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Analysis of the Damages Affecting the Pre-Casted Concrete Liner of a Deep Tunnel

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Abstract

The origin of the fractures that developed in the pre-casted concrete liner of a deep tunnel is investigated through a series of non-linear finite element analyses in plane strain regime. Three possible causes are considered, i.e. the excessive mountain pressure, the anisotropic in situ stress field and the incomplete filling of the rock/lining gap. The calculations are based on two numerical models, one of which accounts for the strain softening behaviour of the lining and for the rock/lining mono-lateral contact. The results of analyses show that the observed damages are likely to depend on the mentioned incomplete filling. On this base some conclusions are drawn on the possible procedures for the structural restoration of the opening.

Keywords: Damages, finite elements, pre-casted lining, strain softening.

1 INTRODUCTION

The problem at hand concerns a small diameter service tunnel excavated using a TBM. Its lining consists of pre-casted concrete segments. The gap existing between liner and rock was filled with selected granular material, or pea gravel. A particular aspect of the lining segments is that they are reinforced solely through steel fibres, some centimetres in length, while the standard steel bars are absent.
The lining showed progressive damages since the early stages of construction. They consist of fractures at the tunnel crown and in the vicinity of its springlines. In some location their maximum width exceeds 1 cm. Being afraid that the progress of fracturing could lead to local failures of the lining, the owner decided the installation of steel ribs to reinforce the damaged sections.
Here the causes of the observed damage are investigated through a series of non linear finite element analyses. In particular, the following possible causes were considered in the calculations: the excessive mountain pressure with respect to the concrete strength, the anisotropic in situ stress field and the incomplete filling of the gap between liner and rock.
Two numerical models were developed to this purpose, both based on a two dimensional, plane strain discretization of the rock mass through quadrilateral isoparametric elements. The first one accounts only for the elastic plastic behaviour of the rock mass and introduces the concrete liner by means of linear elastic beam elements. The second model, more refined than the first one, adopts quadrilateral elements also for the liner. In this case the concrete has a strain softening behaviour and the closure of the possible gap between it and the rock is introduced in large displacement regime through suitable gap elements.
In the following, the tunnel characteristics and the available mechanical information on rock mass, concrete and pea gravel are first summarized. Then, the results of calculations based on the two mentioned numerical models are illustrated. They rule out the mountain pressure, and the anisotropic in situ stresses, as possible causes of damage and show that the development of fractures within the liner is likely to depend on the incomplete filling of the rock/liner gap.
Based on the numerical results some conclusions are drawn on the structural restoration of the tunnel and, in particular, on the characteristics of the steel ribs necessary to stabilize the opening.

2 TUNNEL CHARACTERISTICS AND MATERIAL PARAMETERS

The tunnel has a length close to 5 km, an internal diameter of about 4 m and its depth of cover reaches 550 m. Its liner consists of circular rings having length of 1.5 m in the direction of the tunnel axis. Each of them is formed by 6 segments of pre-casted concrete. The segments have a thickness of 25 cm and their external radius is 2.15 m, while the radius of excavation is 2.3 m. Consequently, the nominal thickness of the gap between rock and liner is 15 cm. As already mentioned, the concrete segments are reinforced solely through steel fibres.

After installing each ring, the lower part of the gap between it and the rock is filled with cement mortar, while the upper part is filled with pea gravel. The pea gravel zone has an extension of 120°-130° on the right and left sides of the crown.

The tunnel crosses various rock formations consisting of marl, shale and limestone. Its central part, where the most severe damages occurred, is located within the limestone formation.

To improve the available information an experimental campaign was carried out in the framework of this study. It included in situ pressuremeter tests and laboratory tests on recovered rock samples. In addition laboratory tests were performed on the pea gravel and on samples of the fibre reinforced concrete.

The numerical analyses that will be subsequently discussed concern a damaged section with a depth of cover of 450 m located within the limestone formation. The experimental investigation led to the following mechanical parameters for the rock: rock mass elastic modulus from Menard pressuremeter tests E_{RM}=1.2 GPa; intact rock elastic modulus from unconfined compression tests E_{Lab}=54 GPa; intact rock unconfined compression strength σ^*_{Lab}=110 MPa; friction angle from laboratory direct shear tests φ_r = 27°.

The intact rock cohesion can be easily evaluated, knowing the laboratory unconfined compression strength, assuming that the same residual friction angle characterizes both rock mass and samples. Assuming now that the ratio between laboratory and in situ elastic moduli coincides with that between laboratory and in situ cohesion, the cohesion of the rock mass is c_{RM}=750 kPa.

Since low RQDs characterize the recovered rock, it is possible that the average in situ cohesion is lower than that previously evaluated. For this reason the calculations were repeated varying c_{RM} from 750 to 50 kPa.

Several samples of pea gravel were subjected to confined compression tests, using a large size oedometer, and to triaxial tests. The first group of tests provided the elastic modulus of pea gravel in confined conditions, which are similar to those present within the rock/liner gap, while the second group led to its friction angle.

The confined compression tests (cf. Figure 1a) led to a tangent modulus, for moderately dense material, of about 50 MPa. The modulus increases up to 120 MPa after some loading/unloading cycles.

The multi stage triaxial tests (cf. Figure 1b) led to a friction angle of about 40°.

The limited compaction of pea gravel during grouting suggested using in the calculations an elastic modulus E_{PG} equal to 50 MPa. As to the friction angle, con-

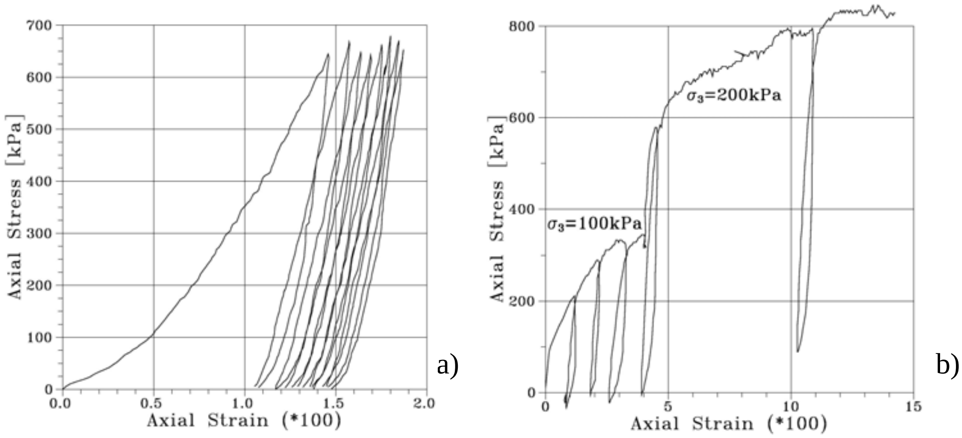

Figure 1: Results of confined compression test (a) and triaxial test (b) on pea gravel.

sidering that the thin layer of pea gravel represents an "interface" between rock and liner, and that the friction angle of the rock (27°) is lower than that of the pea gravel (40°), the friction angle of the interface φ_{PG} was assumed equal to 27°.
As to the fibre reinforced concrete, the unconfined compression tests led to an elastic modulus E_C of about 47 GPa and to a compression strength σ^*_C between 73 and 119 MPa. Also tensile tests were carried out on this material (cf. Figure 2). Before subjecting the samples to increasing elongation, a thin circumferential in-

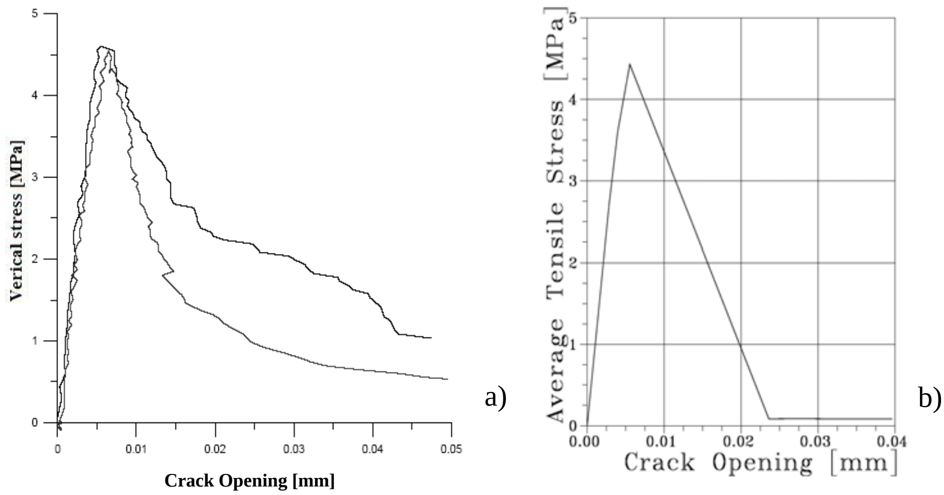

Figure 2: Experimental (a) and numerical (b) results of tensile tests on concrete.

dent was produced at their mid height to allow for the propagation of a crack, but limiting the disturbance caused by the loading caps. These tests provided a tensile resistance of concrete between 3.3 and 4.6 MPa. The tests were numerically simulated in axisymmetric conditions discretizing the samples into a mesh of isoparametric elements and adopting a strain softening behaviour [1,2]. The calibrated mechanical parameters of the numerical model correspond to compression and tensile strengths of 85 MPa and 4.4 MPa, respectively.

3 NUMERICAL STUDY

As previously mentioned, two plane strain finite element models were developed for the purposes of this study. The first one accounts for an elastic-perfectly plastic behaviour of the rock mass, governed by a non associated Mohr-Coulomb yield criterion, while the liner is linear elastic. Some of the characteristic curves of the opening obtained with different c_{RM} values, and with a coefficient of rock pressure at rest K_0 equal to 1, are shown in Figure 3.

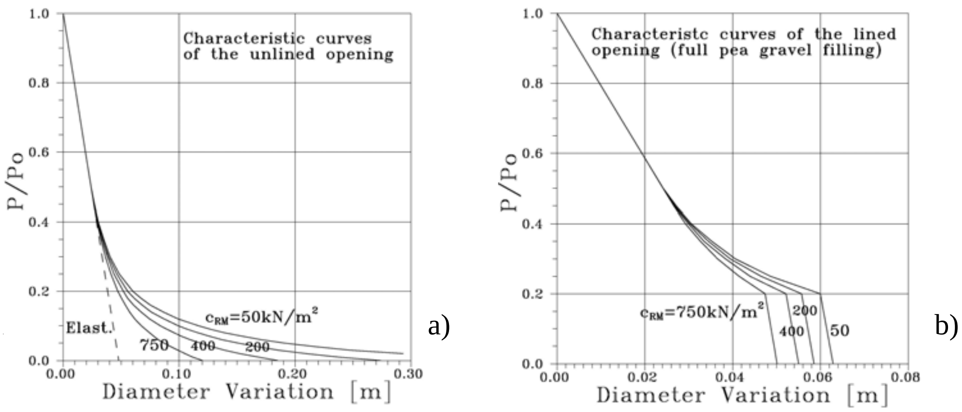

Figure 3: Characteristic curves for K_0=1: a) unlined opening; b) lined opening.

The diagrams of Figure 3b, where P_0 is the in situ hydrostatic stress and P is the decreasing pressure on the opening contour, are based on the assumption that the liner, and the pea gravel filling, are set in place when the pressure on the opening contour corresponds to 20% of the in situ stresses. This seems a reasonable assumption according to the observations contained in [3]. The results shown in Figure 3b correspond to maximum compressive stresses in the liner of the order of 20 MPa, far below the limit resistance of concrete.

Due to the linear elastic lining behaviour the attempts to apply this model to the case of incomplete filling of the liner/rock gap did not provide satisfactory results. To overcome this limit a second model was developed in which the concrete has a strain softening behaviour (cf. Figure 2b). A detail of the mesh is shown in Figure 4a, while Figure 4b reports the characteristic curves obtained with K_0=0.5. Note that the second model, which includes more elements than the first one, provides displacements slightly larger than those shown in Figure 3b.

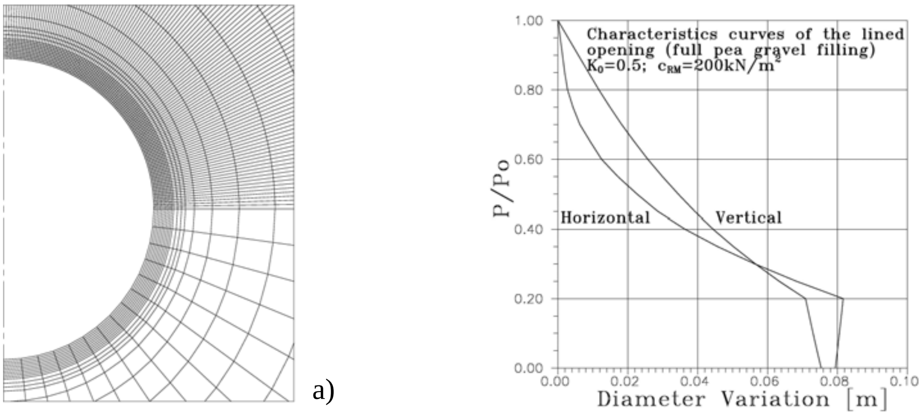

Figure 4: a) Second finite element mesh; b) Characteristic curves for K_0=0.5.

It turned out that, even for K_0=0.5, relatively low compressive stresses develop in the liner (about 30 MPa) and that no tensile stresses are present in it. The results summarized in Figures 3b and 4b rule out the genuine mountain pressure and the anisotropic in situ stresses as possible causes of the observed damages.

The analyses accounting for the incomplete filling of the liner/rock gap were carried out in large displacement regime adopting suitable interface elements able to detect the possible closure of the gap during the loading process. An illustrative example of the performance of these elements is shown in Figure 5.

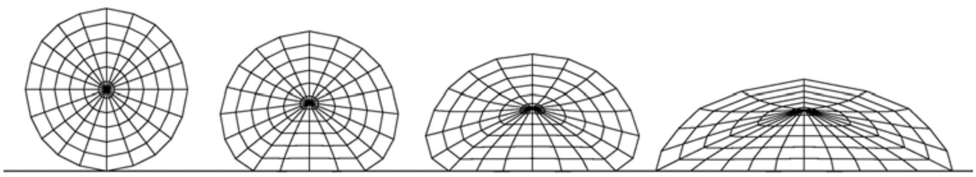

Figure 5: Deformation of a visco-plastic disc subjected to its self weight and resting on a rough plane.

Several analyses have been carried out varying the initial thickness of the liner/rock gap and its extension from the tunnel crown. All analyses assumed symmetric conditions with respect to the vertical axis through the tunnel centre-line. Since the finite elements in Figure 4a are larger than those used in simulating the tensile tests (Figure 2) the softening parameters were suitably scaled according to [4].

For sake of briefness only the results of one analysis are illustrated here. They refer to an initial gap having thickness of 15 cm and extension of 45° on both sides of the tunnel crown.

Figure 6a reports the diagrams relating the variation of the vertical and horizontal diameters of the opening to the reduction of pressure on its contour. Note that the liner and the partial grouting of pea gravel are set in place when the pressure reaches 20% of the in situ stresses. Figure 6b shows, for both liner and rock, the contour lines of the square root of the second invariant of the deviatoric plastic strains corresponding to condition C in Figure 6a.

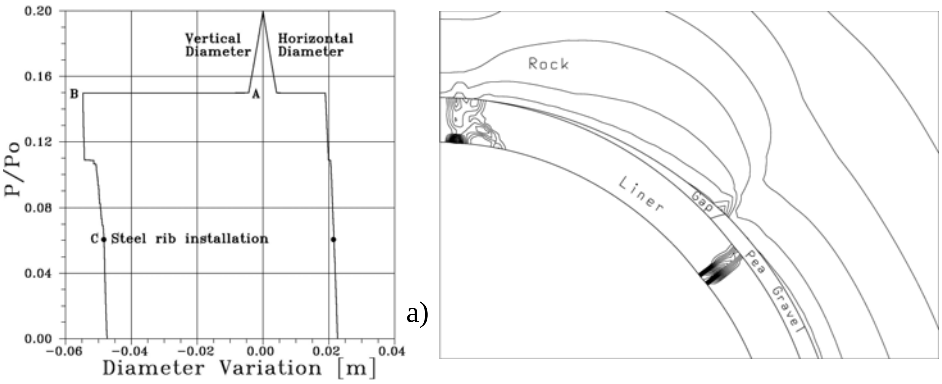

Figure 6: a) Variation of vertical and horizontal diameters with decreasing pressure on the opening (initial gap thickness 15 cm; extension of the empty gap 45° on both sides of the tunnel crown); b) Contour lines of the deviatoric plastic strains for condition C.

It can be observed (Figure 6a) that initially the deformation of the liner develops almost linearly. The horizontal diameter decreases, while the vertical one increases due to the lack of contact between liner and rock at the crown. Then, a sudden displacement occurs (A to B in Figure 6a) that corresponds to the onset of fractures at the tunnel crown and at its springlines. This brings the diagram to condition B, where the closure of the rock/liner gap at the crown occurs. Further decrease of the pressure on the opening contour produce a reduction of both vertical and horizontal diameters until condition C is reached. This, in the calculations, corresponds to the failure of the lining due to the excessive spreading of the fractured zones. To avoid collapse, beam elements were introduced at C that are equivalent to two HE 140 M steel ribs for each liner ring. This provision ensures the stability of the opening and permits completing the numerical excavation process. The calculated final stresses in the steel ribs turn out to be compatible with their structural resistance.

4 CONCLUDING REMARKS

The numerical analyses carried out for the purposes of this study indicate that the damages to the tunnel liner are likely to depend on the non uniform rock pressure caused by the incomplete filling of the liner/rock gap. They also lead to the quantitative evaluation of the stresses within the steel ribs adopted for stabilizing the damaged sections. Once the risk of collapse has been avoided thorough this provision, the gap could be grouted using low pressure injections of cement mortar, see e.g. [5]. In this case particular attention should be paid in reaching a complete filling of the empty portions of the gap and a uniform rock pressure on the liner.

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