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SEISMIC ANALYSIS OF DEEP TUNNELS IN WEAK ROCK: A CASE STUDY IN SOUTHERN ITALY

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ABSTRACT

Underground structures are critical elements in transportation and utility networks and their importance makes the vulnerability to earthquakes a sensitive issue for the community. Underground facilities are usually less vulnerable to earthquakes compared with above-ground structures; but the associated risk may be very high since even a low level of damage may affect the serviceability of a wide network. The seismic behaviour of deep tunnels in rock is investigated in this paper, focusing on the necessary steps to be undertaken for a correct design under earthquake loading. These steps include: (i) characterization of the rock mass for stress-strain analysis in static conditions; (ii) static analysis; (iii) definition of the seismic input; (iv) dynamic analysis under seismic loading; (v) superposition of static and dynamic loads.

The primary objective of the paper is to improve the methods currently used for the seismic analyses of rock tunnels. To this purpose numerical analyses by the spectral element method involving the coupled effect of seismic source, propagation path, and dynamic soil-structure interaction have been performed by a suitable application of the Domain Reduction Method (DRM). The numerical method was used to study a deep tunnel located in Southern Italy. The infrastructure is part of the railway switch line connecting Caserta to Foggia in the northern sector of the Southern Apennines, one of the most active seismic regions in Italy. As a final step, the results obtained from advanced numerical analyses have been compared with those evaluated with simplified methods in order to assess their validity and limitations.

Keywords: Deep tunnels, Seismic analysis, Soil-structure interaction, Spectral Element Method

INTRODUCTION

Underground structures are critical elements in transportation and utility networks (e.g. railway and road tunnels, hydraulic tunnels and hydroelectric caverns, lifelines for transportation of oil, natural gas, etc.). The importance of these structures makes their vulnerability to natural hazards such as earthquakes a sensitive issue for the community. The seismic response of tunnels and in general of underground structures, is considerably different from that of above-ground facilities since the overall mass of the structure is usually small compared with the mass of the surrounding ground, and the

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stress confinement provides high values of radiation damping. As a matter of fact underground facilities are in general less vulnerable to earthquakes compared with above-ground infrastructures. However there are exceptions as numerous tunnels worldwide have been severely damaged by ground shaking. Recent examples include among others the 1995 Kobe (Japan), 1999 Chi-Chi (Taiwan) and 2004 Niigata (Japan) earthquakes. A careful review of the damages suffered by underground structures in these events shows that most tunnels were located in the vicinity of causative faults where the ground motion is characterized by strong and coherent (narrow band) long period pulses.

The analysis of seismic behaviour of a tunnel is a complex task since it involves the interaction with several disciplines including soil, rock and structural dynamics, structural geology, seismotectonics and engineering seismology. So far, relatively little efforts have been dedicated to this subject mainly because, as previously mentioned, underground structures are not considered particularly sensitive to earthquakes, so that tunnel engineers often omit the analysis of the tunnel performance under seismic conditions at the design stage.

This paper focuses on the seismic behaviour of deep tunnels in near field conditions. In particular the attention is devoted to the steps that need to be undertaken for a correct design of a deep tunnel under earthquake loading. These steps include: (i) rock mass characterization for stress analysis in static conditions; (ii) static analysis; (iii) definition of the seismic input motion; (iv) stress dynamic analysis under seismic loading; (v) superposition of static and dynamic loads. The static analysis has been performed using the finite difference-based code FLAC (Itasca, 2000), while the dynamic analysis has been carried out using of the spectral element-based code GeoELSE (Faccioli et al., 1997) coupled with a semi-analytical approach (Hisada & Bielak, 2003) using the Domain Reduction Method (DRM).

The method was used to study a deep tunnel located in the Southern Italy. The infrastructure is part of the railway switch line connecting Caserta to Foggia, in the northern sector of the Southern Apennines, one of the most active seismic regions in Italy.

Note that the mechanical parameters used for static and dynamic analysis are different due to the different level of deformation involved. The main issue of the static analysis is to reproduce the state of stress in the lining due to the excavation and stages of construction. On the other side, the dynamic analysis is aimed to evaluate the stress increment in the lining due to the seismic wave propagation. Static parameters are evaluated on the basis of classical rock mechanics methods, whereas for dynamic analysis the deformability parameters at low strain were used. In both static and dynamic analyses, the rock mass has been modelled as an isotropic continuum medium, since the behaviour of weak rock can be modelled as an equivalent continuum considering that the influence of discontinuities can be neglected.

Finally, the seismic response of a transverse cross-section of the tunnel computed by numerical analysis has been compared with that assessed with a closed-form analytical expression developed by Corigliano et al. (2006), with the aim to test the validity of both approaches. Calculation of ground response without the presence of the structure, a step required by both methods, has been performed using the semi-analytical Hisada & Bielak (2003) approach in both cases. As a consequence the comparison is performed using the same input motion, applied pseudo-statically in the closed-form solution in terms of an in plane shear strain.

THE CASE STUDY

The new "*Caserta-Foggia"* railway line is part of the doubling of the original line which represents one of the most important crossings of the Apennines in Southern Italy. The new railway line has been designed in the late 80's and includes 17 tunnels. The line between Caserta and Apice (a small town near Benevento) was built in the early 90's, whereas the last part of the line underwent a preliminary design stage only.

The focus of this paper is the study of the "*Serro Montefalco*" tunnel (see Figure 1) which belongs to the latter section of the "*Caserta-Foggia*". This is a 11.9 km long tunnel, with a maximum depth of 225 m. It represents one of the most relevant structures of the entire railway line due to the complexity of the geological context (see Figure 2). The lithotypes include varicoloured clay-shales, marl and marly limestone and clay and marl intercalated with limestone (Barla et al., 1986).

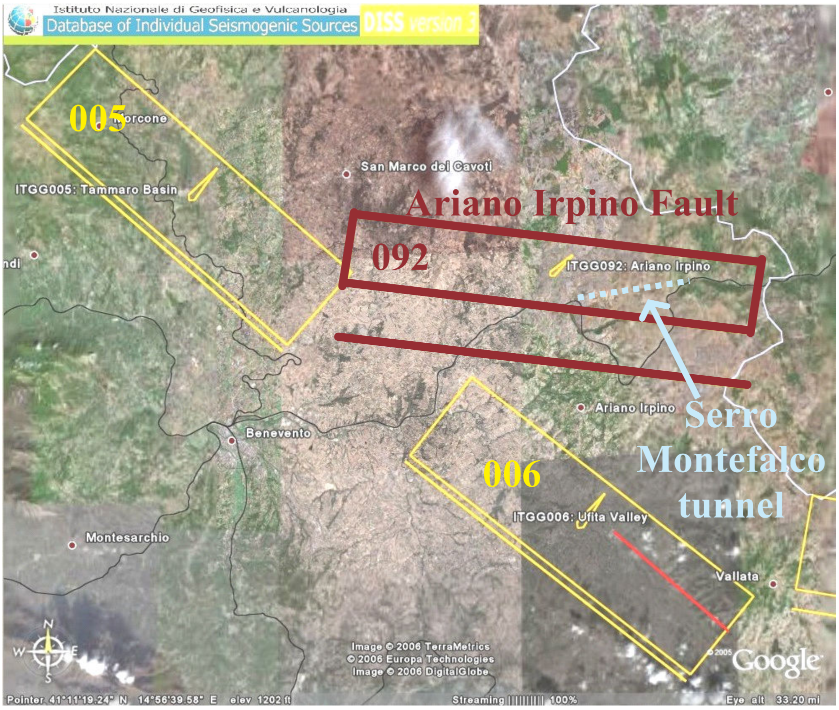


Figure 1. Location of the "*Serro Montefalco*" tunnel (dotted line) along the "*Caserta-Foggia*" railway line (dark solid line).The nearby active faults retrieved from the DISS 3.0.2 database are superimposed. The "*Ariano Irpino*" fault (ITGG092), which is assumed as a potential seismic source in the dynamic analysis of the tunnel, is highlighted. The short segment perpendicular to the tunnel axis, denotes the cross-section of the tunnel.

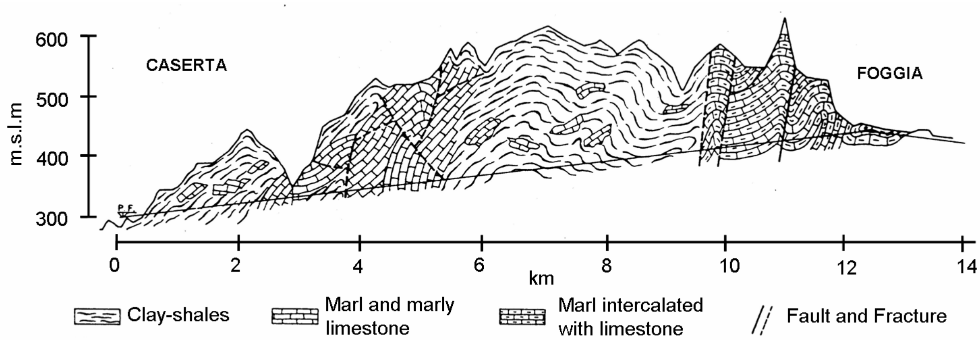


Figure 2. Geological profile along the "*Serro Montefalco*" tunnel (from Barla et al., 1986)

The varicoloured clay-shales (the so-called "*Argille Scagliose*") include expansive clay minerals which exhibit a significant swelling behaviour. Previous excavations of tunnels in this weak rock formation (e.g. the "*San Vitale*" Tunnel see Barla et al., 1986 and Lunardi & Bindi, 2004 for more details) were characterised by severe squeezing and swelling problems which lead to face instability, large convergences, invert-heave and critical loading of the tunnel support.
This case has been chosen because of availability of the tunnel design data associated with a relevant seismicity of the region. In fact the Southern Apennines are characterized by a narrow seismic belt, with NW-SE striking and about 30 to 50 km width following the axis of the mountain range (Improta et al., 2000). The Northern sector of the Southern Apennines, the "*Sannio*" region, is among the most active seismic regions in Italy. In this area five large earthquakes with I_{MCS}>X occurred in 1456, 1688, 1702, 1735 and 1805, causing several victims and severe damage. A long seismic quiescence since1805 event makes the area highly susceptible to a new earthquake.

STATIC ANALYSIS

An important step in assessing the earthquake-induced effect on underground structure is related to the correct evaluation of the state of stress in the lining after construction. This involves the accurate simulation of the construction stages in order to compute the cross-sectional forces in the lining. To this aim numerical analyses have been performed based on the geotechnical site characterization briefly described below.

Geotechnical parameters

The rock mass parameters along the tunnel length have been assessed on the basis of geomechanical classification and scaling rules of the intact rock properties obtained from laboratory tests. For soil-like materials (e.g. varicoloured clay-shales) reference has been made to laboratory testing (i.e. physical properties, CIU, CID triaxial tests, direct shear tests, edometric tests, etc.) of undisturbes samples obtained from borehole drilling and a limited number of field investigation such as dilatometer tests (Barla et al., 1986). The strength and deformability properties obtained for the varicoloured clay-shales formation are illustrated in Table 1.

Table 1 Strength and deformability parameters of varicoloured clay-shales (Barla et al., 1986)

Cohesion	Friction Angle	Young's Modulus	Poisson ratio
50 kPa	22°	200 MPa	0.45

Numerical analysis

Static analyses have been performed using the finite difference-based code FLAC (Itasca, 2000). A two-dimensional model of the tunnel under plane strain conditions (Figure 3b) has been used. In order to simulate the excavation stages, which determine 3-D stress-strain conditions near the tunnel face, the stress relaxation method proposed by Panet (1995) has been used. This introduces a fictitious stress $(\sigma)_c$ on the tunnel profile of the two-dimensional model as follows:

$$(\sigma)_c = (1-\lambda)(\sigma)_p \tag{1}$$

where λ is the stress relaxation factor ranging between 0 and 1, depending on the distance of the design cross section to the tunnel face, and $(\sigma)_p$ is the in situ state of stress.

The calculations have been performed with reference to an in-situ state of stress in the rock mass given by a vertical total stress σ_v = 2.14 MPa (corresponding to a depth of approximately 100 m) and with *at-rest* stress coefficient K_0 = 0.8. The rock mass has been modelled using an elastic-perfectly-plastic constitutive law, with the Mohr-Coulomb yield criterion and a non-associated flow rule (i.e. the rock mass dilatancy angle has been assumed to be zero). The rock mass parameters used in the analyses are listed in Table 1. The construction stages have been simulated through different steps as follows:

1. simulation of the initial in-situ state of stress;
2. full face excavation up to a 50 % removal of the stress on the tunnel profile;
3. application of the primary support at the crown and closure of the ring by the reinforced concrete lining installed at the invert;
4. additional removal up to 75% of the initial state of stress on the tunnel profile, with the above support system installed;
5. installation of final concrete lining at the crown and complete relaxation of the stress on the tunnel profile;
6. degradation of the material parameters of the primary support.

The primary support is composed of a ring consisting of a 30 cm thick shotcrete layer and steel ribs (1 HEB 200 per meter). The primary support has been modelled by using plane strain elements with a linearly elastic isotropic behaviour, characterized by an equivalent Young's modulus (Oreste, 1999).
The mechanical parameters used for the primary support are listed in Table 2.

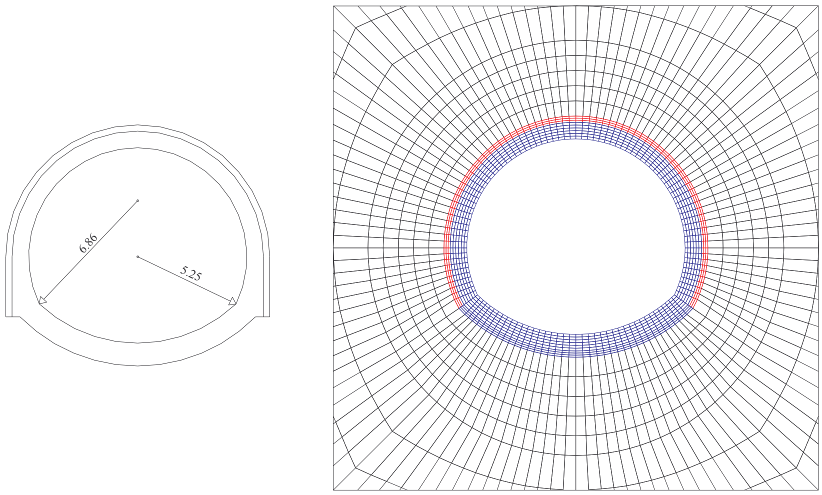
Table 2. Mechanical parameters of the primary support

Shotcrete	**Thickness =** 30 cm	$\nu = 0.2$	E_{shot} = 4000 MPa	
Steel ribs HEB200	A_{steel} = 78.1 cm^2	**D** =1.0 m	E_{steel} = 210000 MPa	J_{steel} = 5696 cm^4
Equivalent parameters	**G** = 3932 MPa	ν = 0.2	E_{eq} = 9438 MPa	**K** = 5243 MPa

The final reinforced concrete lining which will act as a permanent structural support of the tunnel will carry the whole load derived from the static analysis plus the additional earthquake loading. A typical cross section of the tunnel including the primary and final lining adopted in the varicoloured clay-shales formation is shown in Figure 3a). The final reinforced concrete lining has been modelled using linear elastic, isotropic plane strain elements with the material parameters reported in Table 3.
The cross-sectional forces acting along the lining are shown in Figure 4 and Figure 5.

Table 3. Mechanical parameters of final lining

E	γ	ν	K	G	Thickness at the crown	Thickness at The invert
30 GPa	25 kN/m^3	0.2	16.67 GPa	12.50 GPa	80 cm	110 cm



a) b)

Figure 3. a) Cross section of the tunnel considered in the analysis; b) FLAC mesh discretization

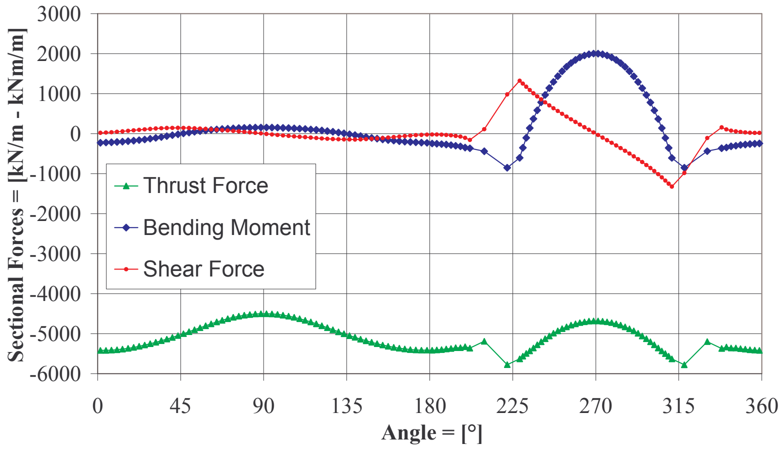
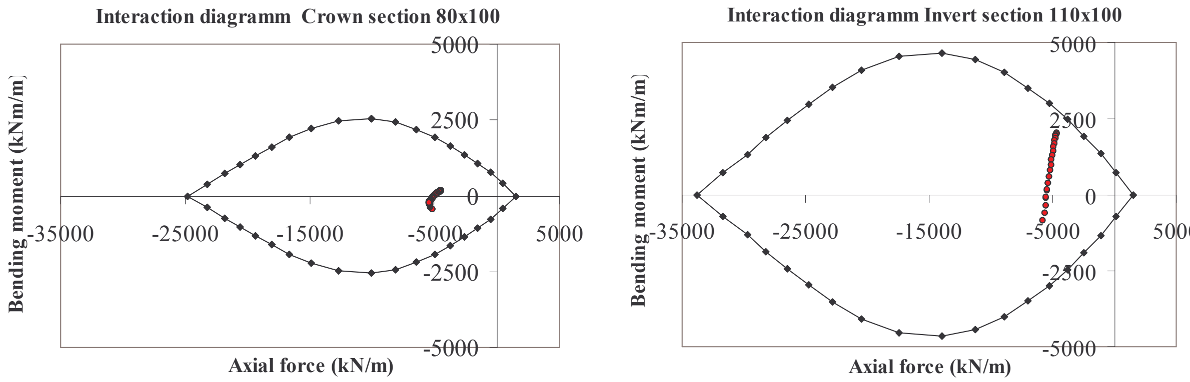


Figure 4. Cross-sectional forces acting on the reinforced concrete lining in the (final stage)

Figure 4 shows that the state of stress in the lining is governed by the construction stages. In the final step this involves large bending effects at the invert. Note that the state of stress at the invert, as derived from the static analyses is approaching the limit design values for the cross-section (see Figure 5b). As a result, underground structures excavated in complex geotechnical conditions, like the "*Serro Montefalco*" tunnel, may undergo significant damages if an additional loading increment (such as that induced by an earthquake) is superimposed. Furthermore, it should be pointed out that for highway and railway tunnels even a low level of damage may affect the serviceability of an entire network with great impact on the emergency services dispatched immediately after an earthquake.

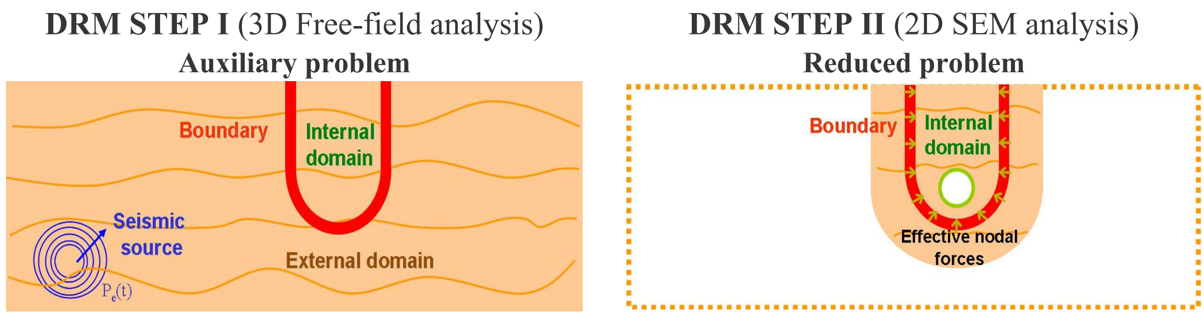


a) b)

Figure 5. Interaction diagrams for the tunnel cross-section at crown a) and at invert b)

DYNAMIC ANALYSIS

A complete analysis of the seismic problem has been performed, which includes simultaneous consideration for the seismic source, the propagation path, the geological site conditions, and the soil-structure interaction. The Domain Reduction Method (Bielak et al., 2003), a powerful substructuring approach, has been applied to reduce the computational effort required by such a large scale numerical problem. The main idea of the Domain Reduction Method (DRM) is the subdivision of the original problem into two simpler problems each solved in two independent steps. The first problem (see Figure 6a) accounts for earthquake source and propagation path and it is solved with a model that includes both the source and a background structure (external domain) from which the structure has been removed and replaced by the same material as the surrounding soil. The second problem (see Figure 6b) simulate with the desired accuracy only a region (internal domain) of the domain of interest which includes the structure and the surrounding soil, but not the causative fault. The input for the solution of the reduced problem is a set of effective nodal forces calculated from solving the auxiliary problem and applied in a narrow strip of elements (see dark boundary in Figure 6). These forces are equivalent to (and replace the) original seismic forces computed in the first step. A detailed description of the implementation of the DRM for applications with the Spectral Elements Method can be found in Faccioli et al. (2005) and Stupazzini et al. (2006).



a) b)

Figure 6. Scheme of the DRM procedure applied to the case study: a) free-field analysis of the source and the wave propagation; b) wave propagation in the reduced domain including soil-structure interaction. The dark line denotes the effective boundary

The main feature of this approach is the possibility of coupling solutions obtained by different methods in two different domains. In this study the auxiliary problem (external domain) has been solved using the 3D by a *semi-analytical* method developed by Hisada & Bielak (2003). The reduced

problem (internal domain) was instead solved by means of a 2D *numerical* analysis using the Spectral Element Method (SEM).
The semi-analytical method developed by Hisada & Bielak (2003) allows to investigate the effects of *fling step* and *rupture directivity* on the computed near-fault ground motion. This method is based on the computation of static and dynamic Green's functions of displacements and stresses for a viscoelastic horizontally layered half space. It takes advantage of an analytical expression for the asymptotic solutions of the integrands of the Green's functions, stemming from the generalized R/T reflection and transmission coefficient method and of the stress discontinuity representations for boundary and source (i.e. kinematic model of the source) conditions respectively. For the case under study only the dynamic contribution of the Green's functions is taken into account.
In the second step, the SEM (Faccioli et al, 1997) was used to simulate the 2D wave propagation and seismic soil-structure interaction along the cross-section of the "*Serro-Montefalco*" tunnel (see Figure 1). The SEM is a powerful numerical technique naturally suited for seismic wave propagation and dynamic soil-structure interaction analyses, that is implemented in the numerical code GeoELSE (Geo-ELasticity by Spectral Elements) jointly developed at Dipartimento di Ingegneria Strutturale of Politecnico di Milano and CRS4 (Center for Advanced Studies, Research & Development in Sardinia) Cagliari. The most recent version of the code includes: (i) the capability of dealing with fully unstructured computational domains, (ii) a parallel architecture, (iii) implementation of the DRM.

Domain Reduction Method: Step I
A 3D analysis of the seismic source and the wave propagation in the external domain has been carried out using the method developed by Hisada & Bielak (2003). The method requires definition of a crustal model for the region of interest as well as a model of the seismic source.

Crustal profile
The geological structure of the "*Sannio*" region is rather complex and characterized by strong lateral heterogeneities in the upper 4 km of the earth crust. Several authors have proposed models of crustal velocity models for the Southern Apennines, especially after the Irpinia earthquake in 1980. Improta et al. (2000) give an interpretation of the crustal seismic refraction data from the Northern Sector of the Southern Apennines thrust belt. Geophysical data were acquired along a 75 km seismic array parallel to the Apennines mountain range. This allowed the definition of a detailed two-dimensional P-wave velocity model of the upper crust.
The velocity model is well constrained by sonic velocity logs obtained from oil wells located in close proximity to the seismic array and gravity data. This profile has been adopted as a generalised crustal model for the "*Sannio*" region. Since this model is too rough in the shallow part of the earth crust (due to the fact that only two layers in the first 5 km from the free surface are used), it has been adapted to fit the soil profile proposed by Cotton et al. (2006) based on the V_{S30} parameter. In this case it was selected a value of V_{S30} = 600 m/s to gradually merge with the V_S profile at greater depths.
The adopted S-wave profile is shown in Figure 7. Since the active fault considered in this study reaches a depth of 25 km and the adopted crustal model is defined only down to 13 km depth, the latter has been extended in depth following the less detailed model proposed by other authors (i.e. Chiarabba & Amato, 1997).

Source model
Based on the database of individual seismogenic sources (DISS 3.0.2) developed for Italy by the *Istituto Nazionale di Geofisica e Vulcanologia* (INGV), the "*Sannio*" region where the "*Serro Montefalco*" tunnel is located, is characterized by three relevant seismogenic structures. They are the "*Ariano Irpino*" fault, "*Ufita Valley*" fault and "*Tammaro Basin*" fault and they are coded as ITGG092, ITGG006, and ITGG005, respectively, by DISS 3.0.2 database (see Figure 1).We have selected the reactivation of the "*Ariano Irpino*" fault as a potential seismic scenario, because it is the closest one to the tunnel and it is characterized by an expected maximum magnitude of 6.9. It was the source of the December 5, 1456 earthquake, one of the most significant seismic events of the Italian seismic history. The main features of this source as well as the seismological parameters adopted for the numerical simulation of this earthquake scenario are summarized in Table 4.

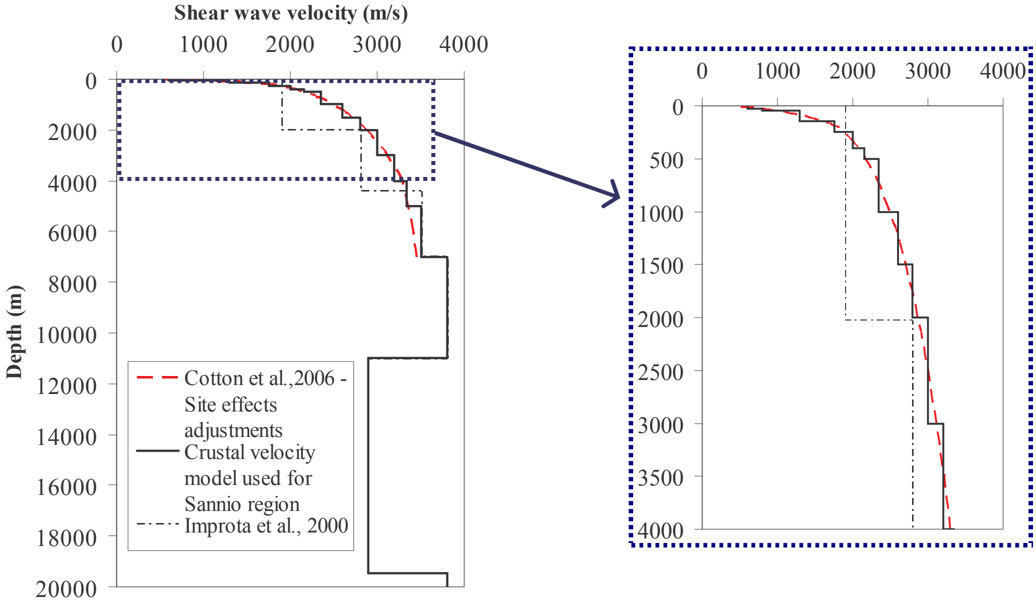


Figure 7. Crustal velocity profile adopted for the solution of the auxiliary problem

Table 4. Features of the Ariano Irpino (ITGG092) fault (from DISS V3.0.2)

Seismic Moment	M_w	L	W	Slip	Strike	Dip	Rake	Min. Depth	Max. Depth	Hypo. Depth	Rupture Vel.	Rise time	Max. freq.
Nm	-	km	km	m	°	°	°	km	km	km	km/s	s	Hz
2.54×10^{19}	6.9	30	14.9	2	277	70	230	11	25	22.7	2.8	1.8	5

Ground response

Figure 8 shows the ground motion computed with the code developed by Hisada & Belak (2003) at a coupled of points along the two-dimensional cross-section of the "*Serro-Montefalco*" tunnel. As expected, the ground motion is nearly the same at all points of the region of the tunnel and this due to the large value of the wavelengths with respect to the dimensions of the tunnel.

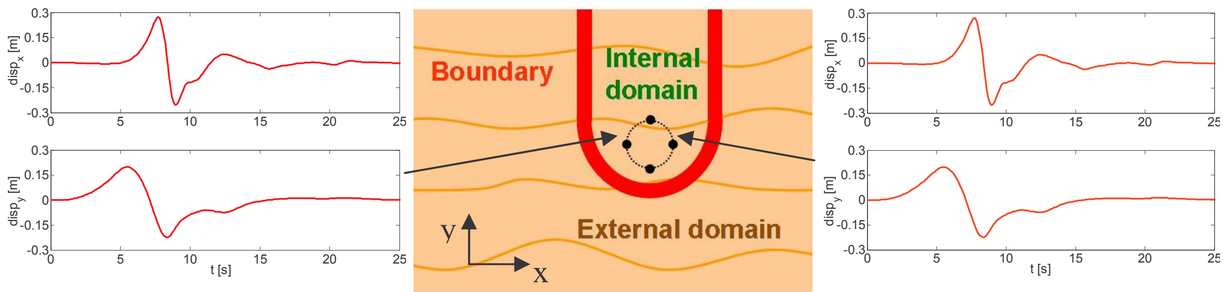


Figure 8. Free-field displacement time histories calculated with the Hisada-Bielak (2003) method

The numerical simulation of the earthquake has been performed by propagating frequencies up to 5 Hz. An example of velocity time histories computed at the tunnel depth is shown in Figure 9 together with the Fourier amplitude spectra. The reached frequency content is concentrated between 0 and at most 2÷3 Hz. The peaks in the Fourier amplitude spectra confirm the narrow band characteristics of the near-fault ground motion.

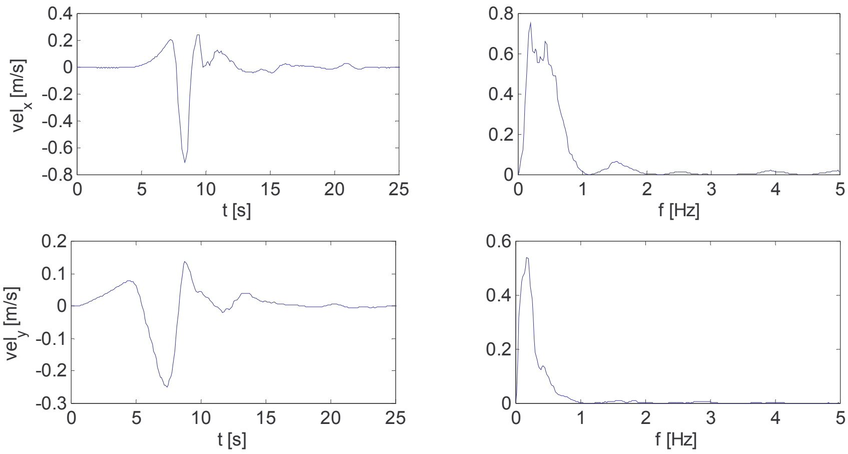
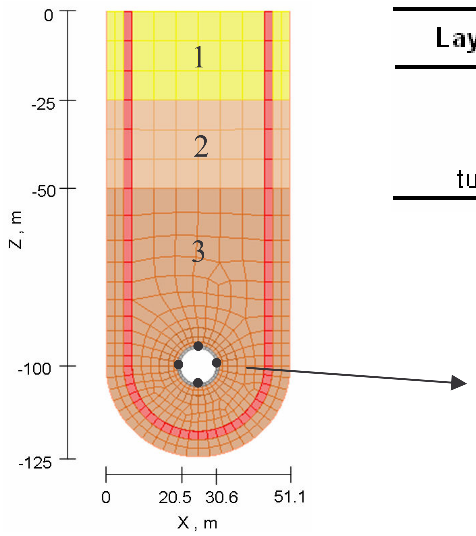


Figure 9. Free-field velocity time histories calculated with the Hisada-Bielak (2003) method and Fourier amplitude spectra computed at the tunnel point highlighted in the previous figure.

Domain Reduction Method: Step II

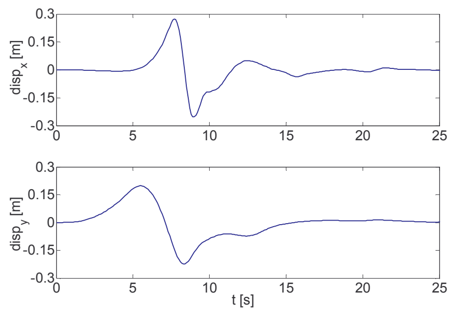
Equivalent nodal forces have been computed along the effective boundaries of the reduced model using the ground motion calculated in the first step evaluated along the effective boundary. These effective forces exactly reproduce the wave field generated by the extended seismic source. These have been computed by taking into account only the vertical and horizontal displacements in the cross-section plane since a 2D model was used. The dynamic analysis has been performed considering a simplified circular tunnel characterized by an equivalent external radius of 5.85 m and a thickness of 0.80 m. The Young's modulus and Poisson's ratio of the lining are reported in Table 3. The reduced problem has been discretised by spectral elements to propagate frequencies up to 70 Hz, as shown in Figure 10a). The rock dynamic properties are shown in Figure 10b). The no slip condition at the soil-structure contact has been considered.

Domain Reduction Method: Step II (2D SEM model)



Layer No	V_S (m/s)	V_P (m/s)	ρ (t/m³)	Q_S
1	600	1039	1.80	30
2	800	1386	1.90	50
3	1300	2252	2.10	75
tunnel	2236	3651	2.50	80

b)



a) c)

Figure 10. Step two of the analysis: a) SEM model of the tunnel cross-section; b) dynamic properties adopted for the reduced model; c) displacements evaluated along the tunnel lining

It is important to remark that the tunnel slightly influences the response of the system with respect to the first step. However, the presence of the structure is irrelevant to the wavefield due to the large propagating wavelengths (on the order of 400 m to 3200 m and more) with respect to the transverse dimension of the tunnel (10.1 m of diameter). It should be considered nevertheless that the extended

kinematic description of the source places severe limitations to the capabilities of the model of reproducing the high-frequency content of the radiated wavefield. In the present case the highest frequency reproduced by the source model is limited to 2÷3 Hz.
The maximum dynamic increment of the thrust force and the bending moment obtained by the SEM approach will be compared with the results obtained with the simplified analytical solution.

A CLOSED-FORM SOLUTION FOR PSEUDO-STATIC SIMPLIFIED ANALYSES

The Pseudo-static analysis of the transversal response has been performed by applying the closed form solution proposed by Corigliano et al. (2006). This solution considers a lined circular tunnel (diameter 5.85 m and thickness 0.80 m) under plane strain conditions and models the earthquake loading as a uniform, quasi-static strain field simulating a pure shear deformation. The relationships for the thrust force Q and bending moment M per unit length of tunnel lining for seismic design associated with the no-slip condition between the lining and the surrounding ground are given by the following relations:

$$Q = \frac{E_g}{2(1+\nu)} \gamma_{ff\,\max} \mathrm{R} \left(1 - \frac{\delta}{3}\right) \cos\left[2\left(\vartheta + \frac{\pi}{4}\right)\right] \tag{2}$$

$$M = \frac{1}{2} \frac{E_g}{2(1+\nu)} \gamma_{ff\,\max} \mathrm{R}^2 \left[1 + \frac{\delta}{3} + \varepsilon\right] \cos\left[2\left(\vartheta + \frac{\pi}{4}\right)\right] \tag{3}$$

where the parameters ε, *δ, β, C** and *F** are defined by:

$$\varepsilon = \frac{{}^{3C^*}\!/_{F^*} \left\{\left[C^*(1-\nu)-2\right] - 3\left[C^*(1-\nu)+2\right]\right\} + \left\{\left[C^*(1-\nu)-2\right] + \left[C^*(1-\nu)+2\right]\right\}\left[1+C^*(1-\nu)\right]}{C^*(1-\nu)\left\{\left[C^*(1-\nu)+4\nu\right] - 2\left[C^*(1-\nu)+2\right]\right\} + {}^{3C^*}\!/_{F^*}\,\beta + \left[C^*(1-\nu)+4\nu\right] - 2\nu\left[C^*(1-\nu)+2\right]} \tag{4}$$

$$\delta = \frac{\left[C^*(1-\nu)-2\right] - \left[C^*(1-\nu)+4\nu\right]\varepsilon}{\left[C^*(1-\nu)+2\right]} \qquad \beta = \left[C^*(1-\nu)+4\nu\right] - 6\left[C^*(1-\nu)+2\right](1-\nu) \tag{5}$$

$$C^* = \frac{E_g \mathrm{R}\left(1-\nu_s^2\right)}{E_s A_s\left(1-\nu^2\right)} \qquad F^* = \frac{E_g \mathrm{R}^3\left(1-\nu_s^2\right)}{E_s I_s\left(1-\nu^2\right)} \tag{6}$$

where R is the average tunnel radius, A_s and I_s are area and moment of inertia per unit length of the lining respectively, E_g, E_s, ν and ν_s are the Young's modulus and Poisson's ratio of ground and lining respectively. The parameters *C** and *F** are the "compressibility" and "flexibility" ratios. They represent a measure of the relative stiffness of the ground with respect to the supporting system (i.e. the lining) under a symmetric and antisymmetric loading respectively (Einstein & Schwartz, 1979). Finally $\gamma_{ff\ max}$ is the maximum shear strain (in absolute value) calculated in free-field conditions. Knowing $\gamma_{ff\,max}$, the imposed stress can be easily computed as follows:

$$\sigma = \tau = \frac{E_g}{2(1+\nu)} \gamma_{ff\,\max} \tag{7}$$

Equations (2) and (3) show that the maximum shear strain in free-field conditions is a key parameter for the definition of the stress in the tunnel lining. To compute the earthquake-induced shear strain field in the vicinity of a causative fault, the displacement time histories *v(y, z)* and *w(y, z)* at four points around the cross section of the tunnel were calculated using the semi-analytical method proposed by Hisada & Bielak (2003). By denoting the direction along the tunnel axis as "x", the shear strain γ_{yz} may be computed as follows (Corigliano et al., 2006):

$$\gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y} \cong \frac{1}{2\,\Delta z}\left[v(y_o, z_o + \Delta z) - v(y_o, z_o - \Delta z)\right] + \frac{1}{2\,\Delta y}\left[w(y_o + \Delta y, z_o) - w(y_o - \Delta y, z_o)\right] \tag{8}$$

in which the partial derivatives are evaluated using the second order central finite difference operators. The maximum shear strain obtained for the present case study is $\gamma_{ff\,max} = 1.39{\cdot}10^{-4}$, as shown in Figure 11.

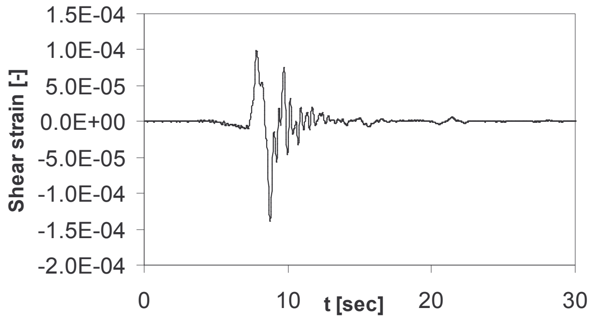


Figure 11. Free field shear strain time histories at tunnel depth

Note that a rough estimate of the maximum shear strain could be computed using the following well-known expression (Newmark, 1967) relating the peak strain (PGS) to the peak particle velocity PGV:

$$PGS = \frac{PGV}{C} \tag{9}$$

where C denotes either the apparent speed of propagation velocity of S-waves in the horizontal direction (V_{Sapp}) or the prevailing phase velocity of Rayleigh waves (V_R). Using an attenuation relationship valid in near field conditions (e.g. Bray & Rodriguez-Marek, 2004) for the PGV, we obtained values ranging from about 50 to 55 cm/s. For C the range of values reported in literature (see O'Rourke, 2003) varies from 2.1 to 5.3 km/s with an average value of 3.4 km/s. Using the average value one would obtain PGS = $1.47{\cdot}10^{-4} \div 1.61{\cdot}10^{-4}$ in good agreement with the maximum value obtained by the Hisada & Bielak (2003) method.

Figure 12a) and b) show the comparison between thrust force and bending moment respectively obtained by means of numerical and the simplified pseudo-static method.

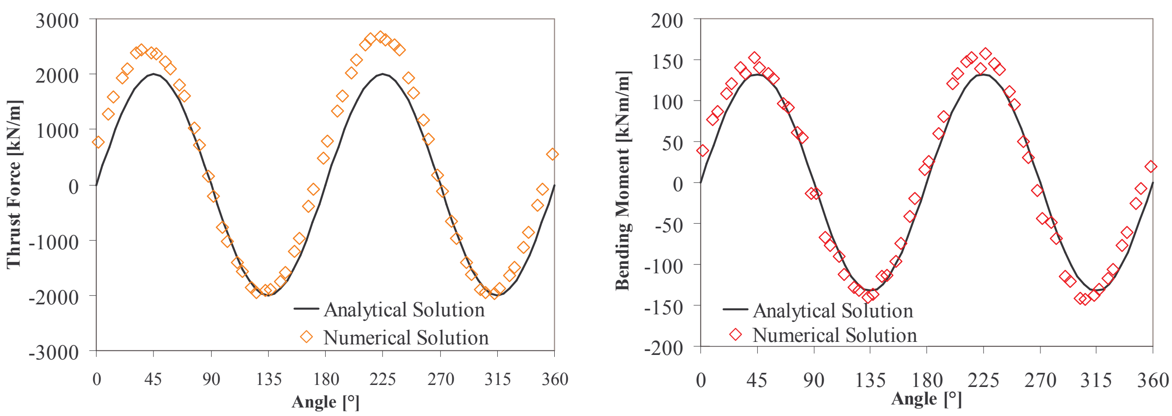
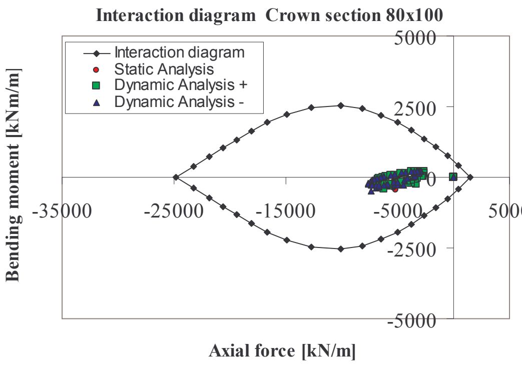
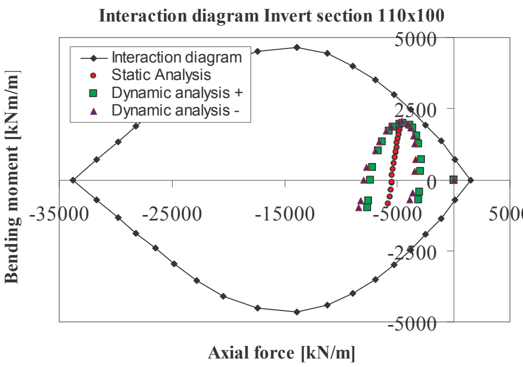


Figure 12. Comparison between numerical (rigorous) and analytical (simplified) solution: a) thrust force; b) bending moment

The figures show that the agreement between the two solutions is good. In particular, the thrust force computed using the numerical analysis exceeds that calculated by the simplified method by approximately 30%. It is important to remark that the bending moment and the thrust force illustrated in Figure 12 represent the dynamic increment of the internal forces. The total moment and thrust force are obtained by adding the dynamic increments to the static values as shown in Figure 13. It turns out that also under the very heavy loading conditions considered, representative of an earthquake with minimum recurrence period of 2000 years, the failure level is not exceeded even though it is very close. It is clear that this is to be taken into account for appropriate design of the final lining of the tunnel. The same consideration cannot be true for un-reinforced tunnels.



a)



b)

Figure 13. Interaction diagram for the cross-section at crown a) and at invert b) adding the dynamic stress increment

CONCLUDING REMARKS

The objective of this paper is throwing some light onto the seismic behaviour of a deep tunnel located in the vicinity of seismogenic faults. The study focuses both on static and dynamic response. Scope of the static analysis is to correctly compute the state of stress in the lining after the phases of construction, whereas the dynamic analysis attempts to calculate the dynamic stress increment of the lining internal forces due to earthquake loading.

For tunnels bored in complex geological formations, like the "*Argille Scagliose*", the static analysis yielded a relatively high state of stress in the lining thereby rendering the tunnel vulnerable to damage in case of earthquake occurrence.

A complete analysis of the seismic problem involves simultaneous modelling of the seismic source, the propagation path, accounting for near-source geological conditions and the soil-structure interaction.

The solution of this complex problem has been carried out by means of Geo-ELSE a Spectral Element-based Method (SEM) equipped with the Domain Reduction Method (DRM). As expected the dynamic soil-structure interaction effects are rather weak, due to the large wavelengths characteristic of near-fault wave propagation with respect to the tunnel size.

Good agreement is obtained between the results of advanced numerical analysis and those calculated using a simplified approach. The thrust force computed by dynamic analysis exceeds by about 30% the value obtained using the simplified approach. Note that tunnels excavated in complex geotechnical conditions, like the "*Serro Montefalco*" tunnel, can suffer damage if a loading increment due to the most severe earthquake scenario considered for that area is added.

From this case study it can be concluded that the simplified approach gives reasonable results from an engineering point of view. Studies are currently underway to investigate the use of a simpler method to compute the earthquake-induced maximum shear strain in the ground under free-field to further simplify the pseudo-static approach in the seismic analyses of underground structures.

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