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Tunnelling on urban areas: 3D numerical analysis of soil/structure interaction

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ABSTRACT: Shallow tunnelling performed by TBM induces volume loss, mainly due to the conical shape of the machine and the consolidation of the injected grout. Ground movements induced on the surface are susceptible to cause damages to surrounding structures. To estimate these damages, it is not sufficient to apply the greenfield deformation, because the structure itself contributes to stiffen the soil. To accurately take into account this soil-structure interaction phenomenon, a three-dimensional numerical analysis is required. This paper presents a finite difference analysis performed with the software FLAC\textsuperscript{3D} of TBM tunnelling under a concrete six-floor building. Tunnelling is simulated by lateral convergence of the tunnel walls.

1 INTRODUCTION

In urban areas, the estimation of the impact of tunnelling on surrounding structures is very important. In order to control the soil movements, and when the tunnel length is large enough, underground works can be performed using a TBM (Tunnel Boring Machine). However, shallow tunnelling performed by TBM induces volume loss, mainly due to the conical shape of the machine and the consolidation of the injected grout. Ground movements are induced on the surface and are susceptible to cause damages to surrounding structures.

A first approximation to predict the damages caused to surrounding structures is done by applying the greenfield deformations induced by tunnelling to the structure’s foundations. “Greenfield” deformations are those estimated by neglecting the presence of the structure. This method is recommended by AFTES (1999). The greenfield deformations can be calculated by empirical (O’Reilly & New 1982, Peck 1969), analytical (Panet 1995, Sagaseta 1987) or numerical methods (Oteo & Sagaseta 1982, Swoboda et al. 1989).

However, this is conservative (Mroueh & Shahrour 2003) because the presence of the structure has an influence on the soil displacements. To accurately take into account this soil-structure interaction phenomenon, a numerical analysis is required.

Potts & Addenbrooke (1997) used two-dimensional numerical calculations considering the structure as an equivalent weightless beam with variable stiffness. They showed the structure’s rigidity influence on surface ground movements induced by tunnelling. Franzius & Addenbrooke (2002) then analyzed the influence of the structure’s weight. They showed that the weight has very low influence on ground movements when rigidity increases.

Nevertheless, with 2D simulations it is worth noting that an empirical parameter such as the deconfinement ratio or the volume loss in tunnel has to be considered as remarked by Benmebarek et al. (1998). Dias et al. (1999) compared results from 2D and 3D numerical simulations with experimental data. They showed that the surface settlement trough obtained with the 3D calculation is more realistic than the trough obtained with the 2D calculation, even with a simple constitutive model for the soil. Moreover, it is impossible to study the damages induced on the structure in the tunnel axis direction when using 2D simulations.

Some authors have already performed three-dimensional numerical analysis. Mroueh & Shahrour (2003) compared the results of a soil-structure interaction calculation of tunnel excavation below a structure with the results obtained by imposing the greenfield movements upon the structure. They showed that this last method is very severe in terms of induced forces in the structure. Netzel & Kaaalberg (2000) have modelled the interaction between TBM digging and masonry structures in order to obtain specific damage criteria.

This article presents a three-dimensional numerical analysis of the soil-structure interaction phenomenon during shallow tunnelling. The tunnel excavation is a simplified simulation of the real phases of a TBM based on the concept of volume loss. The soil behaviour is elastic perfectly plastic. The structure is composed of columns and floors founded on a raft. The influence of the structure’s stiffness is studied. Results are analyzed in terms of ground surface displacements.
2 EXPERIMENTAL SECTION

The case of the Lisboa subway is studied; the experimental section is located near the Ameixoeira Station and is shown on Figure 1.

The tunnel has a 9.8 m diameter (D) and is 26 m deep, which corresponds to 2.65 D (shallow tunnel). The geotechnical properties, given by Ribeiro e Sousa et al. (2003), are summarized on Table 1. These properties result from standard in situ and laboratory tests (Geocontrol 1999). The tunnel is located in the silty sand layer, which presents relatively poor mechanical properties.

In the calculation $\gamma' = \gamma - \gamma_w$ was used under the ground water level. Due to the fast pore water pressure dissipation observed in situ, the calculation was performed in drained conditions.

3 NUMERICAL MODEL

3.1 Numerical method

The numerical analysis is performed using FLAC$^{3D}$ software, developed by Itasca (2002). FLAC$^{3D}$ is a three-dimensional explicit finite-difference program, based on a “Lagrangian” calculation as explained by Billaux & Cundall (1993). This program permits to solve stress – strain problems in a continuum.

3.2 Grid

Due to the symmetry conditions, only half of the ground mass is modelled (Fig. 2). The symmetry plane is the Y-Z plane. The tunnel axis is along the Y-axis, which will be called the longitudinal direction and the X-axis is the transverse direction. The model is 90 m long, 100 m (=10.2 D) wide and 51 m high. The soil layer under C5 (tab. 1) is considered rigid, which permits us to assign boundary conditions on this face (nodes fixed). We considered that there is no tunnelling influence 100 m far from the tunnel axis, then no transverse displacements were allowed on the $X=100$ m plane. The structure will be placed in the middle of the model, on the tunnel axis, this section will be studied more precisely, then no longitudinal displacements are allowed on the planes Y = 0 and 90 m, located far from this section.

There are 58 elements along the tunnel axis with a length varying from 4.90 m for the first and the last element to 1 m for elements in the middle of the model.

3.3 Constitutive law

The constitutive law assigned to the soil is elastic perfectly plastic with a Mohr-Coulomb failure criterion. The parameters are those given on table 1. The flow rule is non-associated and the dilatancy angle was taken according to the Vermeer rule as $\psi = \varphi - 30^\circ$.

Before failure, soil stiffness is taken constant, which seems not to be so accurate for a settlement analysis. In fact, the Mohr-Coulomb law is more relevant for limit analysis problems.

Table 1. Geotechnical properties.

<table>
<thead>
<tr>
<th>Name</th>
<th>Soil type</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
</tr>
</thead>
<tbody>
<tr>
<td>E [MPa]</td>
<td>15</td>
<td>15</td>
<td>266.5</td>
<td>44.7</td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.35</td>
<td>0.4</td>
<td>0.35</td>
<td>0.35</td>
<td>0.37</td>
<td></td>
</tr>
<tr>
<td>$c$ [kPa]</td>
<td>5</td>
<td>5</td>
<td>10</td>
<td>0</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td>$\varphi$ [$^\circ$]</td>
<td>30</td>
<td>32</td>
<td>37</td>
<td>35</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>$\gamma$ [kN/m$^3$]</td>
<td>20.5</td>
<td>20.7</td>
<td>20.5</td>
<td>20.65</td>
<td>20.4</td>
<td></td>
</tr>
<tr>
<td>$K_o$</td>
<td>0.6</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
<td>1.05</td>
<td></td>
</tr>
</tbody>
</table>
The simple constitutive law adopted in this calculation can be justified by the fact that calculation time is considerably increased when using a more complex law, especially when performing a three-dimensional analysis. This analysis constitutes a preliminary study to a more accurate analysis which will use an elastoplastic constitutive law, showing an isotropic and/or kinematic hardening rule, like the CJS model for sands (Cambou & Jafari 1988) and a Cam Clay type model for clays (Roscoe & Schofield 1963). Settlements problems should also take account of small strain soil modulus.

3.4 Simulation of excavation

The adopted excavation process for the calculation is a simplification of the confinement-deconfinement phases induced by the TBM. Benmebarek et al. (1998) showed that when using a TBM, the surface settlements are mostly induced by the lateral convergence of the tunnel walls and not by the tunnel face stability. This permitted us to adopt the following hypotheses: the soil displacements at the tunnel face were blocked, simulating a perfect equilibrium of the confinement pressure. The maximum lateral soil displacements had a linear variation on the distance of 20 m. After this distance, the maximum lateral displacements were constant, as shown on Figure 3.

It is important to be aware that the displacements of the tunnel walls were not imposed, but they converged freely until reaching the limit imposed or the static equilibrium. That led to bottom displacements smaller than tunnel crown displacements. Figure 4 shows a transverse section of the tunnel after free convergence of the tunnel walls. The crown nodes have reached the imposed limit whereas the bottom displacement is only half of this limit.

The adopted process simulates the principal excavation phases:

- Conical shape of TBM
- Grout injection
- Grout consolidation
- Setting of the concrete rings.

The initial position of the tunnel was \( Y = 0 \) m and the numerical phases of excavation were as follows:

- excavation on one element length (varying from 4.90 m to 1 m),
- fixation of the tunnel face nodes,
- convergence of tunnel walls until reaching the given displacement shape, or until reaching the static equilibrium of the full model,
- if a node reaches the limit, it is fixed,
- when the model equilibrium is reached, all the nodes are freed,
- translation of the loading system on one element length.

The excavation is ended when the model is entirely bored, then the tunnel face is at \( Y = 90 + 20 = 110 \) m which corresponds to the entire model length added to the distance between the tunnel face and the position where the lateral displacement is constant.

The maximum radial displacement of the tunnel walls is related to the volume loss in tunnel (ratio between the volume lost in the tunnel and the initial volume of the tunnel). A parametric study of the volume loss influence on the surface displacements and
stress induced in the structure is to be found in Jenck & Dias (2003). The volume loss ranges between 0.5% and 5% and linearity between surface settlements, stresses induced in the structure and volume loss in the tunnel was found.

3.5 Modelling of the structure

The modelled structure is a simplification of existing concrete buildings. It consists of columns and slabs, founded on a raft. Columns have a square section of 0.4 m × 0.4 m, slabs and raft are 0.3 m thick. The building is 28 m high (7 levels of 4 m). The structure elements behave elastically, with properties of long term reinforced concrete: \( E = 19 \text{ GPa}; \nu = 0.2 \). No interface is considered between the raft and the ground mass: the raft is bound to the soil elements.

There is no eccentricity between the tunnel and the building then symmetry is kept. The building is placed transversally to the tunnel axes because it seems to be the more unfavourable case (Dias & Kastner 2002). Figure 2 presents the numerical model coupling soil and structure.

The structure is introduced in the numerical model in one step. Then equilibrium of the model is reached. The structure is only loaded by its own weight (\( \gamma = 25 \text{ kN/m}^3 \)). After this equilibrium the soil displacements are initialized in order to study only the effect of tunnelling. The forces in the structural elements are not initialized: this step represents the initial state in terms of internal forces in the structure. Finally, the obtained numerical model is excavated.

4 GREENFIELD CALCULATION

A reference calculation was done without considering any structure on surface (greenfield calculation), adopting a volume loss of 5%. This volume loss was chosen deliberately high, in order to highlight the influence of tunnelling. With these numerical assumptions, the volume loss on site would be 0.5% (Jenck & Dias 2003).

4.1 Surface settlements

Figure 5 shows the surface settlements along the tunnel axis, for a length bored of 45 m and for the entire model length bored. For a length bored of 45 m, settlements are observed 20 m downstream and the maximum settlement is reached 30 m upstream. At the final state, the settlement is 0.04 m. The settlement is not constant along the model axis due to the varying element length.

Figure 6 depicts the surface settlements at the final state according to the distance to the tunnel axis, normalized by the tunnel diameter D. This curve is called transverse settlement trough. It shows a Gaussian shape, as experimentally observed, following the Peck (1969) relation:

\[
S = S_{\text{max}} \exp \left( -\frac{x^2}{2i^2} \right)
\]

where \( S = \) surface settlement at distance \( x \) to tunnel axis, \( S_{\text{max}} = \) maximum settlement above the tunnel axis, \( i = \) distance of the inflexion point to the tunnel axis, which is here equal to 1.8D.

At the distance of 10.2D to the tunnel axis, which corresponds to the edge of the model, there is a residual settlement of 1 mm, whereas no settlements were expected far from the tunnel axis. This is probably due to the use of the Mohr-Coulomb law, which is known to give settlement troughs wider and shallower than experimentally observed. Bernat & Cambou (1998) found more realistic settlement troughs by using an elastoplastic model and they warn against elastic-perfectly plastic models. In fact the value of \( i \) found here is larger than those given by several empirical relations (Attewell 1977, O’Reilly & New 1982, Mair et al. 1993).

4.2 Surface horizontal strain

Figure 7 depicts the horizontal strain in a transverse section at the final state. Above the tunnel axis, there
is a zone in compression, when moving away from the tunnel axis, the soil is in extension. The demarcation between both zones corresponds to the inflexion point of the settlement through.

5 SOIL-STRUCTURE INTERACTION CALCULATION

A first calculation was performed by introducing the presented building on surface and a tunnel volume loss equal to 5%. Then the influence of the structure’s stiffness is studied by performing simulations including different structures of increasing rigidity.

5.1 Surface settlements

Figure 8 depicts the surface settlements in the longitudinal section, for different lengths bored. The structure has an influence on the surface settlements when the TBM is entirely passed under its foundations (length bored of 66.5 m).

Figure 9 shows that the settlements are increased in presence of the building. However, the trough under the building’s foundations remains a Gaussian curve and the point of inflection remains unchanged.

5.2 Surface horizontal displacements

Figure 10 shows the surface horizontal displacements in a transverse section under the building, at the final state, for the greenfield calculation and for the coupled calculation. This figure clearly shows that the horizontal displacements are negligible under the structure compared to the greenfield case. This is mainly due to the fact that the raft is bound to the soil, and the raft has a large axial stiffness. Consequently, it would be very conservative to apply the greenfield deformation to the structure foundations when estimating the damages induced by tunnelling. This was also pointed out by Mroueh & Shahrour (2003) and by Miliziano et al. (2002).

5.3 Influence of the structure’s stiffness

In order to study the influence of the structure’s stiffness, three additional buildings were considered, presenting an increasing global rigidity. In the following, the previously described structure will be called “structure 1”. Structure 2 is based on structure 1 and it has peripheral walls with openings and a lift cage, located in the centre of the building. The model coupling ground and structure 2 is shown on Figure 11. Structure 3 and 4 are based on structure 2 with respectively one and three 4-m-underground-levels, as shown on Figures 12 and 13. These underground levels modify the ground weight above the tunnel, but due to
the excavation procedure that consists in imposing displacements, only the influence of the rigidity is studied.

Figure 14 illustrates the influence of the structure’s stiffness on the surface settlements. The settlements are only increased in presence of the structure 1. In presence of the structures 2, 3 and 4, the surface settlements are reduced compared to the greenfield calculation. It has to be notified that the surface settlements of structure 3 and 4 are not those under their bottom, but on the edge of the structure, at the same altitude as for structure 1 and 2. This figure shows that the increase of the structure’s rigidity has more influence than the increase of the structure’s weight. Only for the structure 1 the weight has more influence than the rigidity.

Obviously, the horizontal displacements are negligible under all the structures.

6 CONCLUSION

This paper highlights the soil-structure interaction during TBM shallow tunnelling. The presence of the structure led to negligible horizontal displacements under its foundations, compared to the greenfield calculation. To estimate the damages induced by tunnelling, it is conservative to apply the greenfield deformations to its foundations. The influence of the structure’s rigidity was studied. The larger the rigidity, the smaller are the surface settlements.

The tunnel excavation process is a simplification of the real phases of a TBM digging, which takes into account the volume loss in the tunnel.

The use of an elastic perfectly plastic soil constitutive law in this study is debatable, because important features of soil behaviour which are important for settlements estimation are not taken into account. Moreover, the structure behaves elastically, which does not consider most of the reinforced concrete behaviour features, like load redistribution due to cracking for instance.

REFERENCES


