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-

Manuel de Matos Fernandes, Full Professor, Faculty of Engineering, University of Porto, Porto, Portugal, e-mail: mfern@fe.up.pt.
Submitted on ; Final Acceptance on ; Discussion open until April 29, 2011.

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

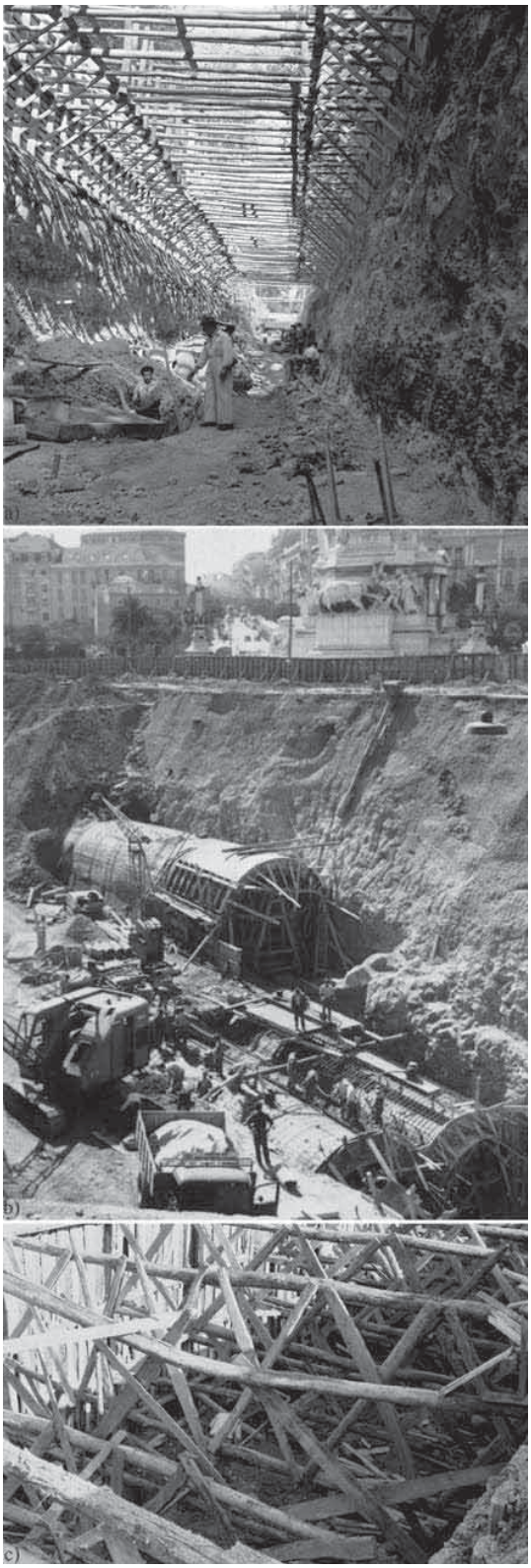


Figure 1 - Construction of the 1st phase of Lisbon Metro (1955-1959): a) Av. Liberdade (1955); b) Praça Marquês de Pombal (1956); c) Av. República (1956).

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

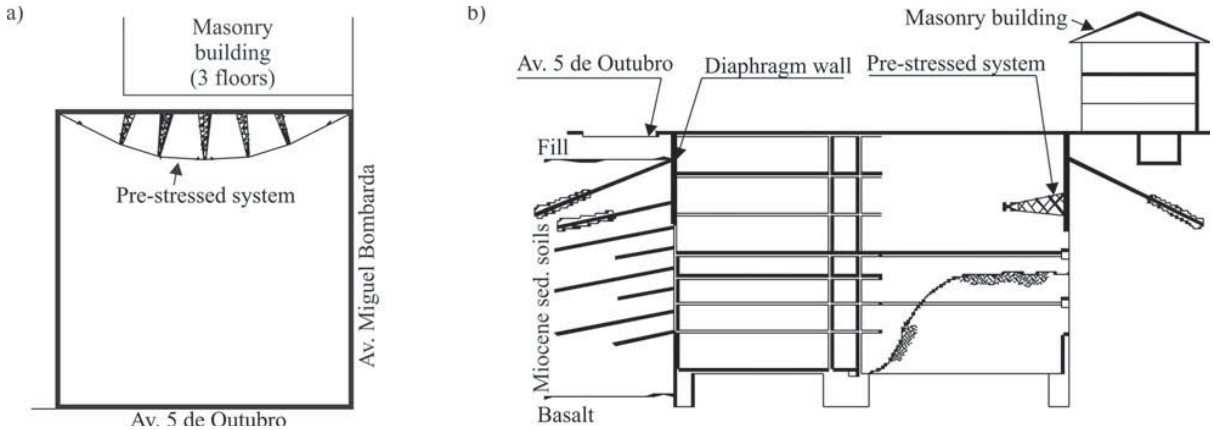


Figure 2 - a) Site plan; b) cross section perpendicular to Av. 5 de Outubro.

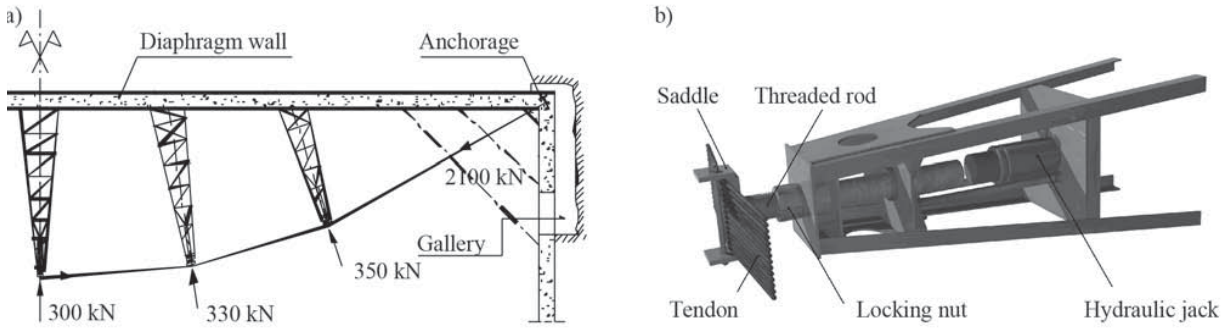


Figure 3 - Pre-stressed structural system: a) plan; b) 3D view of a strut head.

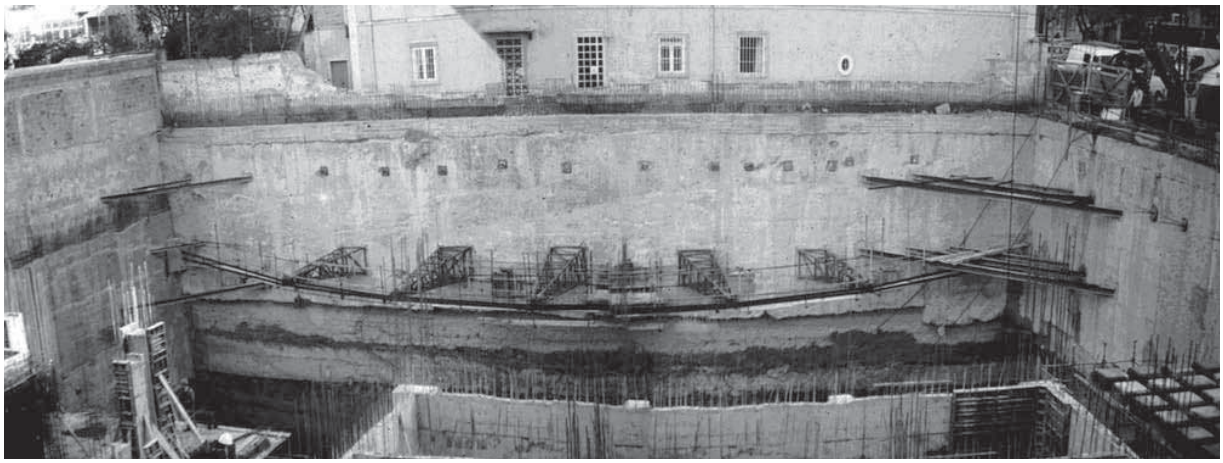


Figure 4 - General view of the solution.

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

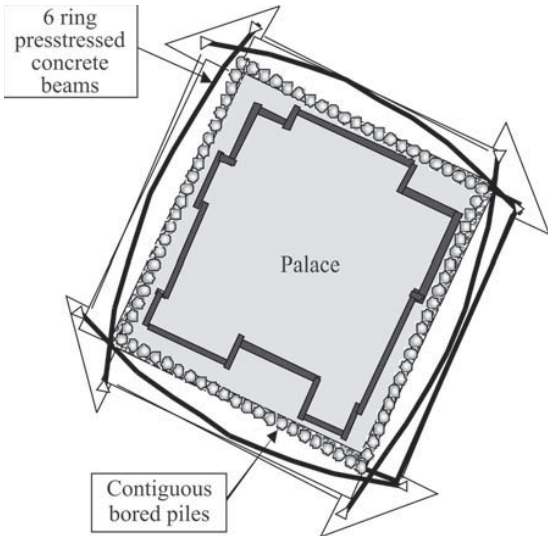


Figure 6 - Plan of the retaining structure of the excavation face adjacent to the Palace (Pinto *et al.*, 2001).

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 retensioning, 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).

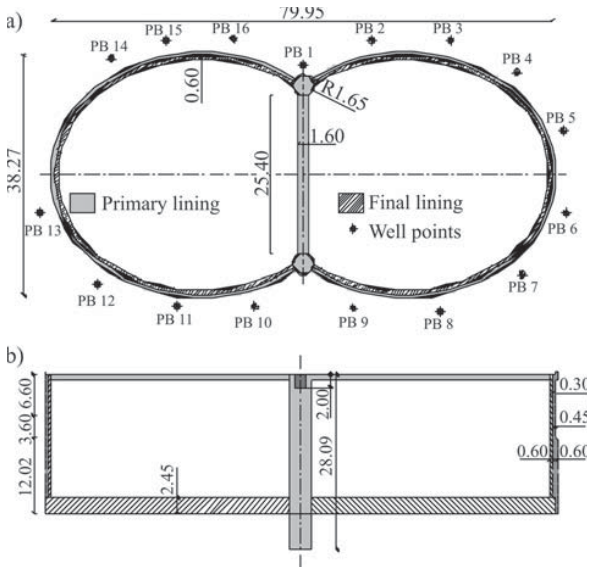


Figure 8 - Salgueiros Station of Porto Metro – plan and section on the longitudinal axis of the station (Topa Gomes, 2008).

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.

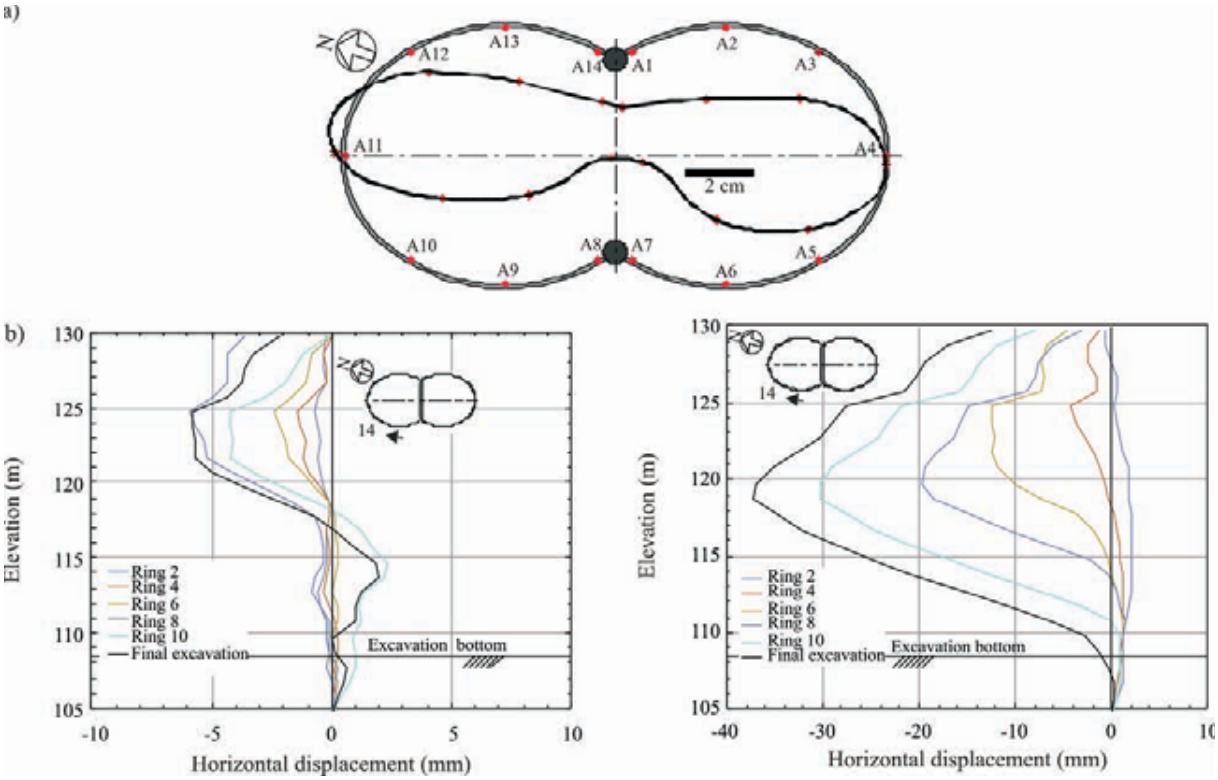


Figure 9 - Monitoring results of Salgueiros Station of Porto Metro (Topa Gomes, 2008): a) deformed shape at the level of ring 4 (9.0 m deep); b) horizontal displacements at a distance of 2.0 m from the cut face, on the direction parallel to the longitudinal axis (on the left) and on the direction normal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

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

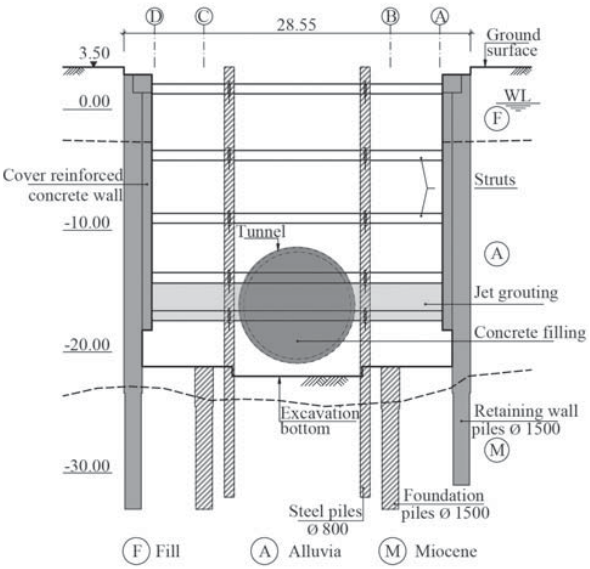


Figure 11 - Cross section of the excavation and of the retaining structure.

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.

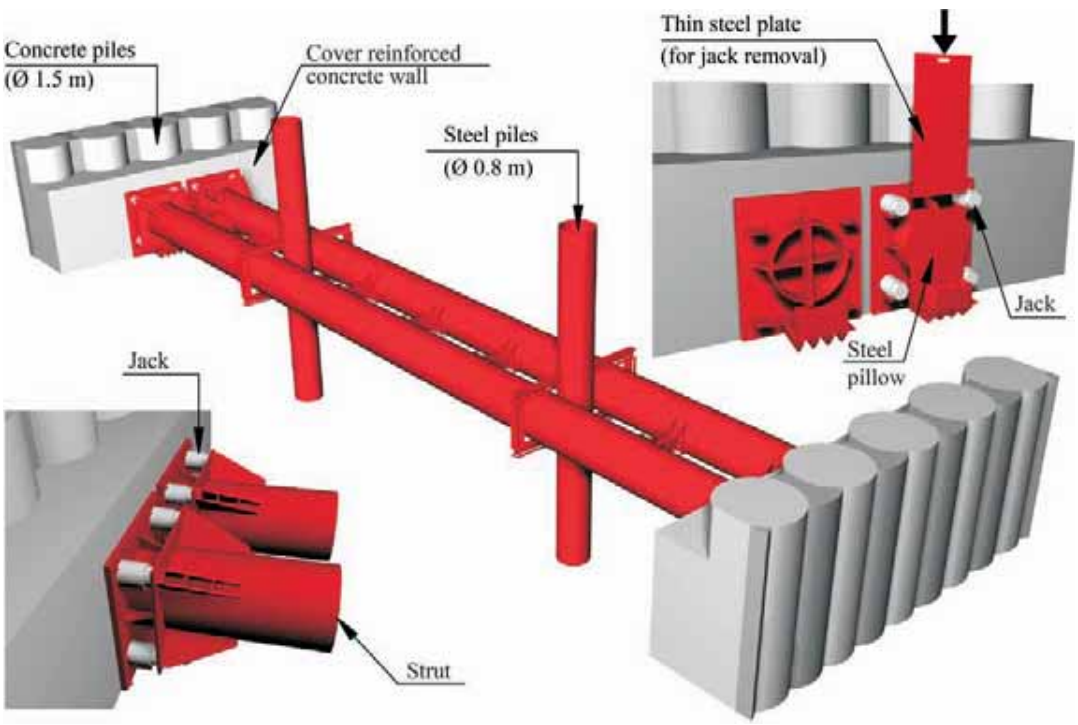


Figure 12 - Strut system: structural arrangement and details of the active strut head.

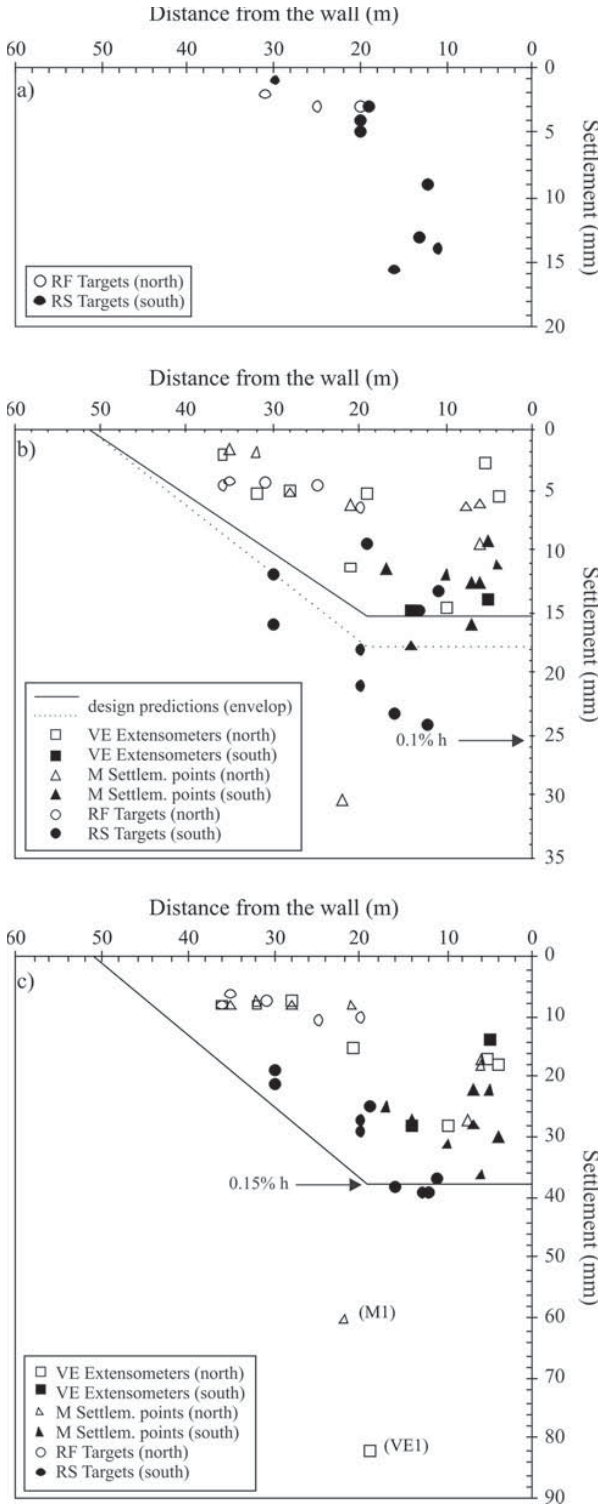


Figure 13 - Observed surface settlements: a) preparatory works and wall construction; b) excavation down to a depth of 22.5 m, with comparison to the settlements predicted in the design for the completion of the excavation (depth 25.5 m); c) total values at completion of the construction.

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.

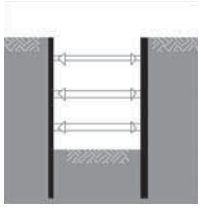
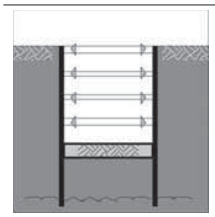
In 50 years – which is not a long period, barely corresponding to an entire professional life time – the evolution of deep urban excavations has been remarkable. We now undertake excavations that a few decades ago would have been unimaginable or would have involved unacceptable cost, construction time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

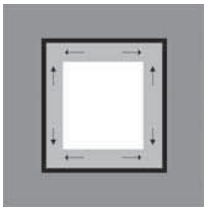
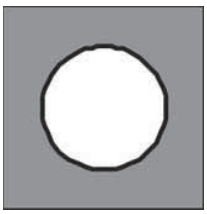
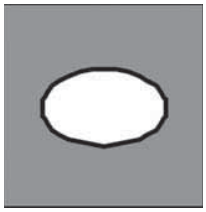
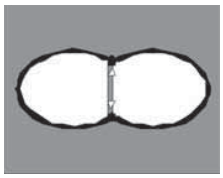
Table 2 - Retaining structures of excavations corresponding to 2D structural systems.

	Type of structure	Main type of structural loading		
		Wall		Supports
		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 excava-tion-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 con-nected 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.

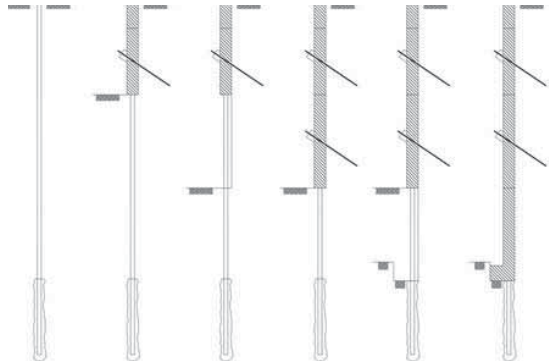


Figure 14 - Construction sequence of a permanent Berlin-type wall.

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.

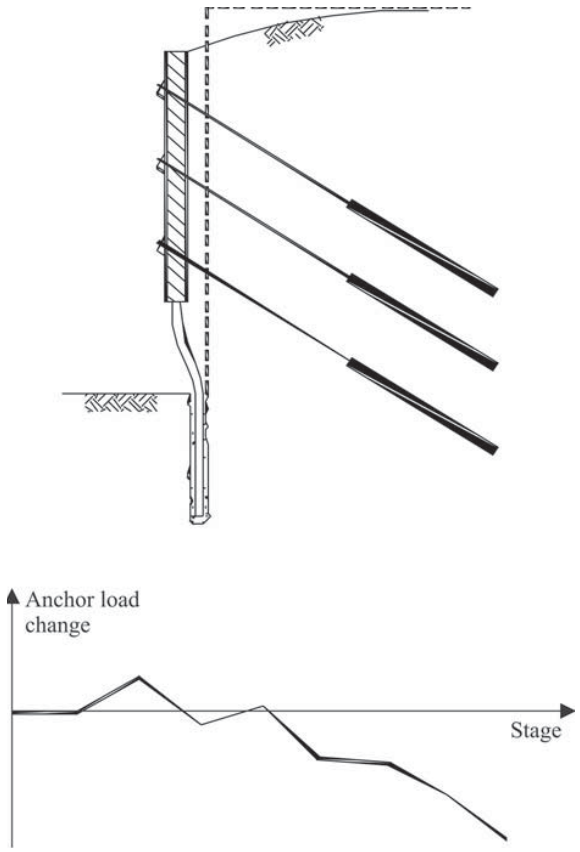


Figure 15 - Pattern of behaviour of Berlin-type walls with marginal stability conditions due to vertical loading.

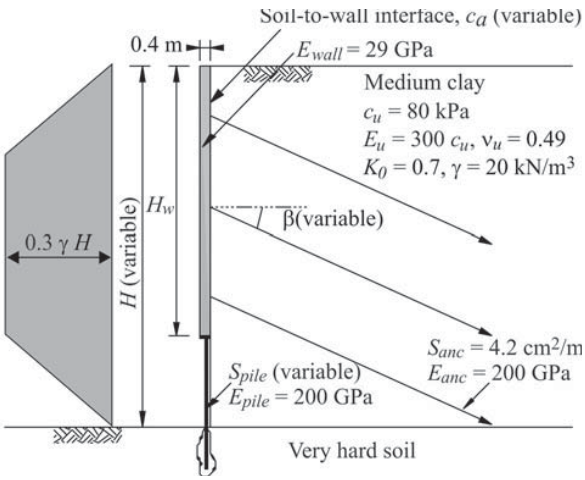


Figure 16 - Assumptions for the numerical case study of Guerra *et al.* (2004).

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).

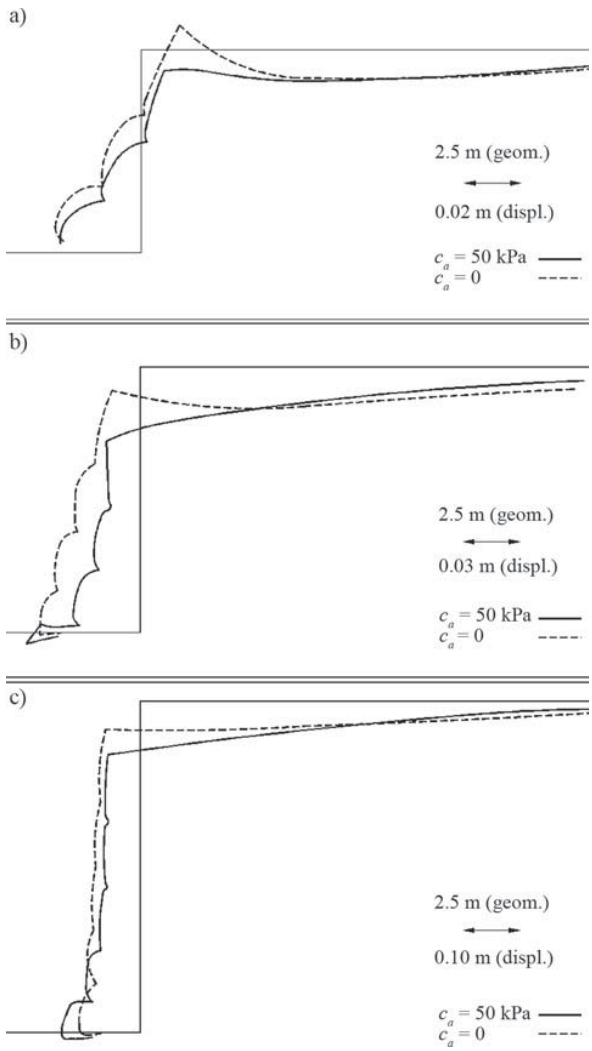


Figure 17 - Displacements of the excavation face and the ground surface for the case depicted in Fig. 16 (Guerra *et al.*, 2004): a) stage 10 (9 m depth); b) stage 13 (12 m depth); c) stage 16 (15 m depth) (note change in displacement scale).

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;

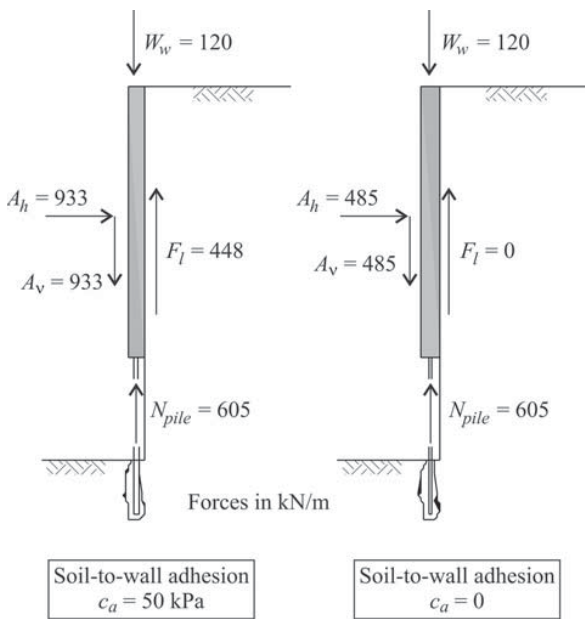


Figure 18 - Forces involved in the wall equilibrium in stage 16 (15 m depth) for the analyses of Fig. 17 (Guerra *et al.*, 2004).

(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-

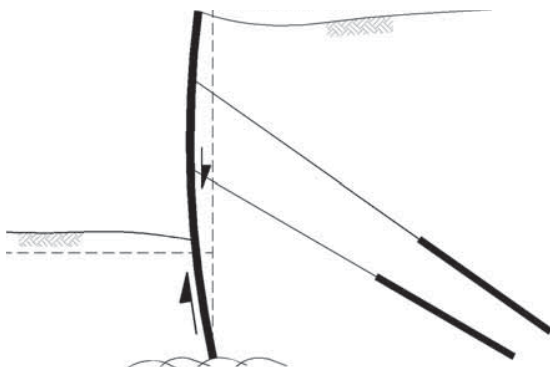


Figure 19 - Tangential forces applied by the soil to a continuous flexible retaining wall with sound foundation conditions.

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.

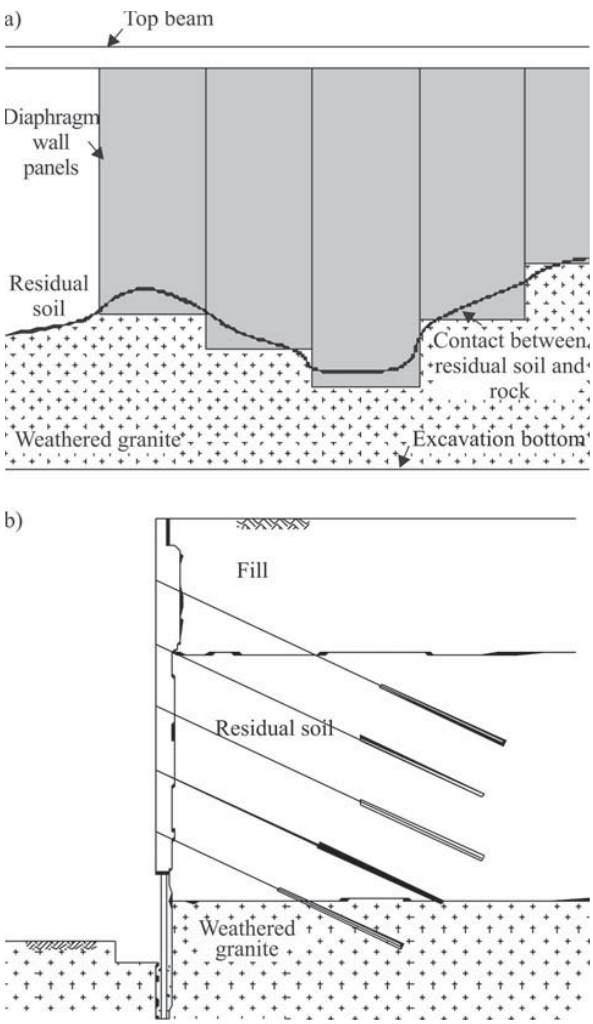


Figure 20 - Examples of difficulties found in excavations in the granite formations of Porto (depicted from real cases): a) diaphragm wall panels of variable height but with the tip above the excavation bottom; ii) permanent Berlin-type wall supporting a thick layer of residual soil and fill but with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

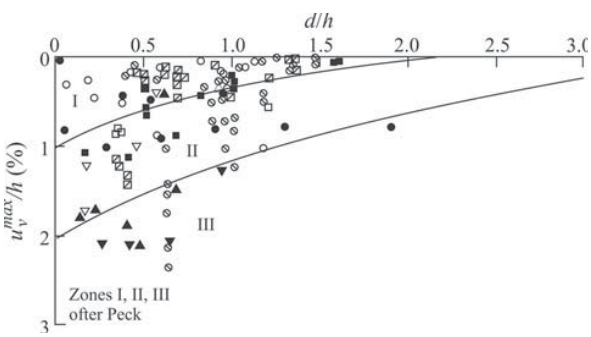


Figure 23 - Summary of measured settlements adjacent to excavations in soft to medium clay collected by Clough & O'Rourke (1990) over Peck's chart.

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-

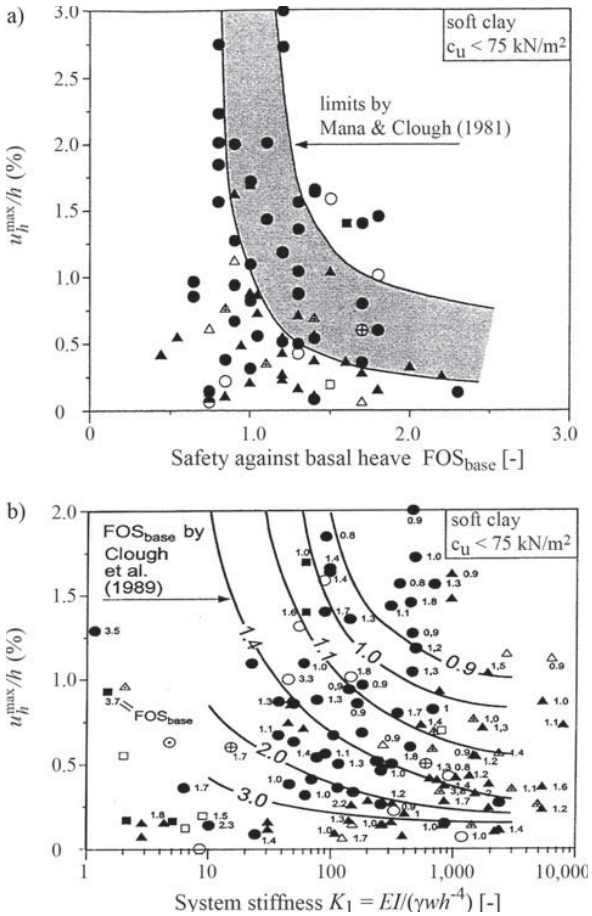


Figure 24 - Maximum lateral wall displacement in excavations in soft to medium clay collected by Moormann (2004) *vs.*: a) safety factor against basal heave; b) system stiffness.

pressed on the basis of the wall bending stiffness, *EI*, and the average vertical spacing of the wall supports, $\bar{h}$ (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.

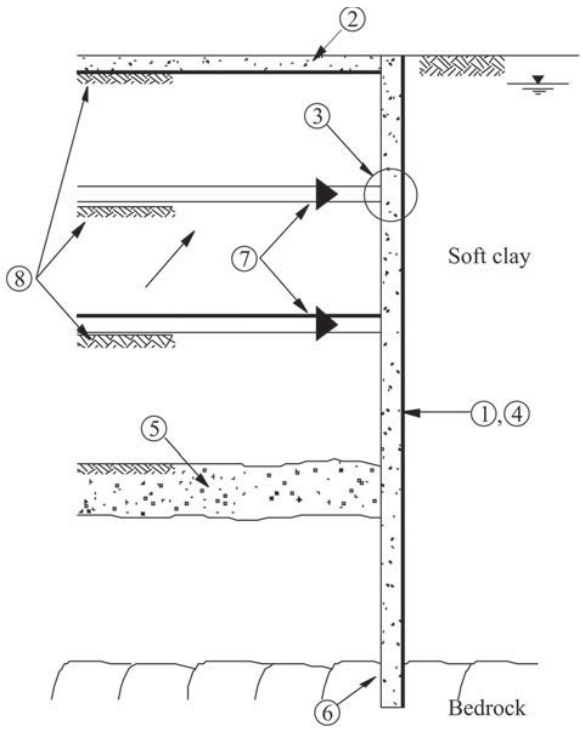
2.3.3. Eight Golden Rules for a reliable control of the excavation induced movements

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.



1 - Reinforced (stiff) concrete wall
2 - Early installation of the first level of supports
3 - Links between struts and the wall carefully detailed
4 - Impermeable wall
5 - Advanced support of the wall by ground treatment
6 - Bedrock holding wall tip
7 - Loading struts in advanced by pre-stressing

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

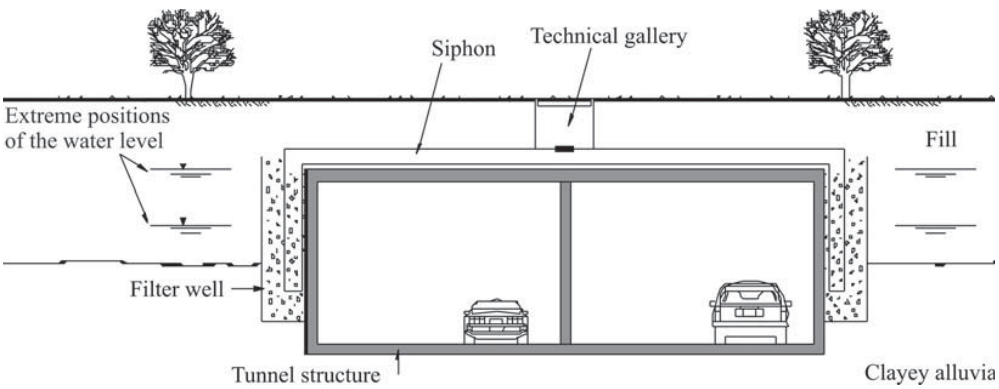


Figure 26 - Solution capable of minimizing the impact on the water level conditions induced by the construction of a permanent long buried structure (Matos Fernandes, 1997).

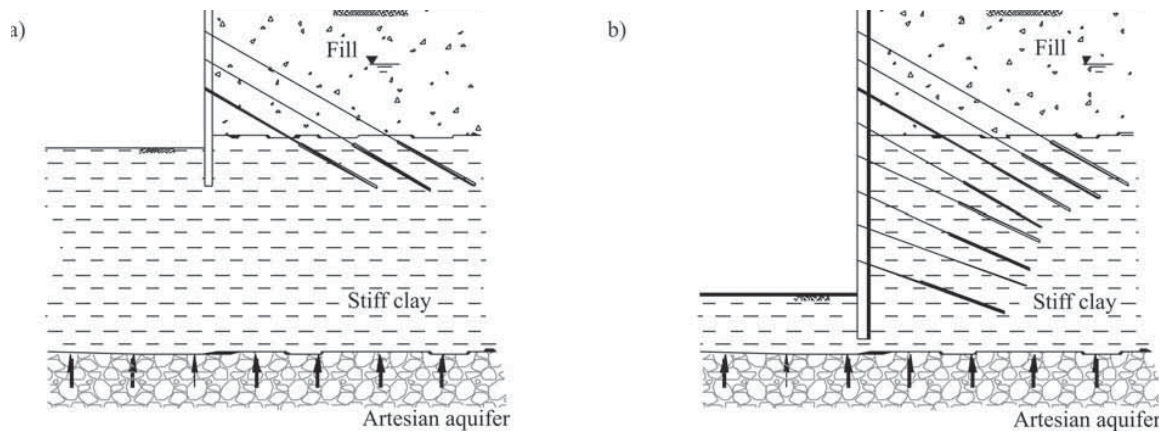


Figure 27 - Influence of a deep artesian aquifer embedded in the substratum: a) negligible influence for an excavation of moderate depth; b) possible influence for a very deep excavation.

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:

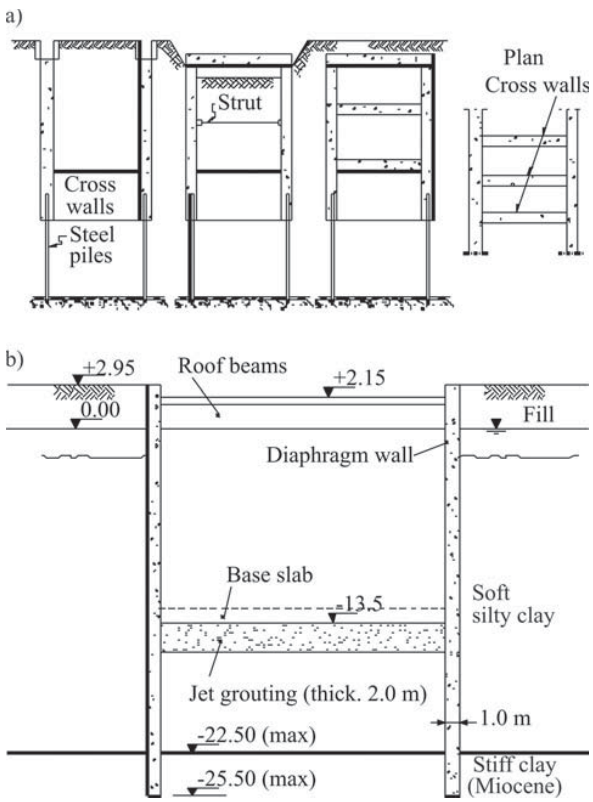


Figure 28 - Solutions for reducing the movements of retaining walls supporting deep excavations in soft clay: a) cross diaphragm wall panels below the base of the excavation (Oslo Metro); b) jet grout slab below the base of the excavation (Cais do Sodré Station of Lisbon Metro).

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-

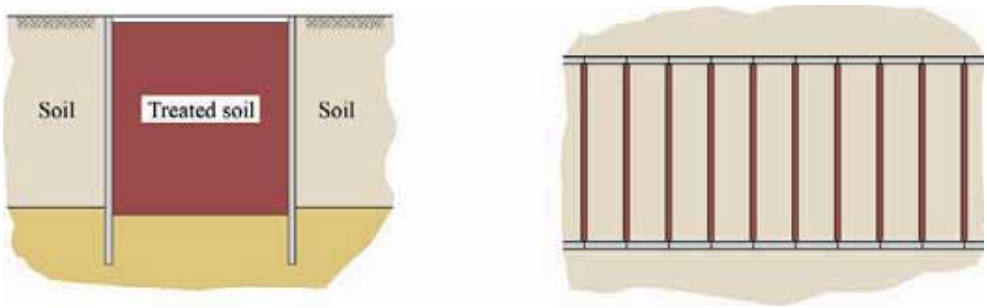


Figure 29 - Transverse supports of treated ground from the top to the bottom of the wall performed before the excavation: a) cross section of the excavation; b) plan.

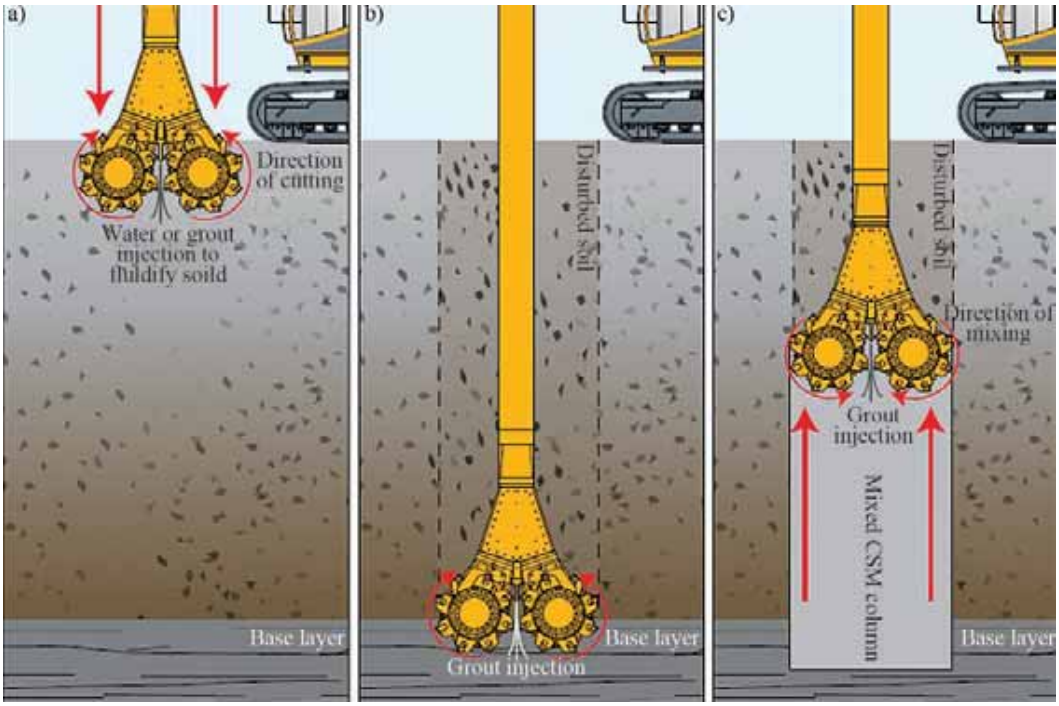


Figure 30 - The CSM technique: a) start of drilling with the ground being softened and broken by the cutter wheels; b) injection of cement slurry begins when the maximum depth of treatment is reached; c) progressive extraction of the equipment maintaining the rotation of the wheels and the injection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

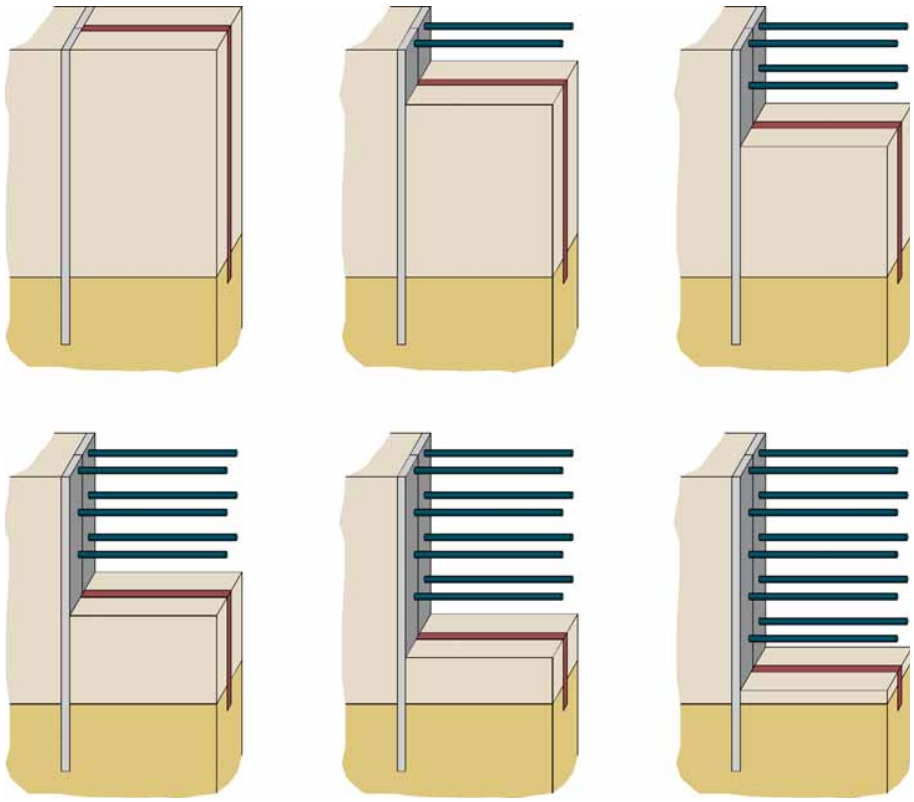


Figure 31 - Simplified sequence of the construction applying the support envisaged in Fig. 29.

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: γ = 18 kN/ m^3; c_u (kPa) = 20 + 0.22 σ'_{v0} ; E_u = 400 c_u;
Miocene substratum: γ = 21.5 kN/ m^3; c_u = 400 kPa; E_u = 400 c_u.

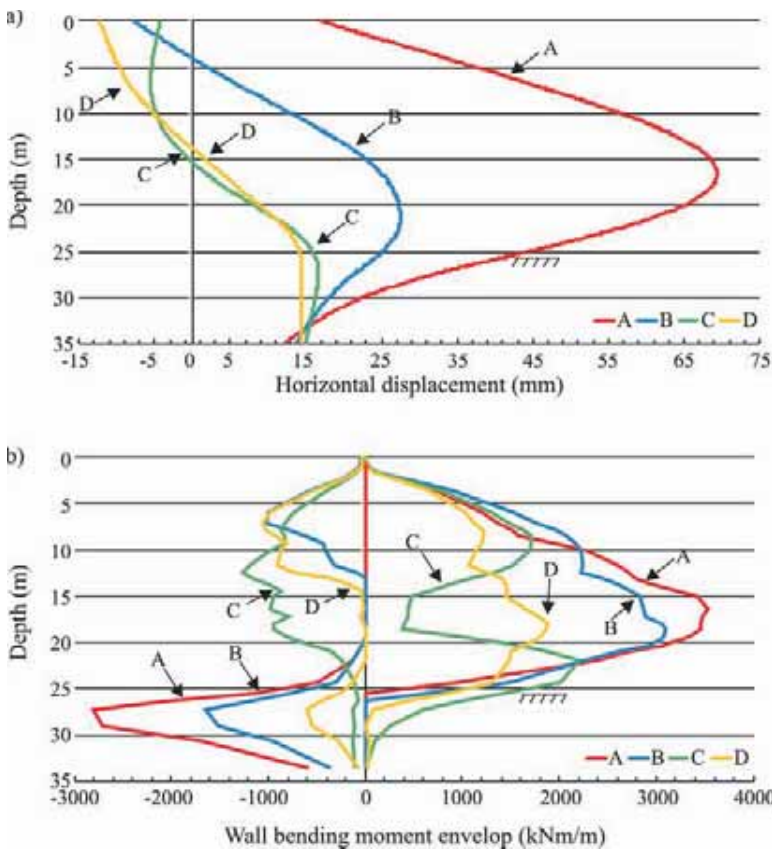


Figure 32 - Comparison of the results from the finite element analyses of Table 7 (in the vertical plane perpendicular to the wall where the results are maximum): a) final lateral wall displacements; b) wall bending moment envelopes.

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.

Acknowledgments

The author expresses his deep gratitude to the Portuguese Geotechnical Society for the invitation to present the XXVI Manuel Rocha Lecture.

He is indebted to the Colleagues that provided documents, reports and other data for the preparation of the lecture. Special appreciation is due to: Alexandre Pinto (JETSJ), Ana Pereira (JETSJ), António Carreto (Teixeira Duarte), Artur Peixoto (Georumo), Baldomiro Xavier (Teixeira Duarte), Gabriel de Almeida (Univ. of Lisbon), João Monteiro (Soares da Costa), José Alves Paula, Miriam Ferreira (CML), Paulo Natário (Rodio), Pedro Malato (NSE), Raúl Abreu (Metropolitano de Lisboa), Rui Pina (Metropolitano de Lisboa), Silvério Coelho (Teixeira Duarte), Vitor Jesus (Metropolitano de Lisboa) and Vítor Pimentel (Geotest).

The author is also indebted to the Colleagues that collaborated in the research presented in the paper, António Cardoso (Univ. Porto), António Pedro (Univ. Coimbra), António Topa Gomes (Univ. Porto), António Viana da Fonseca (Univ. Porto), Armando Antão (New Univ. Lisbon), Emanuel Maranha das Neves (IST-Technical Univ. Lisbon), Francisco Salgado (LNEC), Jorge Almeida e Sousa (Univ. Coimbra), José Couto Marques (Univ. Porto), José Mateus de Brito (Cenorgeo), José Trigo (Inst. Polit. Porto), Laura Caldeira (LNEC) and Nuno Guerra (New Univ. Lisbon).

Great esteem and recognition are due to Professor António Correia Mineiro (New Univ. Lisbon), for his kind

introduction of the Lecturer, and to my Colleague Rui Correia (LNEC), for his generous final vote of thanks.

The careful review of the paper by the Colleagues Peter Bourne-Webb, José Marques, Jorge Almeida e Sousa and Nuno Guerra is gratefully acknowledged.

The author is grateful to Câmara Municipal de Lisboa, Metropolitano de Lisboa, Metro do Porto, Rodio, Tecnasol-FGE and Teixeira Duarte for the permission to publish the data contained in the paper.

References

Addenbrooke, T.I. (1994) Flexibility number for the displacement controlled design of multi propped retaining walls. Ground Engineering, v. 27:1, p. 41-45.

Antão, A.N.; Guerra, N.; Matos Fernandes, M. & Cardoso, A. (2008) Influence of tension cut-off on the stability of anchored concrete soldier-pile walls in clay. Canadian Geotechnical Journal, v. 45:7, p. 1036-1044.

Babendererde, S.; Hoek, E.; Marinos, P. & Cardoso, A. (2004) Characterization of granite and the underground construction in Metro do Porto, Portugal. Proc. 2nd Int. Conf. on Geotechnical and Geophysical Site Characterization, ISC'2, Porto, Viana da Fonseca & Mayne (eds), v. 1, pp. 39-48.

Brito, J.M. & Matos Fernandes, M. (2006) Estação Terreiro do Paço do Metropolitano de Lisboa: concepção, previsões de projecto e desempenho. Proc. 3rd Portuguese-Brazilian Congress, Curitiba, pp. 21-42.

Candeias, M.; Brito, J.M.; Matos Fernandes, M. & Lopes, P. (2007) Jet-grouting solution to link a tunnel to a cut-and-cover excavation. Ground Improvement, v. 11:1, p. 3-10.

Cardoso, A. (1987) A Técnica das Pregagens em Solos Aplicada em Escavações (Nailed Excavations in Soils). PhD Thesis, Univ. of Porto, Porto, 496 pp.

Cardoso, A. & Carreto, A. (1990) Observação e análise de uma escavação com taludes reforçados com pregagens. Geotecnia, 59, p. 47-59.

Cardoso, A.; Guerra, N.; Antão, A. & Matos Fernandes, M. (2006) Limit analysis of anchored-concrete soldier-piles in clay under vertical loading. Canadian Geotechnical Journal, v. 43:5, p. 516-530.

Clough, G.W.; Smith, E.M. & Sweeney, B.P. (1989) Movement control of excavation support systems by iterative design. Proc. ASCE Conf. on Foundation Engineering: Current Principles and Practices, Evanston, v. 2, pp. 869-884.

Clough, G.W. & O'Rourke, T.D. (1990) Construction induced movements of insitu walls. Proc. ASCE Conf. on Design and Performance of Earth Retaining Structures, Ithaca, pp. 439-470.

Dias, H.B. (1995) Muros de suporte Coimbra e Lisboa em substituição dos impropriamente designados genericamente de muros Berlim ou tipo Berlim. Proc. 5th Portuguese Congress on Geotechnics, Coimbra, v. 3, pp. 53-61.

Duncan, J.M. & Bentler, D.J. (1998) Evolution of deep excavation technology. Proc. Int. Conf. Soil-Structure Interaction in Urban Civil Eng., Darmstadt, v. 1, pp. 139-150.

Eide, O.; Aas, G. & Josang, T. (1972) Special application of cast-in-place walls for tunnels in soft clay in Oslo. Proc. Vth ECSMFE, Madrid, v. 1, pp. 485-498.

Ferreira, C.F. (2008) The Use of Seismic Wave Velocities in the Measurement of Stiffness of a Residual Soil. PhD thesis, Univ. of Porto, Porto, 417 pp.

Figueiras, J. (1983) Ultimate Load Analysis of Anisotropic and Reinforced Concrete Plates and Shells. PhD Thesis, Univ. of Wales, Swansea, 520 pp.

Guerra, N. (1999). Mecanismo de Colapso de Cortinas Tipo Berlim por Perda de Equilíbrio Vertical (Collapse Mechanism of Berlin-type Retaining Walls by Loss of Vertical Equilibrium). Ph D Thesis, Technical Univ. of Lisbon, Lisboa, 322 p.

Guerra, N.; Cardoso, A.; Matos Fernandes, M. & Gomes Correia, A. (2004) Vertical stability of anchored concrete soldier pile walls in clay. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, v. 130:12, p. 1259-1270.

Karlsrud, K. & Andresen, L. (2007) Design of deep excavations in soft clays. Proc. XIVth ECSMGE, Madrid, v. 1, pp. 75-99.

Long, M. (2001) Database for retaining wall and ground movements due to deep excavations. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, v. 127:3, p. 203-224.

Lousada Soares, L. (2003) Edgar Cardoso, Engenheiro Civil (Edgar Cardoso, Civil Engineer), Ed. FEUP, Porto, 376 pp.

Mana, A.I. & Clough, G.W. (1981) Prediction of movements for braced cuts in clay. Journal of Geotechnical Engineering, ASCE, v. 107:6, p. 759-777.

Matos Fernandes, M. (1983) Estruturas Flexíveis para Suporte de Terras. Novos Métodos de Dimensionamento (Flexible Earth-Retaining Structures. New Design Methods). PhD Thesis, Univ. of Porto, Porto, 486 pp.

Matos Fernandes, M. (1985) Performance and analysis of a deep excavation in Lisbon. Proc. XIth ICSMFE, San Francisco, v. 4, p. 2073-2078.

Matos Fernandes, M. (1989) An ingenious structural solution to support a deep excavation. Proc. XIIth ICSMFE, Rio de Janeiro, v. 5, pp. 3051.

Matos Fernandes, M.; Cardoso, A.; Trigo, J. & Marques, J. (1993) Bearing capacity failure of tied-back walls – A complex case of soil-wall interaction. Computers & Geotechnics, v. 15:2, p. 87-103.

Matos Fernandes, M.; Cardoso, A.; Trigo, J. & Marques, J. (1994) Finite Element Modelling of Supported Excava-

tions. John Bull (ed), Soil Structure Interaction: Numerical Analysis and Modelling. E & FN Spon, Cambridge, pp 315-358.

Matos Fernandes, M. (1997) Study for Metropolitano de Lisboa on the long-term impact on the groundwater conditions of the construction of a roadway tunnel in the right bank of river Tagus close to Terreiro do Paço, Lisbon (unpublished).

Matos Fernandes, M. & Almeida e Sousa, J. (2003) Deep excavations in soft clay. Classical and new solutions for controlling the induced movements. In: The Challenges of Engineering in Portugal, Odebrecht/BPC, Rio de Janeiro, pp. 203-214.

Matos Fernandes, M. (2004) Escavações suportadas. A interacção solo-estrutura vista através das tensões tangenciais na interface. Soils & Rocks, v. 27:1, p. 3-14.

Matos Fernandes, M. (2006) Mecânica dos Solos. Conceitos e Princípios Fundamentais (Soil Mechanics. Fundamental Concepts and Principles). Ed. FEUP, Porto, 451 pp.

Matos Fernandes, M.; Maranha das Neves, E.; Salgado, F.; Caldeira, L.; Pina, R.; Flor, A.; Brito, J.M. & Tavares, A. (2007) Jet grouting solutions for cut-and-cover subway metro stations in Lisbon downtown. Proc. XIVth ECSMGE, Madrid, v. 2, pp. 895-900.

Matos Fernandes, M. (2007) An overview on induced movements by deep excavations in soft ground and eight golden rules for their control. Proc. XIVth ECSMGE, Madrid, v. 5, pp. 379-384.

Matos Fernandes, M. & Xavier, B. (2010) New earth retention system with prestressed wales in an urban excavation. Journal of Geotechnical and Geoenvironmental Engineering, ASCE (accepted for publication).

Moormann, C. (2004) Analysis of wall and ground movements due to deep excavations in soft soil based on a new worldwide database. Soils and Foundations, v. 44:1, p. 87-98.

Paula, A. (1979) Paredes moldadas no terreno. Técnicas de abertura de valas. Diaphragm Walls. Seminar 229, LNEC, Lisboa.

Peck, R.B. (1969) Deep excavations and tunnelling in soft ground. Proc. VIIth ICSMFE, Mexico City, General Report. State-of-the-Art Volume, pp. 225-290.

Peixoto, A. (2010) Personal communication.

Pinelo, A. (1980) Dimensionamento de Ancoragens e Cortinas Ancoradas (Design of Anchors and Anchored Retaining Walls). Thesis, LNEC, Lisboa, 170 pp.

Pinto, A., Ferreira, S., Barros, V., Costa, R., Lopes, P. & Dias, J. (2001) Sotto Mayor Palace – Design and performance of a deep excavation. Proc. XVth ICSMGE, Istambul, v. 2, pp. 1237-1240.

Poh, T.Y.; Goh, A.T. & Wong, I.H. (2001) Ground movements associated with wall construction: case histories. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, v. 127:12, p. 1061-1069.

Rolo, M.F. (1999) Um Metro e Uma Cidade. História do Metropolitano de Lisboa, 1885-1975 (A Metro and a City. History of the Metro of Lisbon, 1885-1975). Metropolitano de Lisboa, Lisboa, v. 1, 486 pp.

Samuel, A. (1979) Fundações especiais com paredes moldadas no solo. Diaphragm Walls. Seminar 229, LNEC, Lisboa.

Topa Gomes, A.; Cardoso, A.; Almeida e Sousa, J.; Andrade, J.C. & Campanhã, C. (2008) Design and behavior of Salgueiros Station for Porto Metro. Proc. Int. Conf. on Case Histories in Geotechnical Engineering, Missouri Univ., Paper No. 8.04c.

Topa Gomes, A. (2008) Poços Elípticos pelo Método de Escavação Sequencial na Vertical. O Caso do Metro do Porto (Elliptical Shafts by the Sequential Excavation-Concreting Method. The Case of Metro do Porto). PhD Thesis, Univ. of Porto, Porto, 332 pp.

Viana da Fonseca, A.; Matos Fernandes, M. & Cardoso, A. (1997) Interpretation of a footing load test on a saprolitic soil from granite. Géotechnique, v. 47:3, p. 633-651.

Wang, Z.W.; Ng, C.W. & Liu, G.B. (2005) Characteristics of wall deflections and ground surface settlements in Shanghai. Canadian Geotechnical Journal, v. 42:5, p. 1243-1254.

Wong, I.H. & Poh, T.Y. (2000) Effects of jet grouting on adjacent ground and Structures. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, v. 126:3, p. 247-256.