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Soils and Rocks

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XXIII Manuel Rocha Lecture

This lecture has been traditionally promoted by the Portuguese Geotechnical Society (SPG) to honour the memory of Prof. Manuel Rocha. The XXVI Manuel Rocha Lecture was delivered on October 19th 2009, in Lisbon, by Prof. Manuel de Matos Fernandes, from University of Porto, Portugal. The introduction speech was presented by Prof. António Correia Mineiro, who summarized the CV of the Lecturer and emphasised his contributions in geotechnical engineering. At the end, Dr. Rui Correia highlighted the main topics and was warmly joined by all presents in thanking the speaker, who was given a SPG commemorative medal.

The 2009 Manuel Rocha Lecturer is Prof. **Manuel de Matos Fernandes**, 57, Full Professor at the Faculty of Engineering of University of Porto (FEUP), Portugal. He got his degree in Civil Engineering in 1976 at FEUP. The PhD thesis (FEUP, 1984), on the behaviour of flexible earth-retaining structures, was developed at LNEC, Lisbon. He was the main responsible for the development at FEUP of a team devoted to Geotechnics. His areas of interest have been focused on deep excavations, Eurocodes and teaching Soil Mechanics (he is author of a text book). At present, he is the Director of the Civil Engineering Department and the Coordinator of CEC – Research Centre on the domains related to Civil Engineering Construction, with over 40 PhD researchers. In the last 20 years he has had activity of design and consultancy in geotechnical works, particularly bridge foundations and urban excavations.

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Deep Urban Excavations in Portugal: Practice, Design, Research and Perspectives

Manuel de Matos Fernandes

Abstract. The paper corresponds to the Manuel Rocha Lecture delivered by the author in October 2009. It is divided into three main parts. Part 1, Practice and Design, summarizes the most relevant excavations performed in Portugal since the 1950s, with emphasis on selected case histories. Part 2, Design and Research, discusses research work and original approaches and solutions developed for three distinct geotechnical conditions found in Portugal. Part 3, Research and Perspectives, presents some ideas on points considered critical for future developments in the field.

Keywords: flexible retaining structures, soft soils, residual soils, vertical stability, induced movements, ground improvement, ground water, finite element analyses.

Introduction

In the last decades, particularly from the middle of the 20th century on, large and deep excavations for basements and for transport infrastructure became, probably, the most emblematic geotechnical works in urban areas. The evolution of the construction techniques, of the methods of analysis and design, and of the works accomplished – executed under increasingly daring and demanding conditions – has been so intense and continuous that a reflexion on this *itinerary* seems to be justifiable. In fact, the understanding of the chain of relevant technical advances is crucial in the search for new developments.

Why the focus on the works performed in Portugal?

Firstly, because this type of work is strongly dependent on the geotechnical conditions and in Portugal a wide variety of scenarios is encountered: soft and thick alluvial clays; very distinct sandy and clayey secondary and tertiary sedimentary soils; diverse volcanic soils; granite rocks covered by thick and heterogeneous residual soils!

In addition, a *Portuguese know-how* - in the broad and noble sense of the term – has been developed and consolidated throughout the *itinerary* referred above. Drawing on knowledge and experience from projects across the world, solutions appropriate for local conditions have been conceived, lessons from the successful and the unsuccessful works have been learned, significant innovations have been produced and geotechnical works of global significance completed.

Last but not the least, this *itinerary* resulted – to a degree achieved by very few others – from a fruitful collaboration involving Contractors, Designers, Universities and the National Laboratory for Civil Engineering.

This paper is divided into three main parts. Part 1, Practice and Design, summarizes the most relevant excavations performed in Portugal, with emphasis on some special case histories. Part 2, Design and Research, discusses three distinct geotechnical conditions, which have been the subject of research works and required original solutions. Part 3, Research and Perspectives, presents some ideas on issues considered critical for future developments in this field.

1. Part 1 - Practice and Design

The reader should be aware that this paper corresponds to a lecture presented by the author. Part 1 of the paper is quite distinct from the same part of the lecture, which contained a large set of photos of the excavations considered as references for the *itinerary*. It is understandable then that it is not possible to replicate this in a paper. In order to overcome this difficulty, a summary of the main works and technical advances has been prepared (Table 1) and is complemented by a brief description of a selection of special case histories.

1.1. An overview on the excavations performed in the last six decades

1.1.1. General

As shown in Table 1, the *itinerary* begins in the 1950s, when the first phase of the Metropolitano de Lisboa was constructed (1955-59). Some photos of these works are presented in Fig. 1. The satisfactory performance achieved by such a primitive construction process and such light support structures may be explained by the fact that the construction was planned for zones of the city with favourable geotechnical conditions (essentially stiff Miocene and Oligocene sedimentary soils, and volcanic soft rocks) and mainly following wide avenues, thereby avoiding as much as possible buildings or other sensitive structures.

In the 1960s, the introduction of reinforced concrete diaphragm walls and of pre-stressed soil anchors led to a radical change in construction methods. In subsequent decades, other advanced technologies for the execution of these walls and of large diameter bored piles became avail-

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Table 1 - Summary of large urban deep excavations of the past six decades, in Portugal.

Decade	Retaining structure (first applications)	Type of work	References	Fig.
1950	Timber structures for internal bracing or sloped unsupported excavations	1st phase of Lisbon Metro	Rolo (1999)	1
1960	Concrete diaphragm walls. Ground anchors	Building basements	Paula (1979); Samuel (1979); Pinelo (1980)	-
1970	Concrete-soldier-pile walls, so-called permanent Berlin-type walls (anchored)	Building basements	Dias (1995); Guerra (1999); Guerra *et al.* (2004)	14
1980	• Excavation instrumentation	Basements	Matos Fernandes (1983)	-
	• F.E. prediction before construction (14 m deep excavation supported by a multi-strutted diaphragm wall in soft ground)	Basements	Matos Fernandes (1985)	-
	• Nailed excavation (11 m deep excavation in granite residual soils)	Basements	Cardoso (1987); Cardoso & Carreto (1990)	-
	• Pre-stressed arch linked to 5 "flying struts" supporting a diaphragm wall – Case history 1	Basements	Matos Fernandes (1989)	2-4
1990	• Jet grout treatment under the base of excavation (14 m deep excavation in soft clay supported by a diaphragm wall)	Cais do Sodré Station, Lisbon Metro	Matos Fernandes & Almeida e Sousa (2003); Matos Fernandes *et al.* (2007)	28b
	• Large diameter concrete piled walls (horizontal distance between axes larger than the diameter)	Basements. Metro stations	-	5, 6, 22
2000	Large diameter secant piled (reinforced concrete and bentonite-cement) walls	Basements, metro lines and stations	-	11
	• Deep excavation in tertiary soils enveloping a historical building – Case history 2	Basements	Pinto *et al.* (2001)	5, 6
	• Elliptical shafts in granite residual soils – Case history 3	Stations of Porto Metro	Topa Gomes *et al.* (2008); Topa Gomes (2008)	7, 8
	• Deep excavation in soft ground supported by a secant piled wall, jet-grout treatment above the base of the excavation and highly pre-stressed struts, linked to a pre-existing tunnel at the two portals – Case history 4	Terreiro do Paço Station of Lisbon Metro	Brito & Matos Fernandes (2006); Candeias *et al.* (2007)	10-12
	• 40 m deep excavation in tertiary soils supported by an anchored pile wall (depth record in the country)	Public Library, Lisbon	-	-

able, along with a number of methods of soil improvement, such as jet grouting. All of these techniques have been applied in the construction of deep excavations. The influence of the revolution of the sixties is still manifest in many of the excavations performed at present however. In the following, four remarkable excavations in very distinct geotechnical conditions are described, with emphasis put on the conception of the retaining structure, and on the process and sequence of construction.

1.1.2. Case history 1 - "Flying struts"?!

Case history 1 concerns a very original structural solution applied in 1982, to support an excavation in Lisbon (Matos Fernandes, 1989; Matos Fernandes & Xavier, 2010).

Figure 2 shows a plan and a cross section of the excavation, which was approximately square in plan (39 m x 39 m), and 24 m to 29 m deep, for the construction of a seven level basement for a new building. The ground comprised a fill layer and Miocene sedimentary marine soils overlying basalt.

As shown in Fig. 2b, the retaining structure consisted of an anchored diaphragm wall supporting the upper part of the excavation (through the fill and the upper Miocene soils) to a depth of about 10 m, with soil nailing used below this depth where the soils were less fissured and stiffer.

After construction of the first anchor level on the face adjacent to an adjoining masonry building, the owner of this building obtained a court order prohibiting further installation of anchors or nails under his property. Several solutions to overcome this unexpected problem were then discussed. The one adopted was conceived by Edgar Cardoso, an expert in bridge design, and was developed by the

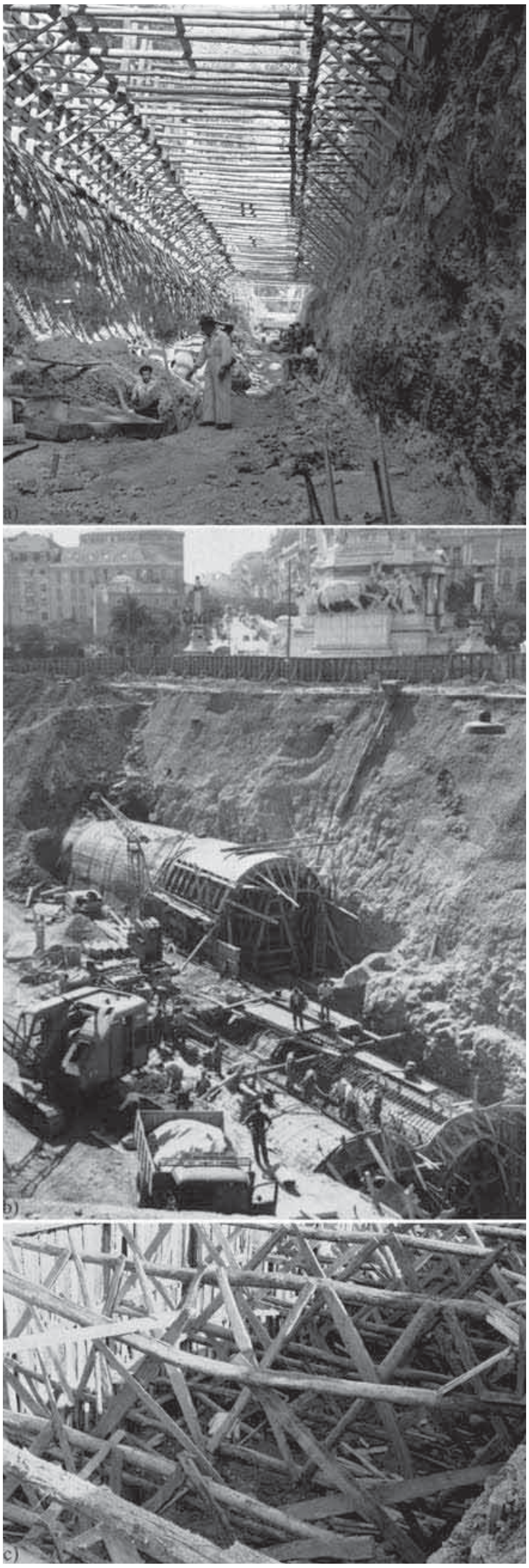


Figure 1 - Construction of the 1st phase of Lisbon Metro (1955-1959): a) Av. Liberdade (1955); b) Praça Marquês de Pombal (1956); c) Av. República (1956).

staff of the Contractor, Teixeira Duarte (Lousada Soares, 2003).

The solution, denoted in Fig. 2 as “pre-stressed system”, is represented in more detail by the schemes of Fig. 3 and by the general view of Fig. 4. It consists of a polygonal tendon of 14 high strength steel strands pre-stressed to 2100 kN, coupled to a system of five steel framed struts applying to the wall forces ranging from 300 kN to 350 kN. The tendon is anchored at the two corners of the cut by steel anchor plates inserted in concrete blocks linked to the diaphragm wall. For the global conception of the system, it was considered convenient that the struts support just axial loads. This required each strut to be placed on the bisector of the angle formed by the two adjacent spans of the tendon, which means that the axes of all the struts converge in a single point.

The construction sequence of the solution is described in the following.

1 - Partial demolition (from the interior of the cut) of the diaphragm wall at the two corners in order to insert the anchor plates connected to the wall reinforcement.

2 - Installation of the framed struts bolted to the diaphragm wall.

3 - Installation of the tendon composed of 7+7 strands in two circular arrangements close to the anchor plates and in a parallel layout over the rest of their length, through the following operations:

3.1 - Introduction and fixing of the 7+7 strands in 1+1 trumpets attached to the anchor plate at the left corner (Fig. 2a).

3.2 - Placement of the 7+7 strands in two parallel arrangements passing through saddles at the heads of the struts (see Fig. 3b).

3.3 - Introduction of the 7+7 strands in the 1+1 trumpets attached to the anchor plate at the right corner (Figs. 2a and 3a).

4 - From the current base of the excavation, and after cutting a door through the diaphragm wall adjacent to Av. Miguel Bombarda, execution of a gallery for accessing the back of the respective anchor plate (Fig. 3a).

5 - Application of a small pre-stress, for adjustment of the strands to the struts, by using jacks operated from the gallery.

6 - Installation of the steel reinforcement at the corners of the diaphragm wall and concreting of the anchor blocks; filling of the gallery.

7 - Application of the pre-stress in 19 stages, by operating the hydraulic jacks at the head of the struts; for a given stage, the pre-stress was applied symmetrically in relation to the central strut (pre-stressing both struts 1 and 5, or both struts 2 and 4 or just strut 3).

As shown in Fig. 2b, in the phases subsequent to the implementation of the pre-stressed system, the excavation first progressed in depth at the left side, allowing for the construction of the foundations and the basement structure, which was then used to support conventional struts for completion of the excavation.

During this phase the pre-stressed system operated well and no significant damage was induced in the vicinity. A maximum horizontal displacement of 3 mm was recorded at the diaphragm wall supported by the pre-stressed system after installation of this solution.

1.1.3. Case history 2 - Excavation around Sotto Mayor Palace, Lisbon

This case is related with the 25 m deep excavation around Sotto Mayor Palace, a monumental masonry building from the first decade of 20th century, located in the centre of Lisbon (Pinto *et al.*, 2001). The construction occurred in the period 1999-2001. As shown in Fig. 5, the excavation

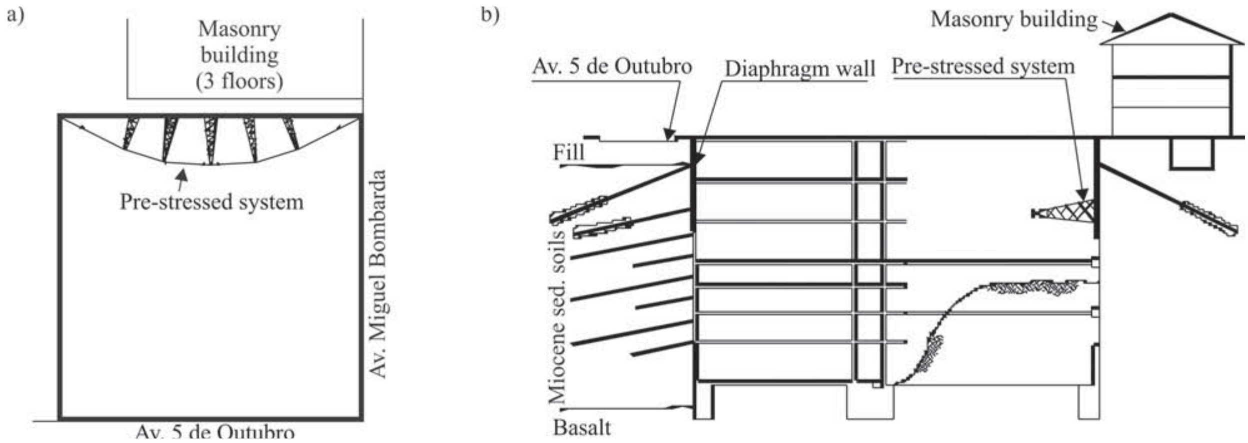


Figure 2 - a) Site plan; b) cross section perpendicular to Av. 5 de Outubro.

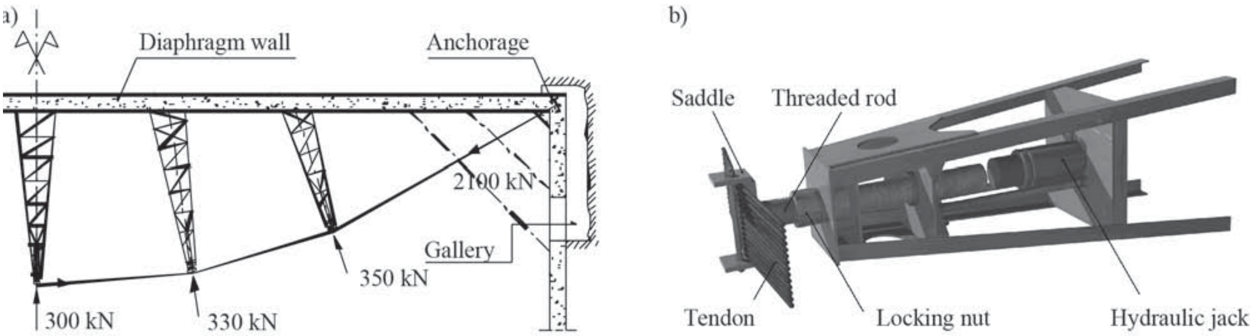


Figure 3 - Pre-stressed structural system: a) plan; b) 3D view of a strut head.

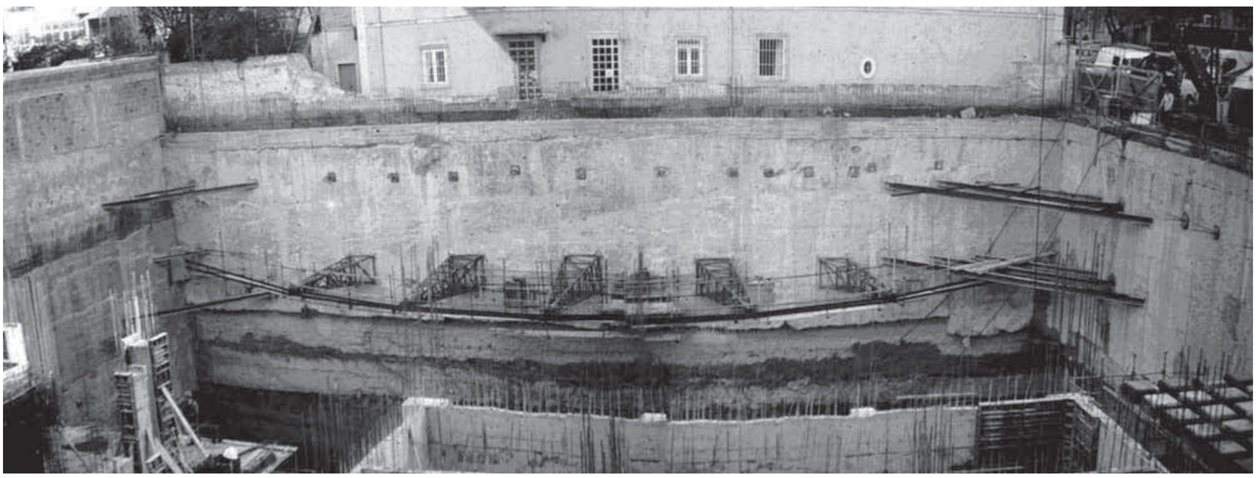


Figure 4 - General view of the solution.

Figure 5 - A view of Sotto Mayor Palace excavation in Lisbon (Pinto *et al.*, 2001).

was carried out in the area of the original gardens of the Palace with its inner limit adjacent to the peripheral walls of the building and the outer limit defined by the surrounding roads. The ground was composed of Miocene marine soils (clays, that became stiffer with depth, with layers of sandstone and limestone), extending down to the bottom of the excavation, overlying very stiff Oligocene sandstone. The water level was close to the bottom of the excavation.

Bearing in mind that the architectural project also included a central gallery under the Palace, the structure of the building was underpinned, prior to the external excavation, with micro-piles capped by a grillage of pre-stressed concrete beams.

The retaining structure of the excavation face adjacent to the building was conceived to work as a wooden wine barrel, as suggested by Fig. 6. The vertical elements consisted of reinforced concrete bored piles (diameter of 0.8 m, spaced at 1.0 m), connected at the top to the masonry walls through a concrete capping beam. The piles were embraced by six levels of horizontal pre-stressed concrete beams (3 m deep, 0.425 m thick). These beams were cast against the ground and their elevation was established in order to permit their incorporation in the permanent basement slabs. The pre-stress load was designed in order to balance

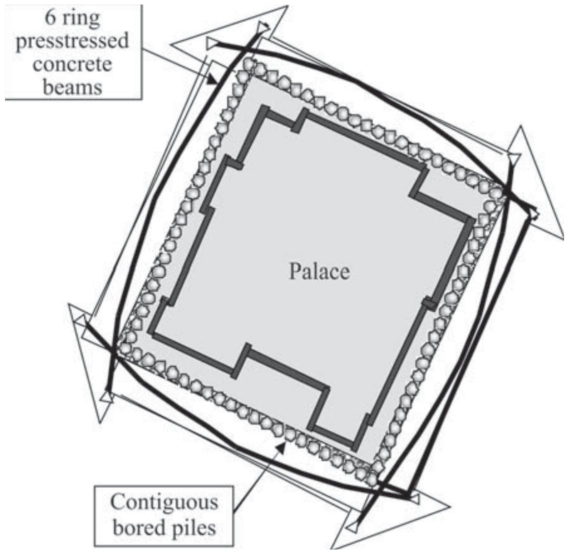


Figure 6 - Plan of the retaining structure of the excavation face adjacent to the Palace (Pinto *et al.*, 2001).

the internal earth pressures, leading to 2x27 high strength steel strands, corresponding to an equivalent normal load applied to the wall equal to 130 kN/m. The beams are supported close to their outer limit by a set of steel soldier piles inserted in the ground prior to the excavation.

The strands were not grouted in order to allow re-tensioning, if demanded by the analysis of the results of the comprehensive monitoring plan that was implemented. The maximum measured horizontal displacement at the Palace façades did not exceed 15 mm, which almost matched the predicted maximum finite element value.

1.1.4. Case history 3 - Salgueiros Station of Porto Metro

The Salgueiros Station of Porto Metro was constructed in 2003-2004 and required an excavation 22 m deep in granite residual soils (Topa Gomes *et al.*, 2008; Topa Gomes, 2008). The fact that the site was located in large open space permitted the implementation of a novel support solution, as shown by Figs. 7 and 8: the rectangular plan shape of the station was contained within two partially overlapping reinforced concrete elliptical rings and full advantage of this shape was achieved by mobilizing the arch horizontal effect on the ground.

The idea was accomplished by using the sequential excavation-concreting method. The first stage consists of the construction of the capping beam which is followed by cycles of excavation and construction of the supporting ring, until the bottom of the excavation. The excavation can proceed to the next ring only after the completion of the previous one (height equal to 1.8 m). The ring is formed by a shotcrete membrane (whose thickness ranges from 0.30 m, in the upper part, to 0.60 m at the base of the excavation) with two layers of wire mesh.

Figure 7 - Salgueiros Station of Porto Metro – a global view of the excavation at its final stage (Topa Gomes *et al.*, 2008).

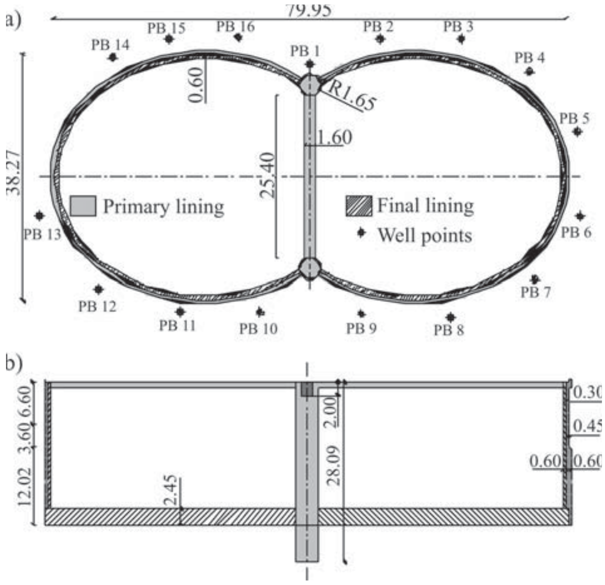


Figure 8 - Salgueiros Station of Porto Metro – plan and section on the longitudinal axis of the station (Topa Gomes, 2008).

In order to equilibrate the forces developed on the vertical plane of intersection of the two elliptical rings, a frame comprising two circular columns (diameter of 3.5 m) and a rectangular cross-lot beam (section equal to 1.60 m x 2.00 m) was cast in situ prior to the beginning of the excavation.

Prior to construction the water table was lowered to a level below the bottom of the excavation with 16 well points, distributed around the periphery of the excavation, 2 m away from the cut face. In addition to the obvious benefit with regard to the construction operations, this measure created a suction in the residual soil which improved its strength and stiffness (Topa Gomes, 2008).

Figure 9a shows the final deformed shape of the support (ring no. 4) at the depth of 9.0 m, and Fig. 9b shows the maximum recorded horizontal displacements, provided by inclinometer I4, installed 2.0 m behind the cut face. It can be seen that the magnitude of the maximum displacements is rather small, barely exceeding 0.15% of the excavation depth. The non-symmetric deformed shape of the support in a horizontal plane (Fig. 9a) is explained by the heterogeneity of the ground, which was more resistant and stiff on the east side of the Station. Anyway, that shape is very expressive with regard to the soil-structure interaction in the horizontal plane, which involves convergence over the minor axes of the ellipses and divergence in the vicinity of the major (longitudinal) axis.

1.1.5. Case history 4 - Terreiro do Paço Station of Metropolitano de Lisboa

The Terreiro do Paço subway station of the Lisbon Metro was built on a reclaimed area adjacent to the River Tagus in the vicinity of a number of historical public buildings, Fig. 10. The main phase of construction occurred between 2002 and 2004 (Brito & Matos Fernandes, 2006).

The 9.7 m diameter tunnel had already been constructed by a TBM, excavated in soft clayey alluvial soils, which extend to about 27 m depth at the site. These young alluvial soils are underlain by stiff Miocene clays. As shown in Fig. 11, the construction of the Station consisted of a cut-and-cover excavation 25.5 m deep linked to the existing tunnel at the two portals, spaced about 140 m apart. The water level is very close to the surface and varies with the tide.

The retaining wall was formed with 1.5 m diameter secant bored piles, embedded to a depth of 8.0 m in the underlying stiff clay. As excavation proceeded, a 0.8 m thick reinforced concrete lining wall, structurally connected to the piled wall, was installed. A piled wall was used instead of a conventional diaphragm wall because the expected presence (and confirmed during construction) of large obstacles buried in the thick fill layer.

After installation of the piled wall and before starting excavation, jet grouting was carried out in order to build a "slab" 3.0 m thick, between the longitudinal retaining walls and the tunnel (which had been previously filled with light concrete).

The wall was supported by five levels of highly pre-stressed large diameter steel tube struts with an average horizontal spacing of 3.5 m. The applied pre-stress (1050 kN for the first level and 3500 kN for levels 2 to 5) was equivalent to 93% of the resultant of the at-rest effective horizontal stresses plus the static water pressures, computed down to 25.5 m depth. As shown in Fig. 12, the struts were paired; this arrangement facilitated the removal of the soil by using suspended clamshells operated from the surface and permitted the implementation of a simple system to reduce the buckling length of the struts in the horizontal and vertical planes.

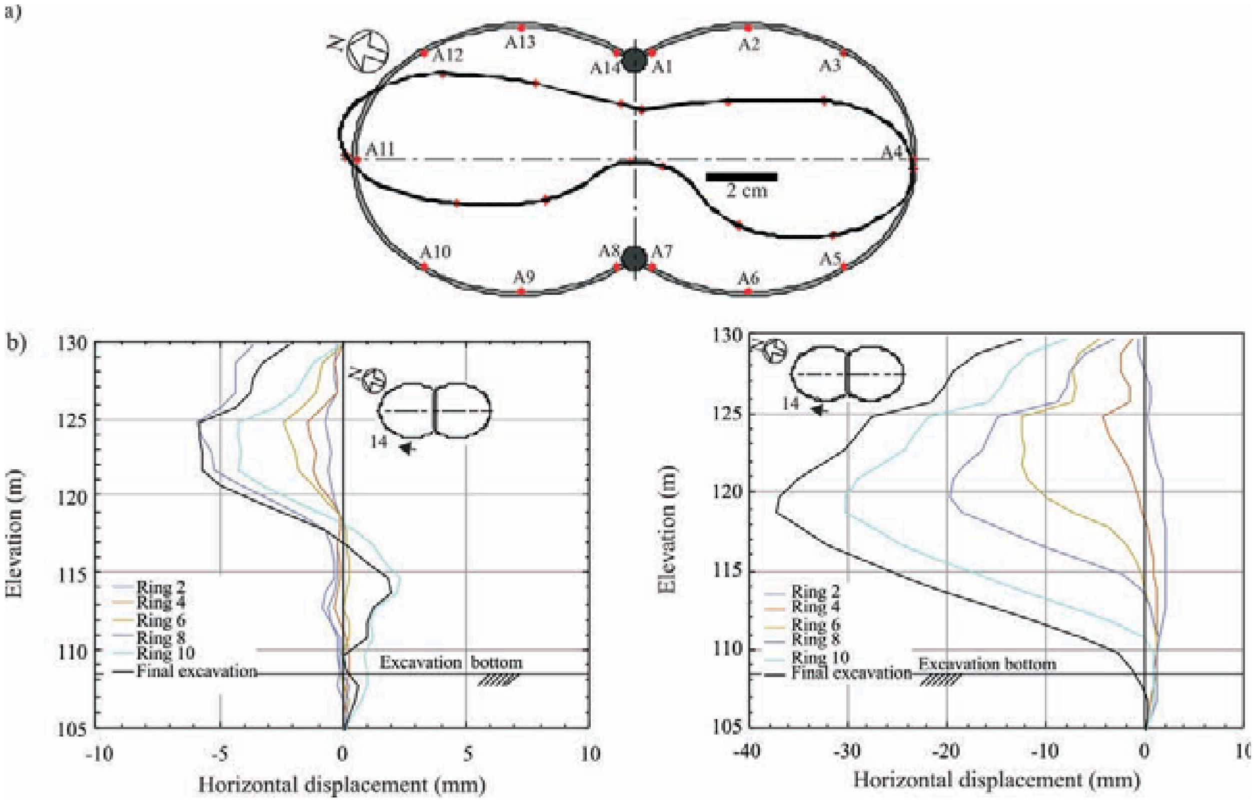


Figure 9 - Monitoring results of Salgueiros Station of Porto Metro (Topa Gomes, 2008): a) deformed shape at the level of ring 4 (9.0 m deep); b) horizontal displacements at a distance of 2.0 m from the cut face, on the direction parallel to the longitudinal axis (on the left) and on the direction normal to the longitudinal axis (on the right).

Figure 10 - Perspective view of the site of Terreiro do Paço Metro Station in Lisbon.

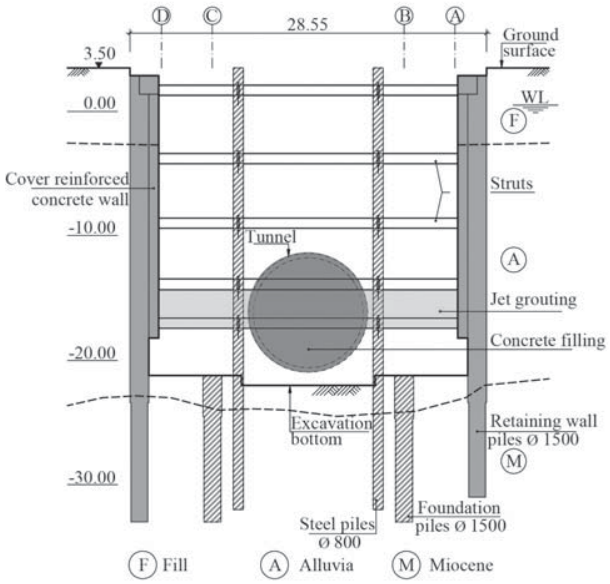


Figure 11 - Cross section of the excavation and of the retaining structure.

Each strut was pre-stressed with the help of a carefully designed and operated system shown in Fig. 12. In order to minimize the loss of pre-stress force, before removing the four jacks, the gap between the strut and the steel-pillow bolted to the reinforced concrete wall was measured and a combination of thin steel plates, fitting the gap size as closely as possible, was installed. There were 1, 2, 5, 10 and 20 mm thick plates, allowing combinations whose global thickness differs by just 1 mm. This original system was very effective in minimizing the load loss after jack removal.

The observed surface settlements in the vicinity of the excavation are summarized in Fig. 13. Figure 13a shows the settlements recorded during the preparatory works and the wall installation. Figure 13b includes the settlements recorded during the phased process of excavation and strut installation, in comparison with the predictions included in the Design Report which has been concluded before the beginning of the work. Note that the recorded values refer to an excavation depth of 22.5 m (and not to 25.5 m, the final excavation depth) because of the occurrence of two incidents which resulted in the intrusion of water and solid material through the joints of the piled wall just before the last excavation stage. These events led to some modifications to the planned construction sequence, namely the addition of soil treatment by grouting down the back of the wall in some zones. Figure 13c shows the total recorded settlements, which incorporate: i) those settlements induced by the preparatory works and retaining wall installation, seen in Fig. 13a; ii) the settlements recorded during the excavation to 22.5 m depth, Fig. 13b; and iii) those resulting from the incidents mentioned above, the construction phases until completion of the excavation and concreting of the base slab, as well as any delayed settlements that occurred during a further period of about 18 months after construction was completed.

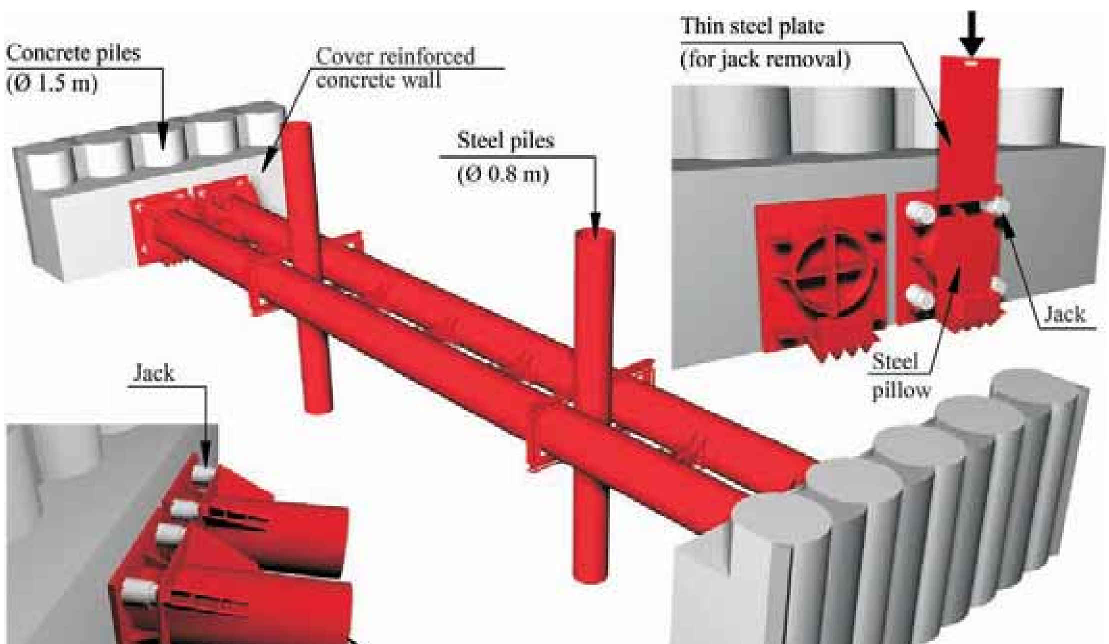


Figure 12 - Strut system: structural arrangement and details of the active strut head.

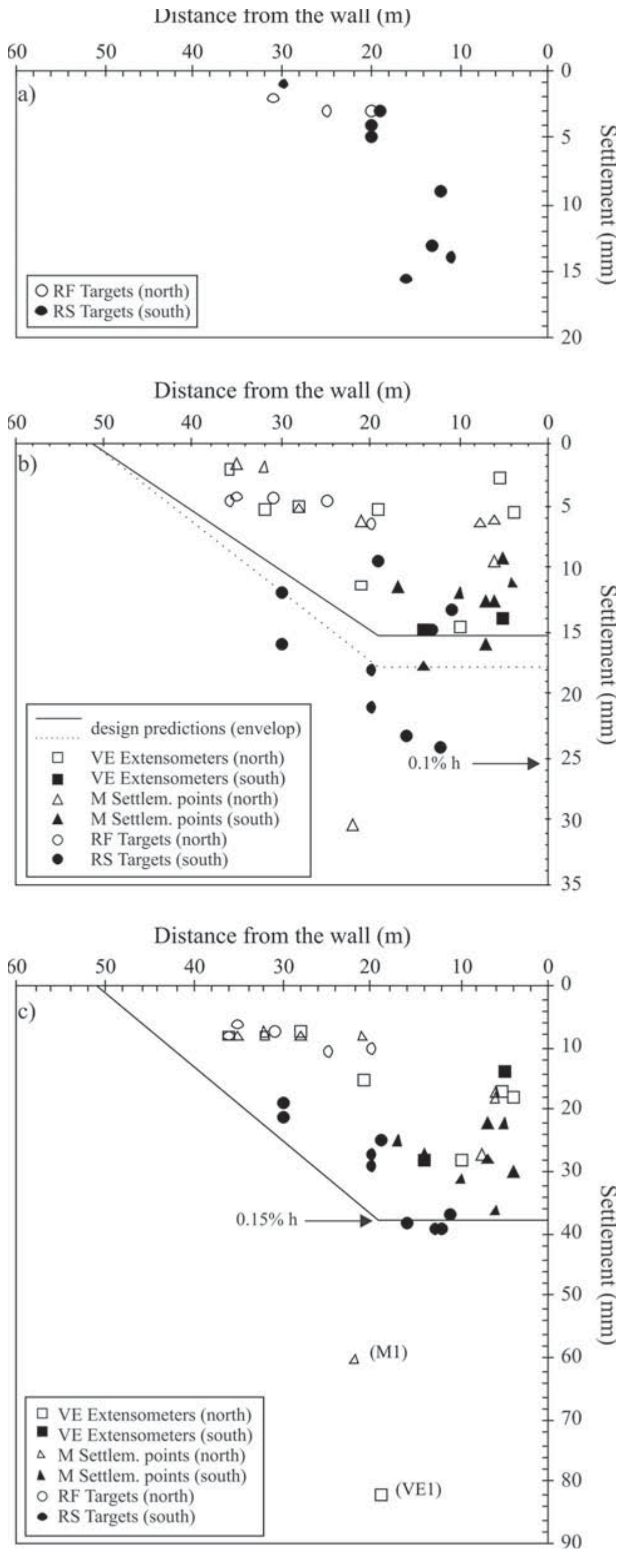


Figure 13 - Observed surface settlements: a) preparatory works and wall construction; b) excavation down to a depth of 22.5 m, with comparison to the settlements predicted in the design for the completion of the excavation (depth 25.5 m); c) total values at completion of the construction.

The results are quite interesting and suggest the following comments.

The magnitude of the settlements induced by the wall construction is in agreement with experience in similar works and conditions (Clough & O'Rourke, 1990; Poh *et al.*, 2001) and is not negligible in comparison with the results recorded during the excavation.

Maximum observed settlements are less than 0.10% of the excavation depth, confirming the excellent level of control maintained during the excavation phase. Further, it can be seen that the cautious design predictions, resulting from the settlement basins provided by finite element calculations adjusted according the proposal of Clough & O'Rourke (1990), envelope most of the observed results.

With regard to the final results, it should be noticed that the largest settlements have been recorded in the vicinity of the location of one of the incidents and are obviously related to this event. Excluding these settlements, the total values are enveloped by a value close to 0.15% of the excavation depth, which can be considered a quite satisfactory result. A significant part of the difference between the settlements represented in Fig. 13c and the sum of those represented in Figs. 13a and 13b may be assigned to the dissipation of some positive excess pore pressure induced in the soft alluvia by the high strut pre-stressing and, possibly, by a general subsidence of the reclaimed area.

1.2. Comments

Some general comments on this overview are presented in the following.

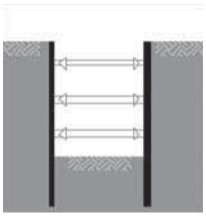
In 50 years – which is not a long period, barely corresponding to an entire professional life time – the evolution of deep urban excavations has been remarkable. We now undertake excavations that a few decades ago would have been unimaginable or would have involved unacceptable cost, construction time and damage in the vicinity.

Before the 1960s, this type of construction was characterized by solutions employing very primitive and limited techniques. At present, it is characterized by the use of advanced and diversified technologies, and by carefully designed and detailed construction operations and structural components.

The solutions that have been constructed encompass a large number of structural systems and soil-structure interaction problems. For a tentative classification, it may be useful to distinguish between the systems employed to support excavations whose longitudinal dimension is very large, which can be treated assuming plane strain conditions, and the cases having similar longitudinal and transversal dimensions and in which the support solution takes advantage of the 3D geometry of the system.

Table 2 represents the 2D systems through a cross sectional vertical plane and Table 3 includes the 3D sys-

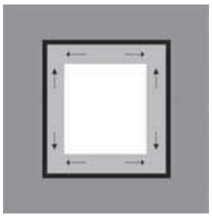
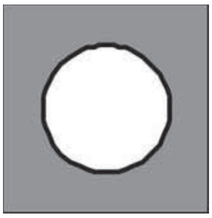
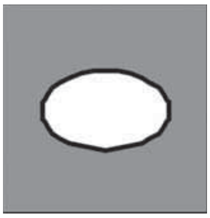
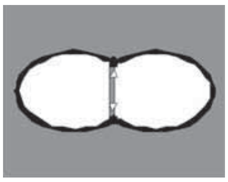
Table 2 - Retaining structures of excavations corresponding to 2D structural systems.

		Main type of structural loading		
		Wall		Supports
	Type of structure	Vertical plane	Horizontal plane	
	Cantilever	Bending	-	-
	Wall supported by slabs of permanent structure	Bending and compression	-	Compression and bending
	Strutted wall	Bending	Bending	Compression and bending
	Strutted wall and jet grout treatment under the bottom of the excavation	Bending	Bending	Compression and bending
	Anchored wall	Bending and compression	Bending	Tension
	Anchored permanent Berlin-type wall	Bending and compression (concrete panels) and compression (soldier piles)	Bending	Tension

tems through a section by a horizontal plane. The order of the presentation in each table corresponds, to some extent, to an increasing complexity of the soil-structure interaction and the order of designation of the type of loading reflects the respective relative importance for each structure.

It is interesting to observe that the type of representation adopted is tacitly related with the mental models of

Table 3 - Retaining structures of excavations corresponding to 3D structural systems.

	Type of structure	Main type of structural loading		
		Wall		Supports
		Horizontal plane	Vertical plane	
	Wall supported by peripheral slabs of permanent structure	Bending and compression	Bending	Bending and compression
	Cylinder shaft	Compression	Bending	-
	Elliptical shaft constructed by the sequential excavation-concreting method	Compression and bending	Bending	-
	Double elliptical shaft with central bracing constructed by the sequential excavation-concreting method (Case history 3)	Compression and bending	Bending	Compression and bending
	Wooden wine barrel type structure (Case history 2)	Bending and compression	Bending	Bending and tension
	Pre-stressed internal arch connected to "flying" struts (Case history 1)	Bending and compression	Bending	Compression (struts) and tension (tendon)

how the structures operate. As a matter of fact, for the structures of Table 2 the main mode of operation develops on vertical planes whereas the effects of the actions on the horizontal planes are null or correspond to simple bending (in the latter case, as a result of the discrete nature of the supports provided by the anchors or struts). On the contrary, for the structures of Table 3 the soil-structure interaction and the effects of the actions are clearly more complex on the horizontal planes whereas the ones on the vertical planes are due to simple bending.

A further curious point arises when the structural systems of Table 3 are considered. In fact, for these cases the conditions for developing soil arching are ideal, which makes very clear that *it is the soil that, ultimately, supports the soil*, the role of the structure being *to assist* the ground mass to adapt to the new equilibrium conditions induced by the excavation. Therefore, for comparable geotechnical conditions and performance, the structural systems of Table 3 require a lower amount of structural material than those of Table 2. Nonetheless, for the latter the statement that it is the soil that, ultimately, supports the soil still applies, by exploiting the loading symmetry, like in the multi-strutted walls, or by transmitting the thrust of the supported soil to self-stable zones of the ground, like in the multi-anchored walls and (or) by prolonging the wall beyond the base of the excavation.

The variety of the structural systems identified above evidence how this domain has been a permanent challenge to Engineers, requiring a well-balanced combination of geotechnical and structural sensibility and expertise.

The understanding of the behaviour of these complex structural systems, whose configuration evolves as the construction progresses and whose deformation strongly influences the magnitude and the distribution of the earth pressures and of the structural stresses and displacements, has been considerably enhanced, from late 1970s on, by the use of finite element models. In fact, the experience and the insights gained from studies using these models, progressively and extensively influenced the applied solutions, even in the cases whose design was, apparently, based on conventional methods alone. Such cases corresponded to the large majority until a few years ago.

Part 2 - Design and Research

This part contains three topics related to excavations in distinct geotechnical conditions occurring in the main urban areas of Portugal: i) stiff sedimentary clayey soils or volcanic soils, in which the so-called Berlin-type walls are very common; ii) weathered granitic rocks covered by residual soils; iii) thick layers of soft silty clays.

2.1. Excavations in stiff soils. The vertical stability of permanent Berlin-type walls

2.1.1. Introduction

A large number of excavations in stiff ground were successfully executed in recent decades with so-called permanent Berlin-type (concrete soldier-pile) walls, which became very common as an alternative to diaphragm walls. The main reason is that conventional diaphragm wall equipment often experiences considerable difficulties when excavating very stiff soil layers or rocks. Figure 14 illustrates the typical construction sequence of a permanent Berlin-type wall.

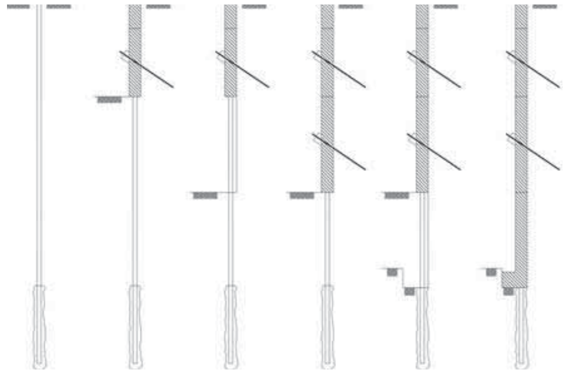


Figure 14 - Construction sequence of a permanent Berlin-type wall.

The same type of retaining structure has also been adopted in other scenarios, *e.g.* where a superficial thick layer of soft ground covers the stiff soil or rocky substratum. In a number of cases, this option has caused a significant degree of damage in nearby constructions and services.

In such situations, as well as in a few cases involving stiff ground, the poor performance of the retaining structure was often associated with insufficient bearing resistance of the soldier piles in relation to the vertical loads applied by the anchors and by the self-weight of the wall. Note that in deep excavations, the weight of the concrete wall may represent a significant contribution to the vertical loading, bearing in mind that the actual thickness may be much larger than the theoretical one, due to over-excavation at the cut face particularly in soft/weak ground, before concreting of the panels.

In general, the deficient resistance to vertical loading has been of structural nature with buckling of the soldier piles observed but a case involving bearing capacity failure of the pile has also occurred.

2.1.2. Mode of failure by loss of vertical equilibrium of Berlin-type walls

The vertical equilibrium of Berlin-type retaining walls was studied by Guerra (1999) and Guerra *et al.* (2004). The studies mainly included the collection of incidents and accidents by local observation and by literature survey, as well as field monitoring and finite element analyses. As depicted in Fig. 15, the field evidence and the numerical results show that the behaviour pattern of a Berlin-type wall under marginal stability conditions consists of a large settlement combined with a pronounced lateral displacement, which induces the progressive unloading of the anchors and, eventually, the overall collapse of the excavation.

An interesting point revealed by the finite element analyses is the fact that vertical failure may occur without full mobilization of upward shear resistance on the back face of the wall. Very similar results had been obtained by Matos Fernandes (1983) and by Matos Fernandes *et al.* (1993; 1994) regarding the vertical mode of failure of continuous retaining walls.

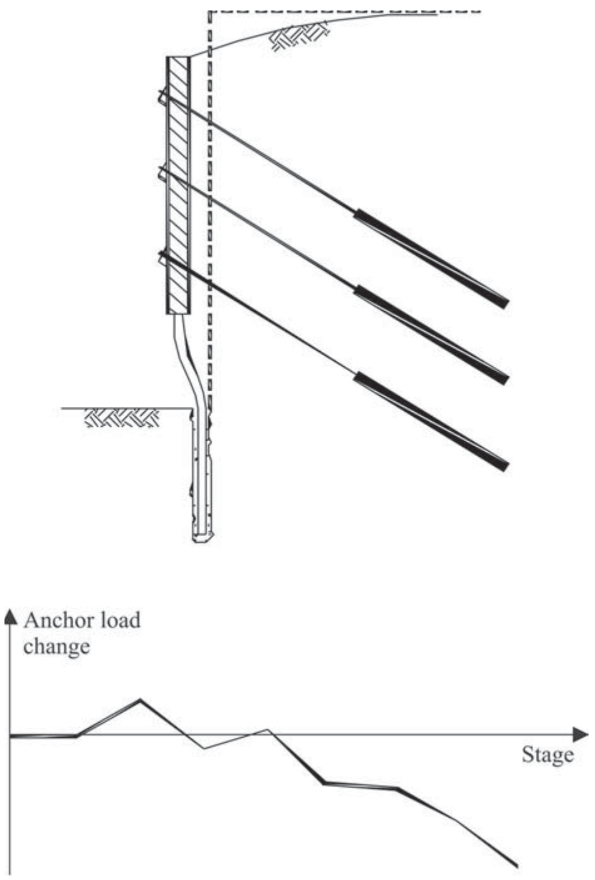


Figure 15 - Pattern of behaviour of Berlin-type walls with marginal stability conditions due to vertical loading.

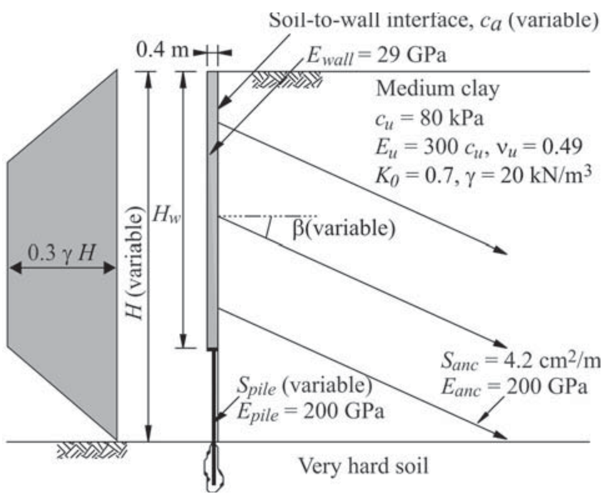


Figure 16 - Assumptions for the numerical case study of Guerra *et al.* (2004).

The following question arises: when calculating the vertical force to be supported by the soldier piles, is it reasonable to allow any mobilization of upward wall-soil interface resistance at the back of the wall?

Figure 16 summarizes the conditions assumed in the finite element study of Guerra *et al.* (2004). To investigate the significance of the resistance of the wall-soil interface for vertical stability, two parallel simulations were undertaken, with the same parameters, except for those defining that resistance. One of the analyses assumed a smooth interface (zero interface shear resistance) whereas the other one adopted a value for the interface adhesion, c_a = 50 kPa, representing a significant fraction of the soil undrained shear strength, c_u = 80 kPa. Figure 17 presents the displacements obtained from both analyses until stage 16 (collapse takes place in the next stage, excavating from 15 m to 18 m).

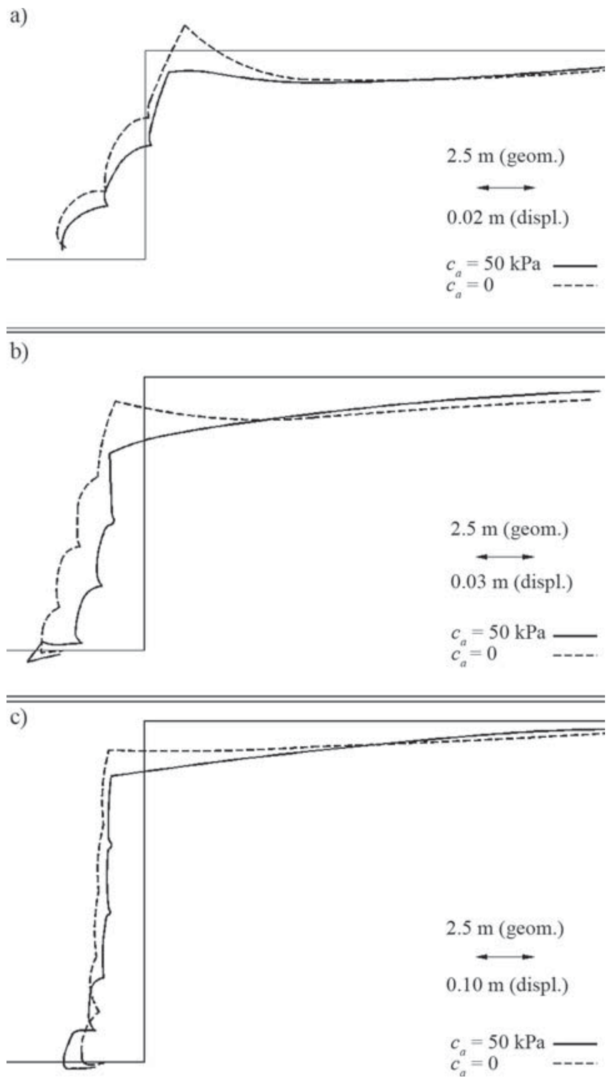


Figure 17 - Displacements of the excavation face and the ground surface for the case depicted in Fig. 16 (Guerra *et al.*, 2004): a) stage 10 (9 m depth); b) stage 13 (12 m depth); c) stage 16 (15 m depth) (note change in displacement scale).

It is rather interesting to note that the performance of the analysis with smooth interface, as far as the movements are concerned, is not worse (it could even be said that it is better) than the analysis with non-zero resistance of the interface.

The whole set of forces applied to the wall at the last stage represented in Fig. 17 was computed, as shown in Fig. 18. Note that the reaction of the soldier piles is the same in both analyses (it corresponds to the buckling load) as well as the wall weight. Since the tangential force at the interface is zero in the analysis with a smooth interface and directed upwards in the analysis with non-smooth interface, the vertical force is greater in the latter and, due to the inclination of the anchors, the same occurs with the horizontal interaction force. A very interesting situation is then observed: the two analyses involve quite different sets of forces but lead to comparable displacements.

These results demonstrate that when discussing the vertical loads on the soldier piles it is crucial to link the equilibrium of the wall with the equilibrium of the soil, not only via the tangential force but also through the normal force mobilized at the soil-wall interface!

In brief, the problem can be described as follows:

(1) considering the equilibrium of the wall only, it would appear that higher mobilization of upward tangential forces applied to the wall would lead to lower required pile resistance;

(2) however, higher upward mobilized adhesion at the back of the wall increases the total vertical downward force on the soil mass;

(3) then, the equilibrium of this mass will demand a larger horizontal force applied by the wall;

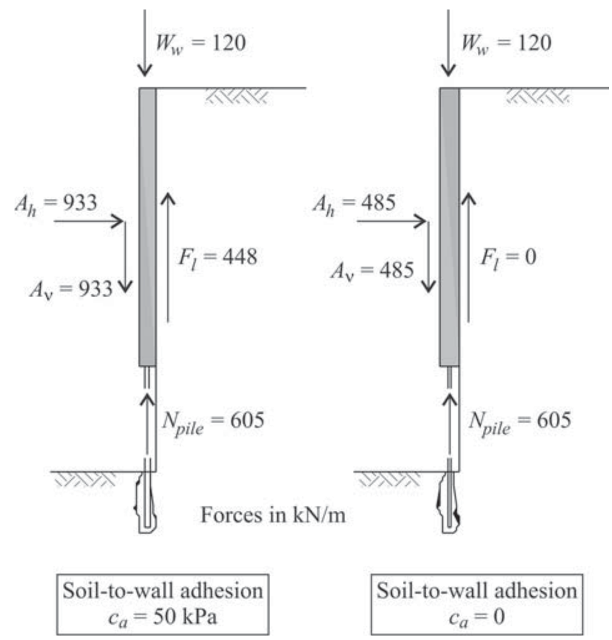


Figure 18 - Forces involved in the wall equilibrium in stage 16 (15 m depth) for the analyses of Fig. 17 (Guerra *et al.*, 2004).

(4) in Berlin-type walls this force is only controlled by the anchors; since the anchors are inclined downwards, this will lead to a greater vertical force on the wall;

(5) then, the mobilization of upward tangential forces applied by the soil to the back of the wall does not necessarily ensure a mitigation of the vertical loading on the soldier piles and a better overall performance of the system.

Further studies on this subject carried out by Cardoso *et al.* (2006) and Antão *et al.* (2008), combining the results presented by Guerra *et al.* (2004) with analytical and finite element upper bound limit analyses, confirmed that the minimum pile resistance to avoid collapse does not necessarily diminish with the increase of upward mobilized adhesion at the soil- wall interface. These studies further show that the soil strength and the angle of anchor inclination are the key factors in that interaction.

2.1.3. Closing comment on soldier pile design criterion

The discussion presented above shows how complex is the behaviour of Berlin-type walls due to the mutual dependence of vertical and horizontal interaction forces and the crucial role of the soldier piles in some conditions.

Anyway, for practical purposes the essential point to be stressed is that a sound behaviour of a flexible retaining wall corresponds to shear forces applied by the soil to the wall according to Fig. 19 (Matos Fernandes, 2004). In fact: i) if the foundation of the wall toe is suitable, the wall settlement will be negligible; ii) the lateral wall displacement towards the excavation, even a very small one, permits the supported ground to settle, which will induce a downward tangential force applied to the back of the wall; iii) the removal of the weight of the excavated soil as well as the wall displacement towards the excavation causes some heave of the soil under the base of the excavation, which will induce an upward tangential force applied to the front of the wall. For Berlin-type walls, only the force at the back of the wall is to be considered in design.

Bearing in mind the above considerations, the attempt to exploit the resistance of the soil- wall interface in order to mobilize upward shear forces on the wall, which could re-

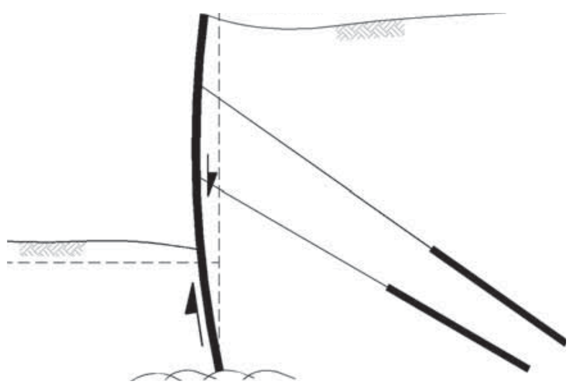


Figure 19 - Tangential forces applied by the soil to a continuous flexible retaining wall with sound foundation conditions.

duce the vertical force on the soldier piles brings with it a significant risk of deficient performance of the retaining structure with regard to movement control. If this risk is not acceptable, the soldier piles should be designed to support a vertical load not smaller than the resultant of the vertical components of the anchor forces plus the actual wall weight.

2.2. Excavations in granite residual soils and the weathered granite of Porto region

2.2.1. General

In the north-western region of Portugal granite rocks are dominant. The upper part of these rocks is typically weathered (with the distinct degrees of weathering from W1, fresh rock, to W5, completely weathered rock) and is commonly covered by residual saprolitic soils, whose thickness may be as much as 30 m (Matos Fernandes, 2006).

There are some important features particular to these geological conditions:

i) the residual soil and the weathered rock are extremely heterogeneous in plan as well as in depth;

ii) the thickness of the residual soil and of the distinct horizons corresponding to a given degree of weathering frequently reveal a very pronounced variation in the horizontal direction;

iii) frequently, horizons of rock covering residual soil layers may be encountered;

iv) very often, round blocks of sound granite ”core stones” are found in the residual soil mass.

These conditions cause serious difficulties for executing deep excavations (and tunnels), particularly in Porto, the most important city of the region, where excavations for deep basements became increasingly common from the seventies.

From that time to the turn of the century, a significant number of problems related with construction difficulties and deficient performance, particularly concerning movement control, and even some serious incidents, have been registered. It should be noted that such episodes did not occur in Lisbon to a comparable degree, in spite of the fact that the pool of designers and contractors is, to a great extent, the same in both regions. There are two main reasons that explain this fact.

2.2.2. Difficulties in applying a construction technique appropriate for granitic formations

Difficulties arose from the adoption of inappropriate construction techniques. In deep excavations supported by concrete diaphragm walls, the drilling equipment could not properly excavate the rocky horizons that were often encountered above the final excavation base. In other cases, that equipment could not cut through the core stones encountered in the residual soil mass.

The option to overcome these difficulties normally consisted of stopping the panels when the rate of the drilling process became unacceptably low. The excavation was then carried out, with installation of pre-stressed ground anchors attached to the diaphragm wall, but with no support on the cut face below the panel tips, as suggested by Fig. 20a. From the bottom, a conventional reinforced concrete perimeter wall was then erected and linked to the base of the diaphragm wall panels. Note that, due to the heterogeneity of the ground, the depth at which the panel excavation process stops varies significantly across the excavation face.

The difficulties faced by the diaphragm walls encouraged the use of permanent Berlin-type walls, with the soldier piles sealed in holes extended beyond the base of the final excavation, as shown in Fig. 20b.

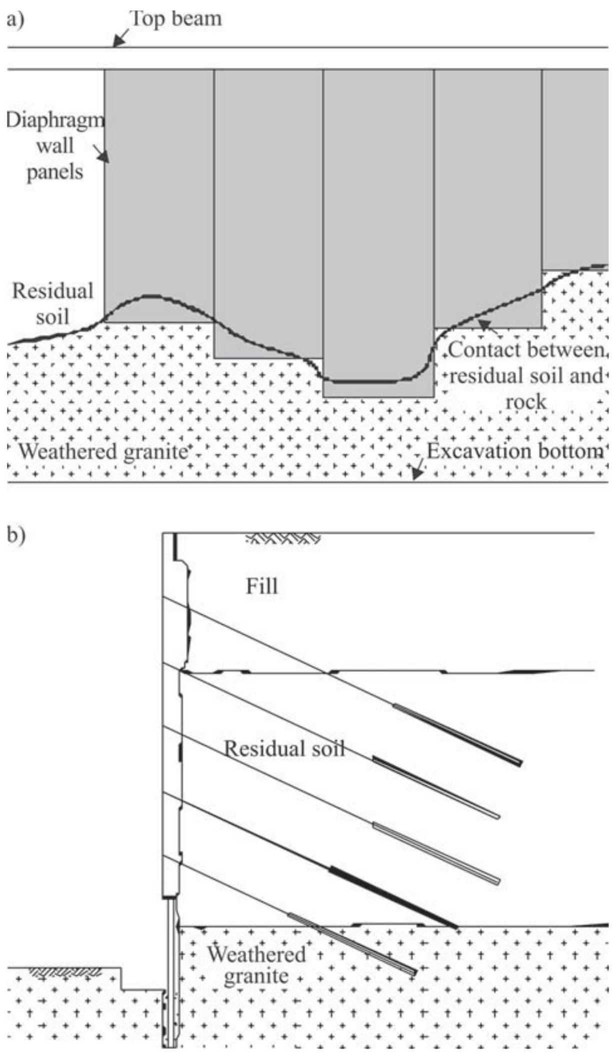


Figure 20 - Examples of difficulties found in excavations in the granite formations of Porto (depicted from real cases): a) diaphragm wall panels of variable height but with the tip above the excavation bottom; ii) permanent Berlin-type wall supporting a thick layer of residual soil and fill but with the rock appearing above the excavation bottom.

Both solutions depicted in Fig. 20 frequently exhibited poor performance, inducing large movements and structural damage to buildings in the vicinity. With regard to Berlin-type walls, in a number of cases, the poor performance was related to insufficient vertical bearing resistance of the soldier piles, as discussed earlier. The conditions for ensuring vertical equilibrium for the diaphragm wall type retaining structure depicted in Fig. 20a seem to be questionable, as well, and they have probably contributed to the inadequate behaviour observed.

2.2.3. The unfavourable behaviour of the granitic formations

The second point that explains the difficulties encountered when performing deep excavations is the unfavourable behaviour – one could even say, the surprisingly unfavourable behaviour – exhibited by the granite residual soils in excavation works (tunnels and excavations). In fact, there is a clear discrepancy between the behaviour of residual soils in foundation works, in which they are loaded under a given (and, in most cases, increasing) confining state of stress and in excavation works, in which they are loaded with reduction of the mean normal stress.

In the first type of problems residual soils normally exhibit satisfactory performance, which seems to be better than sedimentary soils with similar grain size distribution and void ratio. This is testified by the fact that many of the buildings in Porto, even large ones, are founded on footings in these soils. However, in excavation works the behaviour of granite residual soils has been much worse than expected when developing design and construction options.

The stress relief that follows the excavation seems to induce, at a microscopic level, irreversible damage to the particle bonds. This belief is corroborated by the experience gained at the University of Porto when characterizing the stiffness of Porto residual soils. For example, Viana da Fonseca *et al.* (1997) found that the stiffness values estimated in the lab on block samples, carefully collected and instrumented, were just one-third of the values measured in a footing loading test in the field. More recent studies, that compare shear wave velocities measured in the field and in the laboratory on block samples, confirm this observation (Ferreira, 2008). The conclusion is that the stress relief associated with block sampling, similar to the one induced by an excavation, is very detrimental with regard to the cemented structure of the soil.

On the other hand, at a macroscopic level, there is the influence of the countless low strength surfaces in the soil mass inherited from the fractures of the parent rock. These surfaces, in spite of being almost indiscernible as a consequence of the deep weathering that produced the soil, condition in a very palpable way the behaviour of the ground mass, for the type of loading typical of excavations. In fact, the reduction of the effective mean stress facilitates the movement along these surfaces or, in non-supported cut faces, slip failures like the one shown in Fig. 21, which resembles a rock slope failure.

2.2.4. Successful solutions applied in the construction of Metro Stations in Porto

In the last decade, the number of large and deep excavations in the Porto region has increased significantly. In particular, the construction of the first phase of the Metro system required the construction of 13 underground stations in these residual granite soils. Among these, 10 were cut-and-cover excavations (with maximum depth of 25 m) and the remainder were caverns associated with large access shafts.

For the cut-and-cover excavations a successful solution applied to most of them consisted of anchored large diameter concrete pile walls. These walls were of two types:

i) reinforced concrete bored piles whose horizontal spacing between axes is greater than the diameter (which, in some cases, required the application of sprayed concrete on the soil surface between piles, following the progress of the excavation);

ii) secant bored piles, consisting of a combination of plain concrete (with some bentonite mixed with cement) piles, constructed in advance, and reinforced concrete piles constructed in the intervals of the former ones.

The piles have been extended beyond the base of the excavation (with very few exceptions), which required drilling through the weathered rock (and in some cases through the fresh rock) with recourse to either rock augers or rock core barrels. This option enabled the construction of very deep excavations in the centre of the City, close to many historical buildings, with negligible damage, as a rule. See Fig. 22 for an example.

Large elliptical shafts constructed by the sequential excavation-concreting method were adopted for the main excavation at two stations. One of them was the double elliptical shaft for Salgueiros Station (case history 3 above).

The satisfactory adaptation of this solution to the geotechnical conditions of Porto, in comparison with those

Figure 21 - Example of a partial collapse in a sloped excavation in granite residual soils controlled by fracture of the parent rock.

Figure 22 - Aliados Station of the Porto Metro constructed in very heterogeneous residual granite soils: a) view of the large diameter concrete pile wall; b) a view of piece of rock extracted with core barrel.

mentioned in Section 2.2.2, may be explained by the following reasons:

i) the height of unsupported soil for each excavation step, as well as the time between the excavation and the application of the concrete wall were small;

ii) the lowering of the water table in advance of the excavation, provided by vertical and sub-horizontal drains in the retained soil, induces a suction on the soil, which improves the soil strength and stiffness in a similar way of an increase in the effective confining stress (Topa Gomes, 2008).

These two factors seem to be very effective in minimizing the degradation of the mechanical properties of the soil mass, thereby ensuring satisfactory behaviour.

2.2.5. Conclusion

The behaviour of excavations in residual granite soils is now faced by construction professionals as well as academics under a quite distinct perspective from the one prevalent until relatively recently. Such behaviour is much less favourable than expected and recommends, in general, more conservative solutions.

It could be said that, for the complex geotechnical conditions of the Porto region, only in the present decade has a sound base of reference, experience and sensibility been attained with regard to cut-and-cover excavations and tunnelling (Babendererde *et al.*, 2004).

Anchored concrete bored pile walls, with the pile base extended below the base of the excavation, and elliptical shafts constructed by the sequential excavation-concreting method have proved to be quite convenient solutions for cut-and-cover excavations.

2.3. Deep excavations in thick deposits of soft clay

2.3.1. Evolution of the ability to control movements induced by the excavation

The execution of deep excavations in thick deposits of soft soils has been one of the major challenges for geotechnical engineers. The empirical chart by Peck (1969) providing estimates of the limits of settlements induced by cut-and-cover excavations is well-known and widely cited even in recent books and papers. The chart is organized according to the type of soil; for soft clays the expected settlements are extremely large: exceeding 1% to 2% of the excavation depth.

However, as it was emphasized above, supported excavations are a complex soil-structure interaction system, in which the characteristics of the structure, as well as the construction sequence influence the results of the interaction. So, the Peck settlement chart obviously reflects the

Table 4 - Type of walls used for deep excavations and maximum recorded settlements in soft to stiff clays (adapted from Duncan & Bentler, 1998).

Time period	Total number of cases	Diaphragm walls (%)	Soldier pile walls (%)	Sheet pile walls (%)	Other walls (%)	u_v^{max} / h (%) (average)
1962-1975	67	31	33	25	11	1.28
1976-1989	70	40	24	26	10	1.21
1990-1998	47	53	17	11	19	0.41

type of retaining structures and construction methods employed until the 1960s; these consisted mainly of flexible sheet pile or soldier-pile walls with cross-lot (non prestressed) struts or rakers.

From the late-1960s and in the following decades, some remarkable new technologies which were capable of better controlling induced movements, were progressively introduced. These solutions, together with the understanding brought by finite element analyses of the parameters influencing the induced movements, resulted in the completion of excavations in soft ground with much better results in comparison with Peck's chart.

This is well evidenced by the results presented by Clough & O'Rourke (1990), shown in Fig. 23. These results are corroborated by the conclusions of Duncan & Bentler (1998), summarized in Table 4, that highlight the progressive increase of the use of diaphragm walls, accompanied by a tendency for the decrease of the magnitude of the induced movements.

Figure 23 further illustrates a very large scatter of results, thus proving that the specific parameters of the retaining structure and of the construction are extremely important with regard to the induced movements.

A number of studies have been carried out with the purpose of relating some parameters dependent on the soil and on the structure with the induced movements (Mana & Clough, 1981; Clough *et al.*, 1989; Addenbrooke, 1994; Long, 2001). Some examples may be presented, using the results of the novel data base collected by Moormann (2004) in which only a few case histories prior to 1980 were introduced.

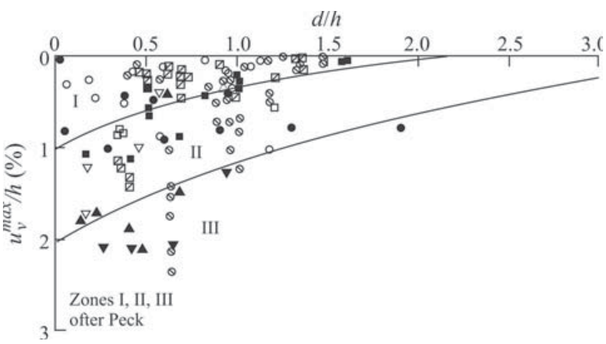


Figure 23 - Summary of measured settlements adjacent to excavations in soft to medium clay collected by Clough & O'Rourke (1990) over Peck's chart.

Figure 24a represents the relationship between maximum lateral wall displacement, expressed as a percentage of the excavation depth, with the factor of safety against basal heave according to the Terzaghi definition. It is interesting to observe that much better results appear in comparison with the trend identified by Mana & Clough (1981). The scatter is, however, very pronounced: for the same value of the safety factor, the displacements may vary by an order of magnitude!

Figure 24b presents the maximum lateral wall displacement plotted with respect to the system stiffness, ex-

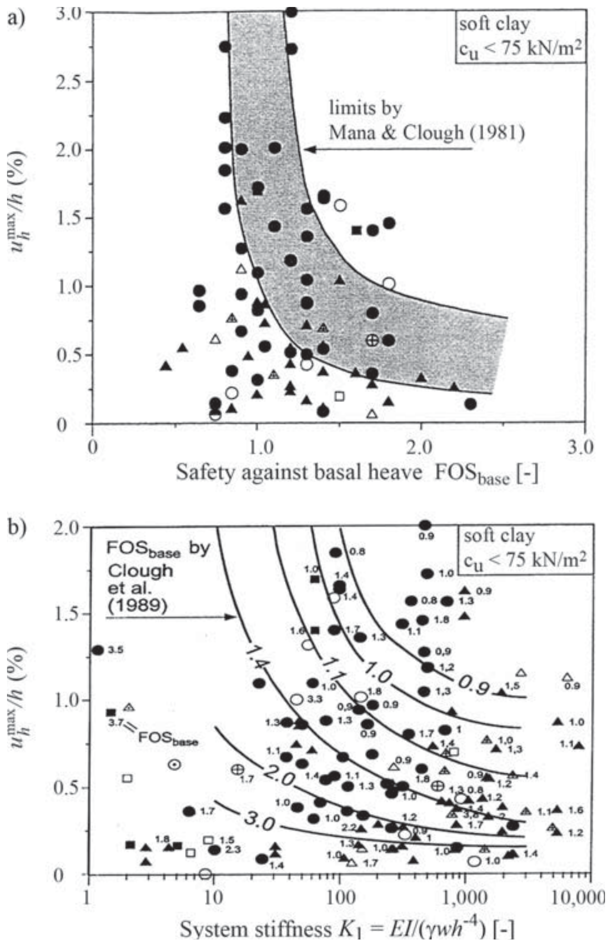


Figure 24 - Maximum lateral wall displacement in excavations in soft to medium clay collected by Moormann (2004) *vs.*: a) safety factor against basal heave; b) system stiffness.

pressed on the basis of the wall bending stiffness, *EI*, and the average vertical spacing of the wall supports, $\overline{h}$ (Clough *et al.*, 1989). The scatter of the results confirms that the behaviour of excavations in thick deposits of soft clay is very complex, depending on a large set of factors and parameters concerning the ground, the structure and the construction.

2.3.2. Higher or lower tolerance with regard to the induced movements when dealing with soft soils?

In spite of the progress in the control of movements revealed by Figs. 23 and 24 and by Table 4, it must not be forgotten that the degree of damage in the vicinity of the excavation obviously depends on the magnitude of the induced movements and in these figures the displacements are expressed as a percent of the excavation depth. This is a relevant point bearing in mind that such excavations are becoming deeper and are being done under more daring and demanding conditions. In fact, many underground works that have been recently carried out in our cities founded on soft clay would be intolerable if the induced movements were in accordance with the majority of the results shown above.

The fact that such results are relatively common even nowadays is possibly related to the idea – which is still commonly accepted in engineering practice – that the achievable performance with regard to movement control essentially depends on the soil type and ground conditions.

As a result of this idea the Profession is tacitly *more tolerant* of excavation induced movements in soft ground. However, ancient structures and services in soft ground might probably be *less tolerant* with regard to further movements than similar constructions over stiff soils! Therefore, a reversal of perspective in facing this matter is imperative.

In any case, the support solutions presently available and the awareness of the factors that control the induced movements permit to achieve similar performance both in deep deposits of soft soils and in other stiffer soils.

2.3.3. Eight Golden Rules for a reliable control of the excavation induced movements

As it was seen in Section 2.3.1, no conclusive correlations can be established between recorded displacements and certain parameters of the structure. The literature search for cases in which the induced movements were rather small, and the identification of common features related with the structure and the construction, appears to be a promising way to obtain consistent guidelines for future projects.

With regard to this strategy, the construction of six cut-and-cover metro stations in the soft to medium clayey soils of Shanghai (Wang *et al.*, 2005), as well as the construction of three metro stations in Lisbon downtown (Matos Fernandes *et al.*, 2007), are references of the utmost importance.

These and other case histories and the substantial amount of insight provided by the use of finite element analyses in the last decades, show that an effective control of the movements (excluding those from wall construction) arises from the combination of all or most of the eight golden rules summarized in Fig. 25. Table 5 includes some comments concerning the proposed rules.

It should be noted that the first letters of the eight rules form the word RELIABLE! It is an auspicious coincidence since this word in common speech means safe, robust, stable, etc. But one can further relate the application of the rules with the technical meaning of that word, since they may be considered as conditions for obtaining *reliable predictions and control* of the induced movements. In fact, such predictions frequently fail because the support solutions employed in the actual construction permit, in many cases, large regions of the ground to reach plastic yield. Under such circumstances, minor variations of the soil undrained shear strength, which in this type of ground is typically anisotropic, may considerably affect the magnitude of the resulting movements.

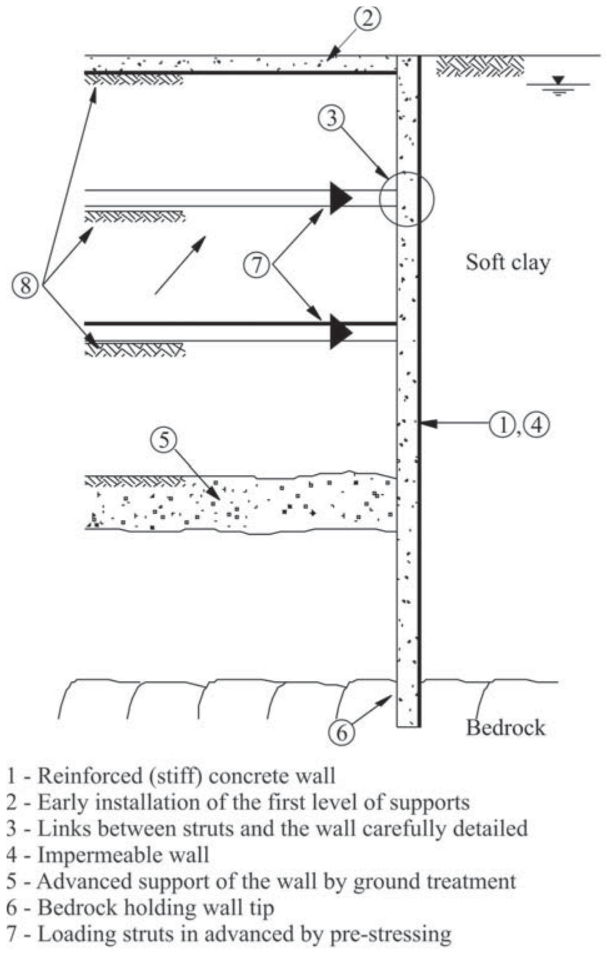


Figure 25 - Golden rules for a reliable movement control induced by deep excavations in thick deposits of soft ground (Matos Fernandes, 2007).

Table 5 - Comments on the 8 golden rules for a reliable movement control induced by deep excavations in thick deposits of soft ground.

Rule	Comment
Reinforced (stiff) concrete wall	Diminishes the wall deflections due to bending. It is particularly important for controlling the displacements below the current base of the excavation
Early installation of the first level of supports	Prevents significant displacement of the upper part of the wall working as a cantilever. The pre-stress of the first level of supports should be small in order not to induce wall displacement towards the supported ground. This level should be connected to the wall in order to be capable of carrying tensile loads, which tend to be mobilized when high pre-stressing is applied in the sequent support levels
Links between struts and the wall carefully detailed	It is of utmost importance to ensure that the effective strut stiffness represents a high percentage of its theoretical value
Impermeable wall	Impedes lowering of the water table in permeable layers, which induces consolidation settlements in soft clay strata. It prevents settlements associated with internal erosion of sandy strata
Advanced support of the wall by ground treatment	Controls the displacements below the current base of the excavation. It is particularly important when the distance from the base of the excavation to the top of the substratum is considerable. In very deep excavations, whose base approaches the top of the substratum, the treatment becomes more effective if it is performed above the excavation bottom
Bedrock holding wall tip	Particularly important for controlling the displacements below the current base of the excavation
Loading struts in advance by pre-stressing	Increases the effective strut stiffness by closing gaps in the system linking the struts and the wall. Pre-stressing forces that represent a high percentage of the at-rest total horizontal thrust are capable of recovering displacements induced by previous excavation stages and condition the state of stress in the ground in a favourable way for the sequent stages
Excavation limited to minimum at each stage	Avoiding over-excavation maximizes the support effect provided by the struts or members like the slabs or beams of the permanent structure

A retaining structure designed and constructed according to these rules is capable of maintaining the major part of the ground mass far from yielding, because its level of deformation will be very small. Thus, its performance shall be easier to predict because it will be mainly dependent on structural and constructional features, exhibiting small sensitivity to those issues whose accurate characterization is more difficult. In short, it will be a *reliable system*.

In such cases, agreement between prediction and performance should be mainly credited to appropriate conception of the structure and to competent construction, and not so much assigned to the sophistication of design prediction analyses.

Part 3. Research and Perspectives

It is difficult to foresee the specific challenges that urban excavations will represent for geotechnical engineers in the future. Anyway, that challenge may be summarized, in broad terms, as *deeper and safer*!

This will require not only progress in distinct fields, such as, construction techniques, structural solutions, methods of analysis, etc. but also that much attention will need to be paid to issues that for shallower excavations were of lesser importance. This part of the paper contains some ideas on future developments to respond to the challenge mentioned above.

3.1. Closing the "analytical gap"

It is well known that in flexible retaining structures the deformations by bending play a critical role in the distribution of earth pressures and of the structural stresses. The ability to experience deformation without stiffness reduction is quite different in steel and in concrete walls. For the bending strain level commonly attained by concrete retaining walls, it is recognized that the behaviour is no longer linear elastic (Figueiras, 1983).

In spite of this fact, the geotechnical finite element models commonly used for design assume a linear elastic behaviour for the structural components, while also offering many non-linear constitutive laws for the soil, some of them highly sophisticated. It is interesting to observe that the opposite very often occurs with regard to the soil-structure interaction analyses performed by structural engineers: more or less complex non-linear constitutive laws for the concrete whereas the foundation soil is assumed to be linear elastic! Table 6 summarizes the situation described.

In the author's opinion, overcoming this *analytical gap* is probably the most relevant task for the near future in

Table 6 - The "analytical gap" between structural and geotechnical analyses.

Analyst (type of structure)	Structural Engineer (conventional structures)		Geotechnical Engineer (retaining structures, tunnels, etc.)
Structure	Non linear model	GAP	Linear model
Soil	Linear model		Non linear model

what concerns the computational methods applied in design.

This goal does not necessarily require the development of new computational models. A convenient solution might be to implement an interaction (a dialogue) between the so-called geotechnical computational code (Code G) and the so-called reinforced concrete computational code (Code RC). Code G corresponds to the models presently available for analysing geotechnical works and particularly cut-and-cover excavations that represent the ground and the retaining structure and simulate the sequence of the construction. Code RC corresponds to nonlinear finite element models of concrete structures, accounting for the contribution of the steel reinforcement. In the present case it would represent just the concrete retaining wall.

For each stage of construction the wall displacements computed by Code G would be introduced (imposed) in the structure represented by Code RC. This code could then calculate the strains in the concrete and the corresponding adjusted stiffness, which would be introduced in Code G. This interaction between the two codes would require, like any other nonlinear analysis, an iterative process for each stage of construction.

3.2. The relevance of hydraulic issues in very deep excavations

Notwithstanding the content of the former section, the support of urban excavations has generally been viewed by the geotechnical engineers from a structural point of view. However, the trend for carrying out deeper excavations requires that greater attention is devoted to the hydraulic aspects of the design.

Firstly, the hydro-geological impact of the excavations has been very often neglected or even ignored in the past. However, for deep and long permanent (impermeable) retaining walls the impact on the ground water conditions may be quite considerable. The increasing environmental awareness of Society and of Administrations will require that these impacts be carefully considered in the future. This opens a stimulating field with regard to the conception of retaining structures capable of minimizing those impacts, such as the example depicted in Fig. 26.

Further, in urban areas of complex geology, unknown deep artesian aquifers, which do not impact significantly on excavations of moderate depth, may become highly influential for much deeper excavations, as suggested by Fig. 27. Therefore, the design of such excavations should include a careful reassessment of the hydro-geological conditions at the site.

A third point deserving particular attention is that the deviations of diaphragm wall panels or of concrete piles (in secant pile walls) from their theoretical design positions may become significant for such great depths, which may compromise the water tightness of the wall. If this occurs in combination with permeable sandy layers behind the wall and with the water table close to the surface, the intrusion of soil and water in the excavation will arise and may be very difficult to control. In such situations, the study of the actual positions of the panels or piles from the very early excavation stages will allow an evaluation of the hydraulic risk and may recommend treatment (jet grouting or similar) of the most problematic points.

3.3. A new paramount advance towards zero excavation induced movements

3.3.1. Introduction

In Section 2.3, the need for a new perspective with regard to the movements induced by excavations in soft clayey soils has been advocated. In brief, if the existing constructions are *less tolerant* to further movements of their foundations, the design options for the excavation to be performed nearby must be, as well, *less tolerant* regarding the induced movements, in comparison with other more favourable geotechnical conditions.

In Section 1.1.5, the case of the excavation for the Terreiro do Paço Station, Lisbon, performed under very difficult and demanding conditions was presented. From

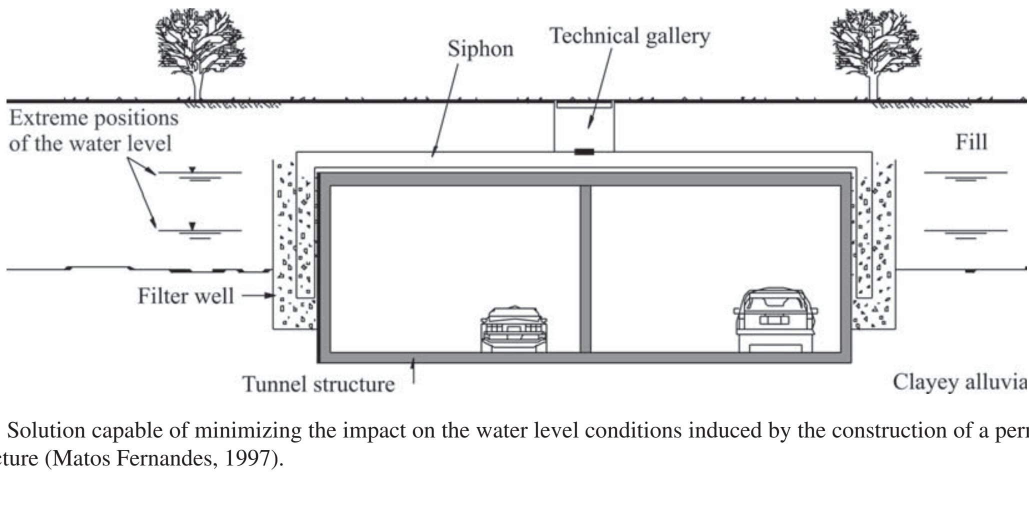


Figure 26 - Solution capable of minimizing the impact on the water level conditions induced by the construction of a permanent long buried structure (Matos Fernandes, 1997).

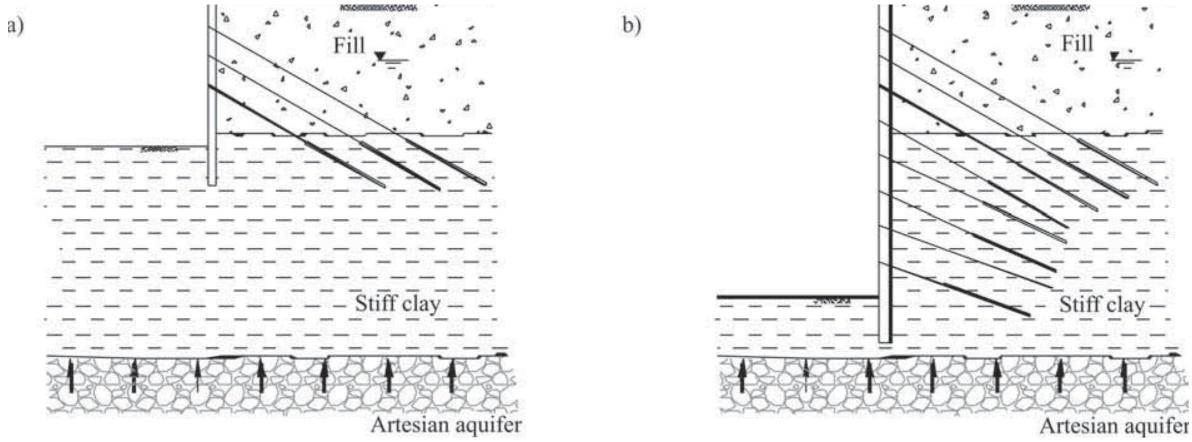


Figure 27 - Influence of a deep artesian aquifer embedded in the substratum: a) negligible influence for an excavation of moderate depth; b) possible influence for a very deep excavation.

the results shown in Fig. 13, it can be concluded that the settlements associated and simultaneous with the phased process of excavation and support installation are just a fraction of the total induced movements. Bearing in mind this conclusion, and adopting the new perspective, endorsed before, that in soft ground the control of the movements should be even more severe than the one achieved in stiffer soils, the settlements associated and simultaneous with the excavation and support installation, the ones that to a major extent depend on design options, must be limited to negligible values.

To see whether this aim is attainable it should be noted that, as shown in Fig. 13b, these settlements (and the corresponding wall movements, as well) seem to be well captured by our models of analysis. Then, these models seem to be a reasonable tool to search for the type of structure and the construction sequence capable of minimizing the wall and surface movements during the excavation.

3.3.2. A second paramount advance: to provide an effective support to the retaining wall before performing the excavation

If the author was asked to select the most important technical advance among those applied in the so-called *itinerary* of Part 1, he would definitely indicate the diaphragm concrete walls as being paramount. In fact, this advance made it possible *to install the entire retaining wall* - stiff, resistant, impermeable, extending beyond the excavation bottom, with minor or modest impact on the ground - *before performing the excavation*.

A second paramount advance would be to *support the retaining wall before performing the excavation*. If that support was effective and if it could be installed with minor impact on the ground, the movements occurring in parallel with the execution of the excavation would become practically negligible.

As with the first paramount technical advance, this second has been experimented with for sometime in a number of distinct forms. Figure 28 summarizes some well-known solutions:

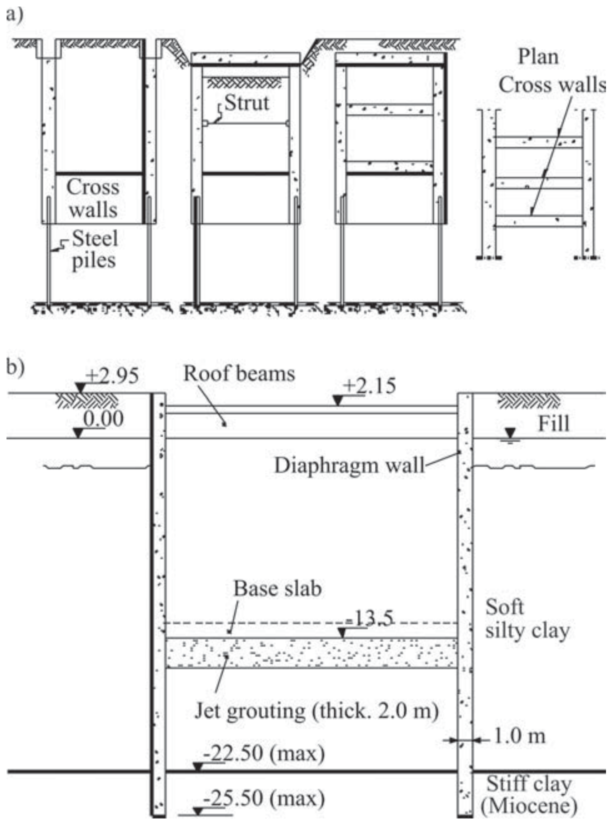


Figure 28 - Solutions for reducing the movements of retaining walls supporting deep excavations in soft clay: a) cross diaphragm wall panels below the base of the excavation (Oslo Metro); b) jet grout slab below the base of the excavation (Cais do Sodré Station of Lisbon Metro).

i) cross diaphragm wall concrete panels, acting as abutments, under the bottom of the excavation (Eide *et al.*, 1972);

ii) jet grout continuous slab under the bottom of the excavation (Wang *et al.*, 2005; Matos Fernandes *et al.*, 2007).

A case with a jet grout slab above the level of the final excavation was already presented in Fig. 11.

These solutions have proven to be more effective than more conventional options in the control of movements. Nevertheless, they have some important limitations.

The cross diaphragm wall panels must remain underneath the excavation bottom due to the difficulty involved in their demolition. Besides, difficulties arise concerning imperfect cleaning of the interface between the longitudinal walls and the cross walls as well as imperfections in the panel joints. This might explain the fact that its application has been very rare and restricted to Scandinavian countries (Karlsrud & Andresen, 2007).

With regard to jet grouting, the control of the geometry of the columns is difficult, particularly in this context because they are executed at great depth. Bearing in mind that soft soil must not remain between columns, these are executed by adopting distance between column axes smaller than the expected diameter. This can lead to outward wall displacements even greater than the inward ones that are to be prevented with the treatment (Wong & Poh, 2000).

A common criticism applicable to both solutions is that they require operations from the surface to form deep structural elements of major importance for the performance of the retaining structure, under very difficult conditions.

As shown in Fig. 29, a more convenient solution might consist of a transverse support, from the top to the bottom of the wall, with a given interval in the longitudinal direction, executed by the novel soil improvement technique called *cutter soil mixing* (CSM), inspired by the experience gained in the production of diaphragm wall cutters.

As shown in Fig. 30, the soil is broken down by cutter wheels rotating about a horizontal axis, and is mixed in situ with cement slurry. A continuous wall is formed by the construction of individual panels in an alternating sequence of overlapping primary and secondary panels. Secondary panels can be constructed immediately after completion of primary panels or by cutting into panels that have already hardened. Panels can be constructed in lengths ranging from 2.2 m to 2.8 m and wall thicknesses of 0.5 m to 1.0 m .

The experience available suggests that this technique could fulfil the following five essential requirements:

i) negligible impact associated with installation, avoiding over-compression or stress relief of the surrounding soil;

ii) constituent material easy to excavate but with good strength and stiffness, as well;

iii) very accurate installation position;

iv) confidence with regard to continuity in the transverse direction;

v) confidence with regard to the connection to the peripheral concrete wall.

As shown in Fig. 29, considering a peripheral diaphragm wall, it would be convenient to install the treated zones coinciding with the joints between the reinforced concrete panels, whose typical interval is around 5 m to 6 m. For this range, and bearing in mind the diaphragm wall thickness commonly adopted in this type of excavations, it may be anticipated that the 3D effects with regard to the wall movements (that is, the differences between the wall displacements in the supported sections and in the sections midway between supports) would be negligible.

Figure 31 illustrates the sequence of construction of a strutted diaphragm wall supporting a deep excavation in soft clay with such a treatment applied before excavation commences. The treated zones are excavated together with the soil, stage by stage, in conjugation with the installation and pre-stressing of the temporary struts.

3.3.3. A preliminary numerical experiment

In order to obtain a tentative evaluation of the effectiveness of the system described, the geometrical and geotechnical conditions of Terreiro do Paço Station, Fig. 11, were selected. A diaphragm wall 1.2 m thick, correspond-

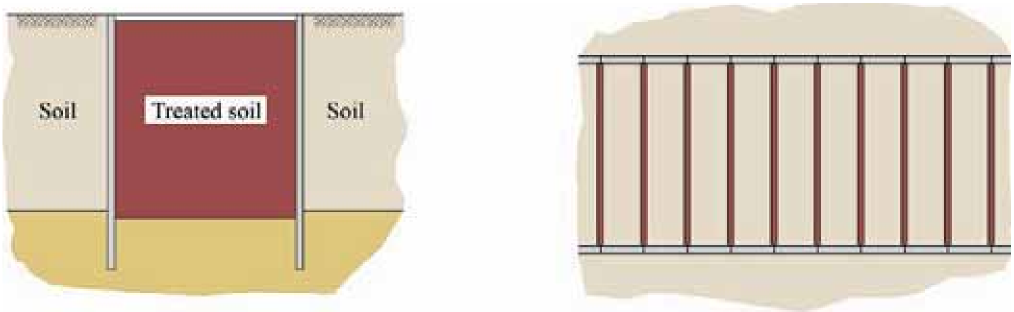


Figure 29 - Transverse supports of treated ground from the top to the bottom of the wall performed before the excavation: a) cross section of the excavation; b) plan.

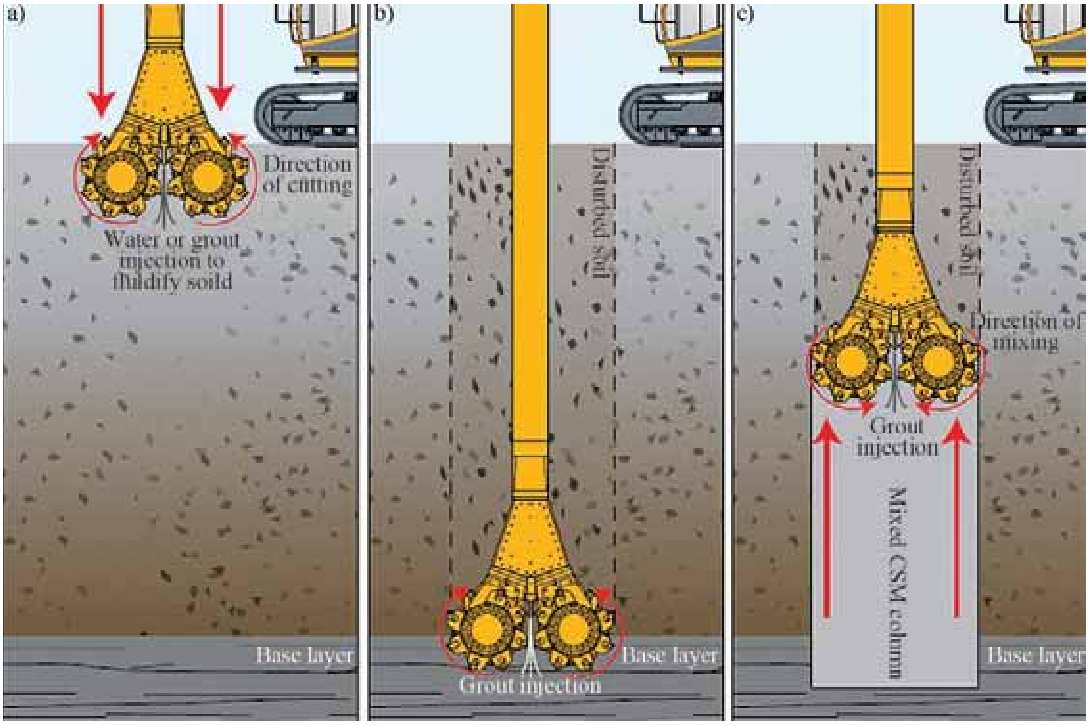


Figure 30 - The CSM technique: a) start of drilling with the ground being softened and broken by the cutter wheels; b) injection of cement slurry begins when the maximum depth of treatment is reached; c) progressive extraction of the equipment maintaining the rotation of the wheels and the injection of slurry (www.golder.ca).

ing to the bending stiffness of the piled wall actually used, was modelled. The five levels of temporary struts were modelled with an average horizontal spacing of 3.0 m and with a theoretical axial stiffness and pre-stress load (per linear meter) equal to the actual structure, as well.

Table 7 summarizes the finite element analyses carried out. Analysis C essentially corresponds to the solution applied to the actual excavation. Analysis D corresponds to the solution depicted in Fig. 31, assuming the transverse panels 0.8 m thick, 30 m high (which corresponds to an embedded height of 3.0 m in the substratum) and spaced 6.0 m horizontally. In order to ensure comparable results, all the analyses were performed with Plaxis 3D. By taking advantage of the symmetry conditions of the problem, the system analysed corresponds to just half of the slice represented in Fig. 31, thus with a thickness equal to 3 m in the longitudinal direction.

Figure 32 illustrates the final lateral wall displacements and wall bending moment envelopes from the four analyses on the plane midway between transverse panels. As expected, a substantial improvement in the movement control can be observed from analysis A to analysis B, due to strut pre-stressing, and from this to analysis C as a result of the restraint provided by the jet grout mass. Moreover, this provokes a considerable reduction of the maximum wall bending moment, in comparison with those provided by analyses A and B. It is interesting to observe that, in spite of the large difference concerning the structural restraint provided by the ground treatment – a horizontal “slab” in analysis C and a vertical “abutment” in analysis D – the lateral wall displacements from these analyses are almost coincident.

Bearing in mind the considerations, outlined above, concerning the control conditions under which the CSM panels are executed, it appears that the envisaged solution is quite promising for the struggle towards zero wall displacement during the excavation process. Moreover, the proposed solution provides some additional very relevant advantages, such as:

i) the CSM panels remaining underneath the final excavation level will provide a foundation for the basal slab of the permanent structure;

ii) the CSM treatment can seal a hole resulting from a serious defect at a wall joint;

iii) the CSM panels may be used, while they are fluid, to install vertical soldier piles for support of the bracing system;

iv) the CSM treatment provides a non-negligible reduction of the maximum positive and negative wall bending moments, in comparison with the jet grout solution.

Note that for better comparison the stiffness assumed for the treated material in analyses C and D is coincident. This seems to be corroborated by the experience with this type of soil (Peixoto, 2010).

In the example presented in analysis D, the CSM treatment corresponds to about 14% of the volume of soft ground enveloped by the peripheral wall. For comparison, it can be said that for analysis C, as well as for the case presented in Fig. 28b (Cais do Sodré Station, Lisbon), the volume of jet grout is about 12% of the soft ground volume. However, for jet grouting the cost of drilling from the surface must be added. The costs per unit volume of treated ground with the two techniques may vary significantly with a number of factors, and so they will not be compared in the paper.

3.3.4. Conclusion

The idea presented in this section obviously needs further numerical and practical studies before it is tested

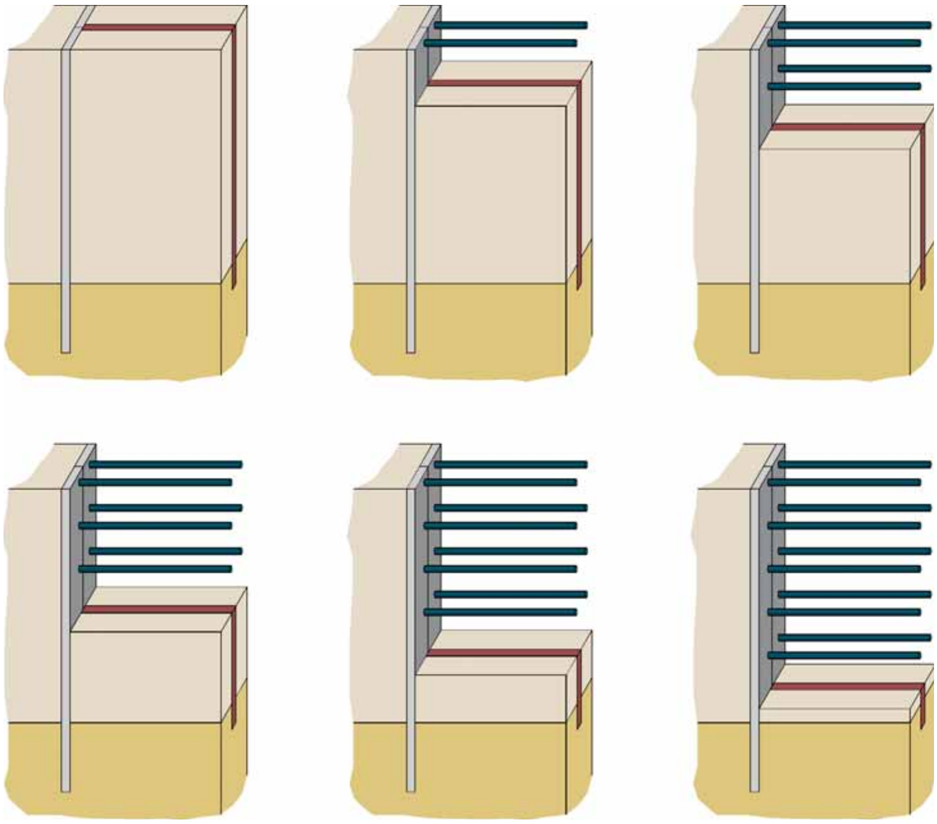


Figure 31 - Simplified sequence of the construction applying the support envisaged in Fig. 29.

Table 7 - Summary of the finite element analyses.

Analysis	Conditions assumed for the support of the wall
A	5 levels of non pre-stressed struts (1) (2) (6)
B	5 levels of pre-stressed struts (1) (3) (6)
C	5 levels of pre-stressed struts + jet-grout slab 3 m thick, from 17.5 m to 20.5 m in depth, executed prior to excavation (1) (3) (4) (6)
D	5 levels of pre-stressed struts + CSM transverse panels, 0.8 m thick, at 6 m intervals, from the surface to 30 m in depth, executed prior to excavation (1) (3) (5) (6)

Notes:
1 - Cross sectional area of the struts: 1st level: 233 cm^2; 2nd level: 306 cm^2; 3rd level: 335 cm^2; 4th level: 479 cm^2; 5th level: 306 cm^2;
2 - Effective strut stiffness = 0.5 x Theoretical strut stiffness;
3 - Effective strut stiffness = 0.8 x Theoretical strut stiffness;
Pre-stress: 1st level = 900 kN; 2nd - 5th levels = 3000 kN;
4 - $E_{\text{jet-grout}}$ = 0.8 GPa; jet-grout assumed as linear elastic;
5 - E_{CSM} = 0.8 GPa; CSM panels assumed as linear elastic;
6 - Soil conditions: soft clay: $\gamma = 18$ kN/ m^3; c_u (kPa) = $20 + 0.22\,\sigma'_{v0}$; $E_u = 400\,c_u$;
Miocene substratum: $\gamma = 21.5$ kN/ m^3; $c_u = 400$ kPa; $E_u = 400\,c_u$.

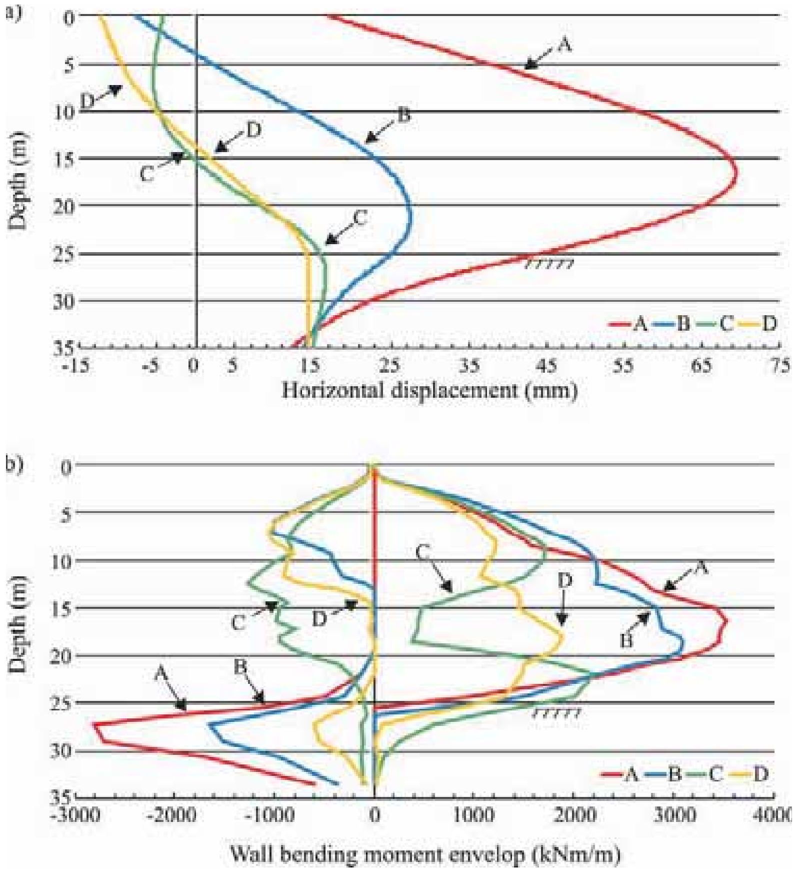


Figure 32 - Comparison of the results from the finite element analyses of Table 7 (in the vertical plane perpendicular to the wall where the results are maximum): a) final lateral wall displacements; b) wall bending moment envelopes.

under carefully controlled conditions in the field. However, the brief study just presented, perhaps allows the observation that the proposed technique seems an encouraging step towards the goal of *zero wall movements*, which – since we are dealing with impermeable walls – would assure null surface settlements during the excavation process.

It may be that progress with regard to movement control in the future will not follow the concrete idea presented above. However, in broad terms, the author is convinced that the right direction for solving this challenge seems to involve the combination of the two paramount technical advances described above: *not only to build the wall, but also to provide effective support to it, before the execution of the excavation*.

Nonetheless, what is absolutely certain is that deep excavations in soft ground will remain a challenge to our intelligence and to our imagination.

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Municipal Solid Waste Sanitary Landfill Compressibility Study with Linear Regression Application

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Abstract. This paper refers to a Municipal Solid Waste (MSW) compressibility study of waste disposed of in small scale sanitary landfills in the municipality of Presidente Lucena in the State of Rio Grande do Sul, Brazil. The research aims at the application and development of settlement prediction models based on settlement data collected on site. The studies were divided into the following stages: application of prediction models based on Soil Mechanics' classical concepts and the creation of a regression model, based on the physical and chemical landfill monitoring results, in order to estimate differential settlements. The results showed that the application of data collected at the monitored small scale landfills through classical settlement prediction models resulted in significant errors. However, the model created based upon the regression analysis, perhaps because it considered the specifics associated with disposal techniques in small landfills, was the most realistic in terms of settlement prediction, such that it is applicable to other similar systems, be it due to the characteristics of disposed waste, as well as to the employed operational details.
Keywords: urban solid waste, sanitary landfill, compressibility, settlement model, linear regression.

1. Introduction

Solid waste generation, as well as its impacts, is directly related to human cultural and technological evolution. Several authors (for instance, Zanta & Ferreira, 2003; Tillmann, 2003; Schneider *et al.*, 2004; Boff, 2005) report cultural, social and educational factors, number of inhabitants, activities carried forth by the population, technology and economic matters as influential factors that affect MSW production, as well as its physical, chemical and biological characteristics.

The Ministry of the Cities/Environmental Sanitation National Agency/Sanitary Sector Modernization Program (MCIDADES/SNSA/PMSS) (2008) has recently published a historical series that includes research on solid waste management in Brazil. Despite the fact that, for the year of 2006, the research has brought forth results from a sample of 48.8% of the Brazilian population corresponding to an urban population mainly from large cities, the final disposal result indicates that 16% of the units reported by managers are open dumps, 18% are controlled landfills, 5% are incineration units and 22% correspond to sanitary landfills.

Jucá (2003), describes a MSW landfill as an engineering workmanship intended for the disposal of such wastes, which undergo mass loss as a result of physical, chemical and biological processes. This phenomenon causes a reduction in the mass height of the disposed waste, known as settlement that is settlement.

Settlements and volume reduction of the deposited waste occur as a result of the transformation of its components through physical, chemical and biological processes resulting in gas emissions and leachate formation (Carvalho *et al.*, 2000). However, the quantification of these landfill geotechnical properties and settlement prediction in sanitary landfills is very complex due to factors such as: diversity, heterogeneity and waste decomposition processes, refuse type individual compressibility, as well as regional climate condition variations (Pereira, 2000; Carvalho *et al.*, 2000; Bowders *et al.*, 2000; Chen *et al.*, 2009).

Both the importance and need to understand landfill geotechnical characteristics may be justified by the imminent possibility to use these areas for waste relocation and/or environmental recovery, such as future reforestation projects, for instance. Park & Lee (2002) report that area usage, after sanitary landfill closure, is restricted mainly due to differential settlements as well as the generation of both leachate and gas emissions.

Among other advantages of settlement prediction is the simulation and/or establishment of applicable mathematical relations as they may be able to help project designers of these workmanships when it comes to the calculation of the space to be created by such phenomena, thus making it possible to discard more waste in the same area, maximizing landfill life cycle. Generally, in sanitary landfills, settlements reach nearly 25 to 30% of the theoretical landfill height (Gandolla *et al.*, 1994 and Santos, 1994).

Accordingly, Santos (1994) adds yet several advantages that point to the study of settlements in MSW landfills, such as: remediation of sanitary landfill or open dump areas with the establishment of parks, gardens, soccer fields, traffic routes and small buildings. These actions depend upon load capacity and settlements that such struc-

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tures may be able to withstand after their implementation. Additional advantages are: settlement evolution curves represent an auxiliary method in the monitoring of physical and chemical modifications and of MSW decomposition; in addition, they may be useful in the assessment and stability of landfill slopes.

Amorim & Bernardes (2007) report that settlement monitoring in MSW landfills is a phenomenon that determines landfill operational control and management procedures, besides the fact that it also affects future area occupation.

Following the same line of thought, Park *et al.* (2007), Liu *et al.* (2006) and Bowders *et al.* (2000) describe that understanding settlements throughout time is both important and critical to a an MSW landfill project, operation and management. Besides, it is essential to the project of landfill rehabilitation, and may be useful in the construction of parks, houses and roads. El-Fadel *et al.* (1999) supplements this statement by mentioning that this phenomenon is an integral part of landfill closure planning, as well as of area reuse.

Finally, an important aspect mentioned by Simões *et al.* (2005) refers to MSW landfill safety. Both horizontal and vertical landfill movement monitoring allow for the identification of movement pattern alterations in such a way that these variations may be indicative of instability problems, thus rendering a geotechnical assessment as important from both a legal and technical point of view.

Gandolla *et al.* (1994) also mentions this preoccupation with the safety element, such that settlement monitoring may be used in a way to improve landfill stability, as it mainly refers to cover layer inclination determination.

Taking all problems and needs pointed out to into consideration, the work to be presented refers to a compressibility study of MSW disposed of in small scale sanitary landfills in Presidente Lucena in the State of Rio Grande do Sul, Brazil, aimed at the application and development of a settlement prediction model.

2. Waste Compressibility

Conventional geotechnical engineering considers settlements as vertical soil deformations resulting from external load application or from its own weight. Both deformability and landfill deformation speed are influenced by waste gravimetric composition. According to Oliveira (1995), landfills with larger inert waste composition tend to be harder in comparison with landfills with a larger percentage of domestic solid waste (decomposing organic matter, plastic and paper). As for this statement, Park & Lee (2002) describe that settlement characteristics in MSW sanitary landfills are particular due to the considerable occurrence of such phenomenon as a result of organic waste decomposition, which lasts for a long time throughout the landfill's life cycle.

The compressibility mechanisms of MSW disposed of in sanitary landfills are described and conceptualized by several authors. According to Sowers (1973), Pereira (2000) and Carvalho *et al.* (2000), MSW compression mechanisms are: 1. Mechanical – particle structural collapse (distortion, bending, crushing and component reorientation); 2. Fine particle migration to empty spaces created by larger particles; 3. Physical and chemical changes – due to corrosion, oxidation and combustion; 4. Biochemical degradation – aerobic and anaerobic fermentation and decomposition processes and 5. Interaction – interaction of physical, chemical and biochemical processes.

On the other hand, Oliveira (1995) defines that sanitary landfill vertical deformations are associated to two periods: 1. Landfill construction period due to the increased load of its own weight, such that larger vertical deformations tend to occur during this period; 2. Landfill post-construction secondary deformations resulting from waste layer consolidation caused by pore expelled water due to landfill material components' deformation.

Finally, another sanitary landfill settlement description and classification is made by Jucá (2003) and Liu *et al.* (2004). For them, there are three types of MSW landfill settlements: immediate or initial, primary and secondary. According to the authors, initial compression occurs due to external pressure caused by compacting machines in the beginning of the waste disposal process. Primary settlements result from liquid and gases being expelled from the interior of the waste mass and occur in the first 30 days according to Wall & Zeiss (1995) *apud* Jucá (2003). Finally, secondary settlements refer exclusively to biodegradation, which can be influenced by humidity level and flow, as well as by buried waste composition.

According to Pereira (2000) and Carvalho *et al.* (2000), several factors affect settlement mechanisms such as: specific weight or void ratio, nutrient availability for microbiological growth, waste composition and moisture content, landfill height, overload, leachate level and fluctuation; operational and project details, in addition to environmental and climatic factors such as moisture content, rainfall, evaporation and temperature.

As a complement to what has been said, Melo *et al.* (2006) describes that MSW landfill settlement magnitude and speed are influenced by various physical, chemical and biological processes, being that the last one is responsible for the majority of this influence.

Pereira & Mañas (2001) monitored superficial settlements in order to evaluate immediate and primary settlements (installation of referential marks) and deep ones to evaluate secondary settlements (installation of a sliding micrometer) at the Valdemingómes sanitary landfill in Madrid, Spain. From the results it may be observed that for a waste disposal height equal to 18 m monitored for approximately 600 days, immediate settlement was measured at 0.067 m; primary at 0.314 m and secondary at 0.860 m, totalizing 1.241 m of measured settlement at the reference positions, which means a deformation of 6.89%. The authors also found that primary and secondary compression transition occurred within 100 monitoring days.

In another research, settlement monitoring of the first installed 2.0 m layer at the Columbia, Missouri sanitary landfill in the United States, which is operated without leachate recirculation, settlement was recorded at 0.3 m during 180 days of monitoring activities; in other words, 15% of total height (Bowders *et al.*, 2000).

The same author shows a study in the Victoria sanitary landfill in Australia, where settlements of 0.7 m were measured in a section where there was leachate recirculation at the portion where plates were placed on top of the landfill (approximate thickness height of 18 m). In the section without recirculation, settlement was approximately 0.5 m.

3. Msw Landfill Settlement Prediction Models

Due to the lack of specific models to determine MSW landfill compressibility, classical concepts of Soil Mechanics have been used with some adaptations, as is the case in the models presented by Sowers (1973), Bjarngard & Edgers (1990), Yen & Scanlon (1975), Gibson & Lo (1961) and Edil *et al.* (1990) *apud* Carvalho *et al.* (2000).

Equation (1) indicates the model presented by Sowers (1973), while Eq. (2) presents the proposals set forth by Bjarngard & Edgers (1990), both of which are used to predict sanitary landfill settlements.

$$S_{(t)} = \frac{H}{1+e_0}\left[C_c \log\frac{\sigma_0^{'} + \Delta\sigma}{\sigma_0^{'}} + C_\alpha \log\frac{t}{t_{(1)}}\right] \qquad (1)$$

$$\frac{S_{(t)}}{H} = C_C^{'} \log\frac{\sigma_0^{'} + \Delta\sigma}{\sigma_0^{'}} + C_{\alpha 1}^{'} \log\frac{t_{(2)}}{t_{(1)}} + C_{\alpha 2}^{'} \log\frac{t_{(3)}}{t_{(2)}} \qquad (2)$$

where $S_{(t)}$ = settlement in time t; H = initial layer thickness; e_0 = initial void ratio ; C_c = primary compression index; $C'_c = Cc/(1 + e_0)$ = primary compression index coefficient ; C_α = secondary compression index; σ_0 = initial vertical stress; $\Delta\sigma$ = increase in vertical stress; $t_{(t)}$ = time to complete initial compression; $t_{(2)}$ = time to complete intermediary compression; $t_{(3)}$ = ideal length of time to predict a settlement; $C'_{\alpha 1}/(1 + e_0)$ = intermediary secondary compression index; $C'_{\alpha 2} = C_{\alpha 2}/(1 + e_0)$ = intermediary secondary compression index, in the long run.

Several studies have been conducted in order to estimate essential coefficients, which aid in the application of settlement prediction models mentioned in the literature. Unfortunately, it is emphasized that they are reasonably complex to obtain; void ratio, as well as primary and secondary compression indexes are examples of this complexity.

Sowers (1973) determined the primary and secondary compression index according to waste characteristics. The values defined for primary compression were $0.15e_0$ (wastes containing little organic matter) and $0.55e_0$ (wastes with high levels of organic matter). The author obtained the following numbers for secondary compression index: $0.03e_0$ (unfavorable degradation conditions) and $0.09e_0$ (favorable degradation conditions).

It is so that Marques (2001) and Simões & Campos (1998) describe a series of research projects that estimate such indexes, which are presented in Tables 1 and 2.

In the Bandeirantes Sanitary Landfill study, Carvalho *et al.* (2000) determined, in the laboratory, these parameters described in Tables 1 and 2. The results indicated a variation in the primary compression index *Cc* from 0.56 to 0.92;

Table 1 - Settlement prediction parameters – Obtained in the laboratory.

Authors	Place	Primary compression		Secondary compression	
		Index (*Cc*)	Coefficient (*C'c*)	Index (*C*α)	Coefficient (*C'*α)
Sowers (1973)	-	$0.15e_0$ to $0.55e_0$	-	$0.03e_0$ to $0.09e_0$	-
Rao *et al.* (1977)	-	-	0.160 to 0.235	-	0.015 to 0.045
Sargunan *et al.* (1986)	-	0.44		0.0036 to 0.005	-
Gabr & Valero (1995)	-	0.4 to 0.9	0.15 to 0.22	0.03 to 0.09	-
Landva & Clark (1984, 1986, 1990) - Sanitary landfills:	Kingston	-	0.17	-	0.0210
	Edmonton	-	0.35	-	0.0180
	Hantsport	-	0.22	-	0.0280
	Ottawa	-	0.21	-	0.0070
	Edmundston	-	0.36	-	0.0020
	Stolport	-	-	-	0.0150

Table 2 - Settlement prediction parameters – Obtained in experimental cells.

Authors	Primary compression		Secondary compression	
	Index (Cc)	Coefficient ($C'c$)	Index ($C\alpha$)	Coefficient ($C'\alpha$)
Espinace *et al.* (1991)	0.13 to 0.40	-	0.14 to 0.59	-
Cartier & Baldit (1983)	0.54 to 0.90	-	0.30 to 0.55	-
Wall & Zeiss (1995)	-	0.21 to 0.25	-	0.033 to 0.056

$C'c$ from 0.175 to 0.229; secondary compression $C\alpha$ from 0.0213 to 0.0442; and $C'\alpha$ from 0.0105 to 0.016.

Marques (2001) reports that sanitary landfill settlements are normally estimated by considering a mechanism of one-dimensional consolidation. The application of settlement estimate models applied to wastes is complex. It is considered that the primary and secondary compression indexes are a function of the void ratio, whose value is variable and difficult to obtain from waste. There occur significant variations in both the primary and secondary compression indexes due to stresses produced in sanitary landfills, and primary settlements are a function of the effective stresses, which depend upon waste specific weight and leachate levels, both of which are parameters equally as hard to assess.

Due to variability, heterogeneity, individual compressibility and MSW degradation, Marques (2001) describes that the determination of the adequate settlement prediction model, just like the calculation of its parameters, presents itself as a limiting factor in sanitary landfill deformability analysis.

According to Liu *et al.* (2006), sanitary landfill estimate models can be divided into the following categories: 1. Consolidated Models: Terzaghi's theory applied to soil settlements is adapted to both primary and secondary settlement calculation; 2. Rheologic Model: waste compression behavior is modeled by using a rheologic model; for instance, in the viscoelastic model by Gibson & Lo (1961), primary and secondary compressibility is simulated by springs and suspension; 3. Biodegradation Model: organic matter gradual biodegradation is considered in model formulation; 4. Regression Model: several common functions (for instance, logarithmic, hyperbolic, *Creep* exponential model, bi-linear, multi-linear) are used to simulate settlements in sanitary landfills. The parameters of these functions are obtained through landfill settlement data.

Of these categories, the most utilized is the consolidated theory; however, there are many fundamental discrepancies between sanitary landfill settlement mechanisms and soil settlements. In the consolidated theory, the condition that soil is saturated is assumed. Thus, settlement is attributed to excess water dissipation in the pores, while secondary compression is responsible for a small portion of the total settlement. Nevertheless, sanitary landfill wastes are not saturated and organic matter degradation produces a significant amount of gas emissions, causing a high consolidation level (Liu *et al.*, 2006).

As for linear regression models, Liu *et al.* (2006) additionally report that this method is also very much used to predict settlements. Regression analysis aims at finding an appropriate coefficient to reach the best possible result; however, according to the authors, this method does not take settlement physical mechanisms into consideration. In addition, the authors describe that although biodegradation models consider the decomposition process associated to secondary compression, they fail to consider mechanical compression.

When applying the Bjarngard & Edgers Model (1990) to monitored settlement data at the Bandeirantes sanitary landfill in São Paulo, Brazil (7 year monitoring period and disposal variation height ranging from 26.3 m to 58.6 m), Carvalho *et al.* (2000) found secondary compression results with a variation of 12% when compared to the initial landfill height, which means settlements varying from 3 m to 7 m. Similar values were found, using the same data, by applying the model proposed by Gibson & Lo (1961). Thus, for the data studied by the authors, secondary settlements may be satisfactorily modeled by any one of the used models. The authors concluded that, although soil mechanics concepts are not entirely appropriate to estimate settlements, they have been the starting point. In addition, the model that was conceived primarily to measure secondary compressibility in peat (Gibson & Lo, 1961) seemed to closely reproduce the results obtained on site.

Armed with settlement monitoring data from the Bandeirantes sanitary landfill, Marques (2001) applied a series of prediction models described in the literature. The results showed that the Yen & Scalon Models (1975) and the logarithmic functions proposed – Models by Yen & Scalon (1975) and Ling *et al.* (1998) - are not recommended for use to predict settlements with the studied data. On the other hand, models based on the hyperbolic proposals by Ling *et al.* (1998) and Gibson & Lo (1961) reproduced settlement curves versus time quite well and, with a few adjustments, can be recommended for use. Proposals with models based on potency functions – Edil *et al.* (1990), Model by Edgers *et al.* (1992), Bjarngard & Edgers (1990) and Sowers (1973) did not prove to be as precise, although they are recommended for use after a few model adjustments. The author concluded that, despite the satisfactory performance of some models (based on simple mathematical formulas, with parameters and coefficients without physical meaning that simply aim at adjusting points on a curve), they must be avoided or used with reservation. Another aspect is the consideration that the sanitary landfill is a solid piece, such that models disregard the entire disposal sequence and the compression processes that act differently on each buried layer.

Park & Lee (2002) applied the biological model in order to predict long run settlements to data obtained from settlement monitoring in seven lysimeters and sanitary landfills of several ages. The authors divided the landfills in three groups: new (few years of operation), middle-aged (approximately 10 years) and old (up to 25 years). Results showed that the biological deformation estimate for new landfills was estimated to be between 11% and 25%, such that the entire settlement shall occur between 10 to 20 years. For landfills that are between 2 and 10 years old, biological deformation total quantity is higher depending on the age of the landfill, such that a full long run settlement rarely occurs prior to reaching 20 years of age.

Jucá (2003) applied the settlement models developed by Sowers (1973) and Gandolla *et al.* (1992) in Muribeca landfill (Recife, Brazil). Analyzing the results, the author concluded that: 1) due to cell age (*C*1 and *C*2 - approximately 18 years old) and consequential low organic matter level, settlements occurred exclusively due to secondary settlements; 2) both settlement measurements varying from 122 mm to 778 mm (for a monitoring period of 17 months) and their speed determination, which varied from 286 mm/day to 2381 mm/day, were considered small by the author due to the low microbiologic activity and final methane generation stage; 3) observed that the overload (placement of 30 cm of soil), as well as liquid and gas drainage (opening of an access channel) increase settlement speed; and 4) the models by Sowers (1973) and Gandolla *et al.* (1994) yielded results with similar values to those measured on site.

In another study, Park *et al.* (2007) classified fifteen MSW sanitary landfills in three different categories according to settlement magnitude and the age at which landfill closure occurs. Later, the authors applied several models used in settlement prediction (Gibson & Lo Model (1961), Hyperbolic function Model by Ling *et al.* (1998), Bjarngard & Edgers (1990) Model, Park & Lee Biological Model (1997; 2002), among others) to settlement data collected on site at these landfills. In the results, the authors verified that for sanitary landfills Type I (young landfills – below 3 years old), settlement estimate was significant for all models, except for the *Creep* exponential model. For Type II (young landfills like the ones in Type I, only that component substantial decomposition and biodegradation was observed), the tested models are appropriate to estimate long run settlements, except for the *Creep* Exponential Model and the Bjarngard & Edgers Model. Finally, for Type III (landfills that range from 8 to 25 years old), all models, except for the *Creep* Exponential Model, adequately estimate long run settlements. Thus, there was a similarity when comparing the application of settlement prediction models studied and the models based on the settlement data collected in the different types of landfills.

Recently, a model based on organic matter degradation was developed by Amorim & Bernardes (2007). In order to test the mathematical formulas, a model adjustment was made with monitoring data collected for forty months in settlements that occurred in an experimental cell built in the sanitary landfill in Brasilia, Brazil. A numeric simulation proved to be satisfactory in comparison to data collected on site.

4. Methodology

This work's methodology followed the stages described in Fig. 1.

4.1. Stage 1: Study area – data collection

Monitoring of the Superficial Settlements was conducted by using six stakes and one reference point through the Simple Geometric Leveling Method with one level and a centimeter rule (precision of 0.015 m), for a period of two years, in small scale sanitary landfills in the municipality of Presidente Lucena in the State of Rio Grande do Sul, Brazil. The municipality has 2100 inhabitants and its economy is basically based on agriculture, although is also has some small shoe making manufacturing companies, fruit processing, wood and textiles.

Data were collected in three different cells (T1 and T2), whose dimensions were approximately 4.0 x 5.3 x 2.5 m. The disposed buried MSW composition corresponds to 50% food scrap, 2% paper, 14% plastic and 34% other materials, mainly biological contaminants (toilet paper, disposable diapers and other sanitary and personal hygiene refuse).

The final landfill cover consisted of local soil (20 cm), being that intermediary coverings were not applied during solid waste disposition. Only T1 received a cover with a PVC membrane prior to the mineral superior layer with the proposal to reduce the entry of rain. High Density Polyethylene (HDPE) 0.8 mm thick geomembranes were used to minimize percolation on the sides and at the bottom.

The following parameters were weekly measured in the generated leachate in the three cells: pH, total solids (TS), total suspended solids (TSS), volatile suspended solids (VSS), chemical oxygen demand (COD), total nitrogen (TN), ammonia nitrogen (AN), phosphorus (PO_4), chrome (Cr), iron (Fe), lead (Pb), cadmium (Cd) and zinc (Zn). All analyses were made according to APHA (1995).

Similarly, differential settlement measurements were taken every week, in addition to counting of total anaerobic micro-organisms. Regional climate conditions data such as rainfall, relative air humidity and temperature were also

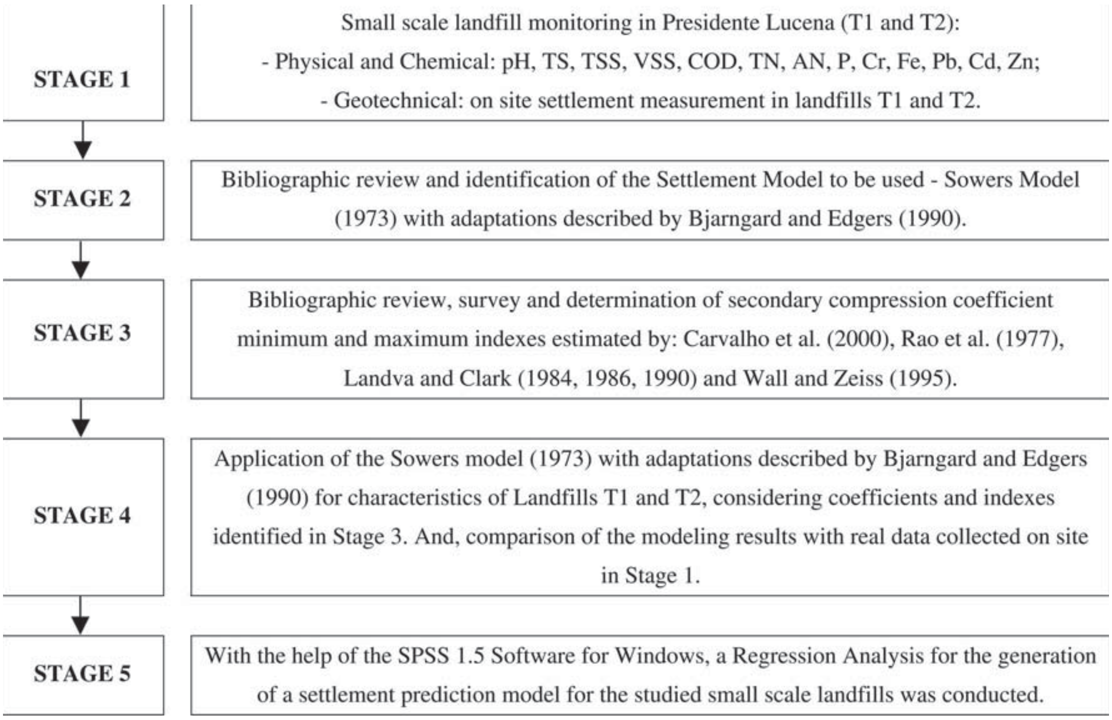

Figure 1 - Research methodological stages.

collected and tabulated. The moment at which the trench was closed was considered as the initial time to monitor settlements ($t = 0$).

4.2. Stages 2, 3 and 4: Application of settlement prediction model in MSW landfills

In order to test sanitary landfill settlement prediction existing models mentioned in the consulted bibliography and, aiming at comparing the generated/estimated curve with the actual data collected on site, data measured *in situ* at the small scale sanitary landfills T1 and T2 were applied to the Sowers Model (1973) with adaptations described by Bjarngard & Edgers (1990) - Eq. (2). This model was chosen because, according to Liu *et al.* (2006), it is the one that is mostly used in settlement prediction.

In this stage of the research, the operation method of landfills T1 and T2, waste characteristics and local geotechnical conditions were taken into consideration, as follows:

• Initial waste layer thickness (trenches' depth) = 2.5 m;

• Waste density in the landfill = 12 tons of MSW were disposed of in the landfill whose volume was measured at 53 m^3. Waste density was determined to be 2.21 kN/m^3;

• Because there was no compression due to the use of machines, initial stress (σ'_0) was considered at 5.55 kN/m^2; in other words, the actual landfill weight alone;

• Increased stress ($\Delta\sigma'_0$) is the actual landfill weight calculated at 5.55 kN/m^2;

• Because waste compaction did not occur either during disposal or in the preparation of the final landfill cover layer, it was assumed that there was no primary compression. Thus, for this paper, the primary compression index coefficient ($C'c$) was considered to be equal to zero;

• Time of 133 days was considered as the period to complete intermediary compression in landfill T1, being that the very same period was used for landfill T2. This parameter was obtained by observing a sudden change in the line angle in the settlement graphs, suggesting a modification in settlement speed. The total settlement monitoring time at each landfill was as follows: T1 = 441 days and T2 = days;

In addition to these characteristics, both primary[(*)] and secondary compression coefficients identified in Table 3 were tested, in order to apply the model expressed in Eq. (2), as follows:

4.3. Stage 5: Data processing and linear regression analysis

From the actual landfill monitored settlement data, corrections were applied, such that values were estimated according to the evolution trend of each parameter. A correlation matrix was created to cross-analyze the several monitored parameters, identifying those with more significant correlations to be used in the creation of the prediction model.

From the data collected on site in T1, a Regression Analysis was made, making it possible to generate three

Table 3 - Primary and secondary compression coefficients used.

Author	Primary compression Coefficient ($C'c$)*		Secondary compression coefficient ($C'\alpha$)	
	Minimum	Maximum	Minimum	Maximum
Rao *et al.* (1977)	0.160	0.235	0.015	0.045
Landva & Clark (1984, 1986, 1990)	0.170	0.360	0.002	0.028
Wall & Zeiss (1995)	0.210	0.250	0.033	0.056
Carvalho *et al.* (2000)	0.175	0.229	0.0105	0.0116

(*) In the case of small scale landfills, as is the case in this paper, the primary compression coefficient is equal to zero.

Models. The Software SPSS 1.5 for Windows was used in the study, which generated model coefficients, as well as the significance of each one of them.

Armed with calculated coefficients, there was an attempt to apply the model to data collected on site in Landfills T2 and T3, in addition to verifying the applicability of the model to other landfills with similar characteristics.

5. Results and Discussion

5.1. Stage 1: Small scale landfill monitoring (T1 and T2)

Leachate monitoring results from the sanitary landfills in Presidente Lucena are presented in Table 4. Daily rainfall measured on site during the experiments varied from 0 to 59.7 mm and environmental temperature ranged between 5.1 and 34.7 °C.

Through monitoring activities, it was proven that solid waste has recently been disposed of as illustrated by organic matter yet to be degraded, as well as leachate high nutrient concentration. Heavy metals were identified according to ranges presented in Table 4.

Through the monitoring of both physical and chemical parameters, as well as settlements measured in landfills T1 and T2, the relationship between organic matter decomposition and landfill consolidation as described by the authors was delineated, thus confirming biochemical degradation as one of the mechanisms responsible for the compressibility of MSW disposed in sanitary lanfills. Figure 2 depicts a graph illustrating the relationship between COD and settlements measured in the T1 landfill in Presidente Lucena throughout time.

5.2. Stages 2, 3 and 4: Sower Model Application (1973) with adaptations described by Bjarngard & Edgers (1990)

The Sowers Model (1973) with adaptations described by Bjarngard & Edgers (1990) was applied to landfills T1 and T2.

Figure 3 demonstrates the graph that was generated from the input of data collected in landfill T1 in Presidente Lucena to the Sowers Model (1973) with adaptations described by Bjarngard & Edgers (1990). Both primary (equal to zero) and secondary compressibility coefficients, as well as data previously mentioned in Table 3 of this article were used. In the same graph, it is possible to observe the curve containing the actual settlement measurements taken on site.

By applying Eqs. (1) and (2) to the maximum limits of secondary compression, a complete settlement in T1 was obtained, with 441 monitoring days, equal to 0.021 m (coefficients by Carvalho *et al.*, 2000); 0.059 m (coefficients by Rao *et al.*, 1977); 0.036 m (coefficients by Landva & Clark, 1984, 1986, 1990) and 0.073 m (coefficients by Wall & Zeiss, 1995). Considering that the actual settlement measured in T1 was 0.118 m (corresponding to approximately 5% of landfill depth), the identified differences between this actual settlement and those that were estimated vary between 0.045 m and 0.097 m, which means, 38% and 82% as related to the actual settlement measured on site.

For landfill T2, the numbers varied from 0.019 m to 0.057 m, thus presenting errors of 26% and 79%, respectively.

Table 4 - Leachate physical and chemical analyses' results from landfills T1 and T2.

Parameters	T1		T2	
	Minimum value	Maximum value	Minimum value	Maximum value
pH	6.1	7.4	6.4	7.5
TS	1165.0	7096.0	1625.5	5763.0
TSS	63.5	820.0	82.0	1000.0
VSS	25.5	440.0	36.0	880.0
COD	152.0	5700.4	310.8	1574.0
P	1.3	401.0	3.5	265.3
TN	26.4	195.7	41.3	373.4
AN	23.3	140.8	34.6	285.4
Cr	0.1	0.4	0.2	0.7
Fe	32.1	78.9	44.0	72.8
Zn	0.1	1.9	0.1	0.3
Cd	0.0	0.9	0.0	0.0
Pb	0.1	0.9	0.5	1.0

Unit: mg/L except for pH, which has no dimension.

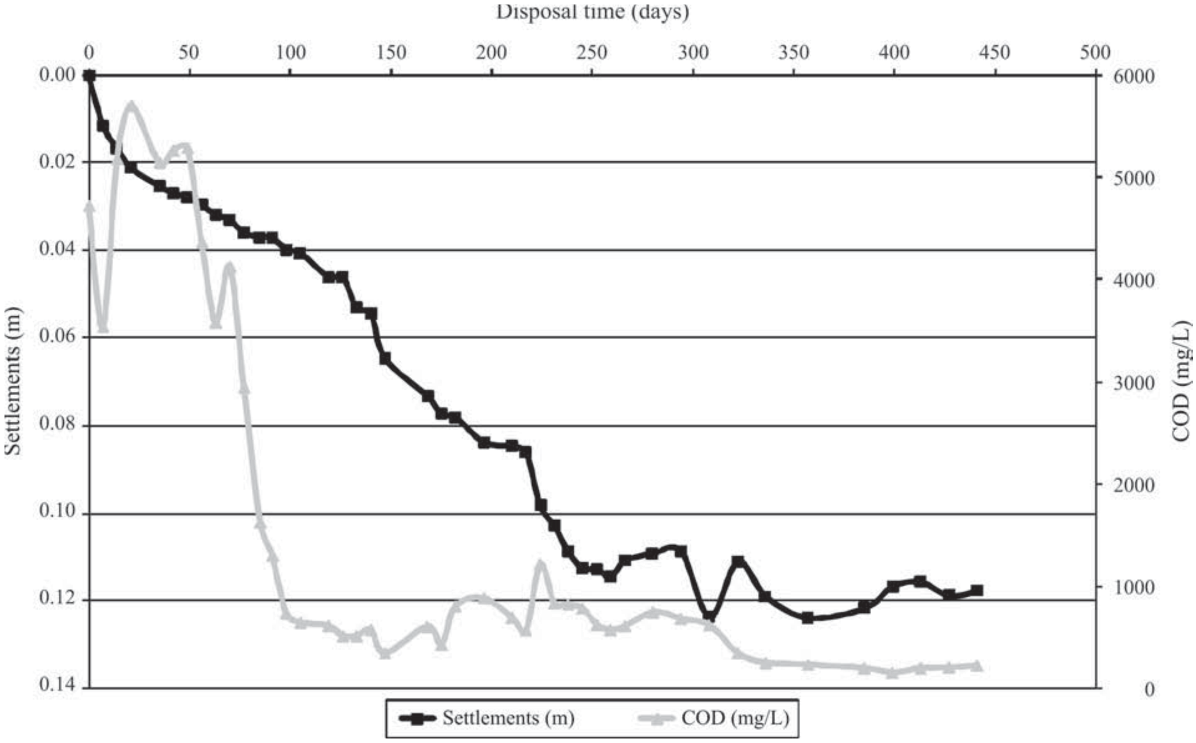

Figure 2 - Actual Measured Settlements and COD x Disposal Time – T1 Landfill.

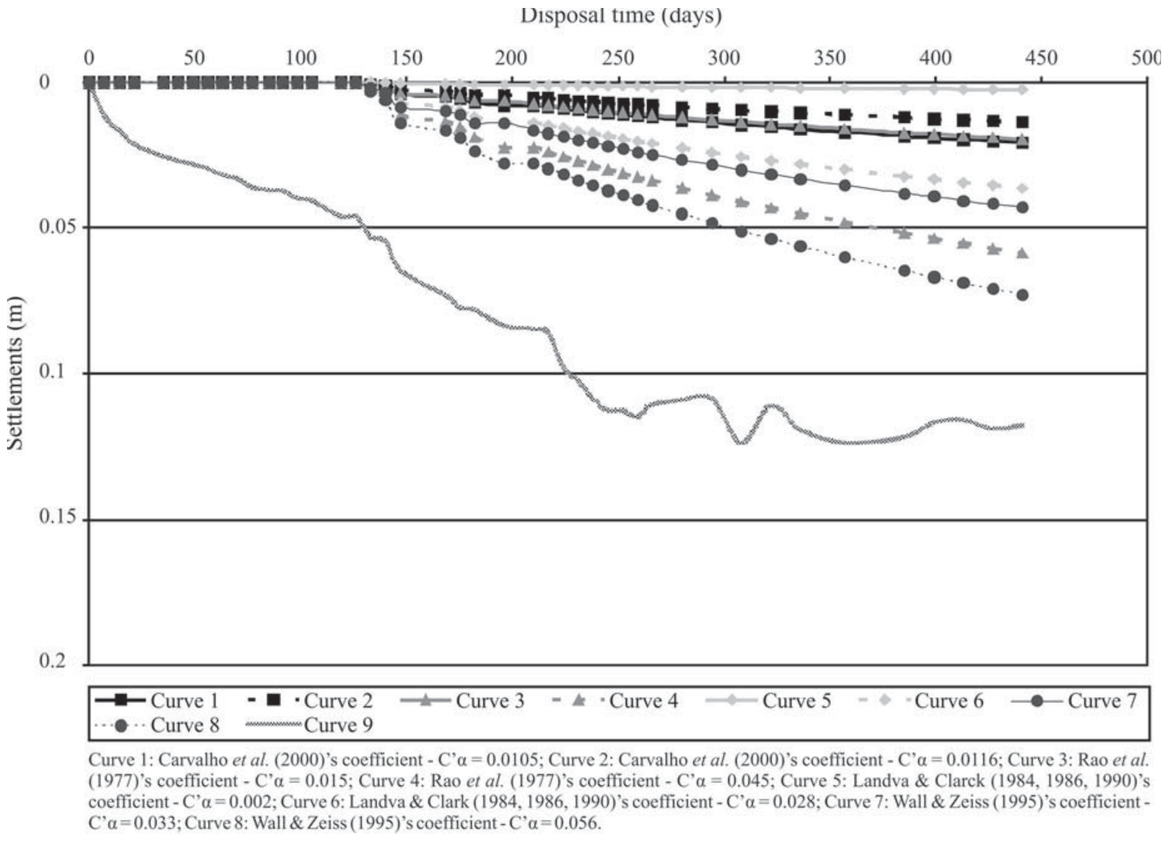

Figure 3 - Settlement data collected on site in T1 and settlement prediction applying the Bjarngard & Edgers Model (1990).

On the other hand, by using secondary compression minimum coefficients, for 441 monitoring days, total T1 settlement was 0.014 m (coefficients by Carvalho *et al.*, 2000); 0.020 m (coefficients by Rao *et al.*, 1977); 0.003 m (coefficients by Landva & Clark, 1984, 1986, 1990) and 0.043 m (coefficients by Wall & Zeiss, 1995). When comparing the actual settlement to the estimate by applying the models found in the literature, it was found that errors varied from 0.075 m to 0.115 m, corresponding to approximately 64% and 98% of the total landfill settlement, respectively.

Similarly, for landfill T2, the errors varied from 0.041 m to 0.070 m, thus showing errors of 56% and 97%, respectively.

According to these analyses and the comparison with Fig. 3, with the model application, the error is higher than that which was measured on site. That may be explained due to the fact that the Presidente Lucena landfill has inferior dimensions than MSW sanitary landfills commonly found in the country (this is the case in Carvalho *et al.*, 2000 in the Bandeirantes Landfill), thus making it possible to present different primary and secondary compression coefficients from those mentioned in this research, which are then probably similar to the inferior limit of the coefficients presented in Fig. 3.

In addition, the majority of the coefficients used from the literature were determined from data from sanitary landfills in developed countries. Therefore, there is a great difference in the gravimetric composition of the disposed wastes, mainly related to the quantity of organic matter, which, in these cases, is known to be lower.

Another reason for the errors found in the model may be explained by using the work developed by Liu *et al.* (2006), which asserts that this model considers that MSW is saturated (the mean moisture content level of the wastes disposed of in sanitary landfills is approximately 60%) and that organic matter degradation produces a significant amount of gas, thus causing an increase in the consolidation degree.

Hence, this research diverges from the studies conducted by Carvalho *et al.* (2000), Park *et al.* (2007), Jucá (2003) and Bowders *et al.* (2000) in what relates to the application of such models. On the other hand, Marques (2001) demonstrated that by utilizing the model by both Bjarngard & Edgers (1990) and Sowers (1973), although presenting not exactly accurate prediction results, they were actually recommended for use in settlement prediction, after some model adjustments.

5.3. Stage 5: Statistical model – T1 landfill

Various data associated to on site T1 monitoring in Presidente Lucena were inputted to Software SPSS 1.5 for Windows, thus generating several models. According to the bibliography, MSW landfills are characterized by a mass of heterogeneous materials with diverse physical, chemical and biological behavior. Therefore, a data variation coefficient of 30% was considered to generate the Presidente Lucena Sustainable Landfill Settlement Model.

One of the considered variables, in addition to the usual environmental monitoring parameters (COD, total solids, nutrients (N and P)) was the landfill leachate recirculation. This is an operational alternative used in Brazil, which is considered to be a leachate treatment method. It must be emphasized that this method must be employed carefully when used in regions or seasons with high rainfall in order to avoid slope stability problems.

Five models were generated, such that models 1, 2 and 3 are linear and find themselves presented in Tables 5, 6 and 7. Two other exponential models were also tested and the obtained results expressed little adherence to the actual data. These adjustments were not considered in this analysis.

Monitoring data from landfill T1 were applied to models 1, 2 and 3 and compared to actual settlements. The errors found in each model as related to actual settlement are presented in Table 8 and were calculated by using the Minimum Square Method. Figure 4 presents the models in comparison with the actual measured settlement.

In analyzing data from Table 8 and Fig. 4, Model 2 obtained the best results, such that it was used in this re-

Table 5 - Statistical Model 1 - based on the dependent variable "Settlement".

Variables	Coefficients	Significance
Constant	-0.0149576	0.3328650
Time	0.0002990	0.0000000
P	0.0000415	0.0287630
Av. env. temp.	0.0005500	0.0850700
Leachate recirculation	0.0182998	0.0000059
COD/TN	-0.0001339	0.3436450
TN/P	0.0005772	0.1784760
TS	0.0000035	0.2235130

R^2 = 0.958. Av. env. temp.: Average environmental temperature.

Table 6 - Statistical Model 2 - based on the dependent variable "Settlement".

Variables	Coefficients	Significance
Constant	-0.1359486	0.0035150
Time	0.0002756	0.0000000
P	0.0000310	0.0754610
Leachate recirculation	0.0173660	0.0000024
TN/P	0.0005716	0.1458620
pH	0.0220027	0.0016710

R^2 = 0.961.

Table 7 - Statistical Model 3 - based on the dependent variable "Settlement".

Variables	Coefficients	Significance
Constant	-0.1327638	0.0182800
Time	0.0002948	0.0000000
P	0.0000198	0.2224990
Leachate recirculation	0.0159054	0.0000122
pH	0.0202842	0.0163770
COD	0.0000011	0.3989860
Av. env. temp.	0.0003281	0.2974790

R^2 = 0.961. Av. env. temp.: Average environmental temperature.

Table 8 - Theoretical Models' Estimated Error as compared to site data from landfill T1.

Models	Sum of model errors
Model 1	0.002771 m
Model 2	0.002588 m
Model 3	0.002607 m

search project to estimate settlement prediction in Presidente Lucena's sanitary landfills. The mean error obtained with the application of the statistical model was 2% (0.002588 m). This error corresponds to the difference during the entire monitoring period and not just to the last point (time = 441 days), indicating that leachate quality monitoring data applied to the proposed prediction model are the ones that must be used. It may be observed that if only the last point were evaluated, the difference between settlement measured in 441 days in landfill T1 was 0.118 m and the estimated value was 0.131 m, presenting an error of 11%. The obtained Model 2 is indicated in Eq. (3):

$$S = -0.1359486 + 0.0002756A + 0.0000310B + 0.0173660C + 0.0005716D + 0.0220027E \quad (3)$$

where S = Settlement; A = time (days); B = Phosphorus (mg/L); C = Leachate recirculation; D = Total Nitrogen (mg/L) / Phosphorus (mg/L); E = pH.

When comparing the obtained results to the model application presented in Eqs. (1) and (2) (errors from 38% to 82% and from 64% to 98% for the minimum and maximum compression coefficient, respectively), it may be observed that the prediction error in the statistical model was smaller. Even the best model results using secondary compression indexes by Wall & Zeiss (1995) resulted in a 38% error.

Another point that may be mentioned is that the obtained results confirm Liu *et al.* (2006), who report that larger errors may exist when the linear regression method is applied to landfills, since these authors did not consider settlement physical mechanisms, which is not the case here (small scale landfills, because of their simplified operation without intense compaction, do not need to consider such mechanisms).

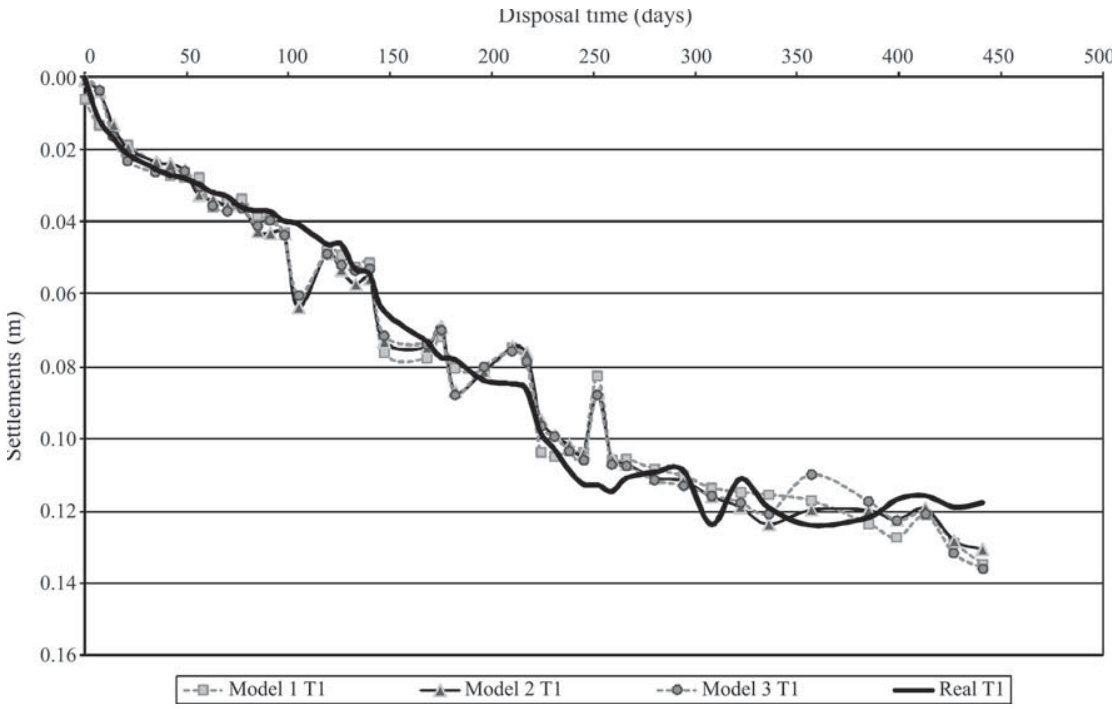

Figure 4 - Estimated Models' Comparison with actual settlement measured in T1.

5.4. Stage 5: Statistical Model 2 - Application generated with data from T1 for small scale landfill T2

The application of model 2 to the on site monitored data from landfill T2 showed the possibility to use the same generated model. After 322 monitoring days at T2 (on site data collection), the mean settlement reached 0.072 m, such that the model calculated result reached a settlement of 0.103 m (difference between actual and estimate equal to 0.031 m or an error of 43% as related to the last measured point). By applying the Minimum Square Method, the error was 16% during the entire period.

5.5. Stage 5: Statistical Model 2 - Application generated with data from T1 for Catas Altas, Minas Gerais, Brazil – landfill

In order to verify the applicability of the generated Linear Regression Model, data from research developed simultaneously to this one in PROSAB (Basic Sanitation Research Program) research network were used. Data (reduced in number – monitoring time of 260 days, with eighteen settlement measurements, which employed a simplified on site determination method) were handed over by UFMG (State of Minas Gerais Federal University) Institutional Coordinator (Lange, 2001).

Statistical model 2 was applied to data collected on site at the small scale sanitary landfill operated by UFMG in the municipality of Catas Altas in the State of Minas Gerais. This landfill was operated in a similar fashion to Presidente Lucena landfill in the State of Rio Grande do Sul and the Catas Altas population generates waste with similar characteristics to the waste generated in the southern city.

Results showed a very significant error. For 260 monitoring days, settlement measured on site was 0.030 m, such that the model generated results presented a settlement of 0.137 m; therefore, a difference of 10.7 cm (356%).

Conclusions

Settlement prediction models seen in the literature proved to be adequate for landfills, according to previous confirmation found in some studies. However, it is noted that such models are generic, based in Soil Mechanics studies, which fail to take some MSW specifics into consideration.

The application of these models to settlement data measured on site at small scale landfills in Presidente Lucena in the State of Rio Grande do Sul reveals significant errors, when comparing real data measured on site to predicted results, ranging from 38% to 98%.

For the case of settlement prediction in small scale landfills that operate without mechanical waste compaction both during disposal and in the preparation of the final cover layer, it is suggested that the primary compression index coefficient ($C'c$) be equal to zero.

The model generated from the statistical analysis (linear regression) proved to be more adequate in terms of prediction, showing an error of 2%.

Some conclusions can be perceived in the use of the generated statistical model:

• The generated model is also useful in estimating parameters that cannot be analyzed in the laboratory due to: malfunctioning equipment, lack of equipment, days without on site data collection, difficulties or operation costs;

• T1 model use in the other Presidente Lucena landfill (T2) showed that it is applicable to the same MSW disposal conditions;

• As for Catas Altas, it may be noted that, even though this landfill has similar characteristics to the one in Presidente Lucena, climatic differences, physical and chemical leachate parameters used in the model and equipment used in settlement monitoring may have influenced the found error; thus, in these cases, the employment of consolidated models is suggested.

The elaboration of a specific model to estimate sanitary landfill settlements, taking MSW specifics into consideration, is not an easy task to be accomplished. The use of the presented Regression Model considered such specifics and presented less significant errors in comparison to the application of on site data to the Sowers Model (1973) with adaptations described by Bjarngard & Edgers (1990).

In addition, the generated regression model is more realistic in terms of settlement prediction, although presenting monitoring time interval limits between 0 and 441 days and application only for small scale landfills such as the ones in Presidente Lucena.

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Symbol List

$S_{(t)}$ = settlement in time t
H = initial layer thickness
e_0 = initial void ratio
C_c = primary compression index
$C'c = Cc/(1 + e\, e_0)$ = primary compression index coefficient
$C\alpha$ = secondary compression index
σ_0 = initial vertical stress
$\Delta\sigma$ = increase in vertical stress
$t_{(1)}$ = time to complete initial compression
$t_{(2)}$ = time to complete intermediary compression
$t_{(3)}$ = ideal length of time to predict a settlement
$C'\alpha_{(1)}/(1 + e_0)$ = intermediary secondary compression index
$C'\alpha_{(2)} = C\alpha_{(2)}/(1 + e_0)$ = intermediary secondary compression index, in the long run
COD = Chemical Oxygen Demand
TN = Total Nitrogen
P = Phosphorus
TS = Total Solids

Time-Dependent Behaviour of a Shallow Tunnel in Overconsolidated Clay

Mafalda Lopes Laranjo, Jorge Almeida e Sousa, Paulo Venda Oliveira

Abstract. The presence of groundwater in clayey soils is known to affect its time-dependent behaviour. This paper presents some of the results of a parametric study performed in order to evaluate the influence of the construction technique and the soil/lining relative permeability, in the time-dependent behaviour of a shallow tunnel in overconsolidated clay. The excess pore pressures generated by the excavation are analysed and the evolution of the stress-paths with time around the tunnel section is presented. The movements induced on the soil surface and in depth as well as the loads acting on the support immediately after construction and in long-term conditions are analysed. A circular tunnel excavated at a depth of 20 m was considered and the analysis was performed using a two-dimensional finite-element code that included the MIT-E3 constitutive model.

Keywords: tunnels, overconsolidated clay, MIT-E3, consolidation.

1. Introduction

In clayey soils the changes in total stresses that occur when a shallow tunnel is excavated are immediately reflected not only in changes on the effective stresses but also on the generation of excess pore water pressures. The dissipation of these excess pore water pressures is time dependent and so are the variations on the stress conditions around the tunnel. As a result, the loads acting on the tunnel lining, and the displacements induced on the surrounding soil are also time dependent.

The relevance of this time dependency is associated with the magnitude and the distribution of the excess pore water pressures induced by the excavation and the final equilibrium conditions (Almeida e Sousa, 1998).

The magnitude and the distribution of the excess pore water pressures are largely dependent on the construction method. When using earth balance shields, excess pore pressures are usually positive. This occurs because the applied pressures at the front are greater than the in situ stresses (Yi *et al.*, 1993) and the tail void grouted (Almeida e Sousa, 1998).

In sequential construction methods, such as NATM, in which the excavation is performed allowing for a significant stress relief, the results of numerical analysis (Schmidt, 1989; Mair & Taylor, 1993), of centrifugal models (Mair, 1979) and field observation (New & Bowers, 1994) show that the magnitude and the distribution of the excess pore water pressures are essentially dependent on the stress relief allowed in the front and on the soil type (its history and stress path).

In overconsolidated clays, the reduction of the mean stress and the increase of the shear stress that derive from the excavation, lead to pore water pressures decrease. This decrease will be more significant as the overconsolidation ratio increases and the stress relief on the cutting face rises.

On the other hand, when a normally-consolidated clay is sheared under undrained conditions, pore pressures will increase, contrary to what derives from the reduction of the normal mean stress. In this case, pore pressures will increase or decrease depending on the relative significance of the changes mentioned above. When the stress relief is very significant, the reduction of the mean normal stress near the tunnel section will tend to surpass the increase of the shear stress, and the pore pressures are likely to diminish.

The final equilibrium pore pressures are mainly dependent on the relative permeability between the soil and the support. In fact, if the tunnel is waterproof-type, the dissipation of excess pore pressures will occur in order to re-establish the hydrostatic initial conditions on the ground. On the other hand, if the soil and the support have similar permeability, the tunnel will act as a drain, (Ward & Pender, 1981; Lee & Nam, 2001) and the final pore water pressures will be lower than those initially in situ. This seepage into the tunnel section will continue with the decrease of the water content of the soil and the increase of surface and subsurface settlements (Lopes, 2004).

This paper presents the results of a parametric study concerning the stress paths around the tunnel section immediately after construction and during consolidation, the excess pore pressures generated by the excavation, the deformations induced at the surface and in depth and the loads acting on the tunnel lining.

The analysis was carried out using a two-dimensional finite element code which allows for the consideration of elasto-plastic models with coupled consolidation and includes, among others, the MIT-E3 constitutive model (Venda Oliveira, 2000).

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2. The Mit-E3 Model

2.1. General

The MIT-E3 constitutive model results from an evolution of the MIT-E1 and MIT-E2, initially developed at the Massachusetts Institute of Technology to simulate the behaviour of normally consolidated clays. In 1987, Whittle developed the MIT-E3 constitutive model, which enables to accurately reproduce the behaviour of saturated overconsolidated clays (OCR < 8) subjected to cyclic loading (Whittle & Kavvadas, 1994).

The MIT-E3 model links the perfectly hysteretic and bounding surface plasticity formulations together with an elasto-plastic model which describes anisotropic properties of K_0-normally consolidated clays (Whittle, 1993). This model contains a number of assumptions concerning the behaviour of overconsolidated clays, namely:

1. - the overconsolidation ratio (OCR) is not sufficient to accurately describe the behaviour of overconsolidated clays; additional information on the loading history is also required to distinguish between unloading and reloading at a particular overconsolidated stress state;

2. - a load cycle always involves some plastic strains so that there is no purely reversible range of behaviour; the perfectly hysteretic model determines the stiffness and non-linearity of the stress-strain response immediately after a load reversal;

3. - the inclusion of plastic strains, using bounding surface plasticity, provides the means for coupling volumetric and shear behaviour.

2.2. Normally consolidated behaviour

The normally consolidated behaviour is described by a yield surface, a failure criterion, a flow rule and a hardening law. The yield surface, which is initially oriented along the direction of consolidation, has the shape of an ellipsoid (Whittle, 1990) and can be written as:

$$F = -c^2 \; p'(2\alpha' - p') + \sum_{i=1}^{5} \left(\{s_i\} - p'\{b_i\}\right)^2 = 0 \qquad (1)$$

where c is the ratio of the semi-axes of the ellipsoid, α controls the size of the yield surface, p' is the mean effective stress, $\{b_i\}$ is a tensor that describes the orientation of the yield surface and $\{s_i\}$ is the deviatoric stress tensor expressed in terms of transformed variables.

The model assumes two hardening rules to describe changes in the size and orientation of the yield surface (Fig. 1). The critical state conditions, describing the behaviour at large shear strains are represented by an anisotropic failure criterion expressed by the equation:

$$h = -k^2 \; p'^2 + \sum_{i=1}^{5} \left(\{s_i\} - p'\{\xi_i\}\right)^2 = 0 \qquad (2)$$

where h describes a cone in the generalised stress space with apex at the origin (Fig. 1).

The scalar constant k and the tensor $\{\xi_i\}$ are soil properties which define the size of the cone and the anisotropy of the failure criterion, respectively. The tensor ξ_i is defined by the friction angles in compression (ϕ'_{TC}) and extension (ϕ'_{TE}) at large strain conditions ($\varepsilon_a \approx 10\%$ in undrained shear tests) (Whittle & Kavvadas, 1994).

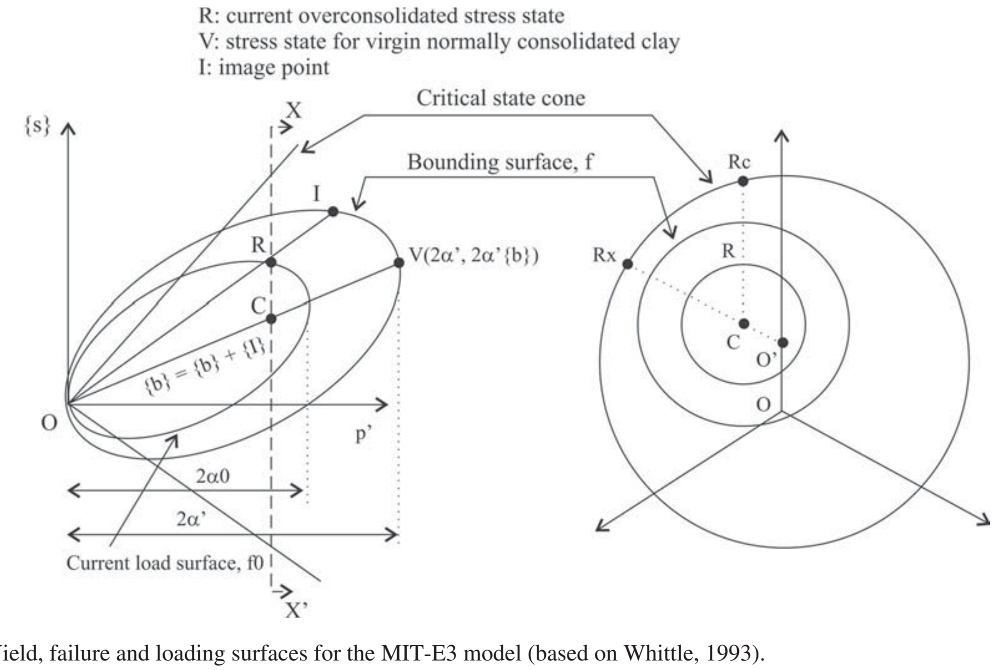

Figure 1 - Yield, failure and loading surfaces for the MIT-E3 model (based on Whittle, 1993).

The model uses a non-associated flow rule in order to describe the critical state failure conditions and K_0 conditions for a one-dimensional compression of virgin normally consolidated clay. Hence, when the stress state approaches the failure cone, there is no volumetric deformation, and in normally consolidated soils in K_0 conditions, any change in the stress state will not change the K_0^{nc} (Ganendra, 1993).

2.3. Overconsolidated behaviour

Laboratory studies on the behaviour of clays show that for unloading/reloading cycles in undrained shear or hydrostatic compression it is possible to observe (Whittle & Kavvadas, 1994):

i) a much stiffer response than the one obtained from the primary loading curve;

ii) a stress-strain hysteretic behaviour;

iii) the existence of plastic strains at the end of each cycle.

To simulate this behaviour of overconsolidated clays, the MIT-E3 model links the perfectly hysteretic model and the bounding plasticity surface. For a load cycle in the stress space, the perfectly hysteretic model describes a closed symmetric hysteresis loop (Fig. 2a), in which the initial stiffness gradually decreases either during the unloading of the virgin consolidation line (from A to B) or during the reloading (from B to A), and that there are no plastic deformations. This model assumes isotropic relations between effective stresses and elastic strain rates, and so there is no coupling between volumetric and shear behaviour.

The MIT-E3 model enables the inclusion of plastic strains in overconsolidated clays (Fig. 2b), providing the means for coupling volumetric and shear behaviour and ensures a smooth transition to normally consolidated behaviour (Whittle, 1993).

In the model, the bounding surface of normally consolidated clay behaviour is described by the yield functions (Eq. (1)), and the plastic behaviour for overconsolidated stress states, R, is linked to the plastic behaviour at the image point *I* (Fig. 1). This situation matches the definition of a load surface f_0 that contains the current (overconsolidated) stress state, homothetic to the bounding surface *f* with a shape coefficient α_0'/α' (Whittle, 1990).

A detailed description of the MIT-E3 model can be found in Whittle (1993), Ganendra (1993), Whittle & Kavvadas (1994) and Venda Oliveira (2000).

3. Base Problem

3.1. Basic assumptions

The analysed problem consists of a 10 m diameter tunnel excavated at a 20 m depth in an overconsolidated clay (Boston Blue Clay – OCR = 6.0), underlying a superficial 5 m thick layer of sand, as shown in Fig. 3. The groundwater table was assumed to be at the surface and the tunnel lining was simulated by a 0.30 m thick concrete ring, considered in perfect contact with the surrounding soil.

The finite element mesh considered in the numerical analysis, which is shown in Fig. 4, includes 308 quadrangular 8 node elements and 961 nodes. "Hybrid" finite elements were adopted, *i.e.*, the displacements were computed in 8 nodal points (quadratic interpolation) and pore pressures in 4 nodal points (linear interpolation).

At the lateral boundaries of the problem, located on the symmetry axis and 225 m to the right, the horizontal displacements were restrained and in the lower boundary, located 60 m below the ground surface, no displacements were allowed.

In terms of hydraulic boundary conditions, the top and the right boundaries were assumed permeable, while the left and lower boundaries were considered impermeable. The excavation boundary was assumed impermeable. For the sand layer a high permeability coefficient was

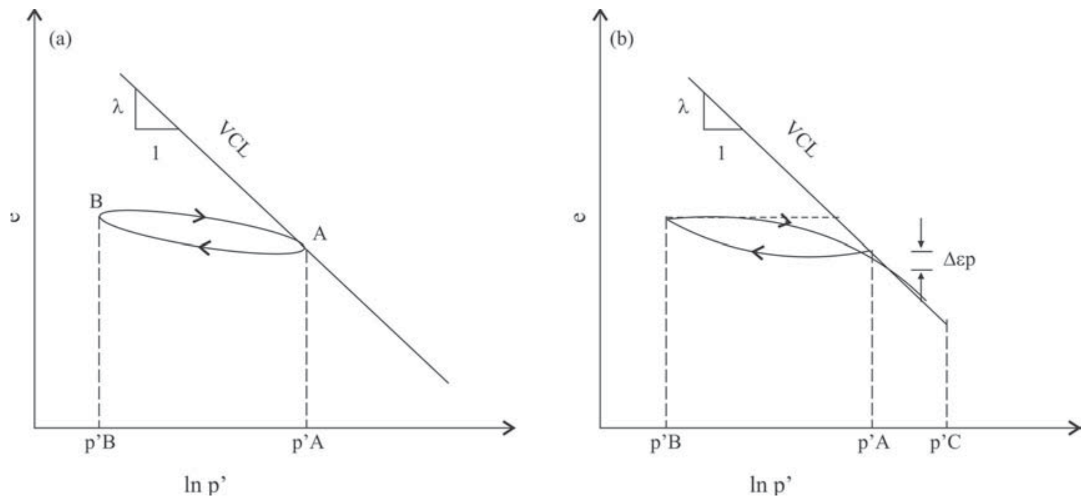

Figure 2 - Conceptual model of unload/reload used by the MIT-E3 model for hydrostatic compression: a) perfect hysteresis; b) Hysteresis and bounding surface plasticity (based on Whittle, 1993).

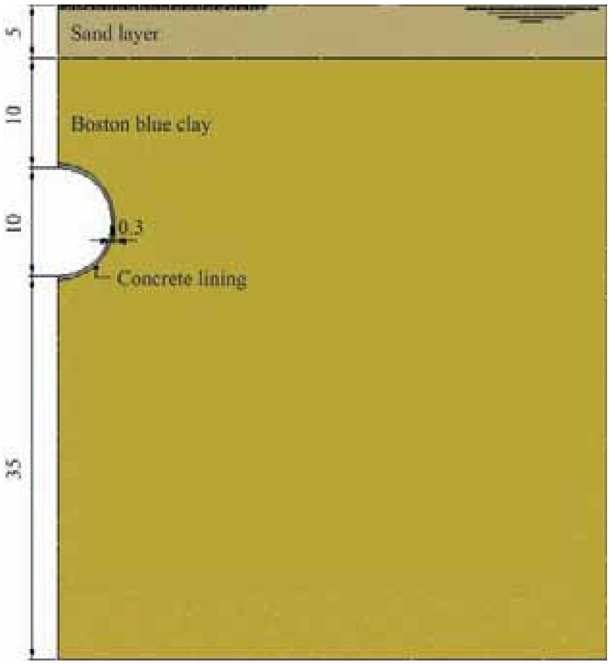

Figure 3 - Geotechnical profile.

Table 1 - Soil properties.

Soil layer	γ (kN/m^3)	K_0
Sand	20	0.5
Boston Blue Clay	20	0.85

adopted and for the clay layer, considered isotropic in terms of permeability, the permeability coefficient adopted was $k = 5 \times 10^{-9}$ m/s.

The ground water pressure was defined by the position of the groundwater table which is considered constant throughout the sand layer.

The geostatic stress state was adopted for the initial conditions and the soil properties shown in Table 1 were assumed.

The behaviour of the sand layer was considered linear, elastic and isotropic, with a Young's modulus (E) of 40 MPa and a Poisson coefficient (ν) of 0.30. For the tunnel lining a linear, elastic and isotropic behaviour was adopted, characterised by E = 30 GPa and ν = 0.2.

The MIT-E3 model, considered for the clay layer, uses 15 input parameters. Some are directly determined from standard laboratory test results: e_0 (initial void ratio); λ (slope of the virgin compression line in e-ln p' space); κ_0 (initial slope of swelling line in e-ln p' space) ; K_0^{nc} ; ν; ϕ'_{TC} ; ϕ'_{TE}. The remaining eight parameters are indirectly obtained from parametric studies: C and n (control the non-linear volumetric swelling behaviour); w (commands the non-linearity at small strains in undrained shear); h (controls the irrecoverable plastic strain); c (controls the undrained shear strength *i.e.* the geometry of bounding surface); S_t (affects the degree of strain softening); γ (regulates the generation of pore pressures induced by shear in overconsolidated clay); ψ_0 (controls the rotation of the bounding surface). The Boston Blue Clay (BBC) parameters are presented in Table 2.

The study was carried out using plane strain conditions, and the convergence confinement method was used to take into account the 3D effects of the excavation (Almeida e Sousa, 1998). In order to simulate soil deformations that occur prior to the installation of the lining, only a fraction (α) of the in situ stresses is relieved. In a second phase, after the installation of the lining, the rest of the equivalent nodal forces are loosened. During these two calculation phases, the behaviour of the soil is assumed undrained and, as a result, excess pore pressures are gener-

Table 2 - MIT-E3 input parameters for Boston Blue Clay (Whittle *et al.*, 1994).

e_0	λ	C	n	h	K_0^{nc}	ν	ϕ'_{TC}	ϕ'_{TE}	c	St	w	γ	κ_0	ψ_0
1.12	0.184	22.0	1.6	0.2	0.48	0.277	33.4°	45.9°	0.86	4.5	0.07	0.5	0.001	100

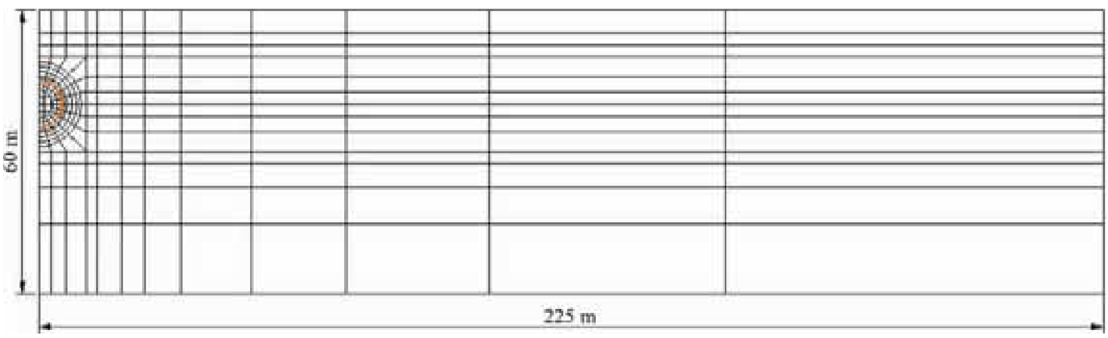

Figure 4 - Finite element mesh.

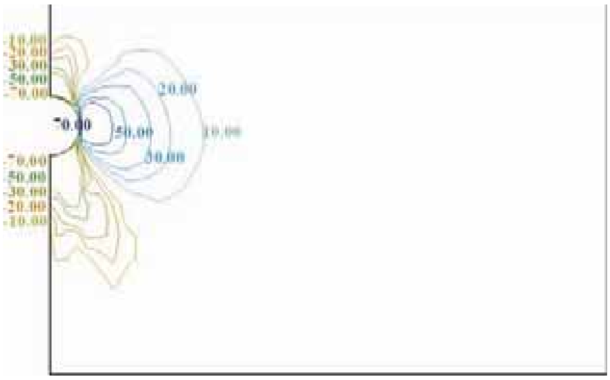

Figure 5 - $\Delta\sigma_y$ immediately after construction (kPa).

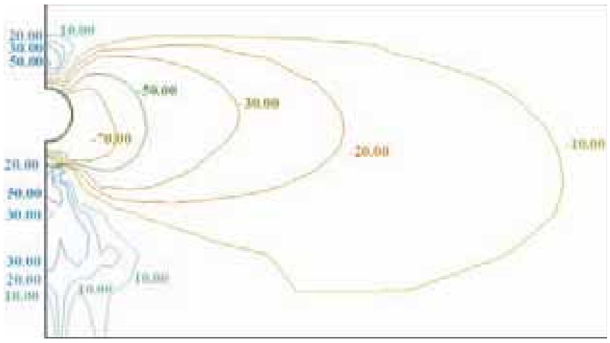

Figure 6 - $\Delta\sigma_x$ immediately after construction (kPa).

ated. For the base problem, a stress relief coefficient $\alpha = 0.63$ was adopted, since it was considered to entail subsidence volumes of approximately 1% of the volume of the excavation on short term conditions (Atwa, 1996; Lopes, 2004), which is in good agreement with the literature (Atwa, 1996; Almeida e Sousa, 1998).

After completion of the construction, the calculations proceeded in order to simulate excess pore pressure dissipation. Several calculation steps were considered, each corresponding to a certain time interval. However, the main purpose of this work is to show clearly the differences between the behaviour immediately after the construction and after consolidation. In this way, the intermediate times of consolidation were not presented in this paper.

3.2. Stress changes

The total stress in the vertical and horizontal direction and shear stress variations immediately after construction are shown respectively in Fig. 5, Fig. 6 and Fig. 7.

The observation of these stress distributions allows identifying five different areas around the tunnel section. Both above and below the tunnel, it is possible to observe a decrease in the vertical stress and an increase in the horizontal stress, which implies a 90° rotation of the principal stress directions. On the other hand, the elements located on the side of the section show an increase in the vertical stress, due to load transfer, and a decrease in the horizontal stress. At these three locations, as there are no shear stress variations, there is no change in the principal stress directions. The other two areas are located at approximately 45° from the tunnel axis (shoulder and bottom of sidewall). In these areas, the shear stress variation is significant and the principal stress directions are rotated.

As mentioned above, and due to the consideration of undrained conditions, the excavation induces excess pore pressures, as illustrated in Fig. 8. It is possible to observe that in general the excess pore pressures are negative, which is related to the expansion of the cavity, caused by the excavation of the tunnel.

The major negative excess pore pressures were obtained near the invert and the lower part of the sidewall, since these are the locations where the most significant stress variations took place. On the side of the section, negative excess pore pressures are only due to the shear stress variation, hence the negative excess pore pressures are less significant.

After completion of the construction, the calculations proceeded to simulate the consolidation of the clay. During the considered time intervals, excess pore pressures were dissipated in order to establish the final equilibrium conditions. Since the lining was considered impermeable, the final equilibrium was established by the hydrostatic conditions. This situation leads to an increase of the ground water content in the areas where negative excess pore pressures

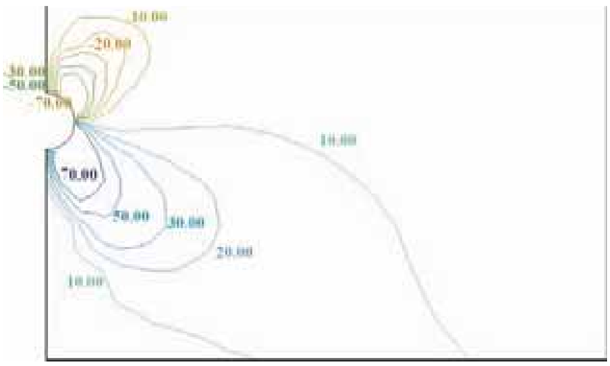

Figure 7 - $\Delta\tau_{xy}$ immediately after construction (kPa).

Figure 8 - Excess pore pressures generated immediately after construction (kPa).

were generated. During consolidation, the variation of the pore pressures will affect the effective stresses around the tunnel, and consequently the ground movements and the loads on the lining.

3.3. Ground movements and lining loads

The surface settlements obtained immediately after construction ($t = 0$) and after consolidation are presented in Fig. 9a. It is possible to observe that the surface settlements tend to decrease with time. Similar conclusion can be drawn from Fig. 9b on what horizontal displacements are concerned. This is due to the expansion of the ground around the tunnel section, induced by the dissipation of the negative excess pore pressures, which is opposite to the vertical and horizontal movements induced by the ground removal.

The ground displacements in depth are presented in Fig. 10. The first observation is that there is significant attenuation of vertical displacements as the distance to the tunnel section increases. In fact, for a final vertical crown settlement (δ_{vc}) of 52.2 mm, the maximum surface settlement (δ_{vs}) was only 4.7 mm, which can be translated by δ_{vs}/δ_{vc} of 0.09.

The consolidation of the clay layer is also responsible for changes in the lining loads. As the effective stresses of the soil decline, the loads acting on the lining rise.

Fig. 11 shows the axial loads acting on the lining immediately after construction ($t = 0$) and after consolidation (t. end), and allows to observe that this load is more significant near the invert and although it tends to increase with time, it does not vary considerably during consolidation.

In terms of the bending moments, Fig. 12 shows that these are practically null after construction and tend to increase with time. In fact, the expansion of the ground around the tunnel section during consolidation is not uniform, hence the bending moment rises.

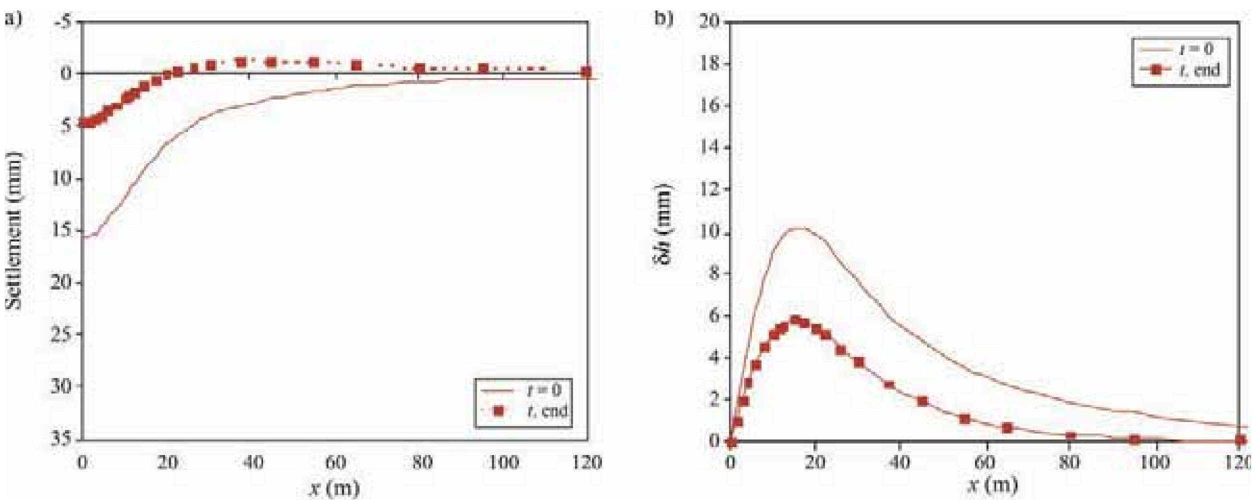

Figure 9 - Displacements induced at the ground surface: a) settlements; b) horizontal displacements.

4. Parametric Study

4.1. General

Table 3 summarizes the calculations carried out during the parametric analysis. The calculations of cases A, B

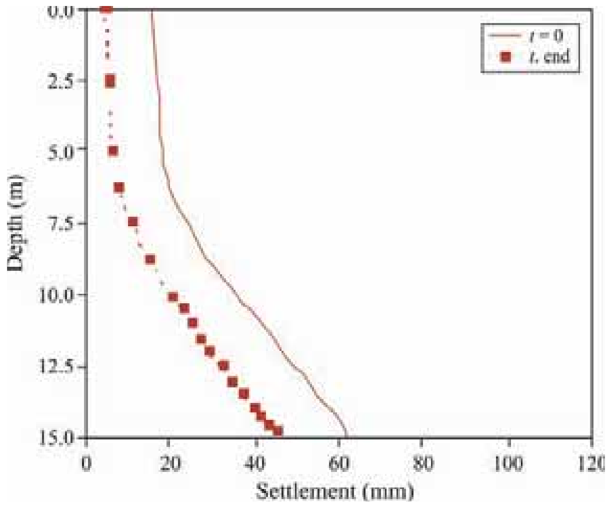

Figure 10 - Ground displacements induced in depth.

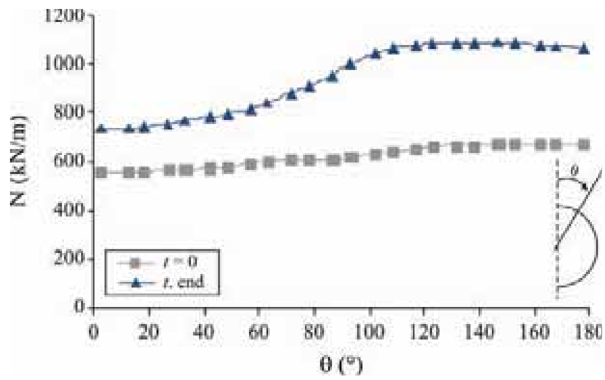

Figure 11 - Axial load on the lining.

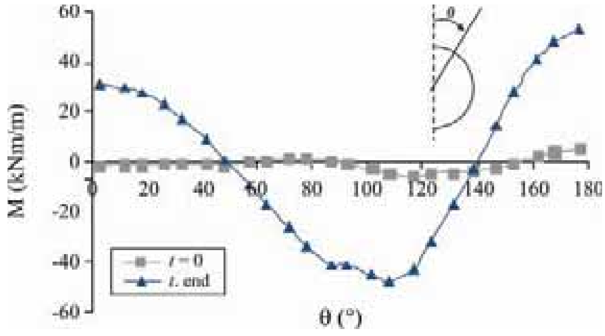

Figure 12 - Bending moment on the lining.

and C were performed in order to evaluate the influence of the construction process on the excess pore pressures. In these cases, the lining was considered impermeable. Case B corresponds to the base problem, which has been presented so far. Cases D, E and F were introduced to clarify the influence of the relative permeability between the soil and the lining, hence the lining was considered permeable and only one relief coefficient was considered.

4.2. Construction process

The analysis of the stress changes induced by the excavation, when considering a larger relief coefficient (case C) in comparison to those illustrated above (case B), suggests the same type of variation during construction, although with longer total stress paths, thus more important variations in both mean and shear stresses.

Fig. 13 shows the pore pressures obtained along the symmetry axis for different relief coefficients immediately after construction. It is possible to observe that the relief of an important part of the equivalent nodal forces prior to the installation of the support (case C), is responsible for larger stress variations on the ground and, consequently, larger negative excess pore pressures. The dissipation of these pore pressures is accompanied by larger displacements in the opposite direction of those induced by the undrained excavation. Numerical analysis performed on the Athens underground show the same behaviour (Drakos *et al.*, 2002).

The ground movements induced by the excavation for cases A, B and C are illustrated in Fig. 14 to Fig. 16. It is

Table 3 - Cases analysed in parametric study.

Case	α	Lining permeability	Soil/lining relative permeability (15 m depth)
A	0.40		
B	0.63	Impermeable	-
C	0.75		
D			1000
E	0.63	Permeable	100
F			10

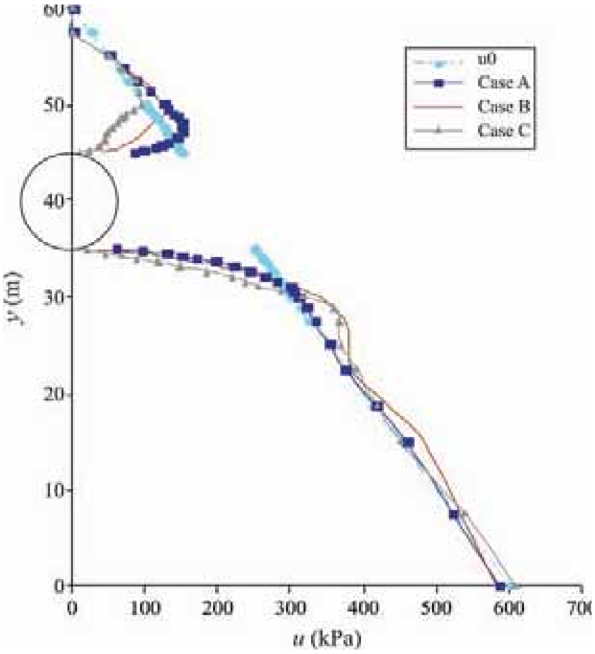

Figure 13 - Pore pressures in the ground along the symmetry axis of the problem, immediately after construction ($t = 0$) with different relief coefficients.

possible to note that the ground movements after construction tend to grow with larger equivalent nodal forces relieved prior to the installation of the support. However, after consolidation, these differences are less significant, since the excavation boundary was assumed impermeable, meaning that the final equilibrium is achieved by the reestablishment of hydrostatic pressures.

The axial loads acting on the lining are presented in Fig. 17 for cases A, B and C, for two time intervals, corresponding to the end of construction ($t = 0$) and the end of consolidation. The results show that the increase of the relief coefficient is responsible for a decrease on the axial load immediately after construction. During consolidation, the pore pressure variation implies an increase of the axial load, which is more significant as the relief coefficient grows. After consolidation the axial load does not significantly differ for the three cases.

The results show a bending moment of approximately zero immediately after construction, which increases with time, especially when the highest relief coefficient is adopted (Fig. 18).

Analysing both axial load and bending moment for cases A, B and C, it is possible to observe that the axial stresses are higher when the relief coefficient is lower, contrary to bending stresses that tend to increase with the increase of the relief coefficient.

Table 4 and Table 5 summarize these conclusions presenting both axial stresses and bending stresses for the three cases, immediately after construction and after consolidation.

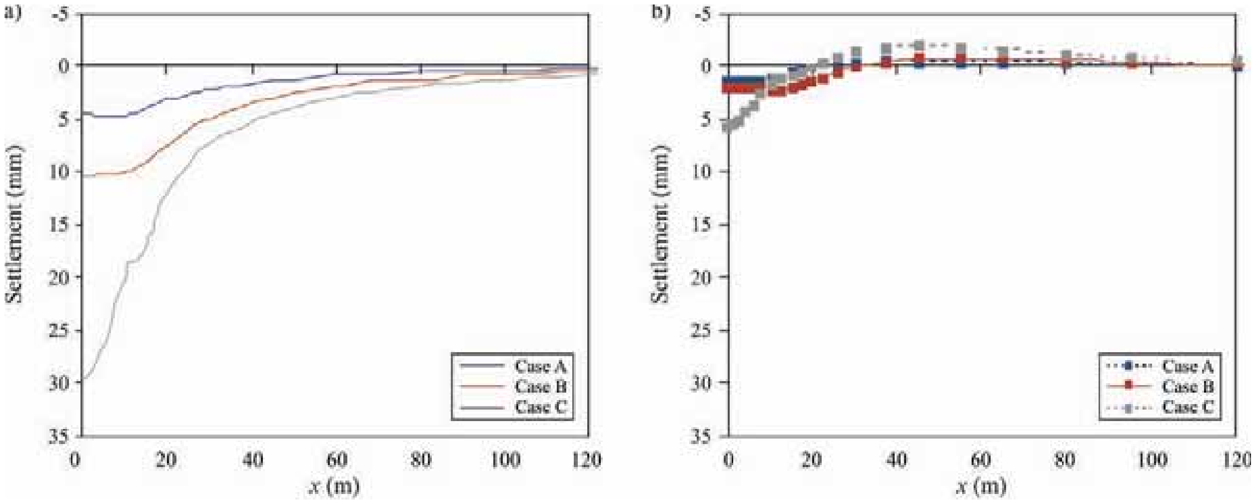

Figure 14 - Ground surface settlements in cases A, B and C: a) immediately after construction; b) after consolidation.

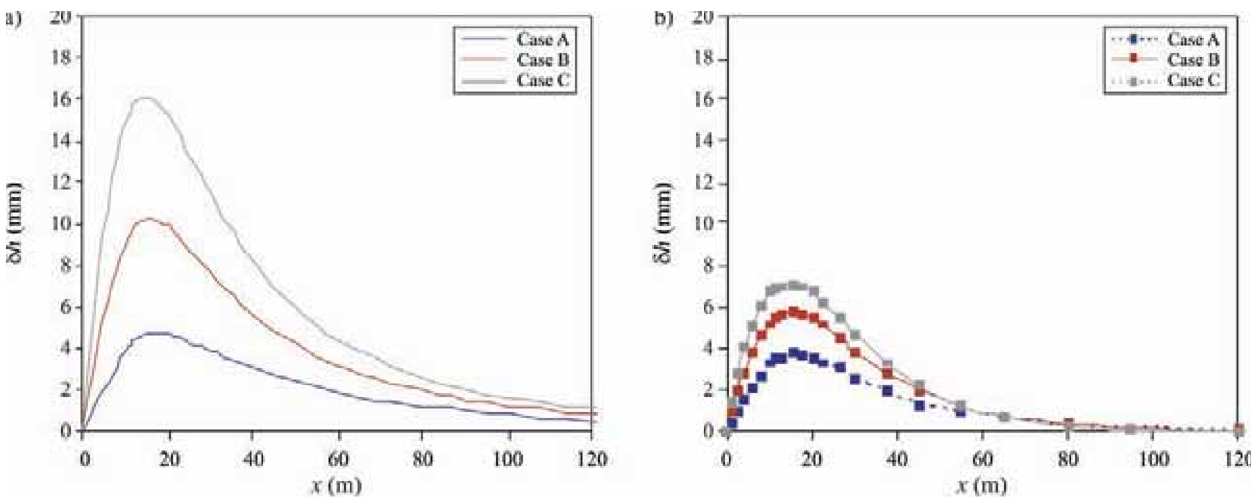

Figure 15 - Surface horizontal displacements in cases A, B and C: a) immediately after construction; b) after consolidation.

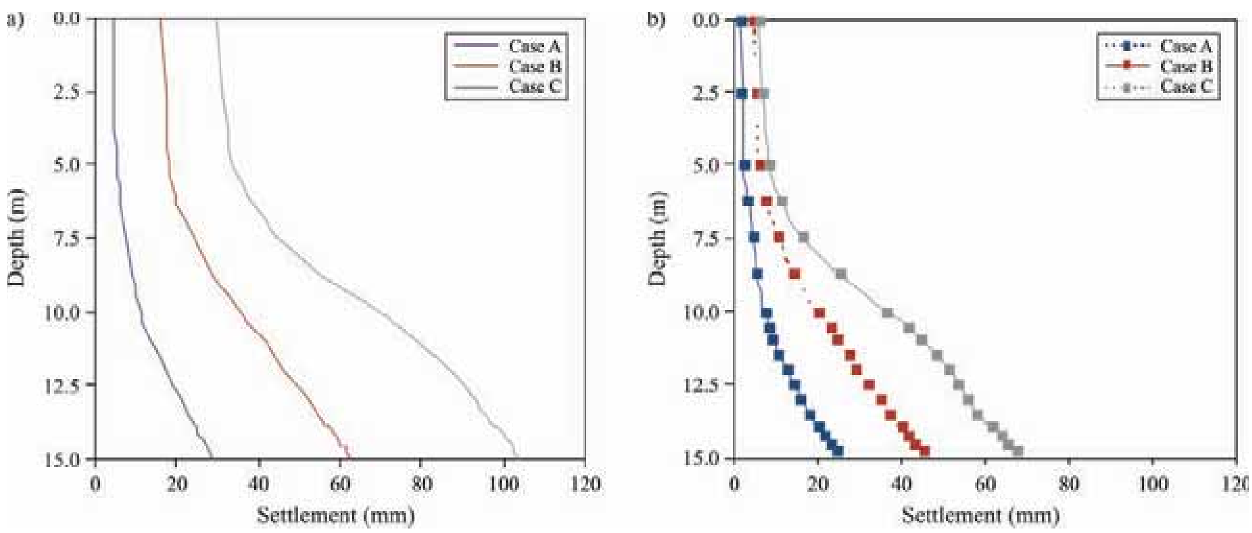

Figure 16 - Ground settlements in depth in cases A, B and C: a) immediately after construction; b) after consolidation.

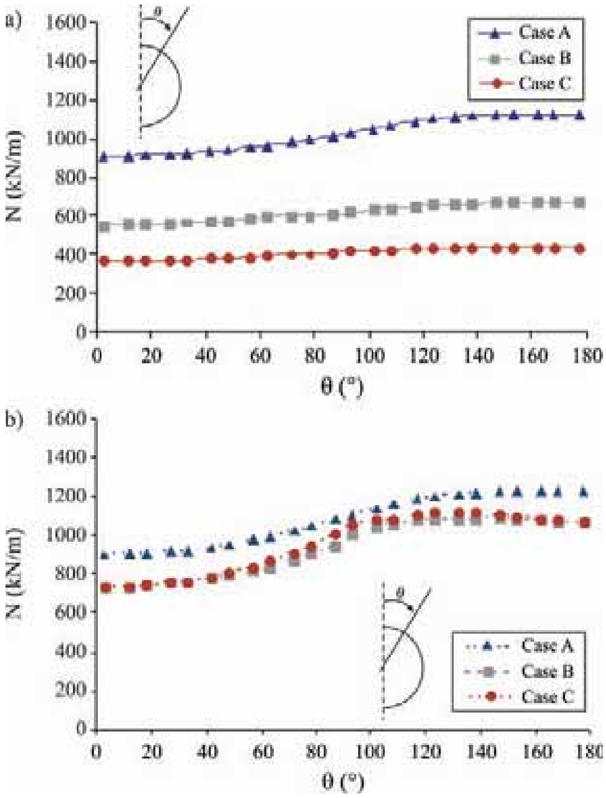

Figure 17 - Axial loads acting on the lining: a) immediately after construction; b) after consolidation.

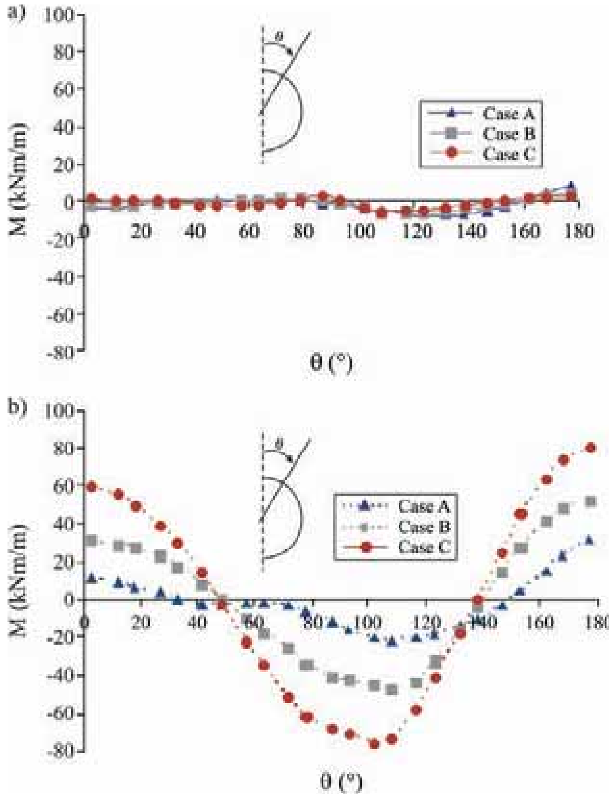

Figure 18 - Bending moments acting on the lining: a) immediately after construction; b) after consolidation.

After construction, the contribution of the axial stresses is more significant than the contribution of the bending stresses, as opposed to what is obtained after consolidation. In fact, when considering a high relief coefficient (case C) the bending stress overcomes the axial stress, thus causing tension in the lining.

4.3. Soil/lining permeability

The effect of the soil/lining relative permeability on the time-dependent behaviour of the tunnel was studied by comparing the results of four different calculations, identified in Table 1 as Cases B, D, E and F. The relief coefficient was assumed $\alpha = 0.63$, as it entails an adequate subsidence

Table 4 - Maximum stresses on the lining immediately after construction.

	$\frac{N}{A}$ (kPa/m)	$\frac{M}{I}$ (kPa/m)	$\sigma = \frac{N}{A} + \frac{M}{I}$ (kPa/m)	% of N	% of M
$\alpha = 0.40$	3750	525	4275	87.7	12.3
$\alpha = 0.63$	2232	328	2560	87.2	12.8
$\alpha = 0.75$	1460	241	1700	85.9	14.1

Table 5 - Maximum stresses on the lining after consolidation.

	$\frac{N}{A}$ (kPa/m)	$\frac{M}{I}$ (kPa/m)	$\sigma = \frac{N}{A} + \frac{M}{I}$ (kPa/m)	% of N	% of M
$\alpha = 0.40$	4086	2140	6226	65.6	34.4
$\alpha = 0.63$	3551	3521	7072	50.2	49.8
$\alpha = 0.75$	3566	5349	8915	40.0	60.0

volume according to the literature. In case B, the lining was considered impermeable, contrary to what was assumed in cases D, E and F, in which the permeability of the lining was considered to be 1000, 100 and 10 times lower that the permeability of the ground, respectively.

In order to simulate a permeable excavation boundary, it was necessary to change the hydraulic boundary conditions of the problem, by altering the permeability coefficient of the finite elements adjacent to the excavation section and by imposing zero pore water pressures on the contact face of these elements and those that materialize the concrete lining, hence allowing seepage into the tunnel.

Since the excavation was performed in undrained conditions, the results obtained for the end of construction are coincident in the four cases and will be referred to as $t = 0$. The obtained differences are significant in the end of consolidation, given that the seepage into the tunnel section provides final equilibrium pore pressures lower than the hydrostatic condition.

Fig. 19 shows the pore pressures along the vertical symmetry axis of the problem for two time intervals (after construction and after consolidation) obtained for cases B and E. The difference between the final and hydrostatic (u_0) pore pressures is more significant for case E, since a drainage-type tunnel was considered.

As the final equilibrium conditions differ, so will the stress-paths and, consequently, the ground movements and the lining loads.

Fig. 20 shows the vertical and horizontal displacements induced in the ground surface for cases B, D, E and F, obtained immediately after construction ($t = 0$) and after consolidation (t. end).

It is possible to observe that both the vertical and horizontal displacements induced on the ground surface when considering a drainage-type tunnel are more significant than those obtained with a waterproof-type tunnel (case B).

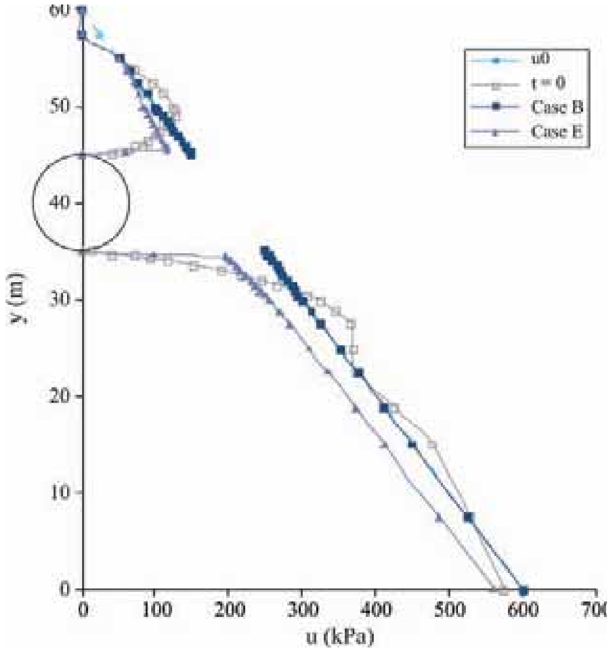

Figure 19 - Pore pressures in the ground along the symmetry axis of the problem, immediately after construction ($t = 0$) and after consolidation (cases B and E).

These differences tend to be amplified with the permeability of the lining, since the global volume of ground surrounding the cavity tends to be reduced as the water flows into the tunnel. In this case, for a soil/lining relative permeability lower than 100, the surface settlements tend to increase during consolidation, contrary to what was obtained with an impermeable lining.

The same behaviour can be observed in Fig. 21, where vertical displacements along the symmetry axis of the problem are illustrated. These results also show an interesting feature regarding the relative vertical displace-

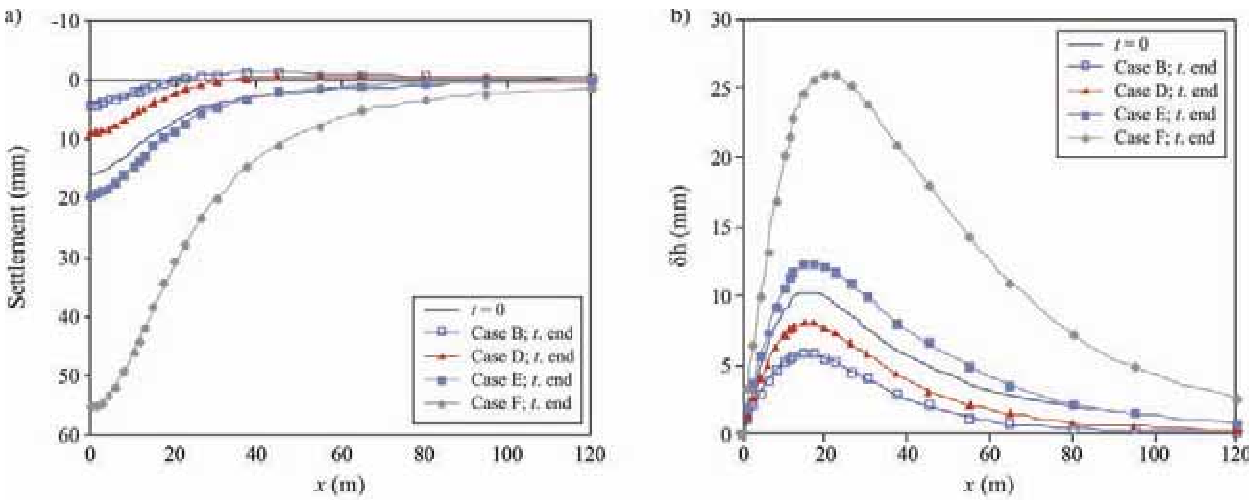

Figure 20 - Displacements induced at the ground surface in cases B, D, E and F: a) settlements; b) horizontal displacements.

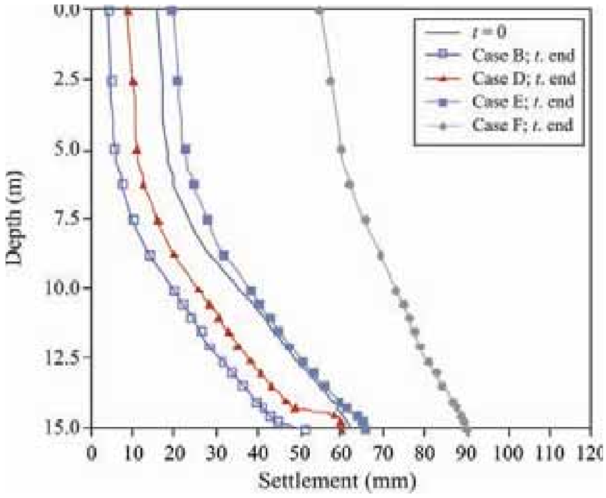

Figure 21 - Ground surface settlements in depth in cases B, D, E and F.

ment between the surface and the tunnel crown (δ_{vs}/δ_{vc}). In fact, as the lining permeability increases, the total head losses in the surrounding ground increase, hence the seepage forces and the effective stresses tend to increase above the tunnel section, causing the approximation of those two points. The ratio (δ_{vs}/δ_{vc}) of these settlements is of approximately 10% for case B (impermeable lining) and 60% for case F.

The axial loads and bending moments on the lining tend to increase during consolidation, more significantly in waterproof-type tunnels, since the water content of the ground in these cases also grows. Fig. 22 demonstrates that an increase of the permeability of the lining will decrease the variation of both axial forces and bending moments during consolidation.

5. Conclusions

In this paper, a numerical study concerning the excavation of a tunnel in clay is presented. The results show that in overconsolidated clay the variations in the normal mean stress and the shear stress induced by the excavation have similar effects on the generation of excess pore pressures. In fact, in overconsolidated clays, due to positive dilatancy, the reduction of the mean stress and the increase of the shear stress, which occur during the excavation, are both responsible for the decrease of pore water pressures. The dissipation of these negative excess pore pressures during consolidation, considering an impermeable lining, occurs with an expansion of the ground volume which, in turn, is responsible for the swelling at the surface. As the ground volume increases with time, so does the water content around the tunnel section and, consequently, so do the axial loads acting on the lining. Since this is not a uniform swelling, bending moments on the lining, which are practically null after construction, also tend to increase with time.

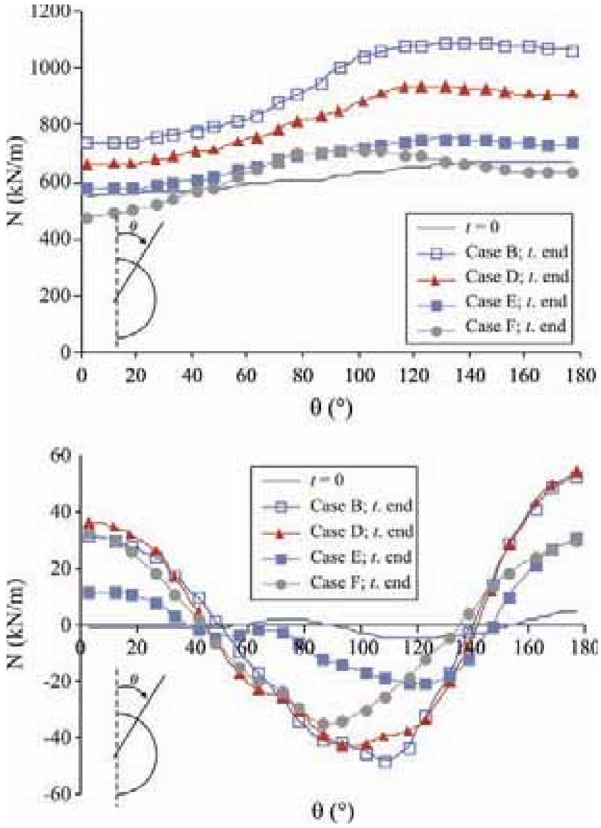

Figure 22 - Loads acting on the lining after construction (t = 0) and after consolidation (t. end): a) axial loads; b) bending moments.

A parametric study was carried out in order to clarify the influence of the construction method, by using three different relief coefficients, and the influence of the lining permeability, which was assumed permeable and with different permeability coefficients.

In what the construction method is concerned, the results show that the magnitude of the negative excess pore pressures tends to increase with the relief coefficient, since the stress variations during construction also increase. As a result, the ground movements induced by the excavation will be amplified. During consolidation, these differences tend to attenuate, particularly at the ground surface, due to the swelling derived from the water content increase around the tunnel.

The same situation occurs with axial loads on the lining, whose variations tend to reduce with time, contrary to what happens with bending moments.

The calculations carried out to analyse the influence of the soil/lining relative permeability on the time-dependent behaviour lead to an enhancement of the surface displacements with the reduction of the relative permeability. Actually, the consideration of a permeable excavation boundary assumes a groundwater flow into the filter layer located between the ground and the lining (drainage-type tunnel), establishing a final equilibrium condition in which pore pressures are lower than the hydrostatic ones. Consequently, the effective stresses will rise more than in waterproof-type tunnels. On the other hand, the decrease of the water content on the ground surrounding the cavity leads to lower axial loads on the lining, as its permeability increases.

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Geo-Environmental Investigation: A Brief Review and a Few Suggestions for Brazilian Contaminated Sites

Giulliana Mondelli, Heraldo Luiz Giacheti, John A. Howie

Abstract. This paper presents a brief review about site investigation procedures for contaminated sites recommended by Brazilian and Canadian environmental agencies as well as discusses the theme of geo-environmental investigation as applied to Brazilian practice. The main definitions on the theme are reviewed and some guidelines are proposed for conducting a geo-environmental investigation of Brazilian contaminated sites using different site and laboratory investigation techniques based on the presented review and on the experience obtained from the investigation of a solid waste disposal site in the interior of the state of São Paulo, Brazil.

Keywords: environmental geotechnics, contaminated sites, regulations, MSW disposal site, tropical soils.

1. Introduction

Contaminated or even potentially contaminated sites abandoned in urban areas are subjects of current interest and concern due to their possible impact on the environment and on human health. Environmental agencies in several countries have proposed different site investigation methodologies to diagnose and confirm different contamination degrees in sites with diverse physical characteristics to guide the remediation plan when it is necessary.

The experience already achieved on site investigation has shown that the best methodology is site specific, and depends on subsoil and chemical contaminants; geotechnical, geological and hydrogeological aspects; evolution of the contamination plume and the possible risks it poses. Several site and laboratory investigation techniques (direct and indirect) have been proposed and used. Sometimes one technique is more suitable than another depending on the physical and natural characteristics of the site.

The demand for geo-environmental investigation of contaminated sites in Brazil has been substantially increased in recent years due to lack of space in metropolitan areas for construction of sanitary landfills. Brown fields and old dumps sites also need to be monitored and investigated for site remediation.

It is important to review the existing regulations and definitions for geo-environmental investigation and suggest guidelines to be followed in the Brazilian practice considering it is a developing country with a large territory to protect. The specific objectives of this paper are:

1) Review the term "geo-environmental investigation" in the practice of Geotechnical, Geological and Sanitary Engineering in Brazil.

2) Compare and discuss Brazil's and Canada's environmental regulations for geo-environmental investigation.

3) Suggest guidelines for geo-environmental investigation of Brazilian contaminated sites based on the presented review and on the experience obtained from the investigation of a MSW disposal site in the interior of the state of São Paulo, Brazil, which was extensively investigated during more than seven years.

2. Definitions of Geo-Environmental Investigation

Geo-environmental investigation is a relatively new term in geotechnical engineering. There are different interpretations for the meaning of the word "geo-environmental". According to Davies & Campanella (1995), it can be understood as "the field of study that links geological, geotechnical, and environmental engineering and their related engineering sciences to form an area of study and practice that includes all physical and geochemical concerns within natural or processed geological media."

Almeida & Miranda Neto (2003) define geo-environmental investigation as the systematic collection of data to determine the degree of contamination of a particular site. These data can be obtained not only by sampling and testing, but also throughout document research, interviews, technical visits, inspections and surveys. According to the authors, the main objective of a geo-environmental investigation is to gather sufficient information of the source, contamination paths and targets to support risk assessment studies and/or site remediation plan, if necessary, aiming to:

• Determine the nature, shape, extension and distribution of any contamination found at the investigated site;

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• Characterize the physical environment, determining its geological, hydrological and geotechnical characteristics;

• Understand the nature of the potential targets of the contamination and the relationship between the source and its effects;

• Provide support for management and remediation decisions.

The Association of Geotechnical & Geo-environmental Specialists (AGS, 2000) proposes to integrate geotechnical and environmental investigations for contamination detection: the first aiming to establish the physical and mechanical properties of soil and the second aiming at determining the chemical composition of soil, water and gases. Brandl & Robertson (1997) comment that the goals of a geo-environmental investigation can be quite different from the traditional geotechnical investigations, since for geo-environmental purposes, in addition to the soil geotechnical characteristics, contaminants distribution and composition must also be identified. In such investigations, the stages of monitoring and sampling should be planned on a long term basis, including the steps of implementation, deactivation and remediation.

Jewell *et al.* (1993) consider the evolution of hydrogeological impact one of the most important aspects to be monitored for heterogeneous municipal solid waste disposal sites. In these particular cases the pollution that reaches the aquifer may spread in different pathways, depending on the type of pollutant (*e.g.* miscible or not) and the characteristics of the aquifer. Thus, a detailed characterization of the aquatic environment must include:

• Assessment of the regional and local geology, including stratigraphy, structure and lithology, weathering characteristics and superficial deposits;

• Assessment of local surface hydrology, including hydrometeorological water balance, major and tributary drainage, flood levels and groundwater contribution to base flow;

• Identification of main hydrogeological units (aquifers and aquitards);

• Assessment of likely groundwater flow mechanisms, and in particular, whether intergranular or fracture flow is likely to be dominant;

• Identification of local structural features which may control groundwater flow;

• Measurement of piezometric levels and calculation of the local hydraulic gradients within hydrogeological units;

• Measurement of the hydraulic conductivity of the main hydrogeological units identified;

• Assessment of the attenuation characteristics of the materials present in both vadose and phreatic zones;

• Analysis of natural groundwater chemistry;

• Identification of any presence or potential beneficial use of groundwater;

• Identification of potential groundwater and surface water receptors of contamination.

In the engineering practice, it is important to keep in mind that it is difficult to identify all the aforementioned factors during a site investigation campaign. Such investigations are constrained by the costs involved, lack of specialists and the geological, hydrogeological and geotechnical natural conditions, which may limit the use of some investigation techniques.

Boscov (2008) defines "geotechnical investigation" and "environmental monitoring" as part of environmental geotechnics, which involves a combination of geotechnical and environmental works. In the present paper, the term "geo-environmental investigation" encompasses all the phases of *in situ* and laboratory testing, field monitoring activities and research, aimed at gaining a better understanding of all the *in situ* geomaterials conditions, and of the aquifers and contaminants present at the site.

It should be noted that "investigation" and "monitoring" do not involve the same activities. Investigation consists of discovering, diagnosing and characterizing *in situ* hydrogeological and geotechnical conditions. Monitoring, on the other hand, involves observing several specific variables over time, for example to evaluate remediation techniques, investigation and/or stability solutions for the conditions at the site. Therefore, monitoring can be considered part of a "geo-environmental investigation", as defined herein.

3. Contaminated Sites

3.1. Definitions

According to CETESB (2004), São Paulo State Environmental Company, a contaminated site can be defined as a location where chemical substances or waste have been deliberately or accidentally disposed, accumulated, stored, buried or infiltrated. However, this definition can be reviewed, since there are two important aspects: natural concentrations of the substances of concern; and allowable concentrations defined by the local environmental agency. Theoretically, a site can be considered clean until the allowable levels are reached. At this site, pollutants or contaminants can be concentrated in the air, in surficial water, soil, sediments or in the groundwater. These pollutants or contaminants can be carried from the site, spreading by the air, soil, groundwater and surface water, altering their natural features or qualities and causing negative impacts and/or risks to a site or its surroundings.

The term "contaminated soil" is more frequently used than "polluted soil". Some authors consider them synonymous and others seek to make a fairly simple distinction between pollutants and contaminants. Yong (2001) and Yong & Mulligan (2004) comment that the term "pollutant" is used to indicate that a given contaminant is becoming a potential source of risk to human health and to the environment. Thus, the term "contaminant" applies more generally to contaminated soils, where site conditions are not sufficiently known to consider the actual existence of risk.

In the context of polluted or contaminated water, this difference is even more complex. Contamination, in this case, refers to the waterborne transmission of microorganisms or substances harmful to human health, which does not necessarily pose a risk to the ecological equilibrium of the environment (Braga *et al.*, 2002). Brazilian Law #6.938/81 defines "pollution" as "the degradation of environmental quality resulting from activity that is directly or indirectly harmful to public health, safety and welfare; creates adverse conditions for social and economic activities; affects the biota; affects the environment's esthetic or sanitary conditions; and discharges materials or energy in violation of the established environmental standards" (Brazil, 1981).

Nass (2002) states that the characteristic that clearly distinguishes "pollution" from "contamination" is the passiveness normally associated with the former. The pollution factor does not usually affect live organisms directly, but impacts the conditions for their survival indirectly, especially in the aquatic medium.

Clearly, the distinction between pollution and contamination depends on the environment that is polluted or contaminated, which can be in air, water or soil. In this paper, it is suggested to adopt the more general term "contaminated soil", which is commonly used by several specialists (*e.g.*, Sánchez, 2001 and Yong, 2001).

The term "contamination plume" is also very common in geo-environmental investigations. A contamination plume is formed when contaminants or pollutants reach the subsoil and spread in vadose and/or saturated zones. Figure 1 illustrates the formation of different types of contamination plumes, depending on the conditions of the site and the characteristics of the contaminants and/or pollutants.

Not all contaminated sites offer risks to the environment or human health. A fundamental aspect is to define the risk of using a contaminated site and it surrounding lands, considering the hazard degree of contamination and its probability to occur. A risk exists only if contaminant concentrations exceed acceptable limits, and if there are sensitive receptors or the possibility of an adverse event to occur. Following the example of countries with tradition in monitoring soil and groundwater quality and controlling contaminated sites, CETESB (2001; 2005) published a preliminary list of guidelines for protecting soil and groundwater, based as far as possible on the Brazilian national data and on human health risk assessment. These quality reference values were established for using as a fast and simple tool to support prevention actions and control soil and groundwater pollution.

The intervention values for groundwater adopted by CETESB (2001; 2005) have been often considered excessively restrictive to assess water quality surrounding a contaminated site, since these values were defined based on the standards for drinking water established by the Brazilian Ministry of Health, Regulation 518 (Brazil, 2004). Similar to CONAMA (2005) (Brazilian Environmental Agency), Resolution 357, which establishes limits for different uses of surface water, CONAMA (2008), Resolution 396, sets limits for different uses of Brazilian groundwater. In addition to the standards for water quality assessment for human consumption, the Resolution 396 also suggests reference values for farm animals, irrigation and recreational uses. Despite recent advances on this resolution, the limit values have not yet been established for several parameters that are important in the assessment of groundwater pollution and contamination, such as biological oxygen demand (BOD), chemical oxygen demand (COD), alkalinity, electrical conductivity, etc.

Sánchez (2001) points out that, although intervention values are helpful, their use may lead to excessively narrow interpretation or, conversely, insufficient protection of a site under investigation. In other words, the use of intervention values represent a limited proposal that does not take into account the site's geological and natural characteristics and its future land use, since certain uses do not require high soil or water quality. Therefore, Sánchez (2001) argues that risk analysis can provide a more reliable basis for decisions about remediation and future use of a site.

3.2. Regulations

Guidelines and regulations for each country establish the way to conduct geo-environmental investigations, considering different local and natural aspects, depending on previous local experience. Some countries, like the United Kingdom and United States have a good experience with brownfield sites, during the transition from areas of heavy industry which are now being redeveloped for residential or "light" industrial use. Countries with recent concern with the environment tend to adapt the more developed countries experience to the reality of their environmental laws, economy, industrialization, size, cultural and social aspects, etc.

The United States Environmental Protection Agency (US EPA, 1996) proposes a detailed plan based on a numerical modeling approach, which provides information about the extension of the contamination plume and of the affected site (Fig. 2). However, this model is not easily applied to complex sites, with heterogeneous geology and hydrogeology, since it is usually difficult to define the stratigraphic profile and particularly the contamination plume. In addition, in sites with complex geology, which is very common in tropical regions, the costs of an investigation program can be very high due to the extensive *in situ* sampling and laboratory testing required by the US EPA (1996) model.

Nnadi (1994) discusses the approach to investigate contaminated sites in North American countries. The au-

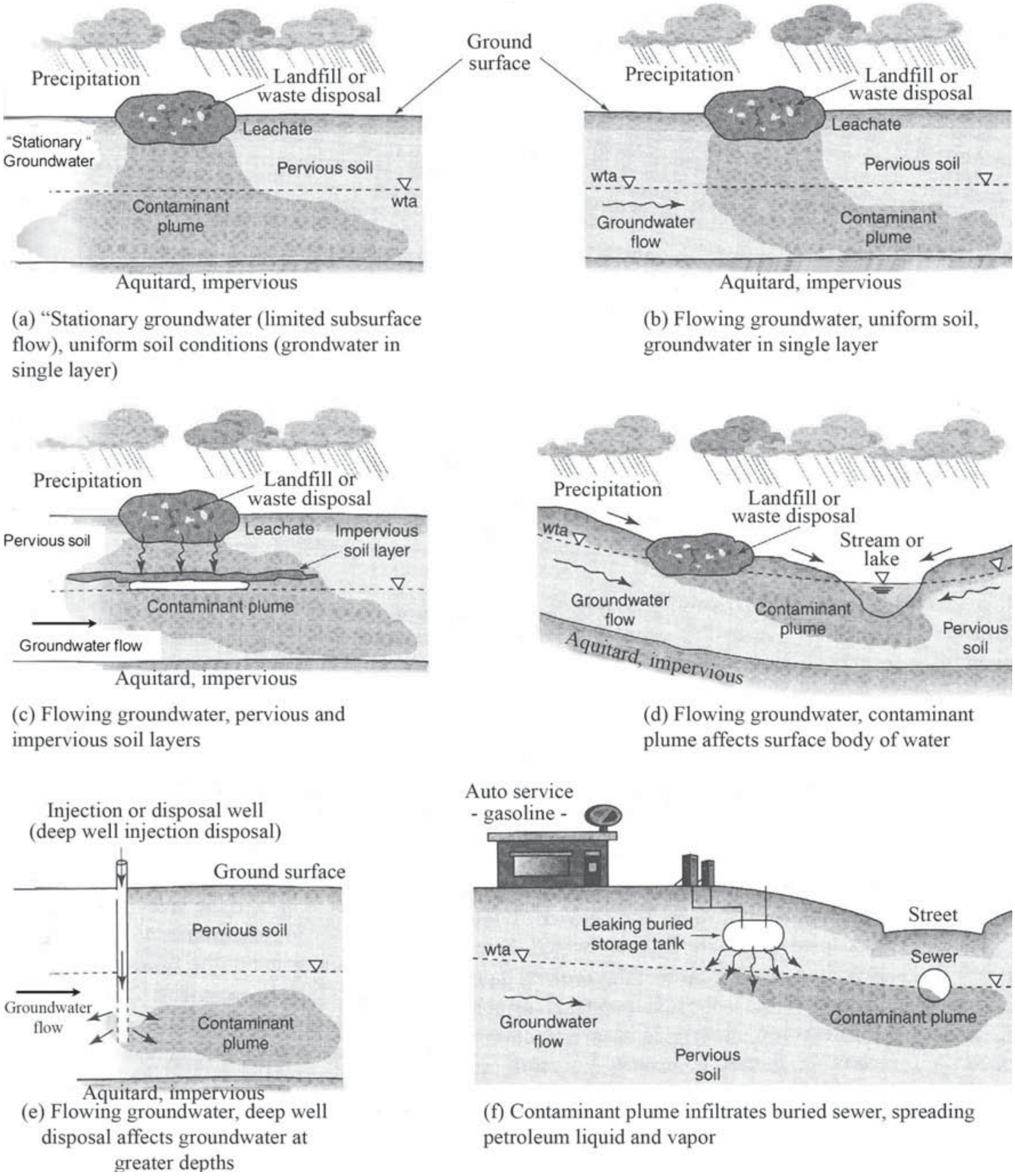

Figure 1 - Contaminant plumes for different site conditions and contaminant sources (McCarthy, 1998).

thor concludes that despite comprehensive environmental laws, United States and Canada differ from Mexico in assessment and enforcement of investigation and remediation of contaminated sites. While the first two countries are proactive, Mexico is reactive due to lack of financing resources and the infrastructure needed to control environmental contamination.

In Brazil, the assessment and registry of potentially contaminated sites (Table 1) began only a decade ago, by CETESB, in the state of São Paulo. Canada has regulations established by Federal law for the entire country, and each province also has its own regulations. The concern with contaminated sites in the state of São Paulo began due to the real estate interests in Greater São Paulo, where there are numerous deactivated industries. Canada, on the other hand, is concerned with the preservation of the natural environment, which includes the health and survival of all living beings that are part of it. Any site development activity can alter the ecosystem, being impossible to return the site to its natural condition. In this case, it is just possible to prevent any further deterioration and restrict the use and occupation of the site.

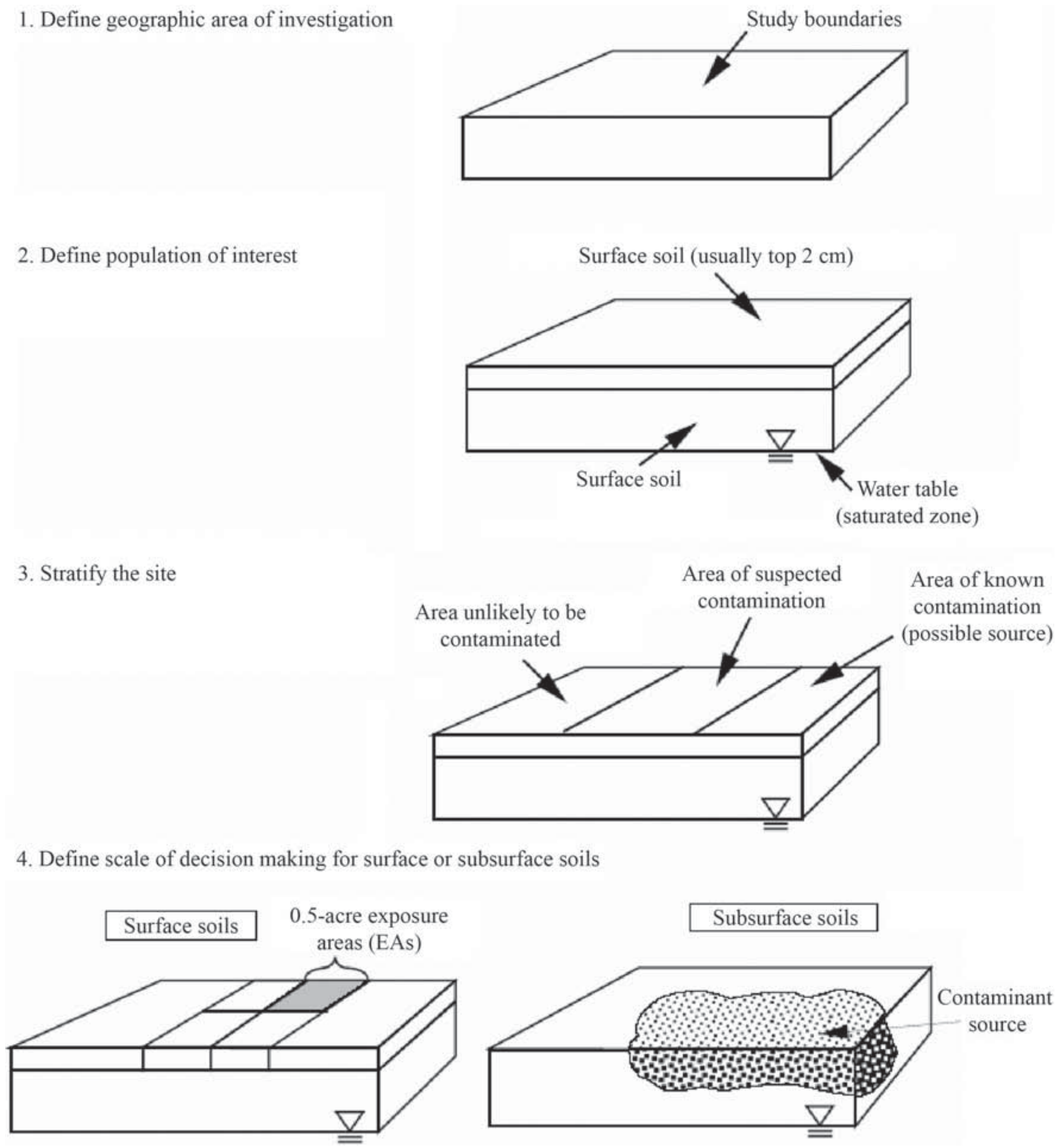

Figure 2 - Geo-environmental investigation of potentially contaminated sites in The United States of America (US EPA, 1996).

Figures 3 and 4 summarize the procedures proposed by CETESB and by Canada Contaminated Sites Management Working Group (CSMWG), respectively. Table 1 compares the regulations from CETESB and from CSMWG, to assess the geo-environmental investigation methodology currently used in both countries, with the objective to explain Figs. 3 and 4.

Independently of the distinction that CETESB does to different phases of investigation to classify the sites as potentially (PS), suspected (SS) and non-contaminated (NS) or contaminated (CS) sites, there are common activities in both procedures summarized in Figs. 3 and 4. These activities are: historical study, initial testing program and detailed investigation. The major difference is that CSMWG presents a more detailed program for remediation and risk management strategies (step 7 in Fig. 4).

CSMWG has a database listing all the potentially contaminated sites in the country, as well as information about the physical environment. This allows for a pre-classification of the site, as described in Table 1. In the state of São Paulo, such records are currently being set up and the investigation proposal is aimed mainly at the preservation of human health and environmental reserves. To expand this database to entire Brazil is a challenge that needs to be carefully planned.

Environmental agencies usually recommend gathering historical information and carry out a site inspection during the preliminary investigation. The real importance

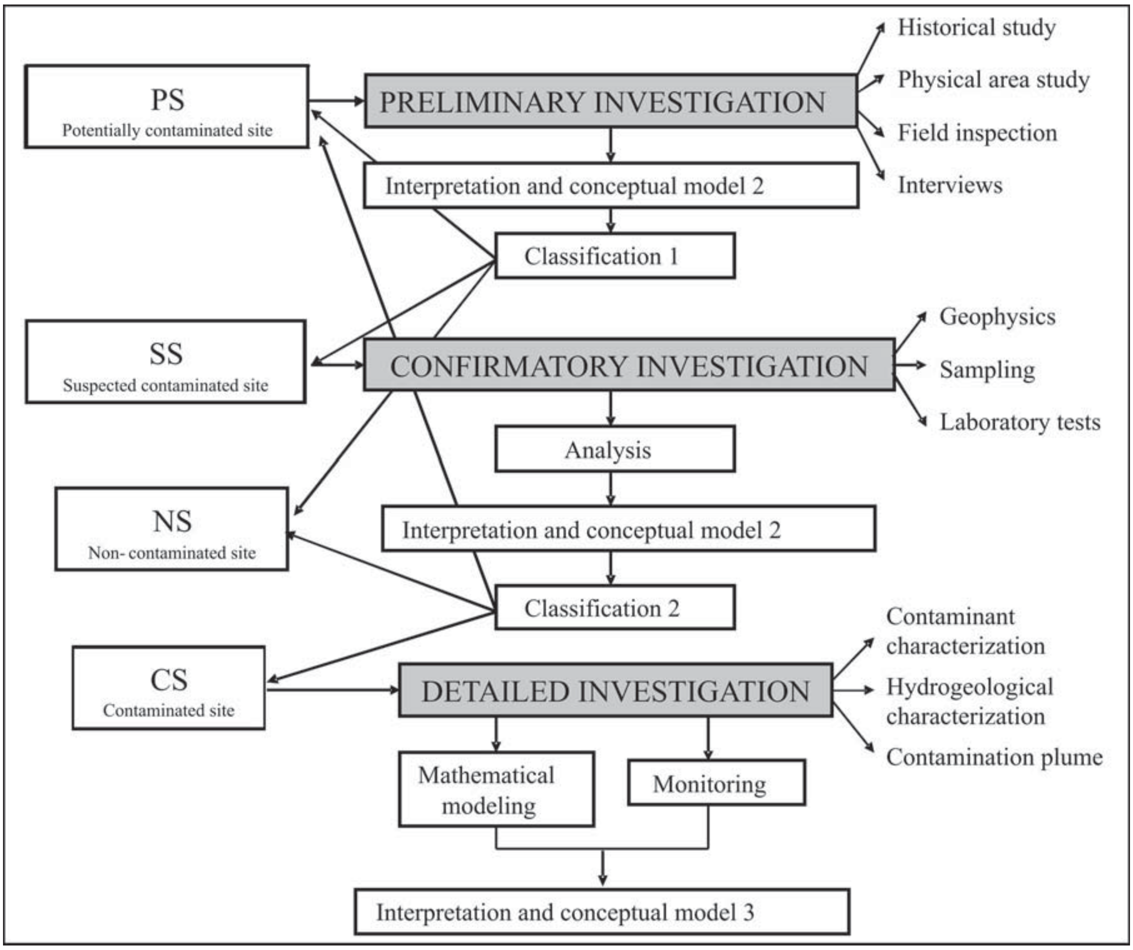

Figure 3 - Summary of different stages of geo-environmental investigation for potentially contaminated sites in São Paulo State, Brazil (CETESB, 1999).

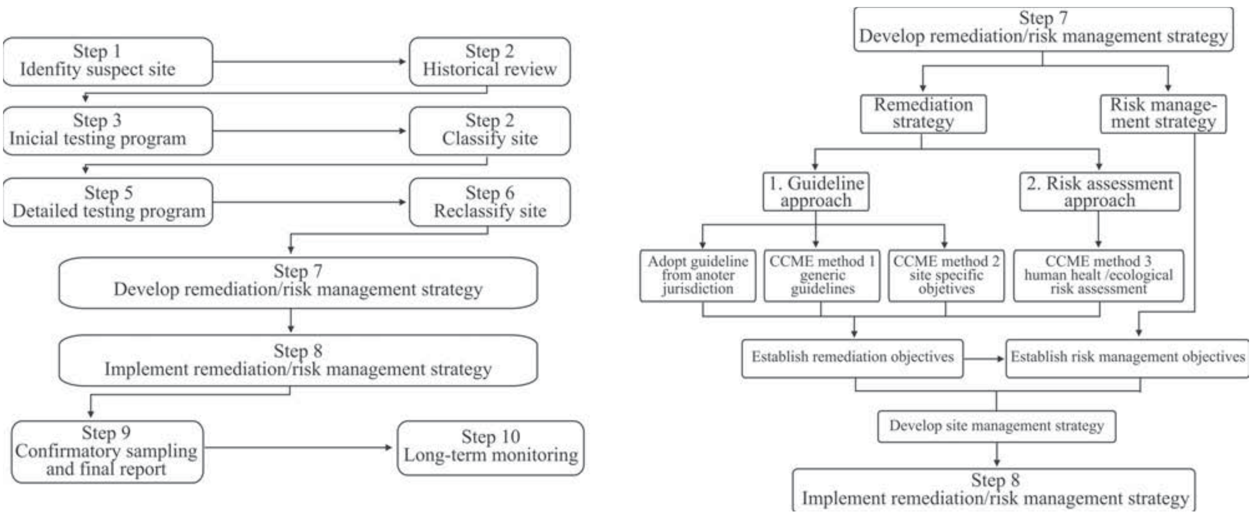

Figure 4 - Canadian proposal for geo-environmental investigation of potentially contaminated sites summarized in a chart (CSMWG, 2000).

Table 1 - Comparing CETESB and CCME regulations for geo-environmental investigation.

	Brazil	Canada
Agency	CETESB (Company of Environmental Sanitation Technology)	CCME (Canadian Council of Ministers of the Environment)
Publication year	1999	1997-2000
Scale	São Paulo State	Federal
Interest	Property and to give the initial step for a federal regulation about this subject	Governmental
Overlapping	Regulation includes notes about the national regulations, added for more detailed information	If there are Provincial and Federal regulations, the most rigorous is used
Objective	Contaminated sites management from São Paulo State	To preserve the natural ecosystem
Groundwater and surface water	Federal (CONAMA e Health Ministry for human supply)	Provincial and Federal (for agricultural use, animal and human supply and aquatic life)
Problems	Who is the guilty or responsible for the contamination?	Who is the guilty or responsible for the contamination?
	Figure 3	Figure 4
	1) Preliminary	1) National Classification System consulting
	2) Registration	2) Assessment of every information about the site
Investigation steps	3) Confirmatory - Geophysics - Sampling - Laboratory characterization	3) Site Investigation - Geophysics - Sampling - Laboratory and field characterization - Hydrogeological investigation - Flow model - Biological assessment
	4) Detailed - Contaminant characterization and hydrogeological investigation - Flow model	4) Detailed - Detailed identification of the limits and every contamination plume paths, to give subsidies for the remediation plan
	5) Risk assessment	5) Remediation Plan (guideline approach or/and risk assessment)
	6) Investigation for Remediation	6) Risk-based environmental management approach (for human health and natural environmental)
	7) Remediation	7) Remediation
Obligatory remediation	To prioritize the most affected sites and close to the major population densities and ecological reserves	Different levels to prioritize: agriculture, parks, residential, commercial and industrial areas
	It is going on now, applying investigation techniques during the different steps are previously described. The sites registration is being built based on detailed information about the sites with potential risk	Study is made with basis on existing site information. There are database and available information registrant. An initial classification can be made, considering the contaminants characteristics, the likely transport paths and the existence or not of targets
	- Potentially Contaminated Site (PS)	- Class 1 (70 to 100 points): action required
Classification	- Suspected Contaminated Site (SS)	- Class 2 (50 to 69,9 points): action likely required
	- Contaminated Site (CS)	- Class 3 (37 to 49,9 points): action may be required
	___	- Class N (< 37 points): action not likely required
	___	- Class I (≥ 15 points): insufficient information
Other aspects	___	- Training courses for workers and for who lives in or around the contaminated or remedied sites - Communication and warning plan to the media and ordinary citizens and residents of the site

of this step was confirmed by Mondelli (2008) in a geo-environmental investigation of the Bauru's municipal solid waste (MSW) disposal site. In this case, the preliminary investigation proved to be very important for the development of future studies, based on the interpretation of the previous and current topography, hydrology, aerial photographs and historical documents. The confirmatory investigation steps in Brazil and Canada differ slightly (Table 1). Both start by using surface geophysics methods, which has proved to be an important in situ testing technique for geo-environmental site investigation and monitoring. Although in Brazilian practice, it has mostly been used only after contamination is detected. In addition to soil and groundwater sampling and laboratory analyses, the Canadian approach includes the option of characterizing all the samples and performing flow modeling and biological assessment in the field.

The greatest difference between the approaches of the two countries is for detailed investigation. The Canadian regulations recommend determining all the requirements for carrying out remediation plans, while CETESB recommend the flow modeling and the hydrogeological and contaminants characterization. According to the CETESB procedures, remediation and/or recovery measures can only be implemented after the phase called "investigation for remediation" (Table 1). In Canada, whether remediation is required or not is one of the first questions to be answered when the site investigation is ongoing. It is important to emphasize herein that the first step is to detect the potential contaminants based on the land use history data. Based on that, site investigation may be required to verify if the levels of any contaminants exceed the maximum accepted values. This is when the question "Is remediation required?" gets asked. When a contaminated site has to be remediated, it can be very expensive and this action may not be effective for its intended use. In these cases, risk assessment came into practice as a way of justifying alternative approaches to deal with the contamination.

Canada's Subsurface Assessment Handbook for Contaminated Sites (CCME, 1994) lists the following investigative steps: review of information; surface and intrusive geophysics; hydraulic and hydrogeological information and monitoring wells; sampling of soil, gas and liquids; chemical, biological, geochemical and geotechnical analyses; mathematical modeling of the contamination plume; and investigation to monitor the remediation program.

Canada has experience in the investigation and control of contaminated sites with the development of modern techniques for site investigation. A very complete investigation program was conducted in a landfill in Cambridge, Ontario, in the late 70s, which was well documented by Dillon Limited (1989). The following steps were taken, which could serve as a reference for investigation of contaminated sites in Brazil:

1) Determine the possible sources and/or targets of contamination.

2) What was the site like before its use as a waste disposal site?

a. Topography, Hydrology, Geology (rocks and deposits), Hydrogeology (aquifers and wells for urban water supply).

b. Determination of the geological and geotechnical profile (mineralogy, grain size distribution, groundwater level and waste density).

3) Determination of changes in Topography, Hydrology, Geology (rocks and deposits), Hydrogeology (aquifers and wells for urban water supply) caused by construction and operation of the landfill.

a. What sorts of materials have been deposited in the landfill?

b. How have the properties of these materials changed since deposition and how are they interacting with the in situ materials and groundwater?

4) Field monitoring activities (monitoring wells, multilevel or not, inside and outside the landfill, sampling of soil, gas, landfill leachate, ground and surface water, *in situ* hydraulic field tests and laboratory characterization tests).

5) Hydraulic gradients, hydraulic conductivities (Hazen formula, pumping and rising-head tests using the monitoring wells), flow speeds, ground and surface flow.

6) Methane gas monitoring (study of the unsaturated-zone contamination plume).

7) Organic and inorganic leachate analysis.

8) Leachate generation study.

9) Assessment of the vegetation.

10) Environmental impact assessment (interpretation).

11) Conclusions and recommendations.

4. Geo-Environmental Investigation of A Municipal Solid Waste Disposal Site Installed In Brazil

Mondelli *et al.* (2007) studied an existing Municipal Solid Waste (MSW) disposal site at Bauru-SP, Brazil with the primary objective of assessing the applicability a wide variety of geo-environmental investigation techniques. Other focus of this study was to assess contamination of a medium size municipal solid waste disposal site, installed over a typical Brazilian tropical soil (residual soils and sandstones from Bauru Group).

A preliminary investigation program was carried out at the site and the previous and current topography, hydrology, aerial photographs and historical documents were used. Figure 5 summarizes the interpretation of all these data, to show an example of integrating all historical data, with tests carried out over the center of the landfill, to support the later investigation programs.

Surface geophysical investigation (geoelectrical survey) and resistivity piezocone penetration tests (RCPTU) were carried out, followed by soil sampling using direct-

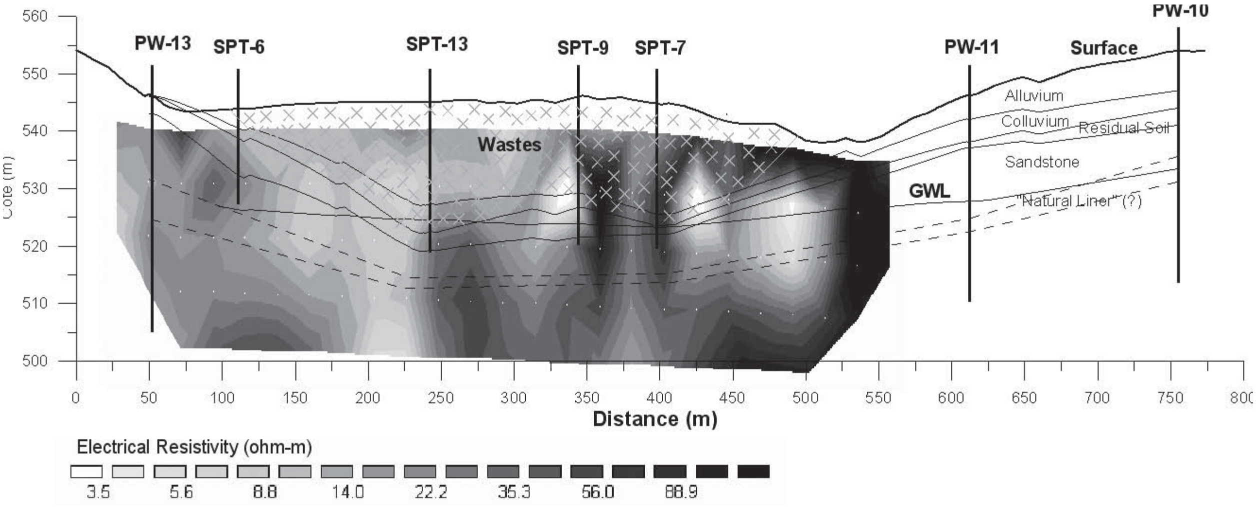

Figure 5 - Results of the dipole-dipole electrical profiling presented together with North-South profile including topographical and geological information (Mondelli *et al.*, 2008).

push samplers and water samples were collected from monitoring wells. The results showed that the application of the geoelectrical methods were indispensable for identifying the presence and flow direction of contamination plumes (leachate) as well as to indicate the most suitable locations for RCPTU tests and soil and water sampling. Chemical analyses of groundwater samples contributed to a better understanding of the flow of the contaminated plume.

A preliminary contamination plume was identified and difficulties in determining a typical soil profile, hydrogeological characteristics and background resistivity values encouraged continuing the site investigation using laboratory tests. The characterization tests included those for geotechnical properties determination and for soil and leachate chemistry (grain size distribution, X-Ray diffraction, blue methyl adsorption and differential thermal analysis). Batch and column tests were carried out using these same geomaterials in disturbed and undisturbed conditions, respectively (Mondelli, 2008). The landfill's leachate was used as a pollutant solution in the laboraratory tests and the dispersion and retardation coefficients were estimated. Electrical resistivity values were measured using different pore fluids and degrees of saturation. The weathering and the fines content of the studied tropical soils was shown to have a significant influence on the resistivity values. It was shown that the laboratory tests could improve the interpretation of the in situ tests and the delineation of the contamination plume occurring at the site (Mondelli, 2008). Figure 6 presents the variation range resistivity values interpreted for the local soils.

5. Suggestions For Geo-Environmental Investigation of Brazilian Contaminated Sites

Based on the review on geo-environmental site characterization and on the experience gained during the investigation of the Bauru MSW disposal site, briefly presented in this paper, a site investigation plan outlined in Fig. 7 is presented, which is considered appropriate for Brazilian contaminated sites. It consists of the following steps:

1) Define the objective of the site investigation.

2) Compile all the existing information about the site (physical and historical), including its topographic, geological, hydrographic, and photographic records and field inspections. This information will be useful for choosing the best investigation techniques, with special attention on additional information such as the position of the groundwater level, texture, evolution, mineralogy and resistance of local soils.

3) Geophysical survey is recommended at the beginning and during any site investigation program, regardless of the potential for or degree of contamination. The types of soils and contaminants should guide the decisions about the method(s) to be chosen.

4) Invasive tests: The choice of invasive technique will depend on the initial studies and on the compilation of all the available geotechnical and hydrogeological information about the site. If the existing information is insufficient, an investigation plan using different invasive techniques is recommended.

5) Direct testing like SPT soundings for subsoil characterization or disturbed soil sampling can be carried out and has been widely used for geotechnical site investigations in Brazil. This test can also be carried out during the construction of monitoring wells.

6) In areas with soft soil and/or clean sand and shallow groundwater levels, which are very common in Brazilian coast, piezocone tests with complementary tools (such as resistivity, pH sensors, MIP detectors and soil and water sampling) are very useful and their uses should be encouraged. These versatile invasive tools cause low

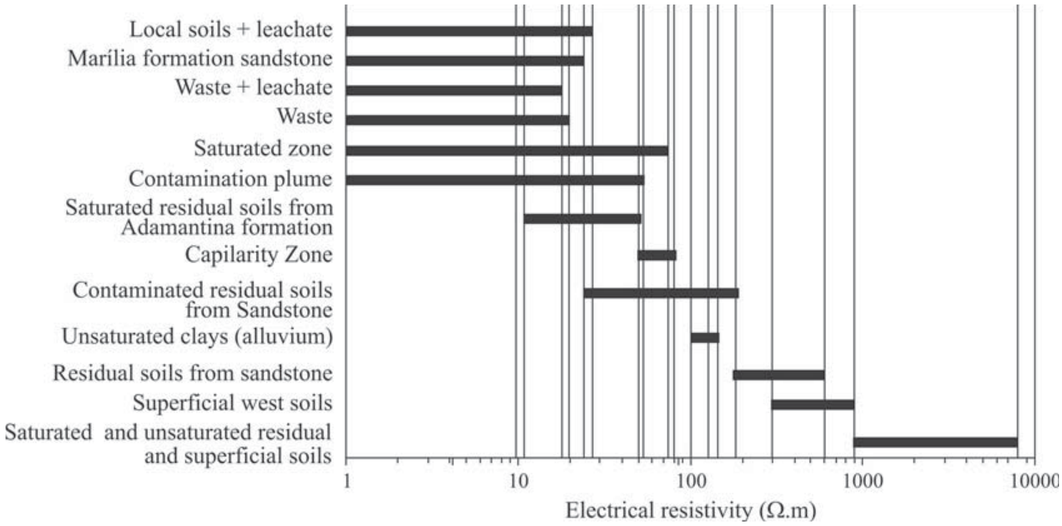

Figure 6 - Range of resistivity values for the study MSW disposal site, based on integration of field and laboratory testing data.

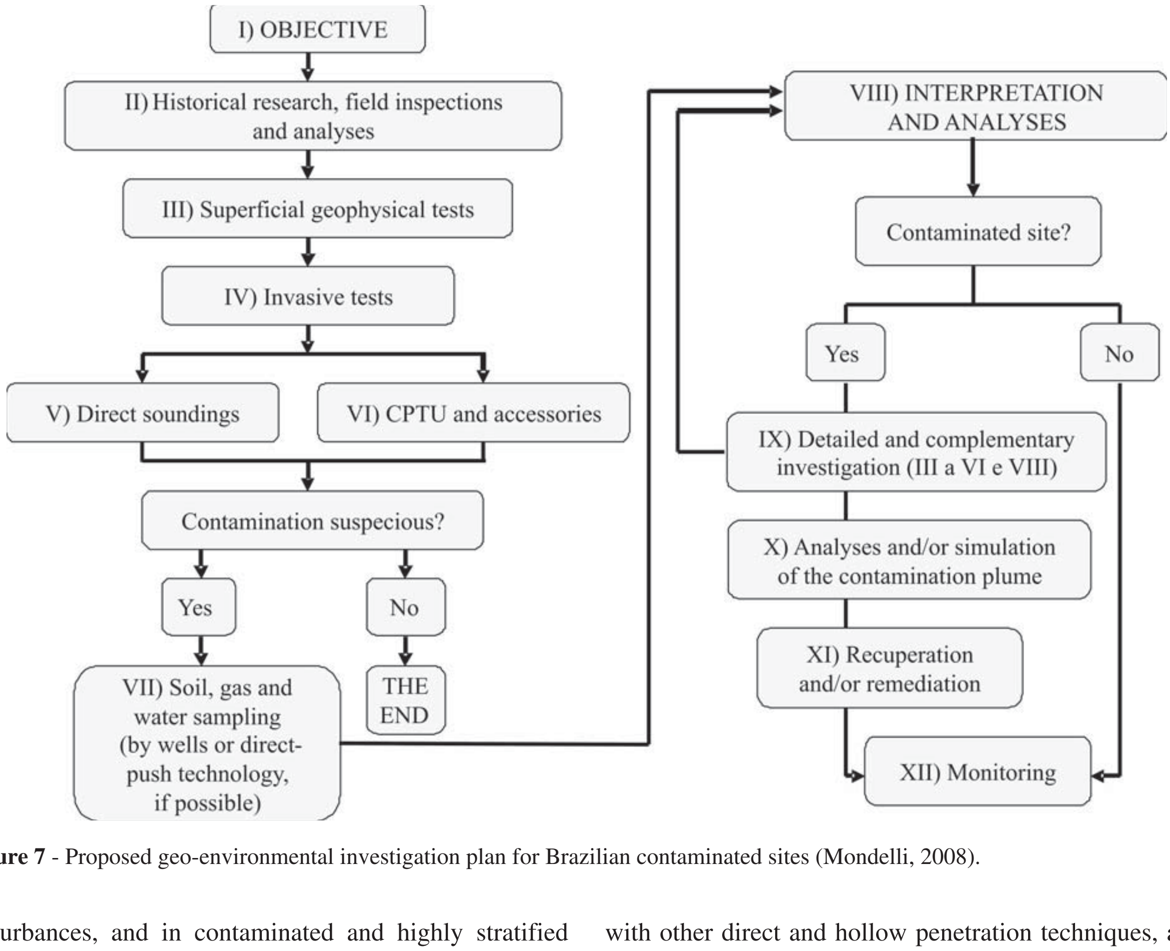

Figure 7 - Proposed geo-environmental investigation plan for Brazilian contaminated sites (Mondelli, 2008).

disturbances, and in contaminated and highly stratified soils allow detailed profiling with high reliability, proving to be very effective for geotechnical and geo-environmental investigations (De Mio *et al.*, 2005). For tropical soils, which can be highly resistant and the groundwater is usually deep, the piezocone test could be used in combination with other direct and hollow penetration techniques, as it was done for the Bauru's MSW disposal site.

7) In areas where geophysical or preliminary tests indicated anomalies or possible contamination, soil, water and gas sampling is recommended for geotechnical and geochemical characterization. This procedure is very im-

portant to confirm contamination. Atterberg limits, grain size distribution and X-ray diffraction can be considered the most appropriate tests for tropical soils for geotechnical characterization based on laboratory tests. The recommended geochemical parameters to monitoring are: temperature, electrical conductivity, pH, Eh and specific tests for possible contaminants likely to be encountered at the site, which are diagnosed in step II and III. With regard to gas sampling, a preliminary identification is required, including propagation pathways and direct contact with individuals that work or live around the site. Monitoring techniques, such as the monitoring wells installation and geophysical survey information, should also be used as the basis for future remediation and recovery plans.

8) An integrated interpretation from the various site investigations testing data is required to define the geological-geotechnical profile and the spatial distribution of the contaminants. The constituents of the contaminants will determine the parameters to be analyzed in geochemical characterization of groundwater and air.

9) A detailed and/or complementary site investigation should be carried out if contamination has been confirmed or if additional data are needed for a conclusive interpretation. This will entail additional sampling of soil, water and gas, laboratory tests to estimate the parameters of pollutant transportation, and additional *in situ* tests during this phase of the site investigation program.

10) After definition of the conceptual model, the contamination (s) plume(s) can be analyzed based on numerical flow modeling and/or on geophysical test results.

11) Remediation plan and remediation phase: If financial resources are insufficient or there is no interest in remediation at that time, a simple plan can be drawn up to recuperate the site or close it.If financial and interest support exists, the remediation plan should be carried out based on risk assessment results.

12) A monitoring plan using surface geophysics and/or monitoring wells is recommended for almost all cases where there is the slightest suspicion of contamination.

6. Conclusions

This paper presents a brief review on geo-environmental site investigation, in order to direct Brazilian environmental specialists to provide an effectiveness investigation plan, considering the practical difficulty of this work for site-specific conditions and different contamination types. It is important to combine the experience of environmental agencies with the knowledge of highly qualified experts, who could benefit from their experience in tackling practical problems, especially for multidisciplinary geo-environmental issues.

In the maintenance of the equilibrium of the natural environment, Canada's environmental agency is also concerned with human health, thus requiring the investigation and remediation of any contamination. The situation in Brazil differs from that of Canada, since only the state of São Paulo has a complete set of environmental regulations, which are primarily concerned with human health and areas of environmental protection. Moreover, Brazil does not have the culture of preservation and people often do not believe in the existence of environmental contamination or do not understand the risks involved on it.

The suggested geo-environmental investigation plan presented herein for Brazilian contaminated sites includes the combination of direct and indirect techniques, since mineralogy and the degree of evolution of tropical soils give rise to a very particular behavior. Therefore, even the guidelines listed here should be followed with restrictions in view of Brazil's wide variety of soils and climates, and should always be complemented by local experience. Indirect tests should be applicable always before sampling and laboratory tests in a site investigation program, to support the decision of their position, number and depth.

The location of monitoring wells based only on topographic and preferential groundwater flow information does not always meet the objective of a geo-environmental site investigation plan, since the contamination plume(s) is(are) influenced by the form, time and location of contamination source. In addition, tropical regions commonly show a complex hydrogeology and a heterogeneous geological evolution of natural materials, as presented herein in the study case of the Bauru's MSW disposal site. The combination and interpretation of different investigative techniques enables these variables to be diagnosed, allowing monitoring wells to be repositioned based on more detailed information, including previously detected contamination spots, and aiding in the interpretation.

It is advisable to use indirect testing methods such as electrical resistivity tests at the outset of any work on contaminated or even potentially contaminated sites to avoid direct contact with contaminants, *e.g.*, domestic or industrial landfills and hazardous waste dumps, since they can provide background values. This step provides a broad picture, facilitating the solution to future problems, especially with regard to the waste disposal on tropical soils, which are highly heterogeneous, and reducing the future investigations costs.

Acknowledgments

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Technical Note

Soils and Rocks
v. 33, n. 3

The Use of Fuzzy Sets in the Evaluation of the Effectiveness of Earth Dam Reinforcement

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Abstract. This paper shows the use of fuzzy logic to evaluate the effectiveness of the reinforcement of Santa Branca dam. The embankment reinforcement was found imperative once the dam had shown signs of instability process, as undesirable seepage and unexpected movements, just after the first impoundment. Stability analyses were carried out considering fuzzy variation of the strength parameters in order to assessing the non-random uncertainties associated with their evaluation. These uncertainties are mostly connected to values varying within a specified interval, whose boundaries express the possibility of occurrences that, in turns, account for vagueness in information. Finally, the fuzzy Factor of Safety distribution resulting from these analyses is then compared to pre-defined fuzzy class intervals in order to classify the failure potentiality of the embankment, before and after the reinforcement has been constructed, reflecting, thus, its effectiveness.

Keywords: fuzzy sets, earth dam, stability analysis.

1. Introduction

Generally, earth dams have been designed for multiple propose applications; however power generation and fresh-water storage are amongst the main objective of these structures.

Unfortunately, in developing countries, due to lack of specific laws, a considerable number of these dams have not always been built with the desirable safety and care, putting in risk goods and lives of people who live downstream.

Several factors, associated to the earth embankment alone, can lead a dam to undermine. Unsuitable materials used as fill and filter, poor compaction, use of overestimated parameters for stability analysis, erroneous design of the spillway (overtopping) and animal activities, can be pointed out as the main reasons for failures and incidents of earth dams.

Stability analysis of an embankment dam is normally made using well-recognized methods where input parameters are derived from investigation using deterministic method. Statistical approach is less common nevertheless nowadays its use is gradually becoming popular in geotechnical field.

However, in order to take into account uncertainties that are not originated from random, but yes, from cognitive errors, it has been proposed herein the use of fuzzy sets concept.

This methodology was, thus, used to assess the effectiveness of the embankment reinforcement used to improve the stability of Santa Branca dam, built in 1959 in Sao Paulo, Brazil, which, since its construction, has been showing worrying signs of instability.

After the occurrence of unexpected important leakages followed by undesirable movements, experts concluded that Santa Branca dam had to be reinforced in order to reach a stable condition.

Regarding that the reinforcement comprised itself as an additional flatter embankment placed at downstream part of the dam, in which the main aim was to improve the stability of the dam, the new factor of safety could be evaluated using standard stability analysis procedures. However, for a more comprehensive analysis, it seems to be desirable to take into account the intrinsic uncertainties of the parameters used in such analysis. In addition, if these uncertainties are of random-type, this can be done using the well-known reliability approaches. However, in case in which uncertainties are of cognitive source, the approach is more effective when based on possibility analysis instead of probabilistic one.

Therefore, in order to allow for vagueness in information and consider this feature in a stability analysis, Fuzzy Set theory can be used as a tool to "classify" the results obtained before and after the installation of the reinforcement, clearly pointing out, thus, its effectiveness by means of linguistic terms instead using numbers only.

2. Fuzzy Logic and Sets

Around 350 b.C., a Greek philosopher Aristoteles has established a Logical Science which is a set of rigid rules to logically validate conclusions. Later, this was become known as Occidental Logic that is based on "false" or "true" statement.

However, in many situations, problems cannot be solved considering only binary approach, because of the presence of ambiguities and vagueness forcing, thus, the answer to be somewhere between "true" or "false" statement.

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In order to overcome this situation and to take into account the ambiguity raised in several common engineering situations, Zadeh (1965) developed a methodology based on fuzzy sets theory, which considers states between "true" and "false" interval. This is done through the degree of belief that is inferred by so-called membership functions that varies from 0 to 1.

The membership function $\alpha_a(x)$ for a fuzzy set A can be defined as:

$$\alpha_a(x): X \to [0,1] \quad (1)$$

where X is the Universe of Discourse.

Membership function reflects the degree of belief of x in A. If $\alpha(a)$ is equal to zero the object x definitely does not belong to X. On the other hand if $\alpha(a)$ is equal to unity, the object x fully belongs to X. Values between zero and unity imply that x partially belongs to X with a degree equals to $\alpha(a)$ The membership function can be represented by convex shapes, but the most common are of triangular and trapezoidal. The higher is the membership function value, the more possible is the occurrence of it. The width of the interval defines its possible variation, which, in turn, reflects the embedded uncertainty.

This method is very suitable to be used in geotechnical engineering where problems are commonly defined by means linguistic terms to treat vagueness. Therefore, the cognitive uncertainties that are always present in design and evaluation can be assessed. This technique allows the mathematic to mimic the human thinking during the process of decision-making. This is particularly appropriated in geotechnical engineering where linguistic terms are commonly used to define the statement of a soil mass as compressibility and density or to define conditions as stability and strength, among others (Giasi *et al.*, 2003).

However, despite its enormous advantage, Fuzzy Sets Theory is not commonly used in geotechnical stability analysis. This can be attributed to the fact that engineers are trained and used to interpreting numbers and Fuzzy Theory deals with, basically, linguistic terms (Dodagoudar & Venkatachalam, 2000)

This paper has as the main aim to show the potentiality of Fuzzy Sets Theory in geotechnical and dam engineering, provided that it is a different manner of interpreting results using possible interval of occurrences instead using classic statistical probability to infer the impact of the uncertainties on final results.

3. Methodology

Santa Branca Dam, built in State of São Paulo, southeastern of Brazil, had its construction completed in the 1959. It comprises a 55 m high dam with a crest extension of 320 m. The cross section of the dam is composed of compacted clayey soil with a vertical and horizontal traditional sand drain/filter system. The upper third portion of the upstream slope is protected with a thin layer of rockfill and the downstream slope has grass as a protective measure against erosion. Details can be reached in (Dell'avanzi, 1995).

After the first reservoir filling in 1959, unexpected problems were observed, including unwanted infiltration with water emerging from downstream slope surface with worrying signs of subsidence. Table 1 gives some details of all occurrences since the first filling.

Therefore, based on these occurrences it was found that the drainage system was not fully operational, showing that repair was necessary followed by a complete reinforcement of downstream slope.

After the expert surveillance conclusion that the vertical filter was not properly operating, for some unknown reason, remedial actions were lately taken only after 1989, when the situation appeared to have reached a critical stage. Thus, a major reinforcement was built in order to increase the dam safety to an acceptable level. This reinforcement consisted basically of an additional downstream embankment covering all over the existing downstream embankment, with flatter slope. A drainage element between the old and the new embankments was placed to keep ground water surface low and to prevent further leakage and piping, as well.

Concerning that Santa Branca Dam had not an instrumentation program to furnish in-situ data, the assessment of the efficiency of the reinforcement had to be checked using traditional methods as slope stability analysis and other less popular such as statistical analysis.

Table 1 - Occurrences observed during the 20 years of the dam's life.

February, 1962	Two shallow failures take place in downstream slope due to saturation
January, 1965	Another two shallow failures appear in downstream slope, but in a different point from those past ones
December 1966	A new shallow failure in the central portion of the downstream slope, takes place
Mar 1967	Water emerging from the central part of the downstream slope
August, 1967	Water emerging from the central part of the downstream slope just above the first berm
February, 1969	Soaking wet spot becomes visible at left abutment
July, 1969	Water emerging again from the central portion of the downstream slope, at mid height of the embankment
August, 1971	Soaked zoned above level 595 m. Important subsidence is found at level 615 m in the central portion of the embankment
November,1983	Soaked zones are found in the abutments and on the central part of the dam. A "waved" deformed surface at the dam crest has been observed

Due to the lack of a comprehensive geotechnical investigation, the main variables involved in analysis had their values estimated by experts. This has stimulated the use of a method based on "possibility" instead of those methods based on "probability" theory. Therefore, the Fuzzy analysis was chosen as a tool suitable for this kind of approach.

In such analysis the variation of the input parameters shall be based on interval of possible values, where a pair of parameters is considered for several membership degrees, varying from 0 to 1.

Therefore, the chosen parameters to be used in the assessment of the reinforcement effectiveness are those believed to have stronger influence on the stability of the embankment, that are: effective cohesion, shear strength angle and reservoir level. Their respective statistical values, from where the Fuzzy distribution was defined, are presented on Table 2.

Thus, a triangular fuzzy distribution is proposed herein in order to take into account the possible values assumed for the variables shown in Table 1. The narrower is the membership triangle function the lesser is the expected variability of the parameters.

The fuzzy distributions for reservoir water level, shear strength angle and cohesion, used in stability analysis are presented in Figs. 1, 2 and 3, respectively.

4. Fuzzy Assessment of Factor of Safety (Fos)

For each membership degree, all parameter values are cross-combined and the resulting factor of safety is then evaluated. Thus, the full analysis involves 2n combination of parameters for each membership function α and n is the number of parameters considered. In the case shown herein five membership degrees were considerable (for simplicity), forming, thus, 33 different combinations that are necessary to complete the analysis, which were carried out using the software Geoslope^{R}, as shown in Table 3. This technique is known as "vertex method" and is fully explained by Juang *et al.* (1998).

After all calculations were performed, the obtained Factors of Safety were sorted in crescent order. After that, both major and minor Factors of Safety are picked up for each membership function and a resulting fuzzy distribution is then defined, as shown in Fig. 4. It can be noticed that, due to the fact that parameters vary in a linear way, the output data can be expected to vary in the similar manner resulting in a regular triangular distribution.

Table 2 - Parameters to be used in stability analysis (Dell'avanzi, 1995).

Parameter	Expected values	Standard deviation
c' (kPa)	83.61	17.05
φ' (degree))	24.43	1.11
Water level (m)	40	1.0

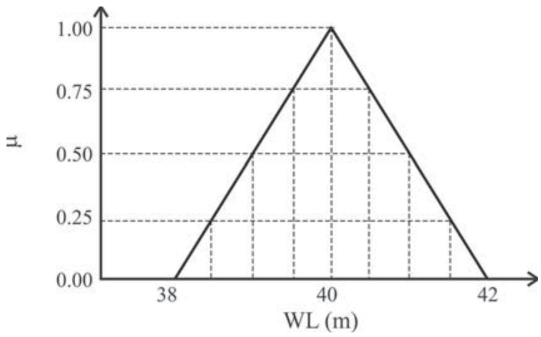

Figure 1 - Fuzzy distribution of the reservoir level (WL).

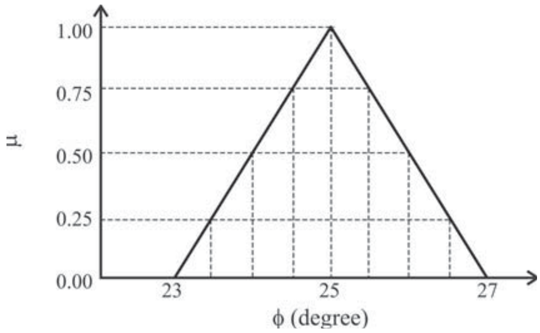

Figure 2 - Fuzzy distribution of the shear strength angle of the dam material.

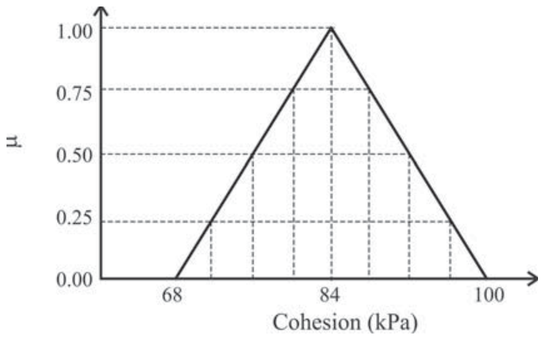

Figure 3 - Fuzzy distribution of cohesion of the dam material.

4.1. Stability state and resemblance degree

The next step to be followed is to compare the resulting fuzzy distribution with pre-defined fuzzy arrangement that describes the "stability state" in order to come up with a final comparison based on so-called "resemblance degree".

Stability state (Fig. 5) is a set of fuzzy distribution defined based on expert's opinion of engineers and geologist that identifies states of stability according to the factor of safety (Juang, 1998). The main objective of the stability state distribution is to create a pattern that can be understood as a standard model in order to classify the obtained fuzzy distribution of the Factor of Safety. This is the subjective portion of the analysis and therefore it strongly depends on judgment (and expertise) and, therefore, it can be

Table 3 - Cross-combined values for each membership degree.

Membership degree	Combined values (WL, C, φ)
μ(0.00)	(38, 68, 23), (38, 68, 27), (38, 100, 23), (38, 100, 27), (42, 68, 23), (42, 68, 27), (42, 100, 23), (42, 100, 27)
μ(0.25)	(38.5, 72, 23.5), (38.5, 72, 26.5), (38.5, 96, 23.5), (38.5, 96, 26.5), (41.5, 72, 23.5), (41.5, 72, 26.5), (41.5, 96, 23.5), (41.5, 96, 26.5)
μ(0.50)	(39, 76, 24), (39, 76, 26), (39, 92, 24), (39, 92, 26), (41, 76, 24), (41, 76, 26), (41, 92, 24), (41, 92, 26)
μ(0.75)	(39.5, 80, 24.5), (39.5, 80, 25.5), (39.5, 88, 24.5), (39.5, 88, 25.5), (40.5, 80, 24.5), (40.5, 80, 24.5), (40.5, 88, 24.5), (40.5, 88, 25.5)
μ(1.00)	(40, 84, 25)

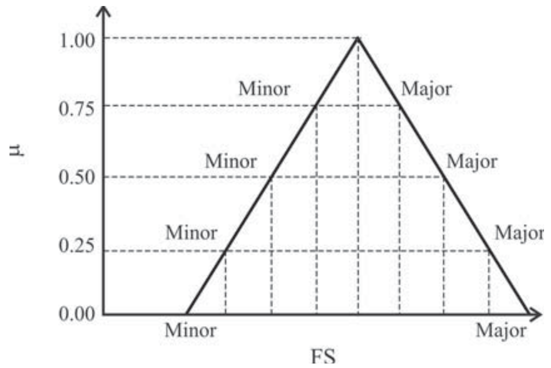

Figure 4 - Definition of Fuzzy distribution from resulting analysis.

changed for adjustment any time during the calibration. Herein the Stability State was divided in four categories as f (failure); pf (probable failure), ps (probable stable) and s stable. The number of categories and also their defined range are (analyst) dependent on the operator decision that constitutes itself the subjective part of the analysis (judgment). Figure 6, aiming to illustrate the method, shows a hypothetical comparison between obtained and pre-defined fuzzy distributions of Factor of Safety.

After superimposing the obtained fuzzy distribution on pre-defined pattern it is now necessary to use an algorithm in order to find which pre-defined distribution is most similar to that obtained one. This is known as resemblance degree.

The simplest way to carry this analysis is to subdivide the distributions in t equal parts and to compare the summation of the correspondent membership degree to each pre-defined FoS distribution. However, if the obtained distribution has other shape than symmetrical, the division must be

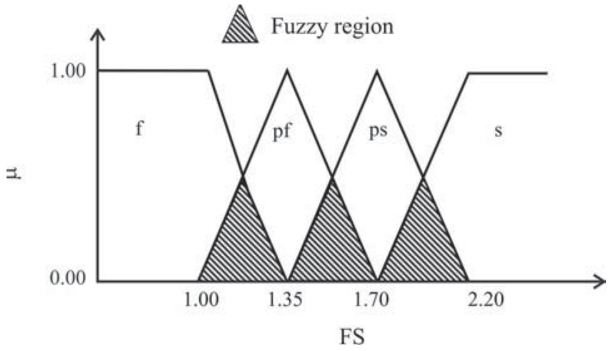

Figure 5 - Pre-defined rules for stability state.

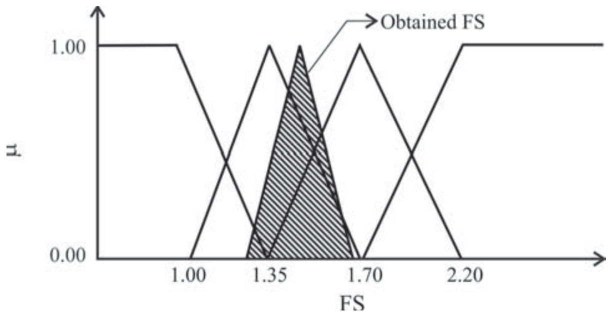

Figure 6 - Hypothetical superimposition of obtained FoS.

done for each side of the distribution in equal parts. Figure 7 shows details of this operation whose algorithm is given by Eq. (2).

$$S_j = \sum_{j=1}^{2t+1} \mu(fsp) \times \mu(fsr) \tag{2}$$

where j refers to the number of attributes and t is the number of subdivisions of each class of factor of safety. $\mu(fsp)$ represents the membership degree of the factor of safety corresponding to pre-defined curve and $\mu(fsr)$ is the membership degree of the resulting factor of safety, after the analysis had been carried out.

In order to define the percentage that describes how the obtained distribution of stability state is most similar to the predefined one, Eq. (3) is used.

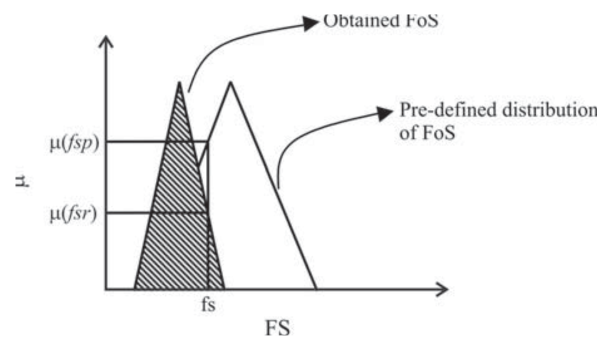

Figure 7 - Example of individual analysis.

$$w_j = \frac{S_j}{\sum_{1}^{4} S_j} \tag{3}$$

Therefore the highest values will point out the stability state amongst those represented in Fig. 5.

This methodology was, then, used for the Santa Branca Dam in order to assess the stability state before and after the reinforcement was placed.

5. Results and Discussions

Following the parameters cross-combination, stability analyses were carried out and the resulting factors of safety are presented in Table 4 for different membership degrees. It is important to mention that the values in bold are those corresponding to the maximum and minimum factor of safety for that specific membership degree, which were used to construct the final fuzzy distribution, shown in Fig. 8.

By comparing the fuzzy distribution for the factor of safety obtained from the analyses before and after the reinforcement, it is clear to verify that the reinforcement really improved the stability conditions of the dam. Before the placement of the reinforcement the resemblance degree calculated using Eqs (2) and (3) was around 61,2% for the possible failure distribution, meanwhile, after the reinforcement, the resemblance degree was around 69,4% of the stable distribution (Table 5).

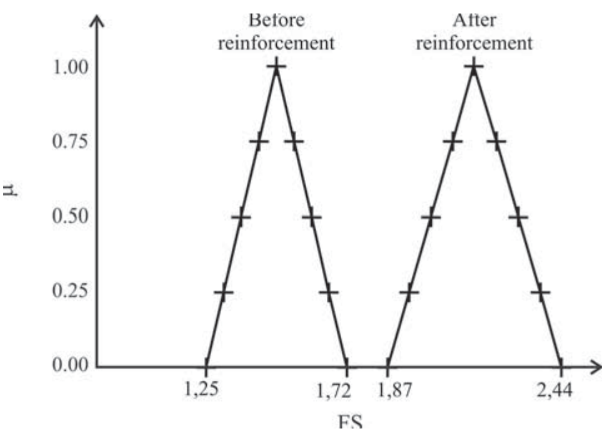

Figure 8 - Final fuzzy distribution of the Factor of Safety for Santa Branca Dam.

Table 4 - Factor of safety before and after the reinforcement.

Membership degree	(WL, C, φ)	FoS before reinforcement	FoS after reinforcement
μ(0.00)	(38, 68, 23)	1.284	1.858
	(38, 68, 27)	1.405	2.113
	(38, 100, 23)	1.594	2.198
	(38, 100, 27)	1.720	2.437
	(42, 68, 23)	1.249	1.856
	(42, 68, 27)	1.365	2.090
	(42, 100, 23)	1.562	2.180
	(42, 100, 27)	1.681	2.414
μ(0.25)	(38.5, 72, 23.5)	1.333	1.946
	(38.5, 72, 26.5)	1.423	2.125
	(38.5, 96, 23.5)	1.565	2.188
	(38.5, 96, 26.5)	1.659	2.367
	(41.5, 72, 23.5)	1.308	1.928
	(41.5, 72, 26.5)	1.396	2.104
	(41.5, 96, 23.5)	1.542	2.171
	(41.5, 96, 26.5)	1.632	2.347
μ(0.50)	(39, 76, 24)	1.384	2.013
	(39, 76, 26)	1.445	2.132
	(39, 92, 24)	1.541	2.174
	(39, 92, 26)	1.602	2.293
	(41, 76, 24)	1.366	2.001
	(41, 76, 26)	1.425	2.119
	(41, 92, 24)	1.522	2.163
	(41, 92, 26)	1.582	2.281
μ(0.75)	(39.5, 80, 24.5)	1.434	2.080
	(39.5, 80, 25.5)	1.449	2.139
	(39.5, 88, 24.5)	1.512	2.161
	(39.5, 88, 25.5)	1.543	2.220
	(40.5, 80, 24.5)	1.426	2.074
	(40.5, 80, 24.5)	1.455	2.133
	(40.5, 88, 24.5)	1.503	2.155
	(40.5, 88, 25.5)	1.534	2.214
μ(1.00)	(40, 84, 25)	1.484	2.144

Table 5 - Resemblance degree before and after reinforcement.

Resemblance degree – before reinforcement (in %)	Resemblance degree – after reinforcement (in %)	Stabilty state
0,44	0	Failure
61,12	0	**Possible Failure**
38,44	30.52	Possible Stable
0	**69.48**	**Stable**

6. Conclusions

Fuzzy Logic is a methodology that uses possibility distribution instead the well-known probability-statistical method. The main difference between both is that Fuzzy Logic does not lunch the concept of probability distribution of random parameters. Instead, it uses the concept based on that an interval between two extreme values embraces a possibility to happen. This gives the engineers freedom to use their experience and judgment to evaluate a boundary in which values are acceptable to vary.

Herein, the parameters were chosen to be the water level and the strength parameters C and ϕ, whose boundaries were evaluated from former published researches.

By cross-combining these values, the final stability state for Santa Branca dam, comprising periods before and after the reinforcement, were evaluated and the results are quite satisfactory indicating, thus, a considerable improvement in the stability of the dam.

Acknowledgments

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Symbol List

$\mu(\ )$: Membership degree

$\mu(fsp)$: Membership degree of the factor of safety corresponding to pre-defined curve

$\mu(fsr)$: Membership degree of the resulting factor of safety

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Book with editors: Sharma, H.D.; Dukes, M.T. & Olsen, D.M. (1990) Field measurements of dynamic moduli and Poisson's ratios of refuse and underlying soils at a landfill site. Landva A. & Knowles, G.D. (eds) Geotechnics of Waste Fills - Theory and Practice, American Society for Testing and Materials - STP 1070, Philadelphia, p. 57-70.

Proceedings (printed matter or CD-ROM): Jamiolkowski, M.; Ladd, C.C.; Germaine, J.T & Lancellotta, R. (1985) New developments in field and laboratory testing of soils. Proc. 11th Int. Conf. on Soil Mech. and Found. Engn., ISSMFE, San Francisco, v. 1, pp. 57-153.(specify if CD – ROM)

Thesis and dissertations: Lee, K.L. (1965) Triaxial Compressive Strength of Saturated Sands Under Seismic Loading Conditions. PhD Dissertation, Department of Civil Engineering, University of California, Berkeley, 521 p.

Standards: ASTM (2003) Standard Test Method for Particle Size Analysis of Soils - D 422-63. ASTM International, West Conshohocken, Pennsylvania, USA, 8 p.

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