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TUNNEL UNREINFORCED FINAL LINING
APPLICATIONS AND LIMITATIONS

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TUNNEL UNREINFORCED FINAL LINING – APPLICATIONS AND LIMITATIONS

The “rising up” of the “tunnel unreinforced final lining” concept has been directly linked with the design and cost optimisations’ needs of the infrastructure projects worldwide in the recent years. The main scope of the concept ‘s application is to limit the construction budgets, at reasonable levels, by avoiding unnecessary over designs in tunnelling, however without risking the final quality and the long term safety and serviceability conditions of the underground structures.

The existing design and construction experiences in tunnelling worldwide have shown that the concept of the “tunnel unreinforced final lining” is not a prohibitive one. A number of highway and railway tunnels structures, with unreinforced final linings, were constructed successfully and are in full operation over the last 30 years in Europe and in Asia. However, the recent increasing demand for design and cost optimisation in tunnelling projects, in combination with the recent very restrict design codes’ regulations and construction specifications for high level long term safety and serviceability conditions of the underground structures, dictate the need to evaluate in detail the application limits of the concept. These limits are related to the geotechnical environment, the seismic / tectonic regimes and the topographies that tunnels will be constructed.

The relevant European design codes, recommendations and guidelines which offer the necessary design framework of the tunnel unreinforced tunnel final linings are:

- Eurocode 2 EN 1992-1 / Section 12, which defines a number of provisions for plain and lightly reinforced concrete structural members, regarding the concrete properties and the appropriate factors of safety for the materials strength.
- AFTES recommendations focus on to the design of the tunnel final lining members, by imposing a restriction on the possibly developed crack depth ($<h_w/2$) and by limiting the allowable eccentricity ($e<0.3h_w$) for all load cases, that result in high axial forces N in relation to the concrete strength ($N>2.7\%$ (bxh_wxf_{ck}), where: (i) h_w is the overall height of the tunnel cross-section, (ii) b is the overall width of the tunnel cross-section and (iii) f_{ck} is the characteristic concrete strength in compression.

Michalis et al [1] performed a parametric 3-D numerical analysis aiming to examine thoroughly the “safe” application limits of the tunnels unreinforced final linings, within the aforesaid design codes framework. This analysis considered the “three lane” typical open and closed sections of the 6km double tunnel, along Maliakos – Kleidi concession motorway project in central Greece (Figures 1 and 2). The analysis examined only typical Eurocode static design load cases, covering all possible range, from the lining construction stage (at the time of de- moulding with the adoption of the effects of the “young” concrete properties and the concrete’s hydration heat emission) to the possible occurrence of an accidental explosion, inside the tunnel, during its operational period. The analysis focused in homogeneous and in mixed face rock mass conditions, where their deformation moduli varied between 150MPa to 1000MPa.

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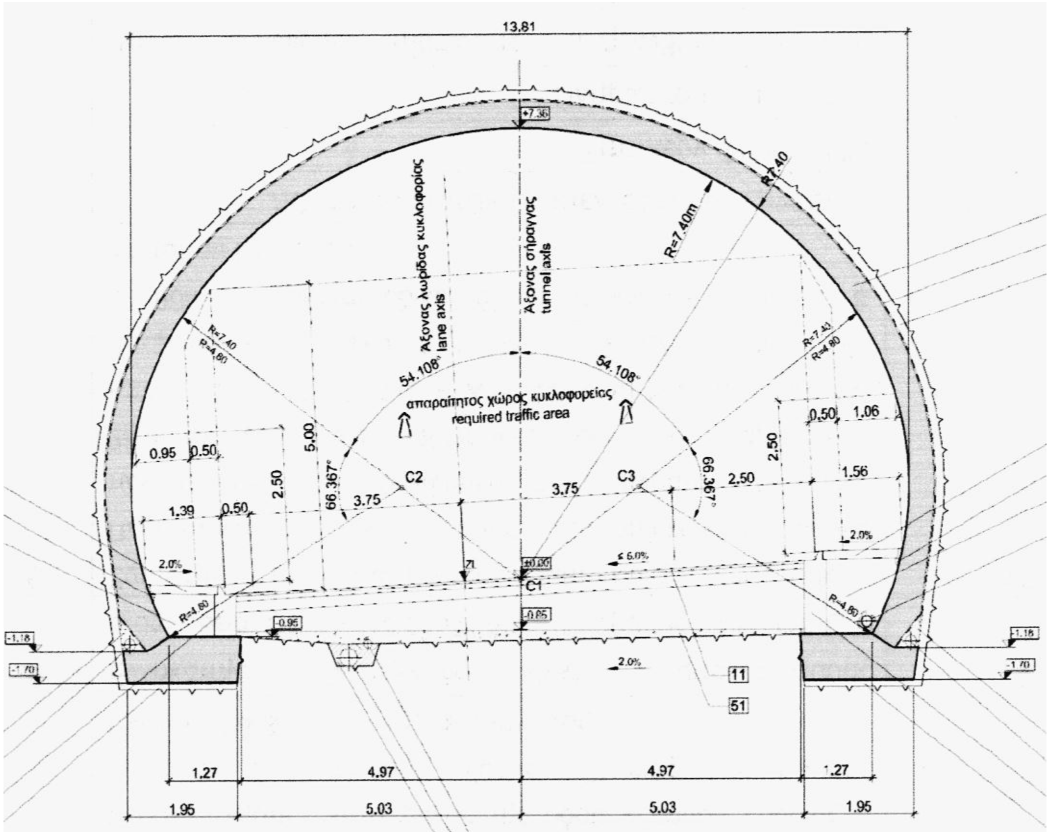

Figure 1. Typical open tunnel section.

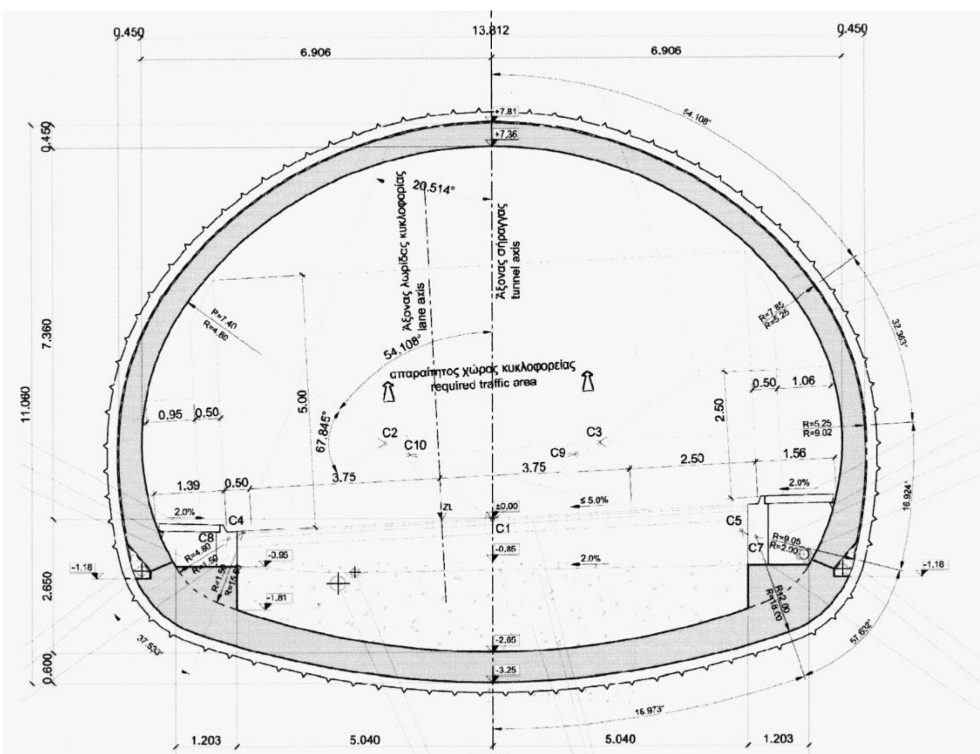

Figure 2. Typical closed tunnel section.

Typical finite models, used in the parametric study, are presented in Figures 3 and 4. The complete dynamic analysis of the tunnel unreinforced final lining, by considering real time history earthquake events, is currently under extensive numerical investigation [2]. However some preliminary conclusions from this sensitivity numerical study are presented in the next final paragraphs of the present paper.

The numerical simulation of the constitutive behaviour of the unreinforced concrete was made with the adoption of Willam and Warnke model [3] and by considering the EC-2 stress – strain curve, as well as the EC-2 concrete characteristic strengths and relevant safety factors. The tunnel lining – rock mass interaction was simulated, in detail, with the use of "stick – slip" elastic springs, where their stiffness values were calculated by considering both the surrounding rock mass deformability properties and the lining geometries (Figure 5).

Figure 3. 3-D finite element model of the tunnel final lining

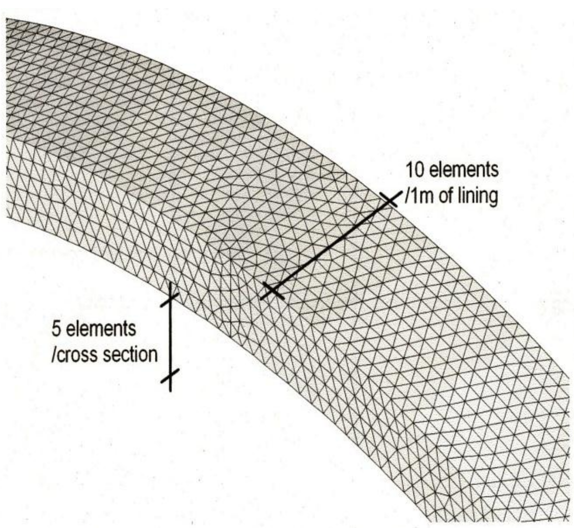

Figure 4. Detail of the 3-D finite element model of the tunnel final lining

The major conclusions drawn from the previously described parametric analysis can be summarised as follows:

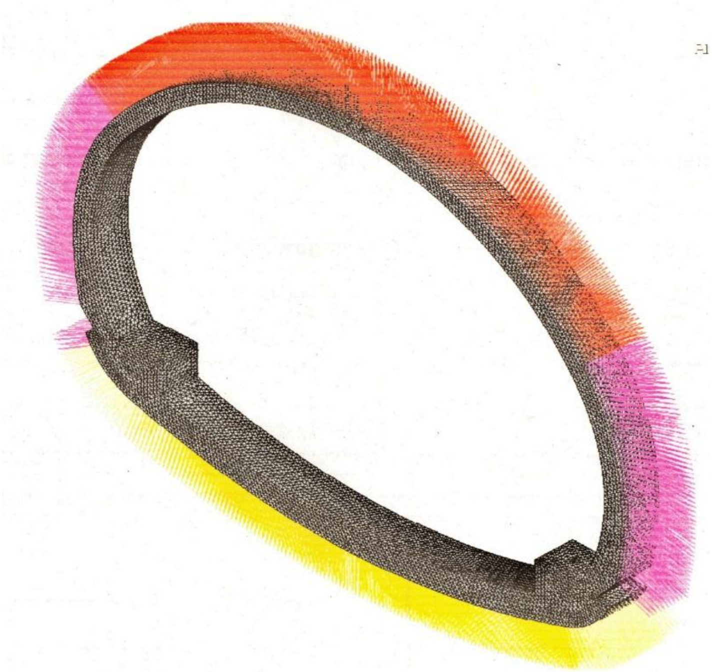

Figure 5. Tunnel lining – rock mass interaction model with " stick – slip" elastic springs, adopted in the parametric analyses

- The structural integrity of the examined tunnel unreinforced (in the vault) final lining sections has been verified in the case of relatively sound rock mass conditions, characterised by deformability modulus $E_m \geq$ 800MPa - 1000MPa, as it fulfils Eurocode 2 Part 1-1 / Section 12 provisions, regarding plain and lightly reinforced concrete members, and AFTES aforesaid design limiting criterion, in respect to the crack depths and the load eccentricities. The aforesaid range of the surrounding rock mass E_m values create enough confidence that the unreinforced (in vault) final linings can be applied even for tunnels that cross high seismic areas, or areas with adverse topographic morphology and relatively low covers.
- In cases where the surrounding rock mass conditions are characterised by deformability modulus 300MPa< E_m < 800MPa, significant cracking depth occurs in the unreinforced tunnel vault, which exceeds the half of the tunnel section's height, in combination to the formulation of secondary horizontal cracking (Figure 6), thus jeopardizing the initiation of spalling phenomena. The long term structural integrity of the tunnel unreinforced vault final lining cannot be achieved, especially in seismic areas.
- In cases, where the surrounding rock mass conditions are characterised by deformability modulus $E_m \leq$300MPa, the unreinforced concrete sections cannot be applied, because of the high risk of the concrete crushing (Figure 7).
- The footings areas and the invert sections (if exist) must be reinforced for any quality of the surrounding rock masses.
- An accidental explosion, inside the tunnel, will result to severe cracking of the unreinforced final lining at the lower bottom arches of the vault (Figure 8). However, by considering the locations that these cracks are expected to appear and the calculated displacement patterns,

no collapse mechanism of the lining can be formulated. However, repairing works after the explosion will be necessary.

- In tunnel portal areas the tunnel unreinforced final lining must be avoided, as well as in areas where the existence of nearby or crossing active faults have been recognised.

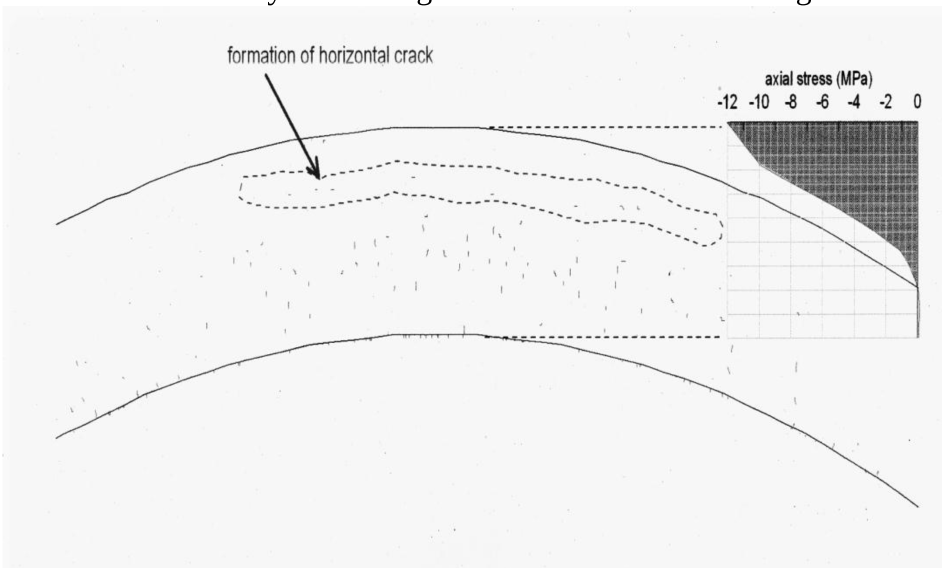

Figure 6. Cracking formation of the unreinforced vault of the final lining. Rock mass deformability modulus 300MPa<E_m<800MPa

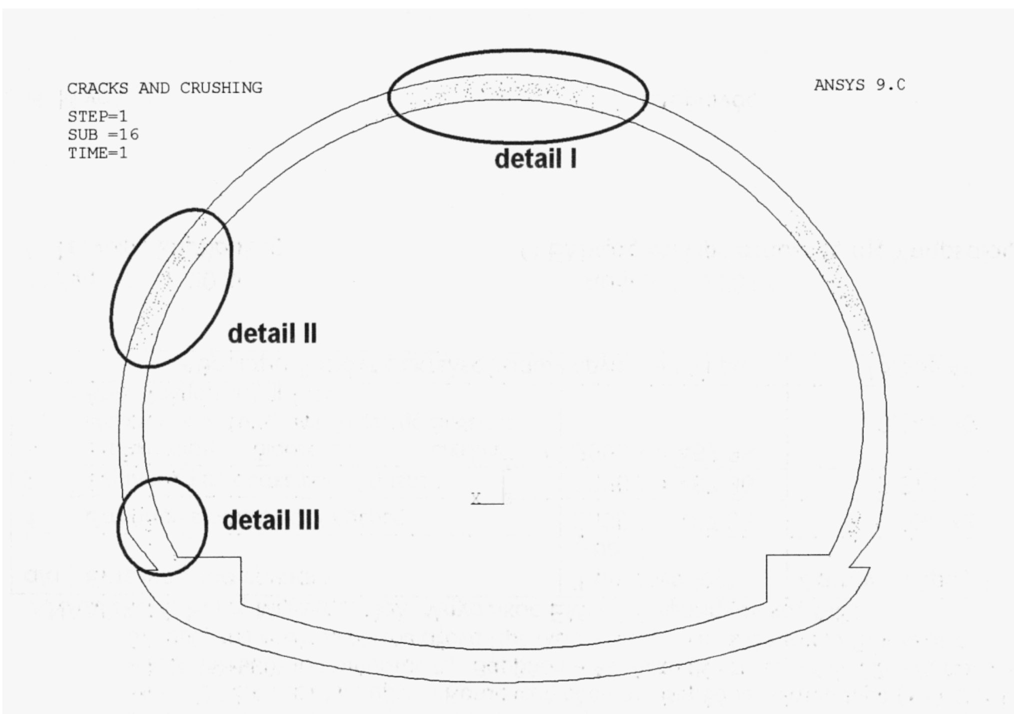

Figure 7. Extensive and severe cracking of unreinforced tunnel final lining in rock mass conditions with E<300 MPa.

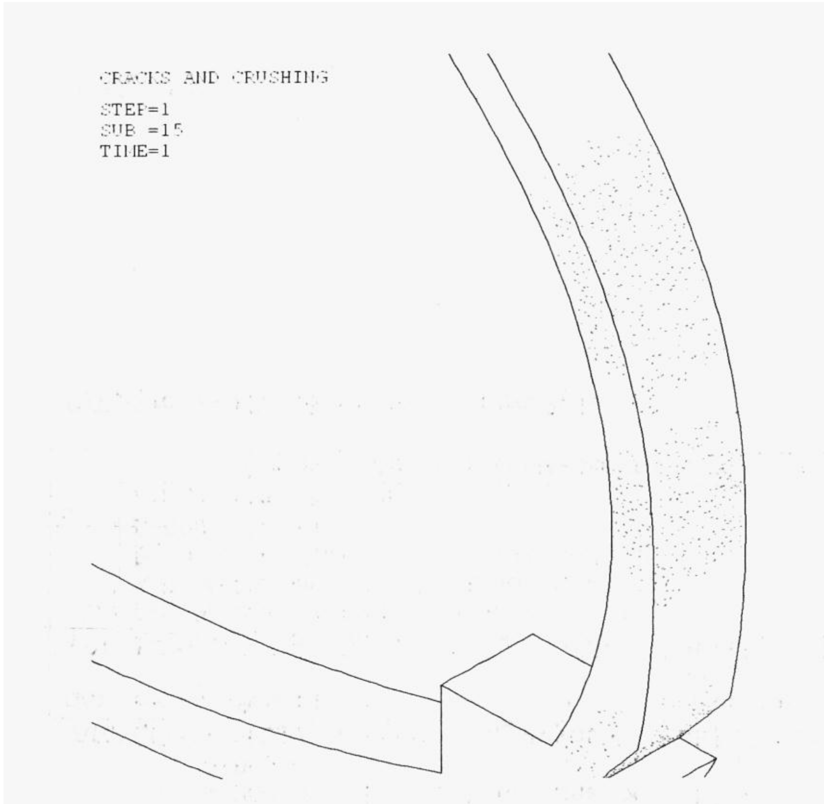

Figure 8. Pattern of cracking of unreinforced tunnel final lining due to explosion

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Ilias K. Michalis, is an internationally acclaimed tunnel expert, who has considerable experience working with tunnel projects across the world. (e.g. Athens Metro, railway tunnels, motorway tunnels and hydraulic tunnels). He currently works with Qatar Rail and is involved in their underground tunnel infrastructure. Michalis is one of the prominent speakers at the 3rd Annual Underground Infrastructure and Deep Foundations Qatar, where he will be providing insight on jet grouting applications in weak rock conditions.

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