Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina, Romancov & Zlámal (eds)

Large deformations in squeezing ground in the Saint-Martin La Porte gallery along the Lyon-Turin Base Tunnel

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ABSTRACT: Within the context of the Lyon-Turin railway project, the excavation of the *Saint-Martin La Porte* inclined gallery (section 80 m^2 – length 2000 m) has encountered a highly fractured Carboniferous rock mass. Different ground support systems were implemented and convergence measurements confirmed the squeezing behaviour of the rock mass. Convergences of approximately two meters were observed as soon as the cover reached three hundred meters. As a result significant changes in the excavation and ground support systems were required. After setting up a ground support system adapted to the squeezing ground, the rate of convergence reached 40 mm per day in the zone of the face and 2 to 4 mm per day at a distance of approximately 60 metres from the face. Due to the difficulties encountered and with consideration given to the exploratory nature of the work, new support systems and organisation works were implemented, combined with an extensive monitoring programme.

1 INTRODUCTION

The Lyon-Turin railway programme requires the excavation of a 53 km long Base Tunnel. The dual national company Lyon Turin Ferroviaire (LTF), a subsidiary company of RFF and RFI, has been appointed to carry out preliminary studies and works involving the excavation of inclined galleries and exploration adits.

The Saint-Martin La Porte inclined gallery is the closest drive to the West portal of the base tunnel. It is to reach the level of the Base Tunnel situated at a distance of 2300 meters from the portal. The cross sectional area of the gallery ranges between 80 and 100 m^2.

The gallery is situated near an important zone of difficult ground and is designed to fulfil several objectives:

- provide additional geological and geotechnical data on the rock mass conditions to be encountered during the excavation of the main tunnel;
- quantify the uncertainties associated with the difficult ground to be excavated;
- provide access for the future tunnelling work.

When the tunnel will be in operation, this inclined gallery will be used for access by emergency teams and also for maintenance and ventilation purposes.

Participants in the programme are:

- Owner: LTF – Lyon Turin Ferroviaire SAS;
- Project management: Group Scetauroute (agent), Antea and Alpina;
- Contractor: Razel inter-company group (agent), Pizzarotti, Bilfinger Berger, Granulats Rhone-Alpes;
- Sub-contractor: Fondazioni Speciali/GD Test.

2 GEOLOGICAL AND GEOTECHNICAL CONDITIONS

The Saint-Martin La Porte gallery is situated near the contact between the Briançonnaise zone, composed of limestone, marl and dolomite, and the Briançonnaise Carboniferous zone, composed of schists, Carboniferous schist and coal (Fig. 1).

Between the two zones a transition zone is present comprising anhydrites (Fig. 1). The contact area between the Carboniferous and the Briançonnaise zone was found after excavating a tunnel length of 800 m and presented characteristics in line with predictions. Also encountered during excavation was a zone with fluvio – glacial alluvium under a cover of

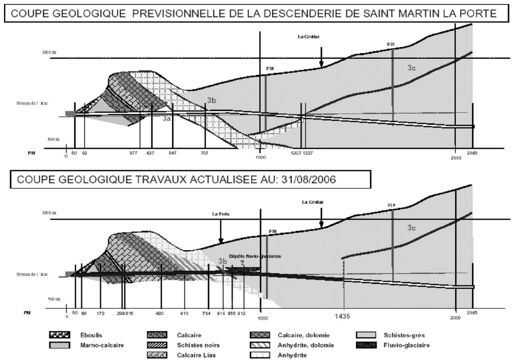

Figure 1. Geological profile.

Figure 2. Appearance of the Carboniferous rock mass at the face. Chainage 1414 m.

150 to 200 meters. This required staged excavation (top heading and bench) and pre-treatment of the ground.

Once the excavation of this latter zone was completed, the gallery encountered the Productive Carboniferous Formation, composed of schists, clays, highly tectonised sandstone and coal. A characteristic feature of the ground in this zone is the highly heterogeneous, disrupted and fractured condition of the rock mass, which exhibits extremely to very severe squeezing problems, Hoek, 2001, Barla, 2001 (see Figs. 2 and 3). As a crude approximation, the GSI (Geological Strength Index, Hoek and Marinos, 2000) is in the range 20–30 (short term conditions) down to 10–20 (long term conditions).

The conditions encountered made the different parties involved in the work to define first a support system based on heavy steel sets (HEB 180) with invert, combined with fibreglass dowel pre-reinforcement at the face, together with a concrete ring approximately 50 cm thick.

Under these conditions, the excavation resumed at a rate of 2 to 3 meters per day and was associated

Figure 3. Appearance of the Carboniferous rock mass during excavation. Chainage 1350 m.

with a specific monitoring programme. As the gallery became deeper, leading away from the valley (Fig. 1), the overburden increased. The tests carried out have confirmed the limitations of the support system adopted when subjected to the high stress conditions expected with an overburden depth in excess of 250 m.

3 SUPPORT SYSTEMS AND GROUND BEHAVIOUR

3.1 *Adoption of the yielding support system*

Several support systems were used following the initial application of a stiff support, as underlined above. It soon became apparent that a stiff support would not be feasible in the severe squeezing conditions encountered, with very high stresses developing in the concrete lining as the overburden increased.

Finally, the design concept chosen was to allow the support to yield in a controlled manner, with the following pre-reinforcement, full face excavation, and support sequence being adopted:

1. fibreglass dowel face pre-reinforcement;
2. full face excavation (1 m face advance);
3. use of yielding steel ribs with sliding joints (TH, Toussaint-Heintzmann type);
4. installation of rock bolts along the excavation perimeter, including the invert, requiring 400 linear meters of rock bolts per meter excavation.

3.2 *Considerations on the observed behaviour*

The excavation-support system, adopted over a length of approximately 100 metres, was marked by high convergences of up to 2 metres, with the radial strains of the tunnel wall ranging between 8% and 16% at a distance of 20 m behind the face. More specifically, with a progress rate of 1 m per day, convergence rates of

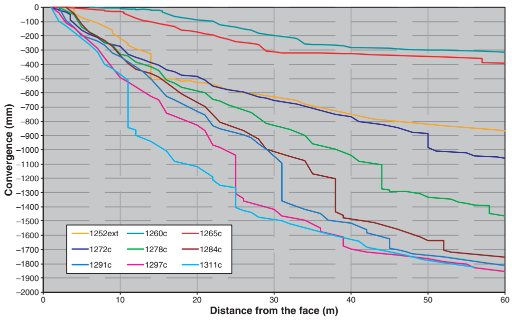

Figure 4. Tunnel convergences measured along the horizontal between chainage 1260 and 1320 m.

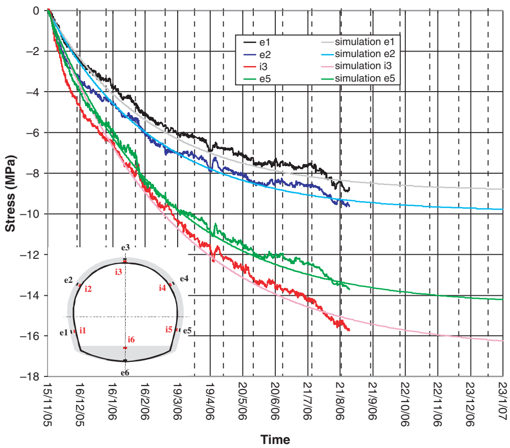

Figure 5. Stress distribution in the lining at chainage 1229 m.

approximately 40 to 50 mm per day were observed. These decreased when moving away from the face and resulted in values of 2 to 4 mm/day at distances between 80 m and 100 m from the face (Fig. 4).

In order to gain in the understanding of the deformational behaviour of the rock mass along the tunnel length, it is of interest to pay attention to the extensometer measurements carried out at chainage 1330 m. It is shown that the zone characterised by a significant strain gradient (say where the strain value is in excess of 0.5%) is un-symmetrically distributed around the opening, with the maximum extent (18 m) being attained at the right rib, in line with the rock mass anisotropic structure (Figs. 2 and 3).

Also to be emphasised, as a means for appreciating the severity of the squeezing conditions being encountered, is the state of stress in the concrete lining at chainage 1229 m as shown in Fig. 5. The vibrating wire strain-meters embedded in the concrete give tangential stresses in the lining, that are still increasing

Figure 6. Re-profiling operations of the opening to the original tunnel profile followed by concrete placing behind steel plates.

8 months following installation with a maximum long term value estimated up to 16 MPa.

The mechanical behaviour of the Carboniferous zone has been shown to be significantly less favourable than anticipated in the design documents. Based on the information gained so far through performance monitoring and back analysis (Barla and Panet, 2006) the ground was found to exhibit a unique deformational response distinguishing between a "short term" and a "long term" behaviour.

In the short term (just behind the face), if the face advance does not stop, the rock mass exhibits a negligible time-dependent behaviour. The tunnel and face deformation can be computed by using an elastic plastic constitutive law. In the long term, the time-dependent behaviour cannot be neglected and an elastic viscous plastic law needs be adopted in order to reproduce the deformational response satisfactorily (Barla and Panet, 2006; Sulem et al, 1987).

3.3 *Improvement of the yielding support system*

Due to the conditions being met, work progress was interrupted for long periods of time, as the reduction in cross section made it necessary to "re-profile" of the opening to the original tunnel profile. With systematic re-profiling operations taking place section by section up to chainage 1325 m, concrete behind the steel plates was placed as soon as possible after completion of the remedial work. Fig. 6 gives a clear view of the re-profiling operations being undertaken with adaptation of the cross section shape.

The observations made, associated with the experience gained during tunnelling in the squeezing ground conditions encountered, made it mandatory to consider a number of possible options for driving the tunnel. The aim was to be able to improve the previous phase of yielding support application, while seeking a different work organisation in order to adapt to the increased cover and to the necessity to contain the residual deformations at a certain distance from the face.

The main objectives to be achieved were as follows:

– reduce/control, where possible, the amount of convergence in the tunnel and the disturbance of the surrounding rock mass;
– use excavation and support systems compatible with the increased overburden to be experienced in the future (up to 600 m) along the inclined gallery and in the base tunnel;
– optimise the pre-reinforcement, excavation and support sequence, including the distance from the face where to install the concrete lining;
– assess the residual deformations to be expected following support installation and the long term state of stress in the concrete lining.

3.4 *Main adaptations required*

Considering the consequences in terms of the tunnel excavation and support costs, including the time delays in completion of the inclined gallery, the following adaptations were made:

– reorientation of the gallery alignment in order to drive nearly perpendicular to the geological structure;
– change the shape of the opening to a near circular section;
– implementation of a pre-reinforcement, excavation and support sequence leading to avoid the need for important re-profiling to the original tunnel section;
– control of the extent of the disturbed zone around the tunnel, thus reducing the ground time-dependent deformations.

3.5 *Pre-reinforcement, excavation, and support sequence adopted*

The new pre-reinforcement, excavation, and support sequence finally adopted can be summarised as follows (Fig. 7):

Phase 0 – face pre-reinforcement, including a ring of grouted fibreglass dowels around the opening perimeter, designed to reinforce the rock mass over a 2 to 3 metres thickness.

Phase 1 – mechanical excavation carried out in segments of one meter length, with installation of support consisting of systematic bolting (length 8 m) along the perimeter, including the invert, and yielding steel ribs with sliding joints (TH, Toussaint-Heintzmann type).

Phase 2 – application of 20 to 30 cm shotcrete with 8 longitudinal slots fitted with HDC (High Deformable Concrete) elements designed to undergo a total tangential strain of 50%, while maintaining a support pressure of 8.5 MPa with controlled movement (Figs. 8 and 9).

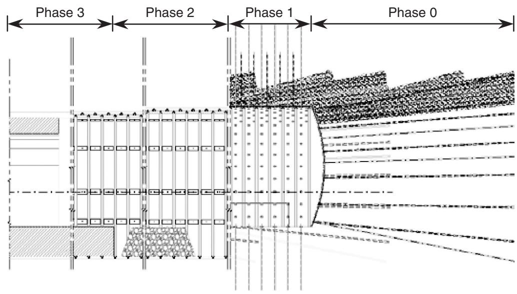

Figure 7. Schematic representation of the adapted pre-reinforcement, excavation, support sequence.

Figure 8. View of the deformable elements installed between the joints of the TH steel ribs.

Figure 9. Photograph taken during Phase 2 implementation.

Phase 3 – installation of a coffered concrete ring when the rate of convergence is low enough to allow this, and outside the zone of influence of the face (Fig. 10).

The guidelines followed for the selection of the excavation-support sequence above were developed

Figure 10. Concrete lining installation.

based on numerical simulations by using both elastic plastic and elastic viscous plastic constitutive laws, with rock mass parameters defined on the basis of performance monitoring. In this context, the support design needs be continuously reassessed according to the results of monitoring and experience feedback from the work site.

It is important to note that the distance from the face at which phase 2 is implemented will depend on the efficiency of the support measures adopted and on the convergence values observed at the face. Diameter reduction for this section is limited to 600 mm. Beyond these values, the shotcrete lining would be overstressed too soon and require repair work on the support structure.

3.6 *First lessons learned*

Use of monitoring equipment during the ongoing work makes it possible to perform back analysis and acquire the experience feedback required for the project. Systematic convergence monitoring takes place continually in both phases 1 and 2. So far also monitored in detail are the strains experienced by the HDC deformable elements installed in phase 2, in conjunction with the tangential stresses induced in shotcrete. The ground response around the excavation in both phases 1 and 2 is being monitored by using multi-point extensometers 24 m long.

Monitoring has shown that the pre-reinforcement and support measures implemented during Phases 0 and 1 have resulted in a better distribution of convergences, while maintaining a tunnel deformation (convergence/diameter ratio) of approximately 2% to 5% at a distance of 20 meters from the face.

Phase 2 (Fig. 9) has been first implemented at a distance of 35 m from the face. The shotcrete ring was rapidly under stress, with a maximum overall deformation being attained in the HDC elements of 20%, i.e. with a reserve for further deformation of up to 25–30% before reaching the limit design value. As a result, phase 2 is being moved closer to the face, to a distance of 15–20 m. Future observations of the overall system behaviour and response will allow one to decide if phase 2 could be implemented at a shorter distance from the face.

A special coffer was designed for the project. It was assembled inside the gallery and served to install the concrete lining in 5-metre long segments, as shown in Fig. 10.

4 CONCLUSIONS

The inclined gallery is currently under construction and is faced with exceptional difficulties over a significant length. This calls for a continuous reflection on the resources and methods to be used. The feedback from experience will be directly used for the excavation of the main tunnel, which is expected to be driven in similar conditions, with a cover close to that to be experienced at the bottom of the inclined gallery.

One of the main challenges ahead lies in the definition of the optimal time for installing the final lining ring, and also in the assessment of the long term state of stress for design. Consequently, particular attention needs to be paid in identifying the long term behaviour and the mechanical properties of the squeezing rock mass. The application of the interactive observational design approach, including design analyses and systematic performance monitoring, become essential for achieving this objective.

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