

Overview of Brazilian construction practice over soft soils

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ABSTRACT This paper presents an overview of Brazilian experience with construction techniques on soft clays. These are often thick fluvio-marine deposits found along the Brazilian coast, where there is a higher concentration of population and industry. The most used site investigation techniques are summarized, some important case histories are presented and the main construction techniques used in Brazilian practice and their performance are briefly described.

1 INTRODUCTION

About half of the population in Brazil lives in a 100 km strip along the coast, where ports and industries are also concentrated. With increasing development, lowlands areas are increasingly occupied, where soft clay deposits may reach a thickness up to 40 m.

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 (Pacheco Silva, 1953; Lacerda et al., 1977; Pinto, 1994; Massad, 1994; Almeida and Marques, 2003; Coutinho and Belo, 2005).

This paper presents an overview of the main techniques used in Brazil for construction on these very soft fluvio-marine deposits. Design and calculation methods adopted in Brazilian practice are beyond the scope of the paper and may be found elsewhere (e.g. Almeida, 1996; Schnaid et al., 2001; Massad, 2009).

2 TECHNIQUES FOR SITE INVESTIGATION

Brazilian soft clay deposits are generally quite compressible, with a compression ratio ($C_c/(1+e_0)$) typically around 0.50. The top clay layer

usually presents very low strength (design strength can be as low as 3 kPa) and very high water content (w_0), which may reach values of 900% in the top layers (Almeida et al., 2008b).

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 σ'_{vm}). The design strength is obtained using the Bjerrum (1973) correction factor μ , which is usually in the range 0.70–0.60 due to the high plasticity of most Brazilian soft clays.

Table 1 presents the main site investigation techniques carried out over soft soils by Brazilian practitioners. Dilatometer (DMT), pressiometer (PMT) and T-bar tests are less routinely carried out.

The Brazilian Code for sampling of soft soils (ABNT, 1997) allows the use of a 75 mm sampler in exceptional situations, but recommends the use of a 100 mm diameter sampler which is mostly used in practice. The sample preparation technique proposed by Ladd and De Groot (2003) for laboratory extrusion has been used in more recent jobs. Sample quality has been evaluated by the method of Lunne et al. (1997) adapted to Brazilian clays (Coutinho, 2007). As it is not easy to obtain good undisturbed samples of the very soft Brazilian clay, *in situ* tests (Schnaid and Danziger, 2000; Coutinho, 2008) are increasingly used.

TABLE 1 TESTS AND GEOTECHNICAL PARAMETERS (ADAPTED FROM ALMEIDA, 1996)			
	Test	Main objectives	Parameters (*)
Laboratory	Consolidation	Settlement magnitude and Settlement x time	$C_c, C_s, \sigma'_{vm}, c_v$
	Triaxial CU	Stability analysis and modulus for 2D FE analys	S_u, c', ϕ', E_u
Field	SPT	Stratigraphy	$w_0 (**)$
	Vane	Stability analysis	$S_u, S_t, (OCR)$
	Piezicone - CPTu	Stratigraphy, Settlement x time from dissipation tests	S_u profile, c_h (OCR; K_0 ; E_{oed})

(*) other parameters that could be obtained from test are given between brackets;

(**) w_0 is determined from the soil collected at the tip of the SPT sampler and could be used in correlations.

3 SOME IMPORTANT BRAZILIAN CASE HISTORIES

Some well-instrumented and analyzed case histories in Brazil executed

during the period 1974–1994 were very useful in forming the basis of soft clay engineering practiced in the country in subsequent years.

Three cases are selected and briefly described below: the Sarapuí Test Embankment I (Ortigão et al., 1983), Juturnaíba Test Embankment (Coutinho and Lacerda, 1989) and the Sergipe Breakwater (Brugger et al., 1997, 1998). The latter was built in stages during four years but the Sarapuí and Juturnaíba embankments were taken to failure in about one month's time. This construction time is usually close enough to an undrained condition in practical terms as far as stability analysis is concerned. However, this may not be the case as far as deformation analysis is concerned, as the results to be presented below illustrate.

The Sarapuí Embankment I (Ortigão et al., 1983) was built until rupture, 30 days after beginning of loading. Subsequent stability studies of this test embankment (Almeida, 1985; Sandroni, 1993) indicated that the actual failure took place when the embankment reached a height of 2.5 m. Almeida and Marques (2003) have described extensive comparative analysis between laboratory and *in situ* test results and field observations.

Back analysis of Embankment I rupture using a 3D failure surface has shown that the correction factor for Sarapuí clay is $\mu = 0.70$ (Sandroni, 1993; Pinto, 1992). This value lies slightly above (for a typical IP = 80) the μ versus IP relationship proposed by Azzouz et al. (1983), for this type of failure, as shown in Figure 1, where other Brazilian case histories are also included, most of which lie on the vane strength correction curves, with the exception of the Juturnaíba test embankment.

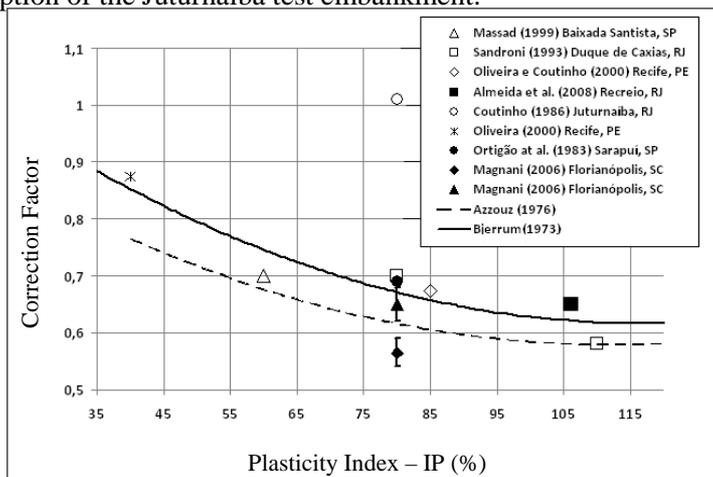


FIGURE 1. Vane strength correction versus plasticity index Brazilian case

histories.

Embankment I was analyzed numerically (Almeida and Ortigão, 1982) using the modified Cam-clay model. Analyses were performed for both undrained and partially drained conditions (using Biot 2D consolidation theory). Good overall agreement between measured and numerical values of base settlements and pore pressures was observed for the partially drained case, but not for the undrained condition, as shown in Figure 2. Therefore, it seems that even for a one month long embankment loading some drainage may have taken place.

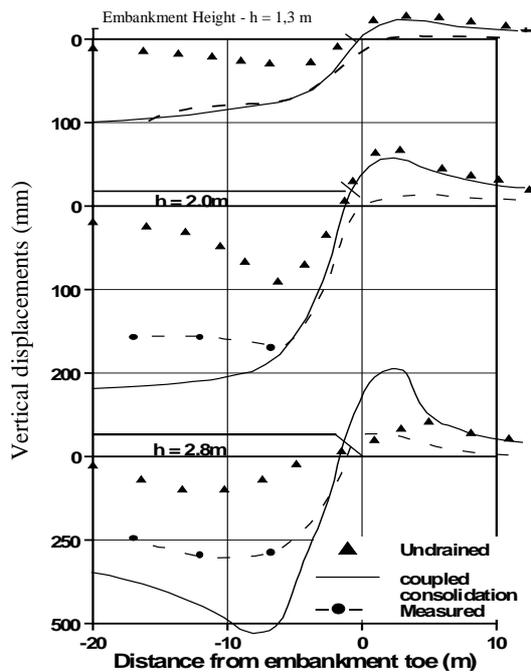


FIGURE 2. Settlements at the embankment I base at various stages of construction.

The Juturnaíba test embankment was another important case history (Coutinho and Lacerda, 1989). This test embankment was brought to failure with the aim of gaining a better understanding of the foundation behaviour of the Juturnaíba dam. The local subsoil consisted of a 7.5 m thick soft clay layer, composed of six well studied sub-layers. The Juturnaíba test embankment was modelled numerically (Antunes, 1996) using the modified

Cam-clay model with 2D Biot consolidation for the soft clay layers. Figure 3 shows measured and predicted displacements for two embankment heights where good agreement is observed. Good overall agreement between measurements and predictions was also obtained for horizontal displacements and pore pressures.

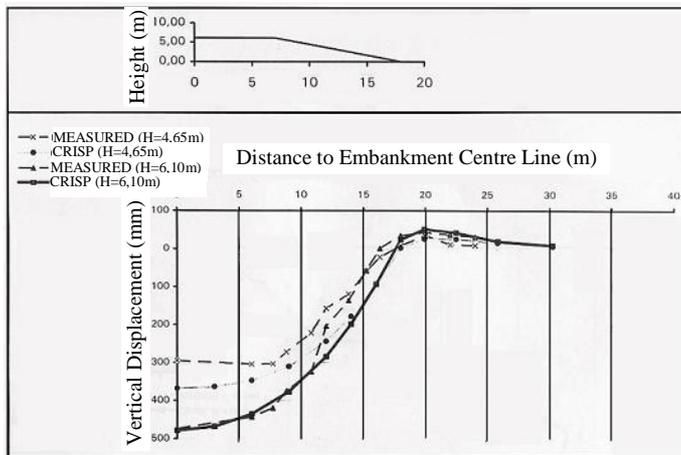


FIGURE 3. *Computed and measured vertical displacements at the base of the Juturnaíba embankment (Antunes, 1996).*

The Sergipe Breakwater is located 2.5 km from the Northeast coast and has a total length of 800 m and height of 15.25 m (10 m underwater). The embankment was built in 5 stages, until the desired elevation was reached. The geotechnical investigations carried out showed the presence of about 4 m of sand over a 7 m thick soft clay layer. The geotechnical characteristics of the local soft clay were summarized by Sandroni et al. (1997). Brugger et al. (1998) presented a numerical analysis of the construction of the breakwater using modified Cam-Clay type computer models for the clay layers.

Figure 4 shows the location of the extensometer (SD-03), installed before the 4th stage construction, along with a comparison of the instrument readings with numerical analysis results. The authors noted that the settlements predicted for the 5th stage are a little higher than those measured, which is a situation that was reversed after 1300 days of reading, and the largest settlements in the field (final expected value of 1.45 m) can be explained by secondary compression, which was not predicted in the

numerical model.

The three cases described above used more conventional construction techniques, with no special requirements regarding stability or settlement control such as reinforcement or vertical drains. Thus they could be more easily analyzed and set the basis for more elaborate construction techniques used subsequently in the country in softer and more compressible clay deposits.

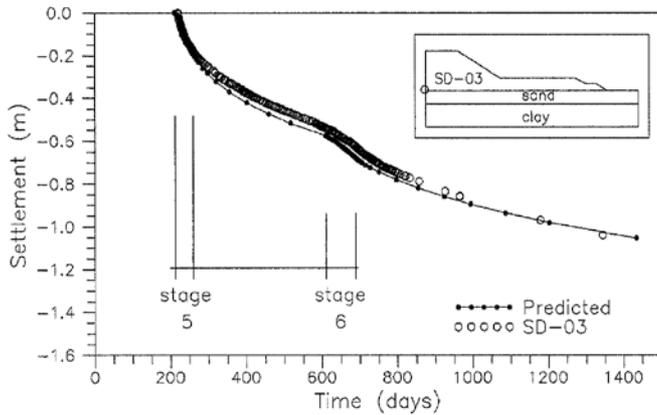


FIGURE 4. *Predicted and measured settlements at Sergipe breakwater (Brugger et al., 1998).*

4 CONSTRUCTION TECHNIQUES

The construction techniques mostly used nowadays to build embankments over soft clays in Brazil are outlined in Table 2. Some of these techniques will be discussed in greater detail in this paper. The methodology mostly used in relation to Brazilian very soft soils combines the use of stage construction, berms, geogrid reinforcement and vertical drains under the embankments.

Although these more conventional techniques have been in Brazilian practice for decades, other construction techniques have been introduced more recently in Brazil. Examples of these are piled embankments on geogrid platforms, use of granular columns (encased or not) and lightweight fills which are described in some length in the present paper. Studies are also under way in some Brazilian projects to use other solutions in the near future such as vacuum preloading.

Jet-grouting and shallow mass stabilization (Stabtec® technique) have also been used in the country in various soft clay projects but is beyond the scope of this paper to discuss these techniques. Other techniques such as lime columns, deep mixing and dynamic compaction have not yet been used in the country, to the authors' knowledge.

TABLE 2 CONSTRUCTION METHODS

Method	Comments	Brazilian studies
Soft Soil Removal	Environmental impact on neighbouring areas are the main concern.	Vargas (1973); Cunha and Wolle (1984)
Soft soil expulsion by controlled failure	Often used for shallow deposits; method highly dependent on local experience; the remaining soft soil left should be assessed by borings.	Zaeyen et al. (2003).
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. (2008)
Vertical drains and/or fill surcharge	Used to accelerate settlements; wide accumulated experience. Temporary surcharge useful to decrease/suppress secondary settlements	Almeida et al. (2001), Sandroni and Bedeschi (2008)
Equilibrium berms and/or reinforcement	Quite commonly adopted; necessary to assess whether specified tensile force is actually mobilized in situ.	Palmeira and Fahel (2000), Magnani et al. (2009)
Light fill materials	Favorable for tight schedules, relatively high cost; its use has increased recently.	Sandroni (2006)
Piled embankments	Favorable for tight schedules, may present different layouts and materials, for very small settlements.	Almeida et al. (2008b), Sandroni and Deotti (2008)
Granular columns (Granular piles)	Granular columns may be encased or not with geotextiles; settlement acceleration owing to draining nature of granular columns; geogrids are sometimes placed above the granular piles.	Almeida et al. (1985), Mello et al. (2008) Garga and Medeiros (1995)
Vacuum preloading	May partially substitute fill surcharge; horizontal displacements much lower than for standard surcharge.	Marques and Leroueil (2005)(*)

(*) Studies carried out by Brazilian researchers on clay deposits of other countries.

4.1 Vertical drains

Vertical drain has 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 1.5 m 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.

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 0.60 m thick drainage blanket provided support for the equipment used to install a prefabricated vertical drain (PVD) in a 1.70 m 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 $\Delta S_u / \Delta \sigma'_v = 0.22$. Values of c_h were measured and the generally good agreement obtained in dissipation tests, using a piezocone to compare with field values, using Asaokas Method (1978) modified by Magnan and Deroy(1980), suggests that during the period of analysis secondary consolidation was negligible compared to primary consolidation, as shown in Table 3.

Method	Range of c_h variation ($10^{-8} \text{ m}^2/\text{s}$)	c_h (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

Figure 5 shows the settlements at the centre of an embankment constructed over a 1.4 m triangular grid of PVD, at Recreio site, Rio de Janeiro city (Almeida et al., 2008). In the frontal region of the site, where the PVD were installed, the soft clay thickness varied from 4 to 11 m and due to its high compressibility, in order to stabilize a 3 m high embankment, it was necessary to build a 7 m thick embankment in three stages, over vertical drains and reinforcement at the borders of the embankment. Due to the high compressibility and low undrained strength of Brazilian soft clays, and also the high values of secondary settlements, the use of PVD associated with surcharge can be very expensive due to fill volumes,

reinforcement and the long construction schedules required. All these factors have led to the increasing use of alternatives construction techniques.

4.2 Reinforced embankments

The use of reinforcement at the base of embankments on soft clays started in Brazil in the 1980s (e.g. Ortigão and Palmeira, 1982) and since then has increased rapidly. In the early years, lower strength unwoven geotextiles were more widely used. However, in the last fifteen years higher strength geosynthetics, woven geotextiles and geogrids have become available and have gained wider acceptance. A number of case histories of reinforced embankments in Brazil have been reported and some of these are briefly described below.

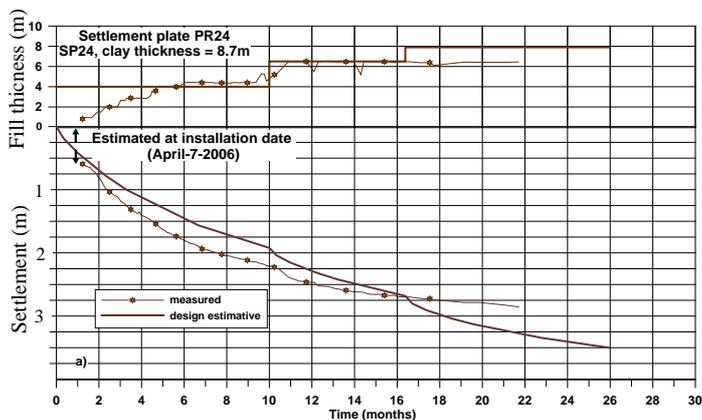


FIGURE 5. Typical settlement plate data of Recreio embankment (Almeida et al., 2008).

Palmeira and Fahel (2000) presented the performances of four geosynthetic reinforcement highway abutments built over soft foundations, including a geogrid reinforced bridge at BR-101 in south Brazil. In this case the new embankment was built with part of its width resting on the old embankment, as shown in Figure 6. The instrumentation showed that the reinforcement was very effective at reducing lateral movement of the soft foundation soil. In spite of the large deformations imposed on the structures, the overall performance of this embankment was satisfactory, mainly because of the flexible nature of the geosynthetic reinforcement used.

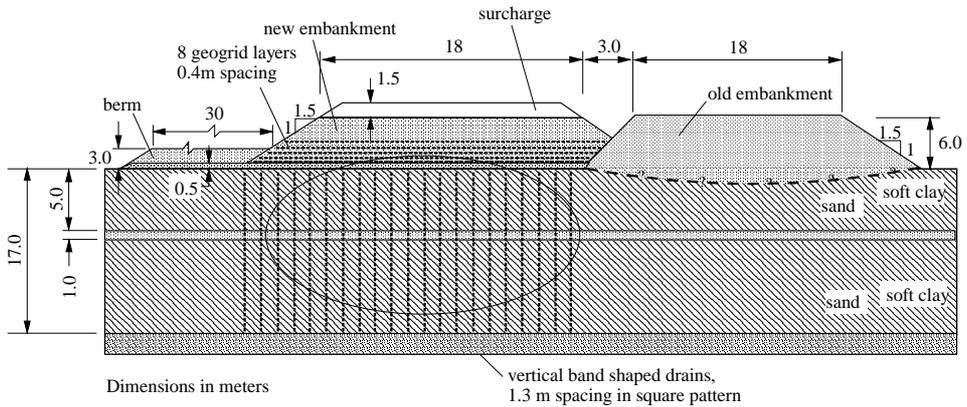


FIGURE 6. Cross-section along the transverse direction of the embankment (Palmeira and Fabel, 2000).

Fabel and Palmeira (2002) described another case history of reinforced bridge abutment which almost collapsed before its planned final height had been reached. Two main reasons for the instability of the embankment were identified in this case: the use of a uniaxial geogrid with quite different tensile strength values along the longitudinal and transverse directions and the rapid rate of construction.

Magnani et al. (2009, 2010a) reported the construction of two reinforced test embankments built to failure with measurement of the tensile forces applied to the geotextile reinforcement. The performances of these embankments are summarized in a paper presented to this symposium (Magnani et al., 2010b), so further information is not presented here.

4.3 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 9 km², located 50 km from Rio de Janeiro city. Due to vertical stresses equal to 340 kPa imposed by the 13 m 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 15 m deep, reaching 17 m in some places. It was estimated that, without the gravel columns, the settlements would reach values around 1.5 m. Figure 7 shows the geometry and geotechnical model used for the solution with gravel columns.

The gravel columns were installed at intervals between 1.75 m and 2.20 m in a square arrangement, with an average length of 12 m and a total length of approximately 400,000 m. 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 3 cm per month were observed in the iron ore stockyard, but decreased after a period of approximately 4 months (Weigner et al., 2009).

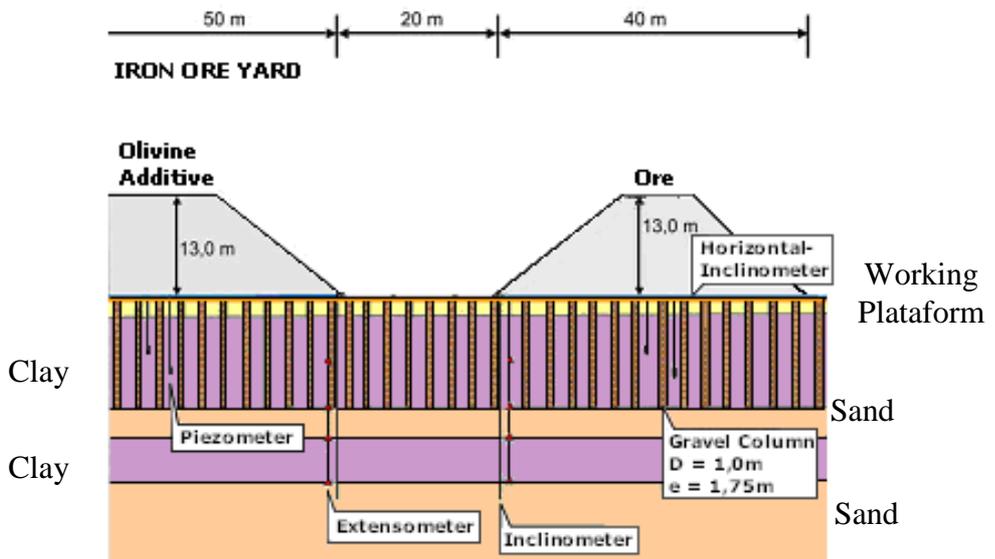


FIGURE 7. Geometry and geotechnical model used for the solution with gravel columns (Wegner et al., 2009).

Other stone column works carried out in Brazil recently (Felix, 2009) include the construction at the BR-101 highway toll station in Araquari, State of Santa Catarina, of 10,305 m of column (diameter 0.80 m; length 7–

10m; spacing 1.80–2.50m). Also, 5378 m of column were installed for the enlargement of the Ubá municipal airport, State of Minas Gerais (diameter 0.70 m; length 6–9 m; spacing 3.5–5.0 m).

4.4 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 Jose dos Campos, 100 km 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 4 shows the range of geotechnical parameters of these two clay layers.

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 8 shows the final stages of implementation of an encased column installation and the column inside. Table 5 summarizes the characteristics of the columns used and some monitoring results.

TABLE 4 RANGE OF GEOTECHNICAL PARAMETERS LABORATORY

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



FIGURE 8. Finished geosynthetic encased sand column.

Characteristics	Values
Diameter of columns	0.70 m
Geotextile used in the jacking	Final stress of 50 kN/m and rigidity of 1000 kN/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

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 6.

Soil parameters	Values
Thickness of clay layers	Up to 20 m of clay layer with 8 m or more compressible clay
Oedometric modulus	0.2–0.5 MN/m ²
c_v	$2-4 \times 10^{-8}$ m ² /s
S_u	5–15 kN/m ²
Columns characteristics	Values
Diameter of columns	0.78 m
Length of columns	10 to 12 m
Spacing	2.0 x 2.0 m
Geotextile used in the jacking	Ringtrac R 100/250 and 100/275

4.5 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 nowadays 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) have been reported, as exemplified in Figure 9.



FIGURE 9 Use of EPS blocks as lightweight fill material: a) cross section showing blocks and protective cover; b) finished pavement (Lima and Almeida 2009).

4.6 Piled embankments

Piled embankments with a geogrid or concrete platform can be an alternative with technical, economical and schedule advantages, when compared to a reinforced embankment with berms on vertical drains, mostly due to the high value of fill material necessary for more conventional solutions. For very soft clays like the ones described here, piled embankment solutions for roads may be more economic than embankments on vertical drains for soft layer thicknesses greater than about 12 m (Almeida and Marques, 2004). However, it is important to analyze each case, since stratigraphy, soil parameters and fill costs vary over a wide range as shown by Nascimento (2009) and thus the clay thickness threshold could be lower than 12 m for some deposits.

The performance of piled embankments built over Barra soft clay deposits were described by Almeida et al. (2007, 2008) and Sandroni and Deotti (2008). The prevailing mechanism in these cases was the membrane effect due to the low ratio of embankment height to pile caps span. When the membrane effect prevails, geogrid deformation should be expected in the future, and the pavement should be light and flexible, allowing future settlements and maintenance.

Based on the performance of these piled embankments, it was observed in very soft clays that the working platform causes important settlements, the magnitude of which is not negligible.

Palmeira and Fahel (2000) describe the behavior of the reinforced abutment with the highest reinforced structure (7.3 m), where no settlement was observed due to the efficiency of the piles along the embankment base.

However, horizontal displacements of the wall crest of the order of 40 mm (approximately 0.55% of the wall height) were measured.

4.7 Vacuum Consolidation

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 1.5 m, inside the weathered crust, and a flow of water towards the underlying till layer. Results of this experiment are summarized in Figure 10 where good correspondence is seen between the applied vacuum pressure, vertical strains (measured by special vertical extensometers) and pore pressure.

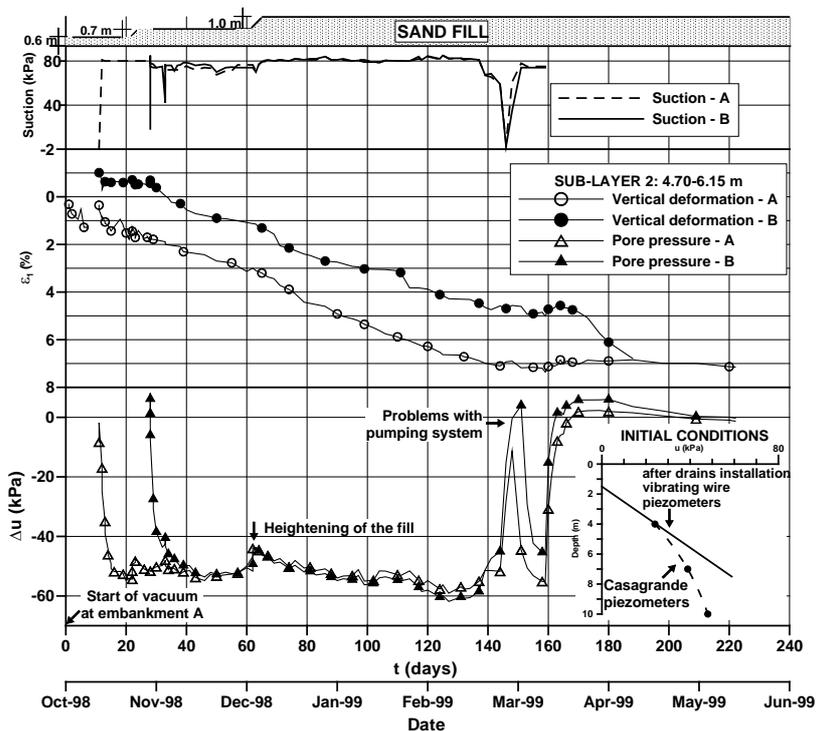


FIGURE 10. Vertical deformation of sub-layers 2A and 2B (fill B) and pore pressure measured in the middle of these layers, at a depth of 5.4 m (Marques and Leroueil, 2005).

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. The main drawback with the use of vacuum consolidation in Brazilian deposits is the presence of sand layers in soft clay deposits (Massad, 2009) as in these cases it is necessary to use plastic diaphragm walls to maintain system tightness.

5 INSTRUMENTATION FOR MONITORING STABILITY

The instrumentation for the construction of stage embankments consists mostly of settlements plates and inclinometers, and sometimes piezometers. To evaluate the gain in clay strength between stages, vane tests are carried out. The use of extensometers is generally associated with a more extensive instrumentation programme, generally for important projects or research.

Settlement control is usually based on settlement and piezometer measurements (e.g. Almeida and Ferreira, 1992; Sandroni and Bedeschi, 2008) but a discussion of these topics is beyond our scope.

For stability control based on inclinometer results, Almeida et al. (2000) proposed that, for distortion velocities (v_d) higher than 1.5%/day, there are regions where plasticization occurs and rupture could be imminent, and the interruption of surcharge is recommended. For v_d values between 0.5%/day and 1.5%/day, special care should be taken, but for $v_d \leq 0.5\%/day$ only the continuous monitoring is required until stabilization.

Sandroni et al. (2004) proposed an empirical method for the evaluation of the safety of embankments over soft soils. The procedure takes into account that the volume due to settlement (V_v) and the volume due to horizontal displacement (V_h), for undrained conditions and plane analysis, should be the same. They proposed that:

- for undrained behavior V_v/V_h are close to one, and non planar undrained behavior $V_v/V_h > 1$;
- for drained behaviour $V_v/V_h > 1$.

During loading V_v/V_h could vary since field behavior is a mix of undrained and drained behavior and deformations are not planar. When close to rupture V_v/V_h tends to 1 very quickly, however when loading is interrupted V_v/V_h increases with time, going to stabilization

Magnani et al. (2008) applied the procedures of Almeida et al. (2000) and Sandroni et al. (2004) to three experimental embankments assumed to rupture, one with drains and reinforcement, another with reinforcement and a third which was conventional (without drains or reinforcement) and obtained satisfactory results for the three embankments.

6 CONCLUSIONS

This paper describes techniques to build embankments on very soft Brazilian clays. The used site investigation techniques involve *in situ* tests (mainly SPT, CPTu, VT) and laboratory tests (oedometer and triaxial tests), which require careful sampling and specimen preparation in order to get reliable parameters for very soft clays. The design strength profile is mainly based on the corrected vane strength supported by normalized undrained strength equations. Coefficient of consolidation design values are usually based on both oedometer tests and CPTu dissipation tests.

The Brazilian case histories of Sarapu , Juturna ba and Sergipe breakwater, which were performed in the 1970s to early 1990s, are well-studied and were very useful in understanding the field behavior of Brazilian soft clays.

A number of more elaborate construction techniques involving the use of reinforcement, berms, vertical drains, temporary surcharge and stage construction are then described by means of practical applications. In very soft Brazilian clays, it is common to make combined use of all these techniques in the same project or site. However the overall construction involved is still extensive due to the nature of these very soft deposits.

Less conventional construction techniques such as piled embankments with geogrid platforms and soil improvement techniques using granular columns and lightweight fills have also been adopted more recently in these

very soft deposits. These less conventional techniques are gaining wider acceptance as they allow shorter overall construction times.

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