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Ground behaviour and potential damage to buildings caused by the construction of a large diameter tunnel for the Lisbon Metro

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ABSTRACT: As a part of the current extension of the Lisbon Metro, over 3 km of double track running tunnels are being excavated by means of an EPB machine. The construction of the tunnel in an old urban environment needed an extensive appraisal of the potential damage which may be induced by the ground movements to the overlying structures. The varying nature of the ground -from very soft alluvium to the Miocene sands, harder but running under water head- was investigated in detail at design stage and the anticipated behaviour to tunnelling was subsequently backanalyzed using data from the instrumentation during construction. In some particular sites the initial analysis showed the need to adop a protection work to the existing buildings before boring operations commenced. This paper summarizes the geotechnical properties of the soils and describes the structural models used to predict behaviour on account of the excavation of the tunnels. Special mention has been given to the refinement of ground behaviour parameters by means of the data collected from the soil instrumentation. Finally, the paper mentions the different techniques employed for ground improvement in those places where protection of buildings was recommended.

1. INTRODUCTION

The scheme under construction includes the following works (see Fig. 1):

- Extension of existing stations: Restauradores and Rossio.

- Construction of two new stations: Baixa/Chiado and Cais do Sodré.

- Construction of a double track running tunnel for the two lines.

- Construction of reversing tunnels at the termini of Cais do Sodré and Baixa/Chiado.

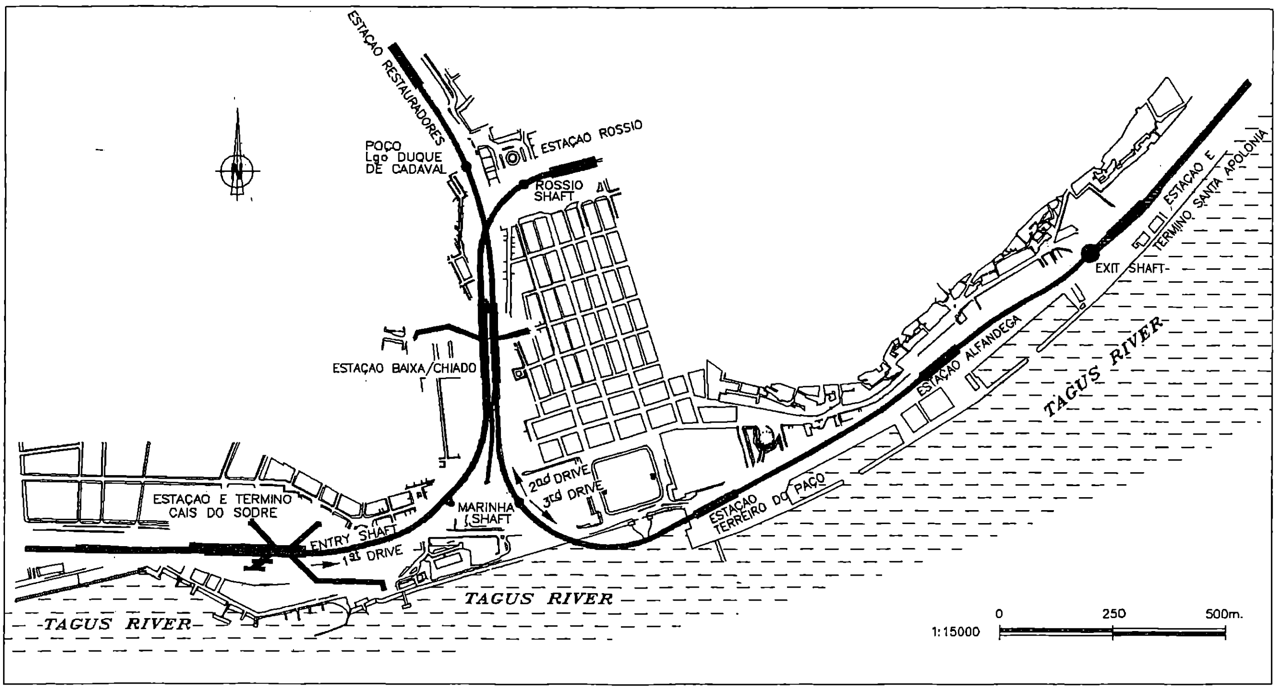

Fig. 1 : LISBON METRO. Southern Extension

The running tunnels, that are the subject of this paper, are designed at the Client's specification as double track tunnels matching the arrangement of the existing underground sections. Fig. 2 shows the typical cross section of the 8.8 m internal diameter twin track tunnel. The construction of the tunnels is being undertaken using a 9.71 m diameter EPB soft ground TBM, with a single pass precast segmental lining. The first tunnel drive commenced from Cais do Sodré station in October 1994 and will be driven through the west side of Baixa Chiado station to be removed at a shaft in Rossio Square. The machine willl then be reinstalled in the Marinha shaft for the second drive towards Baixa Chiado Station where it is planned to drag the machine through the previously excavated eastern half of the station before completing the drive to Largo Duque de Cadaval crossing over the first drive. The machine will then be relocated in the Marinha shaft to commence a third bore which will go southwards to form the running tunnel to the Santa Apolonia Shaft.

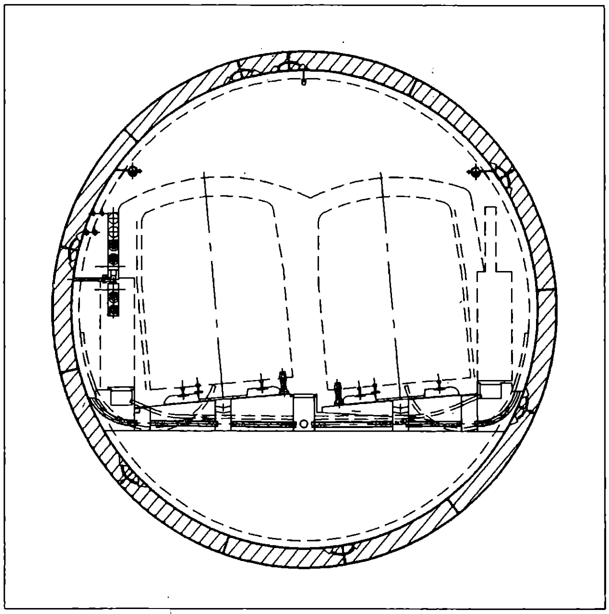

Fig. 2 : Typical cross section of tunnel

A particular feature of this third drive is that the tunnel will be driven below the Tagus riverbed with less than 1 diameter of cover through very soft alluvial silts.

This paper describes the geotechnical properties of the varied soils encountered along the route, discussing the potential effects of ground movements on buildings. It is shown how actual displacements measured during construction were back-analyzed to establish the behaviour of these waterbearing soils.

2. GEOLOGY AND GEOTECHNICAL DESIGN PARAMETERS

The area is included in the tertiary basin of the Tagus River where Miocene deposits occasionally outcrop at hills that form the landscape of the old Lisbon. At the low lying parts of the city, the Miocene is covered by recent alluvial deposits of estuarine origin, overtopped by man made ground.

The main strata are:

- Made ground: remaining of old buildings, stone, brick and wood within an earthfill matrix.
- Alluvial deposits: silts and clays with occasional sandy layers.
- Argilas dos Prazeres: silty clays, marls occasional calcareous interbeddings.
- Areolas de Estefania: clean sands, silty sands and some cemented interbeddings with frequent lateral changes.

Fig. 3 shows a geological profile along the tunnel from Cais do Sodré to Rossio.

An extensive soil investigations program has been completed including boreholes, intact sampling, in situ pressuremeter and piezocone testing. The collected samples were subjected to laboratory tests to establish the

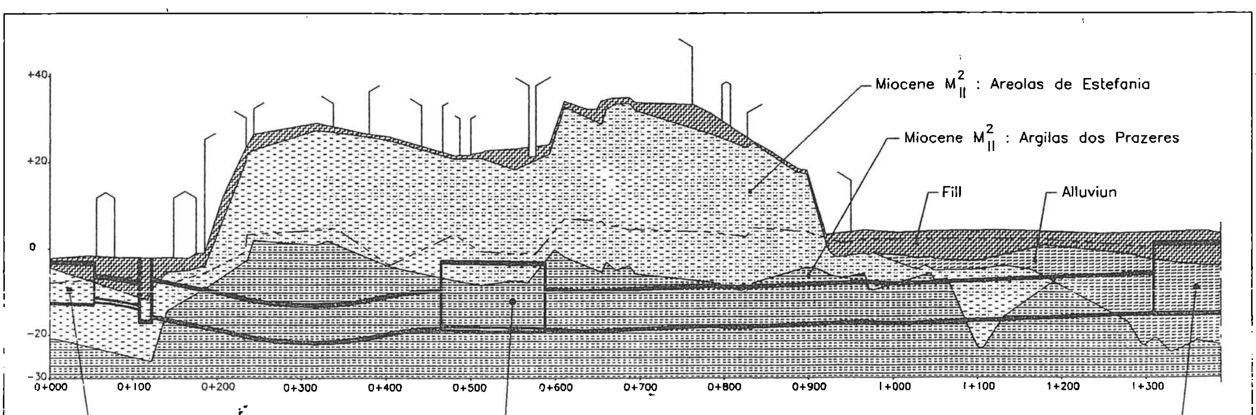

Fig. 3 : Geological profile

basic identification parameters (grain size, plasticity and chemical analyses), soils state (water contents and density), strength (by means of triaxial and unconfined compressive strength tests) and deformability. Complementary information was obtained from plate loading tests carried up to soil failure from pilot galleries bored within the Miocene formations. As a result of such an extensive survey, the basic design parameters were established. Fig. 4 shows the results of shear strength tests in alluvial soils, indicating the soft consistency of these materials.

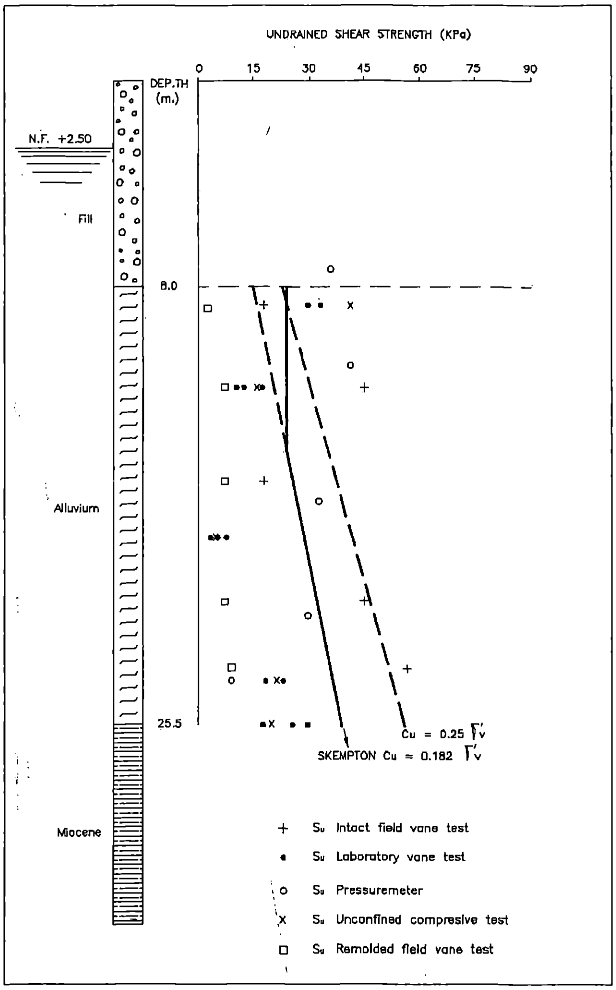

Figure 4 : Undrained shear strength vs. depth of alluvial

3. METHOD OF CONSTRUCTION

Having established the need to use a driven tunnel consideration of the stability ratio of the alluvium is most important. Typically the stability ratio at tunnel axis is as follows.

$$N = \frac{\gamma H}{C_u} = \frac{1.8 \times 14}{2.5} = 10.1$$

Practical experience of real tunnels and testing of model tunnels in a centrifuge confirm that the stability number is much greater than the value at which collapse would occur.

Figure 5 shows how the critical stability ratio at collapse varies with the ratio of clay cover divided by tunnel diameter (C/D). The minimum value of C/D is about 0.9 assuming (Mair, 1993) that the fill is sufficiently cohesive to be part of the clay cover. If the fill is not cohesive the tunnel has a C/D ratio equal to zero. The stability ratio for collapse is therefore between 2 and 5.

The actual stability ratio during construction could be improved by compressed air but to achieve N = 2 at the axis level an air pressure of 2 bar (200 kPa) would be needed.

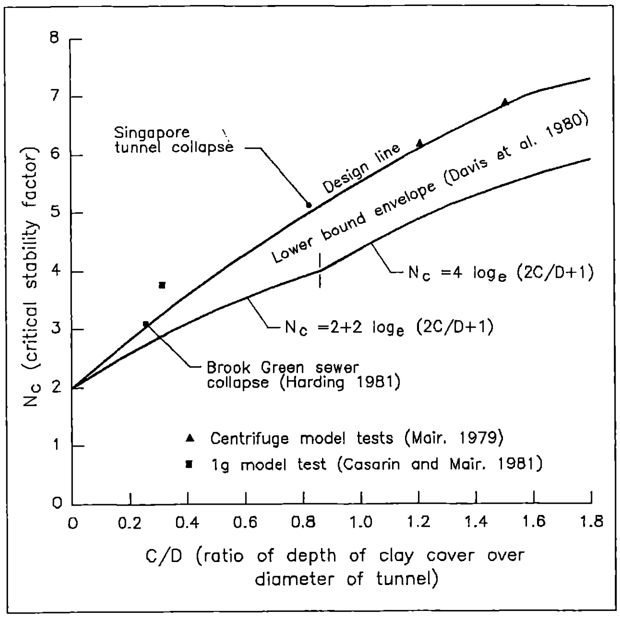

Fig. 5 : Stability of the excavation face (Mair, 1993)

However there would be major difficulties in balancing pressures over the height of the tunnel and in preventing excessive loss of air through inadequate cover.

The best method of ensuring the stability of the face is by means of an earth pressure balanced machine (EPBM) because the density of the soil in the cutting chamber will provide a similar distribution of the in situ earth pressure and the total force on the face can be controlled by the rate of advance and rate of spoil removal through the screw conveyor.

4. SETTLEMENT PREDICTIONS

The tunnel drives pass directly below or close to many old, important and sensitive buildings. The use of an EPB machine was specified to minimise settlements but the large diameter, shallow depth and soft waterbearing soils means that settlements could not be completely avoided. Predictions were therefore carried out to estimate that magnitude of settlements, predict the risk of damage to buildings and decide whether any protection measures were needed.

The method of assessing ground movements is essentially based on the empirical method first proposed by B. Schmidt in his PhD thesis submitted to the University of Illinois in 1969. The method was developed in close collaboration with R. Peck who gave the method more widespread publication in his State-of-the-Art paper at the 1969 ICSMFE Conference in Mexico.

The analysis assumes that the settlement trough will take the form of a Gaussian probability curve and adjusts the volume and width to suit actual measurements on similar tunnels:

$$S_v = S_{\max}\, e^{\frac{-y^2}{2\,i^2}}$$

where S_v is the vertical settlement
S_{max} is the maximun vertical settlement on the tunnel center line
y is the horizontal distance from the center line
i is the trough width parameter and is the horizontal distance to the point of inflexion on the settlement trough.

The ground loss is usually expressed as a fraction of the excavated area of the tunnel. The value of the ground loss in the alluvium will depend upon the details of the method of construction and quality of workmanship. The volume can be expressed as a percentage of the volume of excavation and is referred to as the percentage ground loss. For construction using an EPB shield the ground loss can be divided into 4 components:

(i) Face loss
(ii) Loss around the shield
(iii) Loss behind the shield
(iv) Consolidation effects

(i) Face loss
Recent case histories show that careful control of the face pressures can eliminate any loss into the face and ideally achieve a small heave. It is therefore proposed to assume zero face loss

(ii) Loss around the shield
The stability number of the alluvium indicates that the soil will inmediately close into any space around the shield. It is expected that the minimun over cut needed will be 19mm for straight tunnelling and 45mm for a 200m curve. These correspond to ground losses of 0.8 % and 1.8% respectively.

(iii) Loss behind the shield
As the shield moves forward the annular space around the tunnel lining will be injected with grout. The amount of ground loss will depend on the effectiveness of injecting, the grouting pressure and the time for the grout to gain sufficient strength to prevent ground movement. Assuming that the grouting is carried out carefully through the tail seals of the shield it is reasonable to expect that the grouting will be 95% effective in filling the annular space and therefore limit ground loss behind the shield to 0.2%.

(iv) Consolidation effects
Consolidation settlements in the alluvial clays can occur due to dissipating excess pore presures induced by the EPBM or by groundwater lowering. Provided care is taken to avoid excessive face pressures the consolidation caused by the EPBM tunnel should not be significant. Also, these settlements occur slowly, are more widespread and are therefore less likely to cause damage to buildings. The calculations have excluded consolidation effects.

From the above considerations the ground losses in the alluvium for design purpose were taken as 1% for straight line and 2% for curved aligment.

O'Reilly and New (1982) also correlated many data of observed settlement troughs to show that the trough width parameter i was a reasonably linear function of the depth z and independent of tunnel construction method. It can be assumed that the simple approximate form "$i = K z$" can be adopted. Values of K for tunnels in clay (cohesive soil) and sands or gravels (granular soils) are taken as approximately 0.5 and 0.25 respectively. The choice of an appropriate value of K will often require some judgement, since it depends on whether the ground between the tunnel and the foundation being considered is primarily cohesive or granular.

5. REVIEW OF THE ACTUAL MEASURED SETTLEMENTS

From the review of the instrumentation data, particular attention was given to analyzing the actual measured settlements at some profiles which gave a good range of data. The average values of settlement were then plotted on to a spreadsheet for all points on the profile and then a best fit Gaussian curve was simply plotted to these by varying the values of ground loss and trough width coefficient . This then gave an indicative level of the actual values of % ground loss and trough width coefficient due to tunnelling operations.

Specific points which gave maximun settlements from selected cross sections were then analyzed in the form of a graph of settlement against position of the TBM head. Fig. 6 shows the readings of a settlement cell above at the tunnel axis. The tunnel was excavated in alluvial soft soils in the upper part of the section. This profile demostrates normal soil behaviour with the expected incidence of heave occuring just before the TBM arrives due to the pressure of the face and then a "rapid" settlement occuring after the tailskin has passed which evens off as the grout in the space between hardens as the ground moves back around the tunnel lining. When the machine was situated at 60 m beyond the measuring point, an accidental tail loss occured during excavation which reflected in the graph as a steeper gradient of the settlement curve.

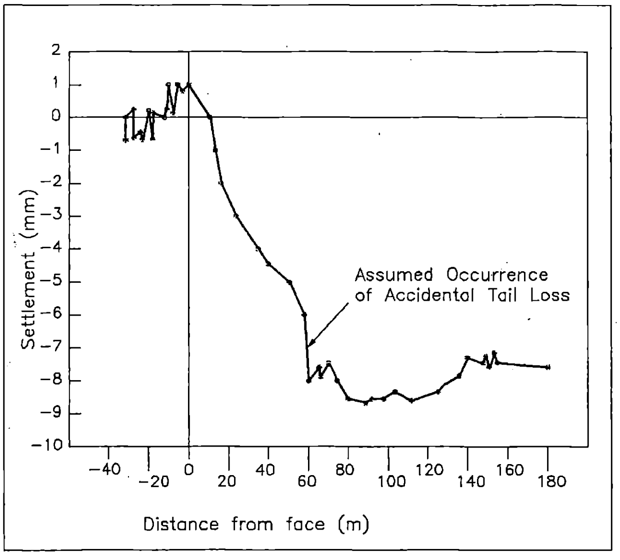

Fig. 6 : Surface Longitudinal Settlement Profile

After review and assessment of the data the following conclusions can be reached:

1. **Alluvium**

   There have not been any profiles analyzed for the situation when the tunnel is wholly within the alluvium. This is because settlement in alluvium is very difficult to interpret, partly due to the influence of jet grouting and other protective works made for buildings.

2. **Miocene Sands**

   From examination of the profiles, it can be concluded that significant settlements have occured in the Miocéne Sand material: Ground losses varying between 0.8% and 1.2% and K factors between 0.3 and 0.5.

The typical value of ground loss has been assessed at an upper bound value to allow for the worst case of settlement occuring: thus giving a better risk analysis of the factors involved. The above values are typical of sands, and the limestone bands indicated in the original geotechnical report do not appear to have any significant influence. As expected with granular soils there is no evidence of consolidation settlement. When inspecting the extensometer results, it was noted that there is considerable volume change with depth due to the loosening of the dense sands.

6.ASSESSMENT OF RISK TO BUILDINGS

Assessments of the influence of ground movements due to tunnelling have been carried out for all buildings along the tunnel route. These assessments have been performed using simplified methods and assumptions for brick and masonry structures based upon the same procedure as outlined by Mair et al (1996) in their paper to this Conference. The building is treated as an idealised beam with span L and height H deforming under a central point load to give a maximum deflection Δ. This will generate tensile stresses in the building which can lead to cracking and general damage, as was studied at length by Burland and Wroth (1974).

Figure 7 shows a general case of a building affected by a tunnel settlement trough. It is assumed that the building follows the ground settlement trough at the foundation level. The point of inflexion of the settlement trough (defined by i for the case of a single tunnel) divides the building into two zones. In the hogging zone ($y > i$), where the neutral axis is at the bottom, all strains due to bending will be tensile. In the sagging zone, where the neutral axis is at the centre of the building, bending will cause both compressive and tensile strains. Within each zone the maximum ratio Δ/L can be determined, i.e. Δ_h / L_h in the hogging zone and Δ_s / L_s in the sagging zone, as shown on Figure 7.

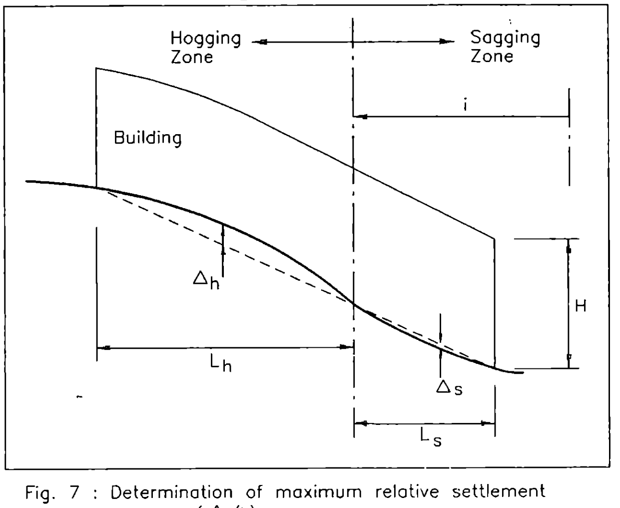

Fig. 7 : Determination of maximum relative settlement ratio (Δ/L)

The maximun bending strain and diagonal strain are derived from the ratio Δ/L and can be combined with the horizontal ground strains to obtain the maximun tensile strain, which is compared to the various categories of risk established by Burland et al (1977) and the Building Research Establishment.

In case that the results obtained indicated untenable degrees of damage it was recommended to provide some form of protection works to the affected buildings before tunnelling works commence. Different types of protective measures have been adopted in this job: underpinning, temporary bracing and compensation grouting. Fig. 8 shows a particular application of the last technique to a 4 storey building.

7. ACKNOWLEDGEMENTS

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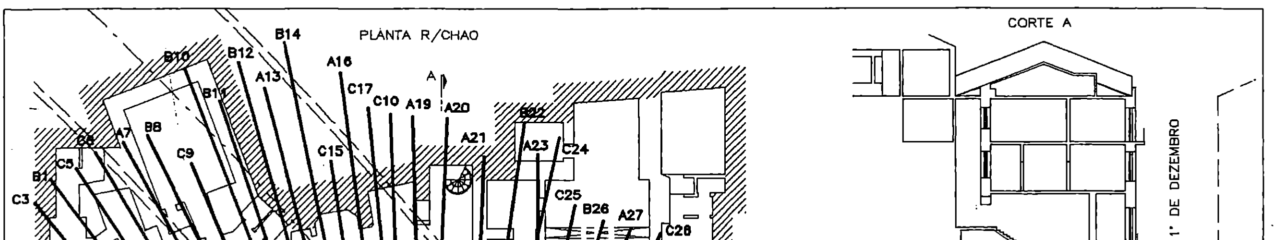

Figure 8 : Compensation grouting treatment