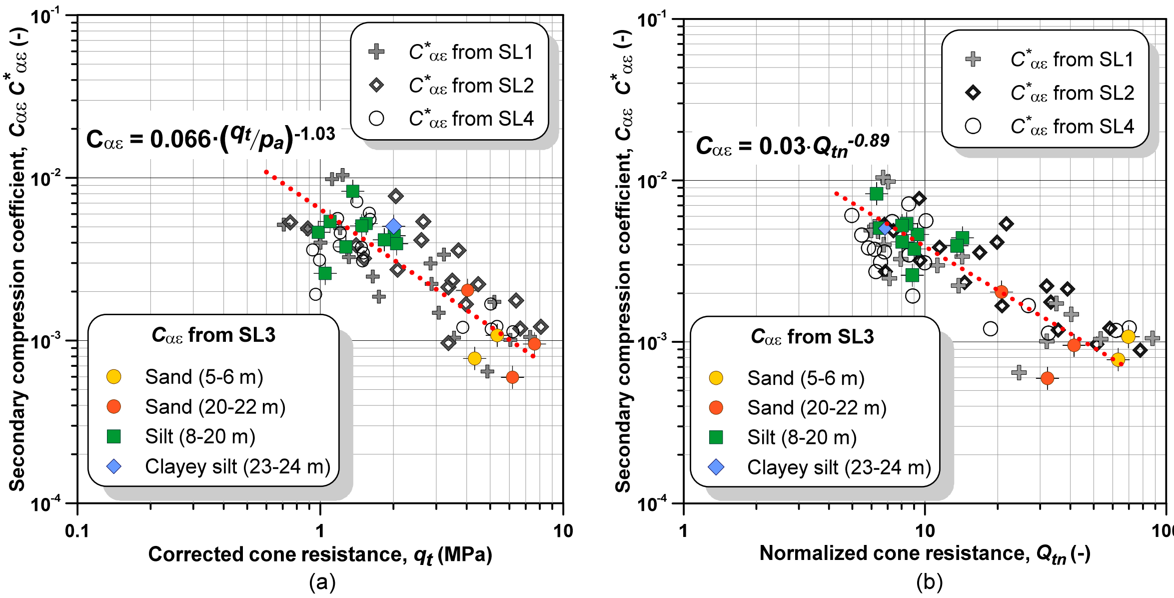

Figure 19. In situ $C_{\alpha\varepsilon}$ as function of the corrected cone resistance q_t (a) and of the normalized cone resistance Q_{tn} (b). $C_{\alpha\varepsilon}^*$ indicates the values obtained from the out-of-centerline sliding deformeters.

Correlations expressed in terms of the dimensionless normalized cone resistance Q_{tn} [20] were found to give higher coefficients of determination R^2. Assuming a log-log relationship, the trend shown by the centerline experimental points reported in Figure 19(b) can indeed be expressed through the equation:

$$C_{\alpha\varepsilon} = 0.03 \cdot (Q_{tn})^{-0.89} \;, R^2 = 0.83 \tag{7}$$

A slightly better fit of the experimental data was achieved by including the excess pore pressure ratio $\Delta u_2/\sigma'_{v0}$ as an additional factor into the Q_{tn}-$C_{\alpha\varepsilon}$ relationship, in an attempt to account for the different pore pressure response of the heterogeneous Venetian sediments as a consequence of the drainage conditions around the advancing cone [35]. The resulting correlations is thus given by:

$$C_{\alpha\varepsilon} = 0.077 \cdot (Q_{tn})^{-1.14} \cdot (1 + \Delta u_2/\sigma'_{v0})^{-0.74} \;, R^2 = 0.86 \tag{8}$$

The application of Eq 8 is restricted to cases in which $\Delta u_2/\sigma'_{v0} > -1$, thus potentially excluding highly dilatant soils. However this is not the case of Venetian deposits, since these sediments generally show a contractive or slightly dilatant behavior. A further possible drawback of Eq 8 is represented by the oscillations observed in the pore pressure profile of the Venetian subsoil, owing to its intrinsic heterogeneity, which might cause in turn significant uncertainties in the selection of a representative value of the excess pore pressure ratio at each depth of calculation. On the other hand, it is worth mentioning that a preliminary validation of the effectiveness of Eqs 7 and 8 was recently carried out with respect to the evaluation of long-term settlements of two existing breakwaters built along the Venetian coastline [7], on the typical silt mixtures forming the lagoon subsoil. In that case, both approaches resulted in similar estimates of $C_{\alpha\varepsilon}$ and, consequently, of the computed settlements, which happened to be in very good agreement with measured vertical displacements.

The predicted $C_{\alpha\varepsilon}$ values deduced from Eqs 7 and 8, in conjunction with the associated settlement predictions occurred in the period July 2003–April 2007 beneath the center of the loading bank, are plotted in Figure 20. A general good agreement between field observations and predictions can be found.

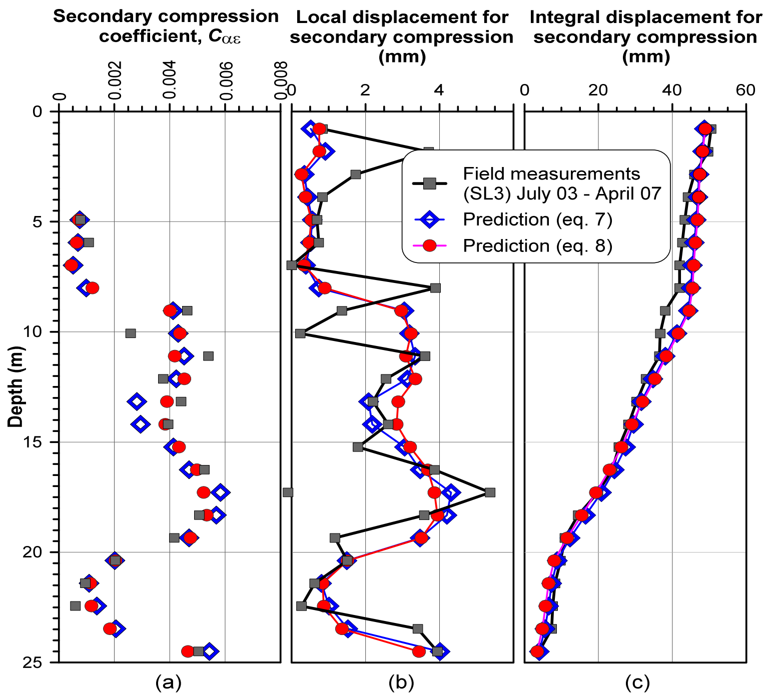

Figure 20. (a) Comparison between field and predicted $C_{\alpha\varepsilon}$, beneath the center of the loading bank, (b) local and (c) integral vertical displacements, from July 2003 to April 2007.

7. Conclusions

The paper has presented a database of research-quality piezocone tests and subsoil deformation measurements assembled at the Treporti Test Site (Venice, Italy), within a long-lasting research project aimed at investigating the stress-strain-time response of the complex assortment of sandy and silty sediments forming the Venetian lagoon subsoil. Attention has been especially focused on the interpretation of the large amount of piezocone data in terms of soil compressibility parameters, in both primary and secondary compression.

As regards deformations in primary compression, the analysis of data has revealed that Venetian sediments do not fit into published frameworks and well-established CPTU-based empirical correlations, typically used for the estimate of the constrained modulus *M* in sands or clays. Despite soil heterogeneity and dense interbedding make sometimes difficult the interpretation of field data, potential advances of the existing approaches and site specific correlations have been proposed, in an attempt to develop a unified formulation equally applicable to all Venetian sediments, irrespective of their grading characteristics.

It is worth observing that difficulties in interpreting in situ tests through standard approaches may be caused by soil consolidation characteristics and partial drainage effects. Indeed, as a consequence of the essentially silty nature and the complex macrofabric of Venetian subsoil, it was proved that different degrees of partial drainage are very likely to occur under a standard rate of penetration. At the same time, although the preliminary evaluation of the actual degree of drainage is recognized to play a crucial role in the interpretation of CPTU data for geotechnical characterization, a few preliminary attempts in that direction have shown that the identification of a simple consolidation trend is difficult to achieve for these natural soils. Hence, the matter is at present far from being satisfactorily solved.

As regards deformations due to creep, the Treporti Test Site experiment has clearly confirmed that secondary settlements of recent soils should not be neglected, even for sands and silts, as typically done in routine calculations. The prediction of such component of the total settlement may prove to be important in some practical applications, and the development of CPTU-based empirical correlations may be very useful for estimating the relevant parameters at sites where undisturbed sampling is very difficult to achieve and geotechnical characterization must essentially rely on in situ testing.

Accordingly, the study has explored the capability of the piezocone test data to predict the one-dimensional secondary compression coefficient of sands and silt mixtures, based on the assumption that frictional response essentially governs both secondary compression of granular soils and cone resistance. Empirical, site-specific correlations for the estimate of $C_{\alpha\varepsilon}$ have been calibrated on the creep deformations measured beneath the experimental loading bank during approximately 4 years. The regression analyses indicate that the predictive capability of correlations is slightly improved when the excess pore pressure measurements are taken as independent variable.

In conclusion, the Treporti Test Site experiment has offered the valuable opportunity to gain a better insight into the mechanical behavior of a class of soils, i.e. silt and sand mixtures, that has been little studied for a long time and that still remains not fully explored and understood. In the light of the main outcomes obtained from this research project, coupled with other significant contributions recently proposed in the literature, it is very likely that the ongoing advances will confirm the leading role of cone penetration testing for geotechnical site characterization also in intermediate sediments.

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Conflict of interest

The authors declare no conflict of interest.

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