BACK ANALYSIS OF HIGH TUNNEL CONVERGENCES IN CLAYEY MARLS

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Figure 1. Tunnel location and typical tunnel cross section
INTRODUCTION

Ganntas Tunnel is part of the modernization project of the railway between Alger and Oran, in Algeria. In order to double and rectify the existing line between El Affroun and Khêmis Miliana, the alignment foresees the excavation of a 7km-long twin tunnel. The excavation works started in June 2011 with the contractor CCECC, under the supervision of SYSTRA.

Excavation is driven in conventional method by hydraulic hammer simultaneously on 8 different faces since excavation was started also from a junction window towards the middle of the tunnel. The minimum longitudinal distance to be respected between two contiguous tunnel faces has been set to 30m. The tunnel cross-section is a 70m² oval shaped profile, temporary support consists of shotcrete, bolts and steel ribs. A 30 to 50cm thick cast in situ concrete final lining is provided as well.

When the tunnel reached a fault zone in soft clayey marls, extreme squeezing occurred, works were stopped, and re-profiling operations were carried out along more than 100m tunnel length. To date, works proceed at slow rate since high convergences are still monitored and completion of works is not expected before December 2016.
GEOLOGY

The project is located within the Tellian Atlas mountain area, including in particular the Djebel Ganntas massif in between Mitidja and Chleff plains. Djebel Ganntas is characterized by an extensive massif aligned in East-West direction with smooth slopes made of soft clays that emerge in multiple cliffs and ravines. On the top of it, along the edge, a more recent sandstone formation is found.

The existing data on the geology did not allow to establish an accurate geological model at tender stage. In-situ investigations allowed at design stage to identify a Quaternary alluvial cover of clays and gravel, followed by a Miocene stratum divided in a sandy top layer and a lower clayey layer.

From a structural point of view, geological maps suggest a north direction dip of layers, but the in situ surveys showed that layers were highly folded. The lack of information at tender stage on the structure of the geological series should have triggered an extensive investigation campaign at detailed design stage which did not take place causing serious consequences on costs and time at construction stage.

The geological longitudinal profile was updated four times between 1991 and 2011, the latest being the one shown in Figure 2. Except for a short section near the South portal, the whole tunnel is driven through the Miocene clayey marls. On the South side, a complicated structure of layers was identified, in particular a major fault zone was found at chainage pk 180+600 where clayey marls were extremely fractured. Nevertheless the geological risk in this zone was underestimated and no further investigations were established to evaluate the mechanical parameters. It is in this zone that the excavation encountered the major problems and where high deformations were registered on tunnel supporting lining.

The geological model has been established on the basis of the information collected through eight boreholes along the alignment and a geophysical surface survey with the AMT method (Audio Mangetotelluric). Laboratory tests have been performed on extracted samples; the main characteristics of the clayey marls are summarized hereafter:

- Passing at 80μm is about 70% (determined on superficial detrital)
- Plasticity index is around 20 (determined on superficial detrital)
- CaCO₃ content varies between 11% and 32% with a mean value of 18.8%
- Undrained cohesion varies between 30kPa and 63kPa (4 direct shear tests)
- Friction angle varies between 15° and 20° (4 direct shear tests)
- Resistance to uniaxial compression varies between 0.9MPa and 7.6MPa (3 tests)

It should be stresses that the totality of the tests and investigations carried out is far from being representative for a 7km tunnel and this has led to an overestimation of resistance parameters of soils. The geological mapping at tunnel face assessed RMR values as low as 14 when high convergences occurred, such value appears completely inconsistent with the set of geotechnical parameters used to design the tunnel supports.

HYDROLOGY

Clayey marls showed an impermeable overall behavior, water circulation is possible in fractures, bedding planes plans and at contact with the sandstone layer. Water inflows are mainly supplied by rainfalls. Experience derived from the excavated sections confirmed that inflows are scarce and dry conditions may be assumes at the tunnel outward.
High Convergences Occurrence

On December 2013, around chainage pk 108+600, with about 100m overburden, excavation was proceeding with A1 excavation profile (see Figure 4), when high convergences of about 20cm started to be measured at about 15m behind the tunnel face. Excavation was driven by top heading with an advance rate of about 1m/day. The temporary support consisted of a 30cm shotcrete layer reinforced with HEB 180 spaced 0,6m, and 12 radial bolts of 25mm diameter and 4m length installed at each rib foot before benching. High convergences caused works to stop in order to proceed to re-profiling operations. The contractor proposed the E1 excavation profile (see Figure 4), the main difference with the A1 profile consists in closing the rib foot with a provisional invert before benching and providing a face reinforcement by means of 34 fiberglass bolts of 25mm diameter and 12m length. A forepoling was added consisting in a line of 42mm diameter tubes at crown. After re-profiling, excavation restarted using the new support profile. The adopted solution showed poor results, convergences continued to raise and a cumulative value of
70cm convergence was reached when works needed to be stopped again to proceed to new re-profiling operations and third excavation profile (E1R, see Figure 4) was proposed. Since convergences were affecting mostly the rib foot and the heave of the provisional invert was remarked (see Figure 6), the Contractor decided to reinforce these elements by installing 2+2 micro piles beneath each rib foot and increasing the bolts length to 6m in this zone. During re-profiling operations convergences did not increased, however, when the tunnel face advancement restarted, 7cm of additional convergence were registered again.

Figure 4. Excavation profile used to cope with the fault zone
Table 1. Excavation profile interventions around chainage pk 108+600

<table>
<thead>
<tr>
<th>INTERVENTIONS</th>
<th>A1 PROFILE</th>
<th>E1 PROFILE</th>
<th>E1R PROFILE</th>
</tr>
</thead>
<tbody>
<tr>
<td>RADIAL BOLTS</td>
<td>6+6 Φ 25mm L=4m</td>
<td>6+6 Φ 25mm L=4m</td>
<td>8/9 Φ 25mm / L=4m</td>
</tr>
<tr>
<td>FORE-POLING</td>
<td>-</td>
<td>12 tubes Φ 42mm</td>
<td>2+2 micro pile / L=6m</td>
</tr>
<tr>
<td>STEEL RIBS</td>
<td>HEB 180 @0,6m</td>
<td>HEB 180 @1,0m</td>
<td>HEB 180 @0,6m</td>
</tr>
<tr>
<td>PROVISIONAL INVERT</td>
<td>-</td>
<td>HEB 180 @1,0m</td>
<td>HEB 180 @0,6m</td>
</tr>
<tr>
<td>SHOTCRETE</td>
<td>5+25cm</td>
<td>5+25cm</td>
<td>5+25cm</td>
</tr>
<tr>
<td>SHOTCRETE AT FACE</td>
<td>-</td>
<td>4cm</td>
<td>7cm</td>
</tr>
<tr>
<td>FACE BOLTS</td>
<td>-</td>
<td>34 Φ 25mm</td>
<td>34 Φ 25mm</td>
</tr>
</tbody>
</table>

Figure 5. Convergence registration at chainage pk 180+600

Figure 6. Rib foot displacement and provisional invert up-heave
ANALYSIS OF THE ROCK MASS BEHAVIOUR

In parallel to the Contractor’s analyses, the Engineer developed his own back analyses of the observed phenomena in order to better understand the problem and the rock mass behavior to cross-check the proposed excavation. The study is divided into two parts: in the former the rock mass is analyzed without any support in order to identify the right set of rock mass parameters which better describe the massif behavior; in the latter the calculated parameters are used to analyze the proposed excavation profile in order to check the adequateness and the coherence with monitoring data.

Back-analysis of rock mass parameters

The back-analysis is carried out by using the convergence-confinement method as proposed by Lombardi-Amberg (1974), identifying the set of parameters which allows to reproduce 70cm of observed convergence on the tunnel characteristic curve at tunnel face when the half-core resistance is reached. Available geological data collected during construction, such as the RMR index were used in order to best fit the solution. Results are shown in Table 2.

Parameters were initially defined according to the Hoek & Brown criteria, in order to use the RMR information, and then they are converted to Mohr-Coulomb strength parameters more suitable to perform F.E.M. analysis.

It is to note that the difference between the original parameters and the calculated ones is quite relevant. The estimated stiffness is reduced to 50% and the resistance almost to 35%.

Table 2. Rock mass parameters according to Mohr-Coulomb criteria

<table>
<thead>
<tr>
<th>Original Parameters</th>
<th>Calculated parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young Module [MPa]</td>
<td>200</td>
</tr>
<tr>
<td>Cohesion [kPa]</td>
<td>200</td>
</tr>
<tr>
<td>Friction Angle [°]</td>
<td>20</td>
</tr>
</tbody>
</table>

Tunnel face stability

High convergence phenomena are the presence of an extended plasticized zone ahead the tunnel face that only the back-calculated parameter could reproduce.

The application of the stability number (see equation (1), Panet, 1982) shows that the stability of the tunnel face is very critical and that an important support at tunnel face is needed (see Table 4).

A tunnel to be driven in difficult cohesive soils that show high convergences needs an important stiffening of the advance core ahead the tunnel face in order to limit the plasticization in this zone and to allow the installation of the temporary support before rock failure is produced around the cavity (Lunardi, 1999). In order to have a stability number lower than 6.7 (to assure face stability according to Panet, 1982), the equation (2) must be satisfied, which means that a mitigation measure capable to increment the material cohesion ahead the tunnel face of about 165kPa is needed.

Using the transformation on the Mohr-Coulomb plan (see equations (3) and (4)) and considering a 200kPa soil-grout shear resistance at bolt interface and standard fiber bolt dimensions,
the minimum density of needed elements is 1 bolt/m². Such density is the double of the one foreseen in E1 and E1R excavation profiles, which both failed in controlling convergences.

\[ N = \frac{2 \cdot P_0}{\sigma_{cm}} \]  
(1)

\[ \frac{2 \cdot P_0}{\sigma_{cm}} < 6.7 \Rightarrow c > \frac{P_0 (1 - \sin \varphi)}{6.7 \cos \varphi} \Rightarrow c > 240kPa \]  
(2)

\[ \Delta c = \frac{1 + \sin \phi}{2 \cos \phi} \cdot \Delta \sigma \]  
(3)

\[ \Delta \sigma = \min \left[ T_F = \pi \cdot D \cdot L \cdot \tau_s, T_T = \sigma_b \cdot A_b \right] \]  
(4)

Table 4. Stability numbers for the two different parameter sets

<table>
<thead>
<tr>
<th>Original parameters</th>
<th>Calculated parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>7,5</td>
<td>21,2</td>
</tr>
</tbody>
</table>

Where:
- \( P_0 \) = lithostatic stress
- \( \sigma_{cm} \) = uniaxial compression resistance
- \( S \) = excavation surface
- \( N_b \) = number of bolts at tunnel face
- \( A_b \) = bolt resistant section of bolt
- \( \sigma_b \) = bolt yielding stress
- \( D \) = drilling hole diameter
- \( L \) = bolt length
- \( \tau_s \) = soil-grout shear resistance at interface

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**Cross-check of the proposed supporting sections**

Calculated rock mass parameters are then used to check the proposed supporting system and to verify the consistency of the results found in the previous analysis. The excavation profiles E1 and E1R are studied with the help of F.E.M. using the calculation program PHASE² (Rocscience). The rock mass is taken into account as an equivalent continuum, soil behavior is modeled as an elastic perfectly plastic material, and the resistance law follows the Mohr-Coulomb criteria. Staged construction is taken into account (see Table 5) in different calculation phases in order to consider the non-linear behavior of materials, deconfinement ratios are estimated on the results of an axisymmetric problem, as intersection of the convergence curve along the tunnel and the characteristic curve as evaluated previously. Forepoling has been taken into account in the model by stiffening a portion of soil at crown.

The principal outcomes of the analysis are the following:
- Results in term of displacement are consistent with monitoring data (see Figure 8),
- The extension of the plastic zone points out that radial bolts are not an effective intervention to limit convergences since they are placed within the plasticized zone,
- The stress analysis of first lining shows clearly that the foreseen intervention could not afford the pressures induced by the failing rock mass.

Table 5. Calculation phases | * distance is calculated from bench, **P1 is the residual stress in bench

<table>
<thead>
<tr>
<th>Phase number</th>
<th>Description</th>
<th>Distance from tunnel face</th>
<th>Deconfinement ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Geostatic conditions</td>
<td>-</td>
<td>0% ( P_0 )</td>
</tr>
<tr>
<td>2</td>
<td>Half top section excavation</td>
<td>0 m</td>
<td>80% ( P_0 )</td>
</tr>
<tr>
<td>3</td>
<td>Steel ribs and shotcrete installation</td>
<td>1 m</td>
<td>90% ( P_0 )</td>
</tr>
<tr>
<td>4</td>
<td>1 day shotcrete curing</td>
<td>2 m</td>
<td>93% ( P_0 )</td>
</tr>
<tr>
<td>5</td>
<td>3 day shotcrete curing</td>
<td>3 m</td>
<td>95% ( P_0 )</td>
</tr>
<tr>
<td>6</td>
<td>9 day shotcrete curing</td>
<td>4 m</td>
<td>98% ( P_0 )</td>
</tr>
<tr>
<td>7</td>
<td>28 day shotcrete curing</td>
<td>5 m</td>
<td>100% ( P_0 )</td>
</tr>
<tr>
<td>8</td>
<td>Invert excavation</td>
<td>0 m*</td>
<td>80% ( P_1 )**</td>
</tr>
<tr>
<td>9</td>
<td>Steel ribs and shotcrete installation on invert</td>
<td>1 m*</td>
<td>95% ( P_1 )**</td>
</tr>
<tr>
<td>10</td>
<td>1 day shotcrete curing of invert lining</td>
<td>2 m*</td>
<td>96% ( P_1 )**</td>
</tr>
<tr>
<td>11</td>
<td>28 day shotcrete curing of invert lining</td>
<td>=</td>
<td>100% ( P_1 )**</td>
</tr>
</tbody>
</table>
Figure 8. Calculated displacements (in meters) for E1 (on the left) and plastic zone extension around E1R (on the right)

Figure 9. First lining verification of E1 excavation profile
CONCLUSIONS

During the excavation of the Ganntas Tunnel, in the Algerian Atlas mountain area, high convergences occurred slightly after temporary support installation, and re-profiling operation were carried out along more than 100m of tunnel. The failure occurred when the tunnel face reached a faulted zone in clayey marls with an overburden of about 100m. The geological investigations proved to be insufficient at design stage since the presence of such material along the tunnel alignment was unknown until the construction stage. This occurrence caused delays and costs increase since the appropriate solution was not found in short term. The Engineer performed a series of independent analyses to back-calculate the design parameters of the rock mass and to calibrate a model able to reproduce the monitoring data in terms of displacement of the supporting lining.

The accordance between the analysis results and the in-situ observations proves that the original set of parameter was not able to properly describe the soil-structure interaction problem and therefore to design an effective supporting system.

Experience in tunneling showed that when face stability is not assured in plastic soils, the support system is not able to control the development of the plastic zone around the excavation and pre-confining measures, such as extended face nailing or grouting, are necessary in order to ensure a safe excavation and a cost effective work progress.

The presented case history reminds finally of the importance of the following main aspects:

- Performing a sufficient quantity of ground investigations before tender stage may highly reduce costs and time of the entire project.
- Construction methods and engineering analyses should be integrated in a homogeneous design process in order to find the most suitable solution.
- When the lack of geological and geotechnical information do not allow to define safe and reliable rock mass characteristics, it is important to analyze the rock-mass behavior during construction. In case of deviation from the foreseen behavior, such information will allow then to perform back-analysis to adapt the construction to the encountered conditions.

REFERENCES

Lunardi, P. 1999. The Tartaguille tunnel, or use of the ADECO-RS approach in the construction of an “impossible” tunnel. Gallerie e grandi opera sotterranee, No 58.