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SOILS and ROCKS

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Articles

Soils and Rocks
v. 32, n. 2

Vulnerability of Rockfill Dams to Seismic Hazard

António A. Veiga Pinto

Abstract. The occurrence of a 5.9 magnitude earthquake, in February 2007, that affected two rockfill dams in Portugal, led to a study presented in this paper about the structural performance of rockfill dams under earthquake influence. Taking into account the world knowledge it can be concluded that the dynamic actions have mainly produced some displacements, and the opening of cracks and fissures of reduced development, particularly in the highest zones of those structures. So, up to now there is not any failure of a rockfill dam related to seismic actions. This paper also presents an expedite method for predicting vertical displacements based on a parameter related with quantification of the dynamic action and with settlements measured on prototypes.

Key words: rockfill dam, seismic hazard, magnitude, displacement, remedial measure.

1. Introduction

Portugal is one of the zones with higher seismic activity, in the world, and particularly in Europe. In February 2007, a 5.9 magnitude earthquake occurred, which, according to the Dam Safety Regulations, led to the analysis of the structural behaviour of such structures that are located closer to the epicentre. For the purpose, both the inspection and the reading of devices of the observation system of Beliche and Odeleite rockfill dams, located on the South-east part of Algarve, were performed. Nevertheless, no effect was observed on those dams as a result of dynamic actions. Subsequently, it was considered that it could be of interest to perform a study about the structural behaviour of such type of dams under the influence of seismic actions.

Earthquakes particularly produce large displacements in dams. Therefore, this paper presents an expedite method for predicting those displacements on the basis of the magnitude of dynamic loads.

2. February 2007 Earthquake

Various countries are adopting the criteria defined by the International Commission on Large Dams for inspecting dams after earthquake occurrence (ICOLD, 1988 and Lamontagne & Dascal, 2006). Considering the earthquake occurred in February 2007, in the South of Portugal, a decision was made to inspect Beliche and Odeleite rockfill dams.

Beliche Dam is 54 m high and has a clay central core. Odeleite Dam is 65 m high and has an upstream reinforced concrete membrane as waterproofing system. Figures 1 and 2 present, for each dam, the standard cross-section and a general view.

Ambraseys *et al.* (1996) and Ambraseys & Simpson (1996) proposed attenuation relationships that can be used to estimate the effect of seismic action in terms of response spectra. So, the peak ground motion can be obtained by the following equation:

$$\log(a_j) = C_{1,j} + C_{2,j}M + C_{3,j}R + C_{4,j}\log R + C_{A,j}S_A + C_{S,j}S_s + \sigma_j P \quad (1)$$

in which a_j is the acceleration ground motion spectra to a frequency j (in g), M the magnitude and R the distance to the epicentre, s_j is the standard deviation of the logarithmic law and P a coefficient equal zero for the average value of a_j. $C_{1,j}$, $C_{2,j}$, $C_{3,j}$, $C_{4,j}$, $C_{A,j}$ e $C_{S,j}$ are numerical coefficients of attenuation estimated by the regression analysis for each frequency and S_A e S_S values are related with the geological local conditions.

For Beliche and Odeleite Dams it was estimated a very approximately period, T, of 0.50 s corresponding to a frequency j. This value was used to obtain from tables the coefficients of the attenuation Ambraseys' laws. The distance of Beliche and Odeleite Dams from the epicentre is approximately 340 km and the earthquake magnitude was 5.9. Therefore, a very low value for the peak ground acceleration was estimated, less than 0.002 g, and due mainly to the fairly attenuating effect of the dynamic impact with distance.

On the two dams, both the visual inspection and the reading of the magnitudes of the observation system were performed, but it was not possible to observe any damage or significant evolution in those magnitudes, which could indicate that these were related with the dynamic action.

3. Performance of Rockfill Dams Under Earthquake

Earthquakes have led to a reduced number of dam failures (ICOLD, 2002). In fact, only a few dozen tailings dams, built by hydraulic fill, collapsed due to earthquakes. Furthermore, there has been no loss of human lives due to the failure of dams as a result of seismic action. Experience has shown that fill dams, compacted in accordance with state-of-the-art techniques, have presented no problems under dynamic earthquake action. Particularly, rockfill dams have exhibited

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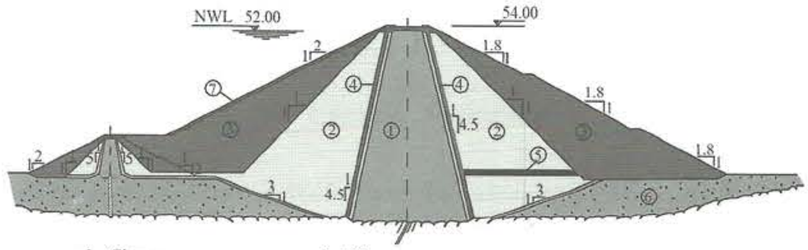

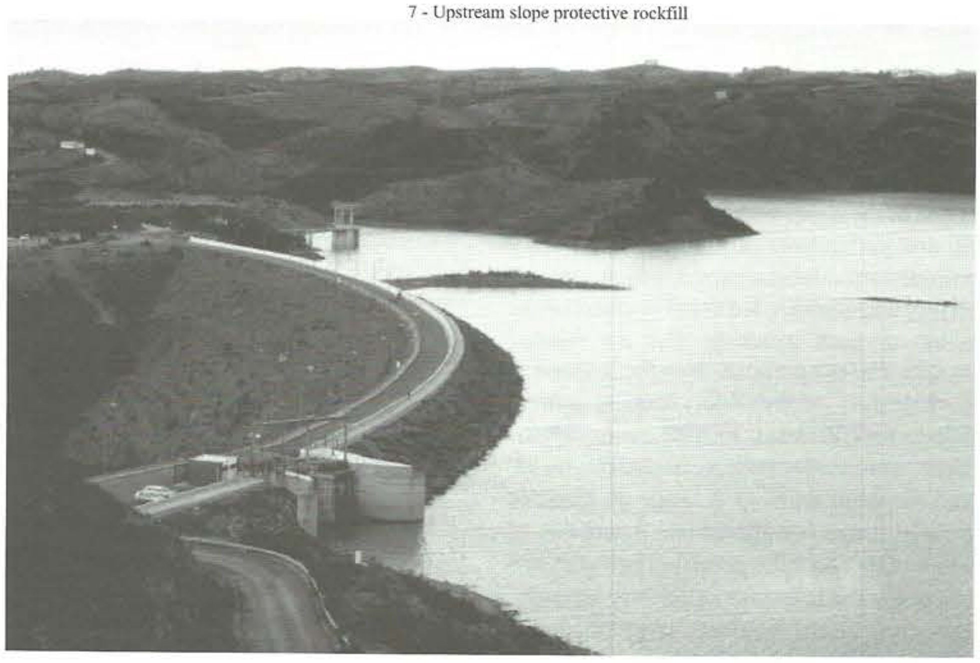

Figure 1 - Beliche Dam - Typical cross section and general view.

a remarkable performance under seismic actions, both the earth-rockfill type with clay core (Ohmachi, 1994) and those consisting of an upstream membrane (Bureau *et al.*, 1985). Only tailings dams, consisting of sand or silt poorly compacted by their deposition by hydraulic fill, have suffered damages caused by earthquakes (ICOLD, 2002).

The next few paragraphs present some aspects referring to rockfill dams that were subject to seismic actions.

3.1. Malpasso Dam

This is a 78 m high dam with an upstream reinforced concrete membrane. The downstream shoulder was built by dumped rockfill and the upstream face consists of both hand-placed and mechanically-placed rock blocks, which makes it possible to achieve a high slope inclination (0.5H:1V). The dam has a concrete membrane, of reduced thickness, at its central zone. A 6.0 magnitude earthquake in 1938, with full reservoir, caused, at the dam crest, a maximum settlement of 7.6 cm and a horizontal displacement towards downstream of 5.1 cm (Ambraseys, 1960). Additional information on this case-history can be found in Table 1.

3.2. Cogoti Dam

Cogoti Dam was built with dumped rockfill and the waterproofing system consists of a reinforced concrete membrane on the upstream face. In 1943, it was subject to an 8.3 magnitude earthquake, which led to a maximum settlement, at the crest, of 28.1 cm. Furthermore, some blocks also fell from the rockfill of the downstream slope, but no fissures on the membrane or an increase in the infiltration flow rates were observed (Cooke, 1984).

3.3. Miboro Dam

This dumped rockfill dam, which is 131 m high and has an upstream inclined core, was subject, in 1961, a year after its construction, to a 7 magnitude earthquake. The epicentre was located at just 16 km from the dam. The displacements measured at the crest were less than 5 cm and no structural damage was observed (Seed, *et al.*, 1978). Subsequently, in 1969, a 6.6 magnitude earthquake occurred, and, similarly, no effect on the work was observed.

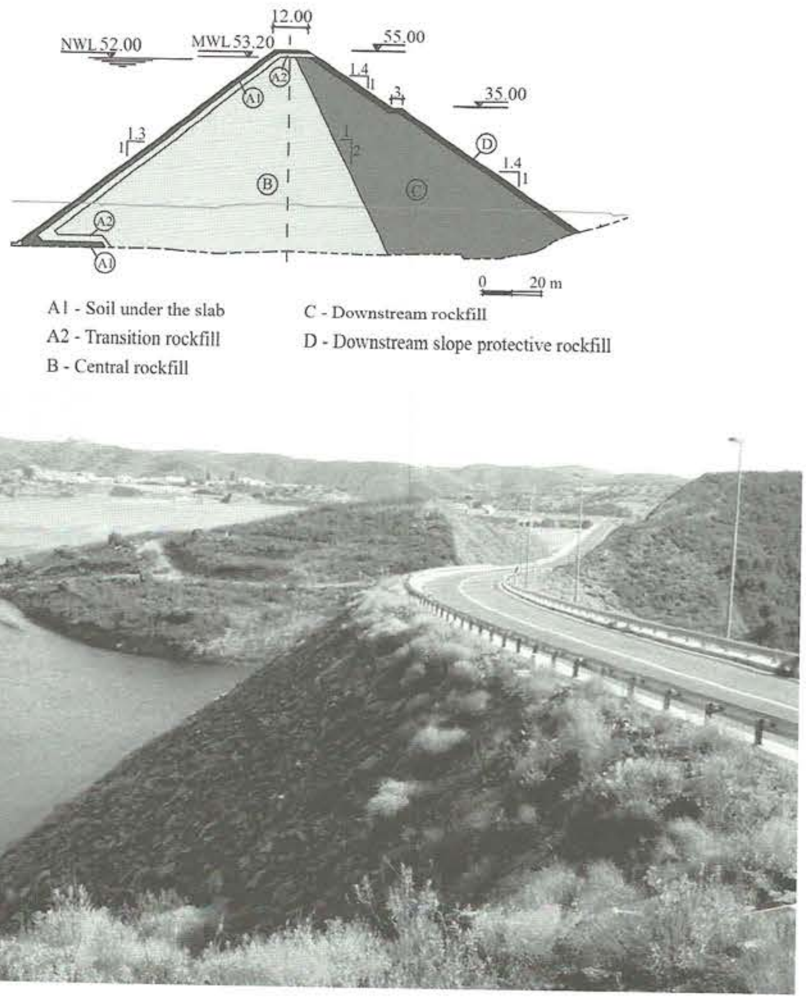

Figure 2 - Odeleite Dam - Typical cross section and general view.

3.4. Minase Dam

This 67 m high dam with an upstream reinforced concrete membrane was built with dumped rockfill and substantial sluicing water. This structure was subject to an earthquake, in 1964, having had a 7 mm settlement. In the same year, another earthquake occurred (Niigata earthquake), with a 7.5 magnitude, and which led to a maximum settlement and horizontal displacement, at the crest, of 61 mm and 40 mm, respectively. Some cracking was observed on the upstream membrane and on the pavement of the road crossing the crest. The infiltration flow rates increased from 90 L/s to 220 L/s, approximately. The dam was subject to new earthquakes, in 1970, 1978 and 1983, which did not produce further damage or a variation in the magnitudes of the monitoring scheme (Matsumoto *et al.*, 1985a).

3.5. Oroville Dam

This structure consists of gravel shoulders and a clay central core. It is the highest fill dam (235 m) in the USA. An earthquake occurred in 1975, with a 5.7 magnitude and a peak 0.12 g acceleration at the crest, which caused a maximum vertical displacement at the crest of 0.9 cm and a settlement on the foundation surface of a few dozen centimetres (Vrymoed, 1981).

3.6. El Infernillo Dam

This dam has a central core and is 148 m high. It was subject to a 7.6 magnitude earthquake and even though it was located at a significant distance from the epicentre (110 km), the accelerations measured at the foundation and at the crest corresponded to about 0.12 g and 0.35 g, respectively. A maximum settlement of 13 cm at the crest was measured, on the upper part of the fill, *i.e.*, for a crest thickness of 40 m. A few cracks were also detected on the dam surface (Romo & Resendiz, 1981).

3.7. La Villita Dam

This 60 m high rockfill structure with a clay central core was subject to earthquake action in 1979, of which the epicentre was at 110 km from its location, and, similarly to El Infernillo Dam, significant seismic accelerations were

Table 1 - Characteristics and effects of seismic actions on rockfill dams.

Dam (country), year of completion (H (m))	Type of dam	Slope (H/V) Upstream Downstream	Earthquake Year	M	R (km)	PGA (g) PCA (g)	Max. displ. crest Vertical (cm) Horizontal (cm)	Reference
Malpasso (Peru), 1936 (78)	CFRD DR	0.5 1.3	1938	6.0	-	0.10 -	7.6 5.1	Ambraseys (1960)
Cogoti (Chile), 1939 (84)	CFRD DR	1.6 1.8	1943	8.3	16	0.20 -	28.1 -	Cooke (1984)
Miboro (Japan), 1960 (131)	ICRD DR	2.5 1.8	1961	7.0	16	0.20 -	3.0 5.0	Seed *et al.* (1978)
Minase (Japan), 1964 (67)	CFRD DR	1.35 2.0	1964	7.5	16	0.08 -	6.1 4.0	Matsumoto *et al.* (1985a)
Oroville (USA), 1968 (235)	CCRD CR	2.8 2.0	1975	5.7	6.9	0.10 0.12	0.9 -	Vrymoed (1981)
El Infernillo (Mexico), 1964 (148)	CCRD CR	1.8 1.8	1979	7.6	110	0.12 0.35	13.0 4.5	Romo & Resendiz (1981)
La Villita (Mexico), 1967 (60)	CCRD CR	2.5 2.5	1979	7.6	110	0.10 0.36	4.5 3.0	Romo & Resendiz (1981)
Leroy Anderson (USA), 1950 (72)	CCRD DR	2.0 2.0	1984	6.2	16	0.41 0.63	1.5 0.9	Tepel *et al.* (1984)
Coyote (USA), 1936 (43)	CCRD DR	3.0 3.0	1984	6.2	25	1.30 -	6.7 3.7	Bennet & Sherbune (1984)
Makio (Japan), 1961 (105)	CCRD DR	3.0 2.25	1984	6.8	5.0	0.4 -	- 10.0	Matsumoto *et al.* (1985b)
La Villita (Mexico), 1967 (60)	CCRD CR	2.5 2.5	1985	8.1	-	- 0.21	30.0 -	Pina *et al.* (1985)
Anbuklao (Philippines), 1956 (129)	CCRD CR	1.75 (2.0) 1.75 (2.0)	1990	7.8	80	- -	80.0 50.0	JSCE (1993)

H - Height. CCRD - Central core rockfill dam. H/V - Horizontal/Vertical. ICRD - Inclined core rockfill dam. M - Richter Magnitude. CFRD - Concrete face rockfill dam. R - Epicentral distance. CR - Compacted rockfill. PGA - Peak ground acceleration. DR - Dumped rockfill. PCA - Peak crest acceleration.

also observed on this dam. However, the permanent displacements measured at the crest did not exceed 5 cm. Furthermore, also similarly to El Infernillo Dam, the duration of the seismic movement at foundation was substantially reduced, comparatively with the vibration time recorded at the crest (Romo & Resendiz, 1981).

In September 1985, another earthquake occurred, with a magnitude equal to 8.1 and an epicentre very close to the dam site. The acceleration at the crest was 0.21 g, and it was possible to observe the opening of many longitudinal cracks on the crest. Cracks of considerable size were also observed on the upstream and downstream faces. In the latter case, one with a 300 m length, a maximum opening of 15 cm and a relative settlement of 20 cm. The vertical displacements on the surface were also significant and reached a maximum value of 40 cm (Pina *et al.*, 1985).

3.8. Leroy Anderson Dam

This 72 m high dam consists of dumped rockfill shoulders and a central core of compacted sand-clayish gravel. In 1984, a 6.2 magnitude earthquake occurred, which caused two longitudinal cracks, each being about 305 m long, as a result of differential settlements between the core and the shoulders. The accelerations measured at the base and at the crest were fairly high, about 0.41 g and 0.63 g, respectively. Both the vertical and horizontal displacements at the crest were less than 2 cm (Tepel *et al.*, 1984).

3.9. Coyote Dam

The dam with a central core was subject, in 1984, to the action of a 6.2 magnitude earthquake, and was located at just 25 km from the epicentre. The seismograph located at the left bank abutment recorded a peak acceleration of 1.3 g and 0.4 g in the horizontal and vertical directions, respectively. Immediately after the earthquake, a few cracks were detected, particularly one on the upstream face, right below the reservoir level, which was at about 1.6 m from the sill of spillway. That crack had a 30 cm width, a 3 m length and a 2 m depth. The maximum settlement of the crest was 6.7 cm. Considering the type of cracks and deformations observed on the dam, it was assumed that its overall safety was not into jeopardy and, therefore, no repair was performed (Bennet & Sherbune, 1984).

3.10. Makio Dam

The 105 m high Dam was subject to a 6.8 magnitude earthquake, in September 2004, and it was located at just about 5 km from the epicentre. The seismic shake was so intensive that it damaged the system for reading accelerations in the foundation. Nevertheless, the earthquake was assumed to have an acceleration of about 0.4 g. The dam presented slight displacements and the opening of small cracks, which did not require the need for repair (Matsumoto *et al.*, 1985b).

3.11. Anbuklao Dam

The 129 m high Anbuklao Dam has a clay central core. In 1990, it was subject to an earthquake of magnitude equal to 7.8, with the epicentre at an 80 km distance. The earthquake caused some damages due to significant displacements, the higher settlement occurred being observed at the rockfill close to the spillway wall, reaching a value of 80 cm and 50 cm, in the vertical and horizontal directions, respectively. Nonetheless, there was no increase in infiltration flow rates and no defective behaviour in the waterproofing system of the dam was detected (JSCE, 1993).

Additional information on the characteristics of dynamic actions and effects caused on the dams is briefly described in Table 1.

4. Predicting Displacements

As previously mentioned, the dynamic actions have led, particularly in rockfill dams, to displacements and to opening of cracks and fissures, particularly in the zones closer to the crest. Therefore, an attempt has been made to deduce, on the basis of case histories, simplified methods for estimating the deformations in dams subject to dynamic actions.

Rockfill dams with upstream membrane have a higher resistance to earthquake than earth ones, because the membrane operates as a waterproof barrier. Thus, the earthquake effects on that type of dams have only been represented by small-scale permanent deformations.

Dumped rockfill dams, due to the less compactness state of the particulate medium, are those having obviously suffered higher deformations. Nevertheless, even for those dams, of which the construction technology is no longer used, no failure has been observed yet.

The analyses to earthquakes in rockfill dams with upstream membrane are mainly intended for estimating the magnitude of displacements and to establish if the possible cracks occurring in the membrane are likely to reduce their watertightness, *i.e.*, to increase the infiltration flow rates. Even though recently new mathematical models have been prepared for designing dam behaviour to earthquakes, the methods related with the performance observed in dams have been among the most reliable ones.

On the basis of observed displacements, Bureau *et al.* (1985) deduced a simplified method for predicting displacements in rockfill dams under seismic actions.

The previously mentioned authors established a non-dimensional parameter, *ESI* (Earthquake Severity Index), which is defined by the equation below (Bureau *et al.*, op. cit.):

$$ESI = A(M - 4.5)^3 \tag{2}$$

In which A is the value of the peak acceleration in the foundation in g's and M is the magnitude in Richter's scale.

Figure 3 presents the relation between the vertical unit displacements and the values of the ESI parameter determined for various dams.

5. Knowledge Gained

The behaviour of fill dams under earthquake action depends on the nature and strength of the material of the dam body and foundation. As previously mentioned, no failure of any dam due to dynamic forces has yet occurred, except for a few hydraulic fill dams, which are very rare and with a construction technology that is been replaced by more reliable ones. The most serious accident caused by an earthquake occurred in 1971, in California, at *Lower San Fernando* Dam (Fig. 4). In fact, even though the dam did

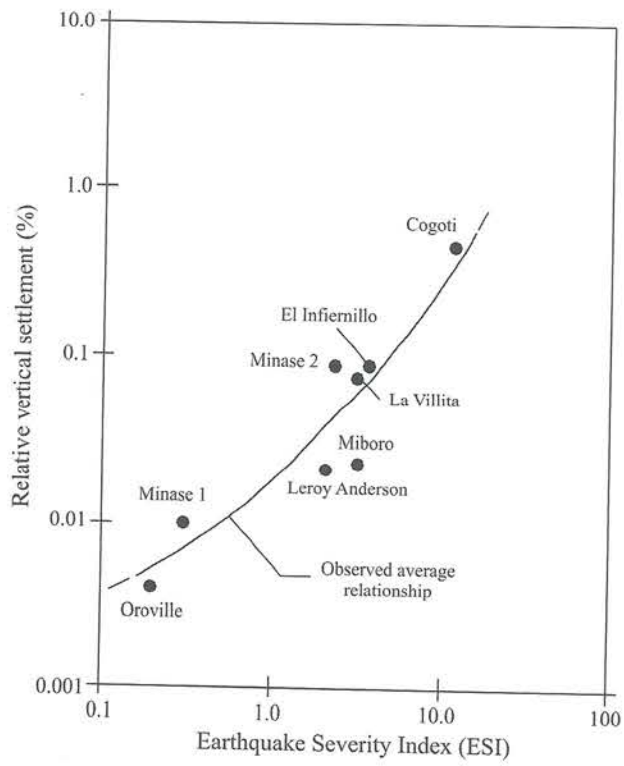

Figure 3 - Relation between settlements observed on rockfill dams as a result of seismic actions and the parameter ESI (Bureau *et al.*, 1985, adapted).

not fail totally, it suffered nonetheless considerable damage, due to liquefaction and, therefore, had to be completely rebuilt (ICOLD, 1974).

The tailings dams, comparatively with other types of dams, are fairly more vulnerable to earthquakes, due to the following reasons:

- they are not compacted;
- they consist of deposits of granular materials of a prevailingly sand size;
- they present a high probability of liquefaction;
- they produce mud seepage zones inside them;
- the observation actions have fewer resources than the observation of conventional dams.

After S. Fernando earthquake, the methods of analysis of the stability of dams to earthquakes were thoroughly revised. Nevertheless, those methods require a sophisticated characterization of materials and mathematical methods, procedures of which the representativeness is difficult to demonstrate, in view of the very poor frequency of response of the prototype to dynamic actions.

Since the S. Fernando Dam accident, it has been considered that such structures and nuclear power plants should be designed for the highest possible hazard, *i.e.*, for the Maximum Expected Earthquake. Nevertheless, on the basis of both the study and the statistical analysis of accidents due to dynamic actions, it has been observed that such method is fairly conservative.

Fill dams designed and built with the present construction techniques have exhibited an excellent earthquake performance. However, hydraulic cracking phenomena may occur in earth dams. Therefore, it is recommended that a particular attention should be paid to the design of seepage control systems (filters and drains), in seismic zones.

Recently, there have been no cases of dams subject to significant earthquakes, except the one occurred in Japan, in January 1995, in Hyogoken-Nambu region (Tamura *et al.*, 1997). The earthquake reached a 7.2 magnitude, leading to extensive damage in residential areas. Fifty dams located at a distance less than 50 km from the epicentre were

Figure 4 - Lower S. Fernando Dam failure.

also affected. Furthermore, a few rather old dams, located close to the city of Kobe were also affected. Immediately after the earthquake, the Authority created inspection teams for assessing the effects of dynamic actions. According to those inspections, it has been observed that no dam had suffered significant damage to the point of requiring corrective measures or emergency repairs (Tamura *et al.*, 1997). There were only a few fissures on the crest pavement of some dams and a slight increase in infiltration flow rates, which, nonetheless, normally decreased and stabilised.

The success of the performance of dams in Japan seems to be due both to the preparation of designs with a careful structural analysis, based on reliable methodologies and criteria, and to a construction with high quality levels, which is associated with a meticulous inspection. Mention must be made of the fact that, whereas the dams had a good earthquake performance, a significant number of other types of structures demonstrated a poor safety, because they exhibited failure and extensive damage (Tamura *et al.*, 1997).

From the analysis of dam failure, it has been verified that rockfill dams are more resistant to earthquakes than earth ones (Yanagisawa, 1990). Due to the absence of pore pressures and to the high values of shear strength of compacted rockfills, that type of dams has a high strength under dynamic actions (Cooke, 1991).

Rockfill dams have exhibited an excellent performance under dynamic actions, even when built with reduced compactness conditions (dumped rockfill), corresponding to a defective technique that was replaced, a few years ago, by compaction with heavy vibrating rollers. The occasional high displacements observed in those dams had no significant influence on their safety (Veiga Pinto, 1990). It is the case of dams mentioned in Table 1. Reference must be made to the fact that, in some of these works, apart from displacements, there are also some cracks, but the increase in infiltration flow rates was only observed in one case. Special mention must also be made to the fact that the total failure of a waterproofing element in rockfill dams with upstream reinforced concrete membrane is not a critical scenario for the stability of this type of dams and, as such, it does not compromise their safety (Wieland & Brenner, 2007).

After the analysis of the performance of dams, when subject to significant dynamic actions (8.1 magnitude at La Villita, 1.3 g and 0.4 g accelerations measured in the vertical and horizontal directions at La Coyotte) and to considerable effects (80 cm instantaneous settlement, at Ambuklao) various authors were led to consider that rockfill dams with clay cores are fairly resistant to dynamic actions.

Furthermore, rockfill dams with an upstream reinforced concrete membrane, when correctly designed and having proper rock foundations are considered to be fairly safe, even when subject to significant earthquakes (Wieland & Brenner, 2007).

The International Commission on Large Dams has promoted studies, namely of a statistic framework, about accidents and failures in that type of works (ICOLD, 1983, 1995). Both from those studies and from others collected in expert bibliography, it is possible to establish the vulnerability of dams to earthquake action.

In Turkey, for instance, as Tosun & Seyrek (2006) mention, just in the last decade, there were about 55000 losses of human lives due to the occurrence of seismic actions in residential areas. Even though there has been the need to perform repairs in some dams affected by those earthquakes, no failure was observed.

From a universe of 14600 large dams built in 33 countries until 1975, about 1014 deteriorations were observed. That sampling corresponded to about 92% of dams built all over the world (excluding China). Thus, 6.9% of deteriorations have been observed, which means, one in every fifteen dams (ICOLD, 1983).

From that universe of 1014 deteriorations, 96 occurred in rockfill dams, 10 failures having been observed in that group. From among the deterioration causes, special reference is made to: deficiency in the outlet works, internal erosion through the foundation, excessive infiltration through the upstream membranes, as well as internal erosion and infiltration through clay cores. Generally, however, the seismic actions had a reduced influence on deteriorations (Veiga Pinto & Rui Faria, 2001).

Foster *et al.* (2000) collected the existence of 136 dam failures from a group of about 11192, which does not include dams in China and in Japan built prior to 1930. Considering all dams, the average failure frequency is approximately 1.2%, to which a historic probability of 4.5×10^{-4}, per dam and per year, corresponds. Those authors only recorded the collapse to earthquake of S. Fernando Dam. Therefore, the average failure frequency due to dynamic actions is only 1.8×10^{-4} (Foster *et al.*, 2000). Even considering these statistical results, a few authors, such as Tosun & Seyrek (2006), assume that more conservative criteria and dynamic analyses should be adopted for dams rather than for other types of structures, in view of the higher potential damages that are likely to occur in the former structures.

Seed *et al.* (1978) performed a detailed analysis about the performance of fill dams under dynamic actions. The study was focused on 127 dams from 3 countries (United States of America, Japan and Russia), which, as a whole, were subject to 16 earthquakes. The magnitudes reached exceptional values of 8.3 in Richter's scale and peak accelerations of 1.3 g.

After the study developed by Seed *et al.* (1978), those authors concluded that: "A knowledge of field performance data of this type can provide a valuable supplement to analytical studies in the final assessment of the seismic stability of an earth dam and in some cases can eliminate entirely the need for analytical studies". They also mention that: "Since there is ample field evidence that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2 g with no harmful effects, we should not waste our time and money analysing this type of problem".

The verification of the performance of dams must be based on dynamic analyses, wherever justified, and particularly, on a discerning reasoning ability. That procedure must be the main factor in the adoption of actions intended for ensuring the stability of dams to earthquakes (Seed *et al.*, 1980).

The requirements defined in the regulations and which are intended to cope with the seismic risk should be better justified. Such is the case of "The guide of seismic risk to UK Dams", published by BRE, in 1992 (Haws, 1994). Thus, in higher risk dams, an acceleration of 0.25 g is required, for a recurrence interval of 3000 years, as well as the execution of special studies with the most updated dynamic analysis models, including the assessment of deformations. Therefore, many owners and engineers responsible for the safety of dams consider those demands as extremely conservative and controversial. Furthermore, they sustain that if those recommendations are to be implemented, for dams built more than 100 years ago, the owners will have to endure considerable costs (Haws, 1994).

In Portugal, the legislation to safety of dams seems also to have adopted rather conservative criteria as regards the design of the earthquake performance of dams. Also according to Serrano Tovar (1994), in Spain, the investment to mitigate the seismic risk seems to be exaggerated in view of the maximum expected acceleration value, which is 0.25 g for a recurrence interval of 500 years. The same author (Serrano Tovar, 1994) considers that the recent installation of digital accelerometers in various Spanish dams is also controversial. This means that the safety in that type of works will be undoubtedly better assessed by investing in other aspects or in other types of monitoring equipment.

As previously mentioned, from the analysis of dam accidents, it is possible to observe that the effects of seismic actions have been less serious than those predicted on the basis of theoretical studies. Nonetheless, presently, a significant number of institutions responsible for the safety of dams are re-assessing the earthquake stability of the oldest dams, even in low risk regions (USA, Martin *et al.*, 1994; Italy, ENEL SpA, 1994; United Kingdom, Haws *et al.*, 1994).

Mention must also be made to the fact that the modelling of the dynamic performance of fill dams is extremely sensitive to proposed hypotheses. Consequently, the design criteria of the seismic stability may sometimes be poorly representative, when compared with real cases.

5. Conclusions

In February 2007, there was an earthquake with epicentre located on the Atlantic Ocean, Southeast of Portugal, at about 340 km distance from Beliche and Odeleite Dams.

Both dams are rockfill dams. Due to the significant distance from the epicentre, a very low acceleration was estimated for the dams site. Consequently, neither the inspection nor the reading of magnitudes in the mentioned dams detected any effect as a result of dynamic actions. Nonetheless, it was considered that it could be relevant to study the structural behaviour of rockfill dams when subject to earthquake action.

This study has demonstrated that this type of dams has suffered minimum detrimental effects caused by earthquakes, even when the actions are considerable, with magnitudes of about 8.3 and accelerations of 1.3 g. The observed effects are represented by superficial displacements, which usually do not exceed a few dozen centimetres, and by the opening of fissures or cracks. Only in one case, there was an increase in infiltration flow rates.

Reference must also be made to the fact that a large number of analysed dams have been built with dumped rockfill, which is a technique that has not been employed for more than 40 years. In those dams, due to the less dense state of granular material, the effects under the dynamic pressures are likely to be much more detrimental than those of current dams with powerful vibrating rollers. Furthermore, almost invariably, after the occurrence of earthquakes it was not necessary to adopt corrective measures.

Bureau *et al.* (1985) presented a method based on values observed on prototypes, which makes it possible to estimate unit vertical displacements, in rockfill dams, as a function of the approximate magnitudes of the dynamic loads.

There is no record of dam failure due to earthquakes. Only the San Fernando tailings dams underwent considerable damages and had to be completely re-built. These are hydraulic fill dams, of which the loss of shear strength occurred due to liquefaction, and therefore, they can be considered as much more vulnerable to earthquakes than the other types of dams.

Earth dams have exhibited a fairly satisfactory seismic performance. Nevertheless, these present higher vulnerability than rockfill dams due to the influence of pore pressures and to the fact of soils usually having less shear strength than rockfills.

From the statistical analysis of accidents in fill dams, it is concluded that this type of structures has a fairly satisfactory earthquake performance. Thus, Seed *et al.* (1978) mentioned that for moderate earthquakes, *i.e.*, for accelerations less than 0.2 g, no significant effect is expectable on fill dams and, therefore, the dynamic analyses should have a reduced development and should be based on judicious reasoning. Regarding this aspect, various countries have stressed the fact that, sometimes, the technical regulations require too severe and too complex studies for the dams under dynamic actions and that the resulting predictions significantly overestimate the values of magnitudes observed in prototypes.

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Effects of an Abutment Construction on Soft Soil on a Neighbouring Structure: Influence of Different Construction Techniques Using Geosynthetics

Íris L. Macêdo, Ennio M. Palmeira, Gregório L.S. Araújo

Abstract. The topographic and geological conditions of several regions in Brazil favour the formation of large deposits of soft soils. Sometimes this requires the use of innovative techniques to allow constructions of geotechnical works in these sites. The presence of deep soft foundation soils has led to an increasing number of reinforced bridge abutments being built in the last years. However, little is known on the behaviour of such works, particularly with respect to displacements and bending moments induced in the neighbouring bridge foundations due to the construction of the embankment. In this paper, hypothetical bridge abutments were modelled, with and without the presence of geosynthetic reinforcement, aiming to evaluate the effects of the embankment construction on the foundations of the neighbouring bridge. The effects of the use of vertical drains and piles with caps underneath the embankments were also investigated. The results obtained clearly indicated the potential of the use of reinforcement, vertical drains or piles to reduce damages to the foundations of neighbouring structures caused by embankment construction. The use of vertical drains in conjunction with geosynthetic reinforcement showed an important effect on the reduction of horizontal displacements and moments in the piles of the bridge.

Key words: geosynthetics, soil reinforcement, soft soil, abutments, foundations.

1. Introduction

The construction of embankments on soft soils requires special design and construction techniques because of the poor stability conditions of the work and large and time-dependent deformations of the soft foundation. Depending on site conditions and project characteristics, the solutions to deal with this type of problems can be partial or total removal of the soft foundation material, embankment basal reinforcement and the use of vertical drains or piles under the embankment (Pilot, 1981; Magnan, 1983; Jewell, 1996; Leroueil *et al.*, 1985; Palmeira, 2002, for instance). Sometimes, some of these solutions can be combined. In this context, the use of the soil reinforcement technique has shown a marked increase during the last decades due to its simplicity and its cost-effectiveness. When a fast dissipation of pore pressures and acceleration of consolidation settlements are required, vertical drains can be used in combination with embankment basal reinforcement. When in-service conditions require limited amount of embankment settlements, geosynthetic reinforcement can still be used in association with piles and caps.

The complexities of designing and constructing embankments on soft soils is significantly increased when there are structures adjacent to the embankments, such as in the cases of bridge abutments. When the abutment is being constructed, not rarely the bridge foundation has already been executed, or the bridge is being constructed simultaneously to the abutment (Fig. 1). Thus, the construction of the latter can cause lateral movements of the soft foundation soil that may yield to serious problems to the existing structure. The use of soil reinforcement and/or soft soil stabilisation techniques in such cases can avoid or significantly reduce the horizontal movements of the soft ground and stresses on foundation elements of the structure. Only a limited number of studies can be found in the literature on this type of problem (Ortigão *et al.*, 2001; Palmeira *et al.*, 2001; Fahel & Palmeira, 2002; Fahel, 2003; Macedo & Palmeira, 2003; Macêdo *et al.*, 2008).

Sometimes soft soil shear strength can be so low that even the construction of rather low embankments may yield to failure. The use of basal geosynthetic reinforcement can then improve the short and long term stability conditions of the embankment (van Leeuwen & Volman, 1976; Volman *et al.*, 1977; Silva, 1996; Rowe & Soderman, 1984; Delmas *et al.*, 1990; Rowe *et al.*, 1995; Rowe, 1997; Palmeira *et al.*, 1998; Fahel, 2003; Oliveira, 2006).

Although short and long term stability of the embankment are of utmost importance, in several cases the limitation of soil deformation is also required to guarantee the project serviceability. However, the estimate of soil deformations is a more complex task than the usual limit equilibrium approaches employed for stability analyses. In these situations, the Finite Element Method (FEM) can be a useful tool to predict soil deformation and several studies involving the use of FEM to analyse the behaviour of

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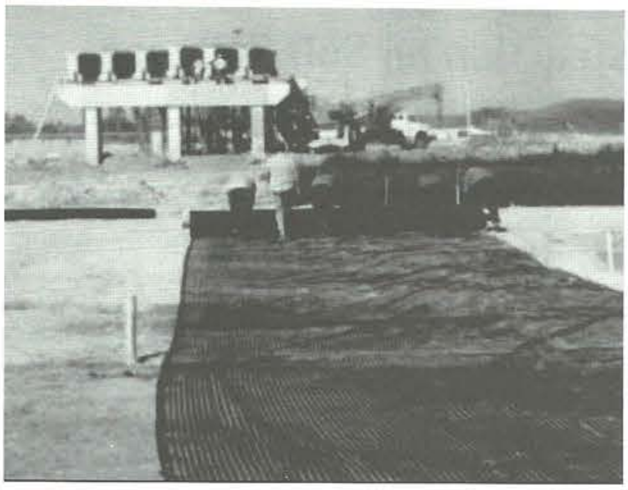

Figure 1 - Example of simultaneous bridge and abutment constructions (Fahel, 2003).

reinforced embankments on soft soils can be found in the literature (Rowe and Soderman, 1987; Schaefer and Duncan, 1988; Loke *et al.*, 1994; Sá, 2000; Borges & Cardoso, 2002; Hinchberger & Rowe, 2003; Araujo, 2004; Li & Rowe, 2008, for instance).

The use of vertical synthetic (geodrains) or granular drains can bring additional benefits to the stability and performance of the embankment, as it accelerates pore pressure dissipation and soil consolidation, with consequent shear strength gains. Besides, after construction, the additional settlements will be smaller and will occur throughout a larger period of time. When there is not enough time to wait for soil consolidation, uncertainties on the performance of the vertical drainage system and the project requires low levels of soil deformation, the use of piled foundation for the embankment can be chosen, though its cost is considerably greater than other solutions. In this case, most of the load of the embankment is transmitted to a stiffer soil layer in depth, minimising the total stress increments in the soft foundation mass.

This paper presents a finite element analysis on the influence of the use of different solutions for abutment reinforcement or soft soil stabilisation on the behaviour of the foundations of a neighbour bridge. Several aspects relevant to the performance of reinforced soil embankments were investigated in these analyses.

2. Characteristics of the Problems Investigated

2.1. Modelling characteristics

In this work the construction of hypothetical abutments were simulated by the Finite Element Method (FEM). The abutments were 5 m high and were built on a 12 m thick soft and saturated foundation soil. The computer code PLAXIS (Brinkgreve & Vermeer, 1998), available at the Graduate Programme of Geotechnics of the University of Brasilia, Brazil, was employed in the analyses, which were carried out under plane strain conditions. Five constitutive models for soil behaviour are available in the program. The mesh generation is automatic and elements can be prescribed with six or fifteen nodes. In the analysis, staged construction of the embankment was simulated and some cases involved the use of vertical drains or piles underneath the embankment. Because of the large number of relevant variables present in this type of analysis, this study focused on the investigation of the performance of abutments on soft soils using typical values for the geometrical characteristics and material properties found in this type of work.

The behaviour of the soft soil was modelled using the elastic-plastic "soft soil" model present in the Plaxis code, which is based on the Cam-Clay model, and uses Biot's theory of consolidation (Biot, 1941). An elastic-plastic model ("Mohr-Coulomb Model" in the Plaxis code) was used for the abutment material and for the soil underneath the soft soil layer. The reinforcement layers were simulated using the "geotextile element" present in the Plaxis code, for which a linear elastic response is assumed.

Two different situations were considered in the simulation of the foundations of the neighbouring bridge. In the first situation, only the piles of the adjacent structure were already constructed at the start of the abutment lifting. Thus, under these circumstances the top of the piles were not subjected to vertical loads or movement restriction yet, except for that from the passive resistance of the foundation soil. Hereafter, these analyses will be referred to as Free Pile Top analysis (FPT analysis). In the second situation, it was assumed that the bridge was already constructed at the start of the embankment lifting. In this case, the line of piles closest to the embankment toe are vertically loaded by a fraction of the bridge weight and there is further restriction to the movement of the piles because of the influence of the presence of the other lines of piles of the bridge foundation system. This type of analyses will be referred to in this paper as Restricted Pile Top analysis (RPT analysis). In both types of analyses, the investigations were concentrated on the study of the effects of abutment construction on the piles of the bridge at varying distances from the embankment toe. In all cases analysed (either FPT or RPT analyses) the piles penetrated 0.5 m in the stronger soil layer underneath the soft soil.

Additional comments and information on the cases studied and methodology used can be found in Macêdo (2002).

2.2. Geometric characteristics and properties of the materials

The height of the abutment was assumed equal to 5 m. Analyses of unreinforced abutments with this height, constructed on the soft soil with the properties used in this study, showed that reinforcement layers would be required in the abutment for that height to be reached. Even one reinforcement layer was not enough in some cases for the abutment to reach its final height, depending on the reinforcement tensile properties. The abutments were constructed in five stages of 1 m. The results presented in this paper are those obtained at the end of embankment construction (28 days total construction time) and 180 days after embankment construction.

Different values of tensile stiffness and number of reinforcement layers were used in the analyses. When more than one layer of reinforcement was used, the vertical spacing between layers was equal to 0.5 m. The bottom reinforcement layer always coincided with the abutment-soft soil interface, except in the case of the presence of piles and caps underneath the embankment.

Figure 2(a) shows a general view of the geometrical characteristics of the problem analysed. Figure 2(b) presents the finite element mesh utilized in the calculations, based on initial trial runs of the programme to evaluate the size of the mesh to be used to minimise boundary effects. The slope of the abutment facing the bridge was equal to 2:1 (horizontal:vertical).

As mentioned above, some analyses considered the presence of vertical drains to accelerate pore pressure dissipation. In these cases the spacing between vertical drains was assumed equal to 1.5 m, with the drains distributed in a square pattern in plan. To model the vertical drains, interface elements were employed and the equivalent wall concept (Indraratna & Redana, 1997) was used to satisfy plane strain conditions.

An additional study involved the investigation of solutions incorporating piles and caps under the abutment with varying spacing between piles. The piles and caps

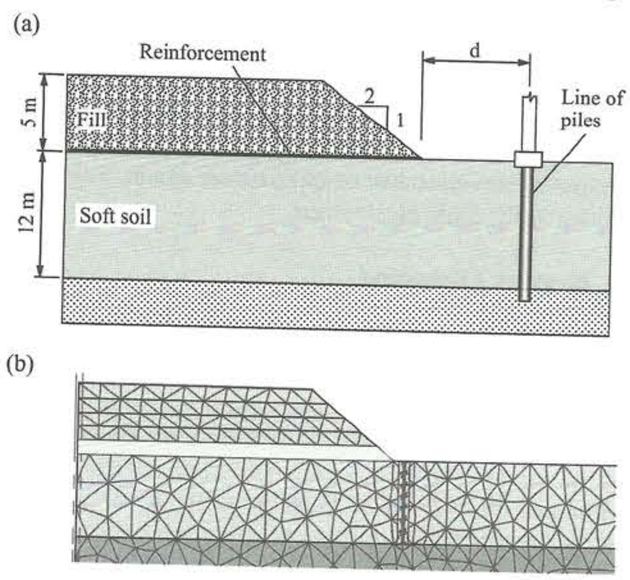

Figure 2 - Geometrical characteristics of the problem and finite element mesh. (a) Geometrical characteristics of the problem, (b) Finite element mesh.

were distributed along the entire base of the abutment, were assumed as made of concrete and were modelled as beam elements. The piles had a diameter of 0.25 m and were 12.5 m long, whereas the caps were 1 m x 1 m x 0.5 m (height). The piles were distributed in plan in a square pattern with varying spacing between them, depending on the objectives of the study. Interface elements were used along the shaft of the piles under the embankment as well as in the piles of the neighbouring bridge to simulate more accurately the interaction between pile and foundation soil. The piles under the abutment were simulated under two-dimensional conditions as equivalent walls. The thickness of the wall was calculated as a function of the pile diameter, spacing and material properties in order to attend stiffness equivalency with the actual array of discrete piles. When present, the reinforcement layer was assumed to be installed on the top of the caps. It is important to point out that in this analysis the reinforcement layer was assumed directly on the cap for the sake of simplicity in the modelling process. In a real situation, the direct contact between cap and reinforcement may cause mechanical damage to the reinforcement (Almeida *et al.*, 2008).

Typical soil properties were assumed for the soils involved in the analysis. Table 1 summarises the properties used in the analyses for the embankment soil, soft soil layer and for the stronger soil layer (3 m thick) underneath the soft soil. These properties are required by the computer code used (Plaxis) when the "Soft Soil" and "Mohr-Coulomb" models are employed.

Some hypotheses had to be made for the analyses of the cases where the entire bridge was already built by the time of the abutment construction (RPT). In this case, a 40 m long and 12 m wide concrete bridge was assumed for this study, as shown in Fig. 3. The bridge is supported by 6 columns. Each couple of columns transfers the vertical load to a group of 10 piles in line, 0.3 m in diameter and with a spacing of 1.5 m. The central columns have diameter equal to 1.5 m and those at the extremities of the bridge have diameter of 1 m. Traditional structural design procedures were used for the hypothetical reinforced concrete bridge (Macêdo, 2002) and the vertical loads acting on each set of two columns were equal to 5000 kN, which yields to an average vertical load per pile equal to 500 kN. This load is consistent with the allowable axial load for a 0.3 m diameter pile reinforced with four 12.5 mm diameter steel bars.

It should be pointed out that, in fact, the problem examined in this paper is a three dimensional one. So, with this regard, the results obtained should be considered as approximations to the actual conditions in the field. To simulate the line of piles of the bridge foundation under plane strain conditions, the equivalent wall concept was used. As in the case of piles underneath the abutment, the thickness of the equivalent wall was calculated as a function of the pile diameter, spacing and material properties

Table 1 - Soil parameters.

Parameter	Symbol	Unit	Material		
			Soft soil (undrained)	Fill (drained)	Bottom soil layer (drained)
Unsaturated unit weight	γ_d	kN/m^3	13.0	19.0	20.0
Saturated unit weight	γ_w	kN/m^3	16.0	20.0	22.0
Horizontal permeability	k_x	m/day	0.001	1,000	1,000
Vertical permeability	k_y	m/day	0.001	1,000	1,000
Young Modulus	E'	kPa	-	15000	60000
Poisson's ratio	υ'	-	-	0.30	0.30
Shear modulus	G'	kPa	-	5769.2	23076.9
Oedometer modulus	E_{oed}	kPa	-	20192.3	80769.2
Cohesion	c'	kPa	5.0	1.0	5.0
Friction's Angle	ϕ'	(°)	25.0	30.0	36.0
At-rest earth pressure coefficient	k_o	-	0.64	0.50	0.41
Modified compression ratio	λ^*		0.08	-	-
Modified expansion ratio	κ^*		0.011	-	-
Over-consolidation ratio	OCR		1.3	-	-
Poisson's ratio (unloading/ reloading)	υ_{ur}		0.15	-	-

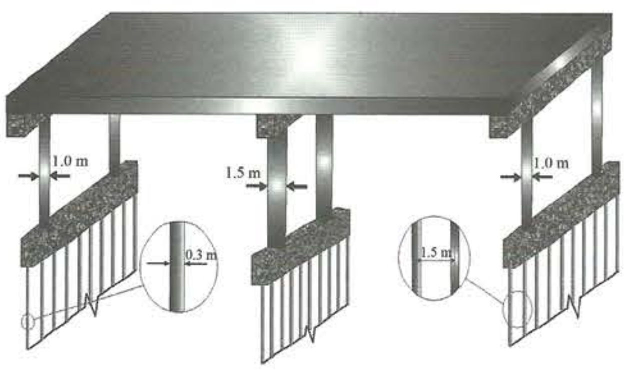

Figure 3 - Bridge geometrical characteristics.

Table 2 - Pile and equivalent wall characteristics.

Parameter	Symbol	Unit	Value
Diameter	D	m	0.30
Young modulus	E	kPa	3.5 x 10^7
Cross section area	A	m^2/m	0.15
Moment of inertia	I	m^4/m	2.65 x 10^{-4}
Axial stiffness	EA	kN/m	5.15 x 10^6
Flexural rigidity	EI	kNm^2/m	9.28
Equivalent wall thickness	d_{eq}	m/m	0.15
Poisson's ratio	υ	-	0.15

in order to attend stiffness equivalency to the line of discrete piles (Fig. 3). Table 2 presents the mechanical and geometrical characteristics of the piles and of the equivalent wall.

A beam element working in a similar way as a spring was used to simulate the restriction to the movement of the pile top due to the presence of the bridge (Fig. 3, RPT analysis). The stiffness (K) of this spring like element was obtained simulating the bridge subjected to different values of an horizontal load (Q) applied to one of its ends, as schematically shown in Fig. 4, and obtaining the corresponding horizontal displacements (δ). Thus, the stiffness (K) of the spring element to be used at the top of the pile, that would restrict its movements in an approximate similar way as the bridge itself, could be obtained.

3. Results Obtained

This section presents and discusses the results obtained for the maximum horizontal displacements (δ_{hmax}), bending moments on the bridge piles, maximum abutment vertical displacements (δ_{vmax}) and maximum tensile forces in the reinforcement (T_{max}) caused by the construction of the abutment for FTP and RTP analyses. The tensile stiffness (J) and number of reinforcement layers was varied, as well as the distance between the piles and the abutment toe and the distance between pile caps (s - a) in the case of piled abutments, where s is the spacing between piles and a is the cap width.

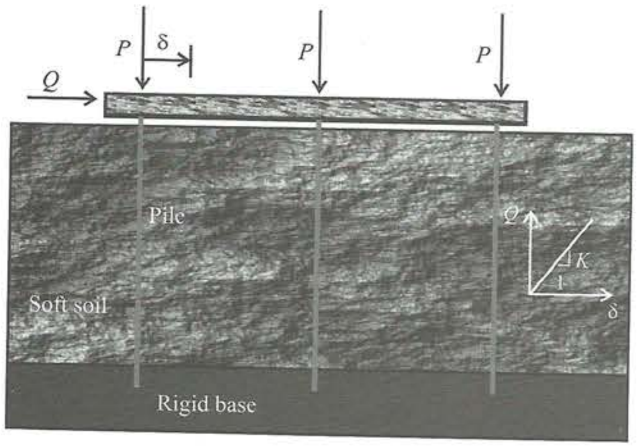

Figure 4 - Estimate of the equivalent bridge stiffness (K) of the spring like element to be used at the top of the piles in RPT analyses.

3.1. Maximum horizontal displacements (δ_{hmax}) of the piles

3.1.1. Influence of the tensile stiffness and number of reinforcement layers

Figure 5 shows the variation of maximum horizontal displacements (δ_{hmax}) of the bridge piles as a function of the geosynthetic stiffness for different numbers of reinforcement layers and for a distance (d) between the line of piles and the abutment toe equal to 1 m for the FPT (free pile top) case. In these cases the maximum displacements occurred at the pile top. These results are those obtained for a time (t) equal to 180 days after the end of construction of the abutment. It can be observed that the reinforcement stiffness has an important effect on the reduction of horizontal displacements. For the conditions analysed, the stiffer the reinforcement and the greater the number of reinforcement layers the smaller the horizontal displacements of the foundation soil and of the piles. However, the influence of the number of reinforcement layers is reduced as the reinforcement tensile stiffness increases. For this and other figures in this paper, missing points in the graph (particularly for low values of reinforcement stiffness - 1 reinforcement layer and J = 500 kN/m in Fig. 5, for instance) indicate situations for which the embankment failed (excessive number of plastic points) before the end of construction.

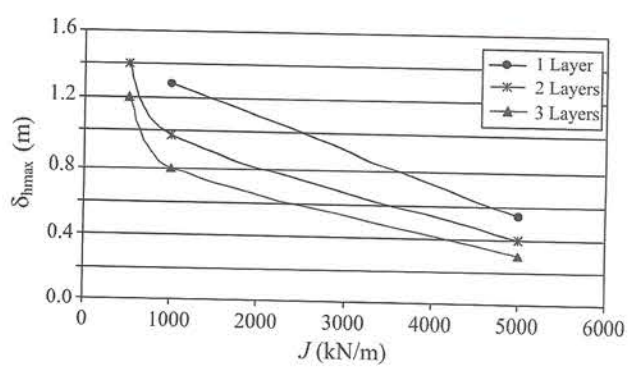

Figure 5 - Maximum pile horizontal displacement *vs.* reinforcement tensile stiffness - FTP analysis, d = 1 m and t = 180 days.

It should be noted that the lines of piles (or equivalent wall) is also a stabilizing element for the abutment, avoiding or minimising the possibility of failure. The line of piles works as a retaining structure, as far as the horizontal movement of the soft soil is concerned, increasing the stability of the system as a whole. For values of d greater than 1 m reinforcements with high values of tensile stiffness are required to make possible the construction of the abutment to its final height. Because of the stabilising effect of the line of piles, less stiff reinforcements were required to construct the abutment to its final height for the cases where the piles were installed closer to the embankment toe (d = 1 m).

The restriction of the pile top (RPT case) influences the displacements of the piles and changes their horizontal displacement profiles, as shown in Fig. 6 for the unreinforced case. For FPT cases, the maximum horizontal displacements occur at top of the pile, whereas for RPT cases the maximum horizontal displacements occur at a depth approximately equal to 0.3 times the soft soil thickness. As expected, the displacements obtained in RPT cases were significantly smaller than those obtained in FPT cases. The displacements in the former case can be up to 75% smaller and are a consequence of the influence of the stiffness of the bridge and of the other foundation elements present.

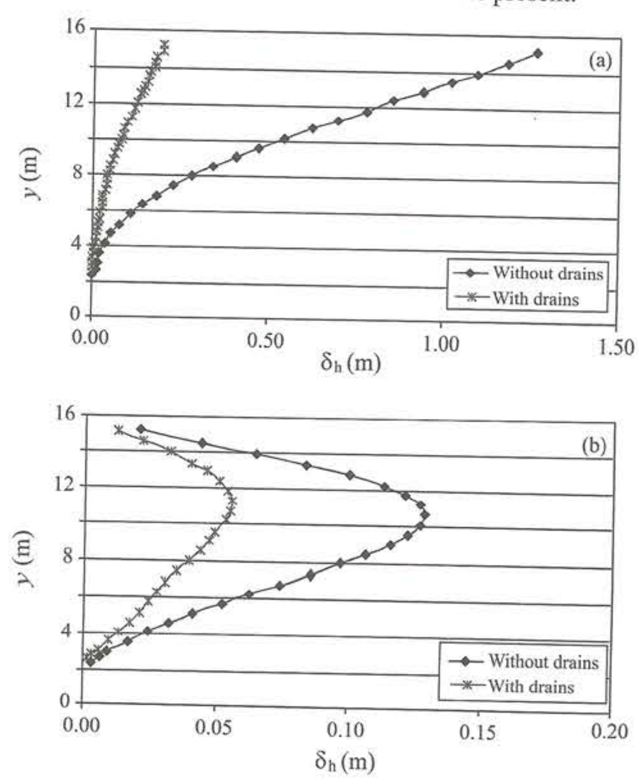

Figure 6 - Horizontal displacement profiles of the piles under unreinforced conditions - d = 1 m and t = 180 days. (a) FPT case, (b) RPT case.

Figs. 7(a) and (b) show the influence of the reinforcement tensile stiffness and number of reinforcement layers on the maximum horizontal displacements of the piles, for cases where the movement of the pile top was restricted (RPT case). The results indicate that the reinforcement stiffness is more important than the number of reinforcement layers for the reduction of the maximum horizontal displacement (Fig. 7a). When the distance (d) between the embankment toe and the piles is increased to 3 m, the pile displacements decrease as well as the influence of the reinforcement tensile stiffness on the pile movements (Fig. 7b).

3.1.2. Influence of the presence of vertical drains

The presence of vertical drains in the soft foundation soil causes a significant reduction on the horizontal displacements of the soft soil associated with the rapid dissipation of pore pressures generated by the abutment construction. Figures 8(a) and (b) show the variation of maximum horizontal displacements of the piles *vs.* reinforcement stiffness for the cases where the presence of vertical drains in the foundation soil were considered. Comparing the results in Fig. 8(a) and in Fig. 5, it can be observed a reduction up to 86% in the pile horizontal displacements close to the abutment toe (d = 1 m) due to the presence of the vertical drains, in comparison with the same case without the drains (Fig. 5). This reduction was observed for the abutment with two layers of reinforcement at its base and with a reinforcement tensile stiffness of 500 kN/m. The presence of the vertical drains allowed the construction of the abutment in all the cases analysed (no failures during construction), including those where reinforcements were not used. This can be explained by the fast increase in effective stresses due to pore pressure dissipation and consequent increase in soft soil strength when the drains were present.

Figure 9 shows that for RTP cases the presence of the vertical drains caused a reduction of approximately 60% on the pile horizontal displacements in comparison with the same case without the drains, for d = 1 m and t = 180 days (Fig. 7a). In RTP cases, the maximum reduction on pile displacement due to the presence of the reinforcement varied between 4.5% and 14% in comparison to the unreinforced case (J = 0 in Fig. 9), depending on the stiffness and number of reinforcement layers considered. The presence of the reinforcement layers was more significant for a number of layers equal to 3 (Fig. 9). However, the presence of the reinforcement layers may be fundamental during abutment construction to guarantee short-term stability conditions.

3.1.3. Influence of the presence of piles and caps underneath the abutment

The movements of the piles of the bridge foundation were also investigated for the case of a piled abutment with and without the presence of reinforcement layers. Only FPT cases were analysed under these conditions. In general, as expected, the shorter the distance between caps the smaller the movements of the bridge piles. As the distance

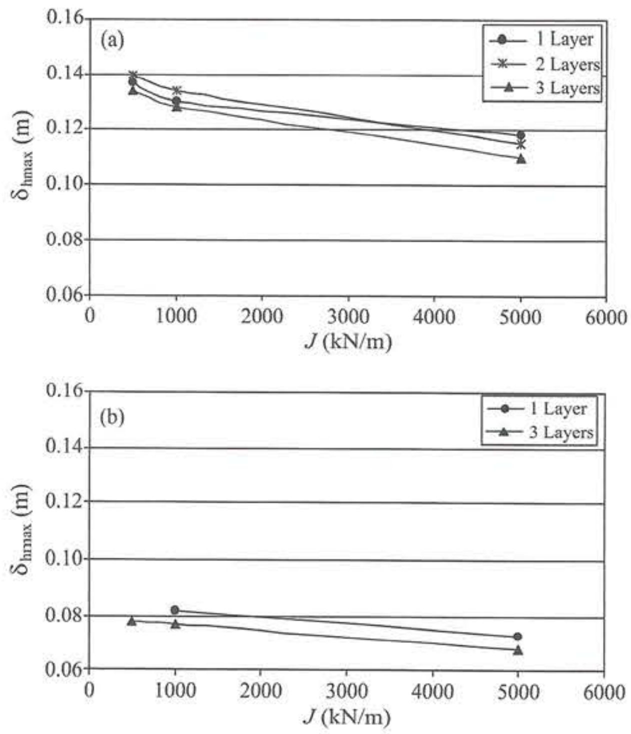

Figure 7 - Maximum pile horizontal displacement *vs.* reinforcement tensile stiffness - RPT cases and t = 180 days. (a) d = 1 m, (b) d = 3 m.

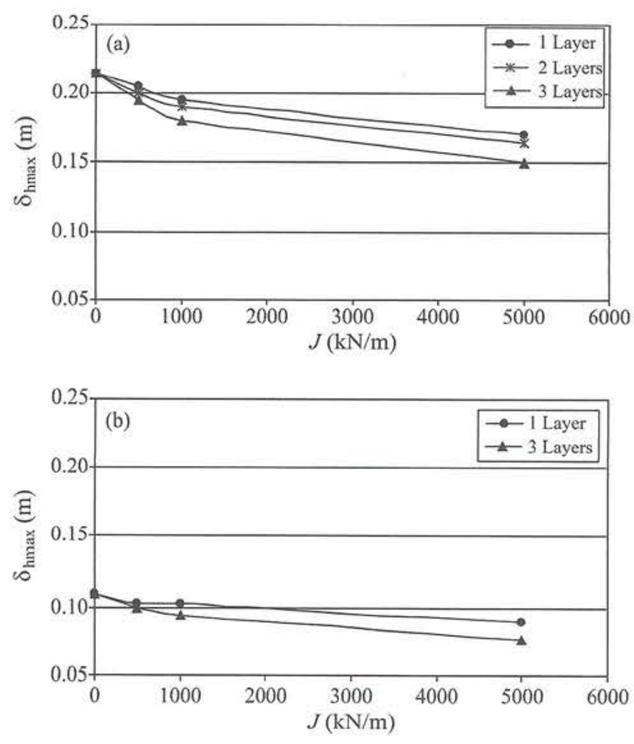

Figure 8 - Maximum pile horizontal displacements *vs.* reinforcement stiffness for the cases with vertical drains - FPT cases and t = 180 days. (a) d = 1 m, (b) d = 3 m.

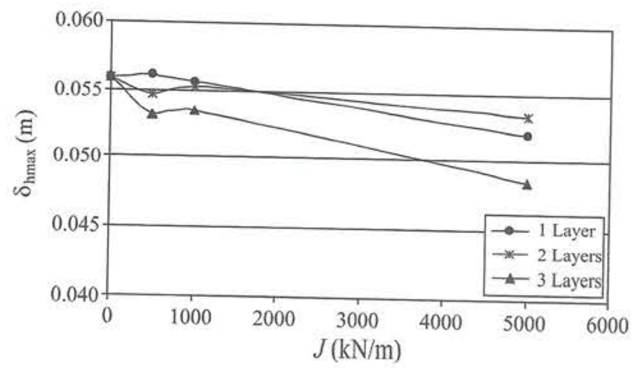

Figure 9 - Maximum pile horizontal displacement *vs.* reinforcement tensile stiffness in cases with vertical drains - RPT cases, d = 1 m and t = 180 days.

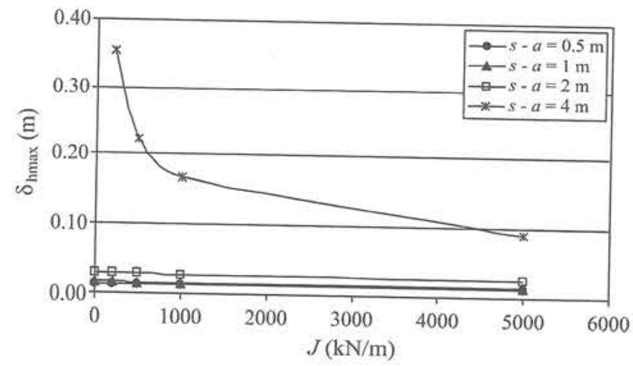

Figure 11 - Variation of pile horizontal displacements with reinforcement stiffness for different values of (s - a), d = 1 m and t = 180 days - FPT cases.

between caps increases, the efficiency of the piles is reduced because of the greater transference of abutment loads to the foundation soil caused by less arching of abutment soil in between the pile caps. The presence of the geosynthetic layer in the abutment improves the distribution of loads to the piles and the performance of the piled abutment.

Figure 10 depicts the variation of horizontal displacements of the piles of the bridge as a function of the distance between caps, given by (s - a), where s is the spacing between piles under the embankment and a is the cap width. It can be noted the small values of bridge piles displacements (less than 0.03 m) for values of (s - a) smaller than 3 m. It can also be observed that, in numerical terms, the construction of the embankment without a geosynthetic layer on the caps was possible only for values of (s - a) up to 2 m. For larger values, excessive horizontal displacements of the piles occurred. However, for the cases where a reinforcement layer with J = 5000 kN/m was used the construction of the abutment was possible up to values of s - a equal to 4 m, with a maximum pile horizontal displacement of 0.09 m.

Figure 11 shows the variation of maximum horizontal displacement of the bridge piles *vs.* reinforcement stiffness for piles distant 1 m from the abutment toe and for different values of the distance (s - a) between the caps of the piled abutment. The results in this figure show that there is little influence of the value of reinforcement stiffness on horizontal displacements for values of (s - a) smaller than 2 m. However, for (s - a) equal to 4 m there is a marked effect of the increase in reinforcement tensile stiffness in reducing horizontal displacements of the piles of the bridge. A similar behaviour was observed by Macêdo (2002) for the case of a 6 m thick soft foundation soil layer.

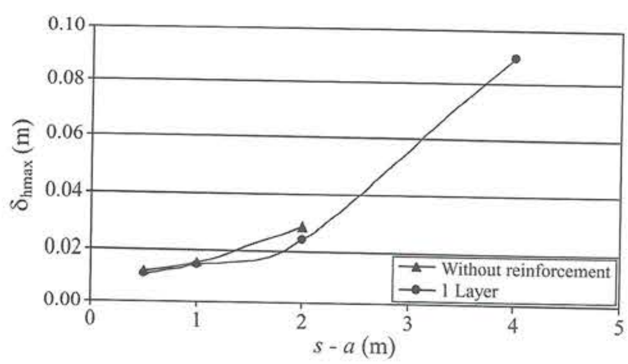

Figure 10 - Maximum pile horizontal displacement *vs.* distance between caps for J = 5000 kN/m, d = 1 m and t = 180 days - FPT cases.

3.2. Maximum bending moments in the piles

3.2.1. Influence of the reinforcement tensile stiffness and number of reinforcement layers

An accurate prediction of the bending moments in the bridge piles is fundamental for the stability of the bridge. Figures 12(a) and (b) present maximum values of predicted pile bending moments *vs.* reinforcement stiffness for FPT and RPT cases, respectively, at the end of the abutment construction (t = 28 days). A marked influence of the presence and tensile stiffness of the reinforcement can be noticed in FPT cases (Fig. 12a). A reduction of maximum bending moments up to 60% was obtained with an increase of reinforcement tensile stiffness from 1000 kN/m to 5000 kN/m in those cases. There is less influence of the stiffness and number of reinforcement layers on the maximum bending moments in the pile in RPT cases than in FPT cases (Fig. 12b).

3.2.2. Influence of the presence of vertical drains

The influence of the presence of vertical drains underneath the abutment is shown in Figs. 13(a) and (b) for values of d equal to 1 m and 3 m (FPT cases and t = 180 days), where it can be seen that the maximum pile bending moment decreases with the increase of the reinforcement tensile stiffness, but there is little influence of the number of reinforcement layers. For piles distant 1 m from the abutment toe the use of a reinforcement with a tensile stiffness of 5000 kN/m caused a reduction of approximately 34% on the maximum bending moment obtained for the situation without reinforcement (Fig. 13a).

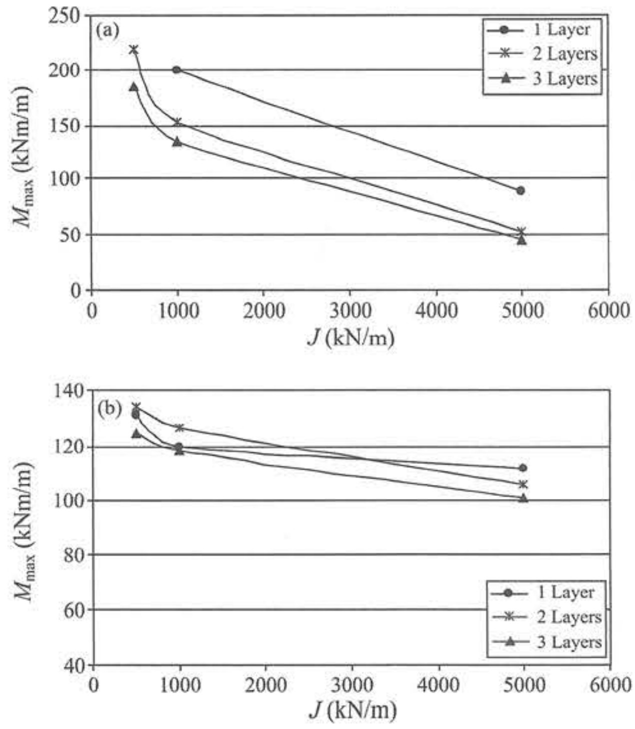

Figure 12 - Maximum pile bending moment *vs.* reinforcement tensile stiffness for $d = 1$ m and $t = 28$ days. (a) FPT case, (b) RPT case.

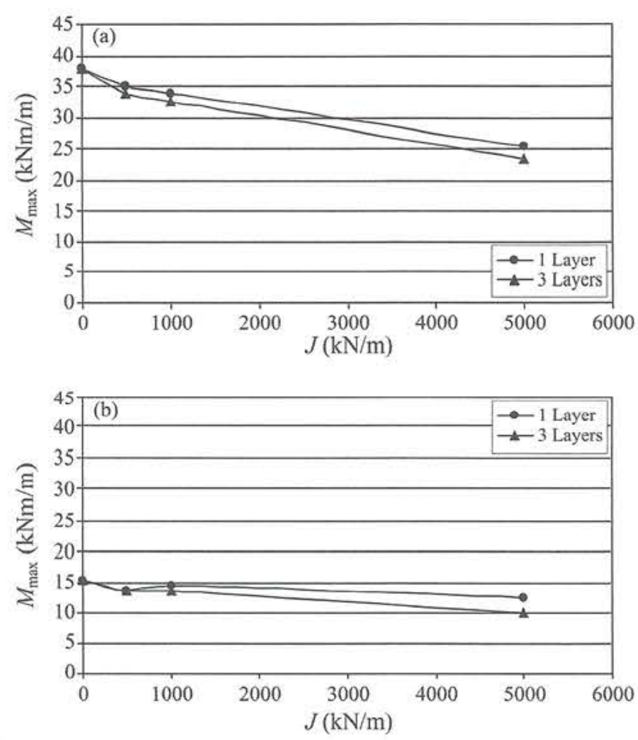

Figure 13 - Maximum pile bending moments *vs.* reinforcement tensile stiffness for the cases with vertical drains - FPT case and $t = 180$ days. (a) $d = 1$ m, (b) $d = 3$ m.

3.2.3. Influence of the presence of piles and caps underneath the abutment

Figure 14 presents the influence of the presence of piles and caps underneath the abutment on the maximum bending moments in the bridge piles at the end of construction for RPT cases. It can be noted that there is a substantial reduction on the pile moments due to abutment piling and that this reduction is a function of the distance between caps (s - a). The tensile stiffness of the reinforcement seems to be relevant only for large values of s - a.

3.2.4. Bending moment distribution along the pile length

Figures 15(a) and (b) show the distributions of bending moments along the pile length for $d = 1$ m, at the end of abutment construction ($t = 28$ days), for FPT cases, with and without the presence of vertical drains underneath the abutment. It can be noted that for the case without vertical drains there is a greater influence of the presence of the reinforcement layers and of the magnitude of the reinforcement tensile stiffness on the bending moment distribution along the pile length (Fig. 15a). The use of vertical drains reduces significantly the values of the bending moments and for the case where a reinforcement layer with $J = 5000$ kN/m was employed, the maximum bending moment was approximately 40% smaller that that observed for the case without reinforcement (Fig. 15b).

3.3. Vertical displacements of the abutment

3.3.1. Influence of the tensile stiffness and number of reinforcement layers

Figures 16(a) and (b) present the results of maximum abutment settlement *vs.* reinforcement stiffness at the end of construction ($t = 28$ days) and for $t = 180$ days, respectively, for FPT cases. The greatest variations of maximum settlement with reinforcement tensile stiffness occurred for values of the latter up to 1000 kN/m. The greater the reinforcement tensile stiffness the less the influence of the number of reinforcement layers. In these analyses it was also observed that increases on reinforce-

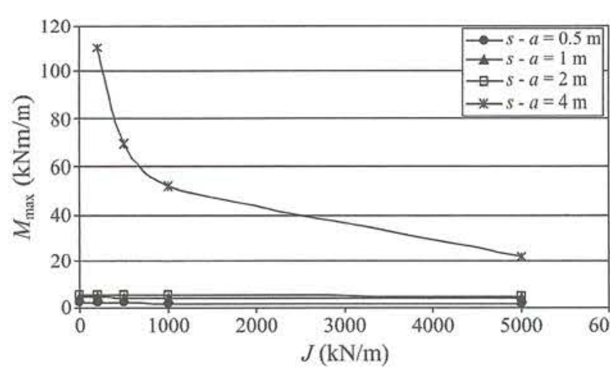

Figure 14 - Maximum pile bending moment *vs.* reinforcement tensile stiffness for cases of piled abutments - RPT cases and $t = 28$ days).

ment tensile stiffness and number of reinforcement layers tend to result in more uniform settlement profiles, reducing the differential settlements of the abutment along its longitudinal direction.

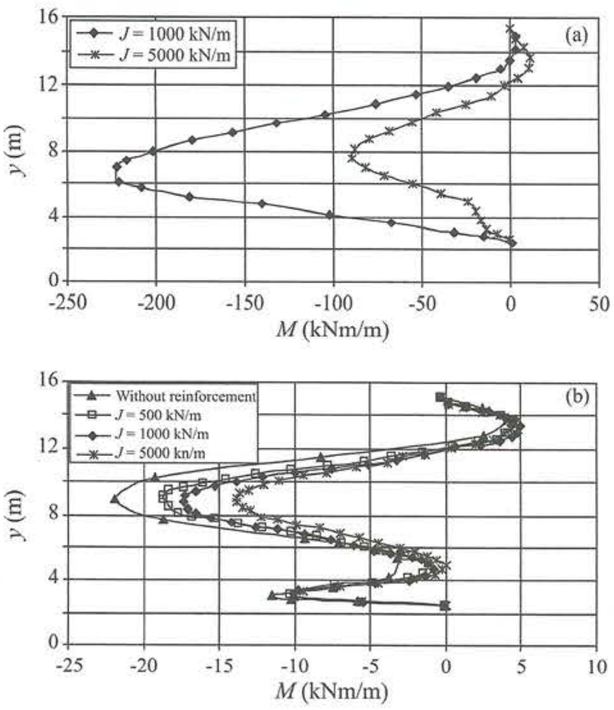

Figure 15 - Distribution of bending moments along the pile length for d = 1 m and t = 28 days (FPT case). (a) Without drains, (b) With drains.

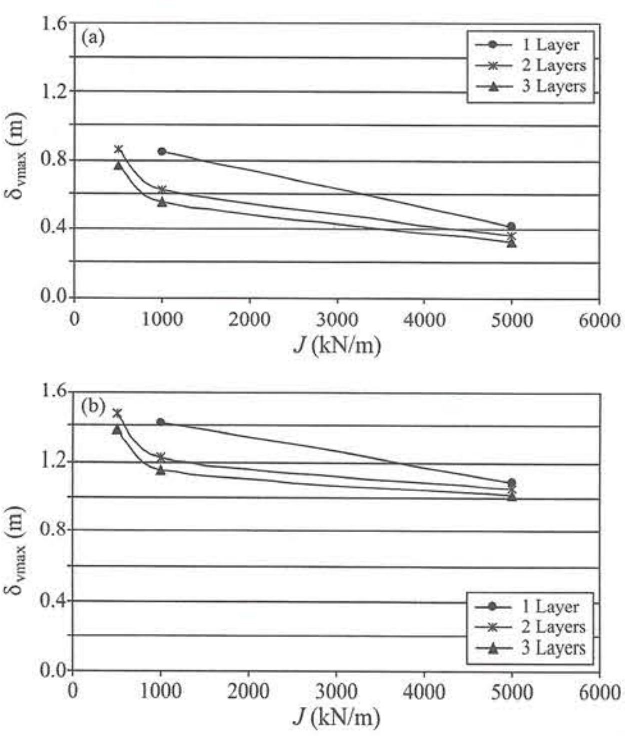

Figure 16 - Maximum abutment settlement *vs.* reinforcement tensile stiffness - FPT cases and d = 1 m. (a) t = 28 days, (b) t = 180 days.

The results of maximum abutment settlements *vs.* reinforcement tensile stiffness for RPT cases are depicted in Figs. 17(a) and (b) for t = 28 days and t = 180 days. With respect to reduction of abutment settlements, the use of three reinforcement layers provides only marginal benefit to the use of 2 reinforcement layers. However, the third layer of reinforcement may be necessary to preserve the stability conditions of the abutment, depending on the tensile strength of the reinforcement and on the project characteristics.

3.3.2. Influence of the presence of vertical drains

Figures 18(a) and (b) shows the settlement profiles along the base of the abutment with and without vertical drains at the end of construction. It can be noted that the presence of the vertical drains causes a more uniform abutment settlement profile independent on the presence or not of reinforcement. For the case of vertical drains, and for the conditions analysed, the influence of the reinforcement tensile stiffness on the settlement profile was negligible (Fig. 18b).

3.4. Forces mobilised in the reinforcement

Figure 19 shows the variation of maximum tensile force (T_{max}) mobilised in the bottom reinforcement layer with tensile stiffness for an abutment with two reinforce-

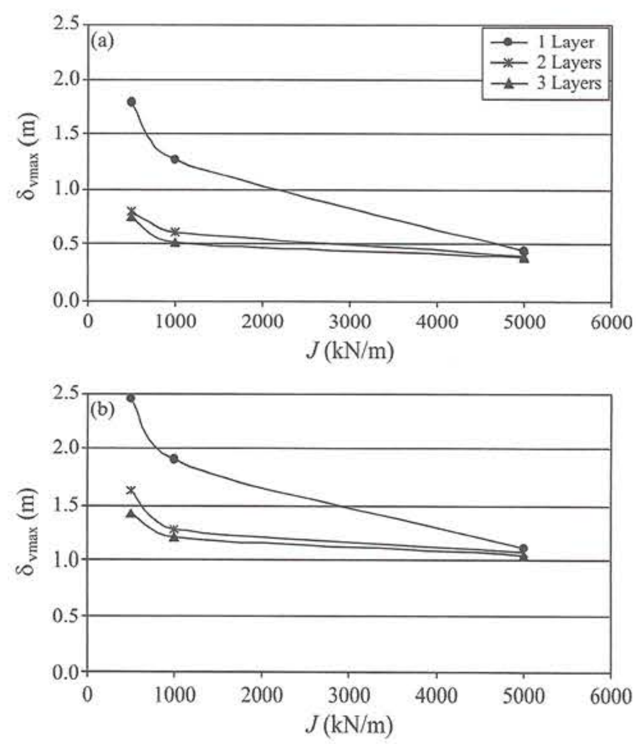

Figure 17 - Maximum abutment settlement *vs.* reinforcement tensile stiffness - RPT case and d = 1 m. (a) t = 28 days, (b) t = 180 days.

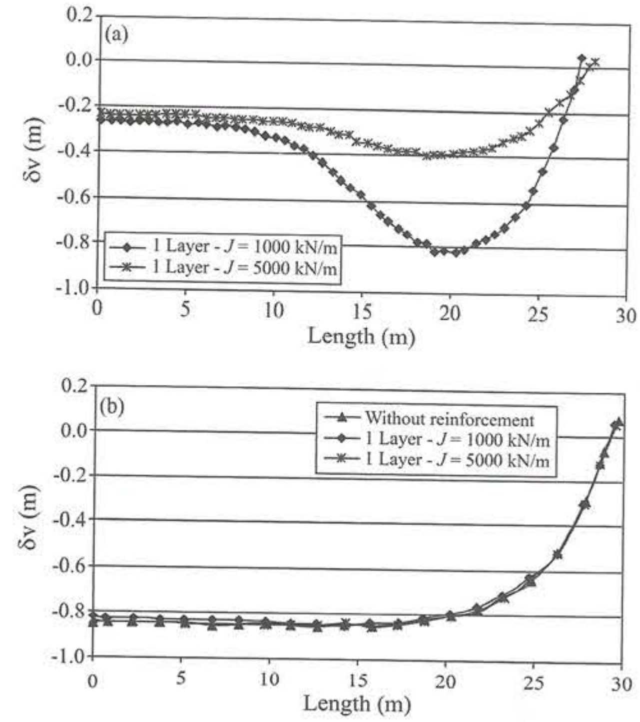

Figure 18 - Abutment settlement profiles with and without vertical drains for d = 1 m at t = 28 days - FPT cases. (a) Without drains, (b) With drains.

ment layers, d = 1 m and t = 180 days in FPT cases. In cases with more than one reinforcement layer in the abutment, the bottom reinforcement layer was the one that presented the highest tensile force. It can be noted in Fig. 19 that the presence of the vertical drains reduces significantly the maximum tensile force mobilised.

The influence of the restriction of the pile movement (RPT cases) on the tensile force mobilised in the lowest reinforcement for the case of an abutment with two reinforcement layers, d = 1 m and t = 180 days is shown in Fig. 20. Comparing Figs. 19 and 20 it can be observed that the restriction of the pile movement alone reduces the maximum

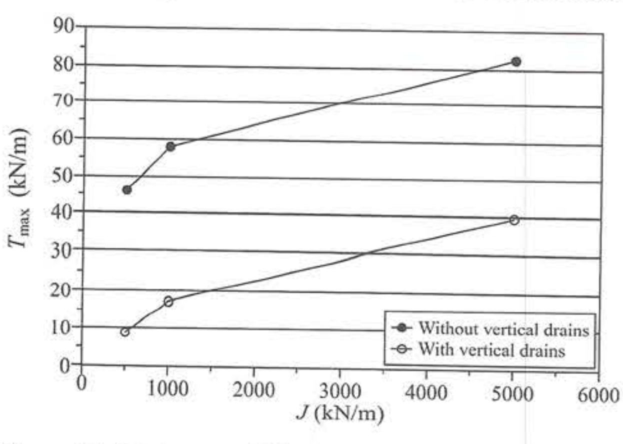

Figure 19 - Maximum mobilised tensile force in the bottom reinforcement *vs.* reinforcement tensile stiffness for cases with vertical drains - FTP cases, d = 1 and t = 180 days.

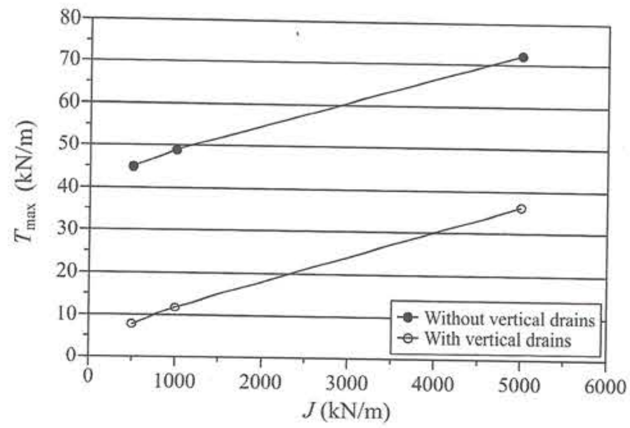

Figure 20 - Maximum mobilised tensile force in the bottom reinforcement *vs.* reinforcement tensile stiffness - RTP cases, d = 1 and t = 180 days.

force mobilised in the reinforcement. However, the greatest reduction on mobilised tensile forces is caused by the presence of the vertical drains. The vertical drains cause smaller horizontal displacements of the soft foundation soil, which yields to less mobilisation of tensile forces in the reinforcement layers.

4. Conclusions

This paper presented a numerical study on the influence of the construction of abutments on soft soils on neighbouring structures. The influence of several techniques to stabilise the abutment and to reduce the effects of its construction on the foundations of a neighbouring bridge were examined, such as the use of reinforcement, vertical drains and piles underneath the abutment. The results obtained showed that for the cases analysed the reinforcement tensile stiffness had an important effect in reducing horizontal displacements of the piles of the bridge. The increase in the number of reinforcement layers in the embankment reduced even further the horizontal movement of the foundation soil and consequently the displacements of the piles.

The use of vertical drains had a marked effect on the reduction of soft soil lateral movement with beneficial effects to the behaviour of the piles of the neighbouring bridge. When vertical drains were combined with basal reinforcement of the abutment the reduction of horizontal displacements of the bridge piles was increased even further.

As expected, the use of piles with caps underneath the abutment was the solution that presented the best results in terms of minimising horizontal movements of the soft foundation soil. The beneficial effect of the presence of geosynthetic reinforcement in this case was more evident for large distances between pile caps.

Either the use of high tensile stiffness reinforcements or vertical drains yielded more uniform settlement profiles of the abutment, with more relevance for the use of vertical drains in that regard.

The maximum tensile forces mobilised in the reinforcement were influenced by the presence of vertical drains or restrictions to the movement of the bridge piles. For cases of abutments reinforced with more than one reinforcement layer, the bottom layer was the one presenting the highest tensile force.

It is important to point out the limitations of the analyses carried out, such as the treatment of a three dimensional problem as an equivalent two dimensional one, limitations inherent to finite element analyses and the geometrical conditions assumed. Nevertheless, the results obtained show the potential of the use of basal reinforcement and vertical drains as an effective solution to reduce the effects of the construction of abutments on soft soils on the foundations of bridges or other neighbouring structures. However, one should bear in mind that there will certainly be cases where the presence of the reinforcement (and vertical drains) alone may not be sufficient to guarantee an appropriate margin of safety against damage to the foundations of the structure. Further studies are required for a better understanding and quantification on the use of geosynthetic reinforcement in such applications.

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Sub-Bases Layers of Residual Granite Soil Stabilised with Lime

Nuno Cristelo, Stephanie Glendinning, Said Jalali

Abstract. This paper describes the results of research carried out for evaluating the effects of hydrated lime, with and without chemical activators, for the modification (up to 2% lime) and stabilisation (more than 2% lime) of residual granite soils. The effects of the modification of granitic soils with the addition of 2% calcitic lime were studied; and the results of these modifications on particle size distribution, plasticity and strength gain of two soils with different clay fractions are reported. Furthermore, the effects on compressive strength when 6 and 10% lime were added for curing times up to 10 weeks were monitored. The effects of saturation of the stabilised soils were also studied, as well as the effects of a small percentage of NaCl on stabilised soils with and without saturation. This addition of NaCl proved to be highly beneficial and a better alternative than the addition of larger amounts of lime. Estimated loss of strength due to saturation of cured specimens appeared to be constant with curing time, indicating that this effect is due to loss of suction potential.

Key words: ground improvement, soil stabilisation, residual soils, pavement sub-bases, lime.

1. Introduction

The modification/stabilisation of inadequate soils for base or sub-base of road construction or other engineering works has been used since ancient times. Among the most common additives used for chemical stabilisation are normal Portland cement, lime, lime-fly ash or lime-cement. These materials change the soil properties by reacting with its fine particles or creating a matrix, which surrounds the particles or links them at their contact points. Lime has been used successfully for clayey soils. Modification of clayey soils is achieved with a small percentage of lime, i.e. up to 2%, leading to agglomeration of clay particles and hence modification of the particle size distribution. Little (1995) reports that this modification results in plasticity reduction, improved workability, immediate shear strength increase and reduced volume change potential. These characteristics are generally associated with rapid reactions between lime and the clay fraction of the soil, due to cation exchange and the associated flocculation. Stabilisation, however, is related with strength, deformability and durability, obtained from the pozzolanic reactions and occurs when higher percentage of lime is used. The reactions responsible for the gains in mechanical behaviour can be summarised as follows: the addition of lime increases pH, which facilitates the dissolution of the silica and alumina of the clay minerals and their subsequent combination with calcium, forming calcium silicate hydrate and calcium aluminate hydrate. These newly formed materials bind soil particles together, and continue to do so while silica, alumina and calcium are available. It is important to emphasize that the amount of amorphous Al_2O_3 and SiO_2, together with the lime content and the curing process, are the principal factors in the stabilisation of decomposed granite soils. Nishida (1987) indicates that the ratio of Al_2O_3/SiO_2 increases with the degree of weathering and that stabilisation is more effective with more weathered soils.

This research work studies the use of lime modification and stabilisation of residual granitic soils for the purpose of road construction. This type of soil is very common in Northern Portugal, and its clay fraction is kaolinite, a mineral which is not as reactive with lime as some other minerals. The use of local residual granite soils allows for a lower-cost and environmental friendly road construction. The other main objective of this research was to evaluate the possibilities of enhancing the effect of lime by using small quantities of low cost activators which would decrease curing times and significantly increase durability.

According to Sherwood (1995) the general functions of the sub-base are to provide a working platform; spread the wheel loads so that the sub-grade is not over-stressed; work as a protecting layer against freezing, where sub-grade is likely to be weakened by the action of frost; and to work as a drainage layer (Fig. 1).

Since lime stabilisation reduces plasticity, swell potential and volume change potential; and increases strength and modulus of elasticity, it provides a better and more consistent support for pavement. Stiffening the base and sub-base layers through lime further provides an improved load-spreading capability of the pavement structure, protecting the sub-grade from over stress.

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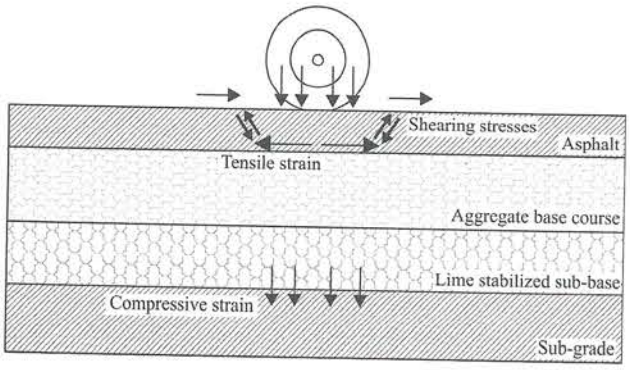

Figure 1 - Critical stresses in a typical flexible pavement (Little, 1995).

In addition, strengthening the base course reduces the stiffness ratio between the surface and the base or sub-base layers. This ratio, together with the contact stresses of the wheel load and the thickness of the layers, controls the stresses and strains developed in the pavement. A stiffer base course will then improve the tensile and shearing strengths of the surface layer, favouring loads distribution and reducing fracture and deformation. Knowledge of the compressive strength of the lime-soil mixture is a useful indicator of the ability of the base to resist shear failure. The unconfined compressive strength (UCS) is the most widely used property to evaluate mixture strength. Tensile strength and flexural tensile strength are approximately 13% and 25% of the UCS (Little, 1995).

2. Methodology and Material Characterisation

The design of lime-soil mixtures supported by laboratory tests aimed at determining the percentage of lime to be added, as a function of the lime type and the nature of the soil, bearing in mind the desired performance. Two different soils were used in this research, both prepared in the laboratory through the mixing of different percentages of kaolin and sand (Table 1). Soils thus obtained are called S1C0 and S2C0.

According to the Unified Classification (ASTM D2487-06), S1 is a "Low Plasticity Sandy Clay" (CL) while S2 is a "Clayey Sand" (SC). According to AASHTO classifications (AASHTO M145-91, 2001) used in road engineering, S1 is A-6 (I_g = 9) while S2 is A-2-6 (I_g = 1).

The plasticity index of soil S1C0 is 17.55% (w_L = 35.70% and w_P = 18.15%) while S2C0 has a plasticity index of 14.30% (w_L = 27.60% and w_P = 13.30%).

Regarding the compaction characteristics of both soils, the Proctor principles were followed using BS 1377. Although there are different types of compaction-related laboratory tests, each with procedural variations related to the nature of the soil, the effect of the water content of the soil on the resulting dry density (dry unit weight) is similar for all methods. The compaction characteristics of the original soils can be found in Table 2.

From X-ray traces it was possible to detect, at peaks 7.08; 3.56; 2.33 and 1.67 ⊕, the presence of clay minerals from the kaolinite type. In both difractograms, the presence of quartz is also very clear.

The kaolin used has significant quantities of alumina and silica which are important for the development of the cementation gel, responsible for the strength gain and durability performance of stabilised soils. The ionic exchange capacity was not high in either soil, which can be justified by the reduced capacity of the constitutive clay minerals (Table 3). Calcitic hydrated lime 99% pure (ASTM D632-01) and sodium chloride were used.

3. Results on Soil Modification

On sample preparation the soil was mixed with water 24 h prior to the mixing with lime, so that the water content could homogenise. The lime was thoroughly mixed with the wet soil before the sample was fabricated in the mould and kept wrapped in cling-film, for the desired curing period, in a 25 °C and 100% moisture chamber.

The particle size was substantially altered after the addition of 2% lime, as can be seen in Fig. 2. The cation exchange and subsequent reduction of the diffused water layer surrounding the clay particles agglomerated them. The effect was an apparent reduction of particles smaller than 0,074 mm, as can be seen from the sieve analysis: from 69% to 11% for S1 soil and from 43% to 5% for S2 soil (S1C2 and S2C2 refer to soils S1 and S2 with 2% lime).

Table 2 - Compaction characteristics of the original soils.

Soil	Optimum water content (%)	Maximum dry unit weight (kN/m^3)
S1C0	17.0	17.7
S2C0	10.9	18.0

Table 1 - General particle size characteristics of the original soils S1C0 e S2C0.

		Particle size distribution						
						Percentage passing		
Soil	Kaolin	Sand	Sand	Silt	Clay	N. 10	N. 40	N. 200
S1C0	65%	35%	34%	15%	51%	100%	89%	69%
S2C0	35%	65%	62%	8%	30%	100%	86%	43%

Table 3 - Most important chemical characteristics of the original soils.

		Soil	
		S1C0	S2C0
Main chemical components	Silica (SiO_2)	54.60%	69.36%
	Alumina (Al_2O_3)	28.69%	20.11%
	Iron (Fe_2O_3)	1.44%	1.04%
	Calcium (CaO)	0.02%	0.02%
	Magnesium (Mg)	0.18%	0.11%
	Others	15.07%	9.36%
Ionic exchange capacity (mE/100 g)	Ca^{++}	1.15	1.36
	Mg^{++}	0.24	0.29
	K^+	0.08	0.14
	Na^+	0.06	0.11
	SOM	1.53	1.90
pH	H_2O	6.7	7.0
	KCL	5.5	5.5

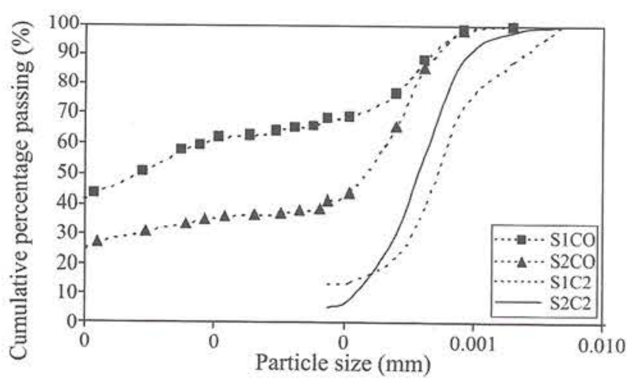

Figure 2 - Particle size distribution of mixtures S1C2 e S2C2 and original soils S1 e S2.

With regard to the plasticity index of soil S2 it reduced from 17.55% to 9.10%, while the opposite happened with soil S1, raising from 14.30% to 23.40%. This raise in the plasticity index of soil S1 is due to a significant increase in its liquid limit (from 35.70% to 46.00%). After treatment, and as a result of the flocculation of the particles, the classification of both soils changed. S1 became a "Well Graded Sand" (SW), and S2 was classified as "Poorly Graded Sand" (SP). Using AASHTO they were classified as A-2-7 ($GI = 0$) and A-2-4 ($GI = 0$).

In both soils it was observed an increase in the optimum water content and a decrease in the maximum dry density (Table 4).

The strength gain of the mixtures with 2% lime was determined using triaxial tests, unconsolidated and undrained, with confinement pressures of 50, 100 and 200 kPa. The improvement of the undrained cohesion (C_u) was only noted in the first days of curing (Fig. 3). The small quantity of lime added was rapidly consumed and so did not contribute to further improvements in mechanical strength. After 3 days of curing, C_u showed increases of 395% and 140% for soils S1 and S2.

Table 4 - Compaction characteristics of the soil-lime mixtures.

Soil	Optimum water content (%)	Maximum dry unit weight (kN/m^3)
S1C2	18.8	16.3
S2C2	12.6	17.5

4. Results on Soil Stabilisation

4.1. Axial deformation

The deformation of the original soils was typical of a ductile material, without a peak value. Mixtures with 6 and 10% lime (C6 and C10) presented a reduction of ductility with curing time and the amount of lime added. Figs. 4 and 5 indicate a similar deformation behaviour for soil S1 and S2.

4.2. Strength gain with time

The strength gain of the mixtures S1C6 (soil S1 with 6% lime) and S1C10 (Soil S1 with 10% lime) can be divided into different phases (Fig. 6). The first phase is a rapid strength gain (Fig. 7), in which the difference between the strength of S1C6 (665 kPa) and S1C10 (930 kPa) is around 40%, as a direct result of the higher quantity of lime present in mixture S1C10. Following this phase a period of practically constant strength is observed, which is due to an induction period for the pozzolanic reactions.

The next phase is due to high rates of pozzolanic reactions and a significant strength gain is observed. After ten weeks curing the strengths were 1159 kPa for S1C6 and 1412 kPa for S1C10, which corresponded to an increase of 460% and 580% in relation to the original soils. The difference of strength between the two mixtures, however, dropped to 22%. The strength gain of the two mixtures fol-

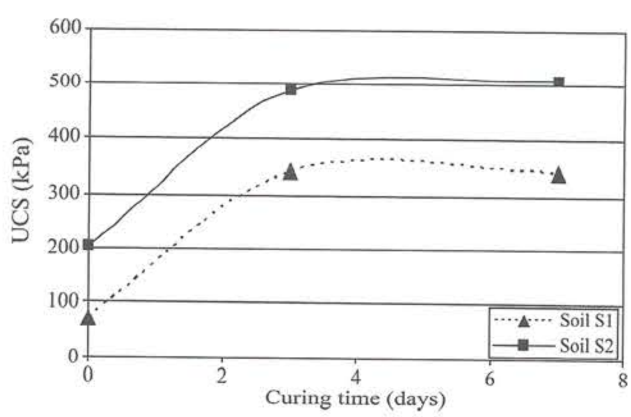

Figure 3 - Undrained Shear Strength (C_u) with 0 and 2% lime.

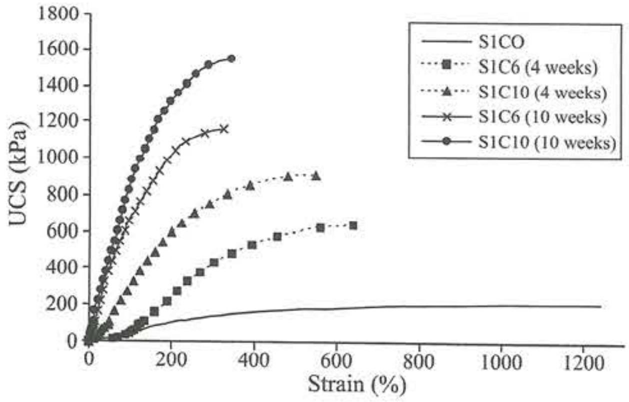

Figure 4 - Stress/Strain behaviour of soil S1.

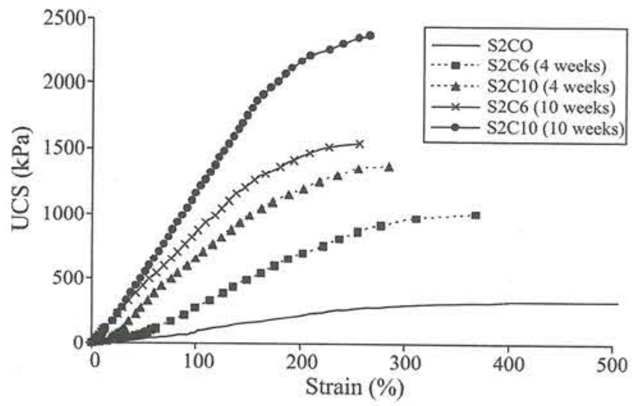

Figure 5 - Stress/Strain behaviour of soil S2.

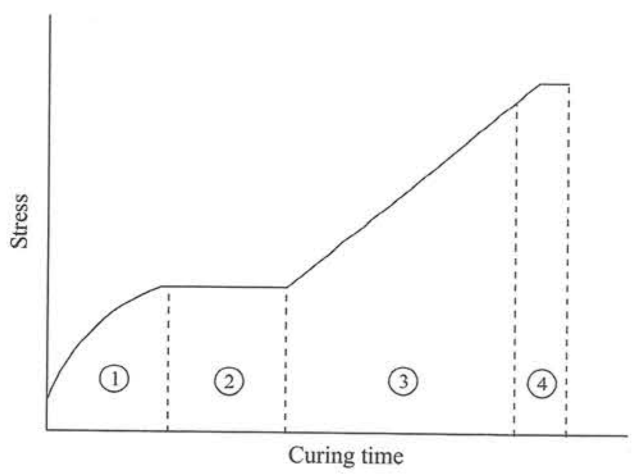

Figure 6 - Proposed model for the strength evolution of a soil-lime mixture with curing time.

lowed a similar pattern and it is expected that with longer curing time, the strength of the S1C6 mixture would start to level off before the S1C10 mixture. Comparing mixtures S2C6 and S2C10, a difference of 60% in strength was observed after ten weeks curing: S2C6 achieved 1575 kPa (345% increase), while the S2C10 mixture achieved 2530 kPa (615% increase). This significant difference, especially when compared with the one observed in the S1 soil, is due to the fact that S2 soil contained a higher quantity of granular material, and so required less cementation material to bond the particles.

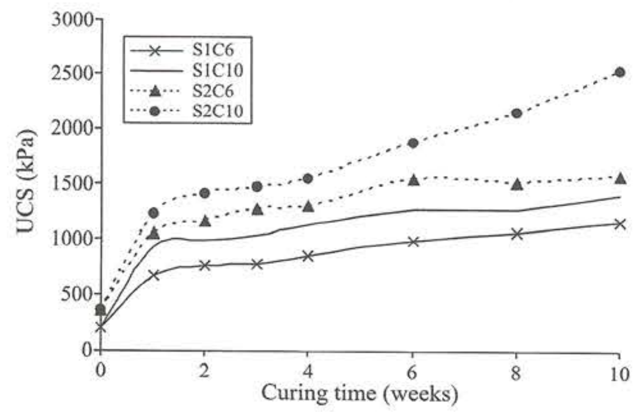

Figure 7 - Strength gain of soil stabilised with 6 and 10% lime with curing time.

4.3. Strength gain using chemical activators

The addition of sodium chloride (NaCl) to a soil-lime mixture has been reported to act as a catalyser, where Cl^-, Na^+, Mg^{++} ions accelerate the pozzolanic reactions (Bergado *et al*, 1994). In order to quantitatively evaluate the influence of this chemical activator in the stabilisation of residual granite soils, tests were made on samples designated as S1C6A (Fig. 8) and S2C6A (Fig. 9).

After 8 weeks curing these specimens with the activator and 6% lime (S1C6A) indicated a significantly higher strength than the mixture without activator (S1C6). The strength gain due to the activator was 42%. It is noted that the strength with activator and 6% lime was 23% higher than specimens with 10% lime (S1C10). In the case of soil S2, the rate and level of strength achieved were similar. This clearly indicates that adding 10% lime to the original soil is basically the same as adding 6% lime and 0,12% NaCl. It is also noted that, the addition of activator in S2C6, resulted in a 51% increase of strength at 10 weeks curing time.

4.4. Durability measured by wet-dry test

To study the water sensitivity, tests were performed on samples submerged in water 24 hours prior to testing. Regarding soil S1, it was noted (Fig. 10) that the loss in mechanical strength (in kPa) was constant with curing time, which resulted in a reduction in the strength loss / strength ratio. The same thing happened with soil S2, where again the difference between strength gain of S2C10 saturated and unsaturated samples was constant (Fig. 11). The strength loss of saturated samples compared to unsaturated samples did not increase with time, and that might be explained by an elimination of the internal suction due to the saturation of the samples (Cristelo, 2001). This strength

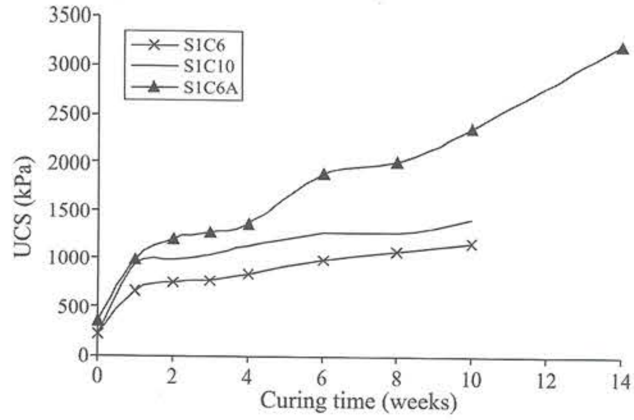

Figure 8 - Unconfined Compressive Strength of Samples S1C6, S1C10 e S1C6A.

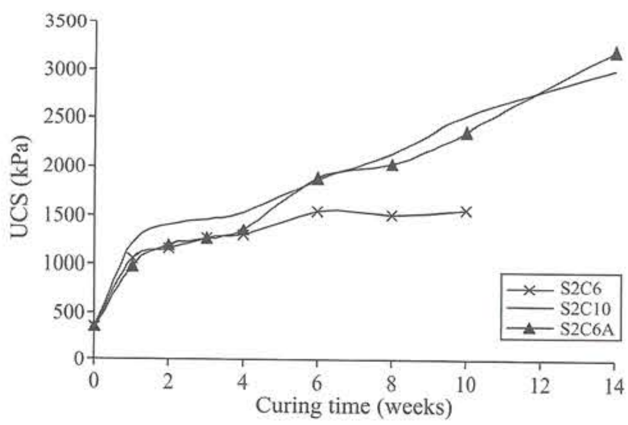

Figure 9 - Unconfined Compressive Strength of Samples S2C6, S2C10 e S2C6A.

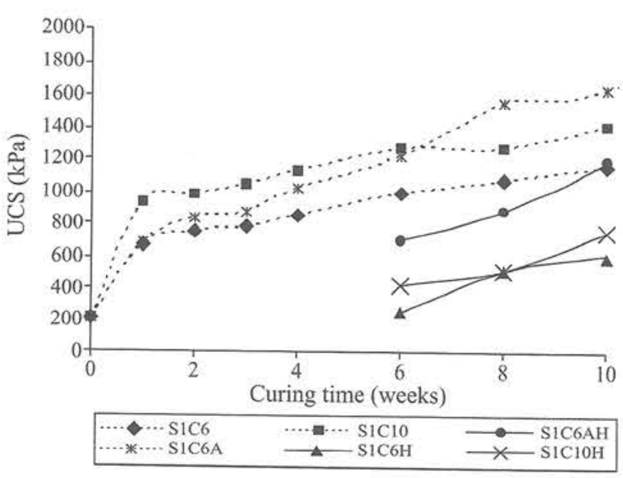

Figure 10 - Strength of the saturated and unsaturated samples - soil S1.

loss, measured after 6 weeks, was a fraction of the strength. Hence, it is possible that at long curing times and with a high enough lime content, the strength gain will be high enough that strength loss due to saturation can be considered negligible.

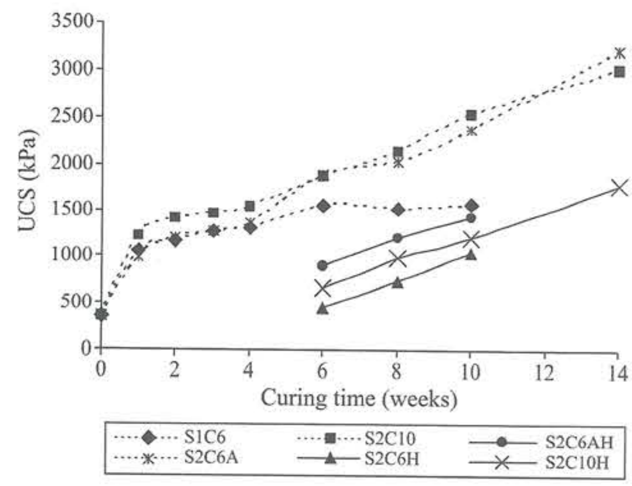

Figure 11 - Strength of the saturated and unsaturated samples - soil S2.

Tests were also performed to evaluate the effect of saturation on samples with NaCl and, once again, it was observed (Figs. 10 and 11) that the strength loss due to saturation remained constant and similar to what occurred with specimens without the activator. This indicates that the cementation materials formed are not sensitive to water and hence can be considered permanent.

Moreover, the development of the reaction products was observed by scanning electron microscope. Micrographs obtained from samples S1C6 and S1C6A are presented in Fig. 12. These figures indicate the formation of cementation materials in the matrix. It is also noted that specimens with activator have higher amounts of cementation materials.

5. Conclusions

The results obtained indicate that 2% lime addition causes: a decrease in plasticity, an increase in optimum water content necessary to obtain the maximum dry unit weight (enabling construction under wetter soil conditions), and a shift in the grading curves due to the reduction of the clay sized particles with a modest immediate gain in strength.

The results obtained with higher percentages of lime illustrated an increase in strength with curing time and lime content.

The strength loss on saturated mixtures remained constant with time, which indicates that the cementation materials formed are not sensitive to water. The loss of strength on saturation is explained by the reduction in capillary suction potential.

The addition of NaCl not only increased the strength at short times, but also decreased the deformation and water sensitivity. In fact, results indicate that adding small quantities of this activator is preferable to increasing the lime content.

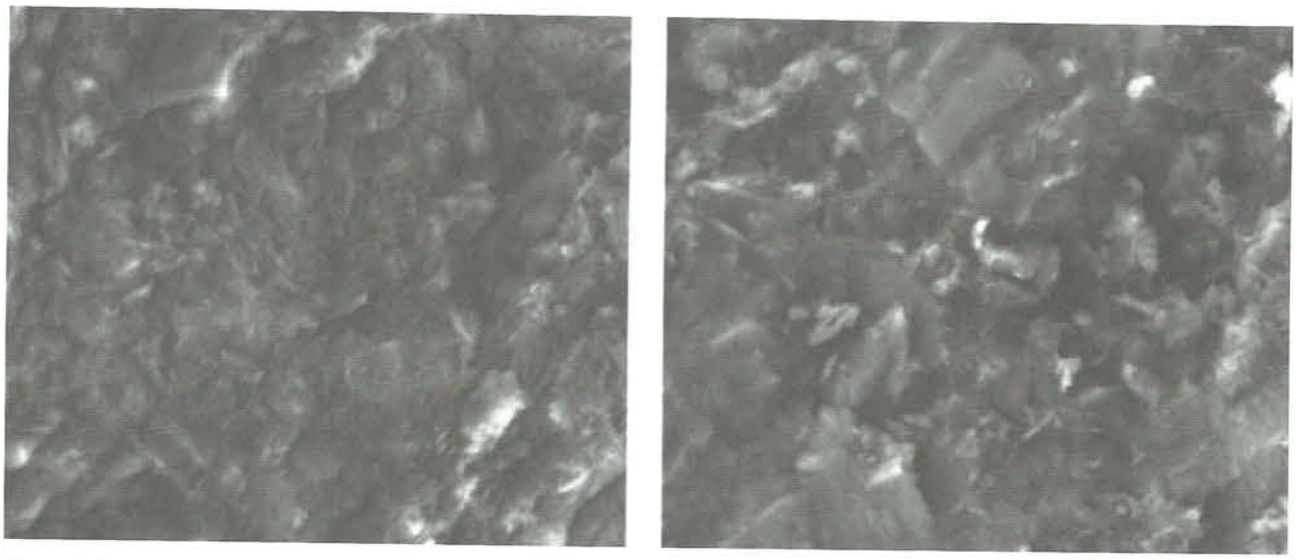

Figure 12 - Micrographs from sample S1C6 at 3 and 15 days curing time (x 5000).

According to the strength gain observed with the addition of lime, the use of this process for stabilizing residual granite soils seems to be a viable practice, as both the strength and durability are enhanced. A sub-base of this material, with a rigid or flexible pavement will provide a stable support.

Furthermore, the construction costs will be considerably lower than using traditional methods, because the existing soil is stabilized in place instead of being removed and replaced, and also because the other layers of the pavement can be reduced in thickness, due to the improved strength and stiffness of the sub-base layer. Not only that, the life span of the pavement is increased, which also reduces costs in the long-run.

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Technical Note

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The Use of a High Capacity Tensiometer for Determining the Soil Water Retention Curve

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Abstract. Soil water retention curve (SWRC) plays a fundamental role in characterizing hydraulic and mechanical behavior of soils and porous material in general. However, the determination of the SWRC is usually time-consuming. The objective of this paper is to describe a procedure for the determination of the SWRC using a high capacity tensiometer (HCT). The procedure described here allows the attainment of the SWRC in less than 7 days, with suction measurements up to 500 kPa. The study was carried out using three soils and using specimen prepared with different sizes. It was then possible to evaluate the influence of the type of soil and dimension of the specimen on the retention curve obtained. In order to evaluate the accuracy of the curve obtained with the HCT others methods were also used (*e.g.* suction plate, pressure plate and filter paper). The results indicated that the use of the HCT presents results equal to the ones obtained using conventional methods. Its use allows a significant economy of time in the determination of the SWRC.

Keywords: soil water retention curve, tensiometer, unsaturated soil, soil suction.

1. Introduction

The water retention curve is an important source of parameters for soils and it is a way to understand and to predict mechanical and hydraulic behavior of porous material in general. Usually the soil water retention curve (SWRC) is obtained using one or more of the following methods: suction plate, pressure plate, filter paper, desiccators, among others. The suction plate is used for low range of suctions, usually below 30 kPa (this value is related to practical height limitation in most laboratories). The pressure plate is used for suction between 20 kPa and 1500 kPa (the upper limit is due to the air entry pressure of the porous material used to apply the axis translation technique). These two methods require an average of three days for each point of the SWRC. The filter paper can be used for suctions higher than 10 MPa. However, the method require at least 7 days for each point.

Independently of the method used for obtaining the SWRC there are two types of equilibrium to be considered. One is related to the method used to measure the suction, and the other is related to the equilibrium within the sample itself. The equilibrium time for the most common methods used for obtaining the SWRC (*e.g.* suction plate, pressure plate and filter paper) varies from 3 to 15 days for each point.

For the suction plate and pressure plate the specimen under test changes its suction according to the level of suction imposed by the system. In the case of the filter paper method, usually the specimen is submitted to air dry for few minutes or hours. After that the filter paper is placed in contact with the sample that may not be in equilibrium. The drying process to bring the specimen to the next level of suction may induce a higher suction at the boundaries. The time for equilibrium of the specimen is included in the time for equilibrium of the filter paper.

The technique presented in this paper uses a high capacity tensiometer (HCT) that measures the suction of the sample in few minutes (*e.g.* Oliveira & Marinho, 2008). The fact that the technique only requires few minutes may induce one to forget about the specimen suction equilibrium.

2. Background

The HCT is one of the most exciting discoveries in soil mechanics of unsaturated soils, although its use is still restricted to some research centers. The high capacity tensiometer used for this investigation is capable of measuring suctions up to 500 kPa. This type of tensiometer was originally developed by Ridley & Burland (1993) and similar equipment has been used in many researches (*e.g.* Marinho & Pinto 1997, Tarantino & Mongiovì, 2002; Take & Bolton, 2003).

The tensiometer used in this research was developed at São Paulo University, and has been used as routine equipment for inferring the suction prior many kinds of tests. Figure 1 presents a schematic draw of the HCT. It consists of a transducer, a high air entry pressure porous ceramic and water as all ordinary tensiometers. The ability to measure high suction is due to the saturation technique imposed to the system (*e.g.* Marinho & Pinto, 1997; Take & Bolton, 2003, Marinho *et al.*, 2008).

The determination of data relating the soil suction, measured using the HCT, with the amount of water the soil can hold can already be found in the literature (*e.g.* Guan & Fredlund, 1997; Fredlund *et al.*, 1997; Boso *et al.*, 2003; Toker *et al.*, 2004; Lourenço *et al.*, 2007).

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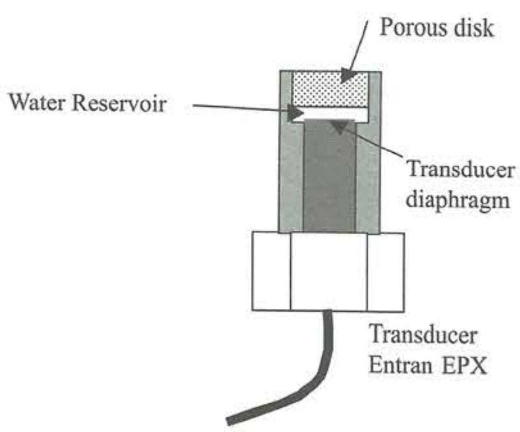

Figure 1 - Schematic representation of the HCT (Marinho & Pinto, 1997).

Guan & Fredlund (1997) used compacted glacial till specimens to evaluate the performance of the HCT in comparison with others techniques (*i.e.* pressure plate, thermal sensor and filter paper). The data agreed reasonably well, apart for the data using the thermal sensor that showed some discrepancies for suctions higher than 300 kPa. For the comparison with the filter paper some discrepancies were also observed for suctions higher than 500 kPa.

Boso *et al.* (2003) presented the results of SWRC of reconstituted silt clay using a HCT to monitor the suction in two ways. One during a continuous drying process (what was called dynamic process) and also under constant water content (static process). For the dynamic process the soil specimen was placed along with the tensiometer on a balance that continuously monitored the change in specimen mass. The static process was performed using one specimen that was air-dried to target water contents. Before measuring the suction each specimen was allowed to equalize for at least two days. The use of the two processes in the silt clay soil presented a good correlation, suggesting that there was a uniformity of suction within the sample even during the dynamic process. When comparing the axis-translation technique with the HCT Boso *et al.* (2004) observed that for suctions higher than 100 kPa, the water stored by the soil was lower than the one measured during the use of the HCT. Although the differences were small they associated that difference to the evaporation phenomenon that occurs on the top of the specimen during the use of the axis translation technique. For more details about that the reader should refer to their paper, since this discussion is out of the scope of this paper. It should be pointed out that during static or dynamic determination of the SWRC using the HCT the effect of temperature of the specimen may be also a problem. Regarding this matter the reader can also report to Fredlund *et al.* (1997).

Toker *et al.* (2004) presented results for the SWRC using the dynamic process applied to specimens of fine sand and glass beads. Their system was very well sealed in order to prevent evaporation other than from the top. The results obtained from four specimens of fine sand proved to be repeatable. The data obtained with the glass beads specimens also presented good results having two points of the curve compared with data from suction plate test, showing good agreement. Toker *et al.* (2004) observed that in almost all experiments, near the dessaturation point, there were drops in suction of 10 to 60%. This sudden reduction in suction was associated to the shrinkage of the sample causing an increase in the degree of saturation. The tests were performed without any paste between the tensiometer and the specimens.

Compacted specimens of sandy clay were used by Lourenço *et al.* (2007) to obtain the SWRC using both dynamic and static techniques. In both situations the specimen remained inside the mould all the time. For the static technique the drying process was interrupted at a time and equilibration was allowed measuring the suction until equilibrium was attained. According to Lourenço (2007) the time for equilibrium ranged from 6 to 48 h. It should be pointed out that the tensiometer used a 15 bars ceramic and no paste was used between the tensiometer and the specimen.

From the two techniques to obtain the SWRC using the HCT (*i.e.* dynamic and static processes) the dynamic is more sophisticated and elegant, but could induce errors due to non equilibrium of suction within the specimen. The static technique can be performed without a balance dedicated to the test. The static process is simpler in terms of instrumentation, although requires more manipulation of the sample.

Despite the fact that the HCT has been used to determine the SWRC it seems to be important to investigate some aspects of the procedure. In order to evaluate the effect of the equilibrium time during static determination of the SWRC tests were performed using three different soils with specimens with two different sizes.

3. Characteristics of the Soils Used

The study was carried out using three soils: A residual soil of gneiss from the experimental site of the São Paulo University, a residual soil of granite from the Serra do Mar area, located at the coast of São Paulo State and the third soil is a clay from São Paulo city (originally a porous red clay). The characteristics and identification of the soils are presented in Table 1.

4. Specimens preparation

From each soil four specimens were statically compacted dry of optimum. Two of them had a diameter and height of 38 mm, and the other two with 70 mm in diameter and height. The objective of using specimens with different sizes was to evaluate the effect of equilibration time within the sample after air drying. A taxonomic scheme was created to easily identify the soil, the size, the specimen number and the technique used to measure the suction. The two initial digits identify the soil used. The third and fourth digits iden-

Table 1 - Characteristics of the soils used.

Soil	w_l (%)	w_p (%)	G	w_{opt} (%)	% sand	% silt	% clay	Site
Residual soil of gneiss - ES	47	34	2.73	23.5	34	46	20	Experimental site (University of São Paulo)
Residual soil of granite - SM	49	43	2.75	22	61	36	3	Coast of São Paulo State at Ubatuba City
Red clay - RC	78	47	2.75	35	8	14	78	Paulista av. (São Paulo)

tify the size of the sample (b for big sample and s for small sample) and the label refers to the specimen number. The information that appears after is related to the type of technique used for measuring the suction (CON for the conventional methods and HCT for the high capacity tensiometer).

Table 2 presents the characteristics of the specimens used for the study. The dimensions of the specimens were obtained using a vernier caliper. All specimens were allowed to absorb water at the suction plate in order to attain the maximum degree of saturation. For the specimens labeled with the number 1 the method used to induce the suction was the suction plate (for suction between 0 and 30 kPa) and pressure plate (between 30 kPa and 500 kPa). After 500 kPa the filter paper method was used. Before increasing the suction the specimen was placed on the HCT for measuring the suction. In this way for specimens labeled with the number 1, there are measurements made with conventional methods and also with the tensiometer. For the specimens labeled with the number 2, only the HCT was used and the suction was induced by allowing the specimen to air dry. In some cases the use of a fan was necessary to speed up the drying process.

For measuring the suction using the HCT the specimens were placed on the base where the tensiometer was installed, and covered with cling film to avoid evaporation. Between the porous ceramic of the HCT and the specimen it was used a kaolin paste. The use of the paste is necessary to establish a hydraulic continuity between the water of the tensiometer and the soil water (Oliveira & Marinho, 2008). A general detail of the system for measuring the suction using the HCT is shown in Fig. 2.

5. Test Results and Discussion

5.1. Comparison between the HCT and conventional methods

The data for the soil water retention curves (SWRC) are presented in terms of volumetric water content versus suction for all three soils tested. The results are presented according to the size of the specimen used and type of method used for measuring the suction. Figure 3 presents the data for the residual soil from the experimental site (ES) of the University of São Paulo. Figure 3a presents the results for the bigger specimens and Fig. 3b for the smaller specimens. The results obtained with the specimens ESb1 and ESs1 suggest that measurement made using the HCT agree reasonably well with the measurements obtained by the conventional methods (*i.e.* suction plate and pressure plate). The measurements made using the filter paper method presented a reasonable continuity for the SWRC, although showing in some cases its inability of measuring matrix suction only. The results from the specimen ESb1 presented a bigger scatter that seems to be due to the volume change measurements (the volume change is performed using a vernier calipers reading

Table 2 - Characteristics of the soils tested.

Soil	Specimen	w (%)	e (initial)	S (%) (initial)	Suction (kPa) after compaction
ES	ESs1	21.8	0.78	76	102
ES	ESs2	21.8	0.74	80	202
ES	ESb1	21.8	0.71	83	166
ES	ESb2	21.8	0.76	79	0
SM	SMs1	23.3	0.86	74	127
SM	SMs2	23.3	0.84	76	127
SM	SMb1	23.3	0.89	72	0
SM	SMb2	23.3	0.89	72	0
RC	RCs1	29.4	0.83	97	430
RC	RCs2	29.4	0.83	97	430
RC	RCb1	30	0.84	98	> 500
RC	RCb2	30	0.85	97	> 500

to 0.01 mm) and it is also associated to the fact that the specimen 2 was less affected by constant manipulation since it was used only with the HCT and not for conventional suction control methods. The results obtained with the specimen ESb2 shows a much smaller variability on the results. Since each specimen size were tested independently, showing a good agreement between methods, the difference observed on the SWRC of the bigger specimen as compared to the smaller one is likely to be due to structural differences among the specimens.

The results obtained with the residual soil from Serra do Mar (SM) is presented in Figs. 4a and 4b, following the same approach presented for the previous soil. Both, for the specimen SMb1 and SMs1, the comparison between the data obtained with the HCT and using the conventional methods showed also a good agreement. When comparing specimens SMb2 and SMb1, some difference is observed. This difference may be justified by sample disturbance or due to the volume change measurement technique or even some difference on the pore size distribution of the two specimens at a suction level between 10 kPa and 100 kPa. The results for the small specimen showed a reasonably good agreement between methods and also between the two specimens tested.

Tests performed with the red clay (RC) are shown in Fig. 5. In this case the eventual error related to the suction measurement may not be detectable due to the shape of the SWRC. The results showed a very good agreement between the conventional method and the HCT, regardless of the size of the specimen. The data showed a behavior associated with stiff clays (*e.g.* Marinho, 2005). It should be pointed out that being a stiff clay the SWRC is almost horizontal for the level of suction measured.

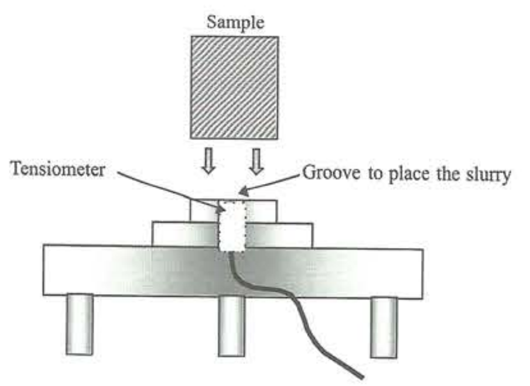

Figure 2 - General detail of the system for measuring the suction with the HCT.

5.2. The effect of the equilibrium time on the HCT measurement

In order to evaluate the eventual effect of the drying process and posterior equilibrium time within the specimen, suction was measured using the HCT at three different

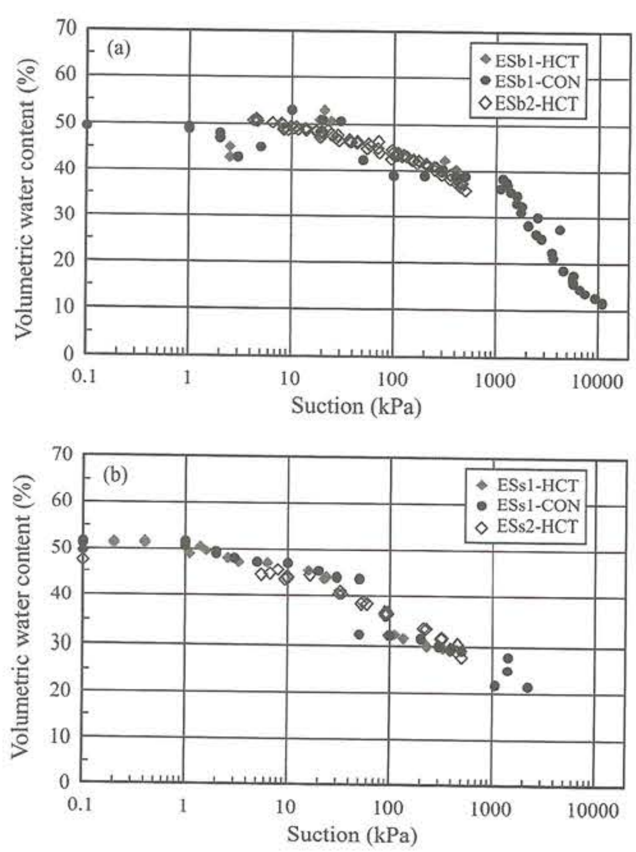

Figure 3 - Conventional and HCT retention data from ES soil.

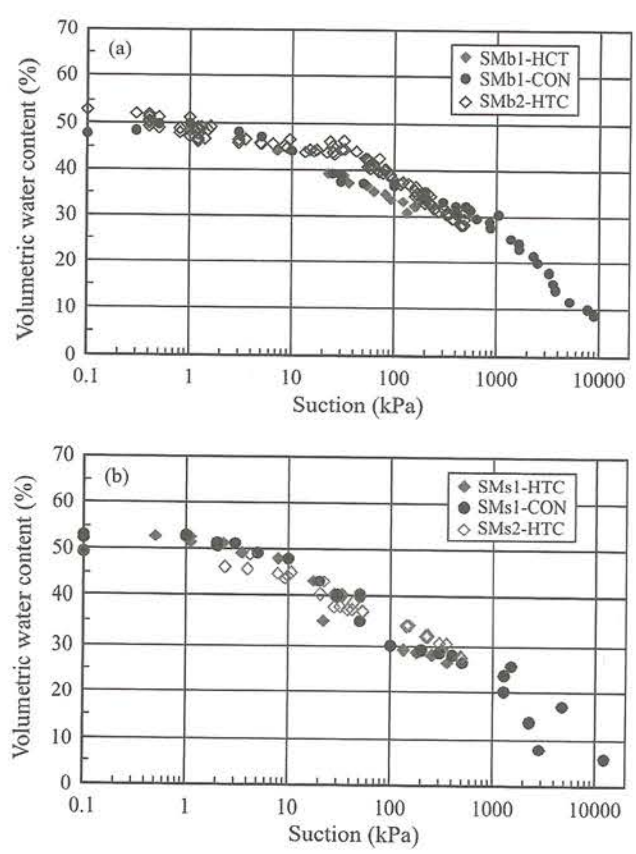

Figure 4 - Conventional and HCT retention data from SM soil.

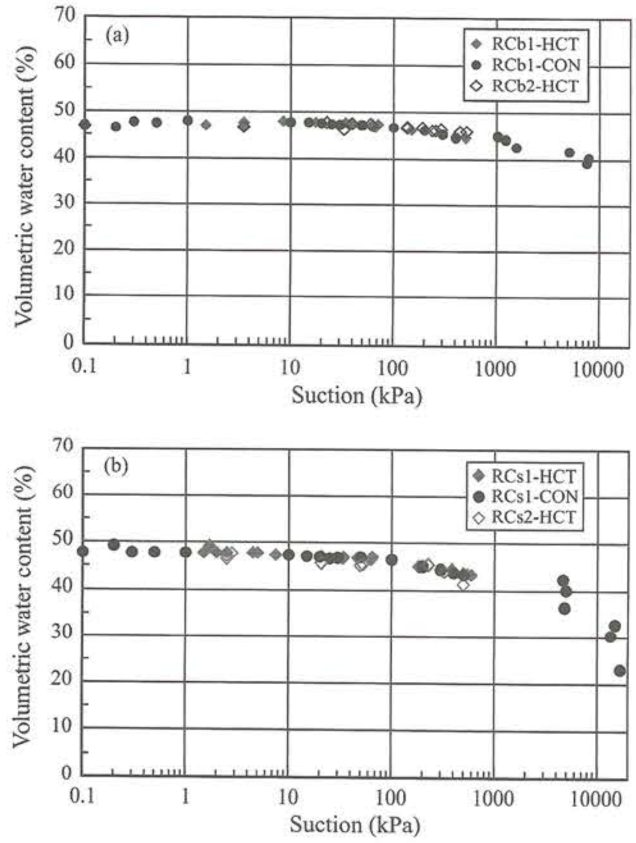

Figure 5 - Conventional and HCT retention data from RC soil.

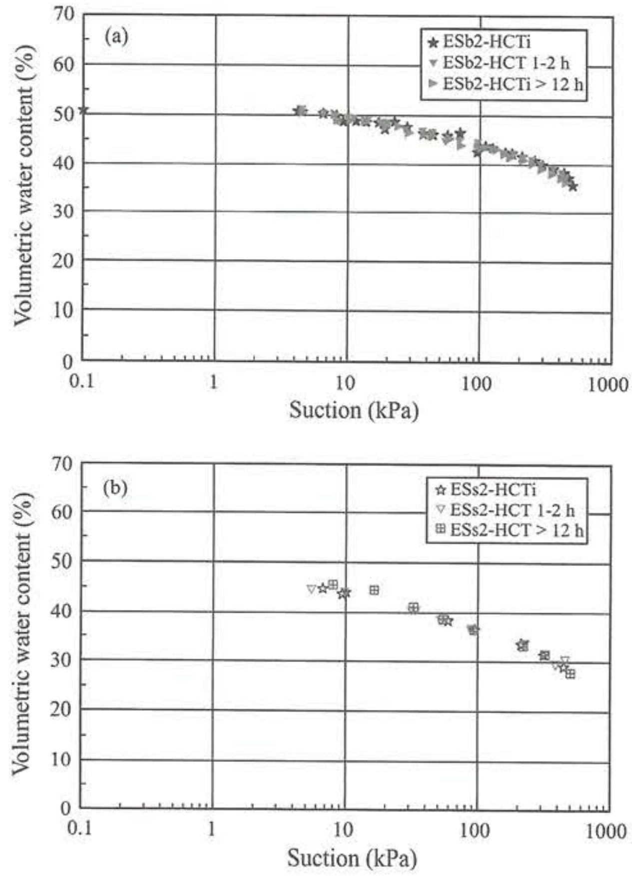

Figure 6 - Retention data at different time for the ES soil.

moments, immediately after the drying process, two hours after, and after 12 h. In some cases more than one measurement was taken for the same equivalent time.

It should be pointed out that the tests were carried out to a suction of about 500 kPa in most cases. For higher suction the equilibrium time may be an important issue due to a lower hydraulic conductivity of the specimen at higher suctions. The results showed that, for the soils tested, there is no difference in the suction measured, regardless the time left for eventual equilibration. In Figs. 6, 7 and 8 are presented the results obtained with the residual soil of gneiss (ES), the residual soil of granite (SM) and the red clay (RC) from São Paulo city, respectively.

6. Conclusions

The paper has presented a procedure to obtain the soil water retention curve (SWRC) using a high capacity tensiometer. The following conclusion can be stated based on the tests performed and the soil used:

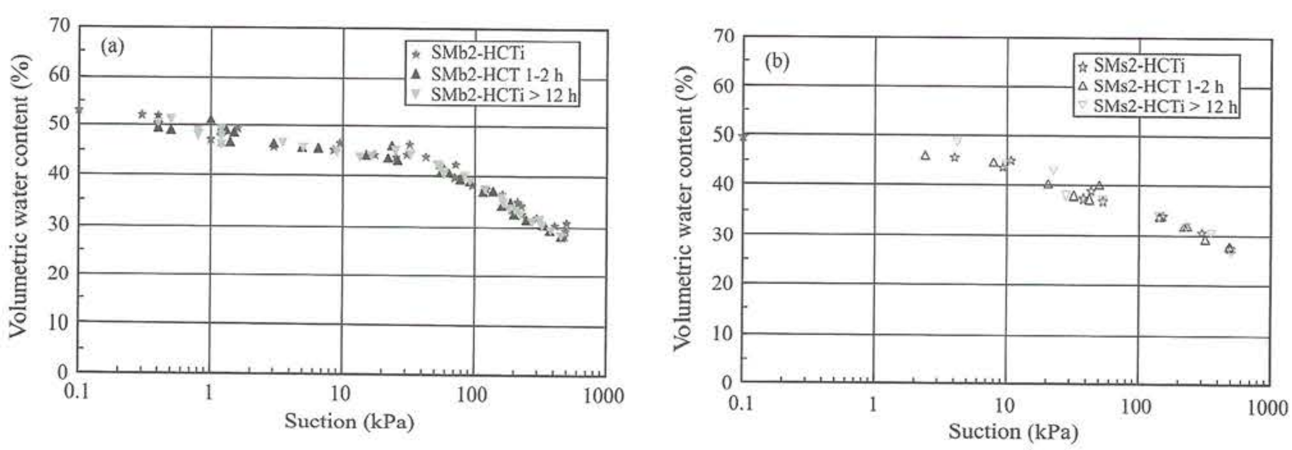

Figure 7 - Retention data at different time for the SM soil.

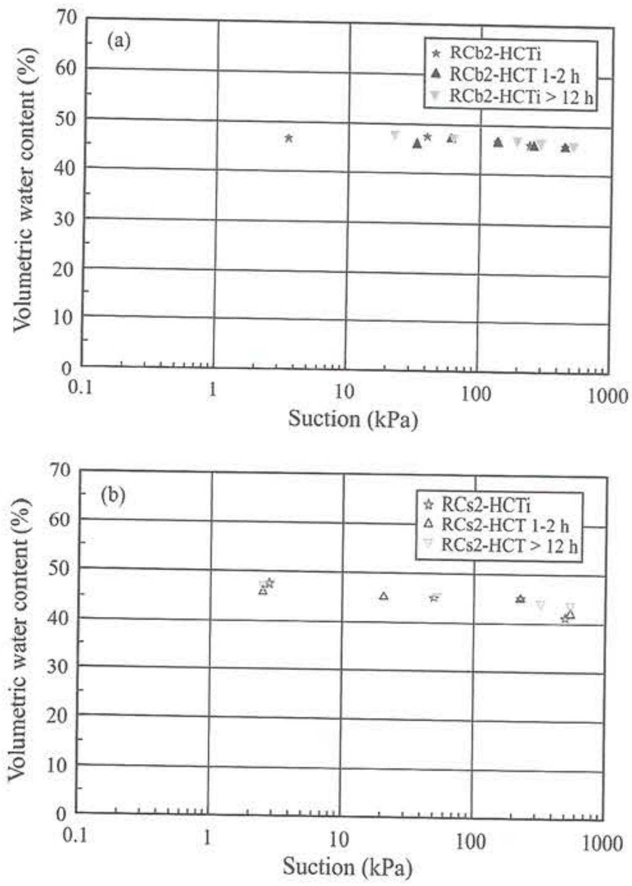

Figure 8 - Retention data at different time for the RC soil.

• The HCT can measure suction in an interval of time that varies from 20 min to 60 min and this is a great improvement when compared to the more traditional methods of suction measurement.

• The use of the HCT allowed the determination of the SWRC (up to 500 kPa) in approximately 5 days. The limitation of the time is related to the ability to dry the sample and not due to the technique used to measure the suction.

• A comparison between the data obtained using the conventional methods and the HCT showed that there is no apparent differences in the SWRC obtained using the HCT and others techniques.

• The drying process did not affect the SWRC obtained regardless the time left for the eventual equilibration and the size of the specimen. This fact suggests that for the soil tested there is no need for equilibration time.

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Discussions

Soils and Rocks
v. 32, n. 2

Laboratory Behaviour of Rio de Janeiro Soft Clays. Part 1: Index and Compression Properties

Discussion by

Kátia Vanessa Bicalho, Reno Reine Castello

The authors presented a comprehensive and useful synthesis of index and compression properties of Rio de Janeiro soft clays. It is useful not only for Rio de Janeiro but for most of Brazil, whose coastal plains share the same origin. The writers congratulate the authors for the work.

Of minor practical importance, but not so for teaching purposes, there is a mistake (maybe a mistyping) on the plasticity classification of clays of the so called Region IV (Fig. 4 and Table 2). They probably are the same clays of Regions II and III (medium to high plasticity) displaced to the right in the Casagrande's chart by a larger content of organic matter. They are not low plasticity clays. The results presented in Fig. 4 have the plasticity index (I_p) limits for each region different from those specified in Table 2. The organic matter increases plastic and liquid limits, but not the plasticity index (Casagrande, 1948; Bain, 1971).

Another point is about the correlation between compression index (C_c) and natural gravimetric water content (w). The writers have also found (Castello & Polido, 1986) better correlations for water contents (a state) than for liquid limits (a soil characteristic). On the other hand Terzaghi & Peck (1948), possibly the pioneers on this type of correlation, tried the liquid limit ($Cc = (w_L - 10)\ 0.009$), which seems more rational. Do the authors have an explanation why the water content works better?

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Received and accepted on December 12, 2008.

Laboratory Behaviour of Rio de Janeiro Soft Clays. Part 1 : Index and Compression Properties

Discussion by

Ian Schumann Marques Martins, Paulo Eduardo Lima de Santa Maria, Flavia Cristina Martins de Santa Maria

1. Introduction

The authors are to be congratulated for presenting a paper with a significant amount of data on soft soils. The kind of data presented is of great value for geotechnical engineering practice, particularly for those involved with soft soils.

To facilitate the discussion's follow-up, the discussers have used the same items that have been used by the authors in the paper.

2. Soil Characterization

As one of the main aims of the paper is to provide practical data for pre-design purposes, it would be adequate to more precisely locate each place studied. Being so, it is suggested to replace the name "Uruguaiana Clay" by "Uruguaiana Street Clay in the Neighbourhood of the Uruguaiana Subway Station" (Vilela, 1976). In a similar way, it is also suggested to use "Botafogo Clay near Botafogo Subway Station" (Lins & Lacerda, 1980), instead of "Botafogo Clay" and so on.

It would be also adequate to present the geotechnical profiles shown in Fig. 1 in a more detailed way because some of them do not reproduce field conditions or do not unveil important information. These are the cases of Caju, Uruguaiana, Botafogo and Barra da Tijuca profiles.

Concerning "Caju Clay", the samples were taken from a soft clay layer beneath a failed embankment. The undrained shearing strains that took place in the soft clay layer due to failure were of such a magnitude that the clay structure was destroyed. Then, despite the care taken during sampling, the retrieved samples could never represent the natural conditions under which the deposit was formed. Although these conditions were reported by Cunha & Lacerda (1991), this information was neglected by the authors. Notwithstanding, if one of the main aims of the paper is to establish a comparison between parameters and properties of Rio de Janeiro soft clays, the conditions under which the determination of such properties and parameters were carried out must be clearly stated.

As a contribution to Table 1, Cunha & Lacerda (1991) have reported that "Caju Clay" has an organic matter content of the order of 5% and the amount of clay given by the grain size analysis is above 60%. Besides, the consolidation tests carried out by Carvalho (1989) have shown that in the normally consolidated domain the average value of the coefficient of consolidation is of the order of 1×10^{-8} m^2/s.

The average value of 9×10^{-8} m^2/s presented in Table 1 for the coefficient of consolidation (c_v) of "Sarapuí Clay" seems to be inadequate since for this clay the values of c_v range from 40×10^{-8} m^2/s in the recompression branch to 1×10^{-8} m^2/s in the virgin compression domain (see, for example, Coutinho, 1976).

As far as "Santa Cruz Clay" is concerned, a complete set of properties and parameters listed in Table 1 can be found in the careful work of De Campos (2006). This is an important additional source of information to all those who intend to deal with soft clays from Rio de Janeiro State coast.

The geotechnical profile near Botafogo Station of Rio de Janeiro Subway is not as simplified as shown in Fig. 1. Besides, it should be once more reminded that if one of the main aims of the paper is to compare soft clays from different places of the Rio de Janeiro State coast, one should give as much information as possible regarding the conditions under which samples were withdrawn. In the case of "Botafogo Clay", Lins & Lacerda (1980) have reported the following information, which must be added to that given by the authors:

a) The tested samples were taken from the depth of 10 m and in the middle of a 6 m-thick clayey-sand layer.

b) Grain size analysis has shown that "Botafogo Clay" was made up of 54% fine sand, 18% silt and 28% clay. So, even showing an appearance of a sandy-clay, "Botafogo Clay" is, in fact, a clayey-sand.

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Laboratory Behaviour of Rio de Janeiro Soft Clays. Part 1 : Index and Compression Properties by Almeida, Futai, Lacerda and Marques. Published in Soils and Rocks v. 32(2), p. 69-75.
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c) The coefficient of consolidation of "Botafogo Clay" in the normally consolidated range is greater than 20×10^{-8} m²/s (*e.g.* twenty times the typical value found for the Brazilian coastal soft clays).

d) Sampling was carried out one year after the local water table was lowered.

The preceding information suggest that at least part of the data obtained for "Botafogo Clay" could be possibly affected by consolidation caused by water table lowering. This has probably occurred because "Botafogo Clay" with its 28% of clay content and its high c_v is in the boundary between the soils with predominantly undrained or drained behaviour. Being so, the discussers are of the opinion that it is inadequate to compare "Botafogo Clay" to the other ones shown in Table 1. It would be more adequate to compare the remaining clays in Table 1 to that one also found in the neighbourhood of Botafogo subway station, between 13 m and 19 m. According to some strength parameters that have been shown by Lins & Lacerda (1980), the referred clay could be more properly classified as a soft clay than that described by the authors. It would be interesting to hear from the authors some comments about this clay layer.

In the case of the geotechnical profile of the subsoil in the neighbourhood of Uruguaiana Station of Rio de Janeiro Subway, there is a mistake. It is not shown in Fig. 1 the 10 m-thick sand layer existing above the soft clay layer, as described by Vilela (1976). Besides, the transition from the sand layer to the clay layer occurs at a depth greater than 10 m. This statement is supported by the grain size analysis of a sample taken between the depths of 11.0 m and 11.45 m, which has shown 70%, 25% and 5% for sand, silt and clay fractions, respectively. On the other hand, samples taken between the depths of 13.0 m and 18.45 m are predominantly clayey. So, if only these samples were taken into account to characterize "Uruguaiana Clay", values shown in Table 1 would undergo a substantial change, as shown in Table 3.

Yet, as a contribution to Table 1, it is worth reminding that the best quality samples tested by Vilela (1976), *e.g.* those obtained at depths greater than 13.0 m with 4" (100 mm) diameter tube samplers, gave $C_c/(1 + e_o) = 0.36 \pm 0.03$ and a c_v of the order of magnitude of 1×10^{-8} m²/s in the normally consolidated range.

The discussers have another set of comments to make on the so-called "Barra da Tijuca Clay", whose parameters and properties are also shown in Table 1. Depending on the place and depth, "Barra da Tijuca Clay" may show many different characteristics. In the region that includes SENAC headquarters, Barra da Tijuca Roundabout and the Panamerican Games Athletes' Village, for example, there is a superficial 3-m thick peat layer whose water content can be higher than 600%. This soil is easily recognized by its dark brown colour and peculiar smell. It is also identified by its specific gravity $G < 2.0$, liquid limit $w_l > 450\%$ and plasticity index (*PI*) of the order of 250% (Da Mota, 1996 and Garcia,1996). In the vicinity of Barra da Tijuca Roundabout, this superficial peat layer looks like a dark pudding and exhibits a natural water content of 630%, a unit weight of 10 kN/m³ and a void ratio $e_o = 12$. One-dimensional consolidation tests carried out by the discussers on this material have revealed overconsolidation stresses σ'_{vm} of the order of 5 kPa and a $C_c/(1 + e_o) = 0.45 \pm 0.03$. An unusual feature of this soil that must be highlighted is its extremely low coefficient of consolidation (c_v), which may reach a value of 4×10^{-10} m²/s, *e.g.* twenty-five times smaller than the typical value of c_v in the virgin compression range, found for clays of the Brazilian coast.

Underneath the superficial peat layer described above, there is a 10 m-thick organic soft clay layer of dark grey colour with natural water content $w = 215\% \pm 85\%$, liquid limit (without previous drying) $w_l = 240\% \pm 80\%$, plasticity index $PI = 165\% \pm 75\%$, specific gravity $G = 2.42 \pm 0.18$ and natural void ratio $e_o = 5.6 \pm 0.6$. One-dimensional consolidation tests carried out by the discussers on this clay have revealed overconsolidation stresses ranging from 7 kPa to 32 kPa and $C_c/(1 + e_o)$ values determined at the beginning of the virgin compression branch of the $e \times \log(\sigma'_v)$ plot, between 0.36 and 0.67. The coefficient of consolidation in the normally consolidated range is of the order of 1×10^{-8} m²/s.

Finally, it would be convenient to hear from the authors about the criterium used to determine *CR* and C_c values presented in Table 1 since the values of both parameters vary with vertical effective stress (σ'_v).

3. Compressibility and Stress History

The features of the *OCR* x depth plot shown in Fig. 5 are indeed very interesting. According to the authors, there are three of them that stand out, namely:

a) The *OCR* profile for all clays is within a narrow zone.

b) The narrow zone of *OCR* values with depth suggests that soft clay deposits of Rio de Janeiro State coastal zone have similar stress histories.

c) Some deposits show a desiccated crust that may reach a thickness as great as 4 m and where *OCR* values are higher.

Table 3 - Geotechnical data of Uruguaiana Clay (taken into account only the samples between the depths of 13.0 m and 18.45 m).

Clay layer thickness (m)	Water content w (%)	Liquid limit w_l (%)	Plasticity index PI (%)	Clay content (% < 2 μ)	Unit weight γ (kN/m³)
7	61 ± 7	83 ± 17	51 ± 9	48 ± 8	15.6 ± 0.3

The discussers are of the opinion that three other observations should be added to the above list, as follows:

i) When clay deposits reach surface (cases where the desiccated crusts show up), the *OCR* trend is to be constant with depth as soon as the crust zone is left behind.

ii) *OCR* values' trend to be constant with depth (see Fig. 5) suggests that clay deposits of Rio de Janeiro State coast have not been affected by the 2 m to 3 m lowering of sea level that occurred 4000 years ago, as estimated by Massad (1994).

iii) Except for "Itaipu Clay", there is a trend for *OCR* values to reach a constant value in the neighbourhood of 1.7, provided the depth considered is out of the desiccated crust zone. This feature suggests that the main cause (perhaps the unique) of overconsolidation of Rio de Janeiro coastal clays was secondary consolidation (aging).

As far as Fig. 6 is concerned, it would be convenient to review C_c and C_r values because they are not correct, at least those given for "Sarapuí Clay".

Finally, the discussers would like to present a more detailed discussion on the relationship between natural water content w and compression index C_c shown in Fig. 7.

It is well known that e x σ'_v (log) plots obtained from one-dimensional consolidation tests carried out on good quality samples are rather curvilinear in the virgin compression range. This means that compression index C_c is not a constant. For this reason, the use of C_c as a characteristic parameter for compressibility may not be suitable (see Fig. 8).

Butterfield (1979) and Martins (1983) have shown that when data from one-dimensional compression tests are presented in a v(log) x σ'_v (log) plot (v being the specific volume $v = 1 + e$), the virgin compression branch of the compression curve appears as a straight line in such a kind of plot. Being so, the compressibility parameter in the virgin compression range, denoted by $\not{C}_c$, is a constant given by:

$$\not{C}_c = \frac{-d \log v}{d \log \sigma'_v} = \text{constant} \tag{2}$$

It is easy to show that (see, for example, Martins *et al.*, 2006).

$$\not{C}_c = 0.434 \frac{C_c}{1+e} \tag{3}$$

Assuming that C_c values shown in Fig. 7 have been obtained at the beginning of the virgin compression branch of the e x σ'_v (log) plots (as C_{cM} shown in Fig. 8) and keeping in mind that all clays presented in the paper have a saturation degree of 100%, one can replace the void ratio e in Eq. (3) by the product Gw between specific gravity G and water content w. Then,

$$\not{C}_c = 0.434 \frac{C_c}{1+Gw} \tag{4}$$

or

$$C_c = \frac{\not{C}_c}{0.434}(1+Gw) \tag{5}$$

The pairs (C_c, w) which satisfy Eq. (5) are, according to Fig. 8, those pertaining to the virgin domain of the one-dimensional compression curve (as, for example, C_{cM} and w_M). Nevertheless, the plot of Fig. 7 does not give the relationship between C_c and w within the virgin compression range but between C_{cM} and the sample water content. Due to sampling operations, the sample water content w is slightly higher than the water content in the field w_o. If the sample water content w is assumed to be equal to the field water content w_o (w_o associated to the vertical effective stress in the field, σ'_{v0}) and if the water content associated to the overconsolidation stress is denoted by w_Y, then w can be evaluated by (see also Fig. 8):

$$\begin{aligned} & G(w - w_Y) \cong G(w_0 - w_Y) = e_0 - e_Y = \\ & C_r \log \frac{\sigma'_{vm}}{\sigma'_{v0}} = C_r \log OCR \end{aligned} \tag{6}$$

The value of the compression index associated to the water content w_Y, which satisfies Eq. (5), is:

$$C_{c_Y} = \frac{\not{C}_c}{0.434}(1+Gw_Y) \tag{7}$$

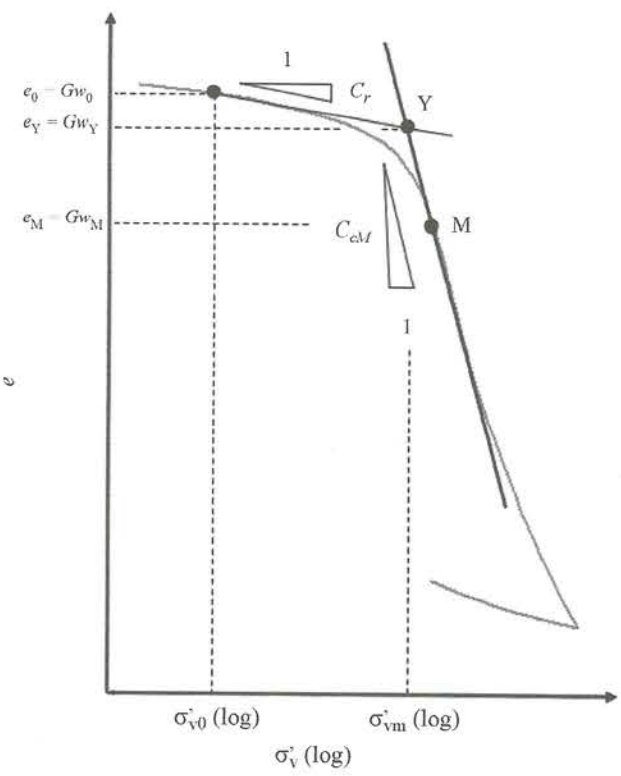

Figure 8 - Typical e x σ'_v (log) plot from an oedometer test carried out on a good quality sample of an organic clay from Rio de Janeiro State coastal zone.

Hence, evaluating w_Y from expression (6) and introducing it in Eq. (7), one obtains:

$$C_{c_Y} \cong \frac{\not\subset_c}{0.434}(1+Gw-C_r \log OCR) \tag{8}$$

Since $C_{c_Y} \cong C_{c_M} = C_c$, the relationship between C_c and w can be expressed approximately by:

$$C_c \cong \frac{\not\subset_c}{0.434}(1+Gw-C_r \log OCR) \tag{9}$$

Taking into account only Juturnaíba, Sarapuí and Uruguaiana clays, for which $\not\subset_c$ values have been determined by the discussers, it can be assumed the following average values for the terms in Eq. (9): $\not\subset_c = 0.21$, $G \cong 2.57$, $C_r \cong C_c / 8$ and $OCR \cong 1.7$. With these average values kept in mind, assuming also that $w_0 \cong w$ and that C_c values presented by the authors have been determined as C_{cM} in Fig. 8, Eq. (9) gives:

$$C_c \cong 0.48 + 1.23w \tag{10}$$

which is a similar expression to that presented by the authors.

It should be emphasized that for the studied clays, $0.180 \le \not\subset_c \le 0.240$. Thus, expression (10) represents an average line through a set of points with a considerable amount of scattering, as shown in Fig. 7. For this reason, both Eq. (10) and the expression presented by the authors should be used with caution.

4. Summary and Conclusions

The discussers would like to emphasize the important contribution given by the authors regarding the pattern of *OCR* profiles. As pointed out by the discussers, the shape shown by the *OCR* profiles suggests that clay deposits from Rio de Janeiro State coast were not affected by sea level lowering that occurred 4000 years ago, as reported by Massad (1994). Besides, the *OCR* trend to be constant with depth towards a value of approximately 1.7 seems to indicate that the overconsolidation observed in Rio de Janeiro State coastal clays are due to secondary consolidation (aging). In the discussers' opinion, this is a subject that deserves a deeper study, which may throw new lights on the understanding of the processes under which the clay deposits of Rio de Janeiro coast were formed.

The empirical expression $C_c = 1.3w$, found by the authors to the set of clays studied, is of great practical interest. The referred expression is similar to the expression $C_c \cong 0.48 + 1.23w$, deduced herein by the discussers. Although very attractive from the practical point of view, both expressions should be used with caution since there is a considerable amount of scattering in the relationship between w and C_c that both expressions intend to represent.

As for the rest, the remaining conclusions stated by the authors should be viewed with caution in face of the questions raised by the discussers herein.

Finally, the discussers would like to clearly state that this discussion was written keeping in mind the purpose of exchanging ideas viewing at a deeper understanding of soil formation for a better practice of geotechnical engineering.

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Authors' reply to the discussions presented by Bicalho, K. & Castello, R.R, and by Martins, Santa Maria & Santa Maria.

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Reply to Bicalho & Castello

With respect to the first point raised by these discussers regarding Region IV, there is indeed a mistyping in Table 2 and the last line should read $I_p > 50\%$ (instead of $I_p > 130\%$). However, contrary to the discussers' statement, the clay of Region IV (basically Itaipu clay) is different from the clays of Regions II and III as all layers assessed have higher liquid limit and also higher organic matter. As a consequence of the higher organic content (presence of fibers) Itaipu clay has shown higher friction angle than the clays from Regions II and III.

The other point raised by Bicalho and Castello regards the correlation between the water content w with the compression index C_c. The reason to use this correlation rather than the correlation between C_c and liquid limit w_L is because it has become common practice recently (*e.g.*, Sandroni, 2006; Almeida *et al.*, 2008) to measure water content using SPT samples. This low-cost and simple procedure also provides a first assessment of the variation of soil characteristics in extensive areas and can be used for preliminary geotechnical design. Therefore, the authors have not in fact assessed the correlation between C_c and liquid limit w_L, simply because the determination of the liquid limit is more time consuming than the determination of the water content. Thus, there is no such conclusion that the w x C_c correlations works better than any C_c x w_L correlation, it is just because the former is simpler and straightforward. More recent work including only data on Barra da Tijuca and Recreio dos Bandeirantes clays (Almeida *et al.*, 2008) has changed slightly the correlation to $C_c = 0.012w$ (w_o in %), as shown in Fig. 9.

Reply to Martins *et al.*

Soil characterization

These writers have presented a number of interesting comments and suggestions, most of which have now been included in the updated version of Table 1 shown below. In fact, Uruguaiana Street clay has a clay content varying with depth (Villela, 1976). It varies from 5% at a depth of 10.5 m reaching a maximum of 54% at a depth of 16 m. The paper shows results of the entire layer, while the writers focused on the material below 13 m, with higher clay content. Therefore, Table 3 provided by the writers is a detail of Table 1 of the paper. Fig. 1 shows a soft clay layer above the mentioned layer, which was clearly a mistake. The authors thank the writers for this important correction.

As far as Barra da Tijuca clay is concerned, the paper is based on the data compiled by Futai (1999) until the late nineties. Later on, a large number of dissertations and theses have dealt with Barra da Tijuca clay (*e.g.*, Macedo, 2004; Nascimento, 2009) and also Recreio dos Bandei-

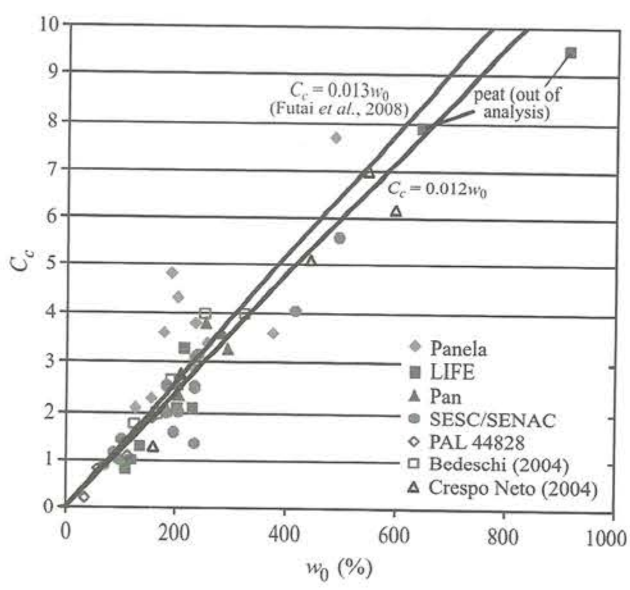

Figure 9 - Compression index C_c *vs.* in situ water content w_o for Barra and Recreio clays.

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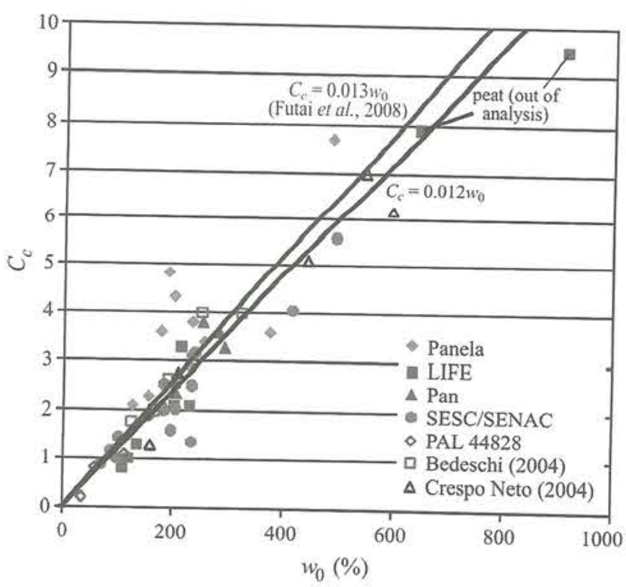

Figure 9 - Compression index C_c *vs.* in situ water content w_0 for Barra and Recreio clays.

rantes (Crespo Neto, 2004). Most of the data mentioned by the discussers regarding water content, liquid limit, plasticity index and compressibility are within the range shown in Table 1. More recently, Almeida *et al.* (2008) presented data with very high water content as shown in Fig. 9. There is indeed a clear difference of behavior depending on the soil depth. However, the purpose of the paper is to present typical and overall data without emphasis on the differences in behavior between different depths.

Compressibility and stress history

Regarding the values of C_c and C_r shown, there are indeed some mistakes and Fig. 10 presents the correct data.

With respect to the *OCR vs.* depth plot, the authors agree with the added contributions and also that aging is the main cause of overconsolidation of Rio de Janeiro clays.

Regarding C_c values, the authors used the same criterion used by the writers, *i.e.*, C_c was determined at the beginning of the virgin compression line.

With respect to the correlation between compression index C_c and water content w, the writers have developed an interesting and new relationship based on the log (e) *vs.* log (σ'_v) plot (previously used by Almeida, 1981 and 1982, for Sarapuí clay). The equation $C_c = 0.43 + 0.0123w$ (w in%) developed by the writers has sound theoretical basis. However, this equation differs from the correlationships developed by a number of authors (Almeida *et al.*, 2008; Bowles, 1979; Kopula, 1981; Nagaraj & Miura, 2001) which are of the type $C_c = K.w$ (*i.e.*, without intercept) with K values in quite a narrow range 0.010-0.013. The equation developed by the writers is plotted in Fig. 11 together with the authors data. The theoretical relationship proposed by the writers is very interesting, but it is not a good fit for the data, as show in Fig. 11.

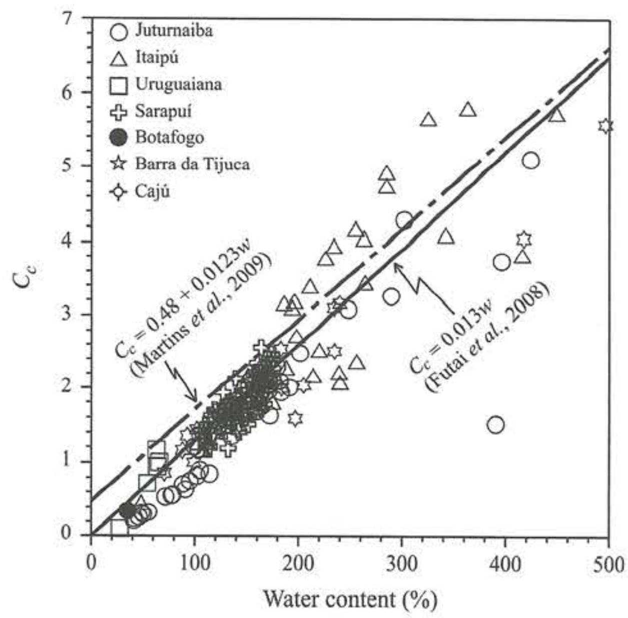

Figure 11 - Compression index C_c *vs.* in situ water content w_0 for Rio de Janeiro clays and the correlation proposed by Martins *et al.* (2009).

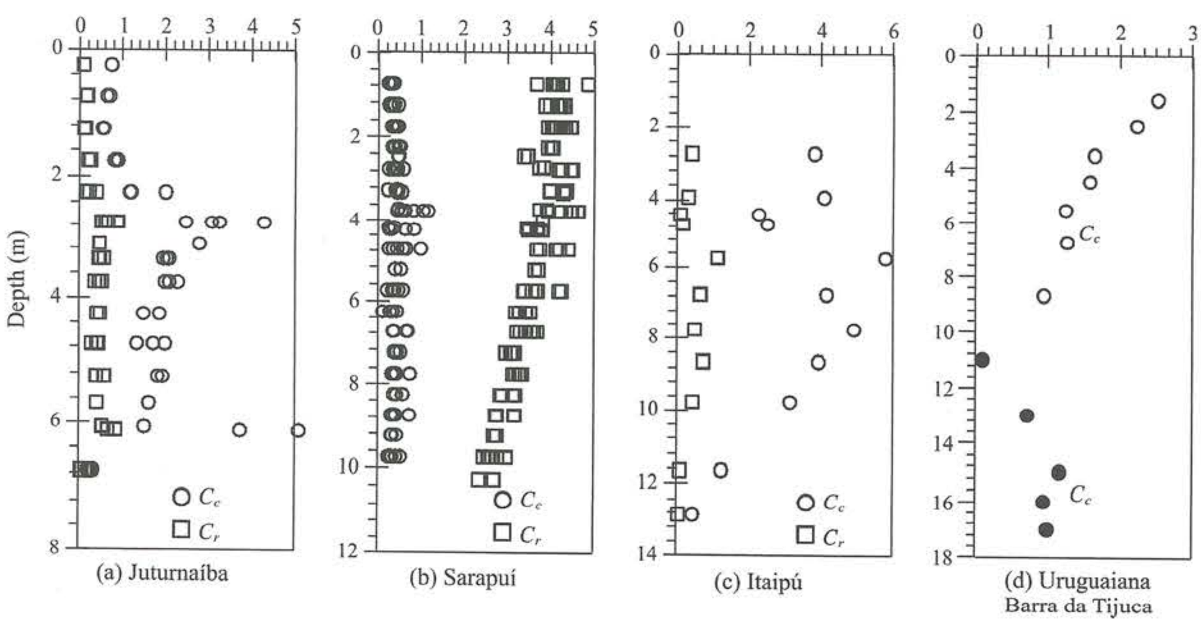

Figure 10 - Compressibility parameter profiles.

Table 1 (altered) - Geotechnical properties of Rio de Janeiro clays.

Parameter / clay	Caju (b)	Sarapuí (c)	Santa Cruz (IZ) (d)	Santa Cruz (SZ) (e)	Nothern coast of Guanabara (f)	Itaipú (g)	Juturnaíba (h)	Uruguaiana Street clay (i)	Botafogo (near the Subway station) (j)	Barra da Tijuca (k)
References	Lira, 1988; Cunha & Lacerda, 1991	Lacerda *et al.*, 1977; Ortigão, 1980; Almeida & Marques, 2002)	Aragão, 1975	Aragão, 1975	Aragão, 1975	Carvalho, 1980; Sandroni *et al.*, (1984)	Coutinho & Lacerda, 1987	Vilela, 1976	Lins & Lacerda, 1980	Almeida *et al.*, 2000
Clay layer thickness (m)	12	12	15	10	8.5	10	7	9	6	12
w (%)	88	143 ± 21.7	112	130	113	240 ± 110	154 ± 95.6	54.8 ± 15.9	35	100-500
w_L (%)	107.5	120.3 ± 18.0	59.6	125.4	122	175.4 ± 82.6	132.5 ± 43.8	71.3 ± 30.0	38	70-450
I_p (%)	67.5	73.08 ± 16.1	32	89	81	74.5 ± 30.1	63.59 ± 22.1	40.5 ± 22.03	11	120-250
% clay	60	70	-	54	35	-	60.7 ± 12.74	39.4 ± 10.11	28	28-80
γ (kN/m^3)	14.81	13.1 ± 0.49	13.24	13.44	13.24	12 ± 1.85	12.5 ± 1.87	16.1 ± 1.39	17.04	12.5
S_t	3	2.59 ± 0.69	3.39	2-6	-	4-6	5-10	3.00	-	5.0
% organic matter	5	4.13 - 5.54	-	-	-	32.63 ± 20.46	19 ± 10.63	2.56 ± 1.04	-	-
$CR = C_c / (1 + e_o)$	0.27	0.41 ± 0.07	0.32	-	0.26 ± 0.15	0.41 ± 0.12	0.31 ± 0.12	0.31 ± 0.15	0.16	0.52
C/C_c	0.21	0.15 ± 0.02	0.10	-	0.16 ± 0.04	-	0.07 ± 0.06	-	0.19	0.10
c_v (m^2/s) x 10^{-8}	1	1-40	0.2-18.2	-	0.4	5	1-10		30	2-80
e_o	2.38	3.71 ± 0.57	3.09	3.37	2.91	6.72 ± 3.1	3.74 ± 1.89	1.42 ± 0.36	1.1	-

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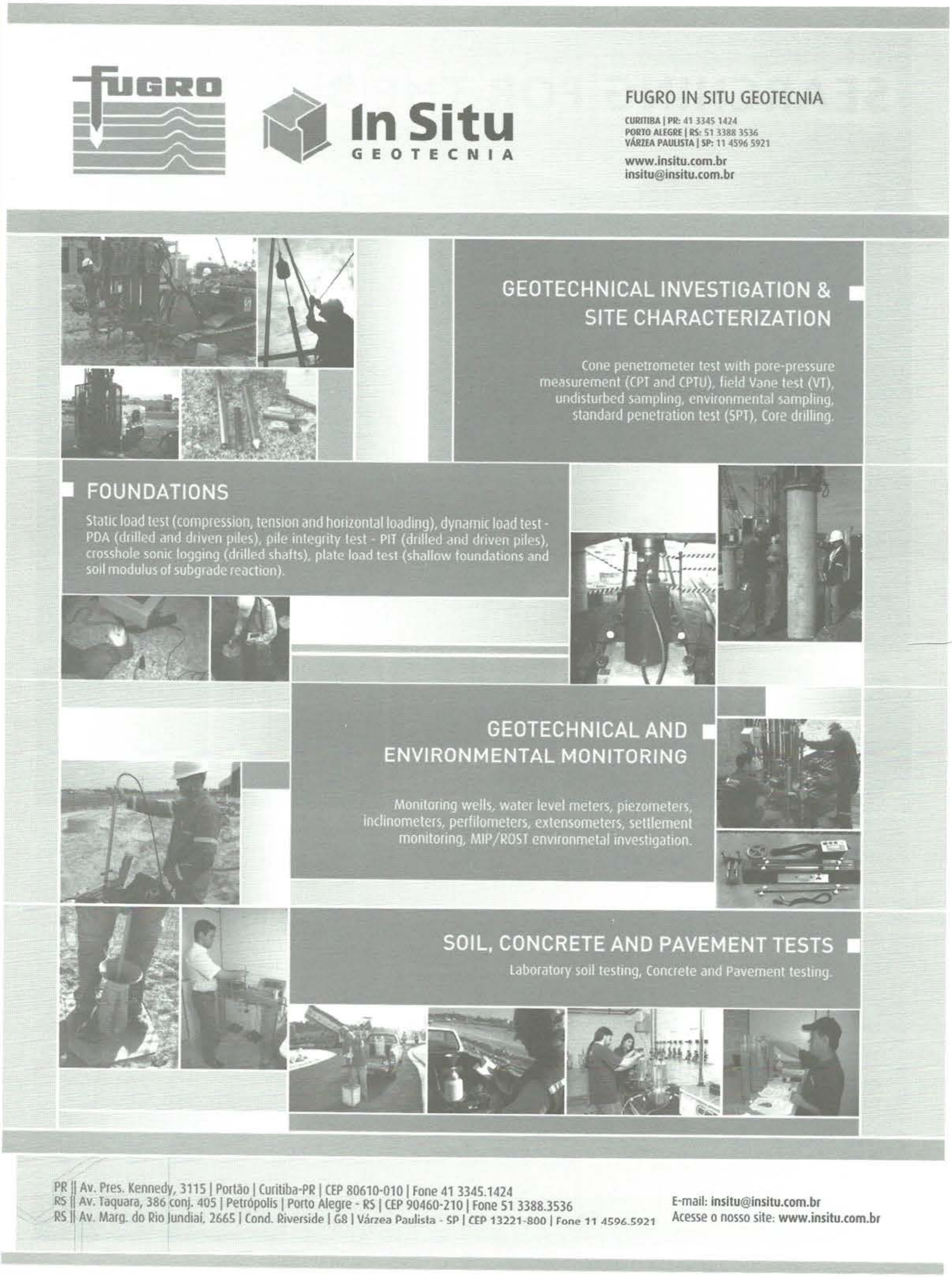
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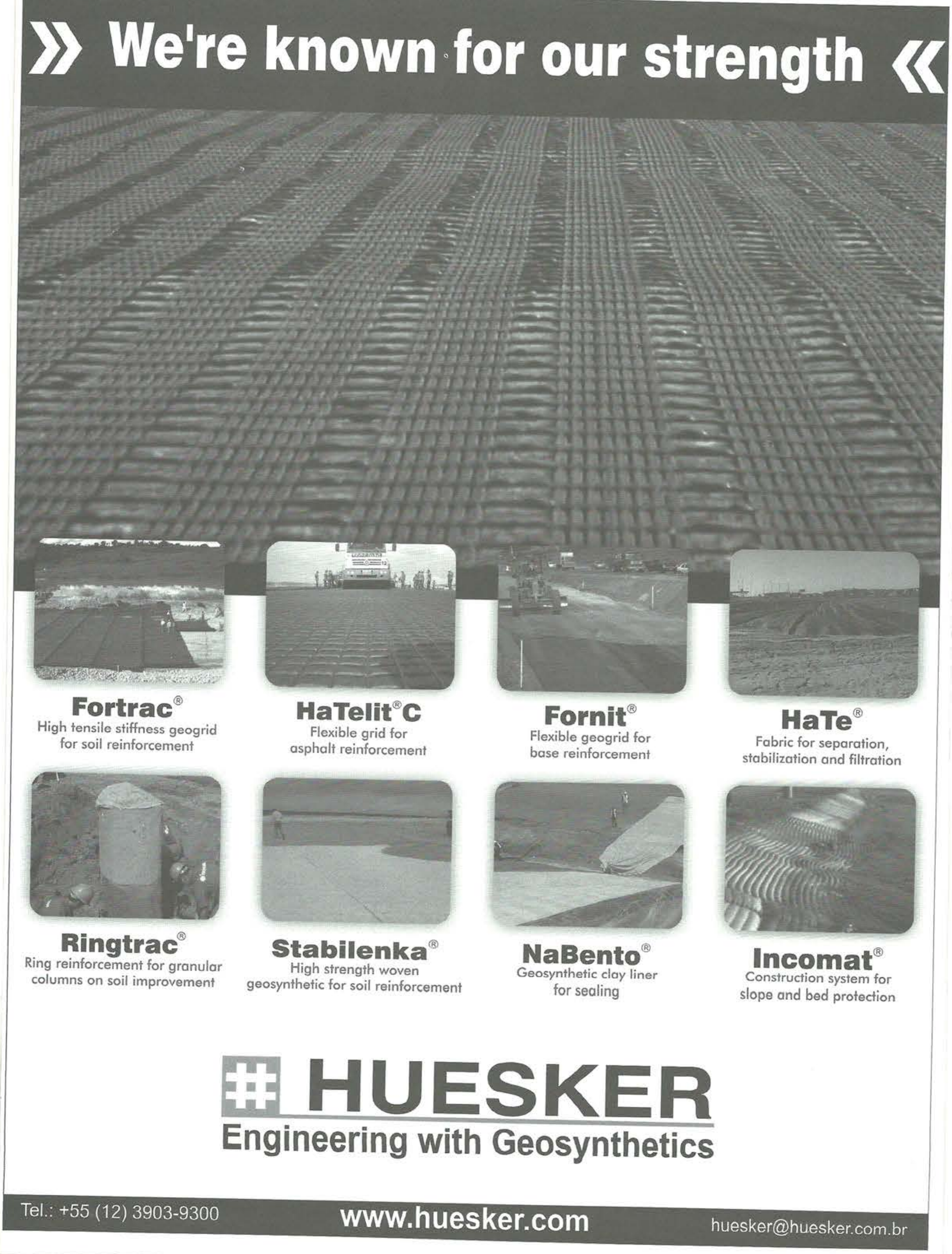
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The accepted papers are classified either as an Article paper, a Technical Note, a Case Study, or a Discussion according to its content. An article paper is an extensive and conclusive dissertation about a geotechnical topic. A paper is considered as a technical note if it gives a short description of ongoing studies, comprising partial results and/or particular aspects of the investigation. A case study is a report of unusual problems found during the design, construction or the performance of geotechnical projects. A case study is also considered as the report of an unusual solution given to an ordinary problem. The discussions about published papers, case studies and technical notes are made in the Discussions Section.

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Although alteration of the sequence and the title of each section may be required, it is suggested that the text contains the following sections: Introduction, Material and Methods, Results, Discussions, Conclusion, Acknowledgements, References and List of Symbols. A brief description of each section is given next.

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Tables shall be numbered consecutively in Arabic and have a caption consisting of the table number and a brief title. This number should be used when referring to the table in text. Units should be indicated in the first line of the table, below the title of each column. Abbreviations should be avoided. Column headings should not be abbreviated. When applicable, the units should come right below the corresponding column heading. Any necessary explanation can be placed as footnotes.

Equations shall appear isolated in a single line of the text. Numbers identifying equations must be flush with the right margin. International System (SI) units are to be used. The symbols used in the equations shall be listed in the List of Symbols. It is recommended that the used symbols

3

be in accordance with Lexicon in 8 Languages, ISSMFE (1981) and the ISRM List of Symbols.

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