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Three-dimensional numerical modelling of the construction of an EPBS tunnel for Shanghai Metro - Line 2

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ABSTRACT: A three-dimensional numerical analysis, based on the finite element method, of the construction of Shanghai Metro Tunnel-Line 2, in China, executed with an earth pressure balanced shield (EPBS) is presented. For the construction of this tunnel, a detailed field instrumentation programme was developed and, therefore, a considerable volume of observation results is available. Results obtained from the numerical model are compared with those observed, mainly for the horizontal and vertical displacements of the soil mass, both at surface and depth, and the earth pressures development around the tunnel concrete lining. Numerical predictions are globally very good, showing the ability of the numerical model to reproduce the behaviour of a tunnel constructed with a shield.

1 INTRODUCTION

For Line 2 of the Shanghai Metro, in Shanghai, China, a tunnel has been executed using an EPB shield (Earth Pressure Balanced). This tunnel was constructed underneath a busy commercial area of the city, with a large number of tall buildings and underground utility services and, therefore, ground and structure movements had to be controlled during construction. With this aim, a detailed field instrumentation programme was developed to obtain a considerable volume of observation results, which are available in Lee *et al.* (1999).

2 SUBSURFACE CONDITIONS

From a geotechnical point of view, the city of Shanghai rests on top of very thick alluvial and marine sediments deposited during the Quaternary, reaching around 400m in depth. The more superficial materials, where the tunnel has been built, are mainly composed of silty clays and sands. For Section P2 of the tunnel (section used in this study) the stratigraphic sequence is presented in Figure 1. The water level is at a depth of 1.5m.

The geotechnical survey of the area included in-situ tests (mainly CPT, vane tests and pressuremeter tests) and laboratory tests (mainly, identification tests and triaxial compression tests). From the results of those tests (available in Lee *et al.*, 1999) one can obtain for the main geotechnical parameters the values presented in Table 1. Since the aim of the numerical studies is to reproduce the construction sequence of the tunnel close to the front face, and taking into account the low permeability of the materials (except for the superficial fills), the analyses are performed under undrained conditions, considering parameters in terms of total stresses.

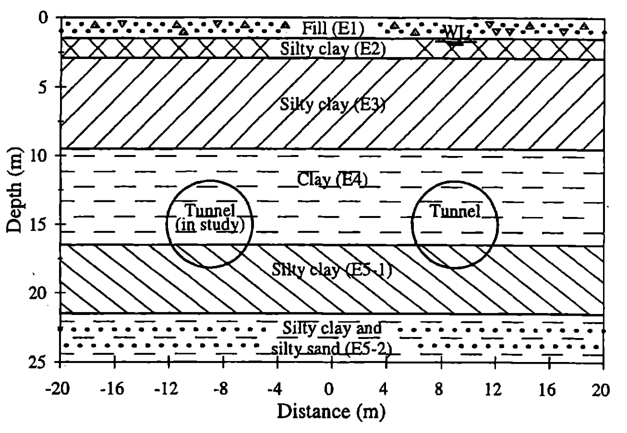

Figure 1. Shanghai Metro - Line 2, Section P2. Geological profile.

With the available set of parameters, linear elastic perfectly plastic models have been used to describe the behaviour of all materials. The Tresca failure criterion has been used for all sequences, except for the superficial fills where the Mohr Coulomb criterion has been considered.

3 PROJECT DESCRIPTION

Only one of the two tunnels that compose this line of the Metro system will be considered in the analyses. This is a tunnel with a circular section, with a diameter of excavation of 6.34m, with the tunnel

Table 1. Geotechnical characteristics considered in the numerical analyses.

Com plex	γ (kN/m³)	E',E_u^{**} (MPa)	ν',ν_u	c' (kPa)	ϕ' (°)	c_u (kPa)	K_0
E1	18.0	10.0	0.30	0	30	-	0.5
E2	18.7	15.6	0.49	-	-	39	0.7*
E3	18.0	14.4	0.49	-	-	36	0.7*
E4	17.0	17.6	0.49	-	-	44	0.7*
E5-1	18.3	20.8	0.49	-	-	52	0.8*
E5-2	18.3	35.6	0.49	-	-	89	0.8*

Note:* mean value of the relation between the horizontal and vertical total stresses; ** due to the lack of information $E_u=400c_u$ has been assumed (Hunt, 1986).

axis being placed at a depth of around 15m (in order to simplify the numerical model, contact between sequences E4 and E5-1 has also been considered at a depth of 15m). This tunnel has been excavated with tunnel boring machines, with earth pressure balanced shields, 6.24m long. At section P2 an earth pressure cell placed at the front of the shield showed, according to the available information, a pressure of 210kPa. The distribution of pressure in front of the shield is unknown.

The tunnel has been constructed with a precast concrete lining, composed of six segmental pieces bolted in the longitudinal and circumferential directions, with an outer diameter of 6.2m, 35cm in thickness and each ring being 1m long. With the advancement of the shield, a tail void, with a 7cm gap, is created. This gap is then filled with pressurized grout material. According to the available information, pressures of grout have been in the order of 250kPa.

4 NUMERICAL SIMULATION OF THE TUNNELLING PROCESS

The numerical model considered is three-dimensional and ABAQUS code (based on the finite element method) has been used for calculations. The element mesh has been built considering a series of vertical slices of quadratic elements of 20 nodes ("brick" elements). In front of the final position of the front face the mesh length is 24m, corresponding to 4 slices of elements. Behind the final position of the front face each slice is 3m long (around half the length of the shield) and a total of 33m (11 slices of elements) have been considered.

The modelling of the construction sequence is based on a series of steps where elements are progressively removed or added, simulating the excavation, shield advancing, lining placement and tail void filling with pressurized grout, and where loads are applied and removed in order to guarantee stability until the lining placement.

Regarding the shield modelling, two aspects must be pointed out. In first place, the elements that model the front part of the shield have the same nodes as the elements of soil that have been removed in the excavation face, except at the cutting face, where new nodes are introduced to keep the elements representing the shield apart from the front face of excavation in order to allow the soil to deform in an unrestricted way. With the excavation, pressures are applied on that front face in order to stabilize it. For EPB shields, the pressure distribution should correspond to a linear increment between the crown and the floor of the section, simulating the equilibrium of the earth pressure at each level. In this case, due to the lack of information concerning this distribution, a constant pressure for all levels of the section has been considered.

The second aspect concerns the shield body. In the tunnel design it was prescribed that the shield should produce a general heave of the surrounding soil in order to compensate for subsequent inward movements of the soil around the tunnel opening. This was achieved, in the field, by adjusting the speed of the screw auger that removes the soil from the soil chamber at the front to a slightly slower rate than that of the soil trying to enter the cutting face. If one numerically modelled the shield body with very stiff elements, as would normally occur, these elements would restrict the movement of the surrounding soil. Therefore, the outer elements of the shield have been modelled with soft elements and, simultaneously, a pressure has been applied to the excavation surface. The value of this pressure has been chosen between the values of the front face pressure and the grout pressure at the tail void. This pressure has been named the "transition pressure". Its value should correspond to the pressure applied to the injection grout, which, due to the loss of efficiency, is not really installed in the tail void.

As mentioned previously, the available information concerning the shield operation (Lee *et al.*, 1999) indicates that a pressure of 210kPa has been applied at the front face and a pressure of 250kPa has been applied to the tail void. The real meaning of these values is unknown. In the first case, the pressure distribution is not defined, although a difference of 75kPa between the earth pressure at the top and base of the front face can be estimated from the in-situ earth pressures. Regarding the value of grout pressure at the tail void (i.e. 250kPa), it is not known if this represents the value really applied or the value registered at the exit of the injection equipment, which, in this case, would be affected by the dynamic effect of the injection operation.

5 NUMERICAL RESULTS *VERSUS* FIELD RESULTS

5.1 Horizontal and vertical displacements along the longitudinal plane

At the beginning of the numerical study, a series of calculations was made where different values of the front face pressure and the tail void pressure were considered. The aim of these initial studies was to evaluate the values of those pressures that would allow the observed field results to be obtained. Only the horizontal displacements, with depth, along the vertical plane that contains the tunnel axis and the surface settlements along the same plane were used for that purpose. This initial procedure is justified, first, by the lack of information already referred to concerning the real meaning of the available values for those pressures and, secondly, by the lack of information associated with the mechanical characterization of the materials from the different sequences crossed by the tunnel, which did not allow a complete definition of geotechnical models for those materials (the values for the undrained deformability modulus, for example, had to be estimated based on correlations with the values of the undrained strength).

The numerical results of the horizontal displacements profiles in front of the front face and the surface settlements profile, both along the longitudinal plane that contains the tunnel axis, are presented in Figures 2 and 3 for a front face pressure of 260kPa, a transition pressure of 240kPa and a tail void grouting pressure of 180kPa. In these figures, the available field results are also presented (results in Figure 3 named "observed/estimated" correspond to observed results that are only available in a time scale; the use of a distance to the front scale had to be estimated based on the "observed" results).

From the analysis of both figures it is possible to

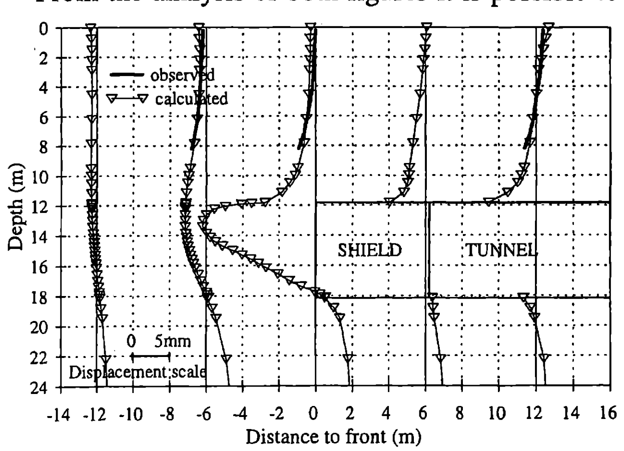

Figure 2. Horizontal displacements along the vertical plane that contains the tunnel axis.

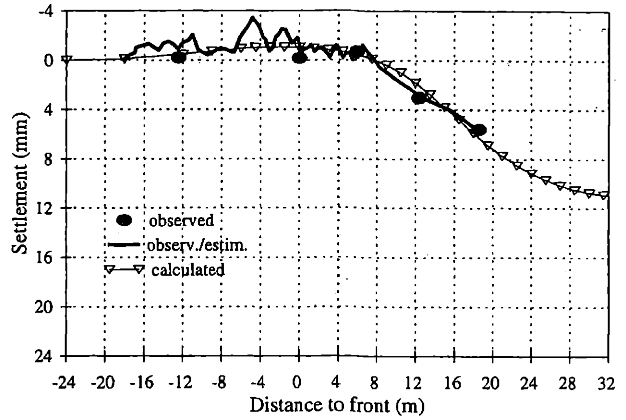

Figure 3. Surface settlements along the longitudinal plane.

conclude that, for the values considered for the different pressures, the numerical results seem to reproduce very closely the observed ones. In general, the observed behaviour of the ground shows the heaving process ahead of the front face, as prescribed in the tunnel design, which was detected not only from the horizontal displacements at depth, but also in the vertical movements at the soil surface. Regarding the numerical results for the horizontal displacements at the front face, the higher value occurs at the top third of the section, due to the fact that a constant pressure has been considered applied at that front. As for the vertical displacements, the soil surface has a sudden and progressive settlement, due to the tail void created as the shield moves forward. This effect has been reproduced in the numerical model considering the transition pressure of 240kPa and an effectively applied pressure of 180kPa.

Comparing the values of pressures considered in the calculations, that allowed a similar pattern of horizontal and vertical displacements to be obtained to that observed in the field, with those referred by Lee *et al.* (1999), it is possible to conclude from the numerical model that at the front face there was a constant pressure of 260kPa, while the available information indicates that during construction there was a pressure of 210kPa from a distribution that is estimated to have a linear increase from crown to floor of 75kPa. Regarding the grout injection pressure at the tail void, the available value of the field pressure is 250kPa, while in the calculations a 240kPa transition pressure and an installed pressure of 180kPa have been assumed. The values of pressures used in the numerical model can, therefore, be considered of the same magnitude to those registered during construction.

5.2 Horizontal displacements along a vertical plane parallel to that containing the tunnel axis

The calculated and observed horizontal displacements of the soil along a vertical plane parallel to that containing the tunnel axis and at a 6m offset are presented in Figure 4. From the figure it is possible to conclude that, although the best prediction of results is obtained for the vertical plane of the axis (Figure 2), the pattern of displacements seems, also here, to have been reasonably reproduced. It should be pointed out that the displacements observed in the field are very small, always lower than 5mm. The biggest differences occur in the distribution corresponding to the zone after the advancement of the shield front face. This is mainly due to the fact that, in the numerical model, the mobilization of shear stresses due to friction/adhesion between soil and shield has not been considered. This effect would naturally lead to an increase of soil displacements in the vicinity of the shield in the direction of the advancement.

5.3 *Surface settlement troughs along a cross-section*

As an example of the surface settlements troughs along a transverse cross-section of the tunnel, the numerical and the field results are presented in Figure 5 for a cross-section three diameters (around 18m) behind the front face, after excavation. As can be seen, numerical results are very close to the observed ones, although the former show higher values of settlements.

5.4 *Subsurface settlement profiles along a cross-section*

For the subsurface settlement profiles, the results show that, as the front face gets closer to the instrumented section until after the shield body completely moves away from that section the soil above the sidewall level generally heaves, due to the application of a face pressure that is higher than the in-situ earth pressure. In Figure 6 the situation is presented for a cross-section at a distance of two diameters (around 12m) behind the front face, after excavation. This figure shows that the observed

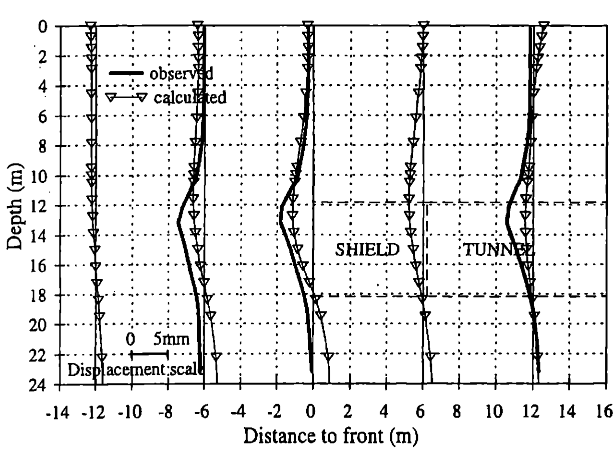

Figure 4. Horizontal displacements along the vertical longitudinal plane placed at 6m from the axis.

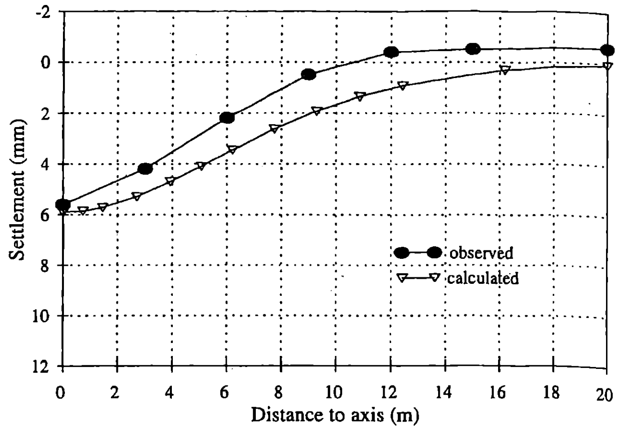

Figure 5. Surface settlements along the transverse cross-section at three diameters (around 18m) behind the face, after excavation.

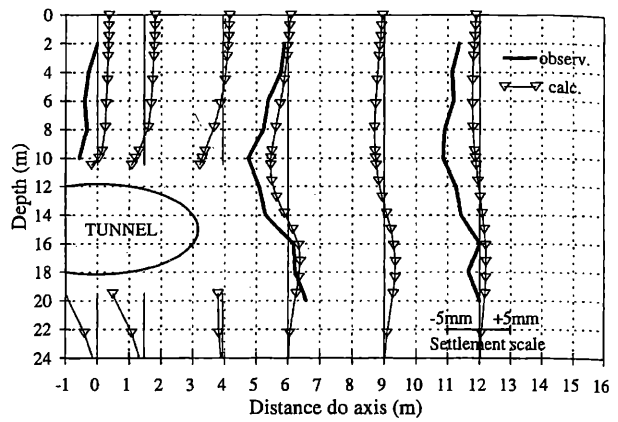

Figure 6. Settlements along the transverse cross-section at two diameters (around 12m) behind the face, after excavation (settlement values are indicated by the horizontal distance between the diagram and the vertical line of reference).

behaviour of the soil is reasonably reproduced in the numerical model, in particular close to the tunnel.

5.5 *Subsurface horizontal displacements along a cross-section*

Regarding the horizontal displacement pattern with depth, the soil again generally moves away from the tunnel because of the application of a front pressure higher than the in-situ earth pressure. Lateral displacements, which are more significant close to the sidewalls, tend to increase slightly as the shield moves forward and the grout is injected at the tail void. Higher values have been registered at levels corresponding to the top third of the tunnel section. In Figure 7, for the cross-section at two diameters (around 12m) behind the front face, the development with depth of the horizontal displacements is presented. These displacements are very small in magnitude, typically lower than 5mm. The behaviour of the soil mass seems to have been reasonably reproduced taking into account the close proximity of numerical and observed results.

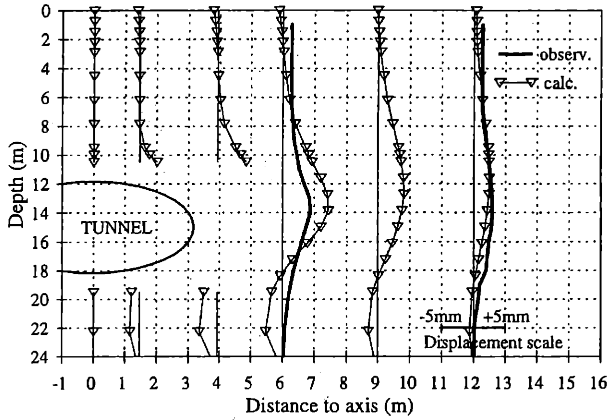

Figure 7. Horizontal displacements along the cross-section at two diameters (around 12m) behind the face, after excavation (displacement values are indicated by the horizontal distance between the diagram and the vertical line of reference).

5.6 *Earth pressures around the tunnel lining*

On the outer surface of the tunnel lining six vibrating wire cells were installed in order to evaluate the earth pressures development. The values registered by those cells are presented in Figure 8, together with the results from the numerical analyses. It must be pointed out that numerical results for cell C5 are not shown because this point is the only one with direct contact between lining and soil and the calculated pressure has an unrealistically high value. The type of pressure diagram determined for this cell C5 is very similar to that of cell C4, but values for the former are 2.5 times higher than those for the latter.

Field results show that cells placed at higher levels (such as C1, C2, C3 and C6) initially registered pressure values around 210kPa, while the cells placed at lower levels reached pressure values around -250kPa. This effect must have been associated, according to Lee *et al.* (1999), to deficiencies in filling the tail void with grout, which, as the shield moves forward, tend to be eliminated. At this stage: the higher values of pressure that were registered close to the tunnel floor are due to the weight of the lining; the intermediate values that were registered close to the tunnel crown are due to the weight of the soil over the tunnel; and the lower values close to the tunnel sidewalls are probably associated only with the pressure of the grout injected at the tail void.

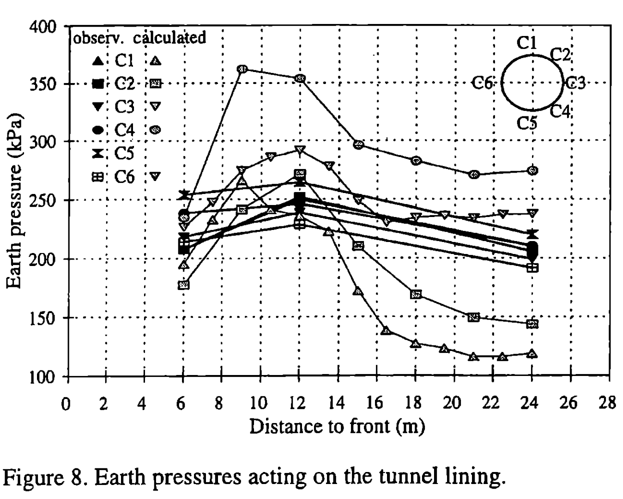

Figure 8. Earth pressures acting on the tunnel lining.

At a cross-section at three diameters (around 18m) behind the cross-section where tail void injection took place (that is, around 24m from front face), the pressure cells registered a generalized reduction of the applied pressures as the grout hardened. It should be noticed that cells C3 and C6, close to the sidewalls, that are considered to reproduce the effect of the grout injection the best, registered pressure values of around 190kPa. This value is very similar to that adopted for the grout injection pressure in the numerical analyses, initially estimated only based on the displacements of the soil along the longitudinal vertical plane. The results of the numerical analyses reasonably reproduce the observed ones for the 14m close to the excavation front face. Only cell C4 shows bigger differences between results, probably due to the proximity of the tunnel floor. A different behaviour of the various cells has been observed for distances from the front face greater than 14m. Close to the crown, analysis results show a reduction of earth pressures, which may be due to an arching effect created in that area. This effect may have produced a stress transfer to the lower levels of the tunnel, leading to higher pressures at cell C4. Analysis values for pressures of cells C3 and C6 have reproduced the observed ones reasonably well.

6 CONCLUSIONS

In this paper, a three-dimensional numerical analysis of soil tunnelling is presented. The analysis is focussed on the tunnel construction of the Shanghai Metro - Line 2, in Shanghai, China, where tunnel boring machines with earth pressure balanced shields have been used. The available information concerning field observation results is significant and part of it has been used to validate the numerical model used.

The agreement between numerical and observed results for the construction phase close to the front face of the tunnel is reasonable. Although some differences exist in the values of some of the observed parameters, the general behaviour of the soil mass seems to have been predicted reasonably well. It should be noticed that soil models (always linear elastic perfectly plastic type) have been conditioned by the available geotechnical information, and, even for these models, the values of some fundamental parameters had to be estimated based on empirical correlations with other parameters (this was the case, for example, with the undrained deformability modulus).

Regarding the numerical model, the construction sequence has been considered simply by adding and removing "brick" elements to the initial finite element mesh, changing the mechanical characteristics of those elements and applying and removing different types of loads. No contact elements have been considered. Reference must be made to the way the shield has been modelled. Since one of the goals of the design was to produce a general heaving close to the front face (by applying at that face a support pressure higher than the in-situ earth pressure), the shield has been modelled with low rigidity elements but with, simultaneously, the application of a support pressure at the excavated surface (named transition pressure) which has an intermediate value between the one applied at the front and the one effectively installed at the tail void. The obtained results indicate that this type of model reasonably reproduces the observed behaviour.

Another reference should be made to the fact that the observation results show that the advancement of the shield produces, in the surrounding soil, displacements with a significant component in the direction of the movement, naturally associated with the friction/adhesion effects between shield and soil, which was not considered in this model.

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