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Site investigation and performance of radial deep consolidation grouting in soft soil

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This paper presents the site investigation and ground stiffening carried out in an 8 m thick, soft soil deposit located in the western zone of Rio de Janeiro, Brazil. The stiffening was achieved through the use of pre-fabricated vertical drains and radial deep consolidation grouting, a form of compaction grouting. In this study, the results of the site investigation, comprising laboratory and in situ tests, are presented and discussed. Soft soil parameters were used to compute the settlement–time curve considering vertical drains without grout injection. This was compared with the settlement–time field curve after grouting, obtained using data from settlement plates monitored for a period of 147 d. Comparison of the two sets of data enabled an improvement factor to be determined, thus quantifying the efficiency of the grouting. Field measurements, provided by settlement plates, were extrapolated to a long-term condition. Also, the final settlement after grouting treatment was evaluated.

Notation

A	transversal area of unit cell
C	empirical constant for cone equations (E_{oed} determination)
C_1	empirical constant for cone equations (overconsolidation ratio (OCR) determination)
C_2	empirical constant for cone equations (OCR determination)
C_3	empirical constant for cone equations (OCR determination)
CR	compression coefficient
c_c	compression index
c_h	coefficient of horizontal consolidation
c_h(NA)	c_h determined from cone penetration testing with pore pressure measurement (CPTu) for normal consolidation range
c_h(PZ)	c_h determined from CPTu for overconsolidation range
c_s	swelling index
c_v	coefficient of vertical consolidation
c_v(NA)	c_v for normal consolidation range
d_e	influence diameter of the drain
$E_{oed,eq}$	equivalent oedometer modulus of clay
$E_{oed,s}$	oedometer modulus of clay
E_u	undrained Young's modulus of soil
e_0	initial void ratio
$F(n)$	drain density function
G	shear modulus of soil
H_d	shorter vertical drainage path in the soft soil layer
h	vertical spacing between bulbs
h_{crit}	critical height of embankment
h_i	sub-layer thickness
I_P	plasticity index
I_r	stiffness index of clay
K	coefficient of lateral earth pressure
K_0	coefficient of lateral earth pressure at rest
$K_{0,NC}$	coefficient of lateral earth pressure for normally consolidated condition
k_h	coefficient of horizontal permeability
k_v	coefficient of vertical permeability
M	Cam-clay parameter (slope of critical state line)
N	specific volume correspondent to the unitary average effective stress on isotropic line
N_b	number of bulbs in a vertical of *consolidação profunda radial* (in Portuguese) (CPR) grouting
N_{ke}	empirical cone factor
N_{kt}	empirical cone factor
N_{spt}	number of standard penetration test (SPT) blows
$N_{\Delta U}$	empirical cone factor
n	exponent dependent on kind of soil
p'	average effective stress after CPR application
p'_p	parameter of the equivalent homogeneous medium method
p'_y	parameter of the equivalent homogeneous medium method
p'_0	average effective stress before CPR application
q_T	corrected cone tip resistance
R	radius of CPTu cone
R_p	parameter of the equivalent homogeneous medium method
RR	expansion coefficient
R_s	substitution ratio
r	$r = c_h/c_v$ (parameter of Asaoka (1978) method)
S	horizontal spacing between bulbs

$S(t)$	settlement at given time
S_t	sensitivity of the clay
S_u	undisturbed undrained shear strength of clay
S_{ur}	remoulded undrained shear strength of clay
S_0	initial degree of saturation
S_∞	final settlement at end of consolidation
$S_{\infty,\text{soil treated}}$	final settlement after soil stiffening (treatment)
T	time factor
t	dissipation time
$t_{50\%}$	50% of the dissipation time
U	percentage of global consolidation
U_h	percentage of horizontal consolidation due to radial flux
U_v	percentage of consolidation due to vertical flux
u_0	hydrostatic pore pressure
u_1	pore pressure measured at cone tip
u_2	pore pressure measured at the cone base
Vg_i	volume of expanded bulb i
w	water content
w_L	liquid limit
w_P	plastic limit
w_0	initial water content
β	improvement factor
β_1	slope of line formed by points $S(t)n$ and $S(t)n+1$ in the Asaoka method
γ	total unit weight of clay
γ_{emb}	total unit weight of soil embankment
γ_s	unit weight of solids
Δt	time interval (Asaoka method)
Δu_{max}	maximum excess of pore pressure generated
$\Delta\sigma_v$	increase in vertical stress due to loading
η	stress ratio ($=q/p'$, based on Cam-clay model)
κ	Cam-clay parameter ($=c_s/2{\cdot}3$)
λ	Cam-clay parameter ($=c_c/2{\cdot}3$)
λ_c	coefficient of reduction volume due to consolidation
σ_{v0}	initial total vertical stress
σ'_v	effective vertical stress
σ'_{v0}	initial effective vertical stress
ϕ'_{cv}	friction angle at constant volume

1. Introduction

A number of soft ground-improvement techniques have been used in Brazil, such as: stone columns, granular encased columns, deep soil mixing, vacuum consolidation, pre-fabricated vertical drains (PVDs), stabilising berms, geosynthetic reinforcement, light-fill materials and others (Almeida *et al*., 2010).

A recent technique, combining vertical drains and grout injections and termed radial deep consolidation grouting, has been introduced in Brazil in the last decade (Almeida and Riccio, 2012). Radial deep consolidation (*consolidação profunda radial* (CPR) in Portuguese) may be considered to be a variation of compaction grouting, introduced in the 1950s for improving silty and sandy soils (Brown and Warner, 1973; Graf, 1969). Compaction grouting procedures and applications were presented by Byle (2000) and Warner (2004). The basic principle of CPR grouting, such as the controlled modulus columns (CMC) technique, is the injection of grouting into the soil to improve its bearing capacity. However, in the CMC technique, the injections produce a controlled pile in terms of shape, that is a cylindrical form. On the contrary, in CPR grouting, the injections produce bulbs without a controlled shape. The injection process is very similar to compaction grouting.

CPR grouting was intended to be used for soft clay improvement, injecting a low-slump grout that displaces and compresses the soil within a pre-installed vertical drain array. Drains ensure good drainage, facilitating the dissipation of grouting-induced pore pressures and allowing faster consolidation rates. As a result, the soil becomes consolidated, becoming stiffer and stronger. Kovacevic *et al*. (2000) investigated the effect of the development of pore pressure on the efficiency of compaction grouting. This topic was later discussed by Jefferies and Shuttle (2002). Riccio *et al*. (2013) measured the settlement of an embankment constructed in Rio de Janeiro, on soft ground that had been improved by CPR grouting. According to these studies, the overall performance of CPR grouting was reasonably good. Soga *et al*. (2004) discussed the factors that affect the efficiency of a multiple grout injection process. The main factors considered were the overconsolidation ratio (OCR) and the time and space separation of the injections.

In this study, a soft surface soil layer (that was up to 8 m deep) was improved with CPR grouting and then an embankment was constructed to a height of 2 m. The embankment was monitored with settlement plates for approximately 150 d, in order to assess the efficiency of the soil improvement. Final settlement was extrapolated from field observations according to the Asaoka (1978) method, modified by Magnan and Deroy (1980). The results were compared with predictions made by the equivalent homogeneous medium method proposed by Cirone (2016). The method proposed by Asaoka (1978) was conceived to interpret field measurements of settlement against time. The method allows the determination of the coefficient of vertical consolidation (c_v) and the coefficient of horizontal consolidation (c_h). Thus, it is possible to construct the settlement against time curve before the end of the entire consolidation process and obtain the final settlement according to real field conditions. The modified Asaoka method is an improved version of the Asaoka method. The equivalent homogeneous medium method is a new procedure to evaluate the final settlement of an embankment over foundations stiffened by the CPR grouting technique. The method considers average ('equivalent') material properties of the constituent materials (stiffened soil and grout bulbs) to calculate the final settlement of the improved ground.

A thorough site investigation (performed before the application of CPR grouting) was carried out using the cluster concept. The site investigation consisted of laboratory tests (index and oedometer tests) and in situ tests (CPTu, shear vane, standard penetration test (SPT) and Shelby sampling).

The site investigation provided information to determine the settlement against time curve using the Barron (1948) theory in a hypothetical case, in which only PVDs were considered. This is a well-known, traditional technique used to accelerate settlement.

The efficiency of CPR grouting was evaluated by comparing the predicted and observed settlement against time curves, for soft soil with and without improvement. The reported results are an important way to verify the readability of the equivalent homogeneous medium method applied in a real case of an embankment over treated soft soil. The capability of the new method to predict the final settlement of an embankment over soft soil after being stiffened using the CPR grouting technique, was verified. By comparing the CPR grouting technique (grout injection + PVDs) with the traditional technique (that employs only PVDs) it is possible to highlight some cost–benefit advantages of the former, such as: a decrease in the time required to fully consolidate, a decrease in the final settlement and an improvement in the bearing capacity of the foundation against failure; in other words, there is no requirement for the use of berms to prevent failure.

2. Site description and investigation

This study was carried out in the western region of Rio de Janeiro (Brazil), more specifically in the Recreio dos Bandeirantes district, a place well known for the presence of heterogeneous soft soil deposits including sand layers and sand lenses (Almeida *et al.*, 2008). The study area covered 21 000 m^2. A total of 24 SPT tests were carried out, as well as two site investigation clusters (SPT, vane, CPTu and Shelby sampling). In this study, only data collected in cluster I01, located near SPT 04, were used for geotechnical characterisation. Figure 1 shows the position of the SPTs, the two clusters (I01 and I02) and the CPR grouting array.

Results from SPT boreholes (e.g. the cross-section BB in Figure 1) show a soil profile consisting of a soft, upper soil layer ($N_{spt} \approx 0$) which was up to 8 m deep, underlain by a sand layer (3 m thick) (N_{spt} varying from 8 to 23). At greater depths, a layer of medium stiff clay was encountered (average $N_{spt} = 11$).

The SPT tests were carried out following the Brazilian standard NBR 6484 (ABNT, 1980) and reached a depth of approximately 34·0 m. Vane tests were used to determine the undrained shear strength of soft soil (S_u) and were conducted in accordance with the Brazilian standard NBR 10905 (ABNT, 1989). Vane tests were performed at the following depths: 0·5, 2·0, 3·5, 4·0, 5·5 and 6·0 m. Sampling procedure followed the recommendations of Ladd and De Groot (2003) and Brazilian standard NBR-9820 (ABNT, 1997). Using Shelby tubes with a 4 in. dia. and a stationary piston sampler, undisturbed soft soil samples were collected at the following depths: 1·25 m (sample S1), 3·25 m (sample S2), 5·25 m (sample S3) and 7·25 m (sample S4). The CPTu tests reached a depth of 8·5 m. Penetration was interrupted to perform four dissipation tests at depths of 1·0, 3·0, 5·0 and 7·0 m. Three settlement plates, designated PR2, PR3 and PR4, were installed near the cluster I01.

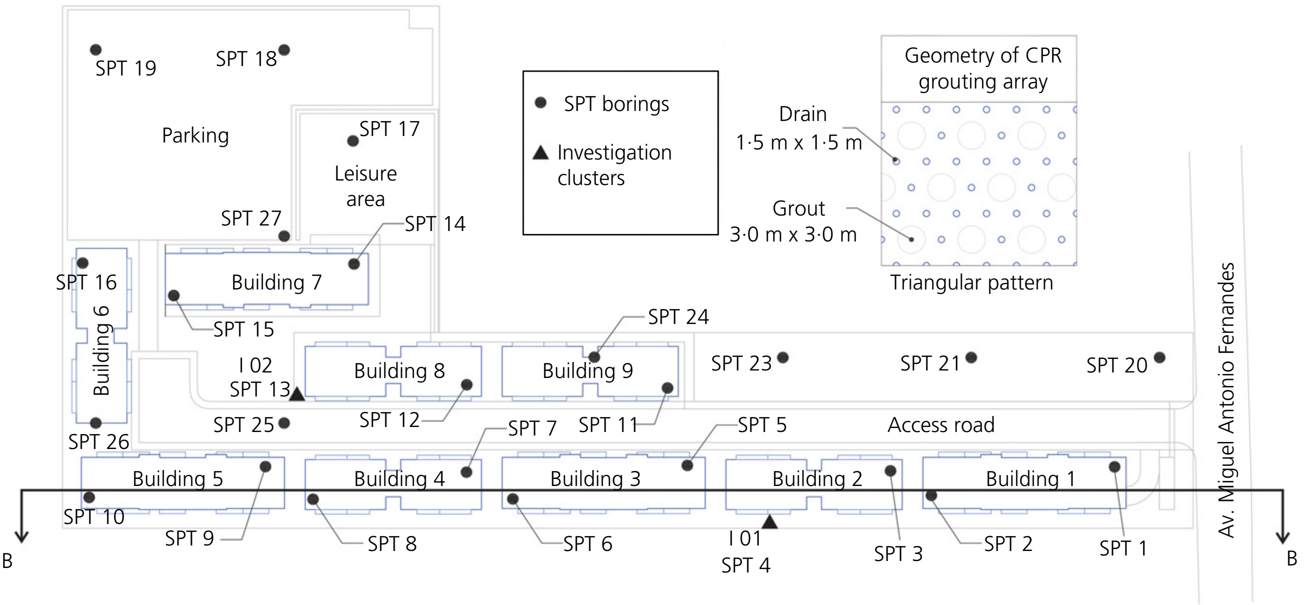

Figure 1. Site layout and site investigation information

CPR grouting was performed according to a triangular pattern, forming successive injected grout bulbs with a diameter approximately equal to 1·2 m. Grout injection points were spaced 3·0 m apart and drains were installed in a triangular pattern, with 1·5 m spacing. Figure 2 shows the soil profile (before the application of CPR grouting) and the idealisation of the bulbs and drains along the depth.

3. Results of site investigation

3.1 Laboratory tests

Laboratory tests were carried out on specimens collected by Shelby samplers. The laboratory experimental programme included full index tests (grain-size distribution, Atterberg limits, moisture content and specific gravity of soil particles) and oedometer tests. Table 1 summarises the index test results.

From Table 1, it is possible to observe that the upper layer (sample 1) is a peaty material. The soil has a moisture content (w) equal to 826·5%, which is much higher than the other samples. The density value of solids is also lower than typical values for clay. The initial void ratio (e_0) is almost five times the void ratio of the other layers. Samples S2, S3 and S4 have

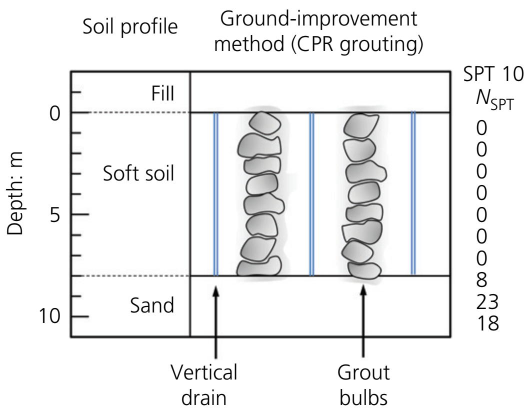

Figure 2. Soil profile (before CPR grouting application) and the idealisation of bulbs and drains along depth

a high clay percentage. Sample S3 has as much as 46·5% clay content. Under these circumstances, soil behaviour is governed by the finest particles. Table 2 gives the results obtained from oedometer testing.

The top layer (sample 1) has the highest compressibility and permeability. Samples S2, S3 and S4 display a more homogeneous behaviour in terms of the coefficients c_c, c_s, CR and RR. These layers are less compressible than the top one. Compression ratios, CR, are in accordance with the published data for Rio de Janeiro clay (Almeida *et al.*, 2008). The final settlement of the unimproved ground was calculated considering the oedometer modulus obtained in the oedometer tests.

3.2 In situ tests

The field tests programme included SPT, CPTu with dissipations and shear vane tests. The undrained strength (S_u) was determined by CPTu tests using the empirical cone factors N_{kt}, $N_{\Delta U}$ and N_{ke}. The values of S_u were also determined by dissipation tests using CPTu, according to the Mantaras *et al.* (2014) proposal (Equation 1). The values of the empirical cone factors (N_{kt}, $N_{\Delta u}$, N_{ke}) were obtained by the correlation of q_T, σ_{v0}, u_2 and u_0 with vane test data (S_u measured in situ), in accordance with the Lunne *et al.* (1997) approach. The values obtained through this procedure were: $N_{kt} = 10{\cdot}7$, $N_{\Delta U} = 3{\cdot}7$ and $N_{ke} = 8{\cdot}6$. Baroni and Almeida (2012) compared three different sites located in the western zone of Rio de Janeiro and obtained an average value of $N_{kt} = 12$. Additional data for other sites in Rio de Janeiro are provided by Baroni and Almeida (2017).

Equation 1 is based on the principles of cavity expansion and critical state soil mechanics and is presented here in its simplified form. The depths of the dissipation tests performed were 1·0, 3·0, 5·0 and 7·0 m.

$$1. \quad S_u = \frac{\Delta u_{max}}{4{\cdot}2(\pm 0{\cdot}2)\log_{10}(I_r)}$$

Table 1. Properties of soft soil obtained from samples S1, S2, S3 and S4

Depth/Property	Sample S1	Sample S2	Sample S3	Sample S4
Depth: m	1·25	3·25	5·25	7·25
w_0: %	826·5	148·4	134·5	115·9
γ: kN/m^3	9·81	12·81	13·09	12·71
e_0: dimensionless	15·61	3·71	3·25	2·75
S_0: %	93	97	98	93
γ_s: gf/cm^3	1·75	2·43	2·37	2·21
w_L: %	—	59·7	57·9	57·8
w_P: %	—	31·3	30·2	31·8
I_P: %	—	28·4	27·7	26·0
Clay/silt/sand: %	—	44·9/23·8/31·3	46·5/24·7/28·8	34·3/20·5/45·2

Note: γ is the unit weight of soil; γ_s is the unit weight of solids. Grain-size distribution based on Brazilian standard (NBR 7181 (ABNT, 2016))

Table 2. Parameters of soft soil from oedometer tests – samples S1, S2, S3 and S4

Depth/Property	Sample S1	Sample S2	Sample S3	Sample S4
Depth: m	1·25	3·25	5·25	7·25
c_c: dimensionless	6·86	1·57	1·18	1·32
c_s/c_c: dimensionless	0·089	0·079	0·117	0·094
CR: dimensionless	0·41	0·33	0·28	0·35
RR: dimensionless	0·037	0·027	0·032	0·033
c_v: m^2/s	$1{\cdot}15 \times 10^{-7}$	$2{\cdot}00 \times 10^{-8}$	$7{\cdot}00 \times 10^{-8}$	$3{\cdot}10 \times 10^{-8}$
$E_{oed,s}$: kPa[a]	161	288	1309	286

Note: CR = (c_c/1 + e_0); RR = (c_s/1 + e_0)
[a]$E_{oed,s}$ from oedometer tests for vertical effective stress varying from 50 to 100 kPa

where Δu_{max} is the difference between the maximum excess of pore pressure and the hydrostatic pore pressure, I_r the stiffness index of the clay, numerically equal to $G/S_u = E_u/3S_u$, observing that G is the shear modulus of the soil and E_u is the undrained Young's modulus of the soil. In this research study, the value of I_r was considered to be equal to 100, which is a typical value for Rio de Janeiro clay (Lacerda and Almeida, 1995; Mantaras *et al.*, 2014).

Figure 3(a) shows the results of S_u provided by the CPTu (using empirical factors of cone and dissipation tests) and vane tests. Vane tests were also performed to determine the sensitivity of the clay ($S_t = S_u/S_{ur}$) as shown in Figure 3(b), where S_u is the undisturbed undrained shear strength and S_{ur} is the remoulded, undrained shear strength of the clay.

From Figure 3(a) it can be seen that Equation 1 (using I_r equal to 100 and values of Δu_{max} provided by the CPTu dissipation tests) leads to good agreement with the vane test data. Also, the profile of S_u which was evaluated using cone factors (N_{kt}, $N_{\Delta u}$ and N_{ke}) also presents very good agreement with the vane test data; it can be observed that the best fit was achieved, in the present case, when N_{ke} was used. The S_u profile shows a very low undrained strength to a depth of 5·0 m (<7·5 kPa). Below 5·0 m depth, the values of S_u vary from 10 to 15 kPa. Considering that the critical height of an embankment (with a safety factor set to unity) can be estimated as $h_{crit} = 5{\cdot}14{\cdot}S_u/\gamma_{emb}$, the critical height is lower than the designed value for the embankment (2·0 m). Therefore, due to the low bearing capacity of the upper part of the clay layer, improvement is necessary to prevent failure of the foundations.

Figure 3(b) shows the values of clay sensitivity (S_t) with depth. Considering all measurements, the average value of S_t is 13·2, but excluding the figure determined at 5·5 m depth, the value of S_t is equal to 8·4. Several authors (Mitchell and Soga,

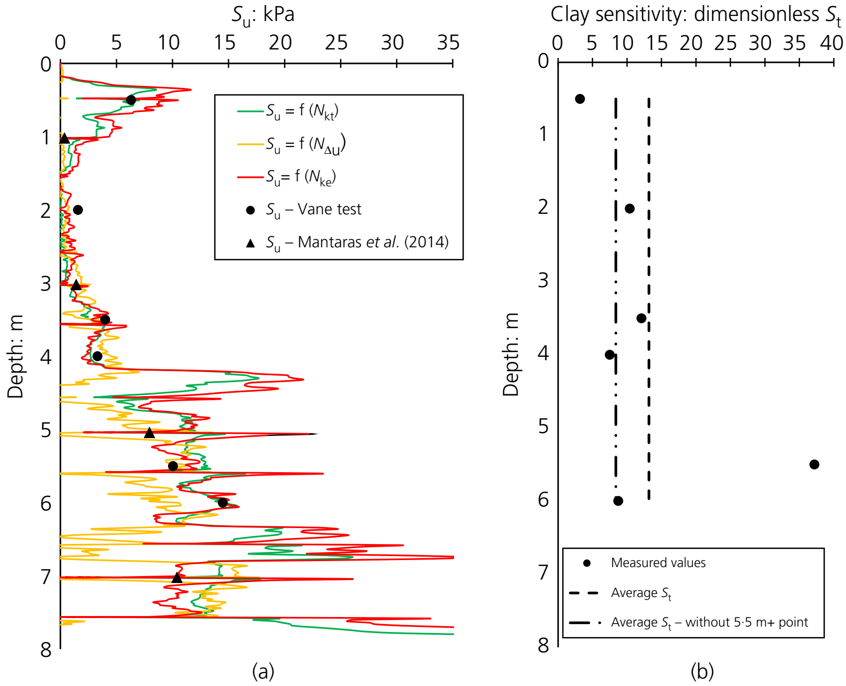

Figure 3. (a) Results of S_u obtained by CPTu and vane tests; (b) values of clay sensitivity (S_t)

Table 3. Parameters related to the dissipation tests and values of c_h(NA)

Parameter	Dissipation test DP 1	DP 2	DP 3	DP 4
Depth: m	1·00	3·00	5·00	7·00
$t_{50\%}$: s	18	870	90	3300
c_h(PZ): m^2/s	$4{\cdot}31 \times 10^{-5}$	$8{\cdot}92 \times 10^{-7}$	$8{\cdot}63 \times 10^{-6}$	$2{\cdot}35 \times 10^{-7}$
c_h(NA): m^2/s	$3{\cdot}84 \times 10^{-6}$	$7{\cdot}05 \times 10^{-8}$	$1{\cdot}01 \times 10^{-6}$	$2{\cdot}21 \times 10^{-8}$
k_h/k_v	33·4	3·52	14·4	0·71
RR/CR: dimensionless	0·089	0·079	0·117	0·094

2005; Rosenqvist, 1953; Skempton and Northey, 1952) have proposed a classification based on clay sensitivity. With a value equal to 8·4, the sensitivity of the clay can be classified as medium to high (Skempton and Northey, 1952).

Determinations of c_h(PZ) values (coefficient of horizontal consolidation) were made from CPTu dissipation tests as shown in Table 3, following Schnaid's (2009) recommendations. The consolidation coefficient for the normally consolidated soil, c_h(NA), was calculated according to Jamiolkowisky *et al.* (1985). Table 3 summarises the results obtained in the dissipation tests.

In Table 3, the value of T (time factor) considered in the calculation is equal to 0·245. Based on the characteristics of the CPTu equipment, the radius R of the cone is equal to 0·0178 m. As mentioned before, the rigidity index, I_r, was set to equal 100, a characteristic value for Rio de Janeiro clay (Mantaras *et al.*, 2014). The average value of c_h(NA) obtained from dissipation tests DP1, DP2, DP3 and DP4 is $1{\cdot}26 \times 10^{-6}$ m^2/s. However, according to Almeida *et al.* (2008) the consolidation coefficient for Rio de Janeiro clay generally lies between $0{\cdot}2 \times 10^{-8}$ and 80×10^{-8} m^2/s. The values obtained from dissipation tests DP1 and DP3 are outside this range. Almeida *et al.* (2008) and Almeida and Marques (2013) emphasise that the typical geotechnical profile of Rio de Janeiro consists of a sequence of sand lenses enclosed within soft clay. Indeed, the value of the relationship k_h/k_v for depths between 1·00 and 1·25 m (DP1 and sample 1) and between 5·00 and 5·25 m (DP3 and sample 3) is related to the occurrence of more permeable material (Jamiolkowisky *et al.*, 1985; Ladd *et al.*, 1977 classification). Therefore, the horizontal coefficient of c_h(NA) was taken to be the average value obtained from dissipation tests DP2 and DP4, leading to a value of c_h(NA) equal to $4{\cdot}63 \times 10^{-8}$ m^2/s. This value was used for evaluating the settlement without CPR grouting, that is using only PVDs.

The oedometer modulus was also determined from CPTu data using

2. $$E_{oed} = C(q_t - \sigma_{v0})$$

where C is an empirical constant, q_t the corrected resistance measured by the piezocone's tip and σ_{v0} the total vertical stress. Kulhawy and Mayne (1990) recommended a value of $C = 8{\cdot}25$. As shown in Figure 4, a better fit of E_{oed} was determined from oedometer testing and CPTu was achieved using a value of $C = 11{\cdot}0$.

The profile of OCR was determined by considering CPTu and the equations proposed by Chen and Mayne (1996), as follows

3. $$\text{OCR} = C_1\left(\frac{q_T - \sigma_{v0}}{\sigma'_{v0}}\right)$$

4. $$\text{OCR} = C_2\left(\frac{q_T - u_1}{\sigma'_{v0}}\right)$$

5. $$\text{OCR} = C_3\left(\frac{q_T - u_2}{\sigma'_{v0}}\right)$$

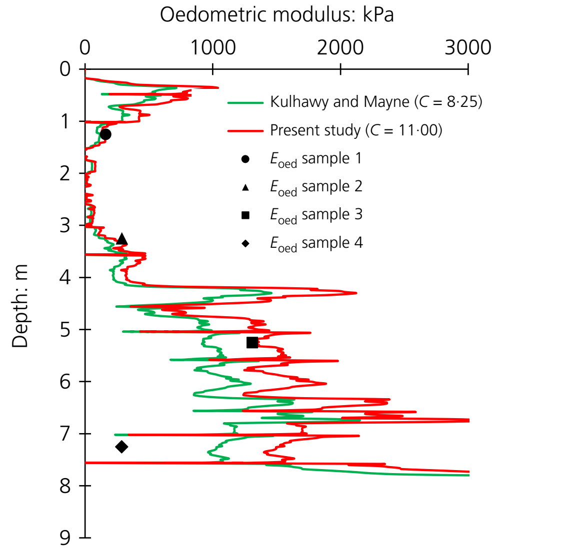

Figure 4. Values of E_{oed} from oedometer tests and CPTu

where q_T is the corrected cone tip resistance, σ_{v0} the total vertical stress, σ'_{v0} the effective vertical stress, u_1 the measured pore pressure at the cone tip and u_2 the measured pore pressure at the base of the cone. C_1, C_2 and C_3 are cone empirical constants. According to Chen and Mayne (1996) they are equal to 0·305, 0·705 and 0·530, respectively. Figure 5 shows the OCR profile, estimated by using Equations 3–5. Values of the empirical constants C_1, C_2 and C_3 were appropriately calibrated for the soft clay at Rio de Janeiro, according to Jannuzzi *et al.* (2015), who found a value of $C_1 = 0{\cdot}105$, and Baroni and Almeida (2012), who found values of C_2 and C_3 equal to 0·375 and 0·265, respectively.

As can be seen in Figure 5, values of OCR determined by Equations 3–5 display the same trend, indicating an overconsolidated layer within the uppermost metre of soil, underlain by a layer which is still consolidating, from 1·0 to 4·0 m depth. From 4·0 to 7·8 m depth, the average OCR value determined using Equation 5 (using C_3 given by Baroni and Almeida (2012)) was 1·00. Despite the OCR peaks registered by CPTu, the soft clay deposit appears to be under consolidation.

4. Foundation stiffened with CPR grouting: monitoring and prediction

Monitoring of vertical displacements was carried out using three settlement plates to assess the efficiency of CPR grouting by comparing the field measurements with the predicted settlement of the unimproved ground. These settlement plates, designated here as PR2, PR3 and PR4, were located near the cluster investigation I01. At first, the objective was to monitor the CPR grouting-improved foundation soils until settlement stabilisation but, unfortunately, readings were interrupted prematurely. Plates PR2, PR3 and PR4 were abandoned after 147, 133 and 80 d, respectively. For this reason, the final settlement

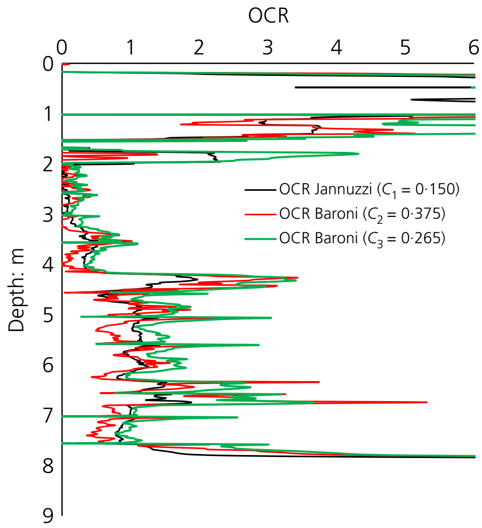

Figure 5. OCR values determined by CPTu using the proposals of Jannuzzi *et al.* (2015) and Baroni and Almeida (2012)

of the treated foundation was extrapolated from field measurements through the Asaoka (1978) method, as modified by Magnan and Deroy (1980). Figure 6 shows that the settlement exhibits practically the same behaviour.

The Asaoka (1978) method makes it possible to forecast the settlement against time curve for primary consolidation. The method also gives an estimate of the horizontal consolidation coefficient. The value of c_h is given by Equation 6, taking into account the combined vertical and horizontal drainage.

$$6. \qquad c_h = -\frac{1}{\Delta t}\frac{\ln\beta_1}{\left[8/d_e^2F(n)\right] + \left(\pi^2/4rH_d^2\right)}$$

where d_e is the influence diameter of the drain, $F(n)$ the drain density function, β_1 the slope of the line formed by points $S(t)$ n and $S(t)n+1$ in the Asaoka method (see Figure 7), Δt the time interval considered to get settlement points in the field curve and r the relationship c_h/c_v.

Figure 7 shows the graphical determination of the final settlement (S_∞), considering a time interval Δt equal to 20 d, despite the normal recommendation for Δt to be between 30 and 90 d (Almeida and Marques, 2013). The final settlement obtained using the procedure illustrated in Figure 7 is $S_\infty = 0{\cdot}167$ m. Table 4 shows the values used in Equation 6 for

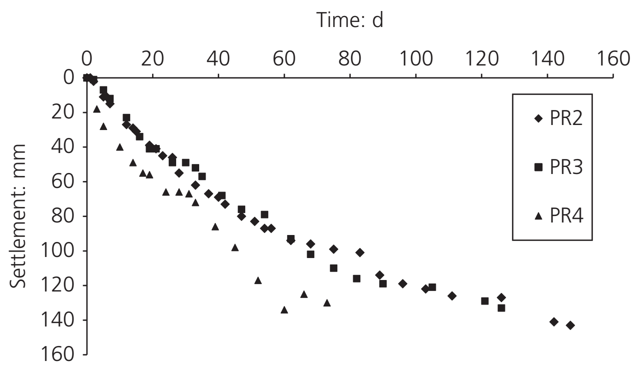

Figure 6. Results of monitoring with plates PR2, PR3 and PR4

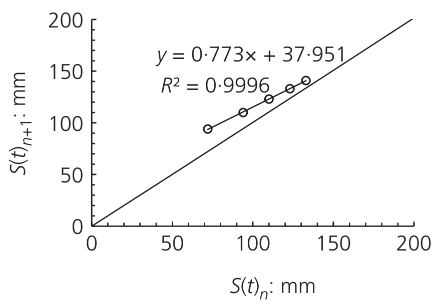

Figure 7. Asaoka procedure for determination of final settlement S_∞

Table 4. Parameters used for calculation of c_h and c_v using a modified Asaoka (1978) procedure

Parameter used for calculation of c_h					
d_e: m	$F(n)$	r	β_1	Δt: d	H_d: m
1·575	2·41	1·50	0·773	20	4·0

evaluation of c_h and c_v. Each drain is 100 mm wide and 5 mm thick.

The parameters shown in Table 4 led to a value of $c_h = 1{\cdot}03 \times 10^{-7}$ m^2/s and, consequently, a value of $c_v = 1{\cdot}55 \times 10^{-7}$ m^2/s, assuming $r = 1{\cdot}50$ (Table 4). The value of c_h obtained by the Asaoka method is different from c_h obtained by means of dissipation tests and can be understood as an equivalent horizontal coefficient for the modified, stiffened soil. The values of c_h and c_v obtained by the Asaoka method and the final settlement S_∞ were then used to draw the settlement–time curve to represent the behaviour after CPR grouting had been applied. To determine this curve, it was necessary to determine the combined degree of consolidation, according to Equation 7 from Carrillo (1942).

$$7.\quad (1 - U) = (1 - U_v)(1 - U_h)$$

where U is the combined degree of consolidation accounting for vertical and horizontal consolidation, U_v is the degree of consolidation due to vertical flux and U_h is the degree of horizontal consolidation.

The vertical percentage of consolidation (U_v) was calculated based on Terzaghi's theory and the horizontal percentage of consolidation (U_h) was determined according to Barron's (1948) theory. The settlement against time curve, extrapolated from field measurements, is shown in Figure 8. By considering the curve provided by the Asaoka method, it is possible to observe that the consolidation percentage $U = 90\%$ is reached after 175 d of construction. Therefore, the 133 d of measurements provided by settlement plate PR3 and 147 d provided by PR2 accounted for a significant portion of the total settlement.

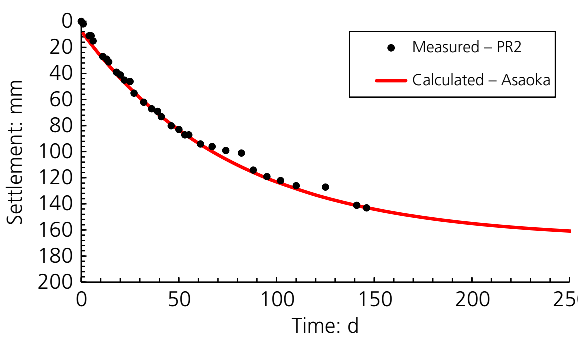

Figure 8. Field measurements of settlement and extrapolated curve using modified Asaoka (1978) approach

5. Prediction of settlement in soft soil only with PVDs (without CPR grouting treatment)

The final settlement of the unimproved soil, that is considering only the use of PVDs or even the condition without CPR grouting or PVDs, was evaluated by means of Equation 8. The values of oedometer modulus considered are those obtained from the oedometer tests and verified by CPTu measurements (see Figure 4). In order to perform the calculation, the soft soil layer was subdivided into four thinner layers, each 2·0 m thick.

$$8.\quad S_\infty = \sum_{i=1}^{n} \frac{\Delta\sigma_v}{E_{oed,i}} h_i$$

where $\Delta\sigma_v$ is the loading caused by the embankment ($\gamma_{emb} = 19$ kN/m^3) and is considered to be semi-infinity and equal for each layer, $E_{oed,i}$ the representative oedometer modulus for each layer and h_i is the thickness related to the layer under consideration (h_i is 2·0 m for all four layers).

Using Equations 2 and 8, the result obtained for the final settlement (S_∞) was equal to 1·06 m. This final settlement relates to the condition without CPR grouting and will be compared with the final settlement provided by the modified Asaoka (1978) method with CPR grouting and the equivalent homogeneous medium method proposed by Cirone (2016).

6. Settlement against time curves with and without CPR grouting treatment

The determination of the settlement against time curve for the condition with no soil stiffening but using PVDs (vertical drains) was made considering both vertical and horizontal drainage. The c_h used was $4{\cdot}63 \times 10^{-8}$ m^2/s (provided by dissipation tests) and c_v equal to $5{\cdot}90 \times 10^{-8}$ m^2/s (average value shown in Table 2). The curve for the condition without CPR grouting stiffening and without vertical drains was produced considering only vertical drainage. For the condition with only vertical drains, the drain mesh pattern is triangular with a spacing equal to 1·5 m. The geometry of the drains used is 100 mm wide and 5 mm thick.

The curve of the CPR grouting improved soil was obtained by extrapolating the settlement plate measurements and using combined vertical and horizontal drainage.

From Figure 9, it can be seen that the use of CPR grouting for soft soil improvement was effective in reducing the final settlement (S_∞) and the time of consolidation. The use of vertical

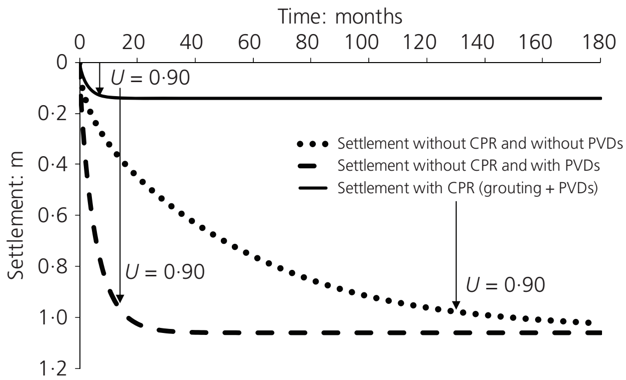

Figure 9. Comparison of settlement against time curves considering: (a) settlement without CPR grouting and without PVDs; (b) settlement without CPR grouting and with PVDs; (c) settlement with CPR grouting (grouting + PVDs)

drains (PVDs) is effective in reducing the consolidation time but there is no reduction in settlement; this is a well-known behaviour.

To reach a degree of consolidation of 90% ($U = 0{\cdot}90$) using CPR grouting, 175 d are required (6 months) and using only PVDs, about 420 d (14 months) are required. Natural soil consolidates in about 3840 d (128 months).

7. Improvement factor (β) and performance of CPR grouting stiffening

Priebe (1995) defines the improvement factor as being the relationship between the final settlement without soil treatment (S_∞) and the final settlement considering the application of treatment ($S_{\infty,\text{soil treated}}$). In the present case the treatment refers to the stiffening of soft soil by the application of CPR grouting. Therefore, the improvement factor (β) is given by Equation 9 as follows

9. $$\beta = \frac{S_\infty}{S_{\infty,\text{ soil treated}}}$$

The value of β was originally proposed to be used to quantify the efficiency of ordinary stone columns. In this study the efficiency of CPR grouting stiffening was also quantified by the improvement factor β. The use of factor β was considered because CPR grouting produces a non-rigid foundation. The final settlement without treatment, S_∞, is equal to 1·06 m and the final settlement with treatment, $S_{\infty,\text{soil treated}}$, was determined by two approaches

(*a*) extrapolation of field settlement against time using a modified Asaoka (1978) approach
(*b*) the methodology proposed by Cirone (2016), called the equivalent homogeneous medium method.

7.1 Improvement factor (β) using S_∞, soil treated obtained by a modified Asaoka (1978) approach

As shown in Section 4, the final settlement of treated soil ($S_{\infty,\text{soil treated}}$) is equal to 0·167 m. Therefore, using Equation 9 and considering the value of settlement without any treatment ($S_\infty = 1{\cdot}06$ m), the value of β is 6·34. Therefore, the use of grout injection and PVDs together with CPR grouting can be considered effective in settlement reduction in this case study.

7.2 Improvement factor (β) using S_∞, soil treated obtained by equivalent homogeneous medium method

Cirone (2016) presents a closed form, analytical solution for determination of the final settlement of stiffened soil with CPR grouting. This formulation is known as the equivalent homogeneous medium method. Some assumptions are made by the methodology: (*a*) the substitution ratio must be calculated in terms of volume; (*b*) the expansion of bulbs produces an increase in the stiffness modulus of the soil; (*c*) the imposed volumetric deformation has a relationship with the substitution ratio and (*d*) the bulbs are rigid inclusions.

In this methodology the substitution ratio is defined by Equation 10, in terms of the volume of the bulbs, and represented by R_S

10. $$R_\text{S} = \frac{\sum_{i=1}^{N} Vg_i}{AN_\text{b}h}$$

where Vg_i is the volume of expanded bulb number i (each bulb with 1100 litres for the present case), A is the transversal section of unit cell ($A = \pi d_\text{e}^2/4$), N_b is the number of bulbs ($N = 8$ for the present case) and h is the height between bulbs in one vertical line ($h = 1$ m for the present case).

For the condition found in the monitored work and using Equation 10, the value of R_S is 14·12%, considering a triangular mesh pattern with spacing $S = 3{\cdot}00$ m ($d_\text{e} = 1{\cdot}05S$), that is one bulb for each metre ($h = 1$). The volume of each bulb $Vg_i = 1100$ litres and the number of bulbs is $N = 8$.

The equivalent compressibility of stiffened soil, considering the effect of an increase in the average effective stress and the presence of hard inclusions and possible soil disturbance, can be estimated through the following equation

11. $$E_\text{oed,eq} = \frac{E_\text{oed,s}}{1 - R_\text{S}^{1/3}}\left(\frac{p'}{p'_0}\right)^n$$

where $E_\text{oed,s}$ is the oedometer modulus of soil at the considered depth, p' is the average effective stress after CPR grouting application, p'_0 is the average effective stress before CPT grouting application and n is the exponent dependent on the soil type ($n = 1$ for soft soils, Cirone, 2016).

The relationship between p' and p'_0 is given by

$$\frac{p'}{p'_0} = \frac{1 + 2K}{1 + 2K_0} \tag{12}$$

where K is the coefficient of earth pressure in the soil after CPR application and K_0 is the coefficient of earth pressure at rest.

The value of K can be determined by Equation 13, as follows

$$\lambda_c R_s = 1 - \frac{N}{1 + e_0} + \frac{(\lambda - \kappa)}{1 + e_0} \ln\left(1 + \frac{\eta^2}{M^2}\right) + \frac{\lambda}{1 + e_0} \ln \frac{\sigma'_{v0}(1 + 2K)}{3} \tag{13}$$

where λ_c is the coefficient of reduction of volume due to the consolidation, R_s is the substitution ratio, e_0 is the initial void ratio, λ is the Cam-clay parameter ($=c_c/2{\cdot}3$), κ is the Cam-clay parameter ($=c_s/2{\cdot}3$), η is the stress ratio ($= q/p'$, based on Cam-clay model), M is the Cam-clay parameter (slope of critical state line) and N is the specific volume corresponding to the unitary average effective stress on the isotropic line.

To solve Equation 13 in terms of K (coefficient of thrust in the soil after the CPR grouting application) it is necessary to determine the value of N according to Equations 14–18. The determination of K is necessary, in order to obtain the value of p' (see Equation 12).

$$p'_0 = \sigma'_{v0} \frac{1 + 2K_0}{3} \tag{14}$$

$$p'_P = \mathrm{OCR} \sigma'_{v0} \frac{1 + 2K_{0,NC}}{3} \tag{15}$$

$$R_p = \left[\frac{3(1 - K_{0,NC})}{M(1 + 2K_{0,NC})}\right]^2 \tag{16}$$

$$p'_y = p'_p R_p \tag{17}$$

$$N = (1 + e_0) - \kappa \ln \frac{p'_y}{p'_0} + \lambda \ln p'_y \tag{18}$$

Table 5. Input parameters of the equivalent homogeneous method

Layer/Sample	1	2	3	4
Depth: m	0·0–2·0	2·0–4·0	4·0–6·0	6·0–8·0
σ'_{v0}: kPa	5	11	17	23
λ_c	0·10	0·65	0·65	0·65
ϕ'_{cv}: degree	30	25	25	25
M	1·200	0·984	0·984	0·845
N	23·25	5·99	5·67	5·52
OCR[a]	2·00	1·00[b]	1·00	1·00
P'_0	3·33	7·90	12·21	16·52
P'	6·82	13·26	26·52	31·89
K	1·97	1·12	1·84	1·58
$E_{oed,eq,i}$[c]	688	1009	5936	1153

[a]Values of OCR according to CPTu measurements (see Figure 5)
[b]Soil under consolidation
[c]Equivalent oedometer modulus of soil for each layer, after CPR application (see Equation 11)

Table 5 sets out the input parameters used. The values of p'_0, p' and $E_{oed,eq,i}$ obtained are shown (see Equation 11). The other input parameters used (such as e_o, c_c, c_s and E_{oed}) have already been presented in Tables 1 and 2. The values of κ and λ are Cam-clay parameters and functions of c_c and c_s, respectively. The friction angles at constant volume (ϕ'_{cv}) in Table 5, were estimated according to Almeida *et al.* (2005). These values of ϕ'_{cv} are typical for the Rio de Janeiro clay region. The values of λ_c were estimated, based on the observations of Komiya *et al.* (2001) and Au *et al.* (2003) regarding the dependency of this parameter on the OCR of the clay.

The oedometer modulus of the entire, equivalent soft soil deposit is given by

$$E_{oed,eq} = \frac{\sum h_i}{\sum h_i / E_{oed,eq,i}} \tag{19}$$

For the condition with CPR grouting stiffening, the value of $E_{oed,eq}$ was found to be 1149 kPa (using data from Table 5) and, for the condition without stiffening, the equivalent oedometer soil modulus was $E_{oed,eq} = 287$ kPa (using data from Table 2). Therefore, the improvement factor β is $1149/287 = 4{\cdot}00$. Considering $\beta = 4{\cdot}00$, the estimated value for final settlement with CPR grouting is 1·06 m/4·00 = 0·265 m.

8. Conclusions

This paper presented an interpretation of a geotechnical investigation of a site located in the western zone of Rio de Janeiro, Brazil. The subsoil is a soft soil layer some 8·0 m thick. An extensive geotechnical investigation was carried out for this study. CPR grouting was used to stiffen the soft soil, with the aim of reducing the settlement and consolidation time due to the loading imposed by a 2·0 m high embankment. Predictions and measurements of the final settlement with and without CPR grouting were carried out. The main conclusions are described below.

The site investigation revealed that the 8·0 m thick layer of clay is an extremely soft soil (CR = 0·41) with low undrained shear strength (values lower than 5 kPa to a depth of 4·0 m).

The values of S_u estimated by the Mantaras *et al.* (2014) approach, using CPTu dissipation tests, agree very well with measurements obtained by shear vane testing and CPTu measurements, using empirical cone factors N_{kt}, N_{ke} and $N_{\Delta u}$.

The oedometer modulus can be evaluated using the equation proposed by Kulhawy and Mayne (1990). In Equation 2, these authors recommend a value for the constant $C = 8{\cdot}25$. In this work, and for the clay analysed, the best fit between values from the oedometer tests and CPTu was achieved using $C = 11{\cdot}0$.

The use of the Asaoka (1978) method, modified by Magnan and Deroy (1980), showed a good approximation when it was necessary to extrapolate field measurements (based on the results obtained from settlement plates) and construct the field settlement–time curve. This result was obtained for the improved soft soil with CPR grouting.

Thus, soil improvement with CPR grouting was capable of reducing the settlement and the time required for stabilisation. The degree of consolidation equal to 90% ($U = 0{\cdot}90$) was achieved in 175 d using CPR grouting. With only PVDs, this time would have been 420 d.

The improvement factor promoted by CPR grouting is approximately 6·43, if the final settlement is calculated using the modified Asaoka (1978) method and approximately 4·00 if the final settlement is calculated using the equivalent homogeneous method (Cirone, 2016).

The equivalent homogeneous method (Cirone, 2016) provided a final settlement (with CPR grouting stiffening) close to the final settlement indicated by the field measurements with settlement plates.

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