Lessons learned during the excavation of the Saint Martin La Porte access gallery along the Lyon-Turin Base tunnel

G. Barla, M. Barla, M. Bonini, and D. Debernardi, Department of Structural and Geotechnical Engineering, Politecnico di Torino (Italy)

1. Introduction

During the excavation of the Saint-Martin La Porte inclined gallery (section 80 m^2 - length 2000 m), along the Lyon-Turin Base tunnel a rock mass of very poor quality was encountered pertaining to the Carboniferous Formation. Different ground support systems were implemented and the convergence measurements carried out confirmed the ground to exhibit a very severe squeezing behaviour. Convergences of approximately two meters were observed as soon as the overburden reached three hundred meters. As a result significant changes in the excavation and ground support system were required, Figure 1 (Barla and Panet, 2006).

The inclined gallery is currently under construction and is faced with exceptional difficulties over a significant tunnel length. The present paper is to discuss the observations and monitoring data collected during excavation. In addition to systematic convergence measurements, the information obtained in instrumented sections will be discussed. The feedback from experience gained will be directly used for the excavation of the main tunnel, which is expected to be driven in similar conditions. One of the main challenges is the optimal time for installing the final lining, and also in the assessment of the long term state of stress for design. Consequently, particular attention need be paid at identifying the long term behaviour and the mechanical properties of the squeezing rock mass.

Figure 1: View of the Saint Martin La Porte gallery with the new support system installed.

2. Geological and geomechanical conditions

As shown in Figures 2 and 3 the Saint Martin La Porte gallery is being excavated in the Carboniferous Formation – “Zone Houillère Briançonnaise-Unité des Encombres“(hSG in Figure 2), composed of black schists (45 to 55%), sandstones (40 to 50%), coal (5%), clay-like shales and cataclastic rocks. A characteristic feature of the ground as observed at the face during excavation (Figure 3) is the highly heterogeneous, disrupted and fractured condition of the rock mass which exhibits very severe squeezing problems (Hoek, 2001; Barla, 2001). The formation is often affected by faulting which results in a degradation of the rock mass conditions. The overburden along the gallery in the zone of interest ranges between 300 m to 550 m. Excavation takes place in essentially dry conditions.

In order to assess the rock mass quality during the excavation a detailed mapping of the geological conditions at the face is undertaken by the site geologist as depicted in the typical plot shown in Figure 3. On this basis, one is put in position to evaluate the percent distribution of “strong”- sandstones and schists – and “weak” – coal and clay-like shales – rocks at the face. Also, a cautious assessment of the geological strength index (GSI), based on examination of the rock exposed in tunnel faces can be carried out. Figure 4 illustrates the results of this assessment. It is noted that the continuous

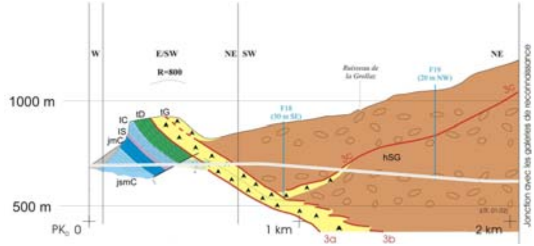

Figure 2: Geological profile.

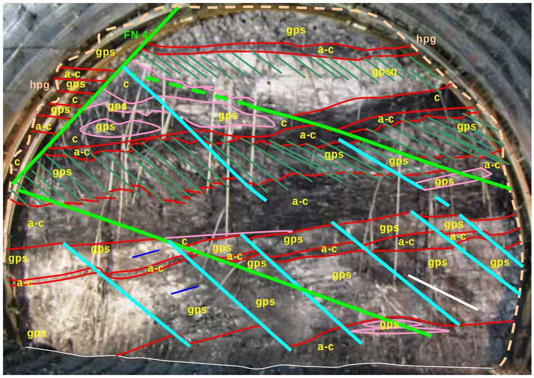

Figure 3: Typical geological conditions at the face (gps-sandstones, a-clay like shales, c-coal, etc.).

variation in GSI values along the gallery is a clear indication of the rock mass heterogeneity. In general GSI is shown to range between 48 maximum to 35 minimum, with the greater value being associated with the greater content in “strong” rocks.

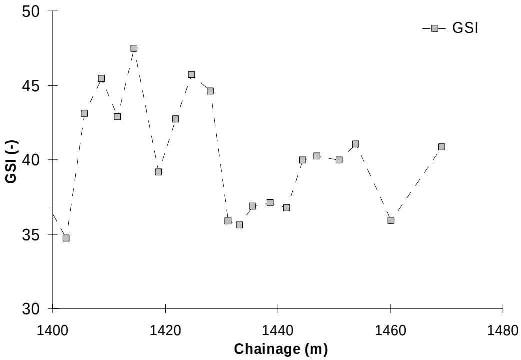

Figure 4: GSI distribution along the gallery.

3. Rock characterisation

With reference to the geological conditions described above, laboratory tests were conducted in order to define the intact rock properties, based on rock samples retrieved from boreholes drilled at the design stage of the Lyon-Torino Base tunnel. Table 1 summarises the data derived from unconfined and triaxial compression tests performed on the “strong” rocks - sandstones and schists - of the “Unité des Ecombres” (LTF, 2006).

Recently, a number of samples were taken from the tunnel face and from new boreholes being drilled in a recent investigation campaign. These samples are being tested in the DIPLAB Geomechanics laboratory of the Politecnico di Torino. With the rock mass being characterised by different rock types, as described above, most of the attention in testing at this stage is based on the “weak” rock components - coal, clay-like shales and cataclastic rock in fault zones. It is expected that the mechanical response of the ground to excavation is likely to be governed by the percentage of “strong” rocks (sandstone, schists) over “weak” ones (coal, claystone and cataclastic rock). This is more evident when time dependent behaviour is taken into account.

Table 1. Parameters derived from laboratory tests on sandstones and schists.

		Sandstones	Schists
Unit weight (γ)	kN/m^3	27.1	26.1
Unconfined compression strength (σ_{ci})	MPa	20.3	16.1
Hoek & Brown constant (m_i)	-	6.4	4.3
Tangent elastic modulus (E_t)	GPa	21.8	19.5

The new tests undertaken at present are focused mainly on investigating the mechanical properties of coal and its time dependent behaviour in triaxial conditions under controlled stress path testing. Also, a number of unconfined compression tests were performed on sandstone and schist in order to validate the database presently available (Table 1). Figure 5 shows typical stress-strain curves for the unconfined compression tests performed on sandstones and schists specimens. It can be clearly seen that the unconfined compressive strength is higher than that previously determined.

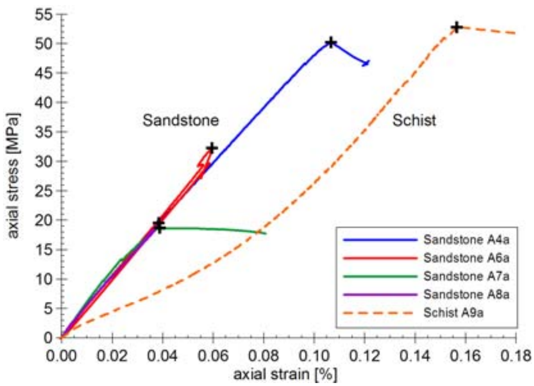

Figure 5: Unconfined compression tests on sandstone and schist specimens.

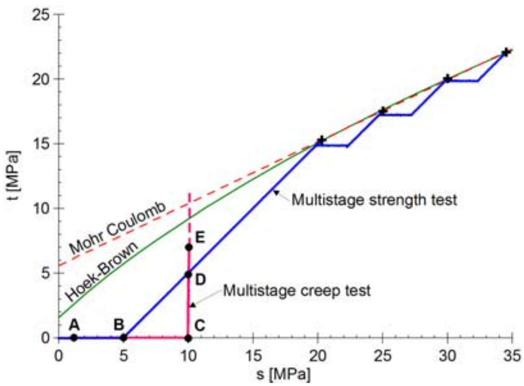

Figure 6: Stress path of the multi-stage triaxial test and strength envelope of coal – Stress path of the multi-stage triaxial creep tests.

With reference to the coal samples, it is of interest to describe the results of a typical multi-stage triaxial test performed with the intent to determine its stiffness and strength properties. Four compression stages were carried out with different confining pressures of 5, 7.5, 10 and 12.5 MPa, by applying a constant axial strain rate of 0.01 %/min, until a peak was reached in each stage on the stress-strain curve. At peak, the confining pressure was increased in order to allow for the determination of a new peak strength. This process was repeated four times allowing one to determine four points on the failure envelope as shown in Figure 6. Stiffness and strength parameters determined are given in Table 2.

In order to study the time dependent behaviour of coal, multi-stage triaxial creep tests are being performed according to the stress path shown in Figure 6. Creep phases were performed in isotropic conditions (points A, B and C) and under a deviator stress state (the mobilisation factor being 0.5 for point D, and 0.7 for point E). The creep curves for the stress state D and E are shown in Figure 7. It is observed that

Table 2: Stiffness and strength parameters for coal

Tangent elastic modulus			
E=6900 MPa			
Mohr-Coulomb criterion			
ϕ=28.5°	c=6.4 MPa		
Hoek-Brown criterion			
σ_{ci}=15.3 MPa	mi=8.97	s=1	a=0.5

a secondary creep deformation is soon attained with a moderate strain rate which is strictly related to the level of the mobilised strength and of the mean stress.

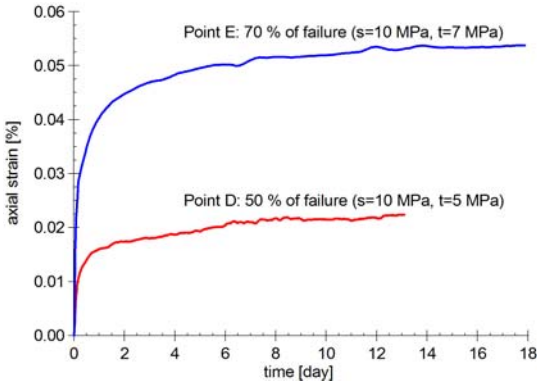

Figure 7: Creep curves for the deviator stress state corresponding to point D and E.

4. New excavation/construction method

Several support systems were used in the Carboniferous zone. However, it became soon apparent that a stiff support would not be feasible in the severe squeezing conditions encountered. The design concept chosen was to allow the support to yield while using full face excavation with systematic face reinforcement by fibre-glass dowels. The support system initially implemented consisted of yielding steel ribs with sliding joints (TH, Toussaint-Heintzmann type), anchors and a thin shotcrete layer in a horseshoe profile. These sections of the tunnel underwent very large deformations with convergences up to 2 m and later needed be re-profiled.

In order to improve the working conditions and to control deformations a new support system was implemented with a near circular cross section (Barla and Panet, 2006). This can be summarised as follows (Figure 8):

- Phase 0 - face pre-reinforcement, including a ring of grouted fibre-glass dowels around the opening perimeter, designed to reinforce the rock mass over a 2 to 3 m thickness
- Phase 1 - mechanical excavation carried out in steps of one meter length, with installation of a support system consisting of anchors (length 8 m) along the perimeter, , and yielding steel ribs with sliding joints (TH type)
- Phase 2 - application of 20-30 cm shotcrete lining, yielding steel ribs with sliding joints (TH type) with 9 longitudinal cuts (one in the invert) fitted with HDC (High Deformable Concrete) elements (Figure 9)
- Phase 3 - installation of a coffered concrete ring at a distance of 80 m from face.

In phase 1 the tunnel is opened in the upper cross section to allow for a maximum radial convergence of 300 mm. In phase 2, which takes place at a distance of 15 m from the face, the tunnel is opened to the full circular section, with concurrent installation of a mixed support system consisting of shotcrete lining, TH ribs, and HDC elements. The yielding strength of the elements is 8.5 MPa, allowing for a maximum 50 percent tangential strain.

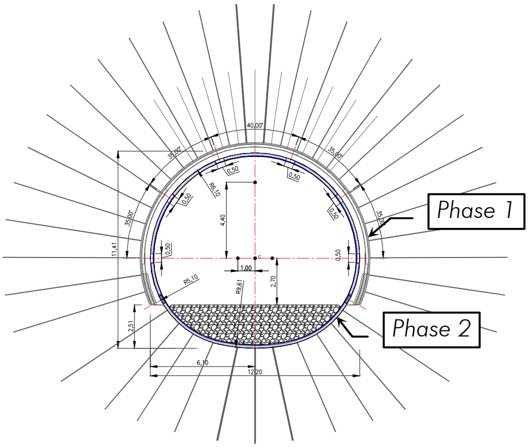

Figure 8: Cross section of the excavation/support system adopted.

Figure 9: Detail of an HDC element installed between the TH ribs.

5. Monitoring data and tunnel behaviour

Systematic monitoring has been performed along the tunnel length. In the following consideration is given to sections between chainage PM1400 and PM1480, with the new support system installed. Figure 10 shows the results of convergence deformations versus time along line1-3 (Figure 11). It is important to note that the monitoring sections installed during phase1 excavation are not in the same position as in phase 2. It is assumed that sections positioned within a distance of 0 to 4 m can be continued with no convergence loss.
The following comments can be made:

- The maximum allowed radial convergence set for phase 1 (300 mm) was overtaken only in the first part of the excavated length under consideration; therefore following chainage PM1438 re-profiling was not needed.
- With the face advance stoppage which occurred between 21/12/06 and 08/01/07 (Holidays Season) the rate of convergence gradually decreased for both phase 1 and 2. With the advance being resumed on 8/01/2007, an increase in convergence occurred in the sections near the face.
- The convergences monitored in phase 2 show that the tunnel cross section has reached a maximum tangential strain of about 68% of its limit value set by the HDC elements , with an average value on the total length under observation up to 37%.

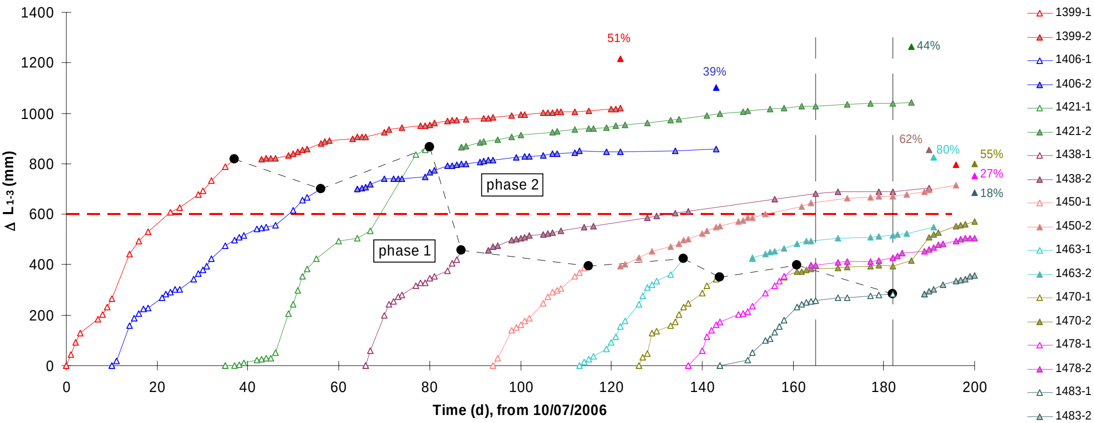

Figure 10: Profile DSM XX - convergences vs time with indication of the amount of deformation capacity.

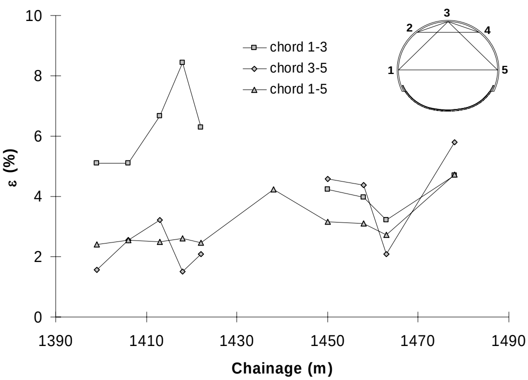

Figure 11: Percent strain (tunnel radial displacement versus tunnel radius) along the tunnel length.

In order to gain in the understanding of the tunnel behaviour during face advance it is of interest to plot in Figure 11 the percent strain (tunnel radial displacement to tunnel radius) for the different sections in phase 1 from PM1400 to PM1480, for lines 1-3, 3-5 and 1-5. It is noted that from PM 1400 to PM1450 the maximum strain occurs along line 1-3, i.e. nearly orthogonal to the most important geological features and discontinuities. From PM1450 onward, a symmetric response takes place as a result of a more isotropic rock mass behaviour.

Figure 12 shows the displacements measured by a multi-position borehole extensometer 24 m long, located on the right sidewall of the access gallery (40° from the horizontal) at chainage PM1443. It is noted that this extensometer is taken as representative of the ground response around the tunnel both in phase 1 and 2, although part of a monitoring station which comprises a total of 6 such extensometers. During phase 1 data were recorded in 7 positions on a length of 24 m. The instrument was replaced in phase 2 with a 15 m extensometer. Notwithstanding the difficulty due to the instrument replacement (reference to the convergences has been made to correlate the data) the following remarks can be inferred:

- Radial displacements occur in the rock surround at least up to a distance of 16 m from the contour. The percent strain (displacement between two subsequent points to distance) ranges from 4% near the opening contour up to 1% at 12 m within the rock mass.
- During phase 1 (25 days) the rate of displacement is almost constant in time and equally distributed versus distance. In phase 2 (39 to 110 days) the rate of displacement decreases significantly to signify that a gradual stabilisation of deformations takes place, in line with the results of convergences shown in Figure 10.

The data of 8 strain meters installed across the HDC elements are available at PM1421 and PM1443. At chainage PM1443 the maximum displacement occurs on the right sidewall (elements n.6, 7) while the crown (elements n.4, 5) exhibits the minimum displacement. PM1421 shows a different trend of behaviour without

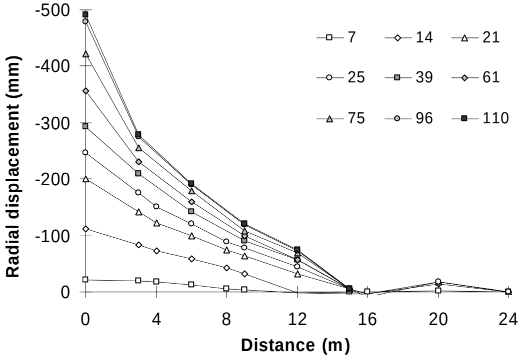

Figure 12: Multi-position borehole extensometer at PM1443. The numbers refer to the days after installation.

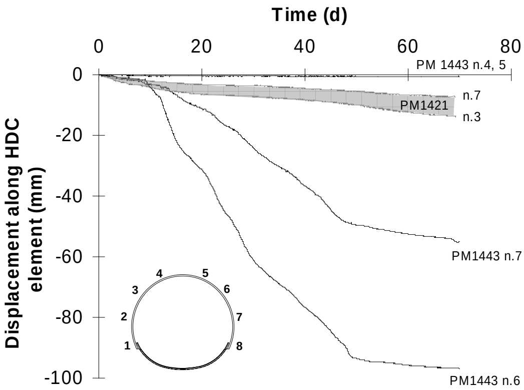

Figure 13: Comparison of the measurements of strain meters installed across HDC elements.

marked anisotropy, mainly due to the fact that most of the displacements takes place during phase 1 (Figure 13).

6. Concluding remarks

The geological and geomechanical conditions encountered during excavation of the Saint Martin La Porte access gallery along the Lyon-Turin Base tunnel, in the Carboniferous Formation, have been presented. The rock characterisation studies in progress at Politecnico di Torino have been mentioned.

The new excavation/construction method adopted in a ground which exhibits very severe squeezing behaviour has been illustrated. Attention has been paid to the sequential installation of a composite prereinforcement-reinforcement system with deformable elements, HDC type, embedded in the shotcrete lining. The most significant aspects of this new method as observed by monitoring during excavation have been described.

For the successful application of the novel excavation/construction method it is essential to adopt an interactive geotechnical design with predictive modelling and to perform monitoring during excavation in the framework of the observational approach.

Acknowledgements

The studies briefly described in this paper have been carried out in the framework of a research contract between Scetauroute, Antea, Alpina and Politecnico di Torino, coordinated by Prof. G. Barla. The authors would like to extend their gratitude to Scetauroute, Antea, Alpina for the permission to publish the results reported. Also to be thanked is LTF, Lyon-Turin Ferroviaire, Chambery.

References

[1] Barla, G. and Panet, M.:
Avis sur les conditions de soutènement de la descenderie de Saint Martin La Porte dans les Schiste Houiller ; Rapport interne LTF, 2006.

[2] Barla, G.:
Tunnelling under squeezing rock conditions; Tunnelling Mechanics. Eurosummer-school, Innsbruck, 2001, D. Kolymbas (Editor), pp.169-268.

[3] Hoek, E.:
Big tunnels in bad rock; Journal of Gotechnical and Geoenvironmental Engineering, vol.127, 9, pp. 726-740.

[4] LTF:
Memoire de Synthese Geologique, Hydrogeologique et Geotechnique. Rapport interne, Rév. 2 – 16/08/06.