INNOVATIVE TUNNELLING CONSTRUCTION METHODS IN SQUEEZING ROCK

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ABSTRACT: Recent innovations in yield-control support systems are allowing contractors to increase the rate of advance when tunnelling in difficult conditions associated with severely squeezing ground. Such systems are being implemented and proven in tunnelling projects using conventional excavation methods. The Saint Martin access tunnel along the Base tunnel of the Lyon-Turin rail line is presented as a case study to illustrate some of these developments which have been implemented successfully to deal with severely squeezing conditions encountered during excavation in a Carboniferous Formation.

Keywords: Tunnelling, Squeezing rock, Full face excavation, Yielding support, Monitoring

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INTRODUCTION

Europe is experiencing today a new "Renaissance" in tunnelling. New infrastructures are being constructed underground and will continue to be constructed, including subways and other urban transportation systems, as well as improvement for existing roads and highways. However, the most significant excavation challenges probably are associated with the construction of high-speed railway lines. Of particular importance are the new crossings through the Alps in the form of base tunnels for rail transportation, such as the recently completed 34 km long Lötschberg Base tunnel and the 57 km Gotthard Base tunnel yet to be completed in Switzerland [1]. Nearly completed in Italy are tunnels with more than 80 km total length for the high-speed rail line between Bologna and Firenze [2]. Other tunnelling projecs in the Alps include the 53 km long Base tunnel along the Lyon-Turin rail line [3] and the Brenner Base tunnel, which is planned to be 54 km long [4].

During the excavation of the Lötschberg Base tunnel, after a 300 m long section in Gastern granite, the tunnel encountered an unpredicted area of sedimentary rock (marlstone, sandstone and siltstone), containing alternating beds of coal (anthracite). Severe squeezing problems occurred with significant convergences and some local instabilities of the tunnel heading [5]. A critical zone with squeezing conditions, about 1150 m long, also was encountered in the Gotthard Base tunnel, while excavating through the Northern Tavetsch Massif. Here, alternating layers of intact and variable strength kakiritic gneisses, slates, and phyllites were encountered (kakirite describes a broken or intensely sheared rock, which has lost much of its original strength) [6]. During the excavation of tunnels between Bologna and Firenze, clay shales exhibited a very severely squeezing behaviour [7]. Severe squeezing problems have significantly impacted the excavation of the Saint Martin access tunnel along the Base tunnel of the Lyon-Turin rail line. Squeezing conditions are also anticipated in some lengths of the Brenner Base tunnel [8].

Tunnel construction in squeezing conditions is very demanding due to the difficulty in making reliable predictions at the design stage. During excavation such conditions are not easily anticipated, even when driving into a specific geological formation and experience is gained on the squeezing problems encountered. Squeezing conditions may vary over short distances due to rock heterogeneity and fluctuations in the mechanical and hydraulic properties of the rock mass. Indeed, the selection of the most appropriate excavation-construction method (i.e. mechanised tunnelling versus conventional tunnelling) is highly problematic and uncertain. Due to the fixed geometry and the limited flexibility of the TBM (Tunnel Boring Machine) allowable space to accommodate ground deformations is restricted. On the contrary, in conventional tunnelling a considerably larger profile can be excavated initially in order to allow for large deformations. The obvious consequence is that in deep tunnels, whenever severely squeezing conditions are anticipated, conventional tunnelling appears to be preferred over mechanised tunnelling.

Conventional tunnelling in squeezing rock generally takes place with a slow rate of advance. However, if the work at the face is well planned and appropriate stabilisation measures are implemented, excavation can proceed even in severely squeezing conditions. On the contrary, if anything goes wrong in mechanised tunnelling the excavation is significantly hindered, and in many cases may come to a complete standstill. Thus, there is a clear need to develop appropriate technological systems that help increase the rate of advance in conventional tunnelling. It is the purpose of this paper to describe some of these recent innovative technological developments and to present a case study to illustrate some of the methods.

SQUEEZING ROCK BEHAVIOUR

The term "squeezing rock" originates from the pioneering days of tunnelling through the Alps. It refers to the reduction of the tunnel cross section that occurs as the tunnel is being advanced (Figure 1). Based on the work of a Commission of the International Society for Rock Mechanics (ISRM) [9], which has described squeezing and the main features of this mechanism, it is agreed today that "squeezing of rock" stands for large time-dependent convergence during tunnel excavation. This happens when a particular combination of material properties and induced stresses causes yielding in some zones around the tunnel, exceeding the limiting shear stress at which creep starts. Deformation may terminate during construction or continue over a long period of time.

Figure 1 Squeezing rock reduces the tunnel cross section. This is shown dramatically in this photograph taken at the Saint Martin La Porte access tunnel along the Lyon-Turin Base tunnel, where re-profiling of the highly deformed cross section took place.

The magnitude of tunnel convergence, the rate of deformation, and the extent of the yielding zone around the tunnel depend on the geological and geotechnical conditions, the in-situ state of stress relative to rock mass strength, the groundwater flow and pore water pressure, and the rock mass properties. Squeezing is therefore synonymous with yielding and time-dependence, and often is largely dependent on excavation and support techniques being used. If the support installation is delayed, the rock mass moves into the tunnel and a stress redistribution takes place around it. On the contrary, if deformation is restrained, squeezing will lead to long-term load build-up of the support system.

The squeezing behaviour during tunnel excavation has intrigued experts for years, and often has caused resulting great difficulties for completing underground works, with major delays in construction schedules and cost overruns. There are numerous cases of particular interest in Europe and world-wide where squeezing phenomena have occurred, providing some insights into the ground response during excavation. A review of these case studies leads to the following remarks:

- Squeezing behaviour is associated with poor rock mass deformability and strength properties. Based on previous experience, there are a number of rock complexes where squeezing may occur if the loading conditions needed for the onset of squeezing are present: gneiss, micaschists and calcschists (typical of contact and tectonized zones and faults), claystones, phyllite, flysch, clay-shales, marly-clays, etc.
- Squeezing behaviour implies that yielding will occur around the tunnel. The onset of a yielding zone in the tunnel surround causes a significant increase in tunnel convergence and face displacements (extrusion); these are generally large, increase in time and form the most significant aspects of the squeezing behaviour.
- Orientations of discontinuities, such as bedding planes and schistosities, play a very important role in the onset and development of large deformations around tunnels, and therefore also on the squeezing behaviour. In general, if the main discontinuities strike parallel to the tunnel axis, the deformation will be enhanced significantly, as observed in terms of convergence during face advance.
- The pore pressure distribution and the piezometric head also can influence the rock mass stress-strain behaviour. Drainage measures that cause a reduction in piezometric head both in the tunnel surround and ahead of the tunnel face often help to reduce ground deformations.
- Construction techniques for excavation and support (i.e., the excavation sequences and the number of excavation stages which are adopted, including the stabilisation methods used) may influence the overall stability conditions of the excavation. In general, the ability to provide an early confinement on the tunnel periphery and in near vicinity to the face is considered the most important factor in controlling ground deformations.
- Large deformations associated with squeezing also may occur in rocks susceptible to swelling. Although the factors that cause either behaviour are different, it is often difficult to distinguish between squeezing and swelling, as the two phenomena may occur at the same time and induce similar effects. For example, in over-consolidated clays, the rapid stress-relief due to the tunnel excavation causes an increase in deviatoric stresses with simultaneous onset of negative pore pressure. In undrained conditions, the ground stresses may be such as not to cause squeezing. However, due to the negative pore pressure, swelling may occur with a more sudden onset of deformations under constant loading. Therefore, if swelling is restrained by means of early invert installation, a stress increase may take place with probable onset of squeezing.

TUNNEL AND FACE STABILITY IN WEAK ROCK

In general, the major difficulties encountered when tunnelling in weak rock, are associated with both the stability of the tunnel and of the face. It is known that the tunnel face follows the same general deformation pattern as the tunnel itself, although the longitudinal face displacements are about 30% smaller than the radial displacements at the tunnel perimeter [10]. This pattern of behaviour is well illustrated in Figure 2, where the typical plastic zone and the deformation processes in the rock mass surrounding an advancing tunnel excavated full face are shown, as obtained by means of an axi-symmetric finite-difference model. For weak rock masses and in deep tunnels the stability problems of the face and of the "core" ahead of the same advancing face may become as important as the stability of the tunnel heading.

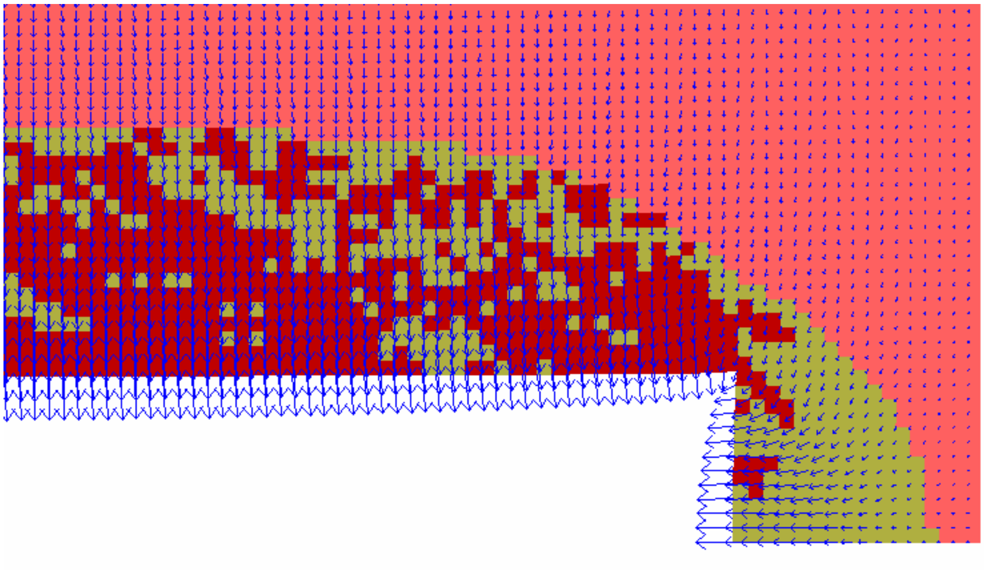

Figure 2 Plastic zone and deformation process (convergence of perimeter and extrusion of face) surrounding an advancing tunnel excavated full face in a squeezing rock mass calculated by means of an axi-symmetric finite difference model

Of the available options for conventional tunnel excavation and construction in squeezing rock (e.g. multiple headings, top heading and bench, full face), the most successful method applied in many cases is the full-face method (Figure 3). A significant advantage of this method is the large working space available at the advancing face, so that large equipment can be used effectively for installing support/stabilization measures at the tunnel perimeter and ahead of the face. In poor rock conditions, this method requires a systematic reinforcement of the working face and of the ground ahead. Generally, the cross section is entirely open and a primary lining is installed as near to the face as possible [11].

When tunnelling in squeezing conditions, one of two basic types of support ("heavy" and "light") can be applied. For the "heavy method" ("resistance principle"), the primary lining is designed to be very stiff (generally composed of steel fibre shotcrete and heavy steel sets), the "ring is closed quickly" (Figure 3), and the final concrete invert (first) and final concrete lining (second) are cast within a short distance from the face. It is apparent that if very high rock pressures are expected, this solution soon becomes impractical. With the "light method" ("yielding principle"), the aim is to accommodate the large deformations likely to develop in the tunnel surround with the expectation that rock pressure will decrease with increasing deformation. The excavation profile is chosen so as to maintain the desired clearance and to avoid the need for re-profiling. A key point when driving the tunnel this way is to be able to control the development of deformations. In addition to the requirement of maintaining stability at the face and the heading during tunnel driving, a suitable tunnel support system is to be adopted that will allow for accommodating deformations without damage of the lining.

Figure 3   Full face excavation/construction method: face reinforcement and ring closure operations.

Face stability

One of the most effective methods for achieving face stability when driving a tunnel full-face consists in reinforcing the rock mass ahead by means of grouted fiber-glass dowels. There are a number of fiber-glass structural elements that may be adopted. Both smooth and corrugated tubes are available. More recently, flat elements (Figure 4) are being used which can be assembled in situ in a wide variety of types; they are very easy to inject and transport, and they allow reinforcement advance steps up to 25 m.

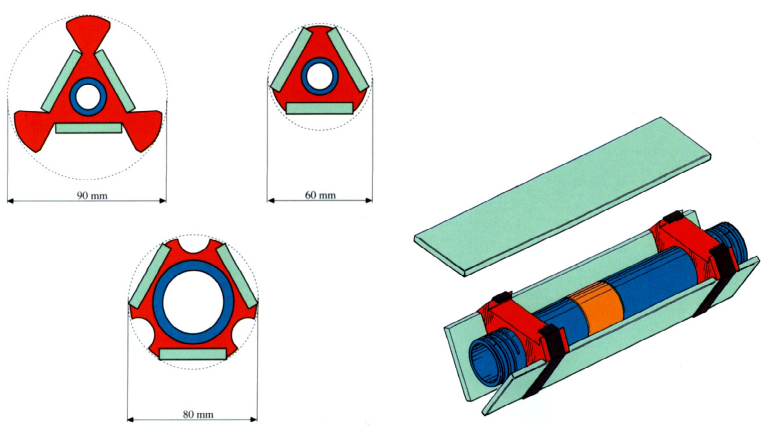

Figure 4   Flat fiber-glass structural elements adopted for face reinforcement in the full-face excavation/construction method.

A typical reinforcement scheme is shown in Figure 5 where the fiber-glass elements are used for both the face and a thick ring surrounding the tunnel. In line with the "resistance principle", closely spaced steel sets are incorporated in a thick shotcrete shell to form the primary lining which is installed close to the face as early as possible.

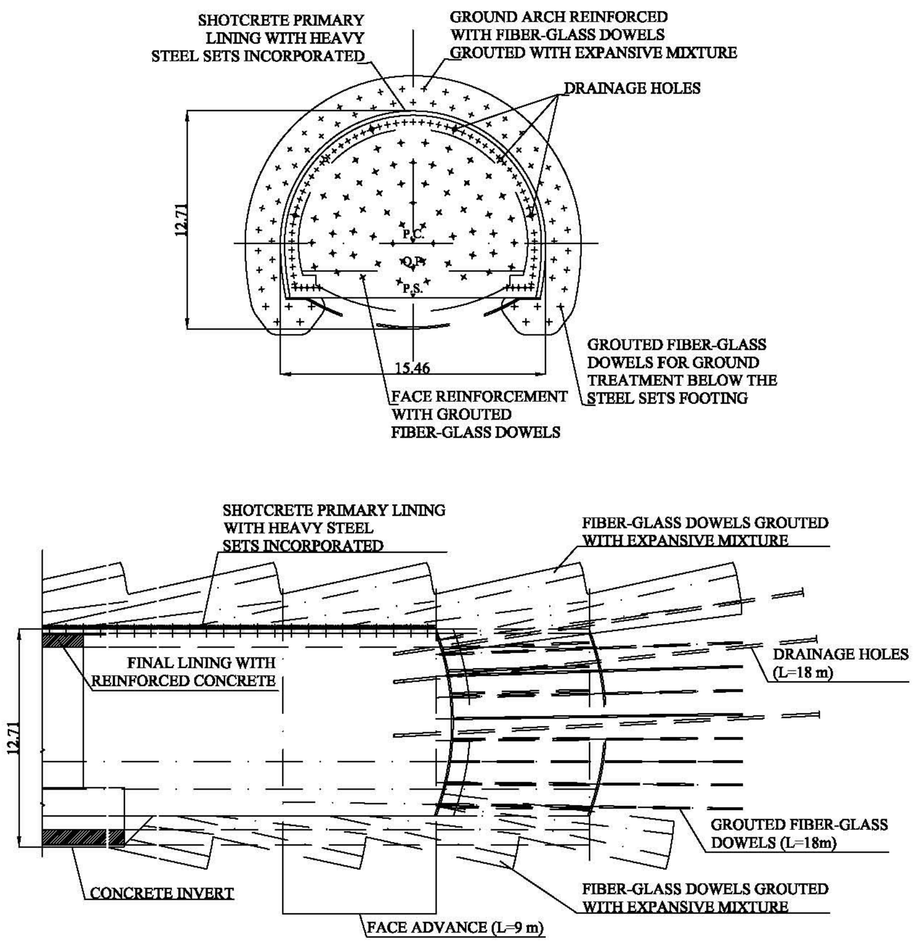

Figure 5 Typical face reinforcement system used with the full face method. Cross section (above) and Longitudinal Section (below). Schematic drawings only, not to scale.

Yielding support

Early applications of a yielding support system consisted of yielding steel sets embedded in a shotcrete lining containing a number of open gaps, in conjunction with dense rock bolting. Various design options have since been proposed and applied in order to deal with squeezing conditions in Alpine tunnels [12]. Two technical options that recently have demonstrated the ability to maintain flexibility in the primary lining and to accommodate deformations without significant damage are shown in Figures 6 and 7 . They both can be installed in the gaps of a shotcrete lining and between steel sets with yielding couplings. In this way, the lining system yields, allowing the tunnel to converge (Δr) in a controlled manner while keeping the tangential stress σ_ϑ constant and applying a confinement stress σ_r (Figure 8).

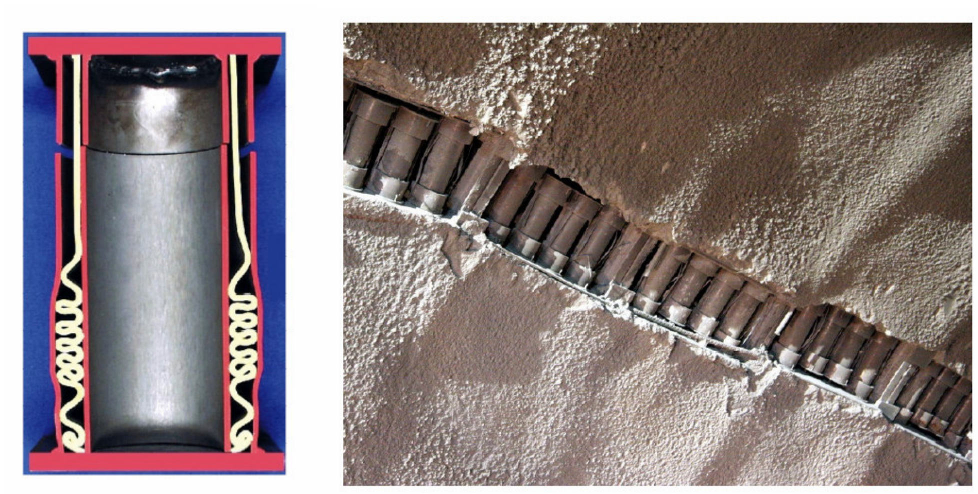

Figure 6 Yielding support with LSC elements incorporated in the shotcrete lining and between steel sets with yielding couplings [12].

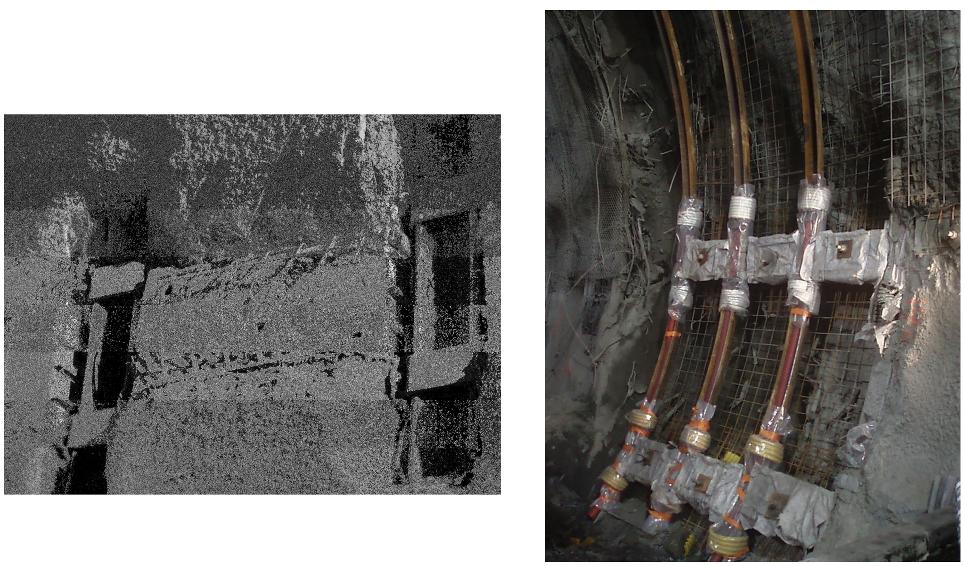

Figure 7 Yielding support with Hidcon elements incorporated in the shotcrete lining and between steel sets with yielding couplings. Installation details as adopted in the Saint Martin La Porte access tunnel.

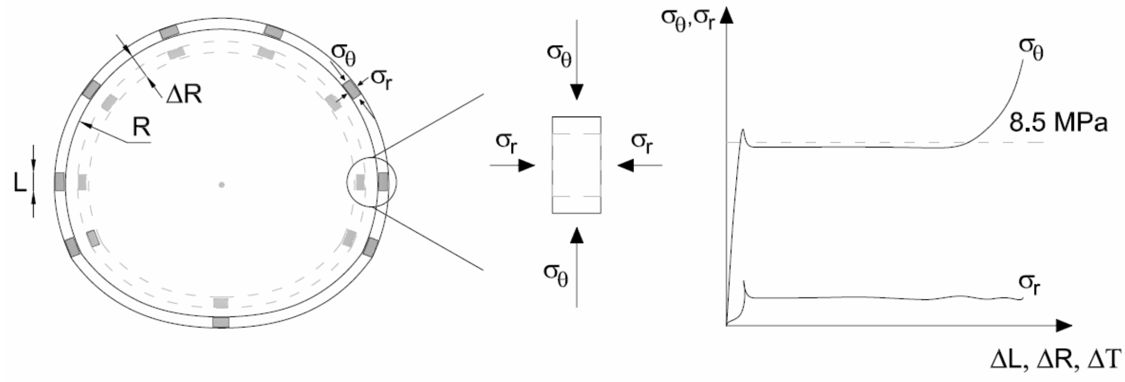

Figure 8 Schematic representation of the behaviour of a yielding support with compressible elements incorporated in the shotcrete lining.

The first compressible element (Lining Stress Controller - LSC) was developed by the Geotechnical Group Graz [12] between 1996 and 1999. In its most recent design each LSC unit (Triple Tube System) consists of three coaxial cylinders which are loaded in the axial direction and buckle in stages, shortening up to 200 mm under a load of 150-200 kN. As shown in Figure 6, three LSC units are installed in the gaps of the shotcrete lining. The second compressible element (High Deformable Concrete – HiDCon) has a beam shape and is composed of a mixture of cement, steel fibres and hollow glass particles [13,14]. The glass particles increase the void fraction of the mixture and collapse at a predetermined compressive stress. The yielding strength depends on the composition of the mixture and ranges from 4 to18 MPa [13]. The maximum allowable strain is approximately 50 percent. Also in this case, as depicted in Figure 7, the elements are incorporated into shotcrete. It is this latter element type that has been successfully applied in the Saint Martin La Porte access tunnel along the Lyon Turin Base tunnel.

THE SAINT MARTIN LA PORTE ACCESS TUNNEL

The Saint Martin La Porte access tunnel (Figures 9 and 10) is being excavated in the Carboniferous Formation, "Zone Houillère Briançonnaise-Unité des Encombres" (hSG in Figure 9), which is 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 observed at the face during excavation (Figure 10) is the highly heterogeneous, disrupted and fractured conditions of the rock mass which exhibits very severe squeezing problems. The formation often is affected by faulting that results in a degradation of the rock mass conditions. The overburden along the tunnel in the zone of interest ranges from 300 m to 550 m. Excavation takes place in essentially dry conditions.

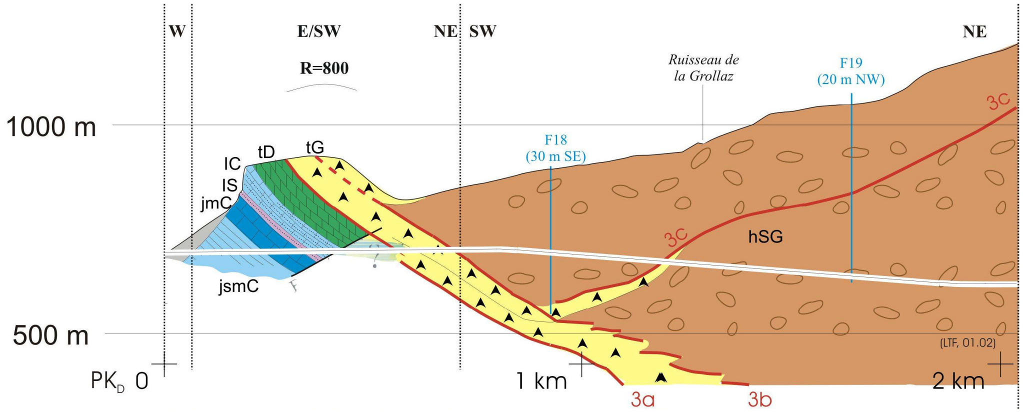

Figure 9 Geological profile along the Saint Martin La Porte access tunnel.

In order to assess the rock mass quality during excavation detailed mapping of the geological conditions at the face was undertaken as depicted in Figure 10. This provides information to evaluate the percent distribution of “strong” (andstones and schists) and “weak” (coal and clay-like shales) rocks at the face.

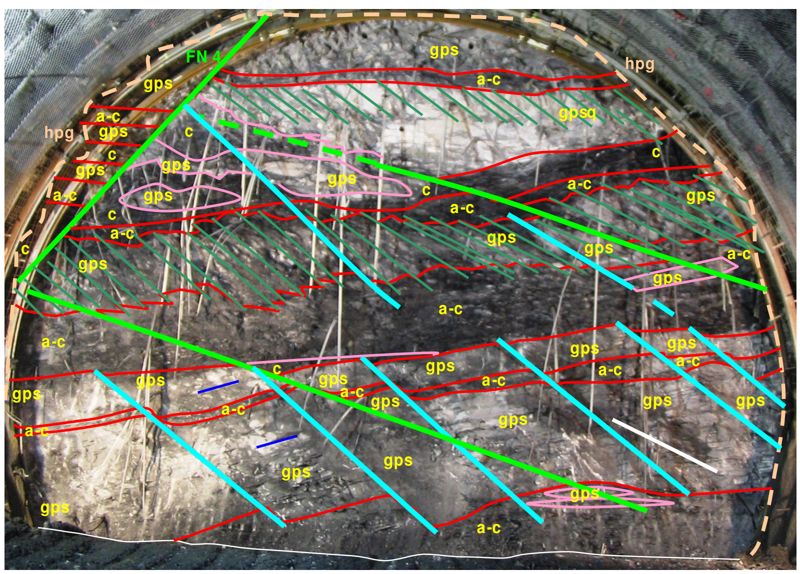

Figure 10 Typical geological conditions at the face
(gps-sandstones, a-clay like shales, c-coal, etc.)

Excavation-support system adopted

Several support systems were used in the Carboniferous zone. However, it soon became apparent that a stiff support would not be feasible in the severe squeezing conditions encountered. The design concept finally chosen [15] was based on allowing the support to yield while using full-face excavation with systematic face reinforcement by fiber-glass dowels. The support system initially implemented (Figure 11) 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 to be re-profiled (Figure 1).

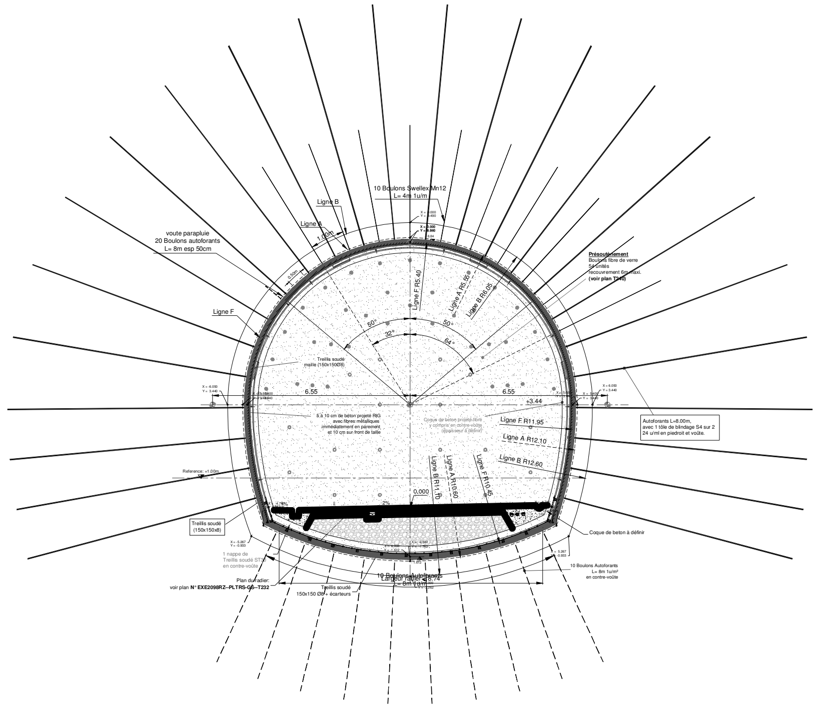

Figure 11 Tunnel cross section (P7.3) showing the excavation-support system adopted between chainage 1267 and 1324 m, where large convergences were experienced and re-profiling was required per Figure 1.

In order to improve the working conditions and to control deformations a novel support system was implemented with a near circular cross section. This can be summarized as follows (Figure 12):

- Stage 0: face pre-reinforcement, including a ring of grouted fiber-glass dowels around the opening perimeter, designed to reinforce the rock mass over a 2 to 3 m thickness.
- Stage 1: mechanical excavation carried out in steps of one meter length, with installation of a support system consisting of untensioned rock anchors (length 8 m) along the perimeter, yielding steel ribs with sliding joints (TH type), and a 10 cm thick shotcrete layer. The tunnel is opened in the upper cross section to allow for a maximum convergence of 600 mm.
- Stage 2: the tunnel is opened to the full circular section at a distance of 15-25 m from the face, with application of 20 cm shotcrete lining, yielding steel ribs with sliding joints (TH type) with 9 longitudinal slots (one in the invert) fitted with HiDCon (High Deformable Concrete) elements. The tunnel is allowed to deform in a controlled manner so as to develop a maximum convergence which should not exceed 400 mm.
- Stage 3: installation of a coffered concrete ring at a distance of 80 m from the face.

As shown in Figure 12, the HiDCon elements, which represent indeed the most recent technological development when tunneling in squeezing rock conditions, are installed in slots in the shotcrete lining between TH type steel ribs (Figure 13). These yield-control support system for the tunnel allows controlled deformations to take place as explained in principle in the diagram of Figure 8. In the project conditions of the Saint Martin access tunnel the HiDCon elements have a height of 40 cm, a length of 80 cm, and a thickness of 20 cm. They have been designed to yield up to approximately 40 percent strain in a ductile manner, while the yield stress has been chosen to be 8.5 MPa. Figure 14 shows the characteristic stress-strain diagrams obtained in our MASTRLAB laboratory.

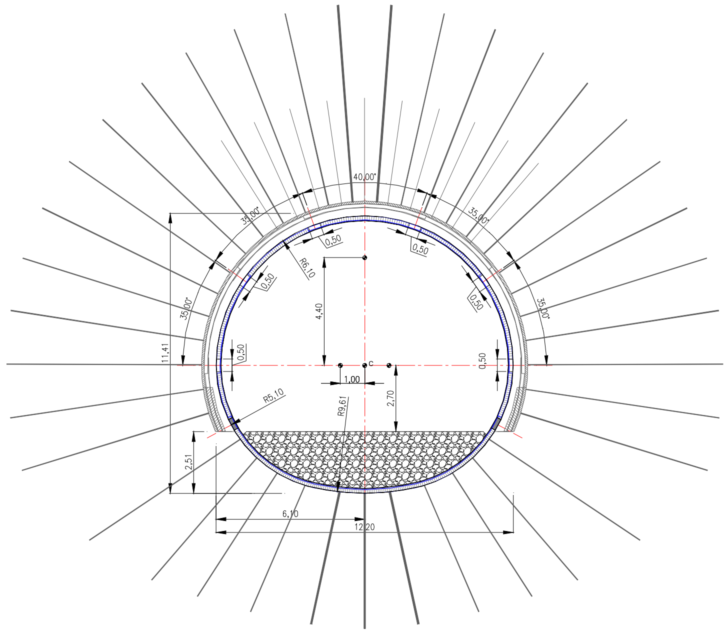

Figure 12 Tunnel cross section (DSM) showing the excavation/support system adopted

(a) (b)

Figure 13 (a) Deformable lining with HiDCon elements, (b) Detail of a HiDCon element at maximum strain during testing in the MASTRLAB laboratory.

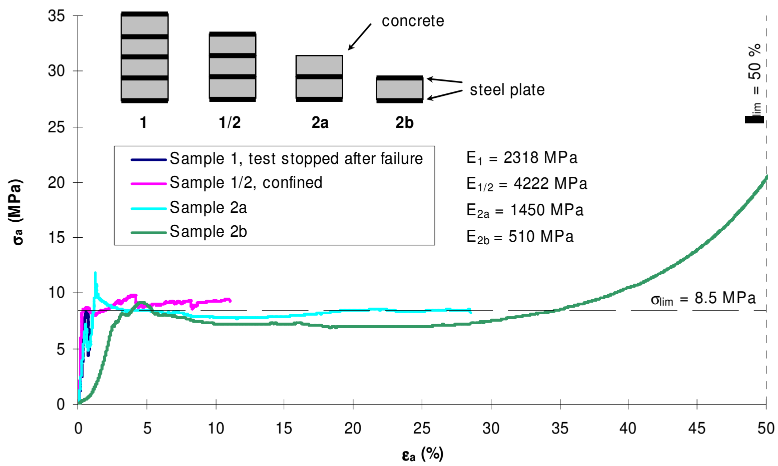

Figure 14 Characteristic stress-strain diagram of a HiDCon element tested in the MASTRLAB laboratory.

Controlled response of the tunnel deformation

Systematic monitoring of tunnel convergence is underway along the tunnel where the support system described above is being adopted systematically. Convergences are measured by means of optical targets placed along the tunnel perimeter (5 in phase 1 and 7 in phase 2). A number of special sections have been equipped with multi-position borehole extensometers and strain meters located across the HiDCon elements. Extrusometer monitoring has been used to measure the longitudinal displacement ahead of the tunnel face. In addition, the strain level in the primary lining has been monitored.
In order to gain an understanding of the tunnel response so far, it is of interest to consider the diagram of Figure 15, which shows the convergences measured along arrays 1-3, 3-5 and 1-5 (Δl_{i-j}) between chainage 1200 m and 1650 m, with the tunnel face being 15 m ahead of the monitoring section. Also illustrated in Figure 15 is the tunnel “deformation” that has occurred (i.e. the convergence divided by the length of each array measured at the time of installation of the optical targets). It is relevant to point out that three stops of face advance took place as follows:

- at chainage 1494 m, for 18 days due to the 2006 Christmas holidays;
- at chainage 1545 m, for 28 days, in relation to the tendering procedure that took place between April and May 2007;
- at chainage 1605 m, for 14 days, for the excavation of a side drift at chainage 1488 m.

The following observations can be made:

- large deformations are associated with cross section P7.3 between chainages 1200 and 1400 m; with cross section DSM the convergences in phase 1 generally are smaller with the tunnel strain never in excess of 6-7 %
- the 600-mm allowed convergence with cross section DSM has been exceeded locally (e.g., between chainages 1525 and 1550 m where the rock mass quality was very poor) and required re-profiling of the tunnel cross section before installing the composite lining adopted in phase 2
- the tunnel deformation associated with cross section P7.3 appears to be rather different in one section with respect to the neighboring one, which is not the case for cross section DSM.

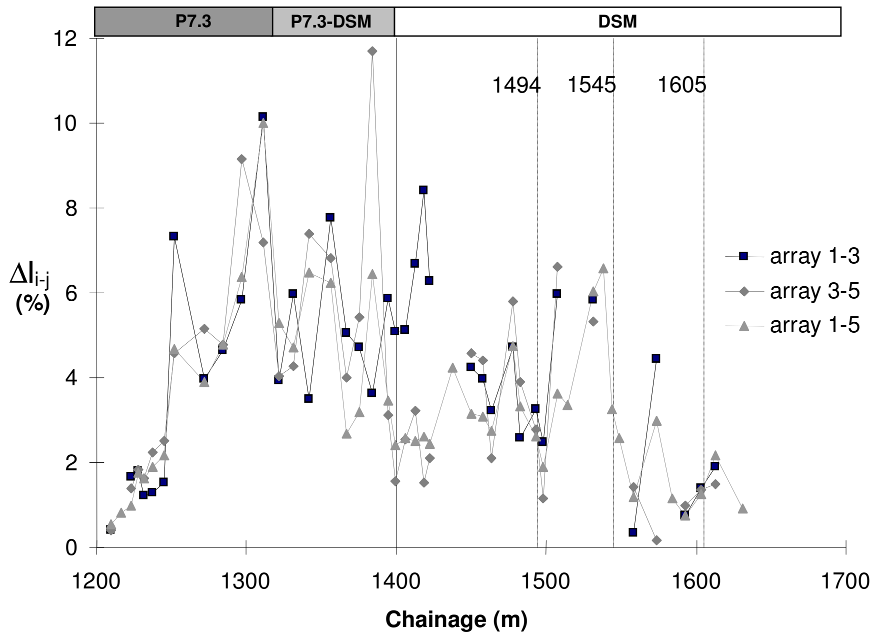

Figure 15 Convergences measured along arrays 1-3, 3-5 and 1-5 at 15 m from face in phase 1.

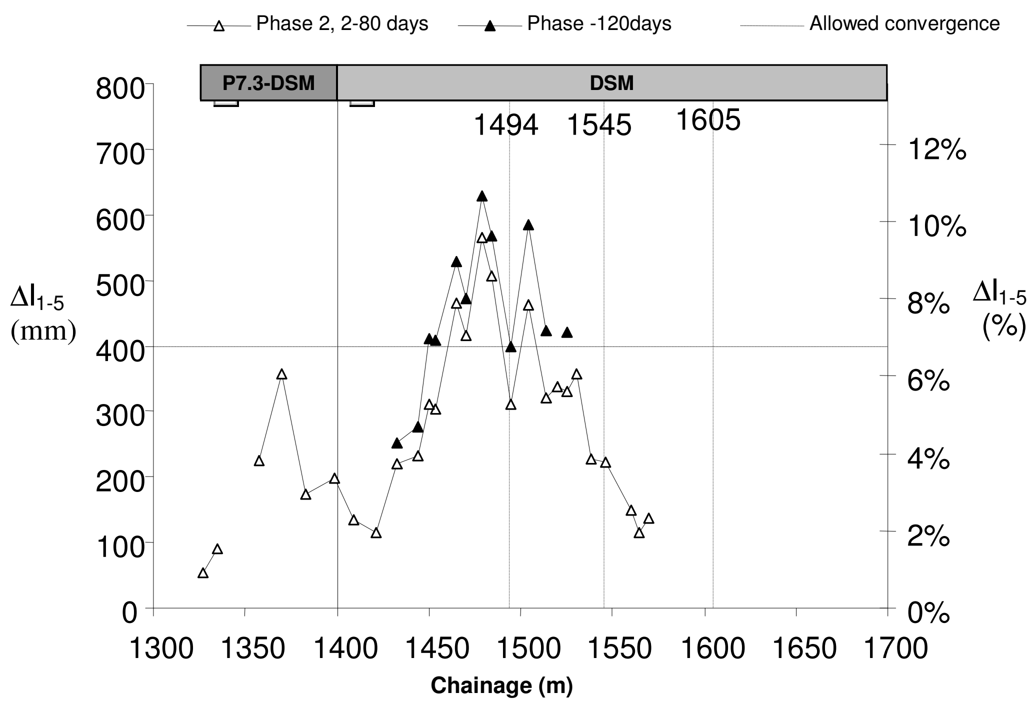

Figure 16 Convergences at 80 and 120 days following excavation with phase 2 installed.

It also is important to consider the tunnel convergence versus time in phase 2 depicted in Figure 16 above. This behavior occurs at a significant distance from the advancing face and when the yielding support has been active for a certain time duration and the final concrete lining has not yet been installed. The graph is for 80 and 120 days following excavation of the monitored section. It is noted that between chainages 1450 and 1525 m the tunnel cross section experienced along array 1-5 deformations in excess of that theoretically allowed (400 mm). In such a case the yielding elements on the right wall (looking at the tunnel face) attained the 40% limit strain (Figure 14) so that visible overstressing occurred in them, which did not seem to be an issue because no difficulty was encountered before installing the final lining.

A back analysis of the monitored data between chainages 1394 and 1507 m gives: $C_{\infty x}$ = 602.6 mm, X = 41.2 m, T = 35.3 days and m = 1.09, based on using the following relationship [16]:

$$C(x,t) = C_{\infty,x} \cdot \left[1 - \left(\frac{X}{x+X}\right)^2\right] \cdot \left\{1 + m \cdot \left[1 - \left(\frac{T}{t+T}\right)^{0.3}\right]\right\}$$

where $C(x,t)$ is the convergence at the distance x from the tunnel face and at the time t, $C_{\infty,x}$ is the convergence at distance x obtained in the case of an infinite rate of face advance (no time dependent effect), m is a non dimensional parameter which depends on the ground conditions, X is a distance related to the distance of influence of the face (for an elasto-plastic model of behaviour $X = 0.84\ R_{pl}$, with R_{pl} taken as the plastic radius of the tunnel), T is a characteristic parameter of the rock mass time dependent properties.

It is of interest to compare these characteristic parameters with those obtained for the Lötschberg Base tunnel when crossing the Carboniferous Formation [5] as shown in the table below:

	$C_{\infty x}$ (mm)	*X (m)*	*T (days)*	*m (-)*
Saint Martin La Porte tunnel	602.6	41.2	35.3	1.09
Lötschberg tunnel	536-890	38.3-48.0	35.7-41.0	0.688-0.768

It is noted that the characteristic parameters in the two tunnels are remarkably similar. In both cases the distance of influence of the face (this length may be estimated to be four times the value of *X* [16]) is greater than 160 m; also, the time dependent properties of the rock mass, which are related to the T parameter, appear to be nearly the same, which is to say that the severity of the squeezing conditions in the two tunnels might be similar.

CONCLUDING REMARKS

This paper discusses tunnel construction in very severe squeezing conditions when excavation takes place with the conventional method and an innovative construction technique is applied which consists of reinforcing the face, while allowing for controlled deformation of the tunnel cross section behind. Central to the construction method adopted is the introduction of a near circular tunnel cross section, the reinforcement of the tunnel face by fibreglass dowels, and the use of highly deformable concrete yielding elements incorporated in the tunnel primary lining which is formed with shotcrete and yielding steel ribs.

The case study of the Saint Martin La Porte access tunnel along the Lyon-Turin high speed rail line, in a length which is being excavated through the Carboniferous Formation, "Zone

Houillère Briançonnaise-Unité des Encombres", is illustrated. Emphasis is placed on the description of the excavation-support method adopted and of the tunnel response as observed by careful performance monitoring during face advance. It is shown that, although the ground conditions met are characterized by an unusually very severe squeezing behaviour, face advancement is now taking place regularly and continuously.

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