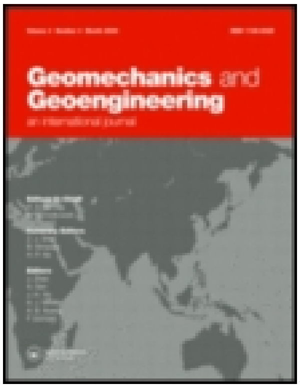

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Evaluation of secondary compression of sands and silts from CPTU

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The estimate of secondary compression behaviour of sandy and silty deposits is not routinely taken into account in the classical settlement calculation, due to its generally limited contribution to the total amount of strains. However, such soils exhibit a non-negligible time-dependent behaviour and the prediction of their long-term response may sometimes be important.

This paper explores the capability of the piezocone test as an effective tool to estimate the one-dimensional secondary compression characteristics of sands and silts. The approach is mainly based on the observation that frictional response essentially governs both secondary compression of granular soils and cone resistance, hence it seems reasonable to establish a valid correlation among them.

The study uses part of the field data assembled over the last years at the Treporti Test Site (Venice, Italy), within an extensive research project aimed at analysing the stress-strain-time response of the predominantly silty sediments forming the Venetian lagoon subsoil.

Empirical, site-specific correlations for estimating the secondary compression coefficient from cone resistance are proposed. The regression analyses indicate that the estimate is slightly improved if the cone resistance-based correlations include a variable accounting for the different pore pressure response associated to the drainage conditions around the advancing cone.

Keywords: piezocone tests (CPTU); silt; sand; time dependence; soil properties; Venetian lagoon

1. Introduction

Over the last 50 years, the historical city of Venice (Italy) has been affected by a significant increase in the frequency of flooding, as a consequence of the eustatic sea level rise coupled with natural and man-induced subsidence phenomena (e.g., Ricceri and Butterfield 1974).

Different engineering solutions, including both nearshore and offshore structures, have been constructed over the years in order to protect human activities of the whole coastal environment as well as the invaluable historical and artistic heritage of the city. In recent years, the MOSE project, consisting in a mobile barriers system designed for the temporary closure of the three lagoon inlets, has been eventually evaluated as the more effective remedy for safeguarding Venice from sea storms, high tides and flooding (Jamiolkowski *et al*. 2009). This innovative defence system, currently under construction, should be fully operational by 2014.

A key issue in the performance prediction of such structures is the estimate of both short-term and especially long-term settlements, hence in the last 30 years extensive geotechnical investigation campaigns have been successively launched over the whole lagoon and the nearshore area, in order to gain a detailed characterisation of the sediments forming the Venetian lagoon basin. From the analysis of the large amount of data collected over more than two decades, it turned out that the upper 100 m of the Venetian lagoon subsoil consist of a chaotic assortment of interbedded normally consolidated or slightly overconsolidated silts, medium-fine silty sands and silty clays (Cola and Simonini 2002).

This paper focuses on the secondary compression behaviour of Venetian silts and sands, as observed from *in situ* settlement records, and provides an attempt to estimate the secondary compression index $C_{\alpha\varepsilon}$ from the interpretation of piezocone (CPTU) measurements. The proposed approach is mainly based on the observation that in these soils frictional response governs both cone resistance and secondary compression, hence this common dependence provides valid reasons for investigating the existence of possible correlations between the two parameters.

The study is based on the large amount of field data collected over the last years at the Treporti Test Site (Venice) within an ambitious research project aimed at thoroughly analysing the stress-strain-time response of the complex and heterogeneous sediments forming the Venetian lagoon subsoil (Simonini 1994).

Field observations have shown that time-dependent phenomena are not negligible in both Venetian silts and sands, hence the proper evaluation of the relevant parameters is of crucial importance for settlement predictions. On the other hand, as a consequence of the predominantly silty nature and high heterogeneity of such sediments, undisturbed sampling is rather difficult, thus

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excluding any possibility of deducing reliable compressibility parameters from laboratory tests. The paper explores the capability of piezocone tests as an alternative way to estimate the one-dimensional creep characteristics of Venetian sediments for reliable predictions of time-dependent settlements.

2. Time effects on soil behaviour

Generally, compression of a saturated soil layer is considered to consist of two successive phases, namely primary and secondary consolidation. During primary consolidation, settlement is controlled by dissipation of excess pore pressures and Darcy's Law, whilst during secondary compression the rate of settlement is controlled by soil viscosity, hence deformation is associated to time effects under approximately constant effective stresses. This latter mechanism is generally known as *creep*.

Over the last 30 years, a large amount of research has been carried out on the one-dimensional compression of clays under constant effective stresses and several rheological models have been suggested to describe soft soils behaviour in one-dimensional conditions. Among them, it is worth mentioning the pioneering work of Zeevaert (1973), recently applied by Manassero and Dominijanni (2010). Most of the studies have focused on cohesive soils, whilst very few contributions on the time-dependent behaviour of sands, especially in confined conditions, are available.

The formulations proposed for clays can be basically ascribed to two different interpretation approaches, the well-known Hypothesis A and Hypothesis B (Ladd *et al*. 1977, Jamiolkowski *et al*. 1985), based on alternative assumptions about the combination of primary and secondary compression during the process of pore pressure dissipation. Despite the large amount of available experimental data and the abundant research contributions on this issue, there are still contradicting opinions on the mechanisms that govern creep during primary consolidation, hence the discussion on the creep hypotheses remains an issue that needs to be solved. A summary of the different contributions proposed in the literature can be found in Mesri (2003) and Leroueil (2006).

When a fully saturated soil sample (typically a clay sample) is suddenly loaded one-dimensionally, the void ratio e decreases with time t, producing the well-known S-shaped consolidation curve in the semi-logarithmic plane e - Log$t$. Such classical way of plotting oedometer data suggests the very common practical working hypothesis of treating secondary consolidation separately from primary compression, although it has been widely accepted that the phenomenon takes place from the beginning of loading (Šuklje 1957, Bjerrum 1967, Leroueil *et al*. 1985). Accordingly, secondary consolidation is typically characterised by the slope of the straight line portion of the consolidation curve, namely the *secondary compression index* $C_{\alpha e}$:

$$C_{\alpha e} = \frac{\Delta e}{\Delta \mathrm{Log}\, t} \tag{1}$$

This coefficient can be either defined with respect to the vertical deformation ε_z, hence it is given by:

$$C_{\alpha\varepsilon} = \frac{\Delta\varepsilon_z}{\Delta \mathrm{Log}\, t} = \frac{C_{\alpha e}}{1+e_0} \tag{2}$$

being e_0 the initial void ratio.

The simplest approach in the framework of the semi-logarithmic law, useful as a first approximation for estimating secondary settlements, is the assumption that $C_{\alpha e}$ (or $C_{\alpha\varepsilon}$) is independent of time and thus constant for a given soil.

However, this does not reflect the real behaviour, since there is experimental evidence that $C_{\alpha e}$ may change with time, both in the laboratory and in the field. A number of investigations have focused on the relationship between secondary compression rates in laboratory tests and the compression index C_c, describing the void ratio decrease during primary consolidation. Research has shown that highly compressible soils in the first compression stage generally exhibit high compressibility in the secondary phase as well. According to Mesri and Godlewski (1977), the ratio $C_{\alpha e}/C_c$ is approximately a constant for a wide variety of natural soils and falls in the range 0.025–0.06. Besides, this ratio was considered to hold at any time, effective stress and void ratio throughout secondary compression. Indeed, it was shown that both $C_{\alpha e}$ and C_c increase as the effective stress σ'_z approaches the preconsolidation stress σ'_p, reach a maximum value in the vicinity of σ'_p, then decrease and finally remain reasonably constant (Augustesen *et al*. 2004).

The unique interrelationship between $C_{\alpha e}$ and C_c, throughout the secondary consolidation stage, provided a simple method for calculating secondary settlement, as shown for example in Mesri (1987) and Mesri *et al*. (1994) among others. Furthermore, the observation that creep behaviour is well-captured by a linear relation in the semi-logarithmic plane e - Log$t$ (or equally ε_z – Log$t$) may be valid for several log cycles of time t, although these findings do not hold true in general. Based on a number of creep oedometer tests, Leroueil *et al*. (1985) reported a general nonlinear strain-time behaviour with respect to a ε_z – Log$t$ diagram and observed a continuously increasing slope of the secondary compression curve when overconsolidated samples are considered. On the other hand, normally-consolidated specimens exhibit a continuously decreasing slope with the logarithm of time. A general nonlinear behaviour in the ε_z – Log$t$ plane was also observed by other authors, among them Leonards and Girault (1961) and Yin (1999).

Finally, it must be emphasised that although creep phenomena are more pronounced in clay, it is now widely accepted that sand shows considerable amounts of time-dependent behaviour as well. Even dense sands may exhibit significant creep settlements (e.g., Burland and Burbidge 1985), which may even double over a period of 30 years (Hight and Leroueil 2002).

As summarised by Augustesen *et al*. (2004), time-dependent behaviour of granular materials can essentially assume two alternative features, in relation to the confining stress level. Indeed, at low confining stresses the deformations are caused to rearrangement over time due to sliding and rolling between sand particles, whilst at high confining pressures the deformations are associated to continuous crushing and deformation of grains. Furthermore, in the low stress regime the strain-time relation seems to be linear when plotted as strain versus logarithm of time. A similar behaviour has been observed in the high stress regime as well (e.g., Leung *et al*. 1996), although further research has shown that a power relation between creep strain and time would be more appropriate to describe the initial stages of creep.

In the following, the parameter $C_{\alpha\varepsilon}$ is used as a reference for describing the one-dimensional creep characteristics of Venetian soils, because the concept is well-known and widely used in geotechnical practice.

3. Basic features of the Venetian lagoon sediments

Due to a rather complex depositional history, the upper 100 m of the Venetian lagoon basin appear as a chaotic system of interbedded sands, silts and silty clays chaotically accumulated during the last glacial Pleistocenic period. Thin layers of peat can be also found, as a consequence of the occasional presence of lacustrine environments. Sediments are generally normally consolidated or slightly overconsolidated due to aging or oxidation.

Despite grain size heterogeneity, research has shown that such sediments have a common mineralogical composition and that their mechanical behaviour, mostly controlled by intergranular friction, can be interpreted within a unified approach, based on a grain size index accounting for the different soil grading characteristics (Cola and Simonini 2002).

As regards mineralogy, Venetian coarse sediments are predominantly composed of carbonates, with quartz and feldspar as other significant components. Furthermore quartz and feldspar grains are generally angular, thus suggesting that sediments experienced a limited transportation process. Minor quantities of mica, approximately in the range 5–10%, are erratically present in the coarse fraction (Sanzeni 2006).

On the other hand, 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) never exceeding 20% in weight. Clay minerals appear generally highly crystallised, thus confirming their detrital origin (Simonini *et al*. 2007).

4. The Treporti Test Site

4.1 *The research project*

The research site of Treporti, located in the North Eastern lagoon (Figure 1), can be undoubtedly considered as truly representative of the typical stratigraphic conditions of the Venetian lagoon basin. Indeed, as discussed later on, the Treporti soil profile shows all the distinctive features of

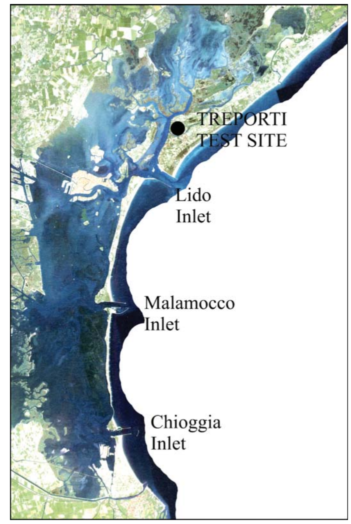

Figure 1. Satellite view of the Venetian lagoon and location of the Treporti Test Site.

the Venetian subsoil, typically characterised by a complex assortment of interbedded normally consolidated or slightly overconsolidated silts, medium-fine silty sands and silty clays.

In this area, a full-scale 6.5 m high, 40 m diameter, vertical-walled cylindrical test bank was progressively built and continuously monitored over approximately 6 years in terms of pore water pressures, induced soil total stresses, horizontal displacements with depth, surface settlements and soil vertical strains. Details on the monitoring instrumentation, including piezometers, inclinometers, GPS, sliding deformeters, can be found in Simonini (2004) and Jamiolkowski *et al*. (2009).

Furthermore, a very accurate geotechnical characterisation of the whole area, based on a large number of piezocone (CPTU) and dilatometer (DMT) tests, boreholes and high-quality laboratory tests, was carried out. In particular, the extensive in situ testing campaign was divided into three main different phases, by first performing a large number of CPTU and DMT tests prior to the loading bank construction, followed by a few tests carried out from the top of the bank, just at the end of its construction, and by some additional tests carried out at the end of its gradual removal (Tonni and Gottardi 2009, 2011).

A scheme of the whole construction history of the test bank (including its removal), together with the associated settlements measured both in the centre and at the edge of the bank by four multiple micrometers, is shown in Figure 2. A plan with the location of the solely CPTUs performed in the first testing phase within the loading bank area is also provided in Figure 2. In particular, tests labelled as CPTU 14, CPTU 15, CPTU 16 and CPTU 17 have been highlighted, since they are relevant to the study described in this paper. Apart from the central CPTU 14 test, these tests are located on an imaginary 30 m diameter circumference, concentric with respect to the loading bank area, as clearly shown in the figure.

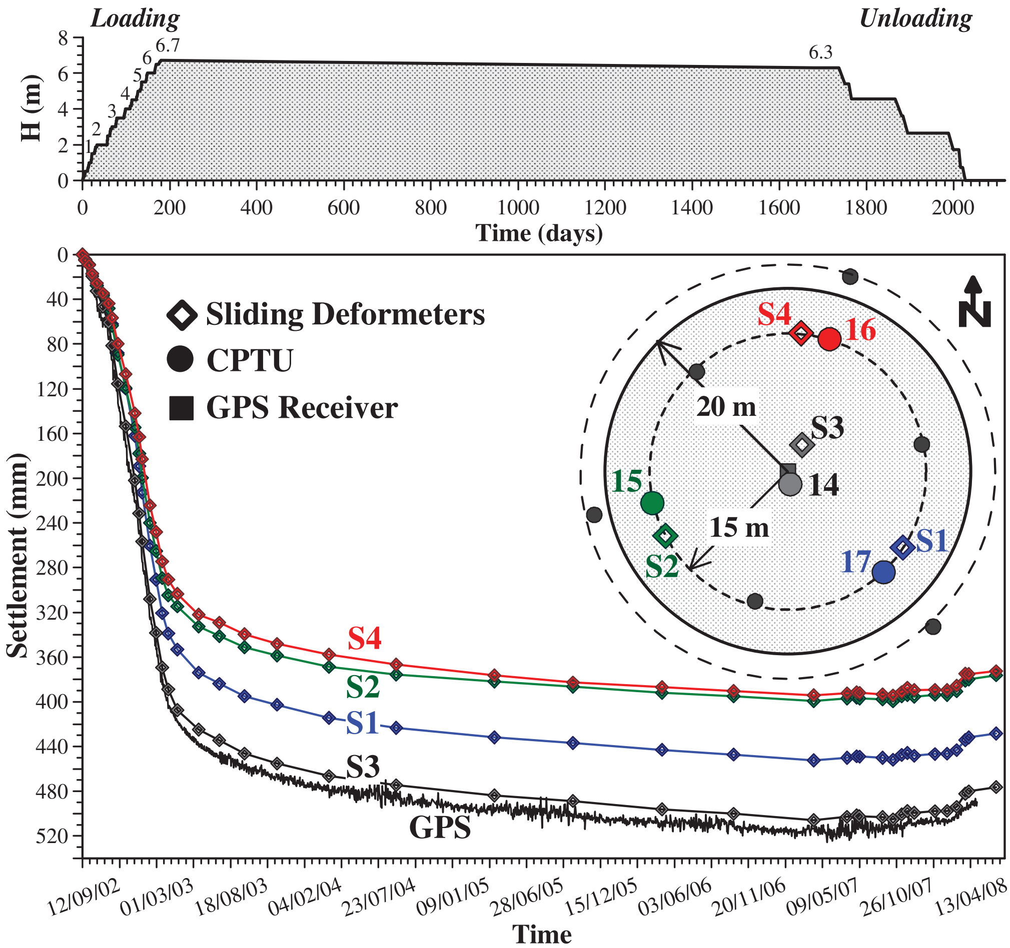

Figure 2. Construction history of the test bank and associated settlements measured beneath the loaded area, together with the location of sliding deformeters and piezocone tests.

4.2 *The piezocone data*

Typical CPTU profiles of Venetian subsoil are reported in Figure 3, showing the corrected cone resistance q_t, the sleeve friction f_s and the pore pressure u of CPTU 15, in conjunction with the grading characteristics obtained from the soil samples of a nearby borehole.

Such profiles fully confirm the highly stratified nature of the Venetian lagoon subsoil. In particular, relevant oscillations in the pore pressure profile can be observed, thus suggesting large differences in drainage conditions due to the complex alternation of different grain sized sediments.

Despite soil heterogeneity, the analysis of all CPTU measurements provides immediate evidence of a well-defined top layer of fine clean sands, 6–7 m thick, followed by a silty unit from 8 m to 20 m in depth. However, the thickness of this unit is not constant beneath the loaded area, being often interbedded with a clean sand layer that progressively reduces its thickness moving eastwards. This is evident from comparison of the q_t profiles depicted in Figure 4. Indeed, it can be easily observed that the sandy lens, particularly evident in the CPTU 15 profile, totally disappears in tests CPTU 14 and CPTU 16.

Another significant sandy layer is located from 20 m to 23 m depth, while a complex alternation of silty sand, sandy silt and clayey silt, with occasional presence of peat, can be identified down to a depth of 40 m.

Finally, as a key feature of the pore pressure response during cone penetration in these sediments, it can be observed that the profile of u rarely follows up the hydrostatic level, falls at times below it but never develops high excess pore pressures Δu, typical of pure clays. Indeed, in such intermediate soils, CPTUs are often performed under conditions of partial consolidation, i.e. where some (but not full) dissipation of excess pore water pressure occurs locally around the advancing cone (Schneider *et al*. 2008). In these cases, there is a significant uncertainty in the assessment of soil properties, hence the preliminary evaluation of such condition should be taken in careful consideration for a proper interpretation of piezocone data. A study recently proposed by Tonni and Gottardi (2010), based on piezocone tests performed at non-standard penetration rates, confirmed the occurrence of this phenomenon in Venetian silty sediments, although only qualitative information on the degree of drainage during penetration could be achieved from the available data.

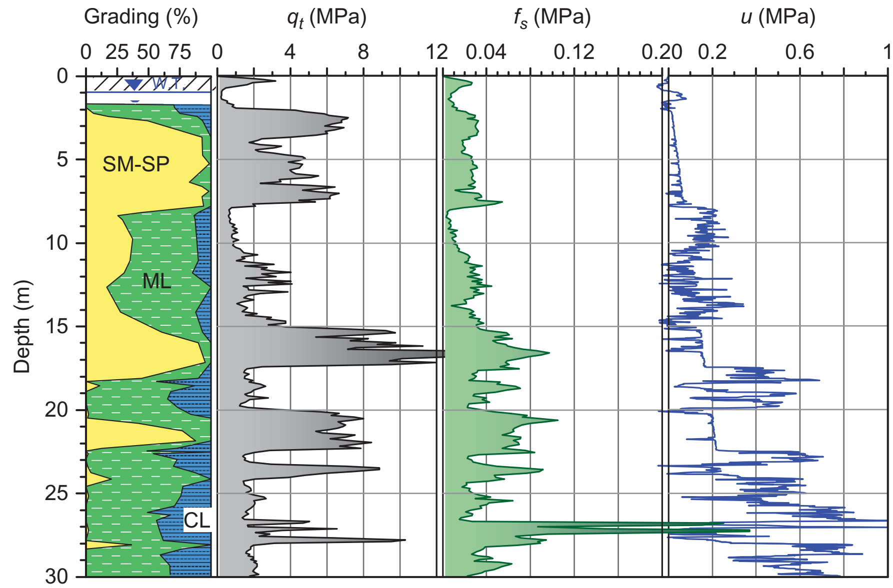

Figure 3. CPTU 15 log profiles.

Figure 5 shows the application to Treporti piezocone data (CPTU 14) of a simple procedure (Hight *et al*. 1994; Schnaid *et al*. 2004) for a qualitative assessment of drainage conditions during a standard rate of penetration. The approach is based on the well-known normalised cone resistance Q_t and pore pressure parameter B_q (Wroth 1984), given by:

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \tag{3}$$

$$B_q = \frac{u - u_0}{q_t - \sigma_{v0}} \tag{4}$$

where u_0 is the equilibrium pore pressure, whilst σ_{v0} and σ'_{v0} are the total and effective vertical stress respectively.

As evident from the figure, most of the points from 7.7 m to 20 m depth, referable to the silty unit, fall in the range where $B_q < 0.3$, corresponding to the domain in which partial drainage is prevailing when testing normally consolidated soils.

5. Vertical deformations beneath the test bank

A crucial role in the analysis of the compressibility characteristics of Treporti subsoil was played by four sliding deformeters installed beneath the bank, one in the centre and three symmetrically located along a 30 m diameter concentric circumference. As shown in Figure 2, they were located very close to the piezocone verticals CPTU 14, CPTU 15, CPTU 16, CPTU 17, the distance between each pair of sliding deformeter and CPTU being not more than 4 m.

The sliding deformeter is a device for the measurement of soil axial strains along a vertical or inclined direction. Such a rather sophisticated piece of equipment was able to monitor, every meter and down to 57 m in depth, distance variations with a probe precision of ±30 μm (Kovary and Amstad 1982).

Plots of the local vertical displacements measured by the sliding deformeters S1, S2, S3 and S4 are shown in Figure 4. Curves have been obtained by plotting the subsoil displacements measured just after the end of the bank construction (March 2003) as well as just before starting the gradual removal of the bank itself (April 2007). Futhermore, the corrected cone resistance profiles of the adjacent piezocone tests have been included in the graphs.

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, contribution of soil layers deeper than 25–30 m 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 S3, located very close to 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 prior to the gradual removal of the bank, the sliding deformeter measurements provided a total displacement of approximately 506 mm, mainly due to the silty unit (≈ 311 mm) and, to a lesser extent, to both the shallow clayey layer (≈ 69 mm) and upper sandy layer (42 mm). Furthermore, as a consequence of the horizontal spatial variability of the Treporti subsoil, the deformation process didn't develop symmetrically with respect to the bank centreline. Indeed, a comparison of the sliding deformeter profiles depicted in Figure 4 makes it clear that sandy laminations, present at times within the silty unit, may have had some effect on the amount of deformation measured from 8 m to 20 m depth.

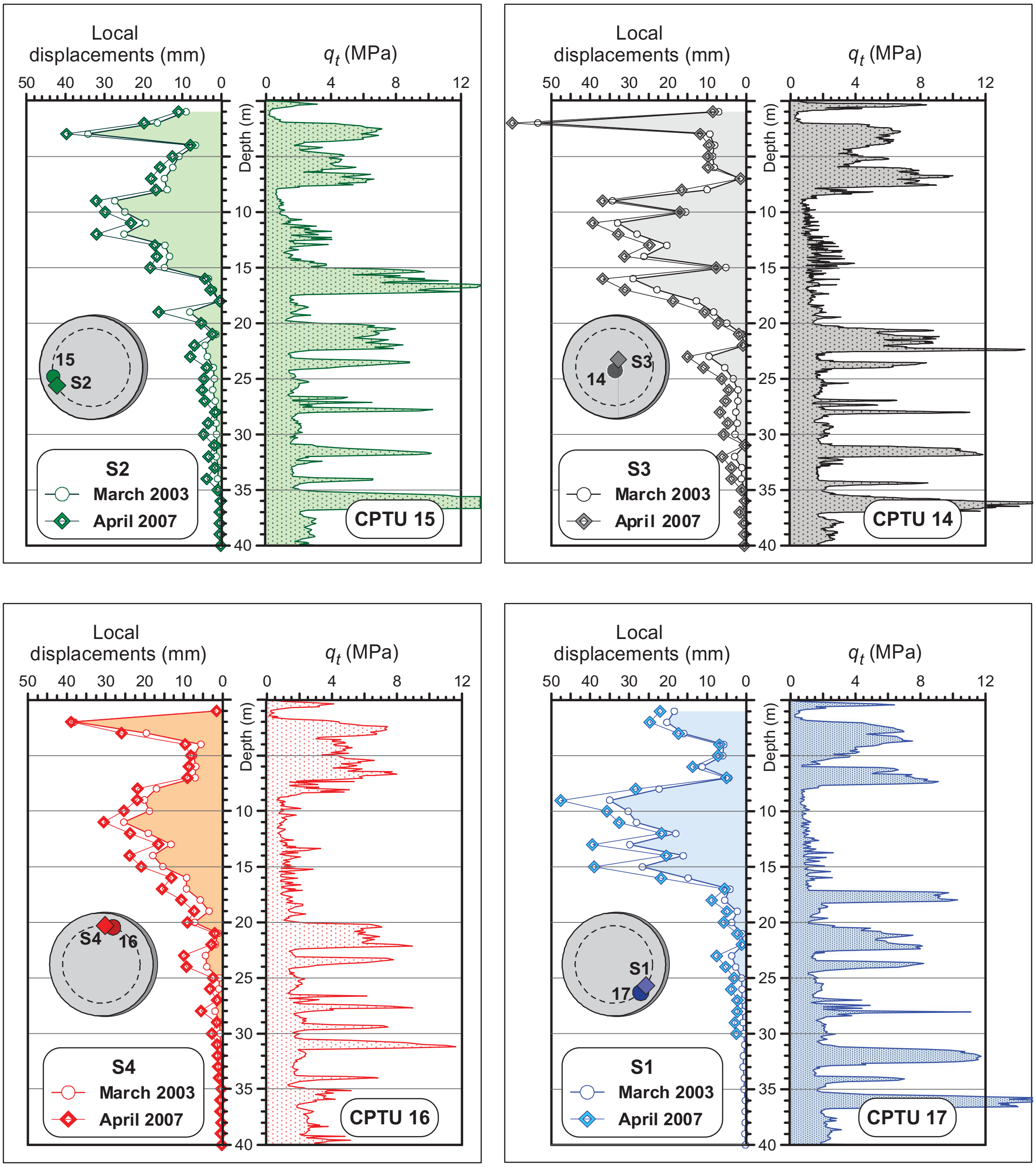

Figure 4. Local vertical displacements recorded at different times by the sliding deformeters S1, S2, S3 and S4, together with cone resistance profiles obtained from adjacent piezocone tests.

On the other hand, it worth remarking that the maximum horizontal displacements provided by the inclinometers located just outside the bank, close to S1, S2 and S4, turned out to be one order of magnitude smaller than the total vertical displacement recorded by the adjacent sliding deformeters, thus confirming that the deformation process beneath the test bank prevalently developed in the vertical direction.

Finally, the pore pressure monitoring revealed that shortly after the bank construction was completed, the piezometers measured pore water pressures essentially the same as those detected before construction began. These findings confirm that the rate of consolidation was quite high and that it can be reasonably assumed that primary consolidation was completed throughout the entire depth by the end of

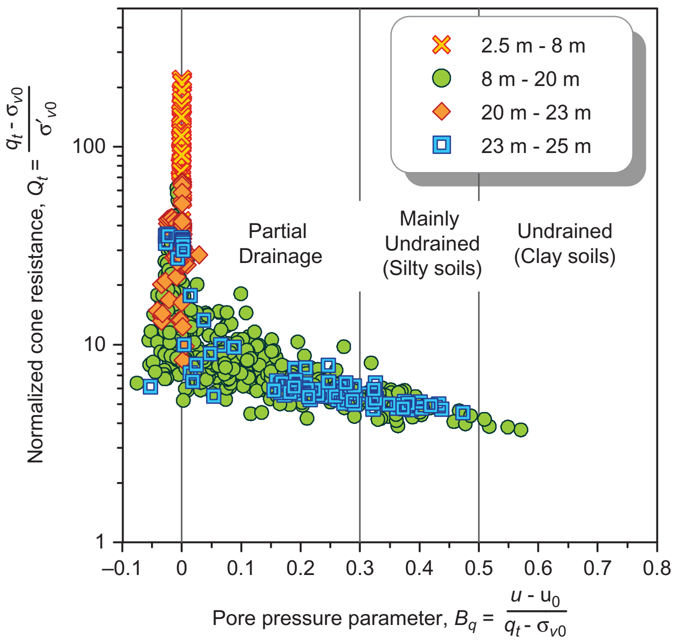

Figure 5. Qualitative assessment of drainage conditions on Venetian sediments.

March, i.e. it developed simultaneously with the loading bank construction.

An effective description of the subsoil deformation throughout the loading process can be obtained by plotting the field differential displacements in terms of the effective vertical stress σ'_v. As an example, Figure 6 shows a selection of the ε_v - Logσ'_v curves derived from strain measurements occurred along the centreline (sliding deformeter S3), between 6 and 23 m depth. Both sandy and silty layer responses are thus depicted in the plot.

All curves, independently of the soil classes, exhibit a similar shape which basically recalls the typical trend of ε_v - Logσ'_v plots obtained from oedometer tests. As evident from figure, the first cycle of loading has generally produced, on the semilog plane, a somewhat straight and flat curve, followed by a smooth but rather pronounced transition into a steep and relatively straight line. A similar trend could be observed also in the curves derived from measurements at S1, S2 and S4, although it must be emphasised that only for the central sliding deformeter, S3, the assumption of 1D conditions can be reasonably accepted.

Simonini (2004) remarked that the slope change observed in the curves of Figure 6 can be interpreted in terms of yielding stress σ'_Y, which can be in turn assumed as an approximation of the preconsolidation pressure σ'_p. Such interpretation of the Treporti field data confirmed that Venetian sediments are basically normally consolidated or slightly overconsolidated, with OCR values ranging in the interval 1.1 – 2. The latter part of the ε_v - Logσ'_v curve is due to the strains occurred at constant vertical stress, i.e. from the completion of the bank construction until its removal (March 2003–April 2007).

Alternatively, field differential displacements recorded by the sliding deformeter S3 during both loading and stationary stages, have been plotted as curves of vertical strain ε_v against Log$t$ (Figure 7). As evident from figure, the resulting curves have very similar shapes to those obtained by plotting

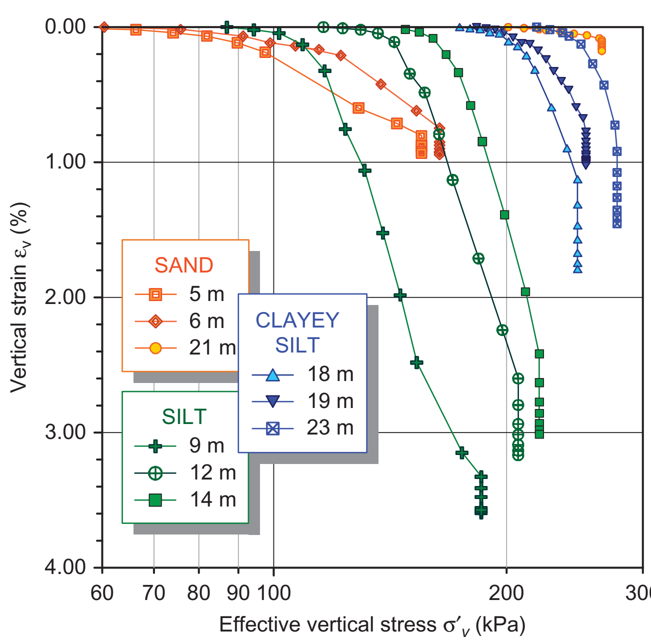

Figure 6. Field axial strains versus corresponding stress beneath the centre of the loaded area.

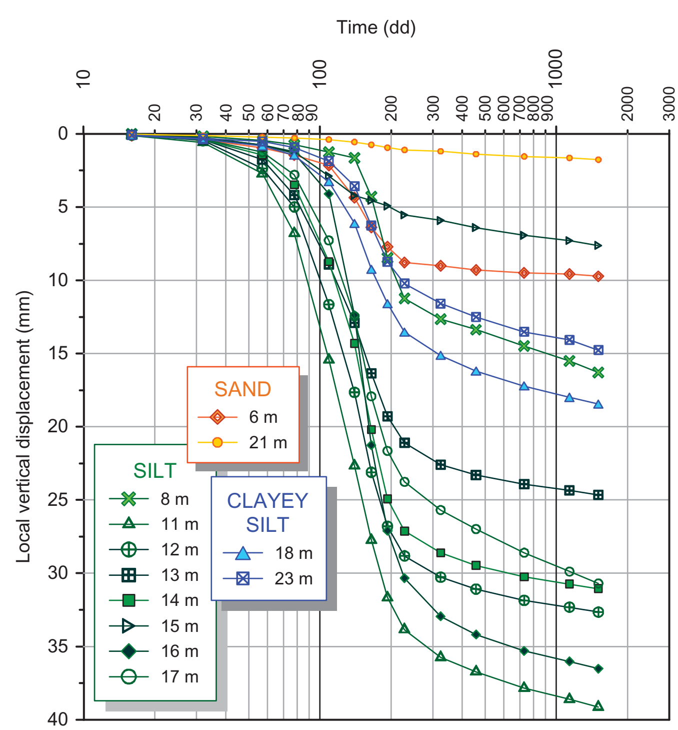

Figure 7. Local vertical displacements occurred beneath the centre of the loaded area, from the beginning of the bank construction until its removal.

deformation dial readings versus log time for a given load increment in an oedometer test. The graphs clearly show that the latter part of the curves, derived from deformations occurred after the bank construction, is usually found to be sloping and approximately linear.

6. Vertical strain analysis

6.1 *Primary compression*

As already remarked, a proper interpretation of the Treporti field data must take into account that the time rate of consolidation turned out to be quite high and that primary consolidation essentially took place during the bank construction stage. Therefore, deformations occurred after the bank completion are most likely to be ascribed to secondary compression. Furthermore, although secondary compression might have certainly occurred as part of the primary consolidation phase, the widely-accepted assumption of treating such components of deformation separately has been adopted in this study. Accordingly, a compression ratio $C_{C\varepsilon}$ $(= C_c/1{+}e_0)$ has been obtained from curves of the type depicted in Figure 6, by calculating the slope of the straight line describing soil compressibility in the normally-consolidated domain. The analysis has mainly focused on the ε_v - Logσ'_v curves provided by the central sliding deformeter (S3) measurements between 5 and 25 m depth, hence a total of approximately 20 different values of $C_{c\varepsilon}$ have been determined along this vertical. Results are shown in Figure 8a.

Despite a certain scatter of results, most likely due to high heterogeneity and chaotic alternation of different grain-sized sediments within the 1 m interval, it was found that typical values of $C_{c\varepsilon}$ for sands generally fall in the interval 0.04–0.05. In very few cases such parameter turned out to be one order of magnitude smaller, around 0.005. Higher values of $C_{c\varepsilon}$ have been provided by field data for the silty layers from 8 to 20 m depth, and a mean value equal to 0.18 was calculated within this soil unit. However, the computed compression ratio in silts seems to vary quite significantly, ranging from 0.09 to 0.34. The highest values (0.23–0.34) have been computed below 15 m depth, where the clay content in the silty sediments becomes significant.

The procedure has been also applied to the ε_v - Logσ'_v curves associated to S1, S2 and S4, and a compression ratio $C_{c\varepsilon}{}^*$, though not referable to 1D conditions, has been obtained.

6.2 *Secondary compression*

Secondary compression of the Treporti sediments has been described in terms of the coefficient $C_{\alpha\varepsilon}$, which was determined from the slope of the approximately straight line portion of the ε_v - Log$t$ curves depicted in Figure 7. In this way, a set of $C_{\alpha\varepsilon}$ has been computed at 1-m intervals, from 5 to 25 m depth. This definition is analogous to the previously defined secondary

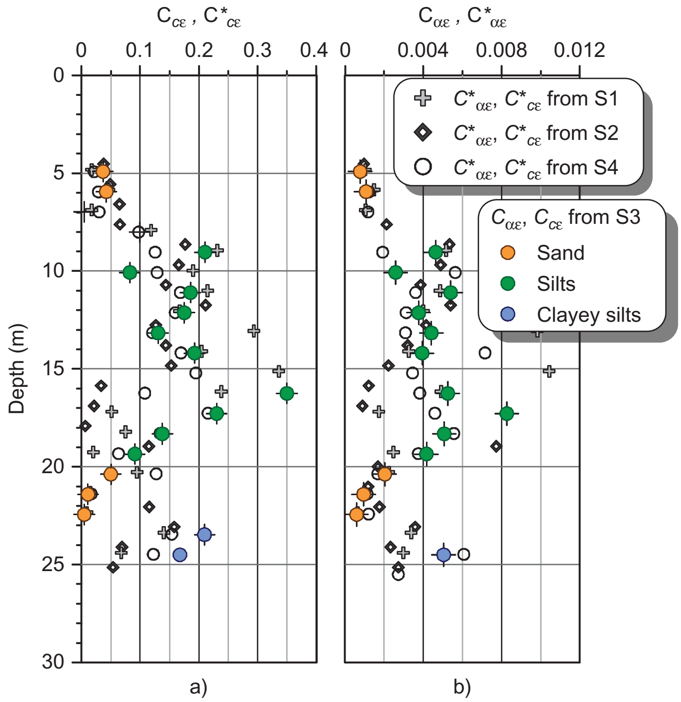

Figure 8. a) Back-calculated $C_{c\varepsilon}$ and $C_{c\varepsilon}{}^*$ values from the sliding deformeters S1, S2, S3 and S4; b) back-calculated $C_{\alpha\varepsilon}$ and $C_{\alpha\varepsilon}{}^*$ values from the sliding deformeters S1, S2, S3 and S4.

compression parameters $C_{\alpha e}$ and $C_{\alpha\varepsilon}$, obtained from oedometer tests.

It must be observed here that, on the basis of field evidence, $C_{\alpha\varepsilon}$ seems to slightly change with time, as shown by the straight to concave-upward shape of the ε_v-Log$t$ curves associated with the central sliding deformeter S3. Such behaviour essentially seems to confirm results obtained by other researchers, both in the laboratory and in the field. Indeed, as already mentioned in this paper, Leroueil *et al*. (1985) reported a general non linear response based on long-term creep tests and observed that normally consolidated specimens show a continuously decreasing slope with the logarithm of time after primary consolidation, similarly to Treporti field results.

In this study, $C_{\alpha\varepsilon}$ values have been eventually derived from data associated with the early stages of the secondary compression process. In this way, $C_{\alpha\varepsilon}$ in silts turned out to generally vary between 0.0026 and 0.0054, with a mean value of approximately 0.0046. Secondary compression in sands is typically described by $C_{\alpha\varepsilon}$ in the range 0.00058–0.00095.

A modified coefficient of secondary compression $C_{\alpha\varepsilon}{}^*$, though referable to 3-D deformation conditions, has been also calculated from the slope of the ε_v-Log$t$ curves associated with the sliding deformeters S1, S2 and S4. Results are plotted in Figure 8b. Figure 9 provides the computed values of both $C_{\alpha\varepsilon}$ and $C_{\alpha\varepsilon}{}^*$, plotted as function of the corresponding values of $C_{c\varepsilon}$ and $C_{c\varepsilon}{}^*$ obtained from primary compression data. In this way, the relationship between $C_{\alpha\varepsilon}$ and $C_{c\varepsilon}$, expressed in terms of $C_{\alpha\varepsilon}/C_{c\varepsilon}$ (Mesri and Godlewski 1977) could be analysed, also in relation to the different classes of sediments forming the

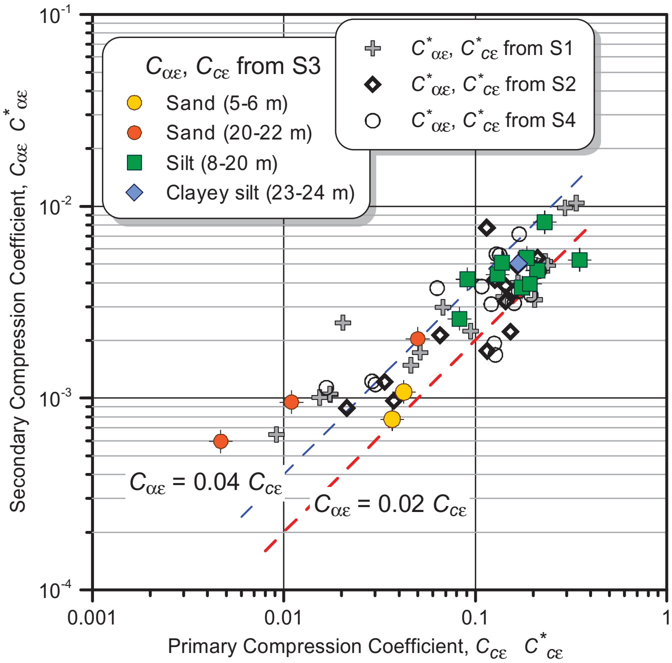

Figure 9. Secondary compression coefficient $C_{\alpha\varepsilon}$ ($C_{\alpha\varepsilon}{}^*$) versus compression ratio $C_{c\varepsilon}$ ($C_{c\varepsilon}{}^*$), from field data.

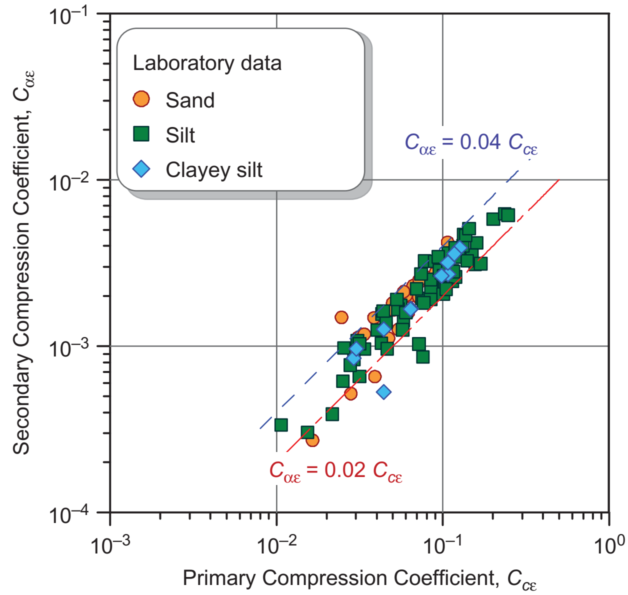

Figure 10. Secondary compression coefficient $C_{\alpha\varepsilon}$ versus compression ratio $C_{c\varepsilon}$, from laboratory tests.

Venetian lagoon basin. As evident from Figure 9, points generally fall in the interval $C_{\alpha\varepsilon}/C_{c\varepsilon} = 0.02$–$0.04$, even when the ratio $C_{\alpha\varepsilon}{}^*/C_{c\varepsilon}{}^*$ is considered. Only the coarse sediments from 20 to 22 m depth significantly exceed this upper limit. In all the other cases, the ratio $C_{\alpha\varepsilon}/C_{c\varepsilon}$ seems to be independent of the soil type.

Results deduced from field observations can be compared to those represented in Figure 10, which shows values of $C_{\alpha\varepsilon}$ and $C_{c\varepsilon}$ obtained from a large number of one - dimensional laboratory compression tests performed on high-quality samples. According to laboratory evidence, the ratio $C_{\alpha\varepsilon}/C_{c\varepsilon}$ generally falls in the range 0.02–0.04; typical values for sands (0.031) are close to the upper limit, whilst intermediate values of $C_{\alpha\varepsilon}/C_{c\varepsilon}$ (0.0281) are basically associated with silts. The range for Clays is somewhat lower (0.0267), although it must be emphasised that no significant differences in terms of $C_{\alpha\varepsilon}/C_{c\varepsilon}$ can be appreciated between the three Venetian soil classes. This result may be partly due to the common geological and mineralogical origin of the lagoon sediments, but even more to the high heterogeneity of Venetian subsoil. Indeed, the complex assortment of sands, silts and clays makes homogeneous specimens rather unlikely to obtain for laboratory testing, whilst the effect is less pronounced at the macro-scale level.

Finally, it is worth remarking that the in situ secondary compression parameters $C_{\alpha\varepsilon}$, as obtained from the sliding deformeters measurements, generally turned out to be higher than values obtained from laboratory tests.

This result confirms the observations reported by Leroueil (2006), based on a number of very well documented embankments on clays for which high quality samples were also available. As pointed out by several authors, differences between values measured in the laboratory and deduced from field observation are not only due to sampling disturbance, but more likely to strain rate effects. According to the well-known hypothesis B and the isotache model (Leroueil 2006), such a kind of response is not surprising: laboratory tests provide compression curves corresponding to strain rates that are generally larger than $3{\cdot}10^{-8}$ s^{-1}, whilst strain rates under embankments are somewhat lower, generally lower than 10^{-9} s^{-1}. In the case of Treporti test bank, the calculated strain rates generally turned out to vary in the range 10^{-11} s^{-1} – 10^{-9} s^{-1}. On the other hand, comparison between field-deduced and laboratory values of $C_{\alpha\varepsilon}$ doesn't seem to be very meaningful, due to the difficulties in obtaining undisturbed samples of such predominantly silty sediments.

7. CPTU-based interpretation of secondary compression

Over the last years a large amount of research has been carried out on the interpretation of the field data collected in Treporti, attention being mainly paid to the evaluation of the mechanical properties governing primary compression. The application to Venetian sediments of existing and well-established empirical correlations for the estimate of the constrained modulus, M, revealed evident limitations of classical approaches, thus suggesting that the experience gained for clays and sands cannot be simply extended to such highly heterogeneous silty sediments. Accordingly, more suitable site-specific correlations have been developed (Tonni *et al.* 2010, Tonni and Gottardi 2011).

In this section we provide an attempt to relate the secondary compression parameter $C_{\alpha\varepsilon}$, as obtained from in situ measurements, to the CPTU data collected in Treporti. Indeed, we have already pointed out that the whole mechanical response of Venetian sediments, including their time-dependent behaviour, is mostly controlled by inter-particle friction. Similarly, cone penetration into these soils is basically governed by frictional response, hence there are valid reasons to expect an effective relationship between $C_{\alpha\varepsilon}$ and the cone resistance, both being affected by a common soil behavioural factor.

Empirical correlations between the secondary compression coefficient $C_{\alpha\varepsilon}$ and penetration test results, expressed in terms of the solely corrected cone resistance q_t, have been investigated previously. Figure 11 shows a total of about 70 paired observations of $C_{\alpha\varepsilon}$ (also including the extra-centreline values $C_{\alpha\varepsilon}{}^*$) and q_t, obtained from side-by-side sliding deformeters and piezocone tests, i.e. S1-CPTU15, S2-CPTU16, S3-CPTU 14 and S4-CPTU17. Again, points associated with strain and cone resistance data recorded along the centreline have been depicted using different symbols, in relation to the different grain size characteristics of sediments.

Despite a certain scatter, plotting data in a log-log plane provides evidence of a rather straightforward trend between $C_{\alpha\varepsilon}$ and q_t, which can be described by a power function of the form:

$$C_{\alpha\varepsilon} = k \cdot \left(\frac{q_t}{p_a}\right)^h \tag{5}$$

where p_a is the atmospheric pressure, expressed in the same units as q_t. The regression analysis performed on the whole set of data points gave $k = 0.033$ and $h = -0.80$, with a coefficient of determinations R^2 equal to 0.64.

The best-fit regression line depicted in Figure 11 is solely based on the data recorded along the centreline of the loaded area, where 1D conditions are very likely to apply. In this case, constants k and h turned out to be equal to 0.066 and -1.03 respectively, with $R^2 = 0.74$.

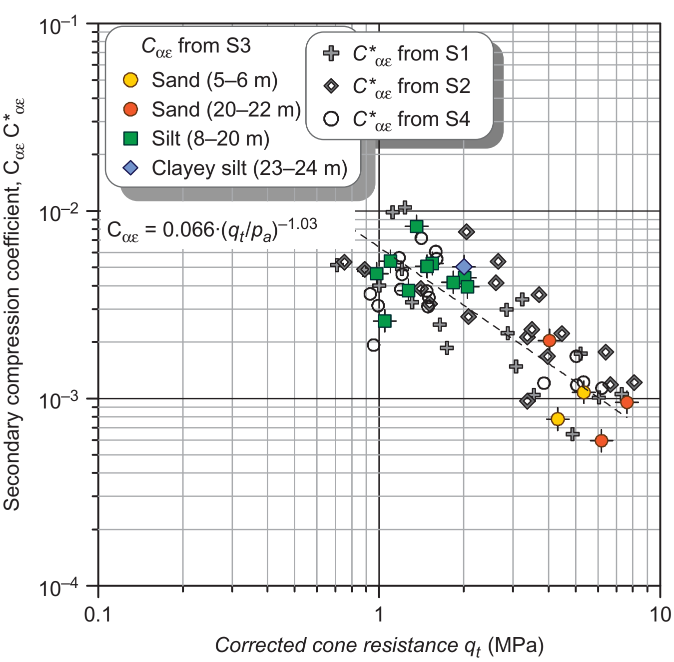

Figure 11. *In situ* $C_{\alpha\varepsilon}$ as a function of the corrected cone resistance q_t.

Afterwards, in view of the recommended procedure of using dimensionless variables in CPT-based correlations (Wroth 1988), attempts have been made to develop a power function expression for $C_{\alpha\varepsilon}$ in terms of the dimensionless normalised cone resistance Q_{tn}, as recently defined by Robertson (2009). In this approach, an iterative, nonlinear stress normalisation procedure is applied to the corrected cone resistance q_t, leading to the following expression:

$$Q_{tn} = \frac{q_t - \sigma_{v0}}{p_a} \cdot \left(\frac{p_a}{\sigma'_{v0}}\right)^n \tag{6}$$

where p_a is the atmospheric pressure and n is a stress exponent that depends in turn on both stress level and the well-known soil behaviour type index I_c, originally defined as:

$$I_c = \sqrt{(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2} \tag{7}$$

with:

$$F_r = 100 \cdot \frac{f_s}{q_t - \sigma_{v0}} \tag{8}$$

Q_t is the normalised cone resistance, previously defined in Equation (3).

According to experimental observations, Robertson (2009) suggested the following expression for n:

$$n = 0.38 I_c + 0.05 \cdot \left(\sigma'_{v0}/p_a\right) - 0.15 \tag{9}$$

where the soil behaviour type index I_c should be defined using Q_{tn} instead of Q_t, hence the iterative nature of the scheme.

In this way, a variable exponent n (≤ 1) is adopted for stress normalisation, depending on the soil class to be considered. Such exponent turned out to typically vary between 0.5–0.6 in the upper sandy layer, whilst in the silty unit it tends toward 1.0. The effectiveness of the normalisation procedure has been recently confirmed with respect to the Treporti piezocone database, by applying the method to adjacent tests performed in the first and second phases of the testing campaign. Details can be found in Tonni *et al*. (2011). As observed by Robertson (2009), the above recommended normalisation is not an arbitrary approach, but is based on experimental observations designed to improve correlations with various geotechnical parameters.

Figure 12 provides the profile of the computed I_c index (labelled as I_{cn} since it is expressed in terms of Q_{tn}) deduced from CPTU 14 data, in conjunction with the I_{cn}-based boundaries of the soil type zones, originally identified by Robertson (1990) on the well-known normalised *Soil Behaviour Type* (SBT) Q_t - F_r chart and later included in the recent SBTn Q_{tn}- F_r chart (Robertson 2009). Furthermore, the profile of the normalised cone resistance Q_{tn} has been plotted.

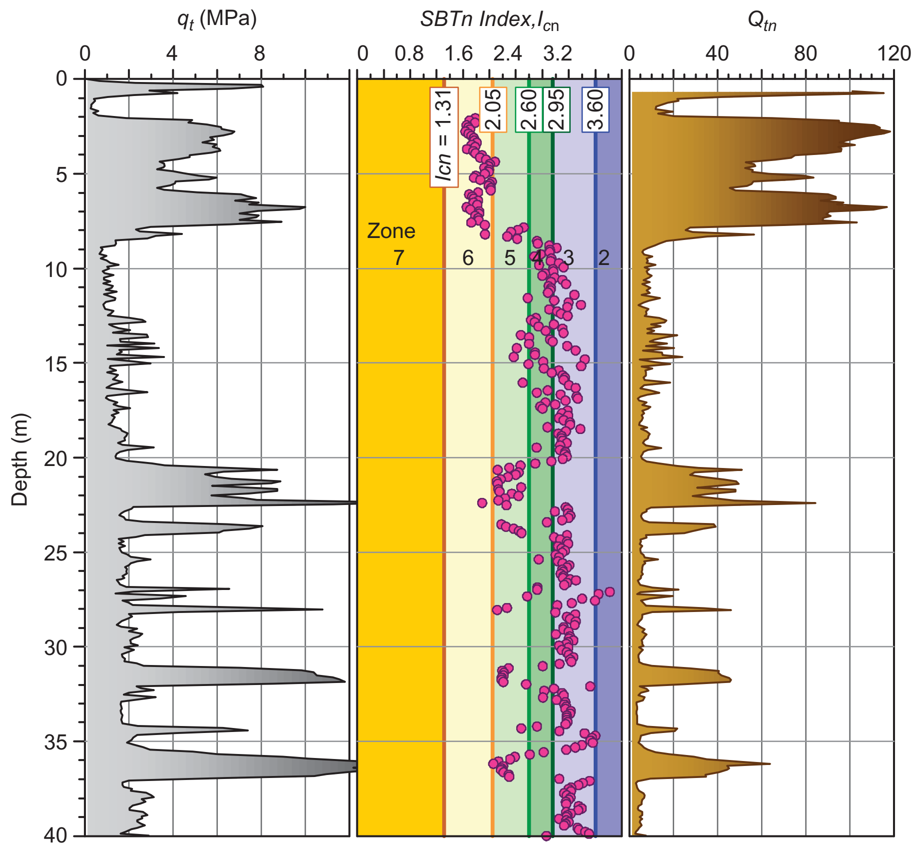

Figure 12. Profiles of the corrected cone resistance q_t, soil behaviour type index I_{cn} and normalized cone resistance Q_{tn} for CPTU 14.

The observed trend between $C_{\alpha\varepsilon}$ (including $C_{\alpha\varepsilon}{}^*$ values) and Q_{tn} is presented in Figure 13. Again, data from the centreline are separated according to the soil type. Compared to the q_t - $C_{\alpha\varepsilon}$ correlation, the regression relating $C_{\alpha\varepsilon}$ to Q_{tn} gives a better fit. Assuming a log-log relationship, the equation of the best fit regression line depicted in Figure 13, based on data from the centreline, is given by:

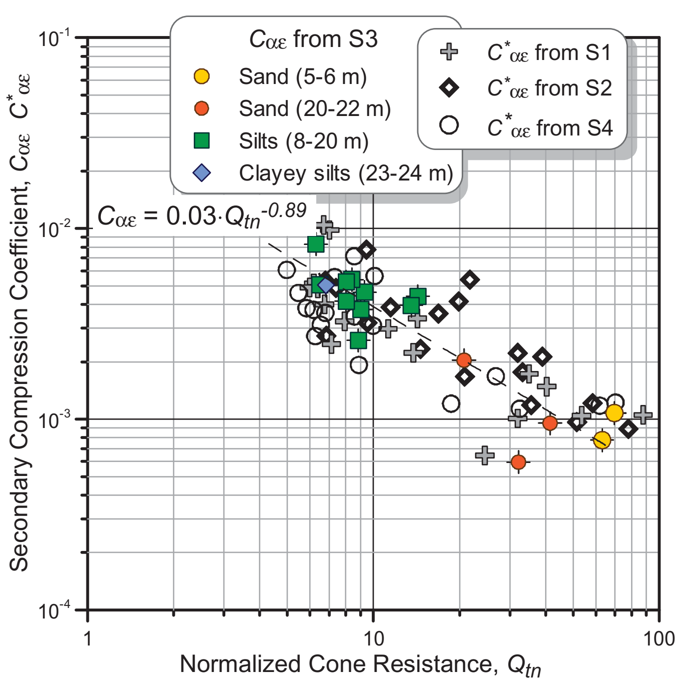

Figure 13. *In situ* $C_{\alpha\varepsilon}$ as a function of the normalized cone resistance Q_{tn}.

$$C_{\alpha\varepsilon} = 0.03 \cdot (Q_{tn})^{-0.89} \tag{10a}$$

with $R^2 = 0.83$.

Considering all the available data, the general regression trend is given by:

$$C_{\alpha\varepsilon} = 0.018 \cdot (Q_{tn})^{-0.69} \tag{10b}$$

with $R^2 = 0.68$. In this latter case, $C_{\alpha\varepsilon}$ stands for both $C_{\alpha\varepsilon}$ and $C_{\alpha\varepsilon}{}^*$, according to previous definitions.

It's worth observing here that the use of a variable exponent, accounting for the soil state under a given stress condition, was proved to provide better estimates of $C_{\alpha\varepsilon}$ also in comparison to the dimensionless cone resistance Q_t (Equation (3)), based on a linear stress normalisation scheme (Wroth 1984, Robertson 1990). On the other hand, no significant improvements seem to be attained if the soil behaviour type index I_{cn} is included as an independent variable in the multiple regression analysis.

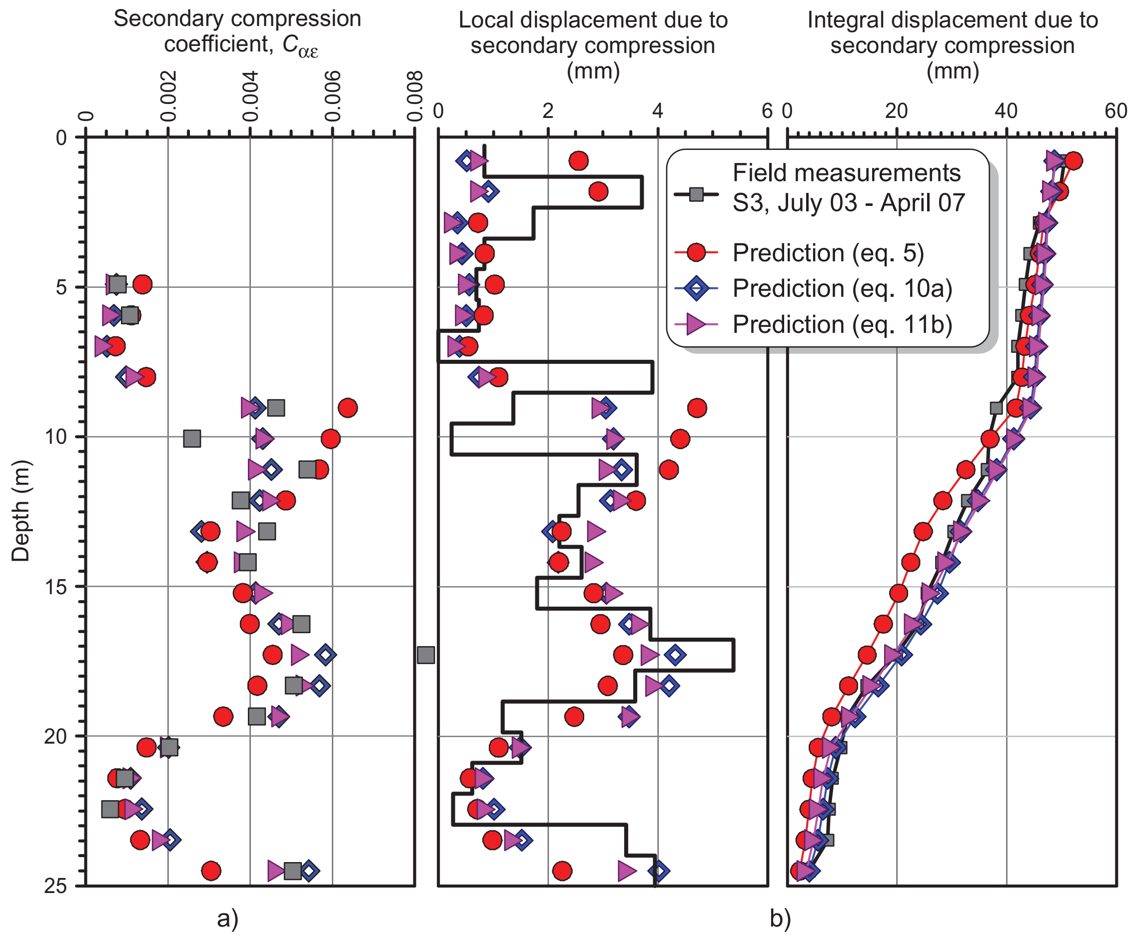

Figure 14. a) Comparison between field and predicted $C_{\alpha\varepsilon}$, beneath the centre of the loaded area; b) local and integral vertical displacements, from July 2003 to April 2007.

The predicted $C_{\alpha\varepsilon}$ values deduced from Equations (5) and (10a), in conjunction with the associated settlement predictions occurred in the period July 2003–April 2007 beneath the centre of the loading bank, are plotted in Figure 14. A general good agreement between field observations and predictions can be found.

Despite the satisfactory predictive capability of the above correlations, it is worth remarking now that neither of them explicitly takes into account partial drainage phenomena during cone penetration in Venetian silts. On the other hand, it has been widely emphasised (e.g., Schnaid *et al*. 2004, Tonni and Gottardi 2010) that such effect should be taken in careful consideration in order to improve understanding and interpretation of penetration data in intermediate soils.

Accordingly, it seemed reasonable to include an additional factor into the Q_{tn}-$C_{\alpha\varepsilon}$ relationship, accounting for the different pore pressure response in relation to the drainage conditions around the advancing cone. Following recent studies on trends in normalised piezocone response as a function of the drainage degree (e.g. Schneider *et al*. 2007, 2008), the excess pore pressure ratio, $\Delta u/\sigma'_{v0}$, was eventually considered as the more appropriate and simple variable to adopt in order to tackle the issue. Thus, a multiple regression analysis was performed in a log-log format to provide a power function expression for $C_{\alpha\varepsilon}$ in terms of the normalised cone tip resistance Q_{tn} and the excess pore pressure ratio-based factor, $(1+\Delta u/\sigma'_{v0})$.

For all the available data, the least square analysis gave the following relationship:

$$C_{\alpha\varepsilon} = 0.035 \cdot (Q_{tn})^{-0.87} \cdot \left(1 + \frac{\Delta u}{\sigma'_{v0}}\right)^{-0.55} \tag{11a}$$

with $R^2 = 0.72$.

When the analysis is restricted to the sole data collected over the centreline of the test bank, the equation of the best-fit regression curve turns out to be:

$$C_{\alpha\varepsilon} = 0.077 \cdot (Q_{tn})^{-1.14} \cdot \left(1 + \frac{\Delta u}{\sigma'_{v0}}\right)^{-0.74} \tag{11b}$$

with $R^2 = 0.86$.

It is interesting to note that the use of $(1+\Delta u/\sigma'_{v0})$ gives the highest R^2, whether the analysis includes the only centreline-located data or all the available measurements, thus confirming that this specific factor and $C_{\alpha\varepsilon}$ can be related in a rational manner. Settlements predictions obtained from Equation (11b) have been plotted in Figure 14.

8. Conclusions

The paper has focused on the use of piezocone measurements for the estimate of the one-dimensional creep characteristics of sands and silts, as an alternative on the classical laboratory tests. Unlike clayey deposits, the time-dependent behaviour of sands and silts has been traditionally disregarded, although there is experimental evidence that creep strains in granular soils are not negligible. Hence, the development of empirical correlations based on piezocone data may be very useful for estimating the secondary compression coefficient at sites where undisturbed sampling is very difficult to achieve and geotechnical characterisation essentially relies on *in situ* testing. The approach proposed in this paper arises from the observation that frictional response essentially governs both cone resistance and secondary compression in granular soils, therefore it seems reasonable to examine possible correlative trends between q_t and $C_{\alpha\varepsilon}$.

The study uses, as a reference database, part of the large amount of CPTU data and subsoil deformation measurements assembled over the last years at the Treporti Test Site (Venice, Italy), within an extensive research project aimed at thoroughly analysing the stress-strain-time response of the complex assortment of sandy and silty sediments forming the Venetian lagoon subsoil. Despite soil heterogeneity makes sometimes difficult the analysis of field data, the study seems to indicate a general trend between the secondary compression index of both silts and sands and the cone tip resistance. According to the statistically-derived relationships proposed for $C_{\alpha\varepsilon}$, a significant improvement in the coefficient of determination is attained if the cone resistance q_t is expressed in a dimensionless form, Q_{tn}, using a recent nonlinear stress normalisation procedure, accounting for the stress level and the soil class effects on piezocone measurements.

Regression analyses also indicate that the estimate of the secondary compression coefficient may be slightly improved if measurements of the excess pore pressure Δu, normalised to σ'_{v0}, are included as an additional independent variable, accounting in some way for the different pore pressure response in relation to the drainage conditions around the advancing cone. Indeed, due to the intermediate nature of Venetian sediments and the complex assortment of different grain-sized soils, different degrees of partial drainage are very likely to occur under a standard rate of penetration, with significant implications on pore water pressure and cone resistance measurements as well as on reliable application of design correlations. From the application of the proposed solutions, a general good agreement between field observations and long-term settlement predictions has been found.

Research is currently focusing on the verification of such site-specific correlations at different areas of the Venetian lagoon, with particular reference to the evaluation of long-term settlements of the Venetian coastal defences. The validation of the proposed solutions to independent cases, where all the relevant parameters are well documented, is likely to constitute an important contribution to the practice of geotechnics in this area.

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