Original Research Paper

Simplified Analysis of the Lining Behavior During Earthquakes in Deep Rock Tunnels

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Abstract: The sizing of the lining of a tunnel requires to consider not only the static loads transmitted from the surrounding rock, but also the effects of earthquakes on the stresses and strains of the lining. The detailed evaluation of the interaction earthquake-lining using numerical methods typically requires very long calculation time and can be limited to the verification of the project final configuration. In this study a simplified procedure for the assessment of the effects of an earthquake on the tunnel lining is presented. This procedure is based on the calculation method Einstein and Schwartz and quickly allows the estimation of maximum moments induced by the earthquake in the lining. Through this procedure it was possible to develop a parametric analysis that allowed to evaluate the pseudo-static loads that is necessary to consider on the rock around the tunnel, according to the earthquake characteristics and to the geomechanic quality of the rock mass. A final calculation example illustrates the modalities for the analysis of a tunnel lining in the presence of a defined earthquake, using the charts developed in the present paper.

Keywords: Tunnel Lining, Deep Tunnel, Circular Tunnel, Dynamic Effect on the Tunnel, Earthquake Action on the Structure, Shear Wave, Concrete Lining, Shear Strain, Wave Propagation Velocity, Peak Particle Velocity, Peak Particle Acceleration

Introduction

The analysis of an earthquake effect on the concrete lining is an important stage of the tunnel design (Power *et al*., 2004). Sophisticated numerical methods allow assessing in the detail the interaction between seismic waves and tunnel linings (Karakostas and Manolis, 2000; Penzien and Wu, 1998; Penzien, 2000; Hashash *et al*., 2001; Do *et al*., 2015). The two and three dimensional numerical methods are widespread in the field of tunnels and underground works in general for many years (Do *et al*., 2013; Oreste, 2002; Do *et al*., 2014a; 2014b). These methods are, however, quite complex and may require very long computation times when the behavior of the lining and of the rock around the tunnel is analizedin the dynamic field.

The analytical methods, instead, generally provide a closed form solution or a simplified numerical solution, able to assess the stress and strain state of supporting structures or of the rock mass around the tunnel (Ribacchi and Riccioni, 1977; Lembo-Fazio and Ribacchi, 1986; Panet, 1995; Oreste, 2003a; 2003b;

2009a; 2008; Osgoui and Oreste, 2007; 2010) or forward to the excavation face (Oreste, 2009b; 2013; 2014a). In recent times, many analytical methods have been developed in the tunneling field, to resolve static problems of great importance (Oreste, 2007; Do *et al*., 2014c). The analytical methods require computing time remarkably low and permit, therefore, to develop parametric analyzes (Oreste, 2014b; 2014c), probabilistic ones (Oreste, 2005a) or back-analysis (Oreste, 2005b), all very useful in the design phase or during the construction of a tunnel. In these cases, the use of numerical modeling is limited only to the verification of the final configuration obtained by the dimensioning with analytical methods.

In this study a new simplified analytical procedure for the assessment of the effects of an earthquake on the stress state of the tunnel lining, considering the presence of pseudo-static loads, is presented. After analyzing the magnitude of induced strains around the tunnel by the shear waves due to an earthquake, the formulations to derive the maximum bending moments in the lining, using the Einstein and Schwartz (1979) method, are shown.

An extensive parametric analysis has then allowed to evaluate the pseudo-static load that is necessary to apply to the rock mass around the tunnel, to account for the presence of an earthquake, according to the intensity of the earthquake (Power *et al.*, 2004), the distance from the source and the GSI of the rock mass (Marinos *et al.*, 2005; Hoek *et al.*, 2013; Marinos and Hoek, 2000; Cai *et al.*, 2004). A calculation example, referring to a specific case, illustrates, finally, the procedures for verification of the lining, considering the loads both of static and dynamic nature (due to the presence of an earthquake).

Materials and Methods

In deep tunnels realized in a rock mass, the more dangerous seismic waves are represented by the shear waves. These waves have a peak particle velocity v_s, a peak particle acceleration a_s and a propagation speed C_m that can be obtained from the following equations (Power *et al.*, 2004):

$$a_s = 0.7 \cdot a_{max} \left[m / s^2 \right]$$ (1)

for tunnels at depths greater than 30 m

where, a_{max}: Maximum particle acceleration due to shear waves, detected on a rocky outcrop at the surface (Sadigh and Egan, 1998; Power *et al.*, 2004):

$$v_s = \beta \cdot \frac{a_s}{100 \cdot g} \left[m / s \right]$$ (2)

Where:

$$\beta \cong \left(0.0020 \cdot M_W - 0.0166 \right) \cdot$$
$$d^2 + \left(-0.0622 \cdot M_W^2 + 0.7844 \cdot M_W - 1.7771 \right) \cdot$$
$$d + \left(-2.8835 \cdot M_W^2 + 71.036 \cdot M_W - 279.47 \right)$$

g = 9.81 m/s²
M_W = Earthquake intensity
d = distance of the earthquake point source from the tunnel:

$$C_m = \sqrt{\frac{g \cdot G}{\gamma}}$$ (3)

Where:

$$G = \frac{E}{2 \cdot (1 + v)}$$

E: Elastic modulus of the rock mass; for the estimation of this value, is generally used the following formulation, which depends on the GSI, on the

disturbance factor D and on the uniaxial compressive strength of the intact rock σ_{ci} (Hoek *et al.*, 2002):

$$E = \sqrt{\frac{\sigma_{ci}}{100}} \cdot \left(1 - \frac{D}{2} \right) \cdot 10^{\left(\frac{GSI - 10}{40} \right)} \text{per} \sigma_{ci} < 100 \text{ MPa}$$

$$E = \left(1 - \frac{D}{2} \right) \cdot 10^{\left(\frac{GSI - 10}{40} \right)} \text{per} \sigma_{ci} \geq 100 \text{ MPa}$$

v = Poisson ratio of the rock mass
γ = Specific weight of the rock

In the case of deep tunnel and for wavelength larger than the tunnel diameter, on the basis of v_s and C_m is possible to obtain an estimate of the maximum shear strain induced in the rock mass around the tunnel (Power *et al.*, 2004):

$$\gamma_{s,max} = \frac{v_s}{C_m}$$ (4)

This maximum shear strain, when multiplied by the shear deformation modulus G, allows to obtain the shear stresses acting on the rock, or directly the normal stresses applied to the rock around the tunnel, according to the simplified load scheme of Fig. 1 (El Naggar *et al.*, 2008; Penzien and Wu, 1998):

$$q_{dyn} = \gamma_{s,max} \cdot G$$ (5)

The lining ovalization produced by the shear wave is critical when the direction of the wave is equal to λ=±π/4 with respect to the vertical direction. In that case the maximum moment that develops in the crown or on the sides of the tunnel due to the presence of an earthquake, is added to the maximum moment due to static loads.

The maximum moment produced by the load scheme of Fig. 1 can be evaluated with the method of Einstein and Schwartz (1979). This method provides a closed form solution of the stress state in the lining; it considers the medium around the tunnel as homogeneous and isotropic with an elastic behavior. The state of stress is applied to the ground at a great distance from the tunnel. The lining is represented by a closed ring with its own axial and flexural stiffnesses which depend on the constituent material elastic modulus and the lining thickness.

Adopting the solution of Einstein and Schwartz (1979), which refers to an allowed relative displacement lining-rock (full slip case), the maximum moment that develops in the crown (when the angle λ=±π/4) is obtained:

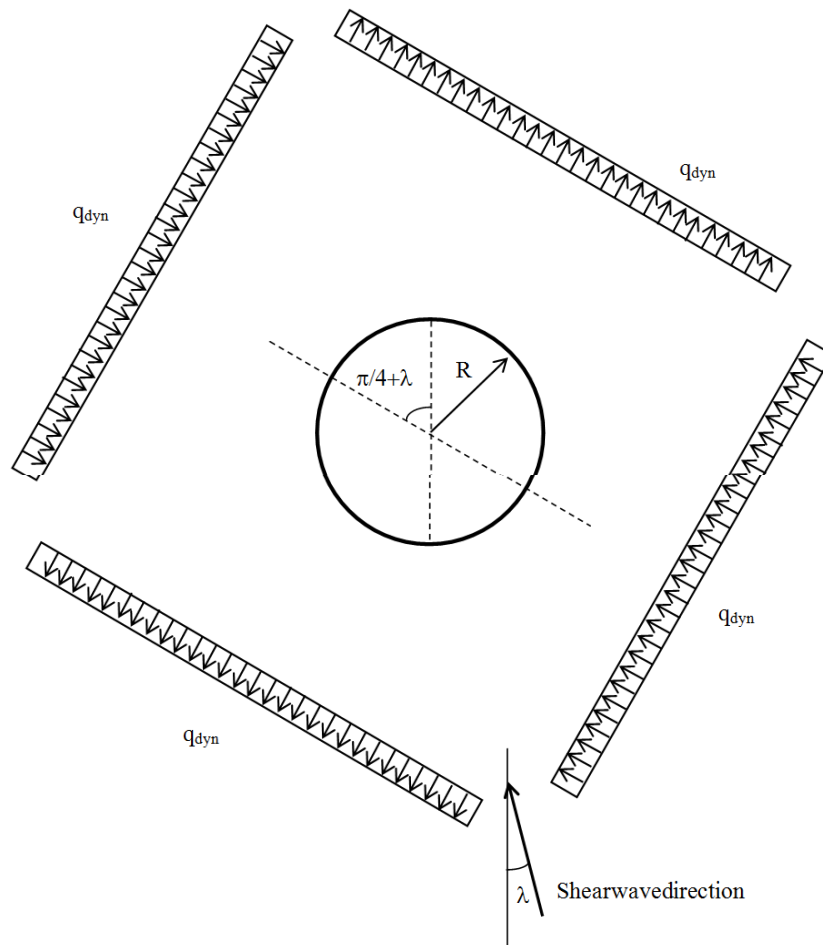$$M_{max} = q_{dyn} \cdot R^2 \cdot \left(1 - 2 \cdot a_2^* \right)$$ (6)

Fig. 1. Simplifiedload scheme of the rock around the tunnel, to simulate the dynamic effects of the presence of an earthquake on the lining of a circular and deep tunnel. Key: q_{dyn} is the normal stress that simulates the presence of an earthquake; R: The tunnel radius (El Naggar *et al.*, 2008; Penzien and Wu, 1998)

where, a_2^* is a dimensionless coefficient:

$$a_2^* = \frac{(F^* + 6) \cdot (1 - \nu)}{2 \cdot F^* \cdot (1 - \nu) + 6 \cdot (5 - 6 \cdot \nu)}$$

$$C^* = \frac{E \bullet R \bullet (1 - \nu_{sup}^2)}{E_{sup} \bullet A_{sup} \bullet (1 - \nu^2)} \quad F^* = \frac{E \bullet R^3 \bullet (1 - \nu_{sup}^2)}{E_{sup} \bullet I_{sup} \bullet (1 - \nu^2)}$$

A_{sup} = The area of the lining section: $A_{sup} = t_{sup} \cdot 1$

I_{sup} = The inertia moment of the lining section: $I_{sup} = t_{sup}^3 \cdot 1/12$

E_{sup} and ν_{sup} = The elastic modulus and Poisson's ratio of the material constituting the lining

According to the Einstein and Schwartz analytical method (Einstein and Schwartz, 1979) and for the assumptions made, in particular for the eligibility of lining-tunnel wall relative displacements (full slip case), the normal forces induced by the earthquake in the lining are everywhere equal to zero. That fact is a consequence of the simplified calculation model adopted: Actually values of normal force, even if low and negligible, can be detected even when it is allowed a lining-rock relative displacement.

The presence of an earthquake, therefore, could produce an increase of the positive moment in the crown, leading to an aggravation of the stress conditions of the lining.

It is, therefore, essential to be able to assess with a certain precision the effect of an earthquake on the stress state of the tunnel lining, with particular reference to what can happen on the crown, when the direction of the wave has an angle $\lambda = \pm \pi/4$ with respect to the vertical.

Results

In order to evaluate the trend value of the normal stresses q_{dyn} due to an earthquake varying the parameters most influential, a parametric analysis was performed

considering five different values of Geological Strength Index (GSI) (GSI = 40 to 80) (Marinos *et al.*, 2005; Hoek *et al.*, 2013; Marinos and Hoek, 2000; Cai *et al.*, 2004), three values of the maximum particle acceleration a_{max} (a_{max} = 0.4 to 0.6·g), 5 values of the earthquake intensity (MW = 6.5 to 8.5) and different values of the distance from the source (d = 10 to 100 km) (Power *et al.*, 2004).

In the parametric analysis has been considered a strength of the intact rock σ_{ci} variable with the GSI: σ_{ci} = 35 MPa for GSI = 40; σ_{ci} = 60 MPa for GSI = 50; σ_{ci} = 85 MPa for GSI = 60; σ_{ci} = 110 MPa for GSI = 70; σ_{ci} = 135 MPa for GSI = 80 (Hoek and Brown, 1980; 1997; Hoek, 2006; 2007). The disturbance factor D was considered equal to

zero, assuming a rock mass not disturbed by the excavation of the tunnel (Hoek *et al.*, 2002).

In Fig. 2-6 are shown the results of the parametric analysis.

It is noted as q_{dyn} tends to increase with the increase of the geomechanical properties of the rock mass (i.e., the value of GSI), with the increase of the distance from the earthquake source and obviously with the increase of the maximum particle acceleration on the surface and of the earthquake intensity. Figures 2-6 can be used for the estimation of q_{dyn} also for intermediate values of the influential parameters, different from those considered in the parametric analysis, through the use of the linear interpolation.
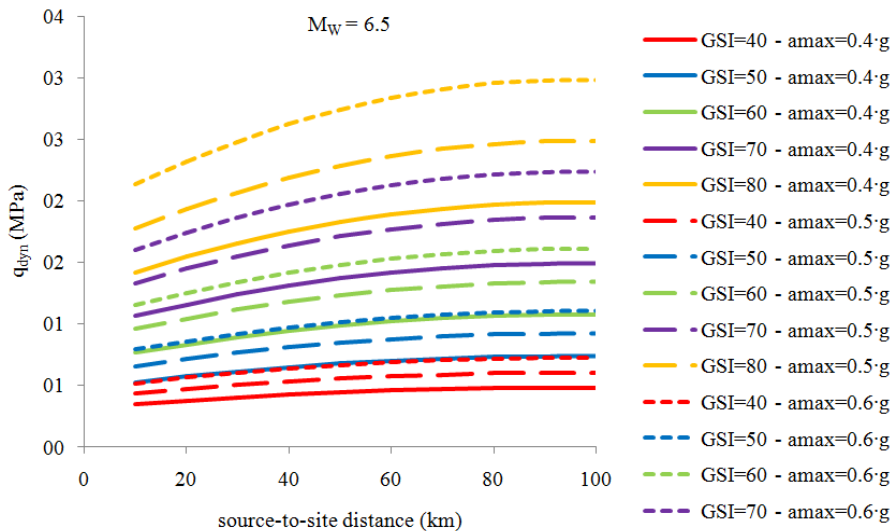

Fig. 2. q_{dyn} load trendvarying the distance d from the earthquake source, the GSI and the maximum particle acceleration on the surface, for an earthquake intensity of M_W = 6.5
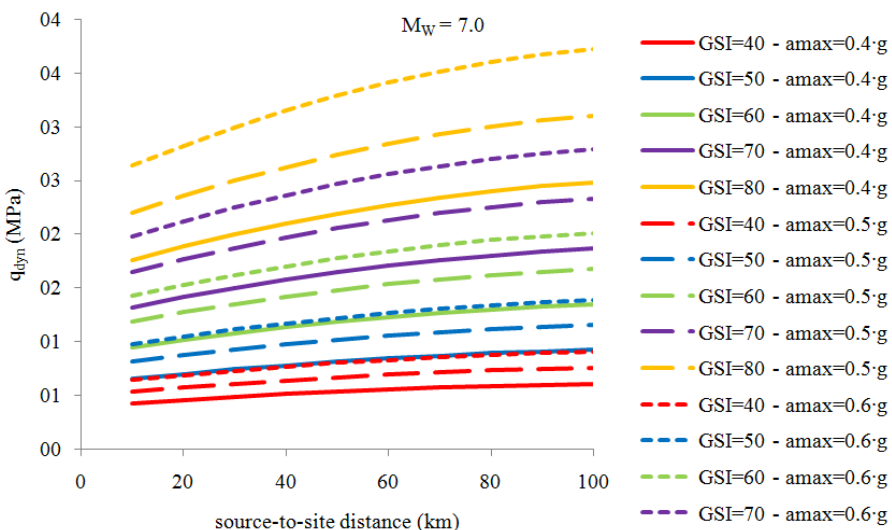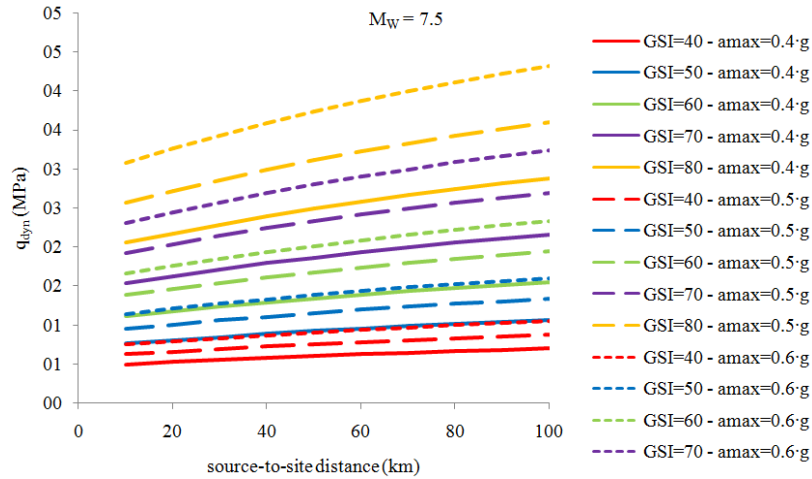

Fig. 3. q_{dyn} load trendvarying the distance d from the earthquake source, the GSI and the maximum particle acceleration on the surface, for an earthquake intensity of M_W = 7.0

Fig. 4. q_{dyn} load trendvarying the distance d from the earthquake source, the GSI and the maximum particle acceleration on the surface, for an earthquake intensity of $M_W = 7.5$
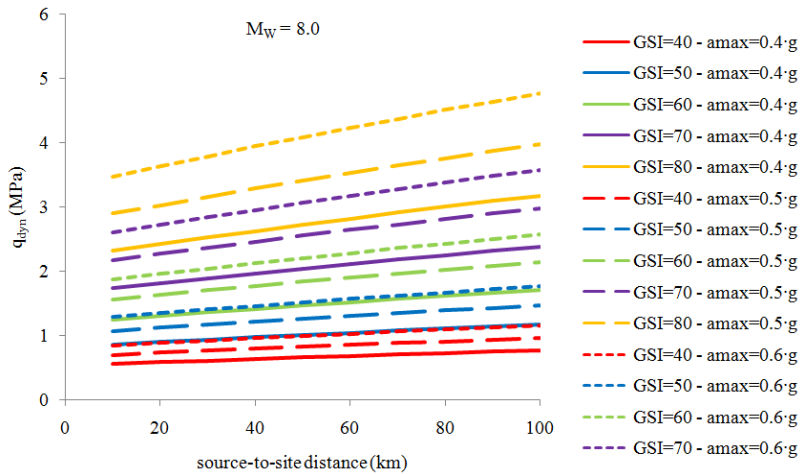

Fig. 5. q_{dyn} load trendvarying the distance d from the earthquake source, the GSI and the maximum particle acceleration on the surface, for an earthquake intensity of $M_W = 8.0$

Fig. 6. q_{dyn} load trend varying the distance d from the earthquake source, the GSI and the maximum particle acceleration on the surface, for an earthquake intensity of $M_W = 8.5$
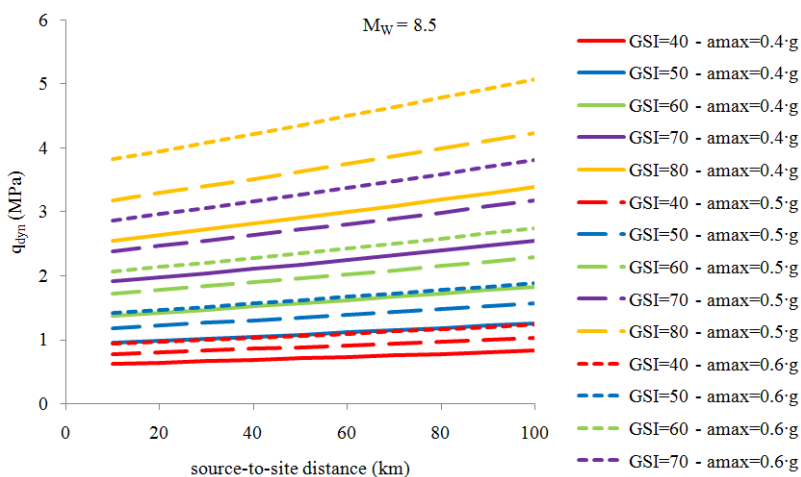
Discussion

From the carried out study, it is noted that the loads acting on the lining are greater when the geomechanic quality of the rock mass is better and, therefore, when its stiffness is higher. The dynamic effects of the earthquake are greater, then, when the static loads are lower. In fact, in a tunnel excavated in a rock mass of good quality, the static loads applied to the lining are generally modest, while the dynamic effects of an earthquake can produce loads that are likely to become critical for the stability of the lining and, therefore, also of the tunnel.

Also the tunnel radius is an important parameter. The moments due to the presence of a seismic wave, in fact, depend on the square of the tunnel radius.

In order to illustrate how to check the effect of an earthquake on a concrete lining of a circular and deep tunnel excavated in a rock mass, the following calculation example, based on the results of Fig. 2-6, is shown.

It refers to a tunnel radius of 3.5 m, excavated into a rock mass with GSI = 55 at a depth of 200 m (p_0 = 5.2 MPa). The uniaxial compressive strength of the intact rock is equal to 72 MPa and the rock mass disturbance factor D is equal to 0. The elastic modulus for the rock mass is estimated to be 11315 MPa (Hoek *et al*., 2002) and Poisson's ratio ν = 0.3. Also the following parameters for

cohesion (2.88 MPa) and friction angle of the rock mass (27°) (Hoek, 2006), relevant for the evaluation of the static load acting on the 40 cm thick concrete lining (without reinforcing), are evaluated. The study was conducted using the convergence-confinement method (Ribacchi and Riccioni, 1977; Oreste, 2003a; 2009a; 2014b; 2014c) and has led to an estimate of the static vertical load acting on the lining equal to 1.06 MPa.

Adopting this value of static load at the crown and a coefficient of lateral thrust k_0 = 0.5 in the Einstein and Schwartz (1979) method (full slip case), it was possible to estimate the maximum moment in the crown due to static loads ($M_{max, stat}$ = 0.0336 MN·/m), associated with a normal force N = 3.65 MN/m. Figure 7 shows the trend of the bending moment in the lining only due to the static load with a variation of ϑ angle.

Assuming thereafter an earthquake of intensity equal to M_W = 8.5, with a distance from the source point of 80 km and a maximum particle acceleration on the surface a_{max} = 0.5·g, from Fig. 6 a dynamic load q_{dyn} equal to about 1.85 MPa is obtained. From Equation 6 the maximum moment produced in the crown of the lining by the considered earthquake can then be evaluated: $M_{max,dyn}$ = 0.0369 MN·m/m. Figure 8 shows the trend of the bending moment in the lining only due to the dynamic load with a variation of ϑ angle.
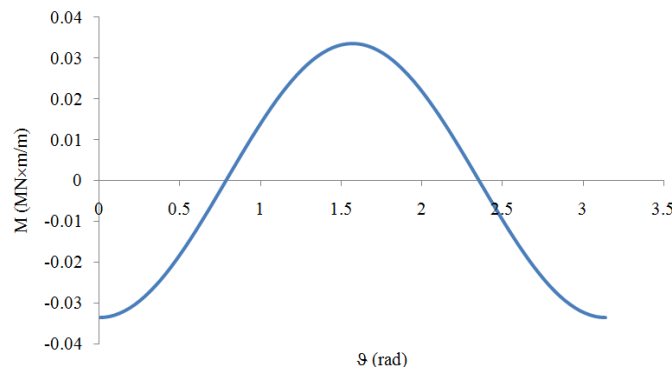

Fig. 7. Trend of the bending moment along the lining progress, for the studied case, with a variation of the ϑ angle (ϑ = 0 on the right side of the tunnel, ϑ increasing counterclockwise), in static conditions. The trend of the bending moment was calculated
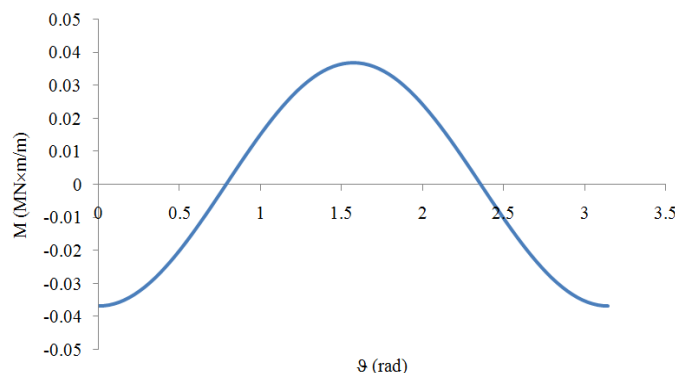

Fig. 8. Trend of the bending moment along the lining progress, for the studied case, with a variation of the ϑ angle (ϑ = 0 on the right side of the tunnel, ϑ increasing counterclockwise), in dynamic conditions. The trend of the bending moment was calculated by the Einstein and Schwartz (1979) method (full slip case)
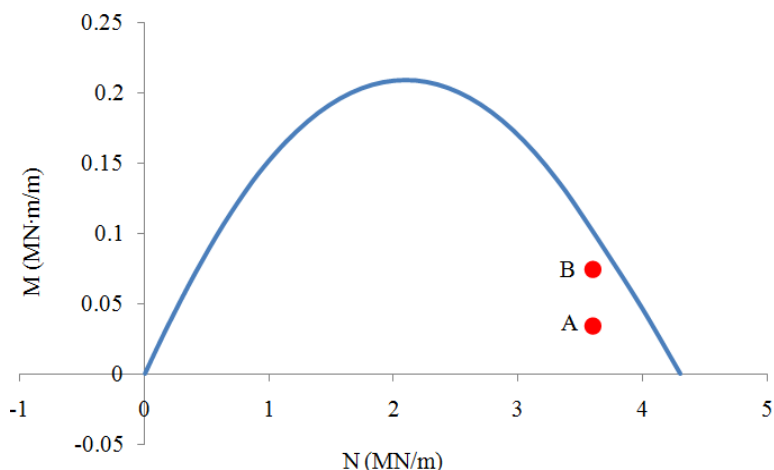
Fig. 9. Comparison of the lining stress state in static conditions (point A) and in static+dynamics conditions (point B), with the strength domain of the concrete section (0.4 m×1 m) in the absence of armature (blue solid line)

The value of the maximum moment due to dynamic loads is greater than the value of the maximum moment obtained in the static field. In order to verify the suitability of the assumed lining to bear the static and dynamic loads together, a comparison is necessary between the induced moments and axial forces obtained from the calculation and the concrete section strength (Fig. 9). In the present case and for the hypothesized earthquake, the assumed lining would be considered suitable to jointly withstand the loads produced by the earthquake and the static ones. Points A and B of Fig. 9 have the same value of the normal force, since the dynamic load produced by the earthquake had no effect on the axial forces that develop within the lining.

Conclusion

The verification of the ability of a tunnel lining to withstand the action of an earthquake is an important stage of the tunnel design. Detailed analysis of the interaction between the earthquake and the lining can be conducted with two-dimensional or three-dimensional numerical methods. These methods, when used in the dynamic field, may require very long calculation times and for this reason a detailed numerical dynamic analysis is limited to the final configuration identified by the project.

In the field of tunnels, very useful are the analytical calculation methods, since they allow to obtain useful results with reduced calculation times. For this reason they are often used to develop parametric analyzes, probabilistic ones or back-analysis.

In this study is presented a simplified analytical procedure for the evaluation of the effect of an earthquake on the lining of a tunnel excavated in rock mass. This procedure has also allowed to evaluate, through an extensive parametric analysis, the pseudo-static loads that is necessary to consider in the calculations according to the type of earthquake and the qualitative characteristics of the rock mass. These results have been summarized in some abacuses easy to use.

A final example has shown the modalities to estimate the maximum bending moments in a tunnel lining due to an earthquake, using the abacuses developed in this study.

Ethics

This article is original and contains unpublished material. The corresponding author confirms that all of the other authors have read and approved the manuscript and no ethical issues involved.

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