

# Construction methods in Brazilian extremely soft soils

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## ABSTRACT

About half of the population in Brazil lives in a 100 km strip along the coast, where ports and industries are also concentrated. The soil deposits found in coastal city areas in Brazil may consist of lowlands with a high water table, low strength and high compressibility. With increasing development, lowlands areas are increasingly occupied, where soft clay deposits may reach a thickness up to 40 m. This paper presents the Brazilian experience in the last couple of decades in issues related to geotechnical investigation, design and construction in lowlands. Emphasis is given to cases involving very soft and compressible ground found in south-east areas of Brazil, particularly on the west side of the city of Rio de Janeiro, where the authors' experience has been concentrated in recent years. The geotechnical characteristics of these uncommon very soft soils are outlined, followed by a description of the main methods adopted for the construction of embankments on these very soft clays. Discussion of new construction techniques on these soils is also presented.

## RÉSUMÉ

Environ la moitié de la population vit au Brésil dans une bande d'environ 100 km le long de la côte, où les ports et les industries sont également concentrées. Les dépôts de sols dans les zones des villes côtières au Brésil peut être constitués de marnes avec une nappe phréatique élevée, faible résistance et de la compressibilité élevée. Avec le développement croissant, les zones basses de terres sont de plus en plus occupés, où les dépôts d'argile molle peut atteindre une épaisseur allant jusqu'à 40m. Cet article présente l'expérience brésilienne dans les deux dernières décennies dans les questions liées à l'investigation et le projet géotechnique, et aussi de la construction sur ces dépôts. Les études sont concentrés aux cas des sol très mous et compressible du sud-est du Brésil, en particulier sur le côté ouest de la ville de Rio de Janeiro, où l'expérience des auteurs a été concentrée dans les dernières années. Les caractéristiques géotechniques de ces sols très mous sont décrites, suivi par une description des principales méthodes adoptées pour la construction des remblais sur ces argiles très molles. Discussion sur les nouvelles techniques de construction sur ces sols est également présenté.

## 1 INTRODUCTION

Soft clay engineering started in Brazil about sixty years ago with the birth of geotechnical engineering in the country and since then several studies have been developed along the Brazilian coastal deposits (e.g., Pacheco Silva, 1953; Lacerda et al., 1977; Pinto, 1994; Massad, 1994; Almeida and Marques, 2004; Coutinho and Belo, 2005).

Brazilian extremely soft clay deposits are generally quite compressible, typical values of compression ratio ( $C_c/(1+e_0)$ ) of Rio de Janeiro clays (Fig. 1), for example, is around 0.50. The top clay layer usually presents very low strength (design strength can be as low as 3 kPa) and very high initial water content ( $w_0$ ), which may reach values of 900% in the top layers (Almeida et al., 2008c). These may be defined (Sandroni and Consoli, 2010) as extremely soft soils which have the following features: SPT blow counts  $N_{SPT} \leq 1$ ; undrained strength  $S_u \leq 12$  kPa and net corrected point resistance  $q_T < 200$  kPa.

Undrained strengths ( $S_u$ ) are mainly obtained by vane tests (using vane borer equipment with shoe protection) but  $S_u$  is also obtained from stress history equations (based on the over-consolidation ratio OCR or pre-consolidation stress  $\sigma'_{vm}$ ). The design strength is obtained using the Bjerrum (1973) correction factor  $\mu$ , which is

usually in the range 0.70 - 0.60 due to the high plasticity of most Brazilian soft clays.

Under these circumstances design and construction should be carried out with special care, so that the ground is raised at the required elevation with settlement and stability control, but also considering the available budgets and schedules.



Figure 1. Localization of the areas (Almeida et al. 2008d).

## 2 GEOTECHNICAL PROPERTIES OF EXTREMELY SOFT BRAZILIAN CLAYS

### 2.1 Soil strata

Geotechnical site investigations have been carried out since the mid-nineties on twelve very soft clay deposits situated on the west side of Rio de Janeiro City, at Barra da Tijuca and Recreio dos Bandeirantes Districts, (henceforth Barra and Recreio). These sites cover a total area of about 7.4 km<sup>2</sup>, as shown in Figure 2.

The investigations carried out at these sites provided data for more than a dozen MSc and DSc theses undertaken at the Geotechnical Group of the Federal University of Rio de Janeiro. These studies were conducted in order to improve site investigation and construction techniques for very soft organic soil deposits. The data also provided geotechnical parameters for design and construction of embankments on these very soft soils.

Figure 3 shows the worst-case scenarios of soft soil strata from 576 SPT boreholes drilled at the sites. At the majority of the sites there are rivers or lagoons surrounding the area and sometimes the SPT shows a superficial layer of sand, which comes mostly from dredging or old fills. The soft layer thickness of these sites varies from zero to 22 m, however it has already been documented in the Recreio clay thickness of approximately 28m.

The superficial peat layer found at the majority of the sites varies from 1 to 4m in depth. The contribution of this layer to the overall settlement is quite substantial, since it is very compressible, with organic matter of about 50 to 60%. This superficial layer may not, however, be fully described by the SPT data alone. In this context, piezocone data as well as water content data measured using SPT samples are quite useful for a better definition of the soil layering.

Some soil parameters such as the compressibility index  $C_c$  can also be related to water content values. This low-cost and simple procedure (Futai et al., 2008) also provides a first approach to the variation of soil characteristics in extensive areas and can be used for preliminary geotechnical design.

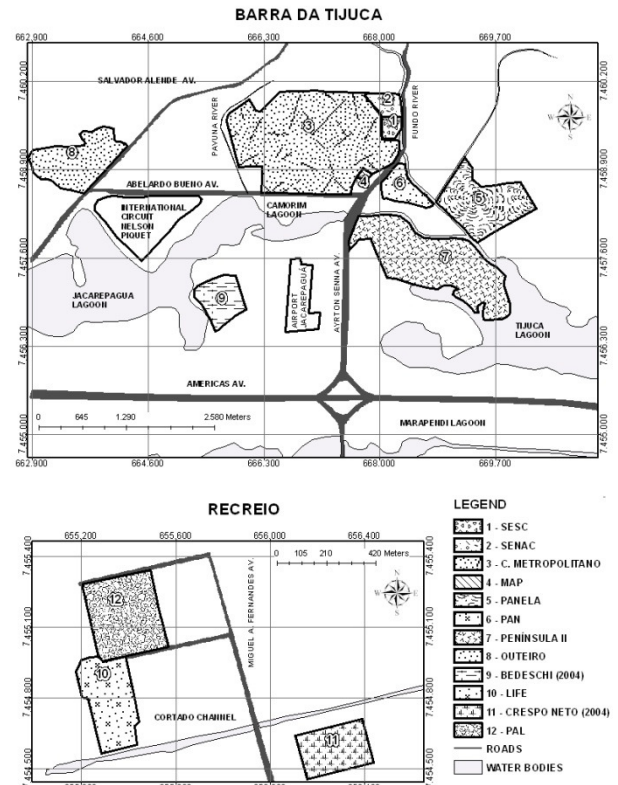


Figure 2. Localization of the sites (Almeida et al., 2008d).

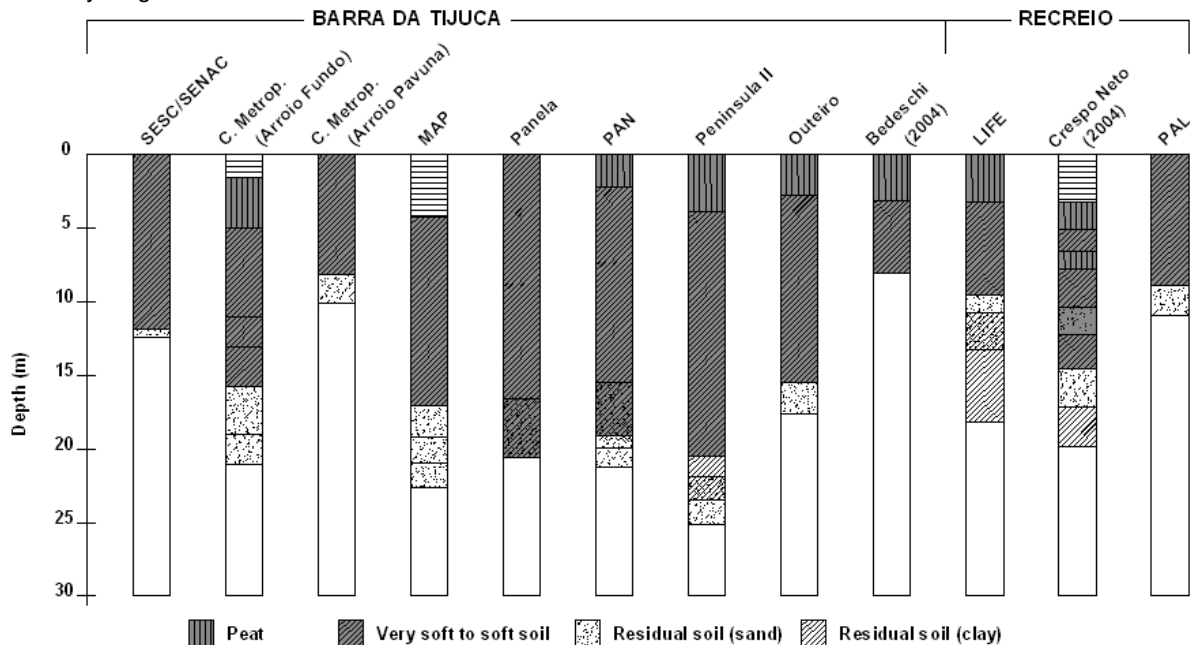


Figure 3. Typical soil profiles at the sites (Almeida et al., 2008d).

Table 1. Geotechnical characteristics of the sites (Almeida et al, 2008d).

Sites	SESC / SENAC	Metro-politan Center	MAP Barra	Panela	PAN	Península II	Outeiro	Bedeschi (2004)	Crespo Neto (2004)	Life	PAL
Reference	Almeida et al. (2002), Crespo Neto (2004)	Baroni (2010)	-	-	Macedo (2004), Sandroni and Deotti (2008)	-	-	Bedeschi (2004)	Crespo Neto (2004)	-	-
Very soft to soft clay thickness (m)	12	2-10	14	4-20	5-16	21.8	15	7.5	11.5	2-11	1.6-9.5
w <sub>0</sub> (%) (1)	100-500	57-789	-	100-488	116-600	70-805	67-207	102-580	72-410	76-913	72-1200
w <sub>L</sub> (%)	70-450	67-610	-	148-312	100-370	27-93	19-65	97-368	23-472	86-636	88-218
I <sub>p</sub> (%)	120-250	47-497	-	80-192	120-250	5-37	12-38	42-200	11-408	59-405	47-133
% clay	28-80	6-60	-	26-54	32	-	10-51	-	-	15-60	2-36
Natural specific weight (kN/m <sup>3</sup> )	12.5	10.01-16.9	13.7-20.2	10.2-13.4	11.6-12.5	-	11.9-23.3	11.2-12.3	11-12.4	10.2-14.0	10.9-14.9
CR = C <sub>v</sub> /(1+e <sub>0</sub> )	0.52	0.2-0.6	-	0.4-0.8	0.36-0.50	-	-	0.32-0.48	0.27-0.46	0.22-0.49	0.11-0.38
c <sub>v</sub> (10 <sup>-8</sup> m <sup>2</sup> /s) (2)	2-80	0,018-19	-	0.6-8.8	0.4-1.2	0.9-2.5	3-70	0.1-19.2	0.1-0.6	0.3-3.3	0.6-6.3
e <sub>0</sub>	1.95-11.1	1.43-12.37	-	3.3-8.2	4.8-7.6	-	0.4-4.7	4.3-9.0	3.8-15.0	3.0-21.9	1.0-11.6
S <sub>u</sub> (kPa)	7-19	4.5-21.9	5-10	3-38	5-23	4-29	7-41	1-22	3-19	4-18	2-19
N <sub>kt</sub> (3)	7.5-14.5	4-15	11	4-16	4-9	6.5-15	-	-	-	4-16	5-14.5

(1) soft clay and peat.

(2) c<sub>v</sub> values from piezocone and consolidation tests at normally-consolidated range, corrected for vertical flow.

(3) cone factor  $N_{kt} = (q_t - \sigma_{vo})/S_u$ , obtained by vane-piezocone correlations at the site, where q<sub>t</sub> = corrected tip resistance from piezocone test and S<sub>u</sub> is the uncorrected strength from the vane tests.

The analysis of the data available at the region shows that there is an important variability of the strata from site to site and even inside a single site. At some sites the clay thickness varies from zero to the high values presented in Fig. 3. This variability in clay thickness poses a major challenge in the design of embankments for reclaimed areas, as different construction techniques are used.

Table 1 shows the range of the soft soil deposits characteristics. The soft clay in all sites is quite compressible, with very high initial water content for peat (w<sub>0</sub> = 1200%) and soft clay (65 ≤ w<sub>0</sub> ≤ 500%) and quite low undrained strength. Owing to the high water content and organic matter the specific weight of peat layers can be as low as 10.2 kN/m<sup>3</sup>. The specific density of soil particles G<sub>s</sub> may also be quite low owing to organic matter content (2.44-2.64).

Massad (1999) described the geotechnical characteristics of south-east Brazilian deposits composed by fluvial-marine sediments (SFL) based on the results of thousands of boreholes. Most of the geotechnical parameters of the organic clays shown in Table 1 are inside the range of parameters presented by Massad (1999), but the parameters are in a wide range of values. This behaviour is quite different from inorganic clays, such as Canadian Eastern clays, for example, which, although structured, present well-related parameters (Marques, 2001).

Despite that, a relationship between compression index (C<sub>c</sub>) and natural water content (w<sub>0</sub>) could be established, as shown in Fig. 4. Futai et al. (2008) obtained C<sub>c</sub>=0.013w<sub>0</sub> for Rio de Janeiro State clays, which is very similar to C<sub>c</sub>=0.012w<sub>0</sub> obtained for Barra and

Recreio deposits. Peat values, however, are often outside this range.

These very soft soil deposits require very good sampling techniques to minimize soil disturbance. At the sites presented here, 4" to 5" diameter samplers were used and in general the Brazilian Code for sampling procedure for soft soils (ABNT, 1997) was used. The Ladd and De Groot (2003) sample preparation technique has been used in the most recent jobs. Even using these procedures, it is not easy to obtain good undisturbed samples, particularly for peat.

Baroni (2010) carried out consolidation tests on 22 samples using these techniques and found that only 60%-73% of samples were good to regular quality samples (by Lunne et al., 1997 or by Coutinho (2007) classification). An overall analysis of all the samples of Recreio and Barra sites, where those techniques were not always used showed that only 24% were good quality samples.

The poor quality of samples has led increasingly to utilization of in situ test data. Undrained strengths are mainly obtained through the use of vane tests and the design strength is obtained with the Bjerrum (1973) correction factor,  $\mu$ , which is around 0.65, owing to the high plasticity of these clays.

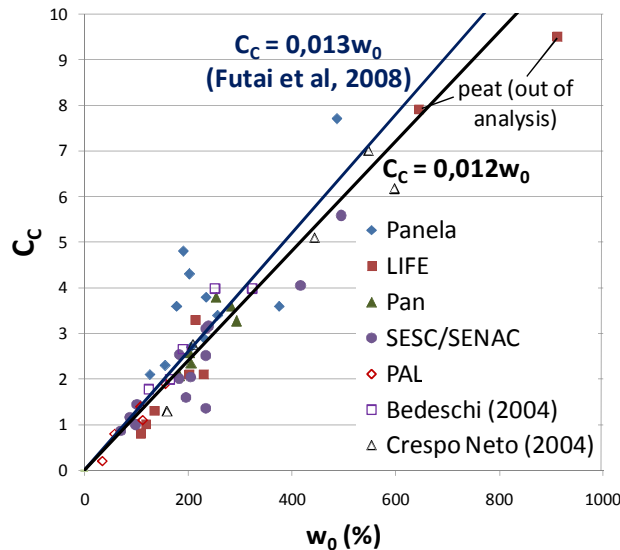
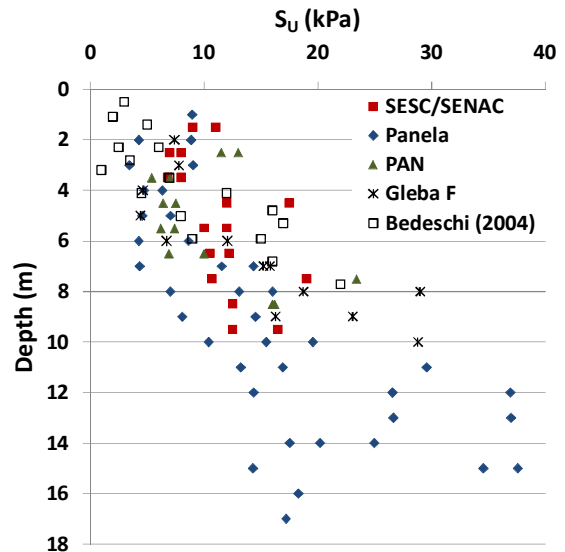


Figure 4. Compression ratio  $C_c$  versus in situ water content  $w_0$  for Barra and Recreio clays (Almeida et al., 2008d).

Figure 5 shows uncorrected undrained profiles obtained from in situ tests for Barra and Recreio. A high scatter of uncorrected undrained strengths values,  $S_u$ , can be seen, owing to the heterogeneity of the site characteristics. The variability of peat thickness at the areas also contributes to this scatter. The occurrence of wood sticks, roots or fibres increases the measurement of  $S_u$  values obtained from the vane tests carried out at the peat layer. In other words, the  $S_u$  values could be very high owing to the presence of the organic matter (Almeida et al. 2008d). The  $S_u$  values from the piezocone tests are not as high as vane test results in the peat layer, apparently owing to the form of the cone tip which is less influenced by the fibres existing in the peat.

### Barra clay deposits



### Recreio clay deposits

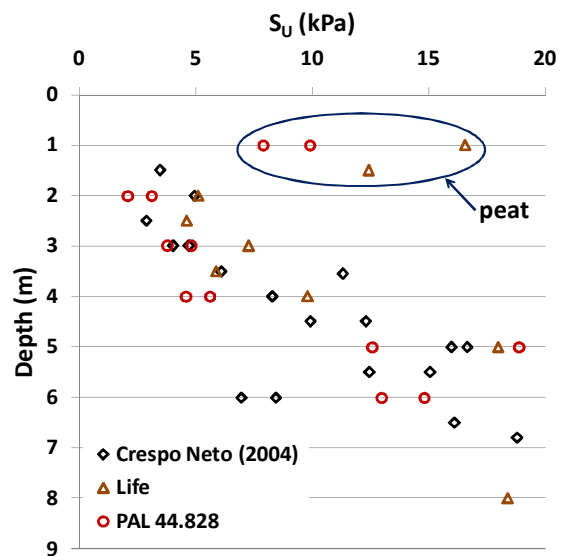


Figure 5. Uncorrected  $S_u$  profiles from piezocone and vane tests for Barra and Recreio clay deposits (Almeida et al., 2008d).

The  $N_{kt}$  values obtained for the Barra and Recreio sites are also inside a wide range. As a matter of fact, even within a single site there is a scatter of  $N_{kt}$  values, as shown in Table 1.

## 3 CONSTRUCTION TECHNIQUES

The settlements of an embankment necessary to raise the ground just above flood level for Barra and Recreio are generally high. For example, in order to stabilize settlements (both primary and secondary) of a 2.8m-high embankment built on 10m-thick soft soil deposit (history case described in section 4.1) it was necessary to have a 7.5m-high fill, which is only attainable in three stages,

since the  $S_u$  is very low, thus leading to construction schedule of about thirty months using vertical drains.

A large variety of construction methods may be used to build embankments on soft clays, as outlined in Table 2. It should be noted that these solutions rely on an increasing use of geosynthetics.

The methodology mostly adopted to build embankments on these very soft soils combines the use of some of the techniques of stage construction, berms, geogrid reinforcement and vertical drains under the embankment. Piled embankments, more often used recently, may present different categories regarding the load transfer platforms, which may be made of geotextile or even concrete.

Some experiences of pile-supported embankments are also mentioned in the present paper. Granular columns (encased or not) have also been used and will be discussed later.

For very soft clays like the ones described here piled embankment solutions may be more economic than embankments on vertical drains for soft layer thickness greater than about 12m (Almeida and Marques, 2004).

#### 4 VERTICAL DRAINS

Vertical drains have been the most widely used technique for soil improvement in Brazil for some time. The accumulated experience in Brazil and elsewhere (Almeida and Ferreira, 1992; Sandroni, 2006; Saye, 2001) indicates that the efficiency of vertical drains depends on the distance between drains. Nowadays it is common practice in Brazil to limit the minimum distance between prefabricated drains to 1.5m in order to avoid remoulding of the clay, although the mandrel and plate anchor dimensions are also factors to be considered in respect of remoulding and minimum drain distance.

Table 2. Construction methods and related Brazilian applications (Almeida et al., 2010).

Method/Technique	Comments	Brazilian studies
Soft soil removal	Limited to shallow deposits; the environmental impact and neighboring constructions are the main concern.	Also used at the case history described in section 4.1 (Recreio clay)
Soft soil expulsion (partial) by controlled (punching) failure	Quite often used for shallow deposits; method highly dependent on the local experience; the remaining soft soil left should be assessed by borings.	Zaeyen et al. (2003)
Fill surcharge	Usually associated with vertical drains; useful to decrease/suppress secondary settlements; may be applied without drains in case of shallow deposits.	Also used at the sites described in section 4.1
Stage construction	Mostly used with vertical drains; requires careful design and control (increase in clay strength); unfavorable for tight schedules.	Almeida et al. (1985) Almeida et al. (2008c)
Vertical drains	Used to accelerate settlements; wide accumulated experience.	Almeida et al. (2002), Bedeschi (2004)
Vacuum preloading	It may partially substitute fill surcharge (equivalent to about 4.5 m embankment height, if submersion is not considered); horizontal displacements lower than for standard surcharge; not yet applied in Brazil.	Marques and Leroueil (2005)
Piled embankments with geosynthetic platform	Favorable for tight schedules, may present different layouts and materials.	Almeida et al. (2008b), Sandroni and Deotti (2008), Borba (2007)
Granular columns (Granular piles)	Granular columns may be encased with geotextile or not encased; settlement acceleration owing to draining nature of the granular columns; geogrids are most often placed above granular piles.	Almeida et al. (1985), Santos et al. (2008), Mello et al. (2008)
Light fill materials	Favorable for tight schedules, relatively high cost; its use has increased recently.	Sandroni (2006); Lima and Almeida (2009)
Equilibrium berms	Quite commonly adopted; usually associated with reinforcement.	Almeida et al. (2002); also used in the case described in section 4.1 (Recreio clay)
Reinforced embankments	Requires careful design to ensure the right mobilization of the reinforcement tensile force.	Macedo et al. (2008), Magnani et al. (2008)

##### 4.1 Case history: embankment over very soft soil at Barra da Tijuca

A settlement study (Almeida et al., 2001) was performed on an embankment built on Barra da Tijuca highly compressible organic clay, in which water content varies

from 500% near the surface to 100% at the base of the clay deposit. The 0.60m thick drainage blanket provided support for the equipment used to install a prefabricated vertical drain (PVD) in a 1.70m triangular grid. The settlements covered a period up to 2 years, when the average degree of consolidation reached around 90%,

and at that time the average gain in strength of the whole clay layer was close to  $\Delta S_v/\Delta\sigma'_v = 0.22$ .

Values of  $c_h$  measured by piezocone dissipation tests and special radial oedometer were compared with field settlement's data using Asaoka's Method (1978) modified by Magnan and Deroy (1980). The overall generally good agreement obtained shown in Table 3 suggests that during the period of analysis (about two years) the secondary consolidation was negligible compared to primary consolidation.

Table 3. Summary of  $c_h$  values of Barra da Tijuca clay (Almeida et al., 2001).

Method	Range of $c_h$ variation ( $10^{-8} \text{ m}^2/\text{s}$ )	$c_h$ ( $10^{-8} \text{ m}^2/\text{s}$ average)
Asaoka (1978)	3.7 – 10.5	6.8
Piezocone	2.4 – 13.7	8.2
Special radial oedometer tests	3.6 – 6.8	5.0

#### 4.2 Case history: embankment over very soft soil at Recreio

An embankment over a very soft soil was constructed in stages in an area 150 x 350m at Recreio. The thickness of the clay deposit was 2 to 11m. In this case history, vertical drains, berms and reinforcement were used, which is quite common in the region. The soil parameters are presented in Table 1 (Recreio clay). It was monitored with settlement plates, inclinometers and Casagrande piezometers from 2006 to 2008.

Since the construction schedule for settlement stabilization was thirty months, a triangular grid of vertical drains spaced at 1.4m was installed in the front area, and the embankment was built in three stages. At the back of the site, for clay thickness lower than 4m, the embankment was built in two stages, and the second stage was built with the material removed from the surcharge of the third stage of the front area. At the border of the back of the area, due to a nearby channel, the soft soil was removed to increase stability.

Table 4 shows the measured coefficients of horizontal and vertical consolidation, from piezocone (overconsolidated range) and consolidation tests (normally-consolidated range). A wide range of values may be observed, and for design purposes the average values  $c_v = 1.67 \times 10^{-8} \text{ m}^2/\text{s}$  and  $c_h = 2.5 \times 10^{-8} \text{ m}^2/\text{s}$  were used for the normally-consolidated range.

At the front of the area, owing to the deposit's high compressibility and clay thickness it was necessary to stabilize 4.3m of settlement (primary and secondary) at the highest clay thickness. Vertical secondary deformations were considered to be about 7%, as suggested by Martins et al. (1997). Figure 6 shows the

results of a settlement plate installed at the site. The main difficulty was to install the settlement plates directly over the geotextile overlying the 3m peat layer, which presents water content up to 1000%. As soon as the fill is placed, the peat settlement plates may experience erratic movements. Thus the settlement plate was installed above the sandy preliminary fill and further probing was later conducted in order to assess the initial settlement.

Field horizontal coefficients of consolidation ( $c_h$ ) computed with Asaoka's method (1978) from the monitoring of settlement plates, for the final loading stage, was in the range of  $6-9.4 \times 10^{-8} \text{ m}^2/\text{s}$ . The average field value of  $7 \times 10^{-8} \text{ m}^2/\text{s}$  was almost three times higher than the average design value. This average field value does not take into account the differences between peat and soft clay, and the natural heterogeneity of the deposit. The average value from piezocone tests carried out at overconsolidated range is twelve times higher than the field average value at normally-consolidated range, which is consistent, given that the relationship between coefficients of consolidation at overconsolidated and normally-consolidated ranges is about ten. Field  $c_v$  values are in general higher than those from laboratory tests, but it seems that the use of vertical drains results in a clear separation between secondary and primary consolidation, which would lead to similar field and piezocone values (Almeida and Ferreira 1992).

Table 4. Vertical and horizontal coefficient of consolidation of Recreio clay (Almeida et al., 2008c).

Vertical	Piezocone tests		Consolidation tests	
	Depth (m)	$c_h$ ( $10^{-7} \text{ m}^2/\text{s}$ )	Depth (m)	$c_v$ ( $10^{-7} \text{ m}^2/\text{s}$ )
SP3	6.5	3.8	2.7	3.0
			4.7	0.4
			6.7	0.7
SP22	4	26.4		
	5	0.22		
	6	6.4		
	8	19.6		
SP36	2.1	1.8	1	2.2
	6	4.9	3	0.4
	4	7.6	5.5	3.4
			7.5	2.1
			9	2.6

## 5 REINFORCED EMBANKMENTS ON SOFT CLAYS

The placement of reinforcement at the base of an embankment over soft clay may substantially improve the embankment stability. Geosynthetic reinforcement is widely used at present, but the mobilization of the tensile force at the reinforcement is not well understood in general.

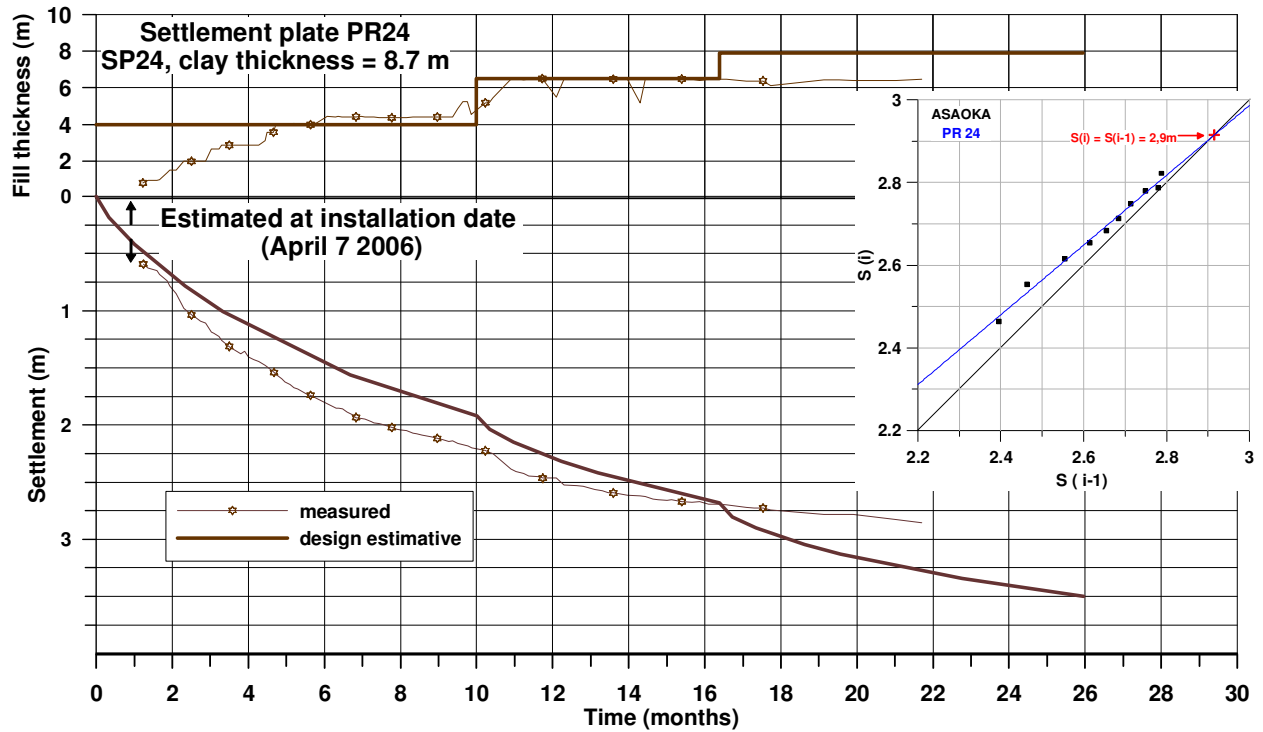


Figure 6. Settlement evolution measured at plate 24 (Almeida et al., 2008c).

It is not uncommon in design practice to use a fixed value of the reinforcement force  $T_{fix}$  without due consideration of the stress-strain compatibility of the soft clay-embankment-reinforcement system. The value of  $T_{fix}$  is usually based on the maximum short-term reinforcement strength  $T_{max}$  divided by strength reduction factors  $f_1, f_2, \dots, f_i$ , which take into account effects such as creep and mechanical and environmental damages. This maximum service value  $T_{fix} = T_{max} / (f_1, f_2, \dots, f_i)$  may not, however, be all mobilized in situ even when failure is reached (Rowe and Soderman, 1985).

Table 5. Main features of the two experimental reinforced embankments (Magnani et al., 2009).

Characteristics	Embankments	
	TE1	TE2
Reinforcement	Stabilenka (200kN/m x 45kN/m)	Stabilenka (200kN/m x 45kN/m)
Vertical drains	Colbondrain CX 1000, 10cm x 0.5cm, triangular mesh, thickness 1.30m.	Without drains
Thickness of the clay layer (m)	8.2	5.6
Thickness of existing sandy working platform (m)	1.7	1.8

These issues were addressed by Magnani (2006), who built two reinforced test embankments which led to failure

with measurement of the tensile forces  $T$  at the geotextile reinforcement. One of the test embankments had prefabricated vertical drains, but owing to the relatively short construction time, about forty days, the embankment behaviour was, in fact, close to undrained condition.

The polyester geotextile reinforcement used had a short-term maximum strength  $T_{max} = 200\text{kN/m}$  and modulus  $J = 1700\text{kN/m}$ . Table 5 presents the main features of the two test embankments, TE1 and TE2.

The failure of the test embankments was clearly defined by the change of gradient of the maximum horizontal displacements measured in inclinometers.

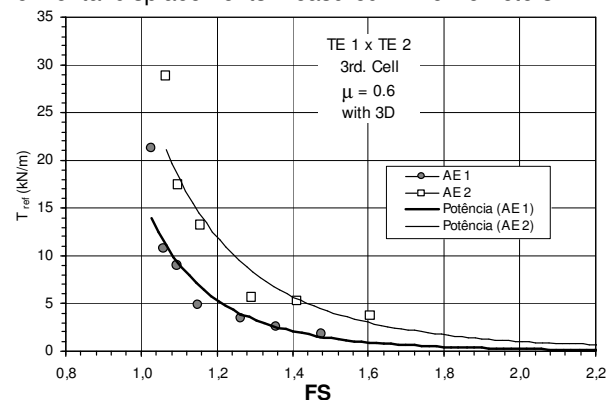


Figure 7. Measured reinforcement forces versus factor of safety for test embankments TE1 and TE2. (Magnani et al., 2009).

Figure 7 shows the relationship between the factor of safety and the reinforcement force  $T$ . The values of  $T$  correspond to the measured values at the load cell located close to the failure surface after the fourth layer of



embankment construction, as measurements for the first three layers resulted in quite small T values. Note that the values of T increase with the decrease in the factor of safety and only values of  $F_s$  lower than 1.2 have mobilised significant values of T.

From the experience of the two reinforced test embankments Magnani (2006) proposed the conceptual curves of reinforced embankment behavior presented in Figure 8.

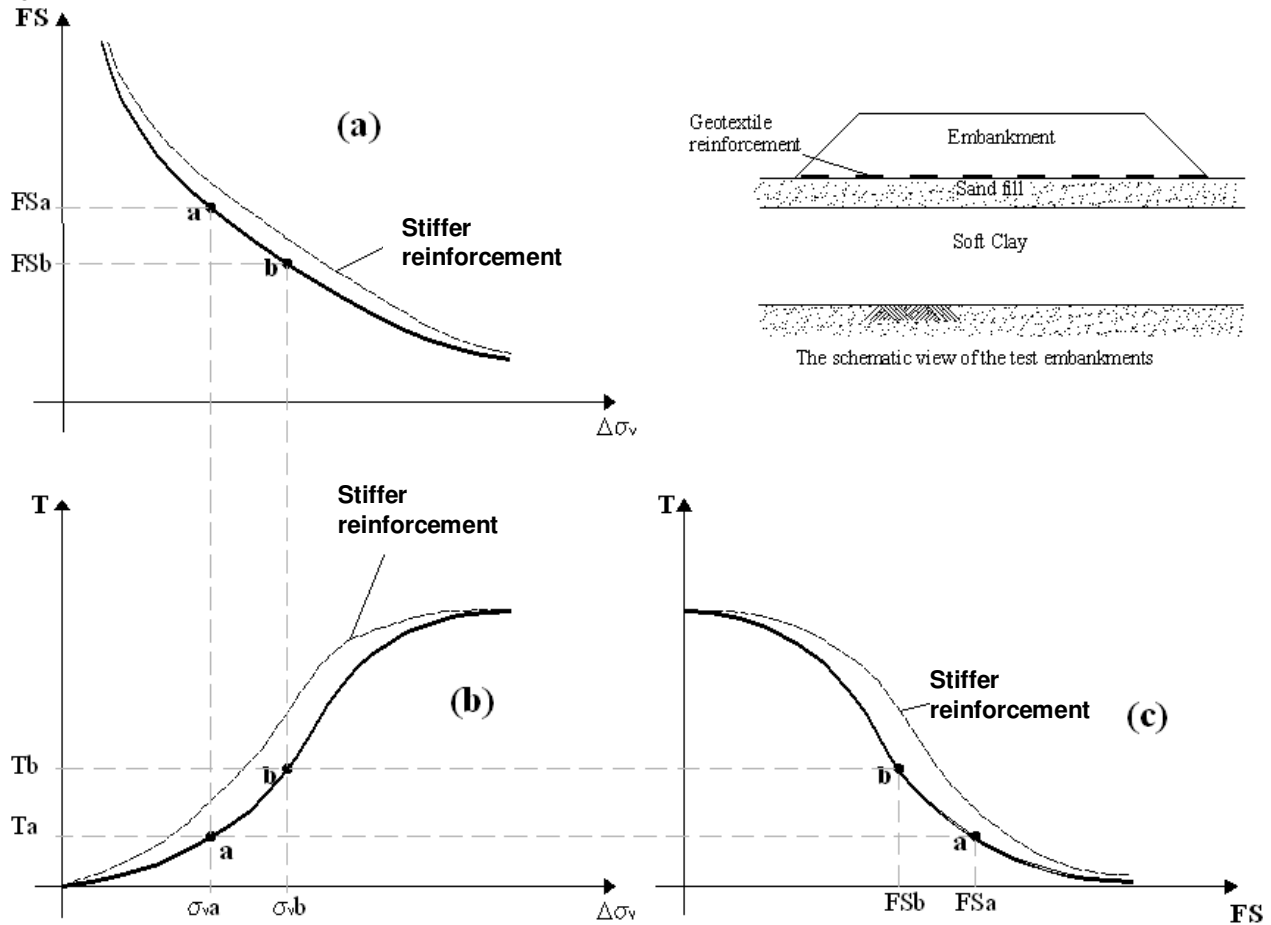


Figure 8. Conceptual curves on the behaviour of reinforced embankment on soft clays (Magnani, 2006).

Figure 8a shows the decrease of the factor of safety with the increase of the embankment load  $\Delta\sigma_v = \gamma H$ . Magnani (2006) also showed from field data that the mobilization of the tensile force T increased with the increase of the embankment load  $\Delta\sigma_v$  (Fig. 8b). Values of T may increase until a maximum value  $T_{lim}$  is reached. The value of  $T_{lim}$  corresponds to the complete yield of the soft clay and is not necessarily mobilized, as it depends on the overall deformation of the system foundation-reinforcement-embankment.

As the value of T increases, the safety factor FS decreases (Fig. 8c). The value of T may continue to increase substantially, even after failure is reached at  $FS = 1.0$ . The tensile force continued to increase with the increase of the embankment load even after failure ( $FS < 1.0$ ), and  $T = 70\text{kN/m}$  was measured sometime later owing to increased displacements caused by soil yield and creep.

Figure 8 also shows FS and T values of a geotextile with greater modulus J. The use of a greater modulus J allows greater FS for the same embankment load  $\Delta\sigma_v$ , as

shown in Figure 8a, thus mobilizing greater T, as shown in Figure 8b.

Estimated tensile forces at failure at the two test embankments by the method proposed by Rowe and Soderman (1985) were not so close to measured T values, but overall values were of the same order of magnitude.

As shown in Table 5, the test embankments had a sandy working platform, made of hydraulic fill, 1.7-1.8m-thick. Stability analyses showed that the contribution of this top sand layer in the stability was greater than the contribution of the reinforcement (Magnani et al., 2010). Almeida et al. (1995) reached the same conclusion in their comparison of reinforced and non-reinforced embankments.

## 6 PILED EMBANKMENTS

The simplest case of piled embankment is when the load is transmitted by the fill arching (Terzaghi, 1943) directly to the pile caps without the use of reinforcement. In this



problem, the distance between caps in square or triangular mesh is calculated (eg Broms, 1977; Ehrlich, 1993), depending on the height of the embankment, its properties and surcharge.

An evolution of the embankment on piles was the incorporation of geosynthetic reinforcement principle allowing the use of more widely spaced piles.

The construction technique of piled embankments demands lower fill volumes than conventional solutions, with a shorter construction schedule. This section presents the results of two low embankments supported on piles with geogrids, built at Barra da Tijuca, at SESC and SENAC sites, near to the 2007 Pan American Games Village (PAN).

The main components of a piled embankment with geogrid platform are: compacted fill height ( $h$ ), pile space ( $s$ ), pile cap dimensions ( $b$ ),  $a = (s - b)$  is the distance in between pile caps, as shown in Figure 9.

Table 6 presents the main characteristics of the two embankments, both with pile caps disposed in a square grid. The ground level is at elevation +0.5m on both sites.

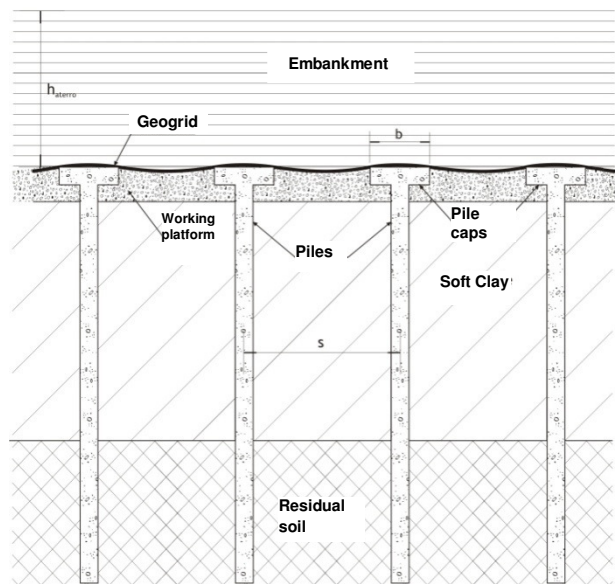


Figure 9. General scheme of a piled embankment (Almeida and Marques, 2010).

The thickness of the working platform varies from 0.6m to 1.0m. The final elevation of the embankment ranges from 2.6m to 3.0m. As a consequence, the height of the piled embankment is generally lower than 1.5m, which is an important characteristic of the cases described here. As shown in Table 6, the relationships between fill height  $h$  and pile caps span  $a$  are lower than unity (in the range 0.70 – 0.78) in both cases.

Table 6. Main characteristics of the two piled embankments.

Characteristics	Embankment 1	Embankment 2
Construction year	2004	2004–2005

Number of piles	1,900	$\cong 10,000$
Pile space, $s$ (m)	2.5	2.8
Square pile cap dimension, $b$ (m)	0.80	1.00
Distance between pile caps, $a = s - b$ (m)	1.70	1.80
Embankment height above pile cap, $h$ (m)	1.2	1.40
Ratio $h/a$	0.70	0.78
Geogrid characteristics	Fortrac R, polyester, biaxial	Fortrac, PVA, biaxial
Nominal geogrid strength (kN/m)	200	200 and 240
Geogrid modulus (kN/m)	3600	3600 and 4400
Fill height below the pile cap (m)	2.0	0.60 to 1.0
Soft soil deposit thickness (m)	8 to 10	9 to 11

## 6.1 Piled Embankment 1

At this site (Spotti, 2006 and Almeida et al., 2007) two configurations of the support condition below the geogrid were studied (excavated and non-excavated layouts). Table 7 presents the main field observations for square pile cap grids and pile centre distance of 2.5m, along a period of six months. For the same embankment height, settlement values of conventional embankments can be 10 times higher. As expected, the settlements of excavated areas were higher than settlements of the non-excavated areas.

Table 7. Monitoring results of the experimental site 1 (Almeida et al., 2008b).

Characteristics	Value
Fill height above pile caps (m)	1.10 to 1.30
Settlements (m)	0.10
Settlements (m), excavation under the geogrid	0.15 to 0.35
Geogrid deformations (%)	0.5 to 2.0

Loads in the reinforcement geogrid were calculated using Kempfert et al. (1997)'s method. For maximum measured deformation, of  $\epsilon_a=2\%$ , and surcharge =0, the calculated load was  $T=42\text{kN/m}$ . The maximum mobilized load value estimated at field, considering  $\epsilon_a=2\%$  and modulus  $J=1400\text{kN/m}$  (of geogrid), results in  $T=28\text{kN/m}$ . It seems that the comparison between measured and calculated values was adequate. However, the period of monitoring was relatively small and the reinforcement load continued to increase with time, as expected.

## 6.2 Piled Embankment 2

Pile embankment 2 was built over an area of  $80.000\text{m}^2$ , surrounding the piled buildings. This was next to the site

of pile embankment 1 and the possibility of constructing the buildings and the piled embankment simultaneously was one of the main reasons for using a piled embankment. Discussions about the interface of piled buildings and piled embankments are presented elsewhere (Almeida et al., 2008a).

Settlements measured until the end of 2005 are presented in Table 8. Most of these settlements occurred due to the settlement of the working platform without piled embankment construction, most of which actually occurred in the three months before embankment completion.

Table 8. Results from about 500 days of monitoring – end of 2005 – site 2 (Almeida et al., 2008b).

Characteristics	Value
Fill height above pile caps (m)	Below 1.50m
Settlements of the working platform (m)	Below 0.40m

### 6.3 Acquired experience

On the basis of the performance of these two piled embankments, and other piled embankments built nearby, a number of lessons have been learned.

Pile caps can be cast inside the fill, as adopted for Embankment 1 (Fig. 10a), or built above the working platform, as adopted for Embankment 2 (Fig. 10b). In both cases it is common practice to use an elevation of about 5cm at the centre of the pile cap, with the purpose of minimizing the high stress levels in the geogrid at the cap corners.

Figure 10 shows the settlement of the working platform surrounding the pile cap, for both cases. The working platform, usually in the order of 0.6m, will settle and these settlements takes a long time to stabilize and are not negligible owing to secondary settlements which could be as much as 7% of the clay thickness, for the soft clay deposits studied here.

The results suggest that sub-grade reaction should not be considered for the design of piled embankments with geogrids. Given the continuous geogrid deformation and the possibility of post-construction settlements, it is suggested that the pavement should be light and flexible, allowing future maintenance works.

In any case, and in order to avoid settlements, the geogrid should be installed soon after the pile cap construction. If settlements occur before geogrid installation, it will be necessary to level the fill before the geogrid installation to ensure that it is well stretched.

The next construction stage is the geogrid installation over the pile caps. Since the concrete pile cap is rough, a non-woven geotextile is generally used in order to protect the geogrid against abrasion. Regarding geogrid installation, the more stretched the geogrid (without pre-stressing), the earlier the strength will be mobilized upon embankment loading.

The settlements observed at the top of the piled embankment in the short and long term are basically the result of the geogrid deformation, as shown in Figure 11. This figure also shows that a void below the geogrid could be expected in the long term, as a consequence of primary and secondary consolidation. In this case, the

installation of a geotextile over the geogrid will be an additional benefit, minimizing soil loss by erosion. Some design methods (e.g. Kempfert et al. 1997) consider the reaction of the soil below the geogrid. As suggested by Figure 11, however, for very compressible soft soils, this reaction should not be considered, since in very soft soils the geogrid loses contact with the soil underneath it.

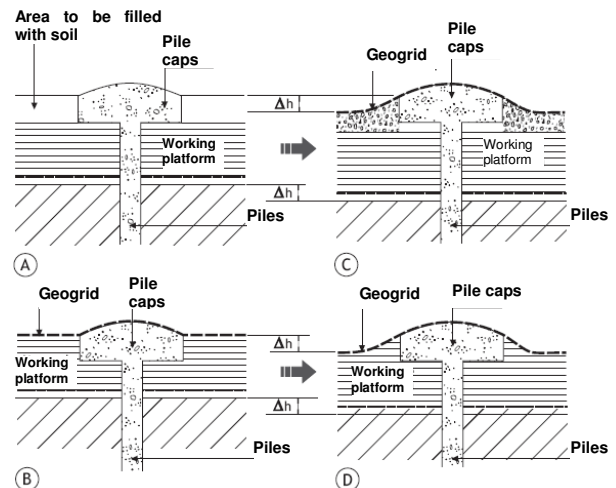


Figure 10. Settlements of working platform: (a)-(c) pile caps above working platform; (b)-(d) pile caps inside working platform (Almeida and Marques, 2010).

In order to mobilize geogrid support, it is necessary for geogrid deformation to take place. This deformation will occur because of both embankment loading and settlement of the working platform. The latter is particularly important in very soft clays, as in the present case. With this construction technique, the final construction of the pavement should be delayed as much as possible.

For very soft clays in Rio de Janeiro, a piled embankment with a geogrid platform can be a solution with better technical, economic and schedule advantages than a reinforced embankment with berms on vertical drains. The two piled embankments described in this paper were built on these soft clays in an area of about 92000 m<sup>2</sup> with a low ratio of embankment height to pile caps span.

Table 9 presents geometric data of some recent case histories of piled embankments in Brazil. It is observed that unlike the cases described above the height-span ratio of these cases is much higher, thus increasing the arching effect, as previously discussed.

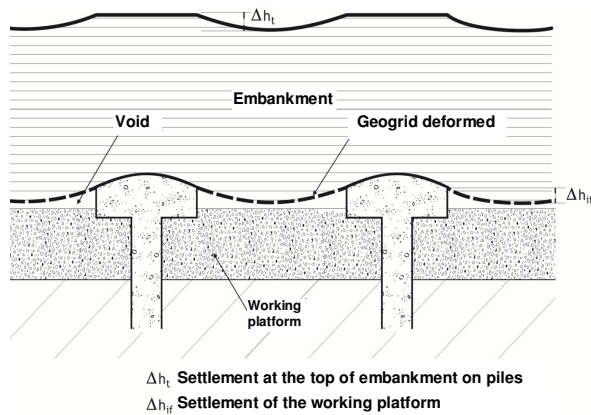


Figure 11. Settlements at the top of the piled embankment (Almeida and Marques, 2010).

Table 9. Some recent case histories of piled embankments in Brazil (Huesker-Brazil, 2011).

State	Embankment height h (m)	Piles spacing s (m)	Pile cap width a (m)	Nominal geogrid force (kN/m)	Height-span ratio h/(s-a)
Paraná	3.0	2.7	1.0	150	1.8
Minas Gerais	11.0	1.7	0.8	200	12.2
São Paulo	5.5	2.4	1.1	100	4.2
Brasília -DF	6	2.0	0.5	100	4.0
São Paulo	1.9 to 4.2	2.0	1.1 and 0.80	400	3.2
Rio Grande do Sul	2.4 to 3.6	2.3	0.9	200	2.1

## 7 EMBANKMENTS ON GRANULAR COLUMNS

Techniques used for granular column installation (e.g. Garga and Medeiros, 1995) did not present adequate performance in Brazil in the last century. This was mainly due to issues such as type of equipment used and selection of granular column diameter and spacing. Modern equipment and techniques have been recently introduced and important soft clay improvement works have been carried out in Brazil using granular columns.

One of these works was the construction of an iron ore stockyard in the CSA ThyssenKrupp Steel Plant, covering an area of approximately 9km<sup>2</sup>, located 50km south from Rio de Janeiro city. Due to vertical stresses equal to 340kPa imposed by the 13m high iron ore stockpiles, vibro-replacement techniques with dry and wet methods were used. Subsoil conditions at the site consist of fluvial and fluvio-marine sediments of quaternary origin, alternating stratification of sands, silts and clays, as well as fluvial gravel and younger mangrove deposits (Wegner et al., 2009 and Marques et al., 2008). The compressible layers are 12 to 15m deep, reaching 17m in some places. It was estimated that, without the gravel columns, the

settlements would reach values around 1.5m. Figure 12 shows the geometry and geotechnical model used for the solution with gravel columns.

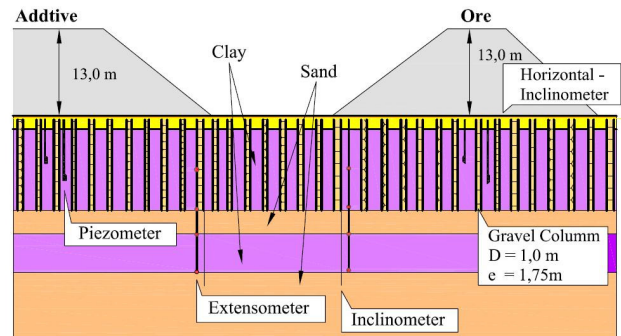


Figure 12. Geometry and geotechnical model used for the solution with gravel columns (Wegner et al., 2009).

The gravel columns were installed at intervals between 1.75m and 2.20m in a square arrangement, with an average length of 12m and a total length of approximately 400km. A load transfer platform was constructed above the gravel columns with the use of two layers of high-strength bi-directional geogrids. Settlement rates from 2 to 3cm per month were observed in the iron ore stockyard, but decreased after a period of approximately 4 months (Weigner et al., 2009).

Other stone column works carried out in Brazil recently (Felix, 2009) include the construction of 10305m of column (diameter 0.80m; length 7–10m; spacing 1.80–2.50m) at the BR-101 highway toll station in Araquari, State of Santa Catarina. Also, 5378 m of column were installed for the enlargement of the Ubá municipal airport, State of Minas Gerais (diameter 0.70m; length 6–9m; spacing 3.5–5.0m).

## 8 EMBANKMENTS ON ENCASED SAND COLUMNS

Encased sand columns were used for the first time in South America on a highway (Mello et al., 2008) near the city of São José dos Campos, 100km from São Paulo city. The subsoil at this site is composed of two layers of soft clay separated by a layer of silty sand. Table 10 shows the range of geotechnical parameters of these two clay layers.

Table 10. Range of geotechnical laboratory parameters

Parameter	Range of values
Bulk weight (kN/m <sup>3</sup> )	13.2-15.4.
Vertical consolidation coefficient $c_v$ (10 <sup>-8</sup> m <sup>2</sup> /s)	1.3-50
OCR	1.2-1.6
CR = $C_c/(1+e_0)$	0.17-0.40
Undrained strength (kPa)	6-12

The columns were installed using closed end Franki pile equipment. After the Franki tube installation, the sand was deposited within the geosynthetic casing and the tube was removed with the aid of a vibratory hammer. Figure 13 shows the final stages of implementation of an



encased column installation and the column inside. Table 11 summarizes the characteristics of the columns used and some monitoring results.

Encased sand columns were also used in sites of the CSA ThyssenKrupp Steel Plant iron ore stockyards (Alexiew and Moormann, 2009) where the local soil was composed of layers of very soft and compressible clay. The soil and column characteristics are summarized in Table 12.



Figure 13. Finished geosynthetic encased sand column.

Table 11. Summary of the characteristics of columns and results.

Characteristics	Values
Diameter of columns	0.70 m
Geotextile used in the jacking	Final stress of 50kN/m, rigidity of 1000kN/m
Length of columns	10.0 m
Spacing	1.8 and 2.2 m
Settlement measured (max.)	100 mm
Time of stabilization after the beginning of readings	6 months

Table 12. Range of geotechnical parameters of the layers and characteristics of columns in the CSA job (Alexiew and Moormann, 2009)

Soil parameters	Values
Thickness of clay layers	Up to 20m of clay layer with 8m or more compressible clay
Oedometric modulus	0.2-0.5MN/m <sup>2</sup>
c <sub>v</sub>	2-4 x10 <sup>-8</sup> m <sup>2</sup> /s
S <sub>u</sub>	5-15 kPa
Diameter of columns	0.78m
Length of columns	10 to 12m
Spacing	2.0 x 2.0m
Geotextile used in the jacking	Ringtrac R 100/250 and 100/275

## 9 LIGHTWEIGHT FILL

Different types of lightweight materials may be used in order to reduce settlements and increase the overall embankment stability. Among these materials, expanded polystyrene EPS is currently the most widely used as it

has the lowest specific weight and combines high strength and low compressibility. The use of EPS blocks as a light fill material is still not widely used in Brazil, owing to its high cost, but some uses in São Paulo (Gonçalves and Guazzelli, 2004), and in Rio de Janeiro (Sandroni, 2006; Lima and Almeida, 2009) and BR-101 highway in State of Paraíba and Pernambuco (constructed by the Brazilian Army) have been reported as exemplified in Figure 14.



Figure 14. Use of EPS blocks as lightweight fill material: a) installation of the blocks b) cross section showing blocks and protective cover (BR-101).

## 10 VACUUM CONSOLIDATION

Vacuum consolidation or vacuum preloading is a technique which uses atmospheric pressure in order to apply a preloading over a soft soil deposit, as described by Magnan (1994), Cognon (1991), Cognon et al. (1994), Jacob et al. (1994), Marques (2001), Chai et al. (2010) and Varaksin (2010).

This technique, proposed by Kjellman (1952), comprises the application of vacuum over the soft soil layer by use of a vertical drains grid connected to a horizontal drains grid placed inside the sandy working platform. The water is then pumped from the soil by vacuum pumps connected to the horizontal drains grid. In order to achieve the vacuum it is essential to guarantee that the system is airtight, so it is necessary to dig trenches around the treatment area as far as the water

table level and then to place an impervious PVC membrane going down the trenches. The PVC membrane should be below the water level inside the trenches, as shown in Figure 15.

The pumping system should be able to pump water and air together, and inside this system the efficiency is about 100%. The suction measured below the membrane, however, is about 70-75kPa, thus giving a vacuum system efficiency of about 70-75%. This change in pressure is equivalent to an embankment 4.5m high.

Soon after the beginning of the pumping there is a strength increase of the sand owing to the apparent cohesion of the sand, which becomes partially saturated. This behaviour is very favourable for equipment traffic if an embankment is designed above the membrane (Marques, 2001). An initial heightening of the embankment is carried out far enough to protect the membrane.

When vacuum consolidation is applied, the pore-pressure decreases and can be negative, depending on depth.

The strain-rates of vacuum consolidation are higher than by conventional preloading owing to the fact that the preloading can be achieved in one single step.

The main advantage of this technique is that horizontal displacements are insignificant and there is no increase in shear stress decreasing the safety factor. Other advantages are: structures nearby are not horizontally charged; there is no significant decrease in total stress owing to submersion; twenty-four hours after pumping begins the fill can be placed over the membrane; volumes of fill and removal of fill are comparatively low.

Disadvantages of vacuum preloading are mostly related to the soil profile: when there are sand layers inside the clay deposit and outside the treatment area, it is necessary to surround the area with a slurry wall barrier, which is the main drawback with the use of vacuum consolidation in Brazilian deposits, otherwise there is diminution of the efficiency of the technique or it cannot be used at all. Additionally, when the natural water table level is low, the elevation of the water table up to the horizontal drains during pumping will decrease the preloading value

available owing to vacuum preloading. In this case, it is recommended to install the horizontal drains as near as possible to the water table level.

As for the technique operation itself, the disadvantages are that it is necessary to maintain the pump system with care and the economic viability of the technique for small sites is questionable.

For very soft Brazilian deposits, with peat layers as deep as 4m, for example, near channels or near previous adjacent structures, where horizontal displacements are not suitable, this technique could be a solution.

Brazilian expertise with vacuum consolidation comes from a study performed at a site in Canada (Marques and Leroueil, 2005), in a research program carried out by Laval University and COPPE/UFRJ. Two trial embankments were built in order to study the technique of pre-consolidation by vacuum (trial Embankment A) and by vacuum and heating (trial Embankment B). The site had a water table at a depth of 1.5m, inside the weathered crust, and a flow of water towards the underlying till layer. Results of this experiment are summarized in Figure 16 where good correspondence is seen between the applied vacuum pressure, vertical strains and pore pressure.

The vacuum technique is a good option for very large areas, where often the superficial very soft clay has very low undrained strength. For these sites, like most Brazilian deposits, the technique presents two advantages: it is possible to attain high stresses without rupture and there is a significant economy with the fill material generally used as surcharge. Another advantage deals with the logistics of obtaining fill material near the embankment area and the transportation and disposal of the material, which could be of great importance for the construction schedule.

Some studies on the subject are being conducted at present in Brazil to use the vacuum technique at nearby shore sites.

Vacuum preload could also be applied without the PVC membrane, however, in this case the top of the vertical drains are airtight and linked to horizontal tubes going directly to the pump system. In this case, all drains are checked individually with regard to the leak.

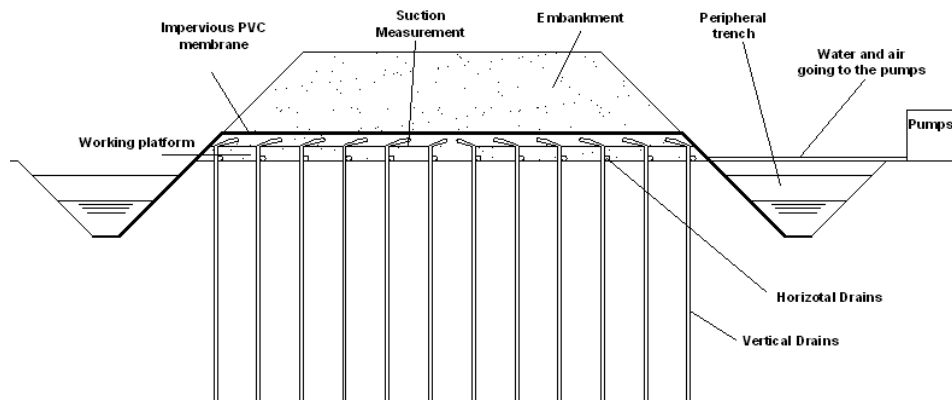


Figure 15. Vacuum consolidation scheme with membrane (Marques and Leroueil, 2005).

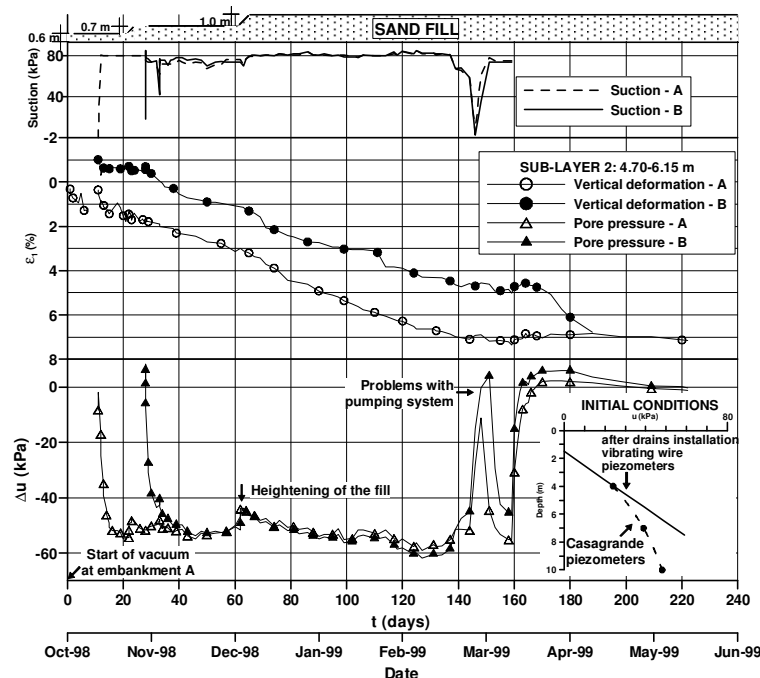


Figure 16. Vertical deformation of sub-layers 2A and 2B (fill B) and pore pressure measured in the middle of these layers, at a depth of 5.4 m (Marques and Leroueil, 2005).

## 11 CONCLUSIONS

Brazilian very soft clays present a wide range of geotechnical parameters due to high organic matter content, which has to be considered for the design over these soft soils. Also, the difficulties regarding sample quality have led to the increasing use of field tests complemented by laboratory tests.

The construction methodologies used at Barra da Tijuca and Recreio are mainly embankments on drains or piles. New techniques are, however, being considered for these soft soils, such as granular columns and vacuum consolidation.

The design of reinforcement embankments on soft clays requires adequate understanding of the overall deformation of the system embankment-foundation-reinforcement, since the maximum service value may not be mobilized in situ even when failure is reached.

With regards to designing embankments over piles with geogrids, it should be noted that geogrid deformation may occur after construction.

## ACKNOWLEDGMENTS

The geotechnical properties of the Barra da Tijuca and Recreio deposits, summarized herewith, result from a large number of studies, particularly MSc and DSc theses. The authors are very much indebted to all involved in these studies, which made the preparation of this paper possible. The authors would like to express their thanks for the continuous financial support received from PRONEX-CNPq, Rio de Janeiro State Agency FAPERJ and National Institute of Science and Technology (INCT-CNPq).

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