A Consistent Approach to the Structural Rehabilitation of Tunnel Liners

Livio Locatelli, Giorgio Borgonovo
Golder Associates, Milan, Italy

Giancarlo Gioda
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

ABSTRACT: The concrete liners of existing tunnels could present relevant damages due to their ageing or weathering, or to improper construction techniques. In some cases, depending on the geological conditions, these faults are due to the overstress caused e.g. by the occurrence of landslides, by swelling or creep of the rock mass, etc. Whilst two-dimensional numerical models are in most cases sufficient to assess the stress-strain state within the liner, a proper analysis of its replacement should be based on three-dimensional calculations. They, in fact, permit evaluating the need of soil/rock treatments around the tunnel and the length of the tunnel portions in which the liner demolition and re-construction can be safely carried out. In addition, they are necessary for a proper design of the new reinforced concrete support. Three level of tunnel rehabilitation are discussed, ranging from light repairs of the inner surface of the liner to heavy structural changes involving its demolition and reconstruction. Some case histories related to highway tunnels are also presented that illustrates the mentioned levels of rehabilitation.

1 INTRODUCTION

A relatively large number of tunnels were built in Italy during the sixties for the development of the national highway system. Many of them presently show some damages of their concrete linings caused by a variety of phenomena, such as ageing, freezing/thawing, chloride penetration, carbonation, or by poor construction techniques [1]. In a few specific cases the damages are related to geological conditions and to the consequent slow time dependent movements of the soil/rock mass surrounding the tunnel.

The steady increase of traffic and the recently introduced new standards make it necessary to improve both quality and safety of the highway network. To this aim, an adequate knowledge of the present conditions of the liners is necessary. Such fundamental information is obtained through an extensive investigation that was planed in the framework of the standard maintenance works.

Here the attention is focused on highway A15 (Autocamionale della Cisa) located in the Apennine mountain chain in central Italy (Fig. 1). The highway is approximately 100 km long and about 15 km of it consist of two-lane twin tunnels.

The state of these tunnels is periodically checked in order to assess possible structural or geological risks. To this purpose visual inspection and hammering are carried out first. If they reveal the presence of damages, drillings and core samplings are performed for a more detailed in-depth investigation.

In order to create a geo-referenced database the observed defects are recorded on a GIS based system that allows for the real time update and control of each repair works and scheduled maintenance. It should be pointed out, in fact, that the continuous monitoring of all relevant damages in the framework of a pre-defined maintenance schedule is crucial for ensuring an adequate safety of the users [2].

Fig. 1. Map with the location of highway A15 (Autocamionale della Cisa).

The extensive inspections carried out so far on the tunnels of A15 showed that the most frequent problems do not necessarily depend on significant geological causes. In fact, they often derive from the limited quality of the materials used for the supporting structures or from poor construction methods.

In spite of this, if relevant damages are observed, the geological conditions of the surrounding soil/rock mass are carefully assessed and investigated. This permits identifying possible geological causes of the damage, like e.g. the occurrence of shallow or deeply seated landslides, or squeezing and creep phenomena.

The typical faults that have been found in the majority of the investigated concrete linings are:

- Honeycombed concrete or poor concrete quality,
- Presence of voids and gaps between the permanent lining and the temporary support,
- Corroded re-bars exposed to the atmosphere,
- Gaps between adjacent concrete castings,
- Presence of cracks and fissures,
- Plugged or damaged drains,
- Water leakage from joints or fissures.

Depending on the level of damage, and on its structural or geological origin, proper provisions are adopted for restoring or replacing the liner. Their design is strictly related to the actual conditions of the concrete and of the surrounding rock mass [3,4,5].

The following Sections discuss the design approach and the techniques adopted for rehabilitation.

To this purpose, three levels of damage and rehabilitation are identified, namely: light, intermediate and heavy. Their salient characteristics are illustrated on a basis of the five years works, still in progress, on the tunnels of the mentioned A15.

2 LEVELS OF LINER REHABILITATION

The mentioned three levels of rehabilitation are described here considering, in particular, the corresponding damages and the works required for the restoring the structural safety of the liner.

2.1. Light rehabilitation

In general terms, a light or "spot" rehabilitation should be considered when the defects are localised within limited zones and mainly involve the inner surface of the lining. In this case the repair works need to be both quick, to minimise the time during which the tunnel will be closed to traffic, and safe. Quite obviously, they should involve a limited overall cost.

The techniques most frequently adopted can be grouped as follows:

- Installation of corrugated steel sheeting secured by nails or small rock bolts in case of limited damaged areas or minor water infiltration.
- Installation of steel sheeting plates secured by anchors or rock bolts in case of wider zones with voids or honeycombed concrete. If the presence of large voids prevents the use of anchors, steel beams secured at their ends to stable rock zones can provide the necessary support.
- The use of exceedingly long anchors would be necessary if the radial extension of the damaged or "decompressed" rock around the tunnel is particularly large.

2.2. Intermediate rehabilitation

The intermediate level involves the general rehabilitation of the entire internal surface of the lining, and the localised restoration of some concrete sections.

In case of extensive damages the cost/benefit analysis may show that the most convenient procedure consists in the general restoration of the inner surface of the liner along the entire length of the tunnel. Hydro-demolition is used in this case to remove a predefined thickness the damaged support. Subsequently a layer of welded wire mesh is installed over entire surface and, finally, a rapid-set dry-mix shotcrete is applied on it.

2.3. Heavy rehabilitation

Heavy rehabilitation is necessary when particularly serious defects, involving large portions of the liner, are detected during inspection. As previously mentioned, they can derive from poor construction techniques or from high geological stresses. Quite often in such cases the tunnel lining needs to be completely demolished and reconstructed, either locally or over its entire length.

It should be considered that the time necessary for rehabilitation would conflict with the highway requirements, which do not permit closing the tunnel to traffic during holiday seasons. To avoid this drawback it is preferable to install steel ribs inside the lining as a temporary safety provision, although this slightly reduces the inner section of the tunnel. Once the risk of damages to the traffic has been minimized, the design of the permanent rehabilitation can develop with proper organization and scheduling.

3 DESIGN APPROACH

Having defined the extension and the characteristics of the damage, and its structural or geological causes, it is necessary to adopt a proper design approach for the structural restoration of the liner.

The first necessary step consists in the evaluation of the state of stress within the existing support in order to assess its safety level with respect to the rock pressure.

This evaluation requires a sufficient knowledge of the mechanical characteristics of the rock mass that are determined through a proper in situ and laboratory investigation. In addition it is necessary to choose a suitable numerical model for analyzing the rock-structure interaction problem.

To support the result of analyses it is advisable to perform also a direct measurement of the stresses within the liner, e.g. by means of flat jack tests.

If the level of damage is limited, the calculations can be based on relatively simple models, like the well-known convergence-confinement method.

In more complex conditions recourse should be made to numerical analyses, based e.g. on elastic-plastic finite element programs.

The correct evaluation of the present stress distribution would require the simulation of the actual excavation and construction processes of the tunnel. In most cases, however, quite limited records nowadays remain of these aspects. Consequently the calculations are often confined to a plane-strain scheme of the opening section in which the excavation/construction steps are modeled in a simplified manner.

If the observed damages are not severe, and the results of calculations and in situ stress measurements show that the present liner can safely bear the rock pressure, a light or intermediate rehabilitation should be chosen.

On the contrary, if one of the above conditions is not fulfilled a heavy structural restoration is necessary. A series of relevant problems shows up in this case that concern, in particular, the quantitative evaluation of:

- The maximum length of the liner segment that can be safely demolished and re-constructed.
- The need of rock treatments before demolition.
- The structural characteristics of the new liner.

Note that the increase of length of the segment to be demolished in general reduces the costs and decreases the safety of the works. This points out that the choice of this length is a crucial aspect of design.

The above quantities can be determined through proper numerical analyses that simulate with sufficient details the rehabilitation works. These calculations, however, cannot be based on the relatively simple two-dimensional model used for the previous stress evaluation. In fact, the demolition and reconstruction process has an intrinsic three-dimensional nature that cannot be disregarded.

To overcome this problem, the previously developed two-dimensional mesh can be easily "extended" in the direction of the tunnel axis to obtain the necessary three-dimensional finite element grid. Possible changes of the tunnel section and of its depth of cover along its length, as well as possible treatments of the rock mass (e.g. tiebacks, grouting, etc.), can be easily introduced in this process.

The "initial" stress state derived from the two-dimensional calculations is applied to the three-dimensional mesh.

The three-dimensional grid is then used for the elasto-plastic analysis of the demolition of a liner segment. The segment length is modified in subsequent analyses evaluating the consequent spreading of plastic strains in the surrounding rock and the increment of stresses in the adjacent portions of the lining.

The critical evaluation of these results permits choosing the maximum length of the segment that can be demolished ensuring adequate safety condition.

On this basis the calculations proceed simulating the construction of the new segment and the subsequent demolition and construction of the segments adjacent to it.

The calculations lead to the stress state in the new concrete liner and to the extension of the plastic zone in the surrounding rock mass. This in turn permits evaluating whether the assumed thickness and the steel reinforcement of the new lining, and the rock treatments, are adequate to carry the pressure exerted by the rock mass.

Three applications of the described design approach are illustrated in the next Section. They concern some damaged tunnels of highway A15 in which different levels of rehabilitation have been adopted.

4 APPLICATIONS

4.1. Light rehabilitation of Casacca tunnel

The in situ investigation on the 350 m long Casacca twin tunnels, initiated in June 1996, led to the quantitative evaluation of the thickness of the lining and of the stress distribution within it. It appeared that the stresses in the lining did not depend on an excessive thrust from the surrounding rock. However, numerous voids were found above the crown, which was already supported by steel ribs and shotcrete since the tunnel construction. The limited thickness of the lining is likely to be the cause for the stress concentration measured at some locations.

A monitoring system, based on electronic vibrating wire crackmeters, was also installed to monitor the possible variation in opening of the observed cracks. However, no relevant changes were recorded over a 3-year span, from 1996 to 1999.

Based on this evidence, the liner rehabilitation initiated in 1999.

The design focussed on the locations where the lining presents insufficient crown thickness or its concrete is highly stressed and fissured. Before proceeding with hydro-demolition, the existing concrete liner was bolted to the surrounding rock mass (using Feb44k steel threaded re-bars 4 to 5 m long) to guarantee safety and to prevent further cracking.

The construction developed through the following phases:

<u>*Phase 1: remediation and protection*</u>

1a) Installation of rock bolts in those crown portions having insufficient thickness and where the concrete characteristics are below acceptable limits (cf. Fig. 2).

1b) Injection of cellular concrete mix, through holes in the existing lining, to fill the gaps between it and the rock. This lightweight material was used to obtain an adequate adhesion between concrete and rock without adding excessive loads to the lining.

<u>*Phase 2: hydro-demolition and rehabilitation*</u>

2a) Hydro-demolition of the entire inner surface, to a minimum depth of 3.5 cm, removal of damaged water drains and demolition of honeycombed concrete areas.

2b) Installation of permanent water drains conveying the leakages from the crown down to the sidewalls and from there to longitudinal drains.

2c) Smoothing of tunnel walls using dry-mix shotcrete and sealing of the drains.

2d) Installation of a single layer of welded wire mesh over the entire tunnel surface and of a double layer where the rock bolts have been installed.

2e) Final application of rapid-set, dry-mix shotcrete over the whole surface.

2f) Further hand smoothing of the sidewalls, up to a height of 4 m, using dry-mix shotcrete with high workability.

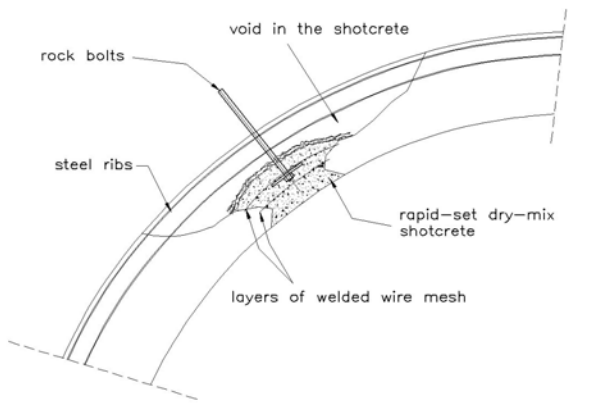

Fig. 2. Details of the remediation works of Casacca tunnel.

<u>*Pase 3: finishing*</u>

3a) Installation of the sidewalks and of longitudinal water collectors along the two sides of the tunnel.

3b) Spraying of fibre-reinforced high resistance white cement to create a smooth and brilliant surface.

3c) Rehabilitation of the tunnel portals with high resistance paint coating.

The restoration of south tunnel was completed in the year 2000, and in year 2001 for north tunnel. Presently Casacca twin tunnels are fully functional as shown in Fig. 3.

As to the mentioned phases, the hydro-demolition requires particular care and careful control to avoid the removal of an excessive thickness of concrete. Particular attention should be also used in positioning the welded wire mesh. The subsequent application of dry-mix shotcrete requires specialised and skilled workmanship to obtain a homogeneous layer with a smooth surface. The white concrete applied up to a height of 4 m on the tunnel sides both increases visibility and is easily maintained by washing.

The described rehabilitation, that involves the complete overhaul of the lining with structural reinforcements, where necessary, and the overall enhancement of its appearance, can be adopted for tunnels up to 500 m in length. Longer tunnels should be rehabilitated in sections. In fact, they can be closed during a low-traffic period, then the rehabilitation of one or their sections can be completed and the tunnels can be reopened temporarily to accommodate heavy holiday traffic. The subsequent sections can then be completed during the next low-traffic period.

Fig. 3. Casacca tunnel after the rehabilitation of its lining.

4.2. *Intermediate rehabilitation of Puntamonte tunnel*

The Puntamonte twin tunnels have an elevation of about 500 m s.l.m. and cross the Apennine chain in the NE-SW direction.

A severely weathered zone was detected during the planned inspections in the vicinity of the north entrance. In fact a liner portion having length of about 10 m contains 7-8 zones of honeycomb concrete with large gaps between them and the rock mass. The area of the largest zone reaches 8-9 m^2.

A temporary support was built up first to allow safe working conditions during the subsequent rehabilitation. It consisted of radial rock bolts 4 m length, with steel plates that support, by means of a narrow welded wire mesh, the damaged lining and protect from localized falls (Fig. 4).

The necessary permanent restoration was then subdivided into five phases. It consisted in the construction of a permanent concrete shell at the tunnel crown that supports the damaged liner.

The first and second phases involve, respectively, the construction of the shell foundation underneath the previously protected area and the hydro-demolition of the inner surface of the liner for a depth varying between 4 and 10 cm, without damaging or removing the temporary support.

Once the demolition is completed, two layers of welded wire mesh are set in place (third phase) at a distance of 2 and 9 cm, respectively, from the previously installed steel plates. Rock bolts and steel bars are also added for strengthening the support of the subsequent concrete shell.

Shotcrete is then sprayed on the steel mesh up to a thickness of 7.5 cm, that increases to 9 cm in the vicinity of the bolt plates. The outer wire mesh should not be completely covered during this process to allow for the subsequent surface finishing. After hardening, the one dimensional compression strength of the shotcrete layer should reach 35 MPa.

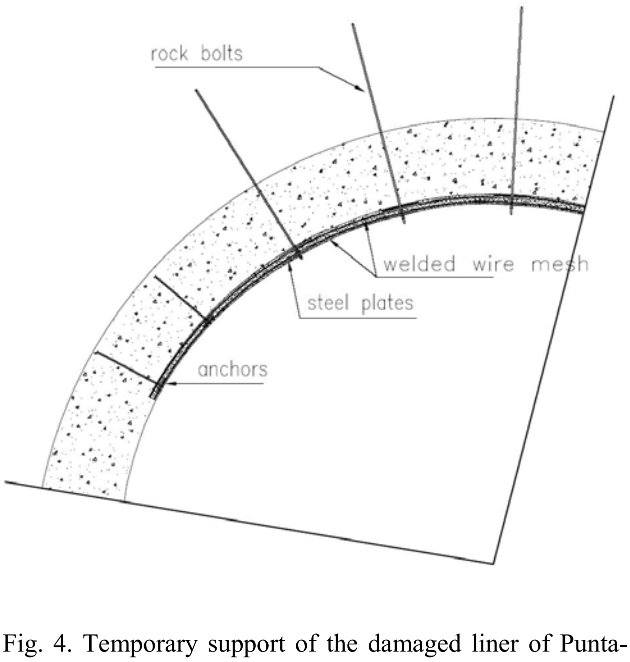

Fig. 4. Temporary support of the damaged liner of Puntamonte tunnel.

The final fifth phase consists in further shotcrete sprayings. The completed concrete shell should have a total thickness from 10 to 12 cm, and at least 2 to 3 cm of shotcrere should cover the outer steel mesh and the steel reinforcements.

As previously observed the design of this rehabilitation can be based on two-dimensional analyses. Following this procedure the stresses within the rock bolts and the concrete shell were evaluated through linear elastic finite element analyses subdividing the structural members into one-dimensional truss and beam elements. Different discretizations were adopted depending on the liner section. Two of them are shown in Fig. 5.

The length of each section along the tunnel axis coincides with the distance between two subsequent rows of anchors. The interaction between adjacent sections and the possible contribution of the old lining were disregarded in the calculations.

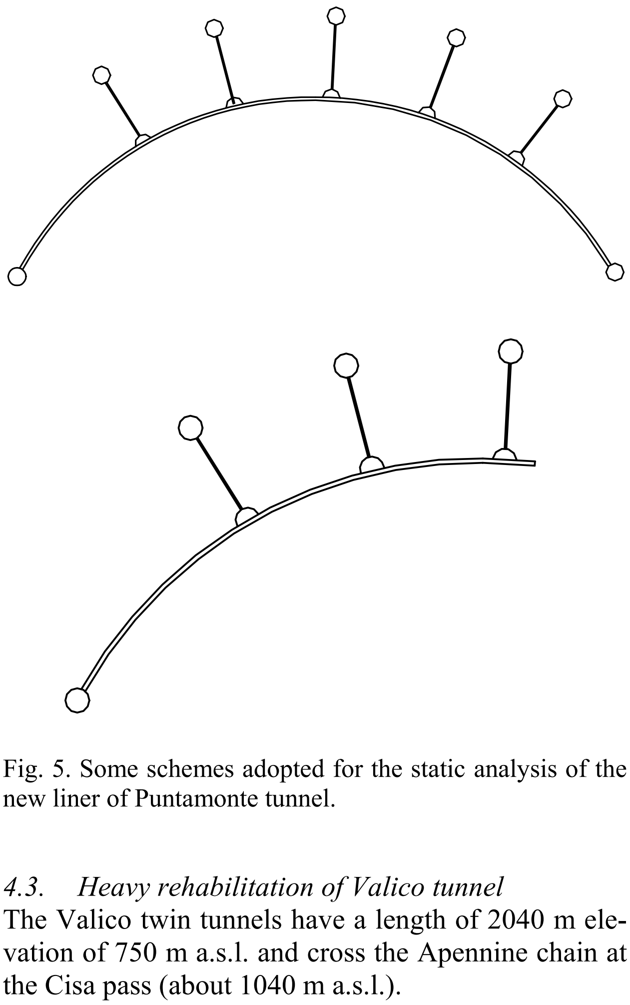

Fig. 5. Some schemes adopted for the static analysis of the new liner of Puntamonte tunnel.

4.3. Heavy rehabilitation of Valico tunnel

The Valico twin tunnels have a length of 2040 m elevation of 750 m a.s.l. and cross the Apennine chain at the Cisa pass (about 1040 m a.s.l.).

During the eighties, transversal and longitudinal cracks developed (up to 50 mm wide) in the concrete linings in the vicinity of the north entrance of the tunnels. The severity of damages suggested the heavy rehabilitation of a relevant portion of the tunnels.

The steps carried out to this purpose can be summarized as follows:

- Geomorphologic and geological study of the area.
- Geotechnical in situ investigation and laboratory tests on recovered samples.
- Evaluation of the effects of the demolition and reconstruction of the liner through two- and three-dimensional elasto-plastic finite elements analyses.
- Detailed definition of all executive phases.
- Verification of the new structural support.

The geological, geomorphologic and geotechnical campaigns included borings and SPTs up to 33 m depth and characterized the formations present in the slope, namely: sandstone, shale and flisch.

The geological section shows a 15 m thick weak layer, of tectonic origin, at 11 m depth. This weak layer crossing the tunnel produced differential displacements and tensile stresses in the concrete linings that caused non-negligible damage to both tunnels.

Two geological sections corresponding to the damaged zones are shown in Fig. 6.

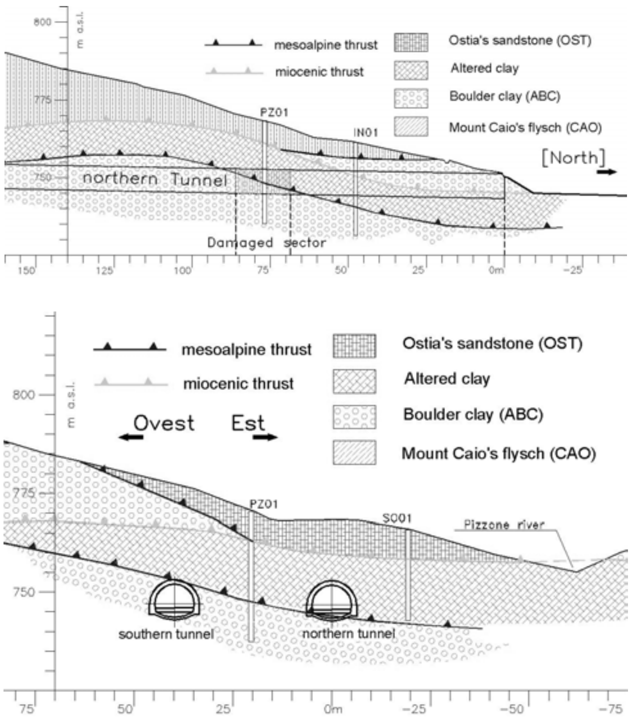

Fig.6. Geological sections of Valico tunnels at their damaged portions.

Piezometers and inclinometers were also installed to record the slope movements and the fluctuations of the water table.

Laboratory tests, that included one dimensional compression tests and direct shear tests, were performed on the recovered samples to define their geotecnical properties.

The discussion is limited here to the northern tunnel (cf. Fig 6) that presents a heavily damaged portion, about 35 m in length, and two moderately damaged sectors, 10 m in length, on each side. The depth of cover of this tunnel ranges between 15 and 23 m.

The poor state of the permanent lining and the presence of fracture and of large gaps between it and the temporary support suggested the demolition and reconstruction of its damaged portion.

According to the design approach previously outlined, two-dimensional plane strain analyses were first performed to evaluate the stress and strain states in the old lining and in the surrounding rock mass. To this purpose the two-dimensional mesh shown in Fig. 7 was adopted that includes both tunnels and a sufficient portion of the slope around them.

The plane strain analysis was subdivided into a number of steps. First, the in situ stress state prior to the excavation of the tunnels was estimated by "activating" the elements of the mesh (subdivided into 12 layers) from bottom to top. The own weight of the corresponding activated elements was applied at each step.

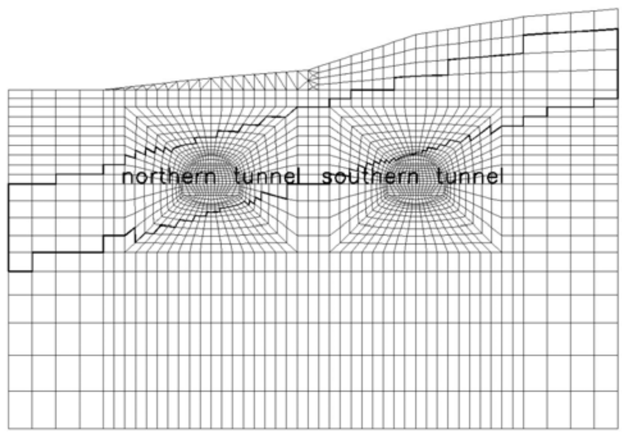

Fig.7. Finite element mesh for the two dimensional analyses of Valico tunnels.

The excavation of the two tunnels was subsequently simulated. The stress state within the element to be excavated was converted into equivalent nodal forces. Then these elements were eliminated from the mesh and the previously evaluated excavations forces were applied to it. The consequent stress/strain variation was evaluated through an elasto-plastic analysis. The installation of the temporary support (steel ribs and shotcrete) was taken into account by introducing equivalent beam elements.

The elasto-plastic calculations have been carried out by means of program FEARSM, specifically developed for the Finite Element Analysis of Rock/Soil Masses [6,7,8].

Since, according to the available information, the two tunnels were excavated simultaneously, the entire analysis was subdivided into the following steps:

0) Evaluation of the in situ stress state.
1) Removal of the elements of the crowns of the two tunnels and application of 30% of the excavation forces.
2) Application of additional 30% of the excavation forces and activation of the steel ribs at the crowns.
3) Application of additional 40% of the excavation forces and activation of the crown shotcrete.
4) Removal of the spring-line elements and application of 30% of the excavation forces.
5) Application of additional 30% of the excavation forces and activation of the steel ribs at the spring-lines.
6) Removal of the invert-arch elements, application of 50% of the corresponding excavation forces and of the remaining 40% of the excavation forces at the spring-lines, activation of the spring-line shotcrete.
7) Application of the remaining 50% of the excavation forces at the invert-arch and activation of the elements of the invert arch.

The spreading of the "plastic zone" for the two-dimensional analysis is shown through the contour lines of the second invariant of the deviatoric plastic strains in Fig. 8.

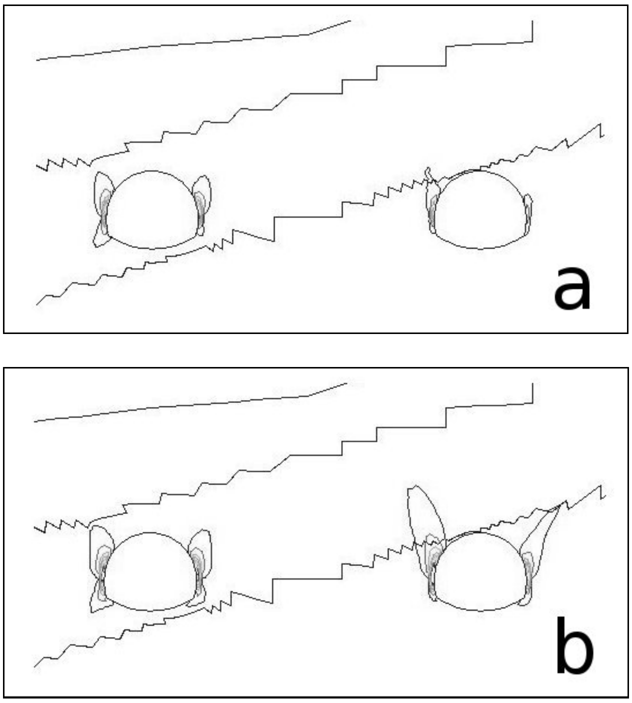

Fig.8. Contour lines of the second invariant of the deviatoric plastic strains at the end of steps 3 (a) and 7 (b).

The calculations show that the rock mass in the vicinity of the northern tunnel is subjected to plastic strains higher than those developing close to the southern tunnel. This is perhaps one of causes of the severe damages present in the lining of the northern tunnel.

Having estimated the stress state prior to the rehabilitation works, the analysis proceeds with the evaluation of the additional stresses induced by the demolition of the old liner. This was based on the three-dimensional mesh depicted in Fig. 9. The rock was subdivided into eight node, isoparametric brick elements, whilst thick shell elements were adopted for the liner.

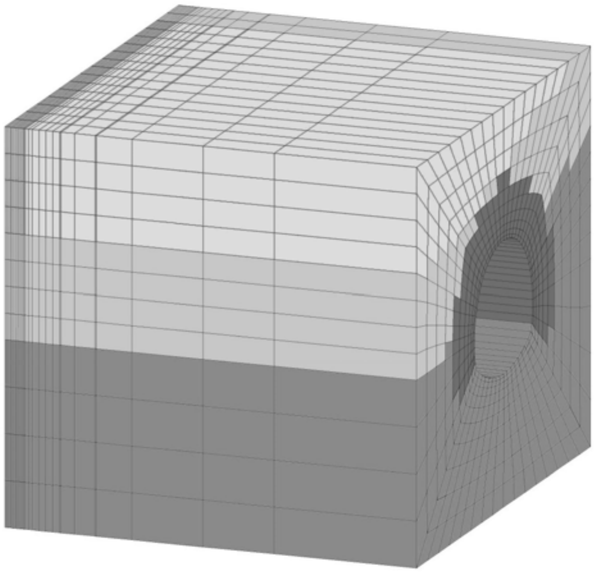

Fig.9. Finite element mesh for the three dimensional analyses of the northern tunnel.

Considering the distance existing between northern and southern tunnels, and the limited length of the liner segments to be demolished and reconstructed in sequence, it was assumed that the interaction between the two tunnels has a negligible influence on the rehabilitation process. As a consequence the mesh was limited to the northern tunnel only. In particular, the finite element grid was refined in a zone that includes three adjacent segments subjected to the works. Because of the problem symmetry only half of the central segment was introduced in the mesh.

In order to define the correct length of these segments, a series of elasto-platic analyses was performed adopting different values for this key parameter.

The analyses were also repeated introducing "improved" mechanical properties for the rock mass surrounding the tunnel, thus taking into account the effect of the rock treatments that will be subsequently described.

The steps of the three-dimensional analyses can be summarized as follows:

0) Application of the initial stress state obtained from the two-dimensional calculations.
1) Removal of the upper portion of the central segment of the old liner.
2) Removal of the lower portion of the central segment of the old liner.
3) Activation of the new liner for the entire central segment and removal of the upper portion of the lateral segments of the old liner.
4) Removal of the lower portion of the lateral segments of the old liner.
5) Activation of the new liner for the entire lateral segments and reduction of the "improved" properties of the rock mass to their standard values.

Note that step 5 leads to the "long term" stresses in the new liner, assuming that the treatment of the rock mass looses its effects with time.

The observed movements of the slope are likely to continue during time. This would certainly induce fractures in the new lining if a continuous reinforced concrete support were installed. To avoid this negative effect, suitable joints are installed between the segments of the new lining to limit the shear stresses exchanged through them. To account for this provision the shear interaction between central and lateral segments was eliminated in the calculations.

Some of the results of analyses are summarized in Figs. 10 to 12. The final distributions of the second invariant of the deviatoric plastic strains are shown in Fig. 10 for the case without rock treatments and in Fig. 11 for the case with rock treatments.

The final distributions of the bending moment in the new liner are shown in Fig. 12a,b.

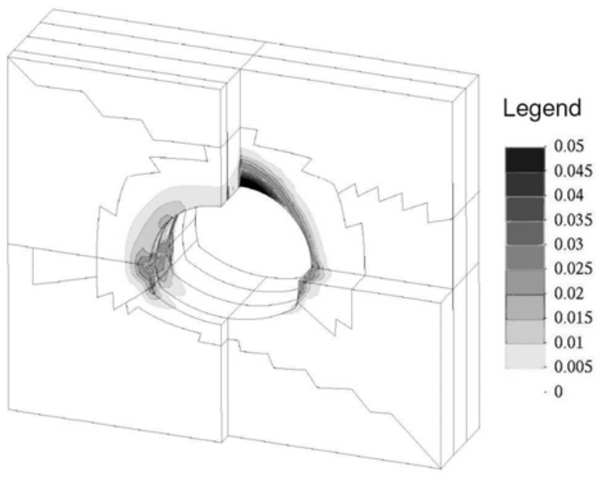

Fig. 10. Distribution of the second invariant of the devistoric plastic strains at the end of the tunnel rehabilitation (without rock treatments).

Based on the above investigation and finite element analyses a procedure was developed for the replacement of the damaged liner. The works, initiated in November 2003, consists of the following phases:

1) Removal of the temporary steel ribs and check of the concrete conditions in the vicinity of the main cracks. Removal of the roadway to expose the invert arch.

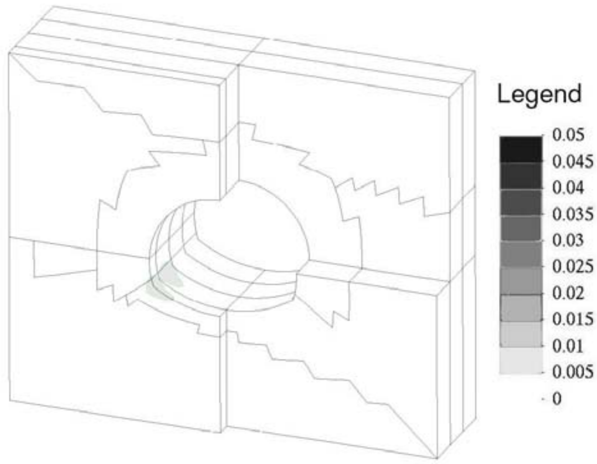

Fig. 11. Distribution of the second invariant of the devistoric plastic strains at the end of the tunnel rehabilitation (with rock treatments).

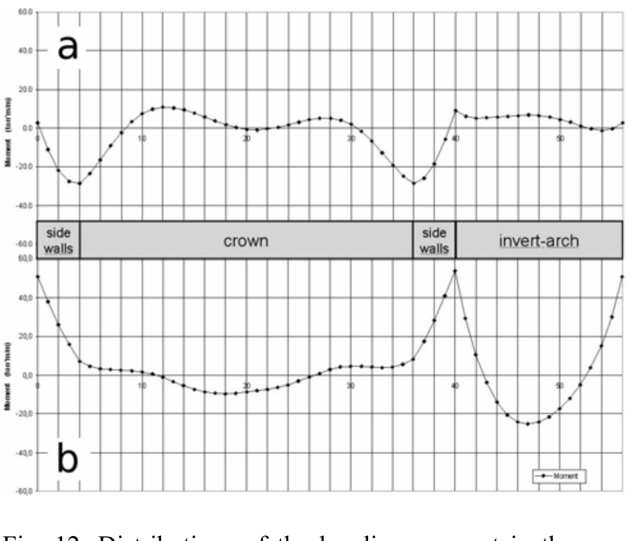

Fig. 12. Distributions of the bending moment in the new liner: a) with rock treatments; b) without rock treatments.

2) Filling of the gaps between lining and rock with cellular concrete.
3) Radial treatment of the rock surrounding the crown with sleeved steel micropiles (Fig. 13).

4) Demolition of limited portions (3.5 m long) of the invert arch, in order to minimise the effect on the old lining, and their reconstruction.
5) Removal and reconstruction of sidewalls and crown. This phase is subdivided into steps as follows: demolition of a first portion (1.30 m in length); installation of 4 coupled IPN180 steel ribs; demolition of a second portion (1.20 m) and installation of a third couple of steel ribs; reconstruction of the whole section. These steps are repeated in sequence until completion of the heavy damaged stretch of 35m.
6) Partial demolition and reconstruction of 5 m portions of the liner on each side of the heavily damaged sector. Demolition is performed by means of a head-scraper removing a 50 cm thick layer of the old liner.
7) Additional hydro-demolition of the previous 5 m portions and installation of layers of welded wire mesh over entire surface. Finally, application of a rapid-set dry-mix shotcrete over the whole surface.

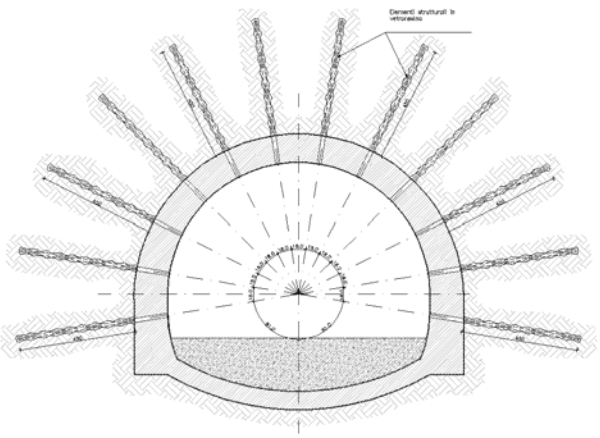

Fig. 12. Micropiles adopted for improving the strength of the rock mass surrounding Valico tunnels.

5 CONCLUDING REMARKS

A variety of structural causes, such as the ageing of concrete, freeze-thaw cycles, chloride penetration and carbonation, or inadequate construction techniques, could produce during time relevant damages to the permanent lining of tunnels. In some cases the damages have a geological cause, like the occurrence of landslides or the swelling/squeezing behaviour of the rock mass.

The continuous monitoring of the tunnels, in the framework of a pre-defined maintenance schedule, permits detecting the onset of possible faults in their liners and is crucial for ensuring an adequate safety of the users.

Depending on the observed level of damage, and on its origin, proper provisions should be adopted for restoring or replacing the liner. Their design is strictly related to the actual conditions of the concrete and of the surrounding rock mass.

A consistent design approach has been presented here that subdivides the problem into three levels of increasing complexity and cost.

The first level, referred to as light or “spot” rehabilitation, should be considered when the defects are localised within limited zones and mainly involve the inner surface of the lining.

The second, intermediate, level involves the general rehabilitation of the entire internal surface of the lining, and possible localised restoration of some concrete sections.

A heavy (third level) rehabilitation is necessary when particularly severe faults are observed that involve large portions of the liner. Quite often in this case the tunnel lining needs to be completely demolished and reconstructed, either locally or over its entire length.

The accuracy of calculations required for design is strictly related to the levels of damage and of the consequent rehabilitation.

If the damage is limited, the calculations could be based on relatively simple numerical models or on closed analytical solutions.

In more complex conditions recourse should be made to numerical analyses based e.g. the finite element method. They could range from relatively simple two-dimensional linear-elastic calculations to more cumbersome three-dimensional elasto-plastic analyses.

The application of this design approach has been illustrated with reference to three tunnel of highway A15 (Autocamionale della Cisa) located in central Italy.

The discussed case histories show that the consistent approach adopted here for design is able to reduce the uncertainties related to the evaluation of the stress distribution in the supporting structures and in the surrounding rock. In addition, when adequate techniques are employed for the restoration, it provides a proper level of safety during construction and leads to acceptable costs of the overall rehabilitation.

6 ACKNOWLEDGMENTS

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