Pavoncelli tunnel case study

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ABSTRACT: This paper describes the back analysis of the Pavoncelli tunnel, an hydraulic tunnel which was excavated between 1904 and 1915 in South of Italy and experienced severe damages during the 1980 Irpino-Lucano earthquake. A numerical simulation which is intended to study the behaviour of the tunnel during this seismic event is presented, also accounting for the swelling phenomenon that took place during and after construction.

1 INTRODUCTION

The paper deals with the seismic design of tunnels. The case study of the Pavoncelli tunnel, in South of Italy, is considered. The tunnel experienced severe damages during the 1980 Irpino-Lucano earthquake, which were localised in the areas where swelling of the local Clay-shales Formation occurred after construction. A by-pass tunnel is being designed and will be constructed close to the existing one. Therefore, the interest is to gain from the back analysis of the existing tunnel useful information on the tunnel behaviour during an earthquake. This will allow to improve the current design, also accounting for the swelling phenomenon that may occur during and after construction.

2 THE PAVONCELLI TUNNEL

2.1 *General overview*

The Pavoncelli tunnel, which is the base tunnel of the Acquedotto Pugliese, crosses the Apennines. This tunnel connects directly the Caposele spring with Conza della Campania, in South of Italy (Figure 1). Caposele is located 80 km far from Naples in the Cervialto massif, a sector of the Lucanian Appenines. The water, after 244 km, reaches Villa Castelli (Le) and spreads towards Foggia, mostly underground, through a highly seismic region.

The construction of the Pavoncelli tunnel (10 km long) begun in 1904 and was completed in 1915. Among the severe difficulties incurred during construction the presence of the highly squeezing/swelling Varicolori Clay-shales is to be mentioned, as they caused large convergences and high stresses to the lining. These phenomena were already experienced during the excavation of the Cristina tunnel along the historic railway line Caserta-Foggia (Barla, 2002).

The masonry lining used for support has a different thickness depending on the soil encountered. Based on what is known (Viggiani, 2001), the tunnel originally had a horse-shoe cross section, while a circular cross section was adopted only rarely when crossing the Varicolori Clay-shales. The thickness of the lining initially installed varied between 26 and 85 cm at the crown and between 26 and 34 cm at the invert. Due to swelling, the tunnel experienced, immediately after completion, heavy damages with longitudinal cracks and invert heave. Due to this, starting from 1922, the tunnel needed be reinforced by placing a stronger lining, with average thickness of 130 cm.

The tunnel cross section of interest in this paper is that adopted after the reconstruction of the tunnel in 1931. The interest is placed over the cross section at chainage 4550 m, where an horse-shoe shaped tunnel with lining thickness between 1.15÷1.60 m at the crown and 0.6-1.0 m at the invert is present (Figure 2). This section was excavated in Varicolori Clay-shales and is located in the area where severe damages occurred in the lining, during the 1980 Irpino-Lucano earthquake: mostly cross fractures and invert heave (Figure 3 and 4).

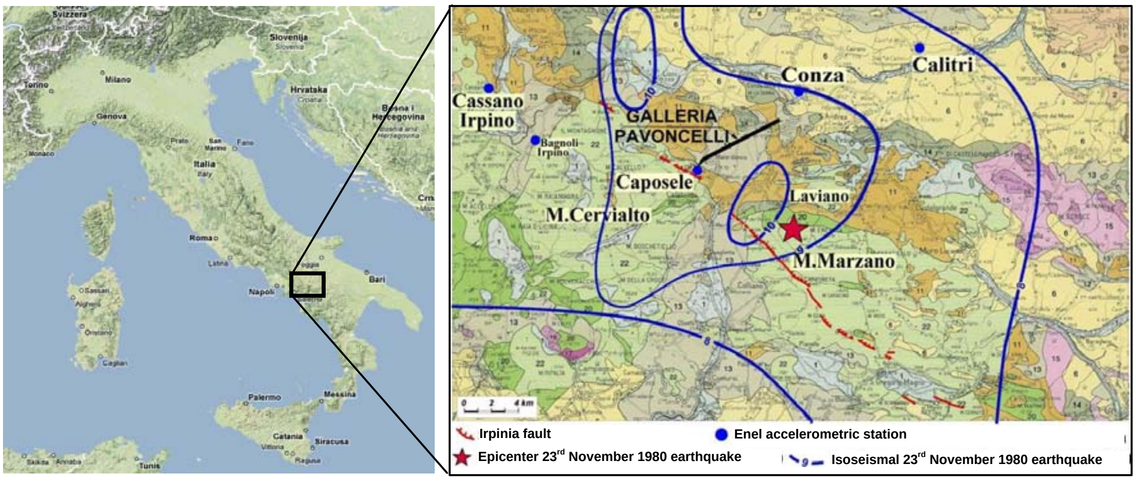


Figure 1. Location of the Pavoncelli Tunnel and isoseismal of the 23rd November 1980 Irpino-Lucano earthquake in MSK scale (from C.H.R., P.F. "Geodinamica", Roma, 1981). Maximum accelerations monitored by ENEL accelerograms network are also shown.

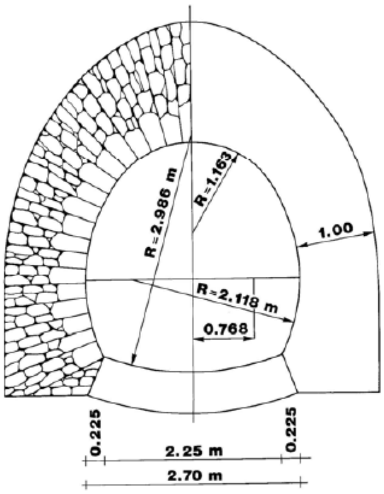


Figure 2. Cross section in the Varicolori Clay-shales (Cotecchia et al., 1986).

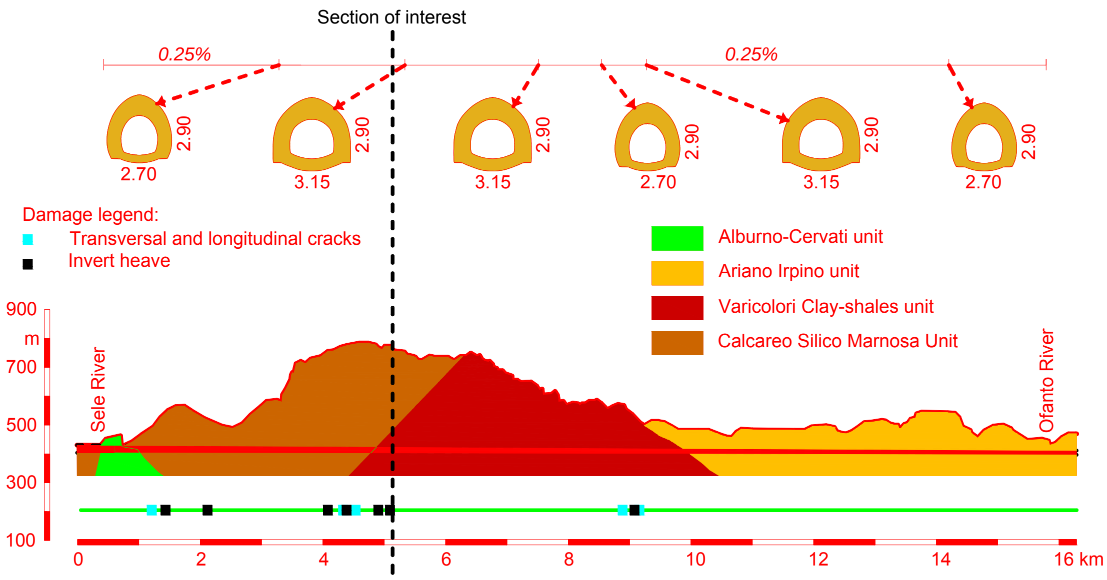


Figure 3. Longitudinal profile of the Pavoncelli tunnel with indication of major damages occurred during the 23rd November 1980 Irpino-Lucano earthquake.

Figure 4. Photographs of typical damages occurred during the 23rd November 1980 Irpino-Lucano earthquake (modified from Cotecchia et al., 1986).

2.2 *Geological-geostructural context*

The Pavoncelli tunnel crosses a very complex area of the Appennines from the structural geologic and hydrogeologic points of views. It has also been subjected to tectonic and orogenetic activity. A precise geologic pattern is not easy to determine due to the different formations, the heterogeneity within a single formation, and the variability in permeability. The following different Units can be defined:

- Calcareo Silico Marnosa Unit
- Alburno-Cervati Unit
- Varicolori Clay-shales Unit
- Irpine Unit
- Altavilla Unit
- Ariano Irpino Unit.

3 THE IRPINO LUCANO EARTHQUAKE

The Pavoncelli tunnel is within an area where the seismic risk is due to the vicinity to a mountain chain which experienced upward heave in the late 700.000 years. The Appennines are still heaving with respect to the Tirrenian sea and are characterised by faults where main seismic events are originated. The isoseismal lines of the main Campano-Lucano Appennines earthquakes show that the areas with higher intensity are along the chain axis (Figure 1).

From the 1987 Italian seismic classification atlas, the area is in a seismic zone of first category with a seismicity degree S=12. Among the most studied earthquakes which occurred in Irpinia, without doubt the 23rd November 1980 seismic event is definitely the one that underlines the connection between deep focal mechanisms, superficial structural elements, displacements and slopes geo-morfological evolution.

For the purpose of the present work, the eathquake recorded at the Sturno station during the 23 November 1980 Irpino-Lucano earthquake (Mw=6.9) is considered.

4 NUMERICAL ANALYSES

A cross section of the tunnel located at 400 m depth was selected for the numerical simulation. The analyses were conducted with the Finite Difference Method (FDM) and the Flac code (Itasca 2005) in plane strain conditions.

The aim is to reproduce the main phenomenona experienced by the tunnel during construction, during its service life, in long term conditions (including swelling of the ground), and due to the earthquake. A set of analyses were therefore carried out in both static and seismic conditions.

4.1 *Model geometry*

The FDM grid used is to be considered as the best compromise between the need of having boundaries sufficiently far, in order not to influence the behaviour around the tunnel, both in static and in seismic conditions, but also not to have excessive computational time.

A number of trial tests allowed to define the length of the model to be at least three times its height in order to avoid boundary influences during the dynamic analysis.

The model geometry is shown, out of scale, in Figure 5 together with the boundary conditions adopted. As illustrated, quiet boundaries (viscous vibration dampers) are applied at the grid contours (Lysmer e Kuhlmeyer, 1969), both in the horizontal and in the vertical directions, so that one can assume the model to be infinitely wide in the horizontal direction, avoiding reflections of the incident waves. Hysteretic damping is applied to the model during dynamic computation.

The elements size has an influence over wave propagation. For this reason the seismic input was filtered for a frequency of wavelength 10 times the maximum length of a single element in the mesh.

The analyses are conducted with constant gravity and in effective stress conditions.

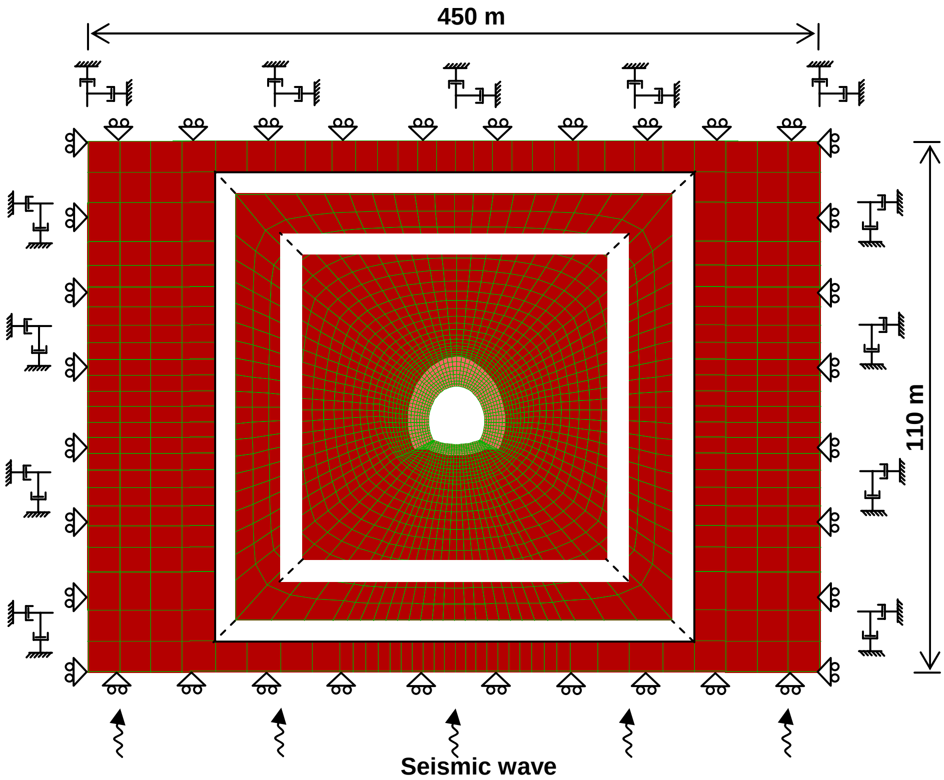


Figure 5. FDM grid and boundary conditions.

4.2 *Initial stress state and material properties*

As mentioned above, gravity is kept constant within the model. The initial stress state for the tunnel cross section of interest is defined as follows:
Vertical total stress = 6.7 MPa
Horizontal total stress = 5.4 MPa
Pore pressure = 200 kPa.

The geotechnical parameters of the Varicolori Clay-shales are assumed on the basis of laboratory tests given in Cotecchia et al. (1992) and are listed in Table 1.

Table 1. Geotechnical parameters for the Varicolori Clay-shales (Cotecchia et al. 1992).

Cohesion [kPa]	ρ [kg/m^3]	n [-]	φ' [°]	E [MPa]	ν [-]	V_s [m/s]
20	1740	0.3	18	580	0.45	340

For the swelling behaviour, reference is made to the method described in Barla (2008), where the swelling potential of the ground is directly related to its tendency to develop negative excess pore pressure during excavation by defining a functional relationship between the volumetric strain (ε_{vol}) and the excess pore pressure (Δu): $\varepsilon_{vol} = f(\Delta u)$, valid for the specific state of stress at the end of consolidation, which is intended to represent the in situ conditions.

This relationship can be adopted to simulate the swelling behaviour by applying to the elements in the FDM grid the volumetric strain as a function of the excess pore pressure computed for each element during the undrained analysis. In order to implement it in the Flac code, the method of Noorany et al. (1999) is used. The ε_{vol} value for each element is computed vs Δu and the corresponding total stress increments are applied to the grid to obtain the desired behaviour.

A relationship for the Varicolori Clay-shales, based on laboratory triaxial tests, is not available. Therefore, the ε_{vol} -Δu curve is estimated as follows, with the threshold value chosen on the basis of oedometer tests:

$$\varepsilon_{vol} = -2.64 + 2.3 \cdot \arctan\left(\frac{\Delta u}{300} + 2.2\right) \quad (1)$$

The tunnel masonry lining, which is assumed to follow a linear elastic behaviour, is given the properties shown in Table 2. A no slip condition is assumed between the lining and the ground.

Table 2. Properties for the masonry lining.

Thickness [m]	E [GPa]	ν [-]
0.6-1.6	3	0.25

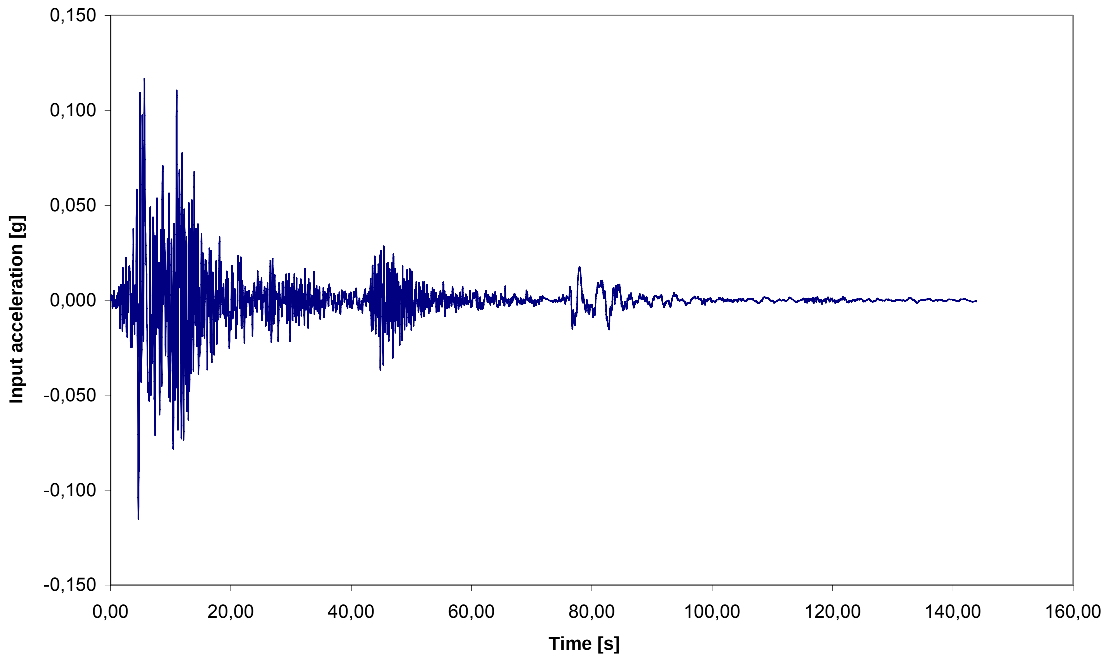


Figure 6. Input acceleration applied to the FDM grid.

The Sturno surface record of the 23 November 1980 Irpino-Lucano earthquake is considered. The record of the horizontal acceleration is applied to the bottom of the FDM grid, reduced by 50% (Figure 6). This is considered acceptable as the model is based on bedrock.

4.3 *Analysis sequence*

As mentioned above, the scope of the numerical analyses carried out is to reproduce the response of the tunnel during construction, during its service life, in the long term (including swelling of the ground), and due to the earthquake.

The analysis was therefore carried out considering the following steps (Figure 7):

1) Assessment of the initial state of stress in the ground by applying gravity loading and initial stress conditions.
2) Excavation of the tunnel full section with a relaxation factor of 0.9 in undrained conditions.
3) Placement of the lining and full relaxation in undrained conditions.
4) Simulation of the swelling process by applying the swelling volumetric strain given by equation (1) to the elements of ground, thus simulating the long term behaviour as described in Barla (2008).
5) Earthquake simulation by running the seismic analysis in undrained conditions.

The assumption of a 0.9 relaxation factor is due to the traditional construction method adopted, where the lining was applied in a multiple stage sequence. For the simulation of swelling, the excess pore pressure developed during the undrained excavation was reduced in order to compute the volumetric strain to be applied. In fact, it is not reasonable to assume that excavation took place in fully undrained conditions. The advancement rate and the traditional excavation method adopted at the time of tunnel construction (early 1900) implies that necessarily some drainage took place, thus reducing considerably the development of excess pore pressures.

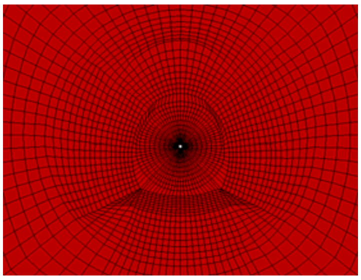
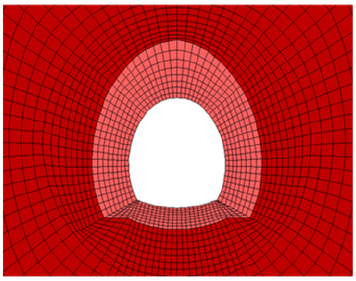
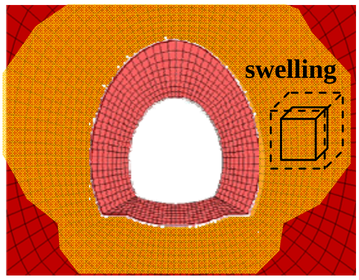
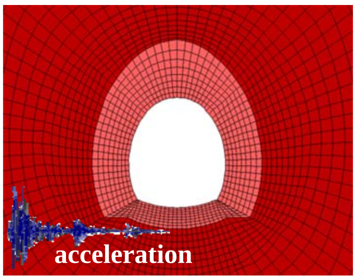
Step 1		Mesh generation, with the ground parameters provided for the different elements and application of the initial state of stress.
Step 2		Undrained excavation of the horseshoe shaped tunnel, by imposing a 90% stress relief on the tunnel perimeter.
Step 3		Installation of the masonry lining (activating the elements representing it), while the remaining 10% stress relief is assumed to take place in undrained conditions.
Step 4		Swelling is assumed to occur in the ground where $\Delta u < 0$.
Step 5		Earthquake simulation.

Figure 7. Simulation steps considered with the numerical computations.

4.4 *Analysis validation*

The first step in the dynamic analysis was to validate the numerical model. This was achieved by comparing the model response in terms of horizontal acceleration. To this extent a numerical analysis was specifically performed. In order to be able to compare the results with EERA, a one dimensional code for surface seismic response (Bardet et al., 2000), the model was constructed to include the ground surface and the tunnel was not excavated. A linear elastic constitutive model was assumed. The input accelerogram was applied at the bottom of the model and the resulting horizontal acceleration measured on the top of it. The resulting acceleration pattern was compared to that computed by using EERA as depicted in Figure 8. The response is very similar, therefore the numerical model can be considered validated for that pertaining to the dynamic behaviour.

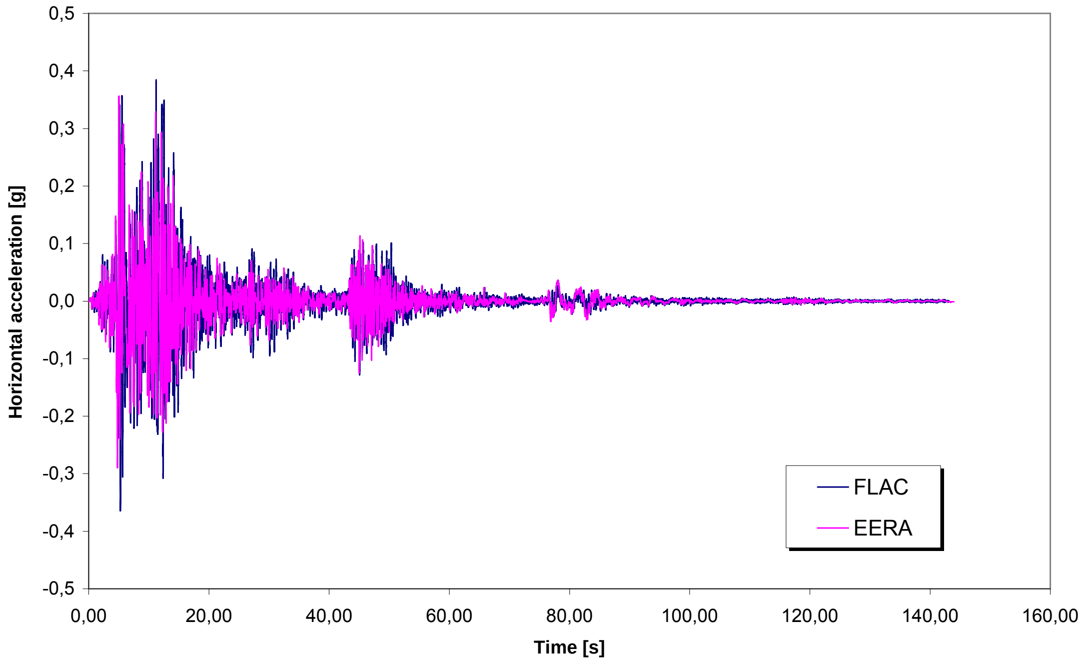


Figure 8. Comparison between Flac and EERA results.

4.5 *Analysis results*

The results of the analyses are compared in terms of stresses in the lining with reference to stages 3, 4 and 5 above (i.e. after excavation, after swelling, after earthquake). The analyses allowed to reproduce the lining response during the three stages in a reasonable way, depicting stress increments in the lining during swelling and dynamic shaking.

Figure 9 and 10 show the axial force and bending moment in the lining. It is noted that in seismic conditions the maximum axial force and bending moment are plotted. As shown, higher hoop stresses occur at the springline and at the crown where the lining is thicker.

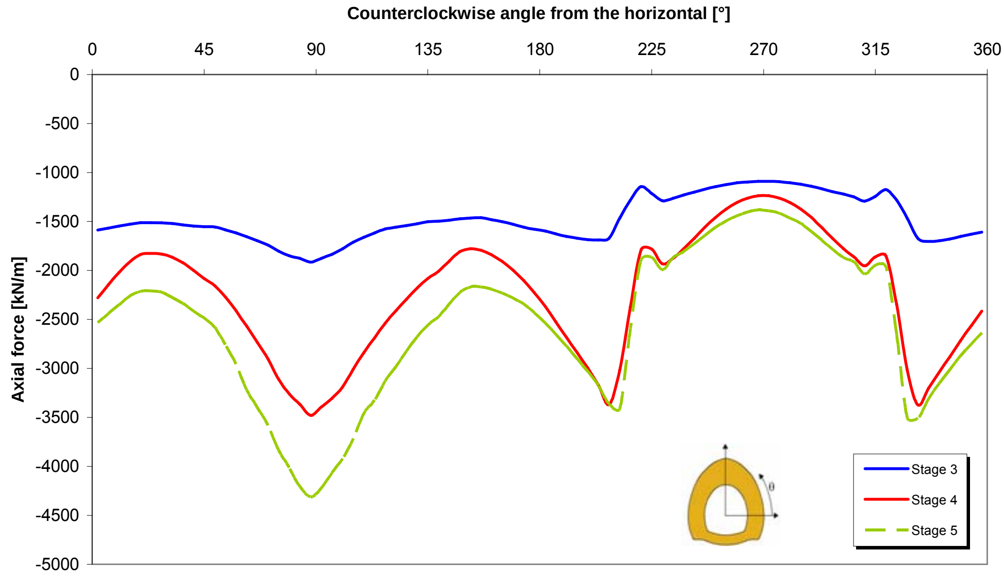


Figure 9. Axial force in the lining.

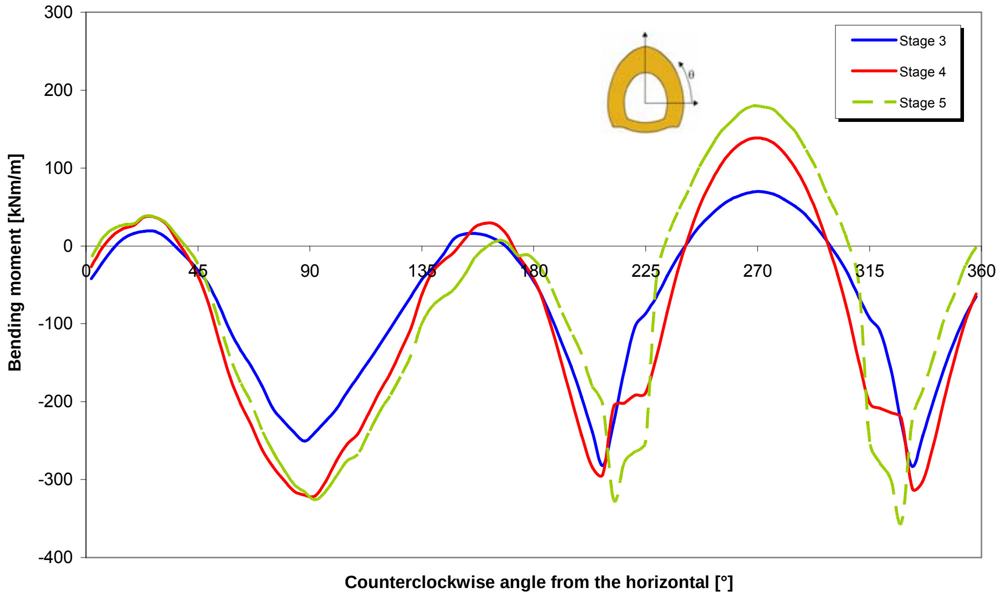


Figure 10. Bending moment in the lining.

To evaluate the damaged sections in the lining, the stress state at the crown, invert and springlines is compared to the strength domain in Figure 11. Three different strength domains are plotted, each one corresponding to a different lining thickness. For simplicity the average thicknesses at the crown, springline and invert have been considered (respectively, 1.32, 1.05, 0.6 m). Each dot in the diagram represents the stress state (axial force and bending moment) for a single element of the lining. Dots outside the strength domain are clearly representative of not allowable stress conditions, which would cause the lining to fail.

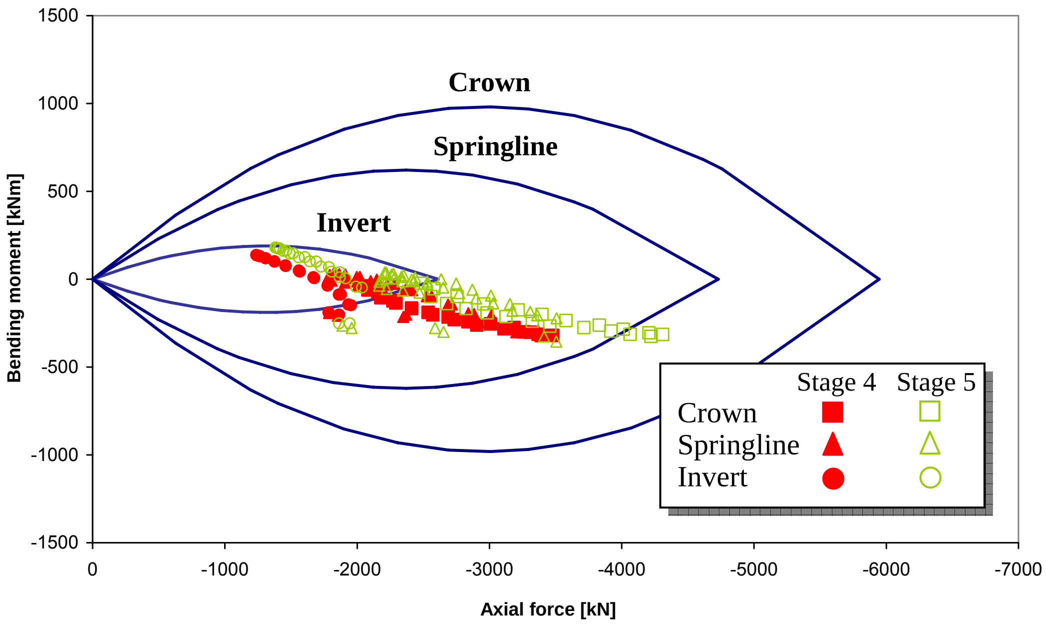


Figure 11. Strength domain and stress state in the lining.

It is clearly shown that the lining is within the elastic domain during the first four stages, getting near to the failure envelope after swelling, particularly at the invert. During the dynamic analysis the stress components cause the lining to go beyond the elastic domain at the invert. The critical sections are at the invert and at the lower portion of the springlines, in good agreement with what observed in the tunnel after the 23rd November 1980 Irpino-Lucano earthquake (Cotecchia et al., 1986). The overall behaviour appears to be in good agreement with field observations as shown in Figure 3 and 4.

It is noted that the invert and the springlines are heavily stressed in static conditions, due to swelling, and the stress states are very close to the strength domain. This allows one to conclude that the increment in stresses due to the earthquake was sufficient to cause failure of the lining which was already highly stressed due to swelling.

5 CONCLUSIONS

The present paper describes the back analysis of the Pavoncelli tunnel. The numerical simulations presented allow to study the behaviour of the tunnel during an earthquake by also accounting for the swelling phenomenon that occurred during and after construction. It is shown that the critical sections in the lining are located at the invert and springlines, highly stressed after construction and subsequent swelling of the Varicolori Clay-shales formation.

The earthquake is shown to have induced failure in these sections. The results are in good agreement with observations in the tunnel after the 23rd November 1980 Irpino-Lucano earthquake (Cotecchia et al., 1986). The information gained so far is of interest to update the seismic design of the by-pass tunnel to be constructed close to the existing one in the near future.

6 ACKNOWLEDGEMENTS

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