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ORIGINAL PAPER

Strength and Compressibility Characteristics of a Soft Clay Subjected to Ground Treatment

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Abstract The compressibility properties, undrained shear strength, and stress history are essential for reliable calculations of settlement and bearing capacity of soft soils. However, it is sometimes a challenge to determine representative parameters for very soft and high plasticity clays, which are often found in Brazilian coastal areas. In this study an extensive site investigation was planned aiming to interpret the behaviour of a trial embankment on stabilized soft ground. The site investigation was carried out in a test area located in the west of city of Rio de Janeiro, consisted of three clusters in which standard penetration tests, vane shear tests, and piezocone tests (CPTu) were performed. A number of correlations was developed and compared with empirical equations in order to verify their reliability. The results of these tests made it possible to define geotechnical parameters of the soft clay to use in the numerical and analytical computations of the embankment on reinforced ground.

Keywords Site investigation · Soft soil · Ground treatment · Undrained strength · Compressibility

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List of Symbols

γ'	Submerged unit weight
ϕ'	Drained angle of friction
c'	Effective cohesion
G_s	Specific gravity
w_n	Natural water content
w_L	Liquidity limit
w_p	Plasticity index
C_c	Compressibility index
C_s	Swelling index
CR	Compression ratio
e_0	Initial voids ratio
$e_{(\sigma'_{v0})}$	Voids ratio at in situ vertical stress level
k_v	Vertical permeability coefficient
k_h	Horizontal permeability coefficient
c_v	Vertical consolidation coefficient
c_h	Horizontal consolidation coefficient
G	Shear modulus
E_{oed}	Oedometer modulus
E_{ur}	Undrained elastic modulus

I_r	Stiffness ratio
K_0	Earth pressure coefficient
σ_{v0}	Initial vertical total stress
σ'_{v0}	In situ vertical effective stress
σ'_{h0}	In situ horizontal effective stress
σ'_{vm}	Pre-consolidation effective stress
σ_3	Confining stress
S_u	Clay undrained strength
S_{ur}	Undrained strength of remoulded clay
S_t	Clay sensitivity
N_{kt}, N_{ke}, $N_{\Delta u}$	Piezocone empirical factors
R	Radius of piezocone probe
u_0	Hydrostatic water pressure
u_1	Pore water pressure at cone face
u_2	Pore water pressure at cone tip
T^*	Time factor
q_T	Corrected cone resistance
z	Depth
J	Geosynthetic tensile modulus

1 Introduction

Increasing development frequently leads to the occupation of lowlands in which the soft clay layer may reach a considerable thickness. Geotechnical design projects carried out on soft soils must account for deformations and bearing capacity concerning the low shear strength and high compressibility of such soft deposits (Gniel and Bouazza 2010). The geotechnical parameters of soft soils are mainly determined using a proper combination of the in situ and laboratory tests. The SPT test is frequently used for preliminary estimation of the ground profile and water content measurement (Almeida and Marques 2013). The vane shear test is usually employed to determine the in situ undrained strength (S_u) and clay sensitivity (S_t); however, test conditions are much less controlled than those of other field or laboratory tests. The piezocone test is particularly effective for soft clays, as it allows the estimation of both the undrained strength and consolidation characteristics, which are key properties of soft soils (Lunne et al. 1997; Schnaid 2009). Nevertheless, the clay parameters such as the compressibility index can also be correlated with the water content values (Nagaraj and Miura 2001; Mesri and Ajlouni 2007; Futai et al. 2008). This low-cost and simple procedure can provide a first estimation of soil characteristics across an extensive area and can thus be used in preliminary geotechnical design. A common procedure employed for determination of these properties includes undisturbed sampling and relevant laboratory tests in a carefully controlled environment such as triaxial and oedometer consolidation tests in order to estimate the time-dependent behaviour of soft soils.

In this paper the geotechnical properties of the Rio de Janeiro soft clay are determined through a suitable combination of the laboratory and in situ tests performed in a test area located at the ThyssenKrupp steel plant. Although previous studies (e.g. Marques et al. 2008; Almeida et al. 2014) have provided significant information regarding the general soft clay properties of the site, owing to its large area of around 0.5 km^2 a more specific site investigation is considered necessary in order to provide a set of geotechnical parameters aiming to interpret adequately the field response of a trial embankment over treated soft foundation. The soft clay properties are also used in numerical and analytical calculations of the test embankment in order to verify the reliability of those parameters obtained through site investigation programme.

2 Site Description

In 2008 the ThyssenKrupp Steel Company (TKCSA) commenced the construction of a steel plant in the Brazilian coastal lowlands near the city of Santa Cruz, including a stockyard for coal and coke material. The total stockyard area was 800 × 600 m, with that part set aside for coal/coke covering 800 × 350 m as shown in Fig. 1a. The entire area consisted of soft clay layers with low bearing capacity and the ground water level close to the clay surface. Different types of soft soil improvement techniques were adopted to overcome the design difficulties over clayey area (Almeida et al. 2015). In part of the area, the optimum solution was composite foundation with granular columns encased by geotextile thus providing adequate bearing capacity whilst simultaneously reducing settlement. In order to assess the effectiveness of the encased columns, a trial embankment was built on a test area shown in Fig. 1b. Prior to the embankment construction, an extensive site investigation was carried out to determine the geotechnical parameters of the soft clay layers for later use in the numerical analysis.

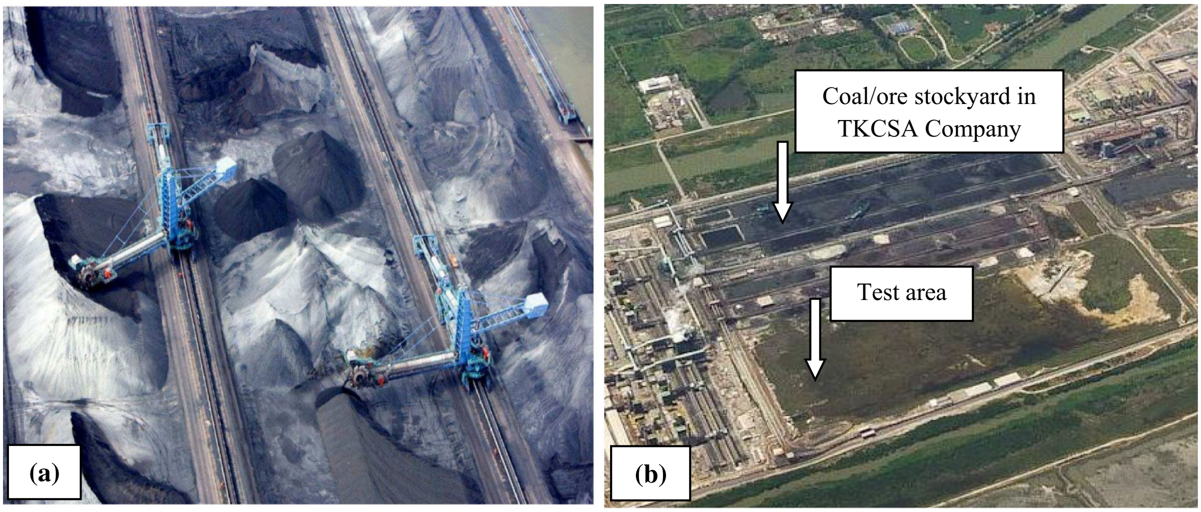

Fig. 1 General view of the TKCSA plant: **a** coal/ore stockyard site; **b** location of the test area

Three standard penetration tests (SPT) were performed as the first step of site investigation with the position of the clusters illustrated in Fig. 2. The site investigation was followed by three vertical piezocone tests comprising 10 dissipation tests (CPTu), 14 vertical vane shear tests (VST) and five undisturbed samples (SM) collected in the different depths as shown in Fig. 3. A 2.0 m-thick working platform was placed on top of the soft layer to provide ground surface for stable construction activities. As seen in Fig. 3, the subsoil is mainly characterised by 6.0 m-thick upper soft clay layer (soft clay I) and about 2.5 m-thick soft clay II intersected by a thin zone of medium sand layer. Soft clay II is underlain by a medium dense sand layer with a thickness of 0.9 to 2.0 m, which is in turn followed by 1.7 to 3.6 m-thick stiff clay. The remaining soil profile comprises dense sand layer to a depth of about 20 m. The groundwater level was also found to locate between 1.0 to 1.5 m below the ground surface.

3 Laboratory Tests

Index tests, oedometer consolidation, and undrained triaxial tests were carried out on five undisturbed

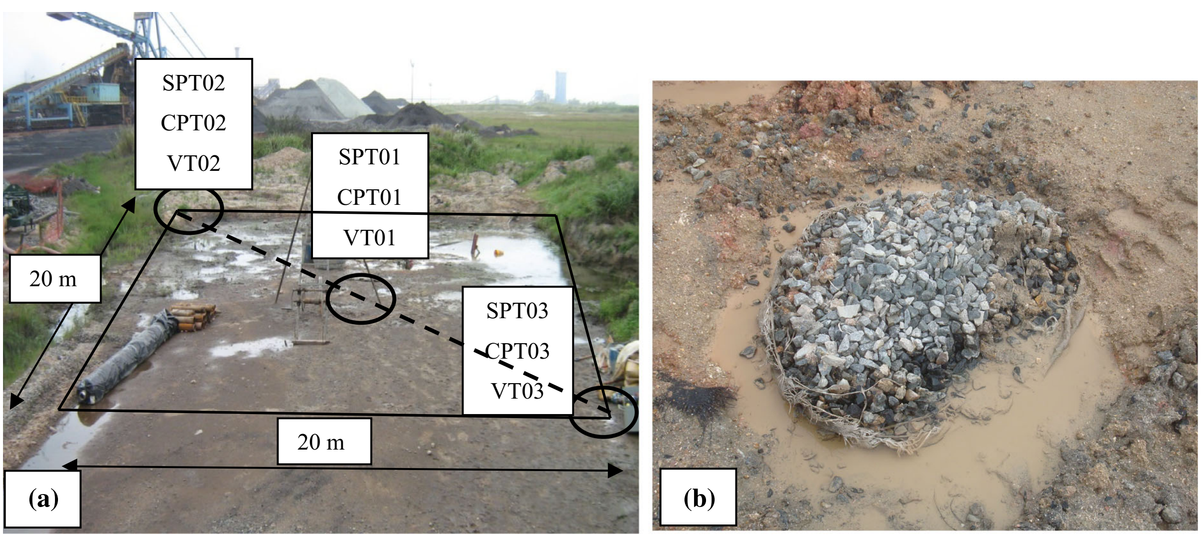

Fig. 2 Over view of the test area: **a** layout of the research clusters; **b** encased granular column implemented in the test area

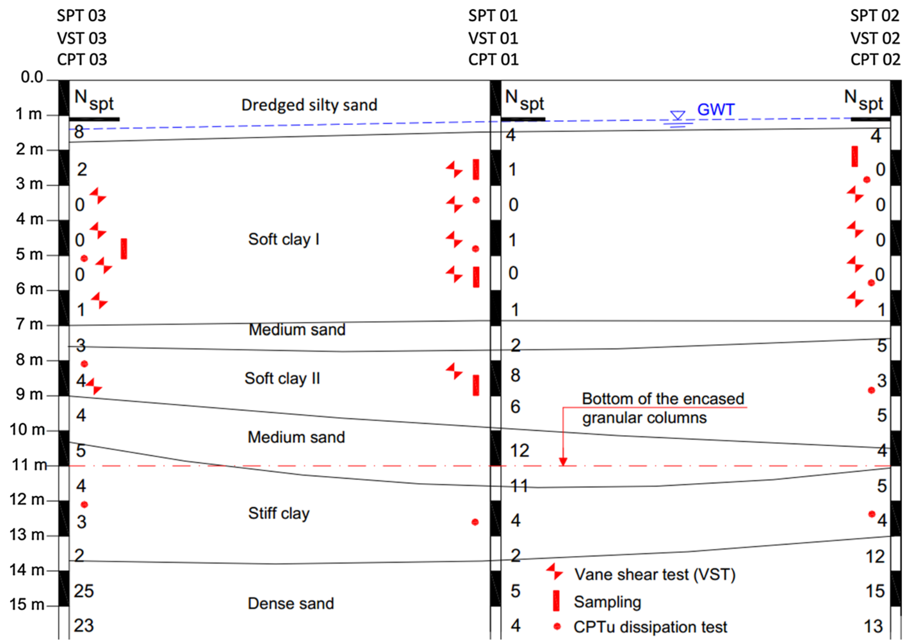

Fig. 3 Subsurface profile in the test area and position of the in situ tests

samples collected in the soft clay layers with their depths illustrated in Fig. 3. Soil sampling was performed using a 10 cm diameter Shelby sampler with a stationary piston following Brazilian recommendations NBR-9820 (1997), as same as those adapted by Ladd and DeGroot (2003). In the following sections the main geotechnical parameters of soft clay studied will be presented and compared with the previous correlation found for Brazilian soft clay.

3.1 Index Properties

As can be seen in Fig. 4a, the natural water content (w_n) is close to the liquidity limit (w_L), with an average value of $w_n = 125\%$ and $w_n = 55\%$ in soft clay I and soft clay II, respectively. The British standard (BS 5930- BSI, 1999) classifies soils with liquidity limit (w_L) higher than 50% as high compressibility soils, calling them H (high plasticity) for the range of $50\% < w_L < 70\%$; V (very high plasticity) for $70\% < w_L < 90\%$; and E (extremely high plasticity) for $w_L > 90\%$. Following this classification, the great majority of studied clayey deposits present extremely high plasticity (E) which is typical for the coastal soft soils of south eastern Brazil (Coutinho and Lacerda 1987; Jannuzzi et al. 2015).

Grain specific density G_s, presented in Fig. 4b, is lower in surface layers, apparently due to a higher concentration of organic matter, and increases with depth with an average value equal to $G_s = 2.55$ along the soft clay studied. As shown in Fig. 4c, the submerged unit weight (γ') varies between 3.5 and 4.0 kN/m^3 for soft clay I, a typical value of very soft clay in the west of the Rio de Janeiro district (Futai et al. 2008). The increasing submerged unit weight of soft clay II indicates a higher undrained strength, as confirmed by vane shear tests and SPT blow counts. Figure 4d shows an increase in the initial void ratio with depth in the soft clay I, reaching a maximum of $e_0 = 3.12$ at a depth of $z = 5$ m, where the highest natural water content ($w_n = 186\%$) is also observed. The void ratio values then decreased gradually in soft clay II, an evidence for less compressibility behaviour of this layer compared to upper soft clay.

3.2 Sample Quality Assessment

A reliable interpretation of laboratory tests is essentially dependent on the quality of the undisturbed samples. Soft soil sampling involves several operations which induce stress state changes and disturbance; even perfect hypothetical sampling inevitably leads to relief of soil stress (Ladd and Lambe 1963; Hight 2001; Almeida and Marques 2013).

Sample quality was assessed using the criteria proposed by Lunne et al. (1997) and Coutinho (2007)

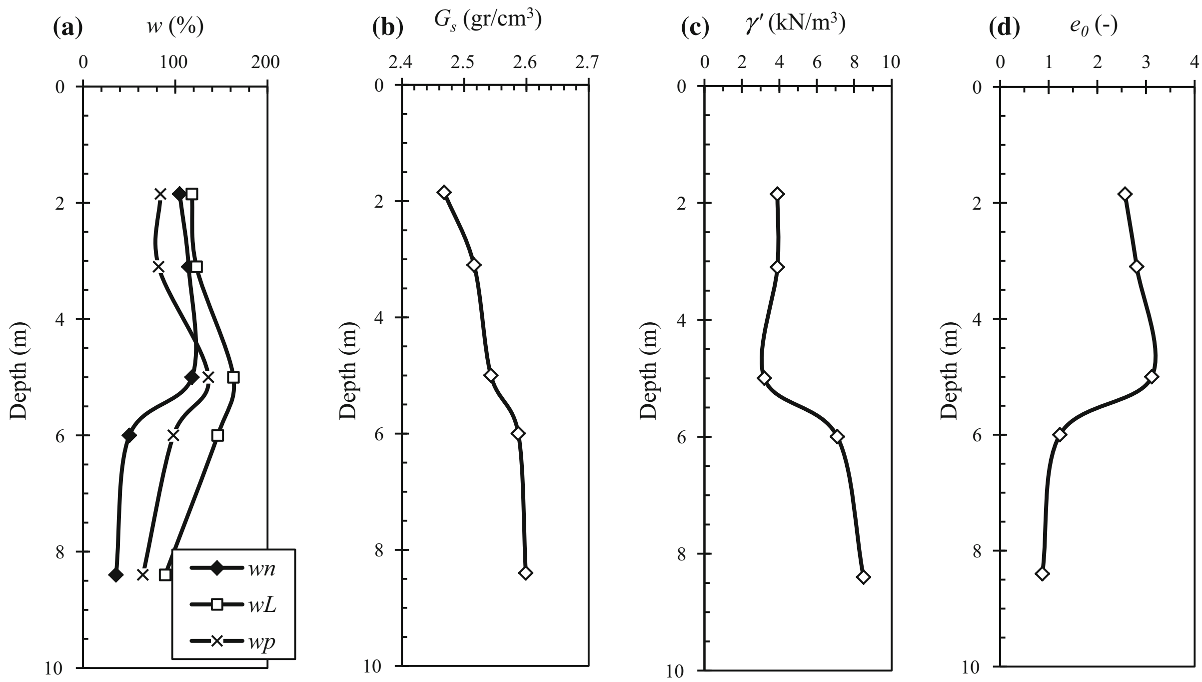

Fig. 4 Variations of the index properties along the soil profile: **a** natural water content; **b** specific grain density; **c** submerged unit weight; **d** initial void ratio

on which he overall assessment is performed based on the variation of the void ratio during oedometer consolidation tests using the following expressions:

$$\varepsilon_v = (e_0/(1 + e_0)) \times (\Delta e/e_0) \tag{1}$$

$$\Delta e = e_{(\sigma'_{v0})} - e_0 \tag{2}$$

where e_0 and $e_{(\sigma'_{v0})}$ correspond respectively to the values of the void ratio at the beginning of the oedometer test and at the in situ vertical effective stress level both obtained by the consolidation curves. Table 1 summarized the results of the sample quality assessment based on the consolidation tests data. It can be seen that all five soil samples show acceptable quality according to the criterion of Lunne et al. (1997). However, using Coutinho's (2007) criterion, three out of the five samples exhibit good quality and the other two samples having good to acceptable quality. These results indicate that the collected samples seem to be quite appropriate for determination of the geotechnical properties of the soft clay deposit.

3.3 Compressibility Parameters

The compressibility parameters obtained from oedometer consolidation tests are shown in Fig. 5, in which variations in ratio of swelling index to compressibility index C_s/C_c and the compression ratio $CR = C_c/(1 + e_0)$ are plotted along the depth. Based on Fig. 5a, b, soft clay I is characterised by an average values of C_s/C_c ratio equal to 0.1 and CR around 0.3, indicating to the high compressibility behaviour of this

Table 1 Sample quality assessment

Depth (m)	e_0	$e_{(\sigma'_{v0})}$	ε_{v0}	Lunne et al. (1997)	Coutinho (2007)
2.85–3.35 (SM1)	2.81	2.62	0.05	Acceptable	Good
5.75–6.25 (SM2)	1.23	1.11	0.06	Acceptable	Good to acceptable
8.15–8.65 (SM3)	0.87	0.79	0.06	Acceptable	Good to acceptable
1.60–2.10 (SM4)	2.57	2.37	0.05	Acceptable	Good
4.75–5.25 (SM5)	3.12	2.75	0.05	Acceptable	Good

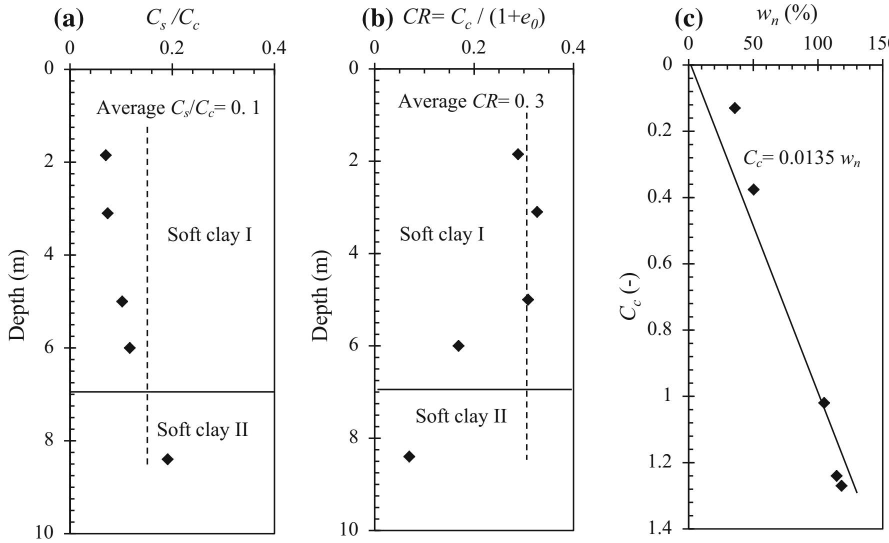

Fig. 5 Compressibility parameters along the depth: **a** relation of *Cs*/*Cc*; **b** compressibility ratio *CR*; **c** relation of *Cc* and w_n

layer. A significant improvement is observed in soft clay II as the average *CR* value reduces to 0.08, demonstrating soft clay II is less compressible than upper soft clay layer, as previously determined by SPT blows. Lacerda and Almeida (1995) have reported that the compression ratio *CR* of Brazilian soft clay varies from 0.2 to 0.4, which is in agreement with the values obtained from the present study. It is common to estimate the compressibility index (C_c) according to the natural water content (w_n) as a preliminary and low-cost result of site investigation. Based on the results shown in Fig. 5c, a linear relationship was found to correlate the compression index with natural water content. Futai et al. (2008) determined $C_c = 0.013w_n$ for Rio de Janeiro soft clays, quite similar to the $C_c = 0.0135w_n$ obtained for the soft clay studied.

The oedometer consolidation test was also used to determine the stress dependent oedometer modulus of the soft clay (E_{oed}). Figure 6a shows the variation of the oedometer modulus for soft clay I with increasing vertical effective stress. It can be seen that the oedometer modulus is about 500 kPa for the vertical stress levels <100 kPa though, at vertical effective stresses beyond 100 kPa, E_{oed} value increases linearly. The relationship between the two variables is represented by the best fit of the average values of E_{oed} at each vertical stress level for soft clay I.

Figure 6b also shows the variation of the coefficient of vertical permeability with the void ratio obtained by oedometer tests. This relationship is quite important for the estimation of the settlement rate and is particularly applicable to the numerical modelling of the embankment on soft clay, as the vertical permeability of the soil changes during consolidation. An increase in soil permeability with a corresponding increase in the void ratio is a well-known characteristic of soft soils, as is confirmed in Fig. 6b where a linear relationship, with the slope of $C_k = 1.03$, can be seen between coefficient of vertical permeability and void ratio values.

3.4 Coefficients of Consolidation

Any procedure for the determination of coefficient of horizontal consolidation (c_h) using piezocone test data requires an accurate estimation of the pore pressure values at the beginning of dissipation (u_i) and the values of the hydrostatic pore pressure (u_o) (Campanella et al. 1982; Campanella and Robertson 1988; Robertson et al. 1986; Schnaid et al. 1997; Danziger et al. 1997). According to the dissipation tests performed in the different depths of the soft clay layer, the coefficient of horizontal consolidation was estimated by following equation which takes into account the stiffness index (I_r) and the time factor, (T^*) (Houlsby and Teh 1988):

$$T^* = \frac{c_h \cdot t_{50}}{R^2\sqrt{I_r}} \tag{3}$$

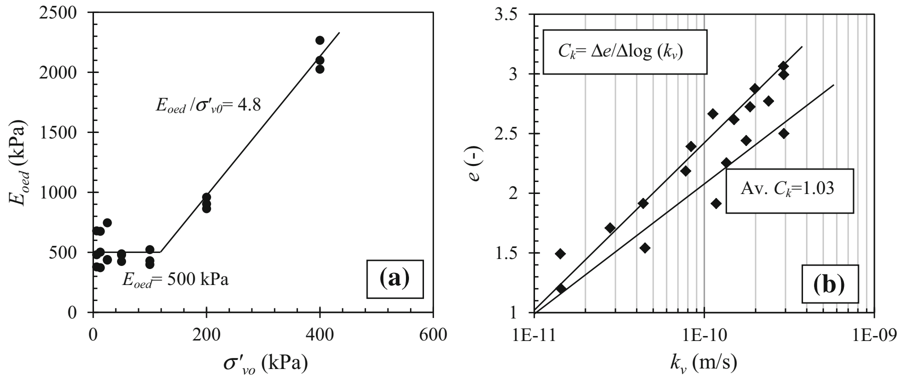

Fig. 6 Results of oedometer consolidation tests: **a** variations of the oedometer modulus versus in situ effective vertical stress; **b** variations of vertical permeability versus void ratio

where R radius of piezocone probe; t_{50} time of 50% of dissipation; and I_r rigidity index (G/S_u) equal to 100 for the present soft clay.

Values of the shear modulus G (=E_u/3) and undrained strength (S_u) were directly calculated from the consolidated undrained triaxial tests data, considering secant Young's modulus for 50% of the maximum deviatoric stress. The time factor was also computed according to the percentage of dissipation proposed by Houlsby and Teh (1988). Figures 7a presents the values of the coefficient of horizontal consolidation calculated using the dissipation tests with an average value equal to $c_h = 3.7 \times 10^{-7}$ m^2/s and $c_h = 2.3 \times 10^{-6}$ m^2/s for soft clay I and soft clay II, respectively. Lower c_h value obtained for soft clay I indicates to a softer behavior of this layer compared with soft clay II, as expected.

The coefficient of vertical consolidation (c_v) was computed from oedometer tests using the method developed by Taylor (1942). The coefficient of horizontal consolidation (c_h), estimated from the dissipation of excess pore water pressures via piezocone tests, corresponds to the properties of soil in the pre-consolidated range (Lunne et al. 1997; Schnaid 2009). According to the approach proposed by Jamiolkowski et al. (1985), it is possible to correct piezocone-estimated c_h values using:

$$c_{h(CR)} = \frac{C_s}{C_c} \cdot c_{h(piezocone)} \tag{4}$$

where the ratio of swelling index to compressibility index Cs/Cc for the present soft clay was found to be 0.1, as shown in Fig. 5a. After conversion of c_h to $c_{h(CR)}$, the corrected coefficient of vertical consolidation $c_{v(CR)}$ was determined using (Taylor 1942):

$$c_{v(CR)} = \frac{k_v}{k_h} \cdot c_{h(CR)} \tag{5}$$

where the ratio of horizontal permeability to vertical permeability was chosen to be 1.5 for lightly to normally consolidated clay (Jamiolkowski et al. 1985; Schnaid 2009; Almeida and Marques 2013). Figure 7b illustrates a comparison between the corrected coefficient of vertical consolidation determined by the oedometer tests and piezocone dissipation tests. It can be seen that c_v values for soft clay I calculated by oedometer tests vary from 4.8×10^{-7} to 9.4×10^{-8} m^2/s quite close to values computed by the piezocone dissipation tests.

4 Undrained Shear Strength

Clay undrained strength may be obtained by the piezocone test data using a number of equations (Lunne et al. 1997; Schnaid 2009). The most commonly-employed equations relate the corrected cone resistance (q_T) to the cone factor (N_{kt}), and pore pressure to the cone factor ($N_{\Delta u}$). Therefore, an accurate determination of the cone factor values is quite important for a precise estimation of undrained strength using piezocone test data.

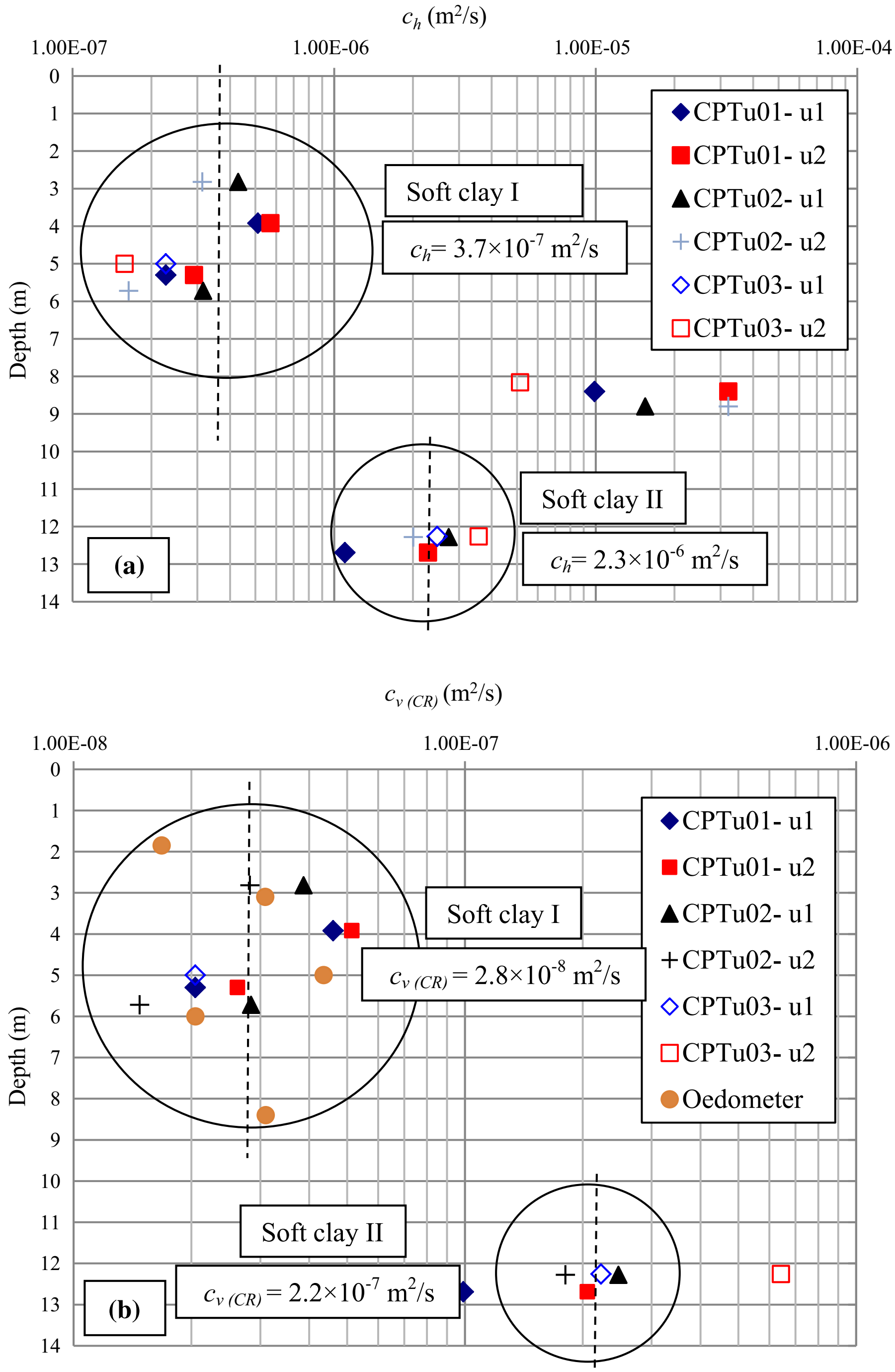

Fig. 7 Variations of coefficients of consolidation: **a** horizontal coefficient of consolidation; **b** corrected coefficient of vertical consolidation

4.1 Empirical Cone Factors

There are some empirical cone factors (N_{kt}, $N_{\Delta u}$, N_{ke}) proposed to determine the profile of the undrained shear strength from piezocone tests data (Robertson et al. 1986; Lunne et al. 1997; Danziger et al. 1997; Robertson and Cabal 2015). These empirical cone factors are estimated by relating the undrained shear strength obtained from the vane shear tests ($S_{u(VT)}$) with either corrected cone resistance (q_T) or pore pressure (u_2) both obtained from the piezocone tests. The estimated values of the empirical cone factors are then used to determine the undrained shear strength profile using piezocone tests data.

The mostly used empirical cone factor N_{kt} is determined by correlating piezocone and vane shear tests using the following expression:

$$N_{kt} = \frac{q_T - \sigma_{v0}}{S_{u(VT)}} \tag{6}$$

where σ_{v0} is the in situ total vertical stress in the same depth on which the vane shear strength is obtained. Figure 8 shows variation of N_{kt} cone factor with soil profile calculated using Eq. (6) and based on data provided from the piezocone (q_t) and vane shear tests ($S_{u(VT)}$) performed in the test area. According to Fig. 8b, the cone factor N_{kt} followed by a constant value equal to 13 in soft clay I, and then increased linearly with an estimated relationship of $N_{kt} = 6.5z$ in soft clay II. The average value of N_{kt} varies between 9 to 14 for Brazilian clays (Schnaid 2009; Almeida et al. 2010), which is similar to that obtained for soft clay I.

There are also two other empirical cone factors (i.e. $N_{\Delta u}$ and N_{ke}) determined by correlating the pore pressure measurements from the dissipation tests and the values of undrained shear strength by vane shear tests (Lunne et al. 1997; Robertson and Cabal 2015). These empirical cone factors are also used to determine the undrained strength profile using piezocone tests data.

The empirical cone factor $N_{\Delta u}$ takes into account variation in pore pressure (u_2) measured in piezocone test in relation to the hydrostatic pore pressure (u_0) through:

$$N_{\Delta u} = \frac{u_2 - u_0}{S_{u(VT)}} \tag{7}$$

However, the cone factor N_{ke} is determined based on the corrected cone resistance (q_T) and pore pressure (u_2) both from the piezocone test using:

$$N_{ke} = \frac{q_T - u_2}{S_{u(VT)}} \tag{8}$$

Figure 9 represents the variations in the empirical cone factors $N_{\Delta u}$ and N_{ke} calculated using Eqs. (7) and (8), respectively. It can be seen that values of $N_{\Delta u}$ and

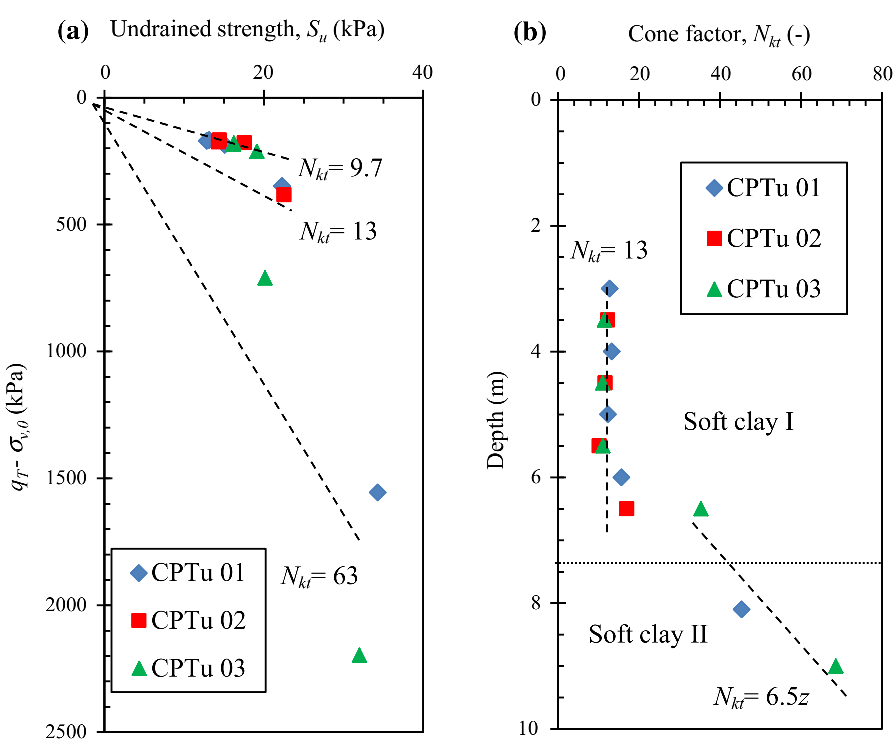

Fig. 8 Variation of cone factor N_{kt} based on vane and CPTu test correlation

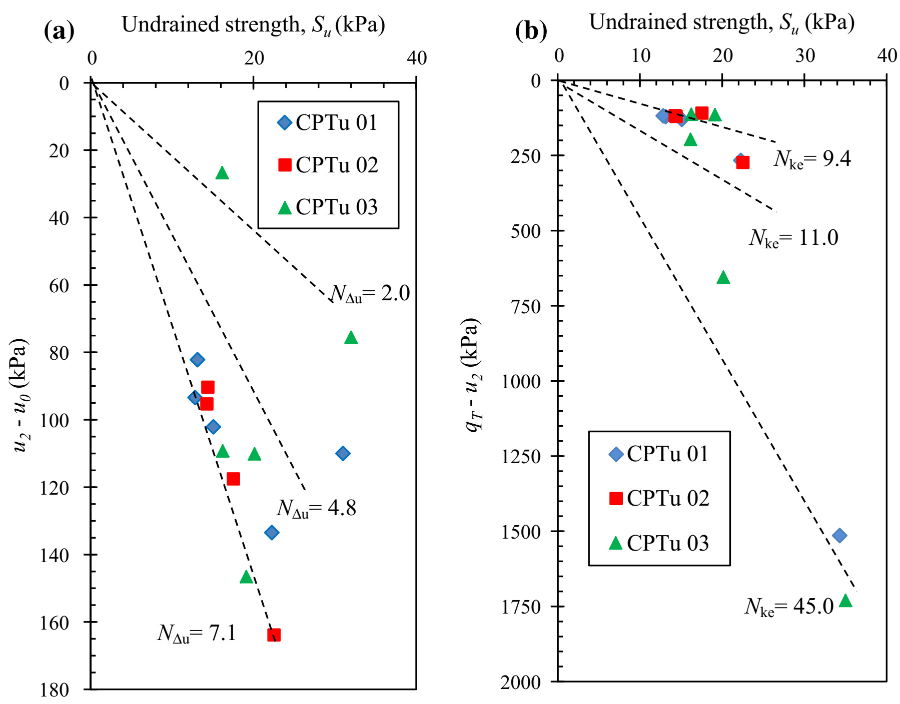

Fig. 9 Empirical cone factors determination: **a** variation of $N_{\Delta u}$; **b** variation of N_{ke}

N_{ke} range from 2.0 to 7.1 and 9.4 to 45, respectively, with average values of $N_{\Delta u} = 4.8$ and $N_{ke} = 11.0$. Robertson and Cabal (2015) have previously shown that the value of $N_{\Delta u}$ typically varies from 4 to 10 and that for a more conservative estimation, values near the upper limit should be chosen. The values of empirical cone factors, determined by correlation between piezocone and vane shear tests, were then used to compute the profile of the undrained shear strength as described below.

4.2 Profile of Undrained Strength

Profile of the clay undrained strength (S_u) was obtained by the results of vane shear test correlated with piezocone tests data and using the corresponding empirical cone factors determined earlier. Undrained strength was also estimated using a mathematical derivation proposed by Mantaras et al. (2015) which takes into account a number of critical factors affecting the S_u values including soil stress history, clay stiffness, and excess pore pressure during piezocone tests. The method of Mantaras et al. (2015) uses the following equation to determine the undrained shear strength:

$$S_u = \frac{\Delta u_{\max}}{4.2 \log I_r} \tag{9}$$

where $\Delta u_{\max} = u_{2,\max} - u_0$, in which $u_{2,max}$ is excess pore pressure measured in cone tip and u_0 is the hydrostatic pore pressure. The data provided by the three piezocone tests performed in the test area was used to estimate the undrained strength using the Eq. (9).

Figure 10a compares the profile of the clay undrained strength estimated from the method of Mantaras et al. (2015) and those obtained directly by vane and piezocone tests data. As expected the undrained strength increased with the soil profile. It is seen that the method of Mantaras et al. (2015) predicted fairly well the undrained strength obtained by vane and piezocone tests. It can be observed that the S_u values are <20 kPa in upper soft clay layer as expected for very soft soils. However, values of S_u in soft clay II increase to about 40 kPa, indicating this layer is less soft than soft clay I.

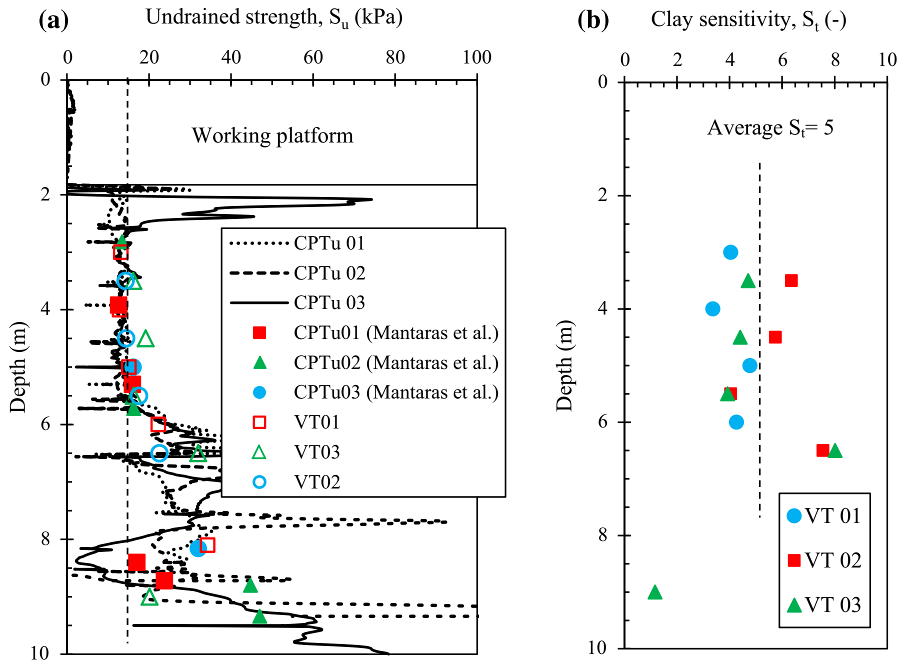

Fig. 10 Profile variation in: **a** undrained shear strength; **b** clay sensitivity (by vane test)

Clay sensitivity (S_t), defined as the ratio of peak strength (S_u) to remoulded strength (S_{ur}), was examined using vane test data. Figure 10b shows the variation in clay sensitivity according to vane tests performed on undisturbed and remoulded clay. The tests produced an average clay sensitivity value of $S_t = 5$; according to Mitchell and Soga (2005) the studied soft clay can thus be classified as a medium sensitive to sensitive clay. Most Brazilian clays have sensitivities in the 1–8 range, with average values of between 3 and 5 (Schnaid 2009).

Both undrained strength and strength properties (ϕ' and c') were determined using a consolidated undrained triaxial test (CU) with accompanying pore-pressure measurement. In the case of the anisotropic consolidation test (CAU), the effective in situ vertical (σ'_{v0}) and horizontal (σ'_{h0}) stresses were estimated beforehand. The earth pressure coefficient for CAU tests ($K_{0(OC)}$) was determined using:

$$K_{0(OC)} = (1 - \sin\phi') \cdot OCR^{\sin\phi'} \quad (10)$$

where both OCR and the drained friction angle (ϕ') were previously estimated via oedometer and isotropic consolidation (CIU) tests, respectively. In the present study, the CIU tests were conducted at two different confining pressures of $\sigma_3 = 100$ and 200 kPa, while the confining pressures for the CAU tests ranged from $\sigma_3 = 18$ to 40 kPa, determined according to the depth of sampling. Figure 11 shows the stress paths determined by CIU and CAU tests carried out at different consolidation pressures. The upper and lower strength envelopes were obtained from values of deviator stress $q = ((\sigma'_1 - \sigma'_3)/2)$ plotted against mean effective stress $p' = ((\sigma'_1 + \sigma'_3)/2)$. According to the failure envelops, the average drained friction angle and effective cohesion are of $\phi' = 27°$ and $c' = 3.2$ kPa, respectively. It is also observed that the values of undrained shear strength S_u obtained from CAU tests

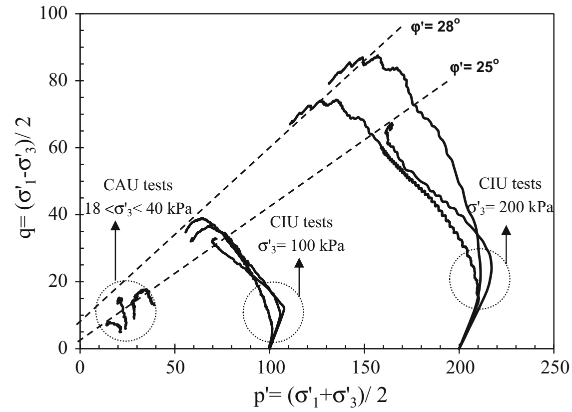

Fig. 11 Stress paths and failure envelopes of the soft clay studied

are lower than 20 kPa quite close to the S_u values determined by piezocone tests shown in Fig. 10a.

5 Stress History

A careful determination of pre-consolidation stress (σ'_{vm}) is particularly important for a precise estimation of the settlement of embankments over soft clay deposit. The pre-consolidation stresses were directly determined using oedometer consolidation tests data with values ranging between 41 and 65 kPa, close to those of in situ effective stress (σ'_{v0}). Figure 12a shows the variation of $OCR = \sigma'_{vm}/\sigma'_{v0}$ with depth, according to consolidation tests performed on five specimens. It can be observed that the present soft clay is lightly over-consolidated in the upper 4 m, with OCR values ranging from 1.1 to 1.6. Chen and Mayne (1996) proposed a number of additional equations to estimate OCR profile. These equations were developed based on more than 1200 piezocone tests and are widely used as follows:

$$OCR = C1\left[\frac{q_T - \sigma_{v0}}{\sigma'_{v0}}\right] \quad (11)$$

$$OCR = C2\left[\frac{q_T - u_1}{\sigma'_{v0}}\right] \quad (12)$$

$$OCR = C3\left[\frac{q_T - u_2}{\sigma'_{v0}}\right] \quad (13)$$

Values of the constants $C1 = 0.305$, $C2 = 0.75$ and $C3 = 0.53$ were used in the original proposal of Chen and Mayne (1996). Figure 12b shows profile of the OCR calculated using Chen and Mayne's equations compared with the values obtained via oedometer tests. It is noted that CPTu01 data were used for the estimation of this OCR profile. A reasonable agreement with the oedometer test was obtained by multiplying the above constants in 0.55, and thus the following were used: $C1 = 0.167$, $C2 = 0.412$ and $C3 = 0.291$. Baroni and Almeida (2012) similarly found that dividing the original constants by two resulted in a proper agreement regarding OCR estimation.

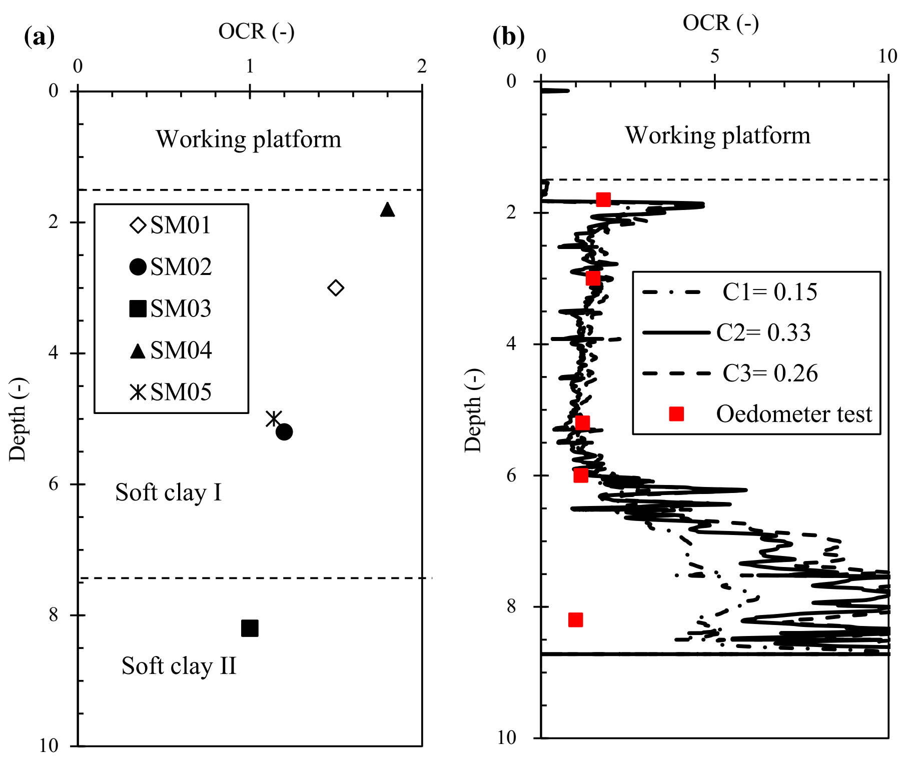

Fig. 12 Variation of OCR along the depth: **a** OCR values obtained by oedometer tests; **b** estimated OCR values using Chen and Mayne's (1996) equations

6 Use of Soft Clay Parameters

The soft clay properties, determined by site investigation, were used in numerical and analytical computations of a trial embankment on treated soft ground, with the objective being to assess their reliability for future practical applications. Data of a trial embankment constructed on geosynthetic-encased columns was used in numerical simulation (Almeida et al. 2015). As can be seen in Fig. 13a, an 18 m wide and 5.30 m high test embankment was constructed on 36 encased columns spaced an average of 2.0 m apart. The reinforced ground was instrumented aiming to measure the settlement, total vertical stresses, excess pore pressures, column radial deformation, and soil horizontal displacement. In the present study the results provided by settlement plates SP1, SP2 and pressure cells CP1 and CP2 were used to verify the simulated results.

Numerical analysis was performed via finite element code PLAXIS 2D (Brinkgreve and Vermeer 2012) using the axisymmetric unit cell approach in which half of a central encased granular column and its surrounding soft soil was modeld (see Fig. 13b). The behavior of the soft clay layers were simulated using the Modified Cam-Clay material model introduced as Soft Soil model in PLAXIS. The linear-elastic, perfectly plastic model with the Mohr–Coulomb failure criterion was used to model the granular materials. The sand properties were correlated by SPT tests data as well as those suggested by Briaud (2013); while the material properties of working platform and embankment fill were determined through the direct shear test performed on corresponding material. The properties of the crushed stone material were not measured in the present study and have been adopted based on the well-established literatures on encased granular columns (e.g. Murugesan and Rajagopal 2006; Yoo 2010). Table 2 shows the material properties used in numerical simulation of the test embankment. The geosynthetic encasement was assumed to be an isotropic elastic material with a tensile stiffness of $J_{en} = 1750$ kN/m. The load application was simulated by actual stage construction of the test embankment followed by six months consolidation period.

Figure 14a shows a comparison of the settlement measured on top of the soft soil and encased column with the values computed numerically (FEA). It can be seen that the FEA results appear to match the trend of the field data fairly well. Both the measured and simulated settlements increased noticeably during load application and then followed by continuous increasing during consolidation period. The difference of settlements between top of the encased column and surrounding soil was also suitably simulated resulting

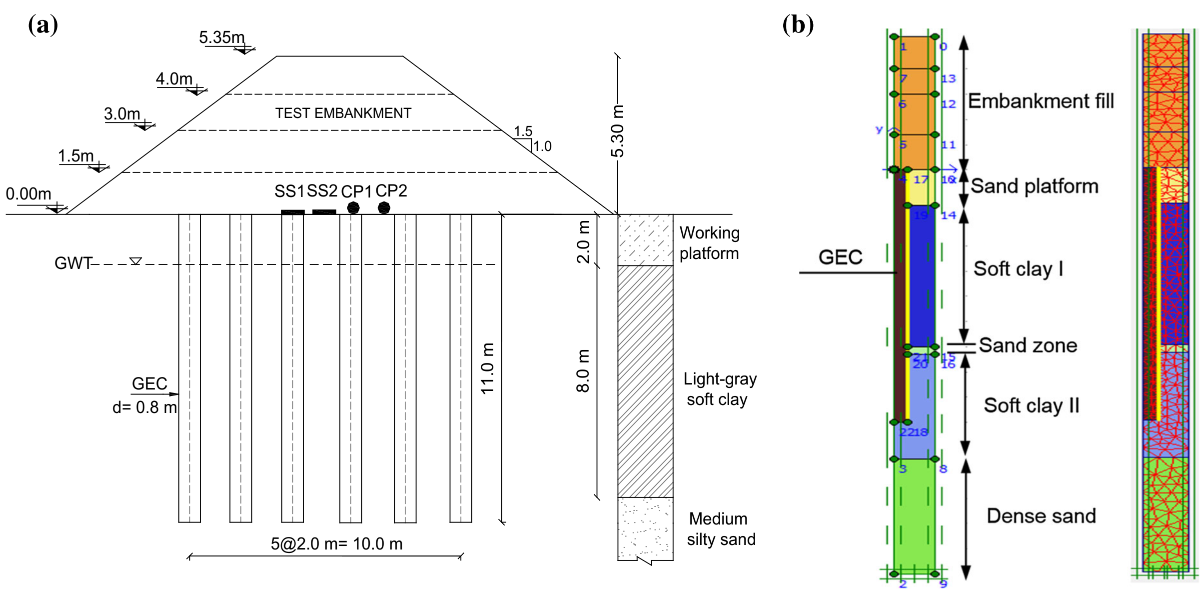

Fig. 13 Case considered for model validation: **a** cross sectional view of the test embankment; **b** axisymmetric model adopted for numerical analysis

Table 2 Material parameters used in numerical analysis of the test embankment

Material and constitutive model	γ_{sat} (kN/m^3)	k_h (m/day)	k_v (m/day)	γ' (°)	c′ (kPa)	E′ (kPa)	C_c (–)	C_s (–)	C_k (–)	OCR (–)
Embankment fill (MC)	28	1	1	45	0	53,000	–	–	–	–
Granular column (MC)	20	10	10	40	0	80,000	–	–	–	–
Soft clay I (SS)	14.4	1.6×10^{-5}	5.2×10^{-6}	26	4	–	0.98	0.084	1.03	1.35
Soft clay II (SS)	16.8	9.7×10^{-6}	4.8×10^{-6}	28	6	–	0.13	0.025	1.03	1.04
Working platform (MC)	19.7	0.6	0.6	33	3	12,000	–	–	–	–
Dense sand (MC)	20	1	1	38	0	30,000	–	–	–	–
Sand lens (MC)	18.5	0.5	0.5	30	0	22,000	–	–	–	–

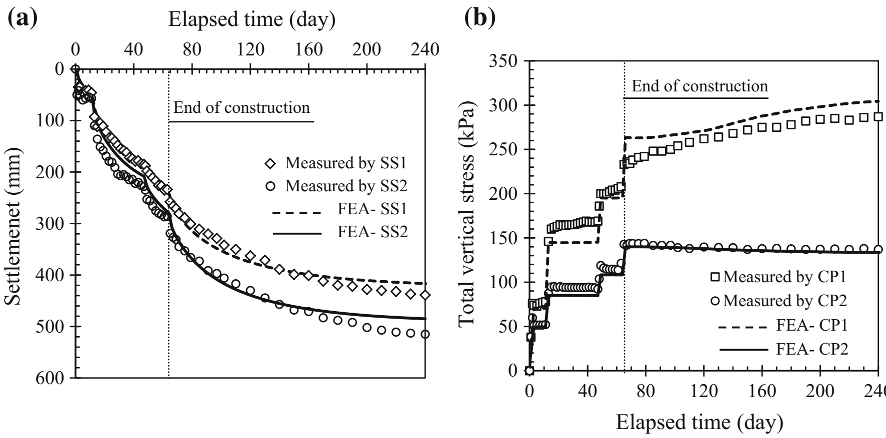

Fig. 14 Comparison of measured and simulated results: **a** settlement below the embankment versus time; **b** total vertical stress versus time

Table 3 Numerical and analytical values compared with measured data

Variable	Calculation method		Measurement
	Axi-symmetric Finite Element Analysis	Analytical Method, Raithel and Kempfert (2000)	
Settlement on surrounding soil (mm)	490	595	525
Differential settlement below the embankment (mm)	80	NA	100
Maximum soil horizontal displacement (mm)	NA	NA	160
Excess pore pressure stabilization (days)	210	NA	240
Vertical stress on column (kPa)	304	1100	287
Vertical stress on soil (kPa)	135	30	137
Maximum column bulging (mm)	16	32	18

NA Not available

to around 80 mm of differential settlement at the end of monitoring time (i.e. 240 days). Vertical stresses transmitted to the encased column and surrounding soil are also compared in Fig. 14b. In general a good agreement is observed between measured and numerical values. It is observed that the simulated total vertical stress on encased column was about 2.5 times greater than that on the surrounding soil, both calculated at the end of monitoring period, which is quite close to the measured value equal to 2.3.

Numerical calculations were also carried out for other measured variables (Hosseinpour et al. 2015), as shown in Table 3, and good overall agreement with measured values is generally observed. Table 3 presents also analytical computations using Raithel and Kempfert (2000)'s method. The most relevant parameters used in the analytical method are the column friction angle, encasement stiffness, soft soil oedometer modulus, and Poisson's ratio values.

It is observed in Table 3 that the analytical method was quite satisfactory in predicting absolute settlements, but on the other hand presented a number of limitations. These limitations result mainly from the fact that the analytical method does not consider the presence of the working platform, thus its corresponding confining behaviour; and also that the computed final settlement are assumed equal for column and soil around columns.

The comparisons presented in Fig. 14 and Table 3 clearly indicate the reliability of the soft clay properties determined by site investigation, thus allowing to predict properly the performance of the treated soft ground subjected to embankment loading.

7 Conclusions

Geotechnical parameters of a typical Brazilian soft clay were obtained through extensive in situ and laboratory tests and then used in the numerical and analytical computations of the stabilized ground beneath the embankment. The main results of this study are summarised below:

1. Oedometer tests showed that the soft clay studied has a high compressibility index, with an average compression ratio (*CR*) of 0.3. Consequently, large settlement and horizontal displacement must be considered in any design approach. In addition, a linear relationship was found for oedometer modulus variation with increasing vertical stress.
2. Undrained strength data obtained by a computational method exhibited good agreement with those measured by vane, piezocone and triaxial tests. Low values of S_u (<20 kPa) indicating a very soft behaviour, particularly in up 7.0 m depth where soft clay I exists.
3. Empirical cone factors were determined by correlating vane and piezocone test data. An average value of $N_{kt} = 13$ was obtained for $z < 6.5$ m, while a linear relationship of $N_{kt} = 6.5z$ was found for depths greater than 6.5 m. The average values of the cone factors N_{ke} and $N_{\Delta u}$ were 11 and 4.8, respectively.
4. Over-consolidation values were obtained by consolidation tests as well as the equations proposed by Chen and Mayne (1996). The results showed a good agreement in terms of the OCR profiles when the equation constants were divided by two, as suggested by Baroni and Almeida (2012) for Rio de Janeiro soft clay.
5. The validity of the clay properties obtained based on the field and laboratory tests was evaluated by numerical and analytical computations of a trial embankment on a treated soft deposit. Numerical results were compared with field data, with the two sets of figures showing reasonable agreement in terms of settlement and vertical stresses. The analytical method was quite satisfactory in predicting final stabilized settlements, but not for the other measured values. Therefore, it can be stated that the soil properties obtained through site investigation were quite reliable to be used in computational methods for prediction of the behaviour of the reinforced soft ground subjected to the embankment loading.

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