South Toulon Tube: Numerical Back-analysis of In-situ Measurements

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Full face excavation with ground reinforcement has become a common technique to build large tunnels in soft rock or hard soil. Nevertheless, at the design phase, it remains difficult to assess the effect of the different construction and reinforcement elements on the control of the ground movements and settlements. In order to improve the understanding of ground response to this tunnelling method, a monitoring section has been installed during the construction of the south Toulon tunnel (France). An important database was obtained and subsequently used for numerical back-analysis. A 3D FE calculation (PLAXIS 3D v.2010), modeling the real pre-reinforcements system and workflow steps, permitted to simulate the in situ measurements.

The south Toulon tunnel will connect motorways A50 and A57 from Nice to Marseille. It is an urban shallow tunnel, 12 m in diameter and 1820 m in length, excavated through very difficult heterogeneous soils and with a limited overburden (maximum of about 35 m). The construction method used is full face excavation and ground reinforcement ahead of the tunnel face by pipe umbrellas (long forepoling) and face bolting. The construction sequences and the amount of reinforcement were continuously adapted to the overburden, the soil conditions and the measured settlements (Janin et al [2011]).

In addition to the regular settlement measures, a specific monitoring zone was set up to improve the understanding of ground response and to collect precise data for validating a 3D numerical model. The complex phenomenon of interaction between the excavation process, the reinforcements and the ground reaction demand in fact a 3D approach. In this paper, the numerical model is described and the simulation is validated by comparing it to in situ measurements.

2.2 Geology
The geological profile of the section has been drawn (see Fig. 1) based on the borehole investigations. They showed that the geological stratigraphy is generally horizontal and the degree of alteration of the phyllitic bedrock is considerably high. Despite the variations of materials characteristics (especially in the bedrock), average ground properties of the different strata were proposed at the detailed design phase (see Table 1).

2.3 Excavation method
The south Toulon tunnel was excavated on the basis of the so-called “ADECO-RS” method developed by Lunardi [2008]. According to this method, pipe umbrellas (6° or 14° of inclination) and horizontal face bolts were installed. The excavation process progressed by 1.5 m steps and after each step one HEB 180 steel rib was installed. In this zone, the tunnel invert (HEB 220 + concrete) was realized with a distance to tunnel face of about 39 m.
### Three-dimensional Back-analysis

A three-dimensional model was realized to simulate the tunnel excavation process of the area where the monitoring zone had been placed. The analysis was carried out by means of the three-dimensional finite element code PLAXIS 3D (version 2010).

#### 3.1 Geometry

Considering the geometry symmetry, only half of the entire domain is taken into account in the analysis as shown in Fig. 2. The real shape of Toulon tunnel with a cross section area of 120 m² is imported from CAD. The tunnel is then modeled by extruding the surface in X direction. The tunnel temporary support and the pre-reinforcement are modeled and the process is explained in the following paragraph.

In order to avoid boundary effects, the extension of the mesh is equal to 150 m in X and Y directions and 70 m in Z direction. The cover depth is about 25 m. A three-dimensional non uniform mesh with smaller elements around the excavation is used. Finally, the model contains 158000 tetrahedral elements and 247000 nodes. All movements are fixed at the bottom of the model and horizontal displacements are blocked in model’s lateral faces.

#### 3.2 Mechanical parameters and simulation of the excavation process

The ground is represented by the non linear elasto-plastic HS model (Hardening Soil Model) implemented in the PLAXIS code. Hejazi [2008] showed, in a tunnel excavation study, that this model might produce ground deformations that better fit with in situ measurements than using the linear elastic/Mohr-Coulomb model. The geotechnical parameters considered in the simulation are listed in Table 1. The initial stress field is considered as isotropic (in conformity with the studies previously realized on Toulon soils by Constantin [1988] and Dias [1999]).

In the numerical model, the shotcrete at tunnel face, the excavation support and the tunnel invert are modeled by “plate” elements with a linear elastic behavior. The mechanical parameters are defined in Table 2. The parameters of support and tunnel invert are determined by homogenization based on steel and shotcrete characteristics and rib’s spacing. A rigid interface is considered between the support and the ground.

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<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soils</th>
<th>$\gamma$ (kN/m³)</th>
<th>$E_{\gamma} = \frac{E_{\gamma}}{E_{\gamma}}$ (MPa)</th>
<th>$E_{\gamma} = \frac{E_{\gamma}}{3}$ (MPa)</th>
<th>$c'$ (kPa)</th>
<th>$\varphi$ (°)</th>
<th>$\nu$ (°)</th>
<th>$\gamma_v$</th>
<th>$K_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 3.5</td>
<td>Fill</td>
<td>19</td>
<td>1.6</td>
<td>4.8</td>
<td>2</td>
<td>20</td>
<td>0</td>
<td>0.2</td>
<td>0.5</td>
</tr>
<tr>
<td>3.5 to 5.9</td>
<td>Colluviums</td>
<td>20.8</td>
<td>40</td>
<td>120</td>
<td>10</td>
<td>30</td>
<td>0</td>
<td>0.2</td>
<td>0.5</td>
</tr>
<tr>
<td>Below 5.9</td>
<td>Bedrock</td>
<td>24.2</td>
<td>240</td>
<td>720</td>
<td>40</td>
<td>25</td>
<td>0</td>
<td>0.2</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 1: Main geotechnical parameters

<table>
<thead>
<tr>
<th>Support type</th>
<th>Support description</th>
<th>E (GPa)</th>
<th>Thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Walls’ tunnel</td>
<td>HEB 180 (spacing 1.5 m) + 25 cm shotcrete</td>
<td>13.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Tunnel invert</td>
<td>HEB 220 (spacing 1.5 m) + 30 cm shotcrete</td>
<td>14</td>
<td>0.3</td>
</tr>
<tr>
<td>Tunnel face</td>
<td>15 cm shotcrete</td>
<td>10</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 2: Mechanical parameters of tunnel support

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Figure 2: Geological section and instruments
The calculation is carried out in drained conditions. The tunnel excavation is simulated in a first stage by 10 steps, 3 meters long, followed by 60 steps with an excavation length of 1.5 m as done in situ. In each phase, the tunnel lining is installed 1.5 m behind the tunnel face, on which the shotcrete application is simulated as well. The tunnel invert is activated 39 m behind the tunnel face progress.

The pre-reinforcements, i.e. the pipes constituting the long forepoling and the face bolts, are simulated using “embedded pile” structures (see Fig. 3). Thanks to in situ pullout tests, a realistic limit skin resistance could be introduced between the piles and the soil. The maximum friction resistance measured along the soil/bolt interface is equal to 135 kN/m.

Figure 4 shows the main characteristics of the pre-reinforcement system considered in the numerical simulation, based on works realized in the studied zone.

As far as the umbrella pre-support is concerned, 13 autodrilling pipes (51/33 mm) were taken into account and renewed every 9 m. The same inclination of 6° has been kept all along the model in order to simplify the meshing.

As for the face bolts, a constant length of 18 m is considered. Besides, Dias and Kastner (2005) proved that the bolting system is characterized by the global stiffness (E*S). Therefore, the same number of bolts (20) is kept all along the model in order to simplify the meshing. The real bolting density, installed in situ, is simulated varying proportionally the bolts’ modulus and the friction resistance based on the material characteristics listed in Table 3.

### Table 3: Face bolts characteristics

<table>
<thead>
<tr>
<th></th>
<th>Steel bolts</th>
<th>Fibreglass bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus E (GPa)</td>
<td>210</td>
<td>40</td>
</tr>
<tr>
<td>Cross section A (m²)</td>
<td>0.488 10⁻⁴</td>
<td>0.8 10⁻⁴</td>
</tr>
<tr>
<td>Moment of inertia I (m⁴)</td>
<td>0.0327 10⁻⁶</td>
<td>~ 0</td>
</tr>
</tbody>
</table>

Figure 5a shows the settlements of different surface points, placed directly above the tunnel axis in the analyzed zone, against their distance from the tunnel face. The excavation started to influence settlements more or less 30 m ahead of the tunnel face. Afterwards, settlements accelerated and finally reach a threshold 50 m behind the tunnel face with a settlement of around 20 mm. The progression of settlements obtained with the numerical calculation fits with the measurements evolution. Similarly, three-dimensional modeling seems to well represent the shape of the transverse settlement trough both in terms of maximum settlement and trough width (see Fig. 5b).

The final measurements of inclinometer movements in the monitoring section plane.
showed two important phenomena (see Fig. 6). On one hand, the first few meters of inclinometers converged towards the tunnel and, on the other hand, a local convergence ("belly") is observed at the tunnel level. The numerical approaches correctly estimate the two phenomena.

A comparison between the tunnel support deformation obtained with numerical calculation and that inferred from in situ measurements is also performed. The 3D model proves to be able to represent the support deformation with an acceptable accuracy. The difference between the measurements and the numerical results, especially in the bottom part of the rib, is probably related to the simplifications related to the way of modeling (homogenization of steel and shotcrete characteristics, excavation of the stress carried out at the same time as that of the tunnel face).

5 Conclusions

The monitoring section, installed during the construction of Toulon tunnel’s south tube, allows analyzing the evolution of soil deformation during the excavation progress. It also permits to create an important database, used later to validate a numerical simulation.

A 3D numerical model was performed with PLAXIS 3D, taking into account the real excavation process and the pre-reinforcements installed in situ. The good fitting with the different measurements recorded in situ shows that the three-dimensional numerical modeling, with discretization of the inclusions, is a reliable tool to simulate the complex phenomenon of interaction between the excavation process, the reinforcements and the ground reaction. Finally, PLAXIS 3D seems to be a useful and efficient tool to predict the movements caused by the excavation of a tunnel, realized with ground pre-reinforcements.

References


Figure 6: Comparison between in situ measurements and 3D simulation - Inclinometer movements in monitoring section plane

Figure 7: Comparison between in situ measurements and 3D simulation – Deformation of the tunnel support