South Toulon tube: 3D numerical back-analysis on in situ measurements

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ABSTRACT: 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, modeling the real reinforcement system and construction stages, was then performed. The satisfactory matching between the numerical results and the in situ measurements shows that the three-dimensional numerical model with an explicit modeling of the reinforcement elements is a reliable tool to simulate the complex phenomenon of interaction between the excavation process, the reinforcements and the ground reaction.

1 INTRODUCTION

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 (about 35 m maximum). The construction method associated full face excavation and ground reinforcement ahead of the tunnel face by pipe umbrellas and face bolting. The construction sequences and the amount of reinforcement were continuously adapted to the overburden, the soil conditions and the measured settlements. During the tunnel construction, the surface and building deformations were measured, with high frequency, by total stations and all measurements were immediately stored in a database with real time access for all the actors. This efficient tool permitted to constantly adapt the tunnelling process as described by Janin et al. (2011). In addition to the regular measures, a specific monitoring zone was set up to improve the understanding of ground response and to collect precise data used subsequently for numerical back-analysis (Janin 2012).

The phenomenon of interaction between the excavation process, the reinforcements and the ground reaction is a three-dimensional problem, especially in the zone of the tunnel face. This was clearly demonstrated by Barla & Barla (2004) on the basis of the analysis of the stress path around the tunnel face. A three-dimensional numerical modelling is therefore necessary to study this phenomenon in all its complexity (Mollon et al. 2011). With this approach, the tunnel geometry, the initial stress state, even anisotropic, the tunnelling method and the phasing of the work can indeed be taken into account. In particular, for the tunnel excavated at full face with ground reinforcement method, as the south Toulon tunnel, only a 3D model can correctly simulate the behaviour of inclusions. An homogenisation approach, often used in 2D simulations, leads to overestimate the positive effects of reinforcements (Volkmann et al. 2006; Eclaircy-Caudron et al. 2006; Dias et al. 2002, Dias 2011).

The following paper presents the instrumentation set up during the south Toulon excavation and the most important in situ results. Afterwards, the 3D numerical model, simulating the south Toulon excavation process is described. In particular, its ability to reproduce the real measurements obtained in situ is tested.

2 ANALYZED SECTION

The monitoring section is situated in “Alexandre I” garden on the west side of Toulon at chainage PM 882. The tunnel face reached the section in March 2009.
2.1 Instrumentation

The monitoring set up in the analyzed section is presented in Figure 1.

The instrumentation from the surface consists of 2 inclinometers on both sides of the tunnel, 1 vertical extensometer on the tunnel axis and 3 surface target prisms (X, Y, Z) close to the previous instruments. In addition, 4 radial extensometers, 6 vibrating strain gauges placed on the steel rib, 5 pressure cells and convergence targets were installed from the tunnel.

2.2 Geology

A geological profile of the section has been drawn (Figure 1) thanks to the drilling investigations. They showed that the geological stratigraphy is generally horizontal and that the degree of alteration of the basement made of quartzophyllades is considerably high. 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). This method is based on the principle that protecting and improving the strength and deformation characteristics of the ground ahead of the tunnel face allows to reduce tunnel deformations and surface settlements in case of shallow tunnels (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.

3 NUMERICAL SIMULATION

A three-dimensional model was realized to simulate the tunnel excavation process in 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 one half of the entire domain is taken into account in the analysis as shown in Figure 2. The real shape of Toulon tunnel was modeled, having 120 m² of section (height of 11.2 m and width of 12.7 m). In order to avoid boundary effects, the extension of the mesh was equal to 150 m in X and Y directions and 70 m in Z direction. The cover depth was about 25 m. A three-dimensional non uniform mesh with smaller elements around the excavation was used. Finally, the model contains 158000 tetrahedral elements and 247000 nodes. All movements were fixed at the bottom of the model and both horizontal displacements were blocked in model’s lateral faces.

3.2 Geotechnical parameters

The ground was represented by the non linear elastoplastic HS model (Hardening Soil Model) implemented in PLAXIS code. This model, especially in excavation problem, permits to obtain ground deformations that better fit with in situ measurements than using the linear elastic perfectly plastic Mohr-Coulomb model (Hejazi et al. 2008). The principal geotechnical parameters considered in the simulation are listed in Table 1. The tangent stiffness for primary

![Figure 1. Geological section and instruments.](image1)

![Figure 2. Three-dimensional model.](image2)

<table>
<thead>
<tr>
<th>Soils</th>
<th>$\gamma$ (kN/m³)</th>
<th>$E^{ref}_{50}$ (MPa)</th>
<th>$E^{ref}_{2}$ (MPa)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>19</td>
<td>1.6</td>
<td>4.8</td>
<td>2</td>
<td>20</td>
</tr>
<tr>
<td>Colluviums</td>
<td>20.8</td>
<td>40</td>
<td>120</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>Basement</td>
<td>24.2</td>
<td>240</td>
<td>720</td>
<td>40</td>
<td>25</td>
</tr>
</tbody>
</table>
oedometer loading ($E_{\text{ref}}^{\text{oed}}$) was taken equal to the secant stiffness in triaxial test ($E_{\text{50}}$). The initial horizontal stress field was considered isotropic ($K_0 = 1$), in conformity with the studies previously realized on Toulon soils by Constantin et al. (1988) and Dias (1999).

The dilatancy angle was taken equal to zero and the power of stress dependent stiffness ($m$) equal to 0.5. These parameters, proposed for the South tunnel final project, were obtained mainly using pressuremeter tests and using a 2D numerical back analysis on settlements measured during the excavation of the previously built North tunnel.

### 3.3 Simulation of the supports and the excavation process

In the numerical model, the shotcrete at tunnel face, the tunnel lining and the tunnel invert were modeled by “plate” elements with a linear elastic behavior. The mechanical parameters are defined in Table 2. The parameters of tunnel supports were determined by an homogenized calculation considering the steel and shotcrete characteristics.

The pre-reinforcements, i.e. the pipes constituting the umbrella and the face bolts, were simulated using “embedded pile” elements. They consist in pile elements that can be placed in an arbitrary direction and are able to support axial and bending forces. They interact with the ground by means of special interface elements. A linear realistic limit skin resistance ($T_{\text{max}} = 135$ kN/m) has been introduced between the embedded piles and the soil thanks to the results obtained from in situ pullout tests in bedrock.

The characteristics of the pre-reinforcement system considered in the numerical simulation were based on real works realized in the studied zone. As far as the umbrella pre-support is concerned, 13 autodrilling steel pipes (51/33 mm, 18 m long, spaced 50 cm on the tunnel contour and with $E = 210$ GPa) were taken into account and installed every 50 cm of advancement. Compared to the reality, it was chosen to keep the same inclination of 6° of the umbrella all along the model in order to simplify the meshing. In addition, previous calculations showed that this parameter had few influence on the results in our study.

As for the face bolts, a constant length of 18 m was 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) was kept all along the model in order to simplify the meshing. The real bolting density, installed in situ, was simulated varying proportionally the bolts’ modulus and the friction resistance based on the material characteristics listed in Table 3.

The calculation was carried out in drained conditions. The tunnel excavation was 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 was installed 1.5 m behind the tunnel face, on which the shotcrete application was simulated as well. The tunnel invert was activated 39 m behind the tunnel face progress.

### 4 COMPARISON BETWEEN NUMERICAL SIMULATION AND MEASUREMENTS

#### 4.1 Surface settlements

Figure 3 shows the settlements of different surface points (PM 847 m, 856 m, 860 m...), 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 for finally stabilizing 50 m behind the tunnel face with a settlement of around 20 mm. The 3D calculation reproduces very well this settlement evolution. The numerical approach is also able to simulate with good accuracy the transverse settlement trend, in terms of maximum settlements and profile’s width (Figure 4).

#### 4.2 Inclinometer measurements

Two important phenomena could be analyzed thanks to the measurements of inclinometer movements parallel to the tunnel axis (Axis B). First, the upper 25 m, after

<table>
<thead>
<tr>
<th>Support</th>
<th>Description</th>
<th>$E$ (GPa)</th>
<th>Thickness (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lining</td>
<td>HEB 180 +25 cm shotcrete</td>
<td>13.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Invert</td>
<td>HEB 220 +30 cm shotcrete</td>
<td>14</td>
<td>0.3</td>
</tr>
<tr>
<td>Face</td>
<td>15 cm shotcrete</td>
<td>10</td>
<td>0.15</td>
</tr>
</tbody>
</table>

Table 2. Tunnel support characteristics.

<table>
<thead>
<tr>
<th>Steel bolts</th>
<th>Fibreglass bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus (GPa)</td>
<td>210</td>
</tr>
<tr>
<td>Section ($m^2$)</td>
<td>$0.49 \cdot 10^{-3}$</td>
</tr>
<tr>
<td>Inertia ($m^4$)</td>
<td>$0.033 \cdot 10^{-6}$</td>
</tr>
</tbody>
</table>

Table 3. Face bolt characteristics.

![Figure 3. Longitudinal settlement profile.](image-url)
having moved towards the tunnel when it approached (measurements of 5/03/2009 and 16/03/2009), went back to their initial values and finally moved in the direction of the tunnel face progression. Secondly, a local displacement at the tunnel level (“belly”) increased as the tunnel face approached and remained even after the tunnel face had passed the monitoring section. The 3D simulation is able to represent both of the above phenomena. The results fitted well also with the measurements in the monitoring section plane, perpendicular to the tunnel axis (Janin 2012).

In addition, the displacements around the tunnel, the rib deformation and the stresses in the ribs obtained with the 3D model are in good agreement with the measurements (Janin 2012).

5 CONCLUSIONS

The monitoring section, installed during the construction of Toulon tunnel’s south tube, allowed analyzing the evolution of soil deformation and tunnel support reaction during the excavation progress.

Furthermore, these accurate measures permitted to create an important database used subsequently for a numerical back-analysis. The real excavation process and the pre-reinforcements installed in situ were simulated in a three-dimensional model. 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.

REFERENCES