Finite difference analysis of an advance core pre-reinforcement system for Toulon’s south tube

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1. Introduction

During the excavation of a tunnel in an initially stable massif, the pre-existing stress field is disturbed. Indeed, the stress on the periphery of the excavation becomes nil. This modification is generally accompanied by a movement of the face towards the excavation as well as a convergence of the tunnel walls.

In the case of full-face excavation, tunnel reinforcement methods are developed recently based on the installation of pre-reinforcement system at the tunnel face that can bring the necessary strength to ensure the stability of tunnels (Lunardi, 2008). It is currently difficult to choose the type of the appropriate pre-reinforcement based on simplified analyses, as they give approximate but not very reliable results, whereas the three-dimensional (3D) analysis takes much time and requires complex operations to develop a numerical model and interpret the obtained results.

Several authors used 3D approaches to study the effect of tunnel advance core reinforcement on the overall stability of the tunnel using either fiberglass bolts (Dias et al., 1997; Yoo, 2002; Oreste, 2013; Perazzelli and Anagnostou, 2013; Anagnostou and Perazzelli, 2015; Li et al., 2015), or bolts which serve as an umbrella arch (Kim et al., 2005; Volkmann et al., 2006; Song et al., 2013; Oke et al., 2014). Other authors have been interested in studying the advance core reinforcement in shallow tunnels in laboratory using centrifugal (Al Hallak et al., 1999; Calvello and Taylor, 1999; Kamata and Mashimo, 2003; Juneja et al., 2010; Yokota et al., 2012) or physical models (Egger et al., 1999; Trompille, 2003; Shin et al., 2008; Hirata et al., 2013). They showed that the installed bolts ahead of the tunnel face can reduce the limit pressure of tunnel lining, the extrusion displacement and the surface settlement. These bolts can also limit the horizontal and vertical extents of the rupture zone ahead.

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

http://dx.doi.org/10.1016/j.jrmge.2016.05.006

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of the face. The optimal length of reinforcement is approximately twice as long as the distance between the excavation face and the possible sliding surface. Although many researches have provided useful information on the behaviors of these reinforcement elements, few studies have been conducted to study the stability of coupled pre-reinforcement type, i.e. the face bolting system with the umbrella arch. A detailed study was conducted by Janin et al. (2012), who used the finite element method to study the influence of two systems of tunnel advance core reinforcement (umbrella arch method and face bolting system) on the reaction of solid rock. Tunnel face bolting and distance of the delay tunnel invert had a leading role in the development of massif movements. A two-dimensional (2D) back-analysis was carried out in parallel to obtain useful indications for calculation parameters that can correctly simulate the excavation process. After comparing the different numerical results with the ones obtained locally, it was indicated that only 3D numerical analysis obtains satisfactory results. Aksoy and Onargan (2010) drew the similar conclusion based on the results of a numerical study for a shallow tunnel in grayish-black limestone bedrock and the lagoon deposits of the Triassic including clays, sandstone, limestone and dolomite. Fig. 2 presents the geological profile of the instrumented section located in the garden “Alexander 1” in MP 880 (MP: Metric point). Fig. 3 shows the longitudinal geological cross-section along the tunnel. It can be seen that the sandstone of bedrock occupies the entire excavation section.

Given the poor quality of the ground encountered, we had to use the ADECO.RS excavation method (analysis of the controlled deformation in rock and soil) conceived by Lunardi (2008). This method consists in reinforcing the advance core to prevent premature extrusion of the tunnel face, and limit the plastic deformation zones and their spread behind the face based on the convergence trend. The total south tube was divided into several main sectors in a previous stage of the project according to the geology, the cover and the acceptable settlement for constructions. Support profile and reinforcement were recommended for each sector. The justification for these support profiles was conducted according to 2D calculations. Taking the instrumented section MP 880 for example, a type of umbrella arch reinforcement and a longitudinal fiberglass dowels have been introduced. The excavation was carried out with a shovel or a hydraulic rock breaker in an advance step of 1.5 m. After each pass, the tunnel lining (HEB 180), tunnel invert (HEB 220) and fiber shotcrete were installed. The layout and quantity of bolts, the implementation of the lining, the shotcrete thickness and other design parameters have been modified during the excavation in order to ensure the steady progress and compliance with limitations in terms of surface settlement and to optimize the work economically.

The methods conventionally employed for the monitoring of tunnels during their construction are auscultations. They aim first...
to control the risk of instability or settlements in the short term and ensure the sustainability of the tunnel in the long term (Lunardi, 2008). The instruments implemented in the studied section (Fig. 2) consist of six pairs of vibrating wire extensometers for measuring the forces in the lining, five pressure cells for measuring convergence, and four radial extensometers installed underground for measuring borehole movements. In addition, two inclinometers at both sides of the tunnel and a vertical extensometer at 2 m away from the tunnel axis were installed at ground surface (Janin, 2012). The recorded data were transferred in real time via a wireless network to an acquisition center. They were stored in a database named “Geoscope” consulted remotely by various stakeholders of the project (company, contractor, project owner) through a secure internet access. A numerical analysis can be performed based on the measurements collected.

3. 2D simulation

The real process of tunnel excavation and reinforcement is very complex where the deformations of tunnel face are a 3D phenomenon (Dias et al., 1997). However, the usual practice of tunnel calculation still relies on 2D numerical simulations to estimate both the surface settlements and structural efforts. As a part of this approach, numerical modeling of the instrumented section was conducted using the 2D finite difference code FLAC2D (Itasca Consulting Group, 2009) to reduce the uncertainty in geomechanical parameters and fit different measurement types to validating the calculation method.
To minimize the influence of boundary conditions and take into account the evolution of movements versus depth, the model size is set to be 80 m wide and 70 m deep, with the cover depth of 25 m. The mesh must be relatively small around the tunnel to sufficiently represent the stress concentration caused by geometric singularities (Mestat, 1997). The 2D model represents a cross-section of the model adopted in the 3D calculations.

The soil behavior was represented by an elastic–perfectly plastic model based on the Mohr–Coulomb failure criterion. It is well known that this constitutive model does not take into account the variation in the deformation modulus as a function of stress state. Nevertheless, several authors like Melis et al. (2002), Mroueh and Shahrour (2008), and Migliazza et al. (2009) used simple constitutive models that require few parameters which can be easily determined.

The tunnel support is constituted of beam elements having three degrees of freedom (two in displacement and one in rotation). These elements can be attached to each other and to the ground mesh, and they can work in tension, compression or bending. Table 1 presents the mechanical properties of tunnel support and invert, where the elastic modulus (E) is calculated by the homogenization method based on the characteristics of metallic lining (E_{lining} = 210 GPa, E_{concrete} = 10 GPa) (Jain et al., 2015).

Recognition of the ground geotechnical parameters based on a series of pressure meter measurements and laboratory tests was conducted to measure the resistance and deformability of soil. The results obtained show a considerable uncertainty in the bedrock deformation modulus (varying between 150 MPa and 240 MPa) and even the value of k_o (Mermet et al., 2005). A parametric study is needed to better identify these two parameters and fit numerical calculations to the measurements. For the other layers (fill and colluviums), the parameters used in Jain et al. (2015) are selected.

Tunnel excavation was numerically simulated by the convergence–confinement method based on the work of Karakus (2007), who compared the suitability of many simulations of the sequential excavation method and showed that the convergence–confinement method has enabled the best agreement with experimental results. The advantage of this method is to convert a 3D problem into a 2D problem. It makes us able to study several successive equilibrium correspondents to the decreasing values of pressure (confinement pressure) applied at the nodes on the tunnel boundary. After excavation, the stress progressively decreases simultaneously as the tunnel face moves away, and the ground will be converged by causing a radial displacement (Panet, 1995).

The value of this stress is calculated using the following formula: \( \sigma(t) = (1 - \lambda)\sigma_0 \) (\( \sigma_0 \) is the initial stress in the ground, and \( \lambda \) is the stress relaxation ratio), with \( \lambda \) varying between 0 (\( t = 0 \)) and 1 (\( t \to \infty \)). The ground deformation is determined for each equilibrium state and the ground characteristic curve at different points of the tunnel circumference (crown, sidewall, etc.) is plotted. FLAC can reduce the initial stress by introducing a stress relaxation ratio that takes into account the mechanical influence of the tunnel face proximity. The main steps of this simulation are listed as follows:

1. Constraints initialization from the ground characteristics.
2. Tunnel excavation with a stress relaxation ratio \( \lambda_1 \).
3. Application of tunnel lining with a stress relaxation ratio \( \lambda_2 \).
4. Installation of the tunnel invert and ending of deformation (\( \lambda = 1 \)).

Fig. 4a shows the first calculations of the parametric study concerning the bedrock characteristics (in this case, the coefficient of earth pressure at rest is 1). It can be noticed that, regardless of the values of deformation modulus and stress relaxation ratio \( \lambda \), the displacement progresses to the contact point of tunnel arch support and the invert. However, the displacement obtained by the inclinometer tube moves towards the middle of the tunnel. Another difference in the measured and simulated displacements below the tunnel leads to an unrealistic vertical movement of the invert, which probably depends on the unloading response of the soil. To solve these problems, the bedrock layer must be divided and a back-analysis must be performed by adjusting the value of the deformation modulus and the coefficient of earth pressure at rest \( k_0 \) to obtain results as close as possible to in-situ measurements.

Generally, the purpose of back-analysis is to optimize the excavation and the support. It allows to validate and/or adapt the models trying to find by calculation the ground behavior observed (due to auscultation). For this purpose, the geomechanical features and/or the constitutive models are adjusted to find good results that are quite close to the measurements (AFTES, 2003). Once the fitting of the calculation is obtained, we can then adjust the excavation process and optimize the supports and concrete lining. In this case, iterative modifications of input parameters E and \( k_0 \) of the bedrock are performed until the output values reproduce the best data compared to that observed. This analysis begins with a limitation of possible variation in each desired parameter value with a reasonable uncertainty. As for the rising of the invert, Duncan and Chang (1970) and AFTES (1999a) recommended to use the unloading modulus instead of the loading modulus (vertical stress reduced at the excavation line) to make a more realistic assessment of ground movements.

Fig. 4b shows the response of the numerical model in terms of the horizontal displacement at \( \lambda_1 = 73\% \) and \( \lambda_2 = 87.5\% \) while installing the tunnel lining. It is clearly seen from the figure that the trend of horizontal movements is well simulated at the surface and in the middle of the tunnel. This confirms that the optimized parameters of the bedrock layer help to fit the in-situ measurements in a satisfactory way. Table 2 summarizes the geomechanical parameters obtained after the back-analysis, which are then used in the 3D modeling. These parameters and the stress relaxation ratios are optimized to confront not only the horizontal movements but also the surface and underground movements and support deformations.

To ensure the stability and study the behavior of decompressed zone, Panet and Guenot (1982) suggested to follow different stress and strain paths at the excavation circumference and to understand if the concentration of such stresses at a point can trigger a failure. Fig. 5 shows the evolution of principal stresses at various points until installation of the invert with a total deformation. At the crown (point A), the vertical stress \( \sigma_1 \) decreases and the horizontal stress \( \sigma_3 \) increases significantly until \( \lambda = 0.45 \) (beginning of plasticity). Then the stress \( \sigma_1 \) decreases as \( \sigma_2 \) decreases until the installation of support at \( \lambda = 0.875 \) where they stabilize. At the point B, it is opposite, i.e. a horizontal deformation at tunnel walls with an increase in the vertical stress \( \sigma_2 \) and decrease in the horizontal stress \( \sigma_1 \). Then the vertical stress \( \sigma_2 \) decreases with \( \sigma_3 \). This stress concentration should be taken into consideration to avoid appearance of cracks in the concrete shell. In the invert (point C), principal directions are not inverted during the
simulation and the variation of vertical stress is large. However, the horizontal stress varies little (up to $\lambda = 0.45$), then it follows the similar path to that of the tunnel crown. The direction of the obtained stresses is similar to that of Bernat et al. (1999) who plotted the path of effective stresses around a lined tunnel and studied the effect of concrete lining installation to slow down the stress evolution.

The advantage of the convergence–confinement method is to plot ground characteristic curve and determine the radial displacement at the tunnel walls. The displacements are calculated from the integration of the stress field associated with a failure criterion. Fig. 6 shows the convergence curves of tunnel without support at points A and B. It can be noticed that the behavior is elastic until $\lambda_e = 0.45$ (where plasticity occurs), corresponding to a displacement of 9 mm ($U_{Re} = 9$ mm). Then, the ground comes into an irreversible deformation state until $\lambda = 0.875$, and a rupture is produced due to excess compression. The difference in the displacements of the two points amounts to the gravity term $P_g$, where the ground weight must be taken into account. It is indicated that arch effect is formed in tunnel crown within the limit of rupture zone ahead of the tunnel face.

The previous analysis clearly highlights that the determination of stress relaxation ratio at the installation of the lining is not easy, because field measured deformations depend firstly on ground parameters whose uncertainties are more or less precise, and secondly on the construction method which incorporates a reinforcements system ahead of the tunnel face. Bernat et al. (1999), Hejazi et al. (2008), and Janin (2012) have emphasized that the use of back-analysis is necessary to simulate correctly the measurements.

4. 3D simulation

The 3D simulation is obviously the more representative of the reality compared to 2D simulation. It helps to avoid the assumptions related to the convergence–confinement method. The work
carried out in this section concerns 3D modeling using the finite difference program FLAC3D to study the impact of the real pre-reinforcement installed in the tunnel on the ground movements. To reduce the effect of boundary conditions, the mesh size is defined as 80 m in X-direction (transverse direction), 70 m in Z-direction (vertical direction), and 90 m in Y-direction (longitudinal direction). The mesh with smaller elements was used around the excavation where the stress and strain fields are very high. The cover depth is kept constant (25 m). The total number of mesh is around 54,000, and only a half section is modeled for symmetry reason (see Fig. 7). The bottom boundary of the numerical model is fixed in the three directions (X, Y and Z), while only the horizontal movements are fixed on the lateral surfaces.

Soil behavior is always represented by an elastic-perfectly plastic model using the Mohr–Coulomb failure criterion. The geotechnical characteristics of the soil are fixed based on the 2D back-analysis results, which are represented in Table 2.

The excavation of the tunnel was simulated in 30 steps with an excavation length of 1.5 m, leading to a total length of 45 m. In each step, the tunnel support (lining and shotcrete) was installed at 1.5 m behind the face. The invert was activated with a delay of 6 m above the tunnel face. In the numerical modeling, the support and the tunnel invert are modeled by shell structural elements, and the mechanical properties of the real support are adopted (Table 1). The shell structural elements behave as an isotropic or anisotropic, linearly elastic material without failure limit. The two elements (support and invert) are attached to each other.

Two types of pre-support were used in the instrumented area (umbrella arch method and face bolting system). The half umbrella arch was composed of 13 steel bolts, which are 18 m long and 51/33 mm in diameter, with a spacing of 50 cm between each other, installed in the upper third or a quarter of the circumference (the thickness of the grout around the bolt is about 7.5 cm). The bolts were installed at a dipping angel of 6° related to the Y–Z plane and renewed every 9 m (Fig. 7). The umbrella arch bolts were simulated by pile elements, and they were considered as the straight segments with uniform bisymmetrical cross-section between two nodes. In addition to providing the behavior of a beam structural element, interaction between the pile and mesh elements is also simulated by a normal-directed friction (including the ability to specify a limit plastic moment) (perpendicular to the pile axis) and a shear-directed friction (parallel to the pile axis). The soil/grout and grout/bolt interactions are characterized by a normal and tangential stiffness, respectively.

The main difficulty lies in determining the exact values of the interaction parameters of pile element. Pullout tests were performed in the bedrock during the south tunnel excavation on bolts with a length of 2 m sealed with cement grout. These helped to determine the limit skin resistance force (perpendicularly to the pile axis) and the cohesion in normal and shear directions (perpendicular to the pile axis) and the shear stiffness (parallel to the pile axis).

The peak frictional force of the interface (Eq. (2)) can be determined as:

$$
\tau_{\text{peak}} = \tau_1 q_B
$$

where $\tau_1$ is the peak shear stress of the interface, $q_B$ is the applied load on the bolt, and $B$ is the cross-sectional area of the bolt.

The peak frictional force of the interface (Eq. (3)) can be determined as:

$$
K_s = \frac{2\tau_{\text{peak}}}{10 \ln(1 + 2t/D)}
$$

where $K_s$ is the shear stiffness of the interface, $\tau_{\text{peak}}$ is the peak shear stress of the interface, $D$ is the diameter of the bolt, and $t$ is the thickness of the grout around the bolt.

Identification of the contact stiffness parameter is essential. Very high values result in an extremely slow convergence and too long calculation time. On the contrary, if the selected parameters are too small, the deformation of the rock mass is small and occurs after that of the reinforcement element without achieving the real interactions. Table 3 summarizes different characteristics of the pile element. Oke et al. (2014) conducted a numerical analysis to study the sensitivity of interaction parameters for forepole element of the arch umbrella system used in the Driskos tunnel in Greece. The results of this analysis showed that the most significant parameter that governs the deflection profile of the forepole element is the normal stiffness $K_n$. The cohesion $c_s$ and the shear stiffness $K_s$ are the second and third most influential parameters, respectively.

With regard to the tunnel face reinforcement, 20 fiberglass bolts were horizontally installed at a spacing of 18 m, uniformly distributed on the half-section of the tunnel face. Considering the extreme complexity of simulating the real renewal bolts (the characteristics of the reinforcements in terms of their number, type and length varied significantly in the studied area), based on the information collected in field, it was decided to renew the entire bolting every 9 m. The bolting density calculated at each advancement step is set as 0.18 bolt per square meter. This will
allow to better approach the numerical simulation in which the real renewal bolts are modeled (Janin, 2012). Oreste and Dias (2012) conducted a parametric study to investigate the influence of the length and number of dowels on the safety factor of the face in the case of Toulon tunnel (north tube). They found that the dowels with residual length less than 5 m have a reduced influence on the safety factor; due to the fact that the interface between the lateral surface of the dowel and the rock can easily break in this condition and thus reduce the contribution of the reinforcing system. On the other hand, if the length is greater than 1.5R (R is the tunnel radius), the efficiency of reinforcement is the same as that of infinite length.

Fiberglass bolting is modeled by cable elements. They are also assumed as straight segments of uniform cross-section between two nodes. These elements behave as an elastic-perfectly plastic material that can yield in tension and compression but cannot resist a bending moment. A cable must provide a link in such a way that the shear forces expand along its entire length in response to relative movement between the cable and the mesh. The grout behaves as an elastic-perfectly plastic material, with its peak strength dependent on the confining stress and without loss of strength after failure (Itasca Consulting Group, 2005). The physico-mechanical properties of these structural elements are given in Table 3.

### Table 3

<table>
<thead>
<tr>
<th>Pile element</th>
<th>Elastic modulus, $E$ (GPa)</th>
<th>Elastic modulus of the grout, $E_{grout}$ (MPa)</th>
<th>Sectional area, $A$ (m²)</th>
<th>Shear coupling spring cohesion, $c_s$ (N/m)</th>
<th>Shear coupling spring friction angle, $\phi_s$ (°)</th>
<th>Shear coupling spring stiffness, $k_s$ (N/m²)</th>
<th>Normal coupling spring cohesion, $c_n$ (N/m)</th>
<th>Normal coupling spring friction angle, $\phi_n$ (°)</th>
<th>Normal coupling spring stiffness, $k_n$ (N/m²)</th>
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<td>25</td>
<td>$3.33 \times 10^9$</td>
<td>8 $\times 10^3$</td>
<td>25</td>
<td>$3.33 \times 10^9$</td>
<td>25</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Cable elements</th>
<th>Elastic modulus, $E$ (GPa)</th>
<th>Elastic modulus of the grout, $E_{grout}$ (MPa)</th>
<th>Sectional area, $A$ (m²)</th>
<th>Grout cohesion, $c_g$ (kN/m)</th>
<th>Grout friction angle, $\phi_g$ (°)</th>
<th>Grout stiffness, $k_g$ (N/m²)</th>
<th>Normal tensile strength, $\sigma_t$ (MPa)</th>
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<td>25</td>
<td>$5.89 \times 10^9$</td>
<td>400</td>
<td></td>
</tr>
</tbody>
</table>

5. Comparison between numerical simulations and in-situ measurements

In order to validate the proposed simulations, the measurements of instrumented section (surface and underground movements, convergence, etc.) are compared with the results obtained by 2D and 3D numerical simulations. We must specify the interest of the 3D approach that takes into account the geometry and the real reinforcement of the tunnel, and also highlight the limitations of the convergence—confinement method.

5.1. Surface settlement

The excavation of tunnels at shallow depths provokes movements that respond to the surface where a 3D settlement trough appears. In urban areas, implementation of an auscultation system is necessary in order to predict and assess the real-time influence of these movements on the existing structures. In the literature, several empirical and analytical methods (Peck, 1969; Cording and Hainsmire, 1975; Clough and Schmidt, 1981; Attewell and Woodman, 1982; O’Reilly and New, 1982; Rowe et al., 1983; Sagasta, 1987; Mair et al., 1993; Oteo, 1993; Serratrice and Magnan, 2002) have been proposed to study surface settlement. The measurements obtained by these methods showed that the transversal profile of vertical settlements at each depth $Z$ can be well modeled by a Gaussian curve. Some authors (Clough and Schmidt, 1981; O’Reilly and New, 1982; Selby, 1988; Mair et al., 1993; Sugiyama et al., 1999) have proposed the correlations linking the point of inflection to the geometric parameters of the tunnel based on observations made in real cases.

The settlement trough at the upper surface of the massive is compared with that measured in vertical alignment of the instrumented section (Fig. 8a). Despite the low number of measuring points, it is clear that the 2D calculation with the values of optimal deconfinement ratio ($\alpha_1 = 0.73$, and $\alpha_2 = 0.875$) fits correctly with the 3D settlement trough, and likewise with the real measurements. In our case, the maximum settlement is 20 mm and the trough half width $i = 17.7$ m, which are in relatively satisfactory agreement with that obtained by the empirical formulae of Sagaseta (1987) and Oteo (1993).

The volume of the settlement trough (per unit length of tunnel) $V_t$ is an important index in the expression of surface settlements, and can be calculated as $2.5iS_{\text{max}}$. Its amplitude depends mainly on soil type, excavation method, tunnel support and quality of works execution (Serratrice and Magnan, 2002). The ratio of calculated $V_t$ to the theoretical excavated volume (120 m³) is 0.73%, indicating good control of massif deformation. Similar results were obtained for tunnels excavated in the London clay (New and Bowers, 1994), but this value reached 3% in the marine soft clays in Singapore with earth pressure balance TBM’s (tunnel boring machines) or compressed air shields (Shirlaw and Doran, 1988). The maximum slope of the transverse trough (located at the point of inflection) should remain below 4%, to avoid any disorders occurring to buildings (AFTES, 1999b). According to our results, the calculated slope at the inflection point is 0.7%, well below the threshold value. It is shown that no damage can appear on the buildings.

As for the settlement calculated as a function of the advance of the tunnel face, Fig. 8b compares numerical results in terms of the longitudinal profile of surface settlement to the in-situ measurements at different points along the tunnel axis. It can be seen that the movements accelerate during the passage of the tunnel face. Thereafter, the velocity of settlement decreases till the face exceeds the section for 40 m (about four times the diameter). Attewell and Woodman (1982) and Mair and Taylor (1997) have conducted studies to estimate the surface settlements generated by the full-face excavation, and concluded that the settlement at the face position is close to $S_{\text{max}}$/2. In our case, the ratio of $S_{\text{face}}/S_{\text{stabilized}}$ at this area is approximately 44%. An interesting study was performed by Serratrice and Magnan (2002), who proposed an empirical expression derived from the optimization of exponential equations based on settlement measurements in the south Toulon tunnel to describe the evolution of surface settlement based on the tunnel advance. They showed that it is possible to predict the final settlement if the magnitude of the settlement ahead of the tunnel face is known. This method helps engineers to intervene at the right time to change the pre-support type.
5.2. Displacement

Numerical results of displacement were compared to the deformations measured by a vertical inclinometer located at 13.3 m away from the tunnel axis (Fig. 9). Corrections were made to the results measured by the southern inclinometer compared to the movements measured by the surface target prism near the inclinometer position when the tunnel face was 95 m away. The numerically obtained values are very close to those measured. A minor difference in the shape of the curves is obtained with 3D calculation after 45 m of excavation.

The curves were analyzed to highlight the movement of the first meter of the ground towards the tunnel due to deformations of surface soil mass. Furthermore, a tilting towards the void was created by the tunnel advance, possibly due to extrusion displacement of the face in the direction of the tunnel axis. These phenomena have been already pointed out by Serratrice (1999) during the measurements of inclinometers placed in a section of the north Toulon tube. On the other hand, in the case of shielded TBMs with the face under pressure of bentonite slurry (Ollier, 1997; Benmebarek and Kastner, 2000), a lateral displacement towards the opposite direction was manifested at the tunnel axis.

5.3. Extensometer measurements and tunnel wall convergence

Extensometers located 2 m away from the tunnel axis were installed on two anchors at 6 m and 12 m deep, respectively, to study the magnitude and the extension of the ground movements in depth. They are the only ones that can be installed before the passage of the tunnel face. The measurements were taken at a varied frequency depending on the distance between the face and the reinforced section, with a maximum of two measurements by week when the face is at 30 m away from the instrument. The results show that the simulations of the two anchors descend more regularly and the magnitude of the displacements transmission towards the surface is very low (Fig. 10). The 3D numerical analysis is the only one that can represent the phenomenon. The settlement in the tunnel crown decreases due to the installation of the umbrella arch system where the bolts installed in the ground mass traverse the slip surface. This observation is in agreement with the measurements of Kamata and Mashimo (2003), Shin et al. (2008), Hisatake and Ohno (2008), and Juneja et al. (2010), which showed that this pre-reinforcement type helps to reduce the propagation of the rupture zone towards the surface. In the case of 2D numerical analysis, a difference of 11% is found compared to the results of 3D analysis. The deformations are slowed down only by a shotcrete associated with the metallic lining.

The radial displacement of the downward extensometer #5 is shown in Fig. 11, and the profile of the ground deformation around a hole is obtained by measuring the change in the position of each anchor (located between 2 m and 12.5 m) over time with respect to a fixed point. In this case, the anchor at 12.5 m had to be cut due to the closing of the hole. It can be seen that different numerical
analyses seem to better simulate the in-situ measurements. In addition, the 2D approach remains strongly dependent on the value of the stress relaxation ratio at the installation of tunnel lining, $\lambda_{th}$, so that the results are in good agreement with the measurements. Low values of beyond 5 m depth indicate that the extension of the plastic zone is exceeded, and a slight difference was found ahead of the extensometer because in reality, the hole was excavated after a period of time during installation of the tunnel invert. This simplification induces the increase in the tunnel support deformation.

The final convergence of the support was measured by topographers. Low convergence magnitudes have been recorded when the face was 45 m away (Fig. 12). A satisfactory approximation between the results of 2D numerical analysis and in-situ measurements was found at either the tunnel crown or the sidewalls ($U_c$ in Fig. 12). On the other hand, 3D numerical analysis presents a difference in support deformation in the order of 60% compared to the measurements. This is probably related firstly to the delay in installation of the invert, which is close to the support ring and can prevent the sidewall from sinking or moving towards the excavated area, secondly to the fact that the excavation of the bottom half-section was simultaneously performed on site when the tunnel invert is activated, and thirdly to the first reference measurement taken by the topographer, which was not carried out immediately after the installation of the lining. In this period, the deformation occurred. This thus confirms that the differences between the results are related to a relatively localized phenomenon, due to the simplification introduced into the modeling described previously. Therefore, it is essential to immediately install a less deformable support ahead of the tunnel face for reduction of tunnel wall convergence and initiation of settlement.

6. Influence of different pre-support systems

In this section, we focus on the influence of some pre-reinforcement systems on the extrusion and axial displacement of the tunnel face. Three cases were considered here with respect to the same numerical and geotechnical characteristics of the models: (1) without any reinforcement, (2) reinforcement with the umbrella arch method, and (3) reinforcement of tunnel face by bolting. Then comparisons are performed among the three cases and the reference model where the tunnel is reinforced by both the umbrella pre-support and face bolting.

Fig. 13a illustrates the comparison of the numerical results in terms of horizontal movements of the tunnel face along the tunnel vertical axis. For face bolting case, the total maximum extrusion is reduced by 50% compared to the case without reinforcement. However, the installation of the umbrella arch pre-support seems not to have an effect on the extrusion and the axial displacement of the tunnel face (Fig. 13b). The same result has been highlighted by Kamata and Mashimo (2003), Song et al. (2006) and Eclaircy-Caudron et al. (2006). Similarly, the advance core reinforcement greatly reduces the pre-extrusion movements. The axial displacement obtained in the face bolting case is actually 48% lower than that obtained in the case without reinforcement, as shown in Fig. 13b.

The presence of bolts seems to give cohesion to the ground mass and keeps the tendency to an extrusion movement. It is also noticed that 80% of the movements are obtained in the first 5 m, with a depth less than the radius. These results conform to those of Dias (2011) where extrusometers were installed in a horizontal hole to measure the extrusion of the core during the excavation of the north tube. It can be noted in this study that the advance core reinforcement plays a crucial role in the movements ahead of the tunnel face and allows to constantly keep the tunnel under control for the success of excavation.

Fig. 10. Comparison of settlement obtained by numerical simulations and in-situ measurements with vertical extensometer.

Fig. 11. Comparison of anchor displacement obtained by numerical simulations and in-situ measurements with radial extensometer.

Fig. 12. Comparison of support deformation obtained by numerical simulations and in-situ measurements.
7. Conclusions

The studies of balance and stability of the south tube excavated in a complex geological context have generated an important interest. Installation of an auscultation system has allowed to regularly monitor the deformations of the massif during the advancement of the excavation. Geomechanical parameters and recorded data have allowed forming a large database based on which numerical approaches have been validated.

In this paper, 2D and 3D numerical analyses were conducted to better study the impact of the pre-reinforcement system actually installed in the tunnel on the movements generated by the excavation. The 2D numerical analysis based on the convergence confinement method has helped to identify the values of some parameters that offer better agreement with the in-situ measurements. However, it is noticed that the method strongly depends on the choice of the value of stress relaxation ratio \( \lambda \) which may be influenced by several factors that have already been highlighted by many authors, such as the excavation process, the pre-support installed ahead of the tunnel face, and the position of the invert. It appears that the 2D numerical analysis is not able to represent the complex effect of the reinforcement on the displacements field of the ground. This study clearly shows that to validate a numerical model by back-analysis and to optimize the excavation and the support to the real conditions encountered, it is necessary to compare the results of different types of in-situ measurements.

To avoid the gaps related to the previous methods, a 3D numerical analysis was then performed. The satisfactory agreement with the different measurements recorded in field shows that the 3D numerical analysis with a full discretization of the inclusions seems unquestionably one of the most reliable approaches to simulate this phenomenon. The analysis of results has clearly shown the role of the reinforcement of the advance core using fiberglass bolts in the reduction of deformations in the ground mass, including the extrusion and surface settlements. The umbrella arch method is limited in reducing the settlements of the tunnel crown. On the other hand, it seems not to have an effect on the extrusion and axial displacement of the tunnel face, so its role is decreasing the risk of instability in the arch. This shows the performance of 3D simulation to refine the calculations and better understand the tunnel behavior.

Finally, it should be noted that a numerical simulation is widely related to the deep understanding of construction operations, which are based on preliminary geological and geotechnical surveys of the ground and implemented techniques. The monitoring instrumentation always remains a useful source of information for engineers to anticipate any difficulties and react as quickly as possible to ensure good implementation of work.

Conflict of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

References


Fethi Kitchach is currently a Ph.D. student at Biskra University. His research focuses on the behavior of shallow tunnels excavated in urban areas. Using a finite difference program, several approaches were examined and compared by him to study particularly the effect of adjacent deep core reinforcement of the tunnel by a pre-support system on massif deformations.