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Full-face excavation of large tunnels in difficult conditions

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ABSTRACT

Following a few preliminary remarks on the tunneling methods at the beginning of the 20th century, the successful applications of the full-face method also in difficult conditions are underlined. The attention is posed on the use of a systematic reinforcement of the face and of the ground, by means of fiber-glass elements. A selection of tunnels where this method was used successfully is reported with the purpose of illustrating the wide spectrum of ground conditions where it has been applied. Then, following a description of the main concepts behind the method, the attention moves from the so-called "heavy method", where deformations are restrained, to the "light method", where deformations are allowed with the intention to decrease the stresses acting on the primary and final linings. The progress in the application of the "light method" is underlined, up to the development of a novel technique, which relies on the use of a yielding support composed of top head steel sets with sliding joints and special deformable elements inserted in the primary lining. The well-known case study of the Saint Martin La Porte access adit, along the Lyon-Turin Base Tunnel, is described. In this tunnel, a yield-control support system combined with full-face excavation has been adopted successfully in order to cope with the large deformations experienced during face advance through the Carboniferous formation. The monitoring results obtained during excavation are illustrated, together with the modeling studies performed when paying attention to the rock mass time-dependent behavior.

1. Introduction

No established method was available at the beginning of the 20th century for the design and excavation of tunnels in difficult conditions. However, with the need of new transportation lines and the increasing mobility, spectacular tunnels were excavated and completed. Important examples in Europe are the Frejus and Simplon railway tunnels through the Alps and "*La Grande Galleria dell'Appennino*" through the Apennines in Italy, as shown in Fig. 1. Different tunneling methods were applied such as the Belgian, Austrian and Italian methods that were characterized by significant differences in the choice of the section where to initiate excavation. In general, tunneling used to take place in stages by the sequential excavation method.

The need to improve the way of thinking of the time was soon recognized in order to make tunneling a more systematic and efficient work. Simplon Engineers Andre and Rothpletz emphasized the advantage of full-face excavation already one hundred years ago (e.g. Andreae, 1956). Rabcewicz (1964) pointed out that tunnels should be driven full-face whenever possible. It is indeed in this context that in the mid-1980s, Lunardi (2000) saw the importance of the stability of the face, in particular with the increase of the size of tunnels and their depth below surface, and promoted the full-face method. He suggested that understanding and controlling the behavior of the "core" ahead of the advancing tunnel face are the secret to successful tunneling in difficult conditions.

Tunnel construction in such conditions (e.g. in poor quality ground or in squeezing or unstable ground) is very demanding and reliable predictions at the design stage are difficult if not impossible, so that most often the use of the "interactive observational approach" is advocated. If consideration is given to the construction of deep tunnels (such as the new Alpine Tunnels in Europe, e.g. Lötschberg, Gotthard, Lyon-Turin, and Brenner Base Tunnels), alignment constraints, and uncertainties of geological exploration, it is not always possible to avoid difficult conditions. Therefore, the selection of the most appropriate excavation-construction method is highly problematic and uncertain. The choice is in all cases between mechanized tunneling (tunnel boring machine, TBM) and conventional tunneling.

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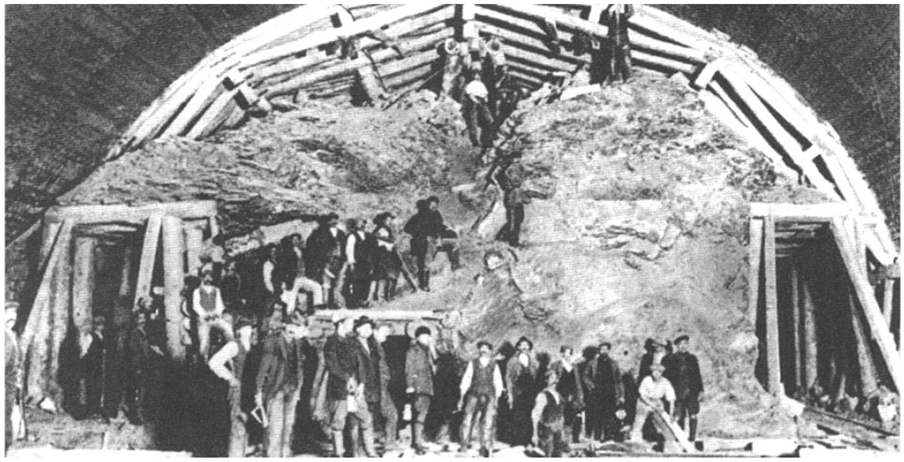

Fig. 1. "*La Grande Galleria dell'Appennino*", excavated through the Apennines along the Bologna-Florence railway line between 1921 and 1934.

In mechanized tunneling, due to the fixed geometry and the limited flexibility of the TBM, allowable space to accommodate ground deformations is restricted. On the contrary, in conventional tunneling where a considerably larger profile can be excavated in order to allow for large deformations, inevitable excavation will take place with a low rate of advance. It is however true that, if the work is well planned and appropriate stabilization measures are implemented, excavation may proceed at an acceptable rate of advance even in very difficult ground conditions, as in squeezing conditions.

The purpose of this paper is to describe the full-face excavation method, which has been experienced successfully so far under various conditions, at depth or near the ground surface. In doing this, the attention is paid to the use of the so-called "heavy" and "light" methods of tunneling, meaning that limited or significant deformations of the ground around the tunnel are allowed to take place in a controlled manner. An attempt is made through a case study to outline some of the geotechnical issues in view of modeling the tunnel behavior and observation and monitoring during excavation.

2. Full-face tunneling

Full-face tunneling has been applied in Italy for more than 30 years, with cross-sectional areas ranging from 120 m^2 to 220 m^2, in

Fig. 2. Photographs showing the face of a few selected tunnels excavated full-face by using the "heavy method" (Lunardi and Barla, 2014).

Table 1
Selected projects (see Fig. 2).

Project Tunnel	Length (m)	Ground type	Overburden (m)	Diameter (m)
Milan–Rome High Speed Rail Line (in 1987), 6 tunnels between Florence and Arezzo	11,900	Sandy silts, lacustrine deposits	80	13.5
Caserta-Foggia Rail Line (in 1991), San Vitale Tunnel	2500	Clay-shales	100	12.5
Ancona-Bari Rail Line (in 1993), Vasto Tunnel	5000	Silty, clays	135	12.2
TGV Mediterranée Marseille-Lyon (in 1993), Tartaiguille Tunnel	900	Swelling clays	110	15.0
Rome–Naples High Speed Rail Line (in 1994), 22 tunnels	21,987	Clays, pyroclastic and volcanic rocks, lava, clay shales, sandstone, limestone	114	13.5
Milan–Rome High Speed Rail Line (in 1996), 9 tunnels between Bologna and Florence	73,000	Silty clays, marls, flysch, sandstone, limestone	560	13.5
Large Open Ring Roma (in 2000), Appia Antica Twin Tunnels	2 × 620	Pyroclastic rock	18	20.7
SS106 "Jonica" (in 2012), twin tunnel	2 × 13,265	Silty clays	120	13.0
Marche-Umbria (in 2013), 11 twin tunnels	2 × 20,000	Soil deposits, limestone, marly limestone, marl	350	14.0
"Pedemontana Lombarda" (in 2013), 3 twin tunnels	2 × 2910	Gravel and sand, conglomerates, sandstone, marly sandstone	70	16.0
SS 212 "Val Fortòre" (in 2014), 4 tunnels	3000	Flysch	36	14.6
Highway A1 "Variante di valico" between Bologna and Florence (in 2014), 8 twin tunnels	2 × 45,000	Flysch, scaly clays, sandstone	150	15.7

different ground conditions, near the surface or at depth (Lunardi, 2000). Fig. 2 and Table 1, taken from Lunardi and Barla (2014), show a selection of these tunnels with the purpose of conveying the wide spectrum of ground conditions where the method has been applied successfully. In addition to major railway, highway and road tunnels in Italy, also reported in Fig. 2 is the "Tartaiguille" tunnel in France, excavated along the railway line "TGV Mediterranée".

Based on the wide experiences gained so far, the most important component of successful design and construction of tunnels when full-face excavation is considered is the ability to understand the importance of the ground deformational response. This is strictly linked to the formation of the "arching effect" (i.e. the ability of the ground to stabilize and sustain itself), needed for reaching stability of the underground excavation, in the short and long terms.

As illustrated in Fig. 3, the designer has to make appropriate predictions of the "extrusion" (longitudinal displacement) of the face, the "pre-convergence" (radial displacement ahead of the face) and the "convergence" of the tunnel perimeter (radial displacement behind the face), before starting excavation. From this point of view, one essential component of the approach is the full understanding of the tunnel behavior in three-dimensional conditions.

When full-face tunnel excavation is carried out, stresses and strains do develop in the surrounding rocks, including the "core" ahead of the advancing tunnel face. This "core" is the main tool for

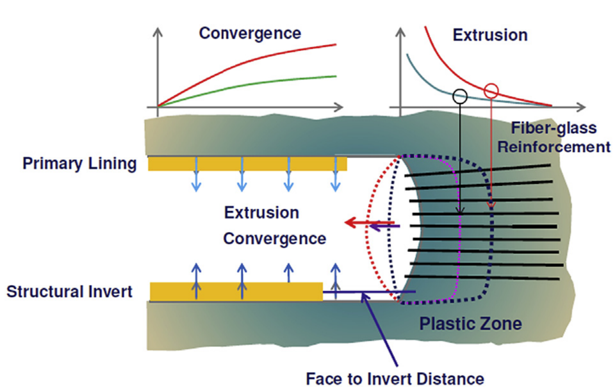

Fig. 3. Simplified illustration of the ground deformational response during full-face tunneling.

reaching the tunnel stability by means of stabilization and reinforcement measures as appropriate. The full-face method, as applied for the first time in Italy (Lunardi, 2000), is indeed characterized for having developed the technologies for "protecting" and "reinforcing" this "core" ahead of the advancing tunnel face, whenever needed (in particular with the use, in weak rock, of fiberglass elements).

The transition from the confinement action due to the "core" ahead of the advancing tunnel face to that of the support along the tunnel perimeter is to take place in the most uniform and gradual way as possible, by placing the invert in the near vicinity of the face, when needed. The key component of the approach is to minimize the extrusion surface, which coincides with the tunnel perimeter and extends longitudinally, from the point of contact between the ground and the support at the crown and at the invert, respectively.

3. "Heavy" and "light" methods in full-face tunneling

In general, the major problems encountered when tunneling in squeezing rock, which is a typical difficult ground condition, are associated with the stability of the tunnel and of the face. As illustrated in Fig. 4, obtained by the author with conventional axisymmetric and plane strain models by the finite element method (FEM), a plastic zone develops in the rock mass surrounding the advancing tunnel. Depending on the rock mass properties, the wall plastic zone may interact or not with the face plastic zone.

As already noted, of significant importance for the understanding of the tunnel response are both the radial displacements of the tunnel wall and the corresponding longitudinal displacements of the tunnel face as excavation proceeds. The tunnel face follows the same deformational pattern as the tunnel itself, although the longitudinal displacements of the "core" ahead of the face are significantly smaller than the tunnel radial displacements. As shown by Hoek (2001), this is well illustrated in squeezing rock by plotting the normalized wall convergence (ε_t) and normalized axial displacement of the tunnel face (ε_f) against the ratio of rock mass strength (σ_{cm}) to in-situ stress (p_0), i.e. σ_{cm}/p_0. Note that ε_t is defined as the percentage ratio of radial tunnel wall displacement u_r to tunnel radius a and ε_f as the percentage ratio of axial face displacement u_f to tunnel radius a.

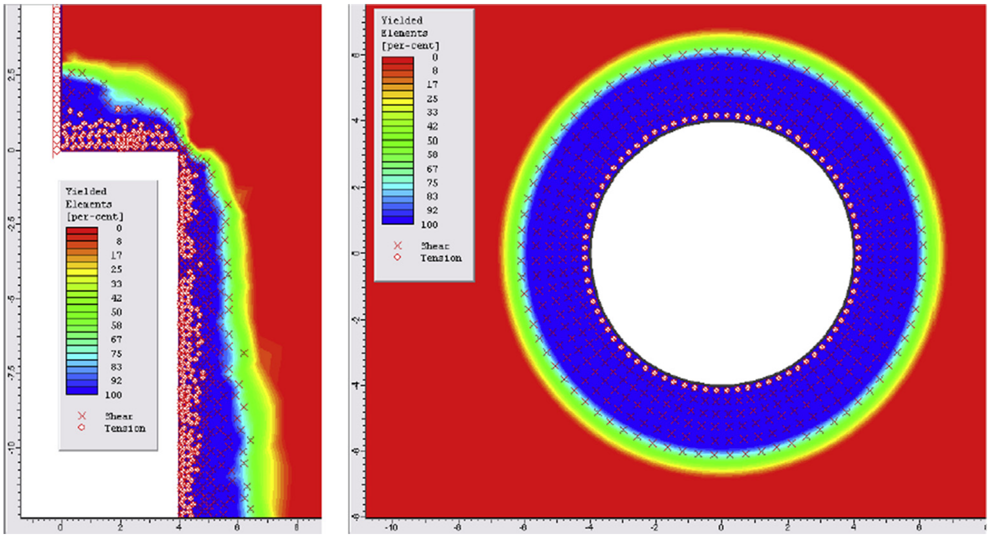

Fig. 4. Plastic zone developing around a 10 m diameter circular tunnel (Left: axisymmetric, right: plane strain FEM analyses; in-situ stress $p_0 = 5.0$ MPa, deformation modulus $E_d = 400$ MPa, rock mass strength $\sigma_{cm} = 0.6$ MPa, dilation angle $\psi = 0.0°$).

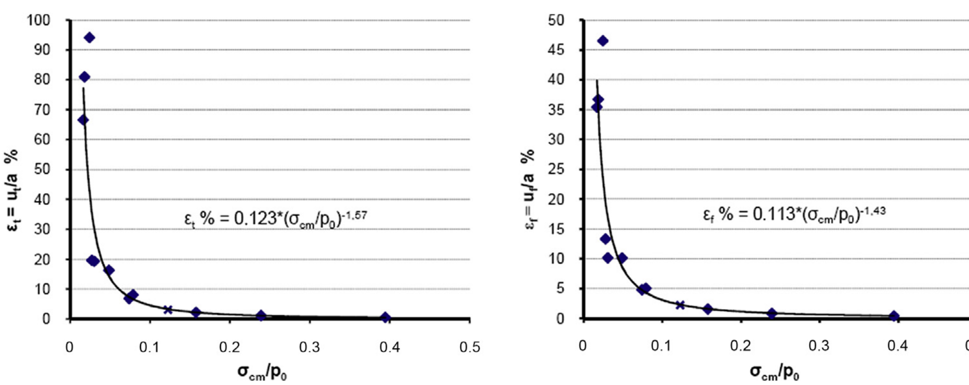

Fig. 5. Normalized wall convergence (ε_t) and normalized axial displacement of the tunnel face (ε_f) versus the ratio of rock mass strength (σ_{cm}) to in-situ stress (p_0), i.e. σ_{cm}/p_0.

Fig. 5 shows such a plot, obtained for a 10 m diameter circular tunnel at depth where the rock mass is represented as a continuum, isotropic, elastic-perfectly plastic model with a Mohr-Coulomb yield surface and by assuming a zero dilation. The results were generated with an axisymmetric FEM model under no support pressure either at the wall or at the face. It is shown that the strains ε_t and ε_f increase asymptotically when the ratio of the rock mass strength to in-situ stress is smaller than 0.2, to indicate the onset of

Fig. 6. Full-face excavation and construction method: face reinforcement and ring closure: “heavy method” (left) and “light method” (right).

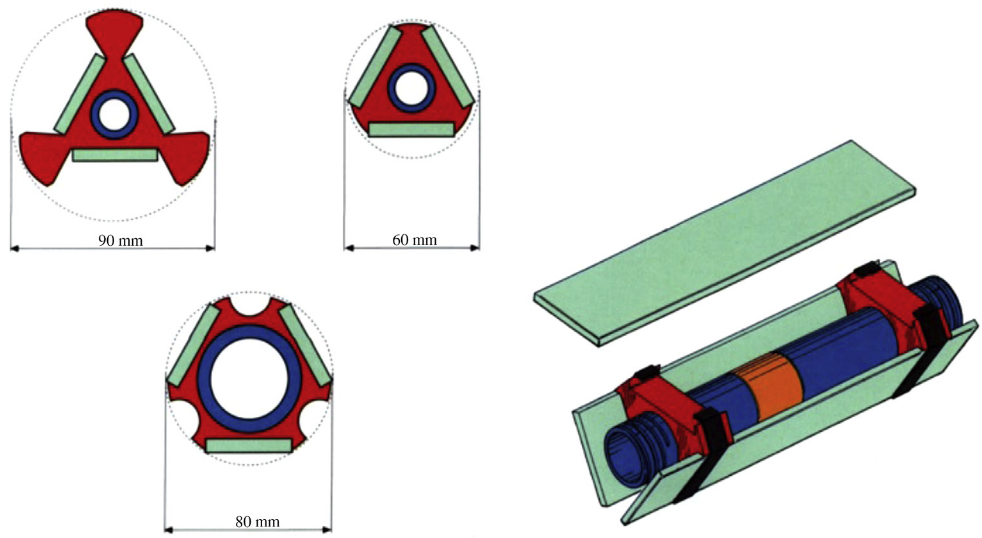

Fig. 7. Flat fiber-glass structural elements adopted for face reinforcement in the full-face excavation and construction method.

severe instability of the tunnel if no adequate support measures are implemented (Hoek, 2001).

Of the available options for conventional tunnel excavation (e.g. multiple headings, top heading and benching down, full-face), the choice falls, also in difficult conditions such as in squeezing rock, on the full-face method (Fig. 6) with a systematic reinforcement of the ground ahead of the face, e.g. by means of fiber-glass elements. However, this method, well-known from several tunnels mostly excavated in Italy (Fig. 2 and Table 1), including tunnels in difficult conditions, was applied in combination with a stiff support. This was in general possible and applied successfully due to a rather moderate overburden and when squeezing was not very severe.

With the need to cope with high overburden and in particular very severe squeezing conditions as, for example, in the Sedrun section of the Gotthard Base Tunnel in Switzerland, the proposal was made (Kovári and Ehrbar, 2008) to combine full-face excavation with a yielding support system composed of steel sets with sliding connections. This challenging combination raised questions on kinematics, stability and handling of the steel sets. Advanced structural analyses and full-scale field tests were carried out and the tunnel was completed successfully.

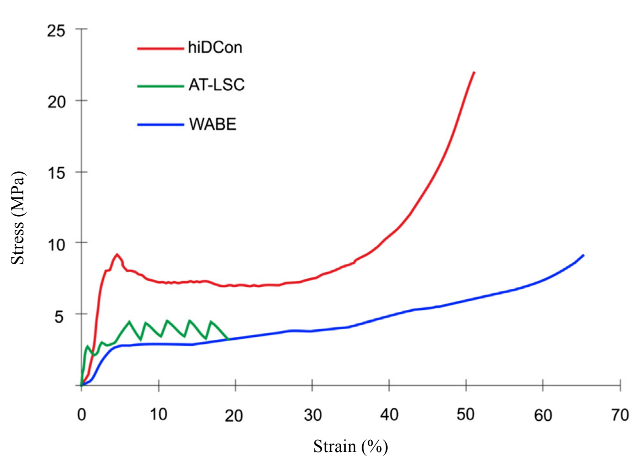

Fig. 8. Stress-strain characteristics of LSC (lining stress controller), WABE and hiDCon elements obtained in laboratory tests.

As illustrated in Fig. 6, with this combination of structural elements, one is moving (Kovári, 1998) from the "heavy method" ("resistance principle"), where only small deformations are permitted, to the "light method" ("yielding principle"), where large deformations are allowed to develop around the tunnel with the expectation that rock pressure will decrease with increasing deformations.

With the "heavy method", the primary lining is designed to be very stiff (generally composed of steel-fiber shotcrete and heavy steel sets). The tunnel cross section is entirely open and the primary lining is installed near to the face (in Fig. 6 left, which is for a typical tunnel in Italy; the "ring is closed quickly" by using a steel set as invert). 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, as in deep tunnels, this solution soon becomes impractical.

With the "light method", the excavation profile is chosen in order to maintain the desired clearance and to avoid the need for re-profiling. A key point is to be able to control the development of deformations. A suitable tunnel support system is to be adopted (in Fig. 6 right, which is referred to the Gotthard Base Tunnel, a yielding support by steel sets with sliding connections was used, including steel bolts for face support) that will allow for accommodating deformations without damage of the lining.

The face stability when driving a tunnel consists in reinforcing the rock mass ahead by means of grouted fiber-glass elements. There are a number of fiber-glass elements that may be adopted. Both smooth and corrugated tubes are available. More recently, flat elements (Fig. 7) 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. In typical reinforcement schemes, the fiber-glass elements are used to reinforce the "core" ahead of the face and in cases to provide a "reinforced ring" around the tunnel.

There are a number of options for the yielding support, in addition to providing sliding joints in the top hat steel sets as in the case of the Gotthard Base Tunnel in the Sedrun section. These include the LSC (lining stress controller) element (Schubert, 1996), the WABE honeycomb element (Moritz, 2011), and the hiDCon element (Kovári, 2008). Fig. 8 illustrates typical stress-strain characteristics of these elements. As shown in this figure, a yielding element initiates yielding at a specified stress level as it undergoes

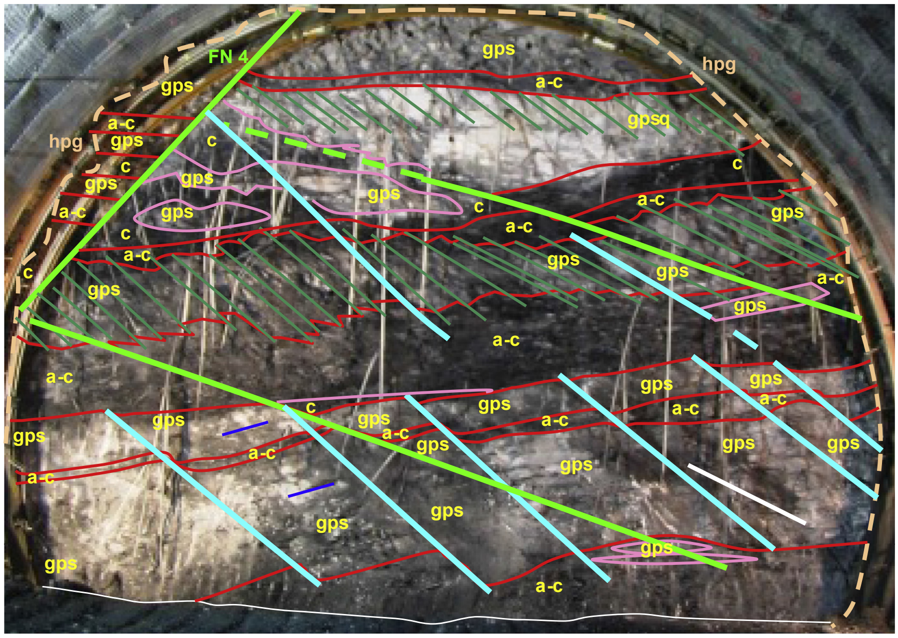

Fig. 9. Typical geological conditions at the tunnel face (gps – sandstone; a – clay-shales; c – coal, etc.).

a very small strain (smaller than 5% approximately). Deformation continues with the element length shortening significantly before the stress in the element increases.

4. Full-face tunneling in severely squeezing conditions: a case study

In order to illustrate full-face tunneling in severely squeezing conditions, a case study is discussed in the following. Reference is made to the Saint Martin La Porte access adit, along the Lyon-Turin Base Tunnel, where a yield-control support system combined with full-face excavation has been adopted successfully in order to cope with the large deformations experienced during face advance through the Carboniferous formation (see Barla, 2009; Barla et al., 2010).

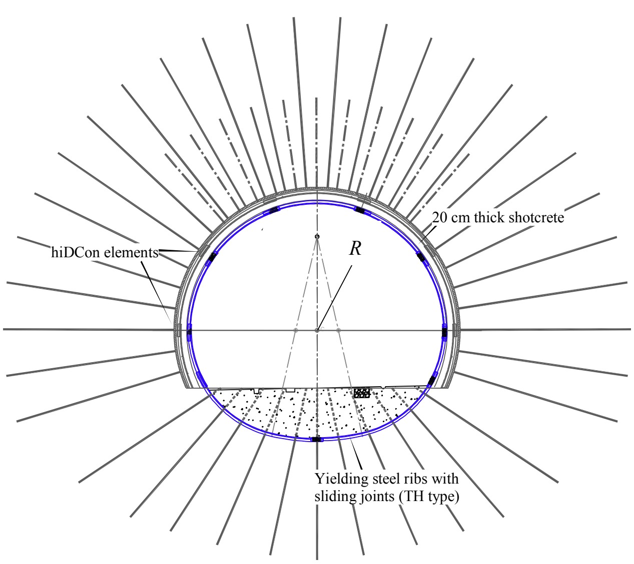

Fig. 10. Yield-control support system. Near circular cross section ($R = 6.3$ m). Stages 1 and 2 are shown. Face and ring reinforcement are not indicated.

4.1. Rock mass conditions

The rock mass encountered during excavation, as shown in Fig. 9, is a highly heterogeneous, disrupted and fractured rock mass, which is often affected by faulting that results in a significant degradation of the rock mass conditions. The overburden along the tunnel in the zone of interest ranges from 300 m to 600 m. Excavation takes place in dry conditions.

4.2. Support system

The design concept consists in the systematic use of full-face excavation and reinforcement, coupled with a yield-control support system by using a near circular cross section (radius of 6.10 m), as shown in Figs. 10 and 11. The excavation-support sequence can be summarized as follows:

(1) Stage 0: face reinforcement, including a ring of fiber-glass elements around the tunnel perimeter, over a 2–3 m thickness.
(2) Stage 1: mechanical excavation carried out in steps of 1 m in length, installation of 8 m long rock dowels along the perimeter, yielding steel sets with sliding joints, and a 10 cm thick shotcrete layer. The tunnel is excavated in the upper cross section to allow for a maximum convergence of 600 mm.
(3) Stage 2: the tunnel is opened to the full section at a 30 m distance from the face, with the application of 20 cm shotcrete lining, yielding steel sets with sliding joints fitted with hiDCon elements. The tunnel is allowed to deform in a controlled manner with a maximum convergence not to exceed 400 mm.
(4) Stage 3: installation of the final concrete lining at a distance of 80 m from the face.

As shown in Figs. 10 and 11, the most important component of such a yielding support is the hiDCon element. Nine such elements (one in the invert) are installed in slots in the shotcrete lining

Fig. 11. Left: view of the tunnel during excavation. Right: the hiDCon elements as installed.

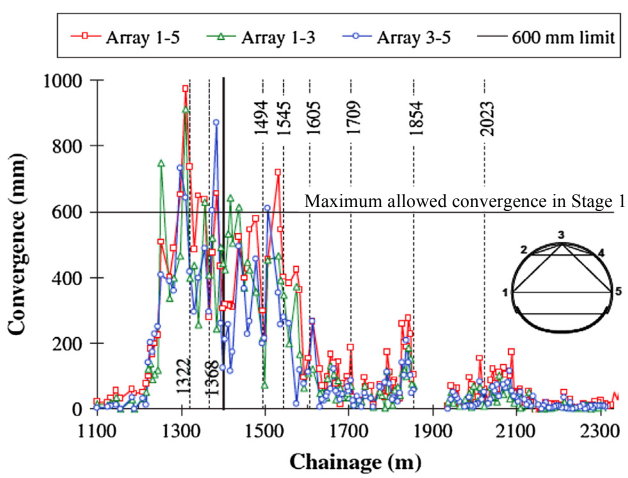

Fig. 12. Tunnel convergence versus chainage in Stage 1 along different arrays.

between the yielding type steel sets. These elements (height of 40 cm, length of 80 cm, and thickness of 20 cm) yield at approximately 40–50% strain with a yield stress of 8.5 MPa. With 9 elements installed, if each element may attain a 50% strain, the maximum allowed radial displacement ΔR is equal to 20 cm approximately, resulting in a total tunnel convergence of 40 cm. Also, if one takes a yield stress of 8.5 MPa, the radial confinement stress on the surrounding rock results to be 0.3 MPa approximately.

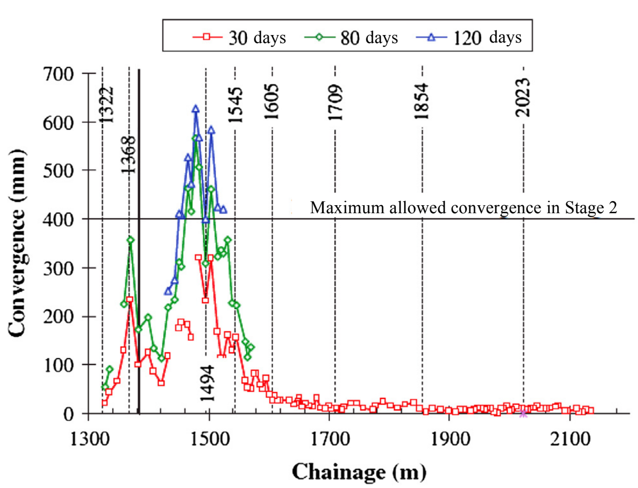

Fig. 13. Tunnel convergence versus chainage in Stage 2 along array 1–5 at 30, 80 and 120 days, following the opening of the full cross section.

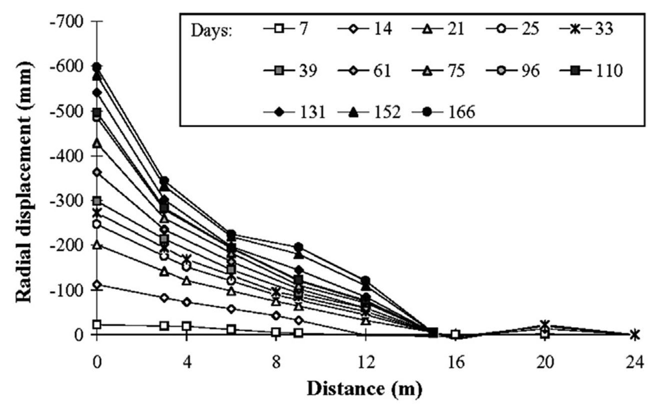

Fig. 14. Radial displacement versus distance at different times around the tunnel at chainage 1444 m.

4.3. Performance monitoring

Monitoring of tunnel convergence has been underway along the tunnel systematically. Convergences were measured by means of

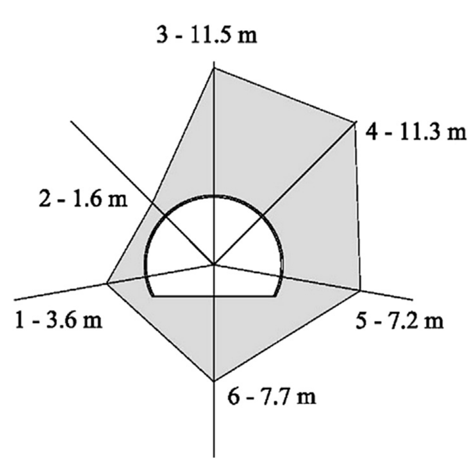

Fig. 15. Zone around the tunnel at chainage 1444 m, where the measured radial strain is greater than 1%, obtained by using multipoint extensometers.

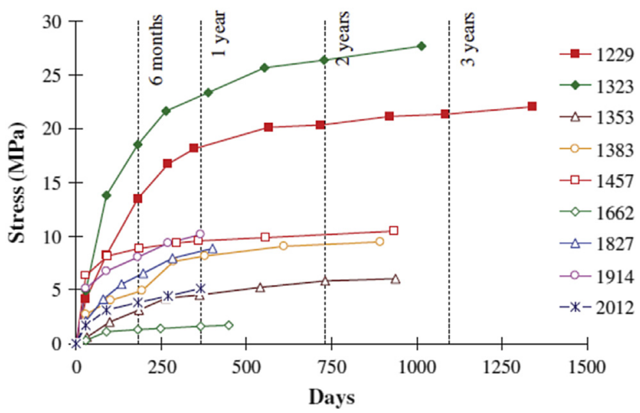

Fig. 16. Variation of tangential stress with time in the final lining at different chainages, obtained by using embedded strain meters.

optical targets placed along the tunnel perimeter. Also measured were the longitudinal displacements ahead of the face. In addition, a number of sections have been equipped with multi-position borehole extensometers and strain/stress meters in the primary and final linings. A few representative monitoring data are described as follows.

Fig. 12 shows the convergences measured in Stage 1, along different arrays (1–5, 1–3, 3–5) between chainage 1100 m and 2300 m approximately, with the tunnel face 15 m ahead of the monitoring section. Similarly, Fig. 13 gives the convergence at 30, 80 and 120 days following the opening of the full cross section in Stage 2.

One should note that the new yielding support system shown in Figs. 10 and 11 was applied systematically only following chainage 1350 m approximately. Prior to this chainage, a horseshoe tunnel section was excavated full-face with fiber-glass reinforcement. The support consisted of steel sets with yielding connections and shotcrete. Starting with chainage 1550 m, the ground conditions were improved substantially. In the sections with the new yielding support system installed, the mean tunnel normalized convergence 15 m behind the face in Stage 1 is 4% and locally never in excess of 6–7%. In Stage 2, the maximum tunnel normalized convergence is 5% around chainage 1480 m, in excess with respect to the target value of 3.5%.

Multi-position borehole extensometers were also used in order to observe the rock mass response in the ground around the tunnel. Figs. 14 and 15 show typical monitoring results for the section at chainage 1444 m. It is of interest to point out the significantly non-symmetric closure of the tunnel, due to the essentially anisotropic features of the rock mass and the presence of "strong" (sandstones and schists) and "weak" (coal and clay-like shales) "layers" which dip from the left to the right of the tunnel cross section.

Fig. 16 illustrates the maximum tangential stress in the final lining, at the sidewalls, versus time. In general, in the cross sections following chainage 1350 m, with the new yield-control support system installed, this stress is between 2 MPa and 10 MPa, with an average value equal to 7 MPa approximately. The corresponding stress in the final lining, where however excavation took place with the previous support formed with steel sets with yielding connections and shotcrete, at chainages 1229 m and 1323 m, is 22 MPa and 27 MPa, respectively. Also noted, as shown in Fig. 15, is the longer transient phase with a more pronounced stress rate value.

Table 2
Parameters for the SHELVIP model, from back-analysis of the monitoring data.

Behavior	Parameter	Description	Value
Elastic	E (GPa)	Young's modulus	0.64
	ν	Poisson's ratio	0.3
Plastic	φ (°)	Friction angle	26
	c (MPa)	Cohesion	0.56
	α_p	Volumetric tension cut-off	0.1
	ω_p	Plastic dilatancy	0.0
Viscoplastic	γ	Fluidity parameters	5.1×10^{-5}
	m	Constitutive parameter	2.2
	n	Shape factor	0.18
	l	Load dependency factor	0.01
	ω_{vp}	Time streching factor	0.735

4.4. Numerical modeling

Significant features of the tunnel response are the time-dependent deformations observed whenever face advance is stopped. Also, these time-dependent deformations do take place during excavation, when it is difficult to distinguish the "face effect" from the "time effect". In addition, laboratory tests on representative specimens show a time-dependent behavior (Debernardi, 2008; Debernardi and Barla, 2009). Therefore, in such condition modeling should consider the use of constitutive laws that account for time-dependent behavior.

Many constitutive laws can be used to describe such a behavior; however, only few of them can reproduce satisfactorily all the time-dependent features involved in tunnel excavation, with a reasonably simple mathematical formulation to be used in design practice.

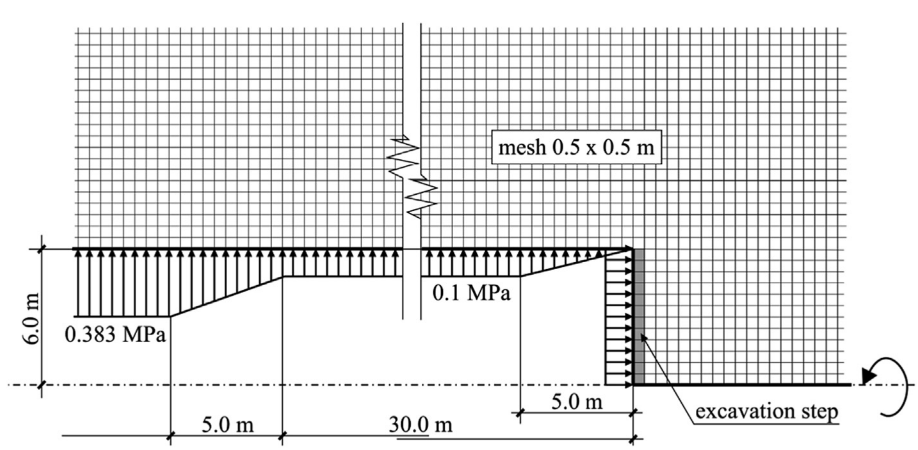

Fig. 17. The axisymmetric finite difference model.

With the phenomena observed in the Saint Martin La Porte access adit in mind, the SHELVIP (Stress Hardening ELasto VIscous Plastic) model was formulated (Debernardi, 2008; Debernardi and Barla, 2009). This model is used as follows.

With the objective to analyze the tunnel response based on the available monitoring data previously described, the axisymmetric finite difference model shown in Fig. 17 has been created with the FLAC computer code (Itasca, 2006). The tunnel cross section in the model is circular, with an equivalent radius of 6 m. The total size of the model (96–280 m) is such as to minimize boundary effects. With the overburden equal to 360 m, the in-situ stress is 9.8 MPa and isotropic. It is understood that the model created oversimplifies the observed conditions during excavation, in view of the significantly non-symmetric closure of the tunnel as shown in Fig. 15.

As shown in Fig. 17, an equivalent pressure of 0.1 MPa applied to the face represents the ground reinforcement ahead of the face. An equivalent radial pressure equal to 0.1 MPa, 5 m behind the tunnel face, represents the radial reinforcement and the first stage lining (10 cm thick). An additional internal pressure 30 m behind the face, which reaches the constant value of 0.383 MPa in a 5 m span, simulates the influence of the second stage lining. This value accounts for the yielding strength of the yielding elements, as known from laboratory tests. The assumed excavation rate is 0.54 m/d.

Back-analysis carried out with the model has led to the values of the constitutive parameters listed in Table 2. Fig. 18 shows the comparison of computed and measured radial displacements for the tunnel section at chainage 1444 m. The numerical results with the mean curve agree satisfactorily with the monitoring data, notwithstanding the scattering due to the high heterogeneity and anisotropy of the rock mass. Also the displacements around the tunnel monitored with the multi-position borehole extensometers are satisfactorily reproduced, as shown in Fig. 19.

5. Conclusions

This paper presented the state-of-the-art in full-face tunneling in difficult conditions, when the tunnel stability relies heavily on the ground reinforcement ahead of the tunnel face (the so-called "core") and, in cases, on a ring around the tunnel. The developments of the method, with the primary lining designed to be very stiff (generally composed of steel fiber shotcrete and heavy steel sets), as initially applied mostly in Italy, were mentioned. Then, the early applications of full-face tunneling under difficult conditions by using a yielding support system were described.

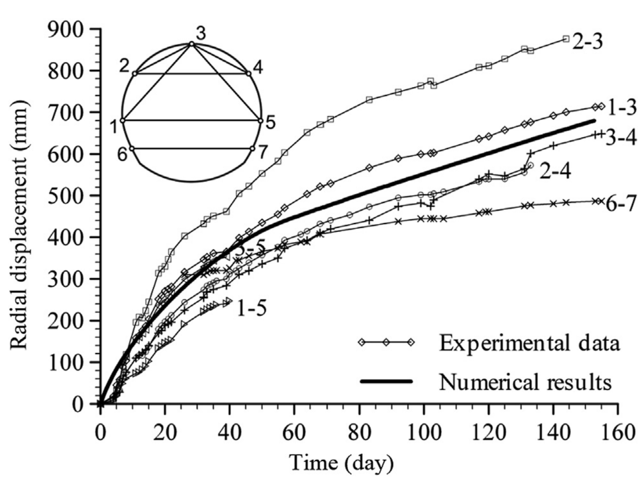

Fig. 18. Computed versus monitored convergences at chainage 1444 m.

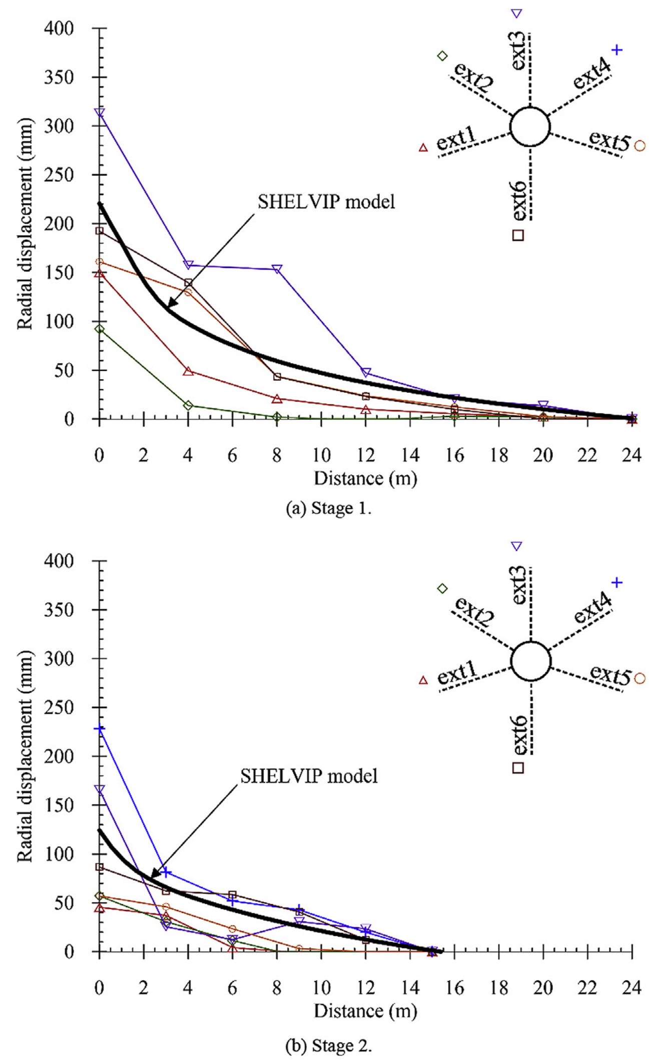

Fig. 19. Computed versus monitored radial displacements around the tunnel at stages 1 (a) and 2 (b), chainage 1444 m.

The well-known case study of the Saint Martin La Porte access adit, along the Lyon-Turin Base Tunnel, which experienced severely squeezing conditions during excavation in the Carboniferous formation, was taken as representative. In this tunnel, which has been widely studied by the author, an innovative tunnel excavation and construction method was introduced which combines full-face excavation and face reinforcement by means of fiber-glass elements with a yield-control support. The results of in-situ performance monitoring and numerical modeling with consideration of the rock mass time-dependent behavior were illustrated.

Conflict of interest

The author wishes 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.

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