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# ANALYSIS OF TUNNEL BEHAVIOUR UNDER SEISMIC LOADS USING DIFFERENT NUMERICAL APPROACHES

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## ABSTRACT

In this paper different approaches aimed at investigating the dynamic behaviour of circular tunnels in the transverse direction are presented. The analysed cases refer to a shallow tunnel built in an ideal soft clayey deposit excited by four different acceleration time histories characterised by significantly different frequency content. The adopted approaches include: 1) one-dimensional (1D) numerical analyses performed modelling the soil as a single phase visco-elastic non-linear medium, the results of which are then used to evaluate the input data for selected analytical solutions proposed in the literature (uncoupled approach); 2) 2D fully coupled Finite Element simulations adopting visco-elastic and visco-elasto-plastic constitutive assumptions for the soil and the lining (coupled approach); 3) 2D quasi-static Finite Element simulations adopting the same constitutive hypotheses of the fully coupled dynamic analyses. The three approaches, when a proper calibration of the soil parameters is undertaken, provide comparable results in the case of the visco-elastic assumption for the soil. The different constitutive hypotheses adopted in the 2D Finite Element simulations prove to play a significant role on the seismic-induced loads and curvature of the lining.

Keywords: Seismic behaviour, tunnels, constitutive models, numerical modelling

## INTRODUCTION

The dynamic response of tunnels to seismic actions can be assessed by means of uncoupled or coupled approaches. The approach is uncoupled when the evaluation of the seismic wave propagation and the corresponding actions on the structure is undertaken in two separated steps, while it is coupled if at this scope a unique analysis is carried out. For both the approaches the constitutive model employed to describe the mechanical behaviour of the ground can be "simple" or "advanced".
In this paper a set of 1D and 2D FE analyses of tunnel behaviour during seismic loading are presented by comparing simple and advanced numerical approaches.
The simplest uncoupled approach consists in 1D site response analyses performed modelling the soil as a single phase visco-elastic non-linear medium and in the application of the computed ground shaking parameters as input data for selected analytical solutions proposed in the literature to estimate the seismic-induced loads on the lining.
A coupled approach require the seismic wave propagation and soil-lining interaction to be modelled in one single analysis. In this paper 2D FE simulations are carried out performing fully-dynamic and quasi-

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static analyses of the same idealised problem. In these analyses both visco-elastic and visco-elasto-plastic constitutive laws are adopted for the soil.
The numerical analyses are aimed at establishing the benefits and the limits of the investigated numerical approaches and the relevance of the different constitutive assumptions on the computed ground deformation and loads acting in the transversal section of the tunnel lining during the seismic wave propagation. The analysed cases refer to a shallow tunnel built in an ideal soft clayey deposit excited by four different acceleration time histories characterised by significantly different frequency content.

## OUTLINE OF THE IDEALIZED PROBLEM

A 60-m thick ideal deposit of soft clay is assumed as the reference soil profile. The physical properties and mechanical parameters are reported in Table 1. The water table is assumed at the ground surface.
The assumed profile of the small-strain shear stiffness $G_0$ with depth (Figure 1) was calculated adopting the relationship proposed by Viggiani (1992):

$$\frac{G_0}{p_r} = S \cdot \left(\frac{p'}{p_r}\right)^n \cdot R^m \tag{1}$$

where $p_r$ is a reference pressure taken equal to 1 kPa, $p'$ is the mean pressure, $S$, $n$ and $m$ are parameters depending on the plasticity index $I_P$ and $R$ is the overconsolidation ratio in terms of mean effective stress. The values of $S$, $n$ and $m$ are also summarised in Table 1.
For sake of simplicity, the small-strain damping ratio $D_0$ was considered constant with depth.
A circular tunnel, located at 15 m depth and with a 10.10 m diameter, is selected as the reference underground structure for the present case study. The lining is assumed to be composed by 0.50 m thick reinforced concrete ring, characterised by a characteristic compressive cubic strength of the concrete $R_{ck} = 45$ MPa and by the following linear visco-elastic parameters: Young's modulus $E_l = 38$ GPa, Poisson's ratio $\nu_l = 0.25$, damping ratio $D_l = 5\%$.

**Table 1. Physical and mechanical parameters of the soft clay deposit.**

| **Parameters:** | |
|---|---|
| Plasticity index $I_P$ (%) | 44 |
| Unit weight of volume $\gamma$ (kN/m$^3$) | 17 |
| Coefficient of earth pressure at rest $K_0$ | 0.60 |
| Overconsolidation ratio in terms of mean effective stress $R$ | 1.5 |
| Small-strain shear stiffness $G_0$ (MPa) | variable with depth (Figure 1) |
| Small-strain damping ratio $D_0$ (%) | 1.0 |
| Poisson's ratio $\nu'$ | 0.25 |
| Cohesion $c'$ (kPa) | 0 |
| Friction angle $\varphi'$ (°) | 24 |
| Parameter of Eq. (1) $S$ | 600 |
| Parameter of Eq. (1) $n$ | 0.820 |
| Parameter of Eq. (1) $m$ | 0.360 |

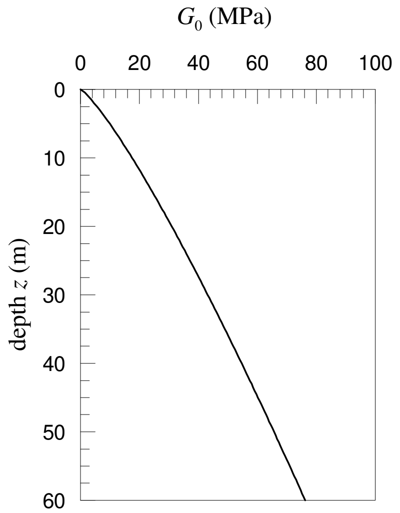


**Figure 1. Profile of the small-strain shear stiffness $G_0$.**

In the present study four different acceleration time histories were considered, namely Kalamata, Gilroy 2, Tarcento and Port Island. A summary of ground motion main characteristics is given in Table 2. All input signals were scaled to 0.35g, in order to study the soil-tunnel interaction under severe seismic conditions, and filtered to prevent frequency levels higher than 12 Hz. This latter frequency was selected in order to limit the minimum element dimension adopted in the Finite Element analyses. The selected acceleration time histories after manipulation are given in Fig. 2 while the corresponding Fourier amplitude spectra and acceleration response spectra are shown in Fig. 3 and 4 respectively. It is evident that the seismic signals are characterised by significantly different frequency content, in order to be representative of a wide range of possible events.
The input seismic signal was considered applied at the rigid base of the deposit.

**Table 2. Main characteristics of the adopted input seismic motions.**

| Station | Comp. | Earthquake | Subsoil | PGA (g) | Duration (s) | Dominant frequency $f_p$ (Hz) |
|---|---|---|---|---|---|---|
| Tarcento | NS | Friuli (Italy), 1976 | Rock | 0.21 | 16.85 | 10.10 |
| Gilroy 2 | 50 | Coyote Lake (USA), 1979 | Stiff | 0.20 | 18.00 | 5.00 |
| Kalamata | X | Kalamata (Greece), 1986 | Stiff | 0.24 | 29.75 | 1.63 |
| Port Island | 90 | Kobe (Japan), 1995 | Reclaimed Land | 0.28 | 42.00 | 0.91 |

## 1D EQUIVALENT-LINEAR VISCO-ELASTIC GROUND RESPONSE ANALYSIS

The 1D ground response analyses were performed by the code EERA (Bardet *et al*. 2000). The code is based on the assumption of equivalent-linear visco-elastic soil behaviour.

Modulus reduction curve $G/G_0$ and variation of damping ratio $D$ with shear strain level $\gamma$ were defined according to typical results reported in the literature (Vusetic and Dobry 1991) as a function of $I_P$ (Figure 5).

A total number of 39 layers were assumed to discretise the profiles of stiffness and damping ratio with depth: 7 top layers of 0.5 m thickness, followed by 10 of 1 m, 8 of 1.5 m, 8 of 2 m, 5 of 3 m and 1 base layer of 3.5 m. In the iterative procedure, the ratio of effective and maximum shear strain is assumed equal to 0.5.

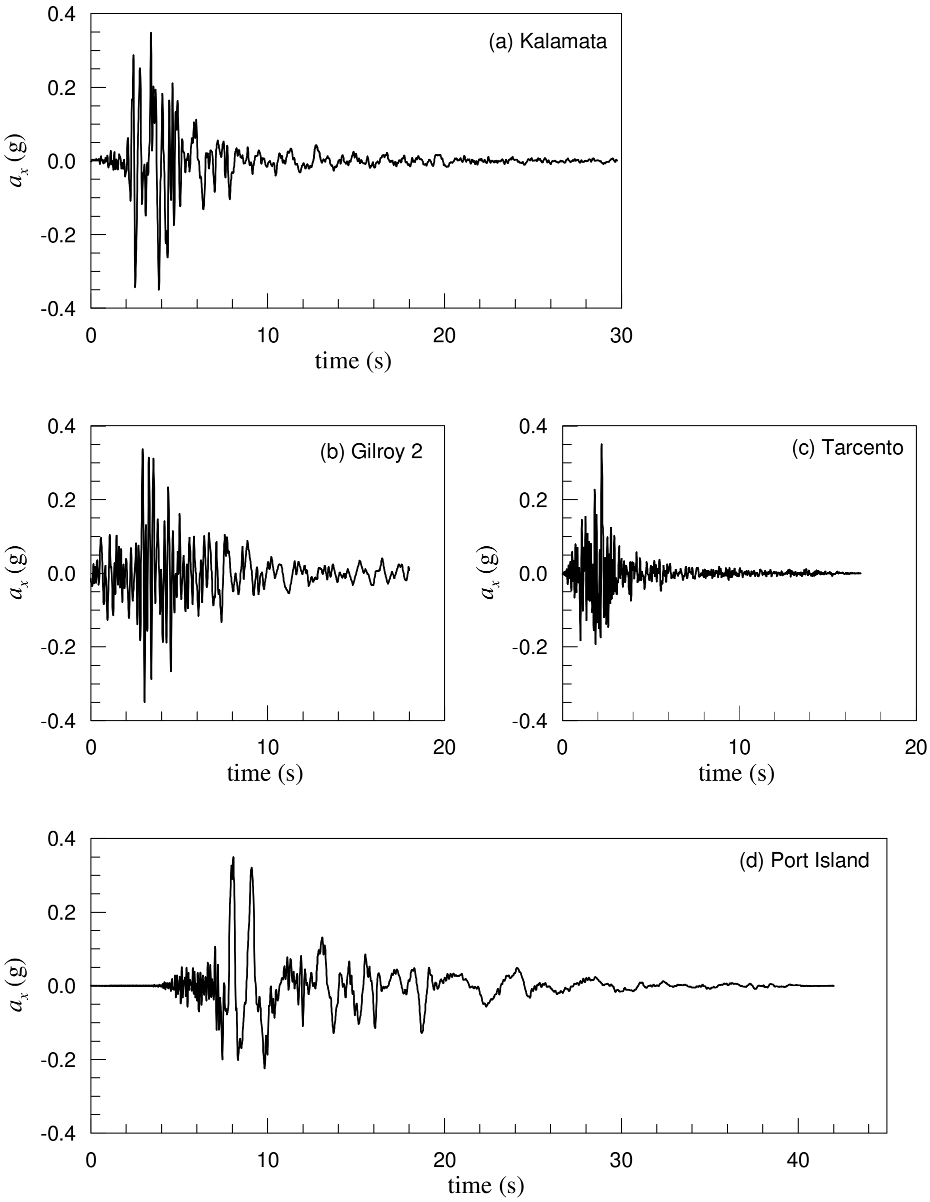


**Figure 2. Plot of the four selected acceleration time histories scaled at 0.35g: a) Kalamata, b) Gilroy 2-050, c) Tarcento e d) Port Island.**

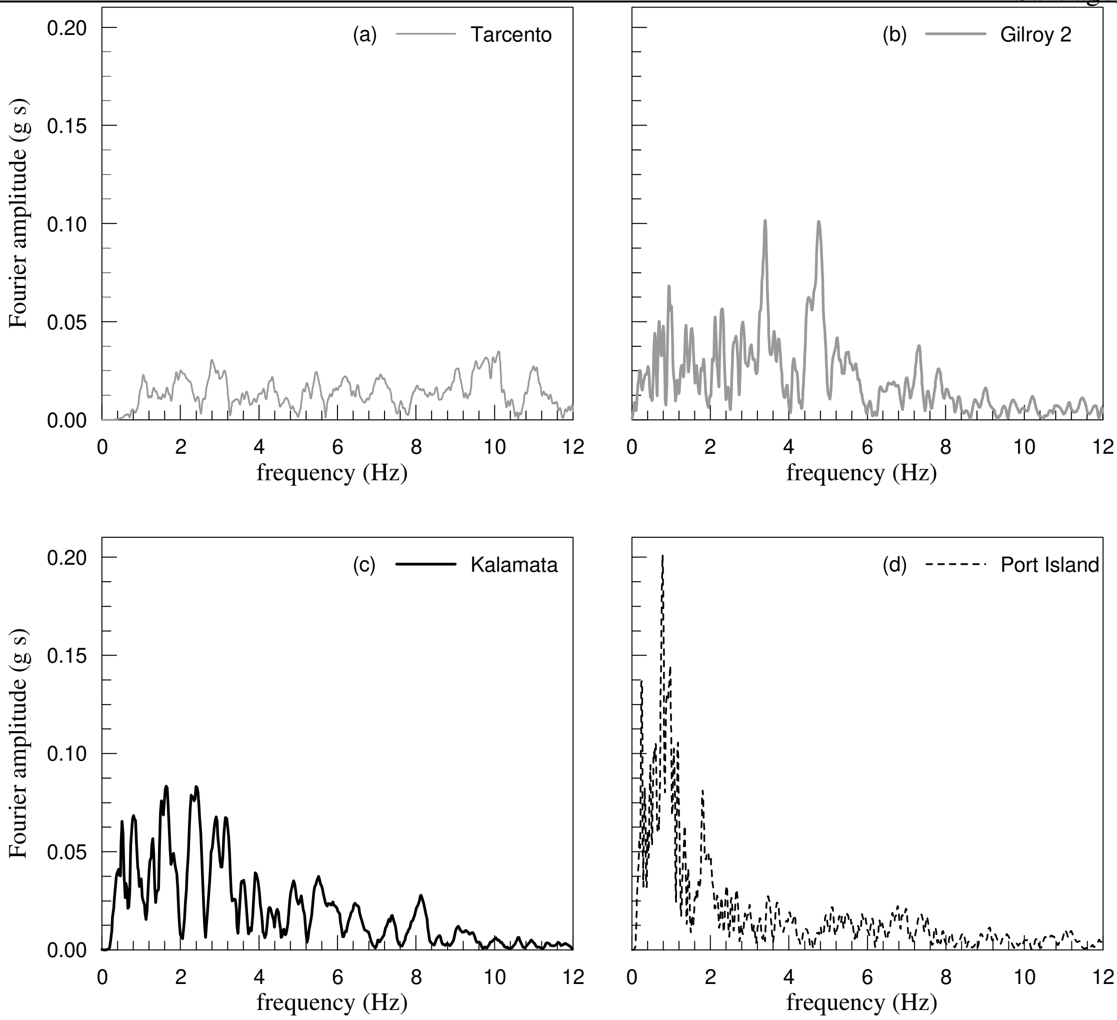


**Figure 3. Frequency-filtered Fourier amplitude spectra of the four selected acceleration time histories.**

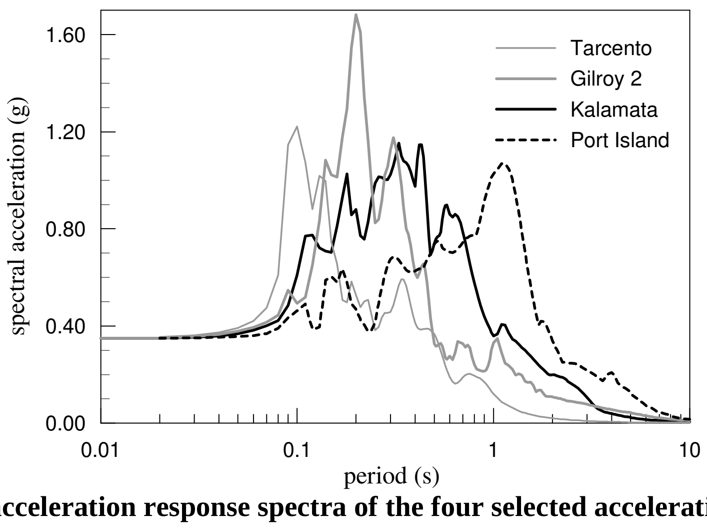


**Figure 4. Elastic acceleration response spectra of the four selected acceleration time histories.**

Figures 6 shows the results of the analysis in terms of maximum shear strain $\gamma_{max}$, normalised shear stiffness $G/G_0$, damping ratio $D$ and maximum acceleration $a_{max}$. Values of $\gamma_{max}$ and $G$ obtained at the depth of 15 m, i.e. at the tunnel depth, were subsequently used to evaluate the increments in the hoop force and bending moment in the tunnel lining during the earthquake, according to selected analytical solutions discussed in the next paragraph.

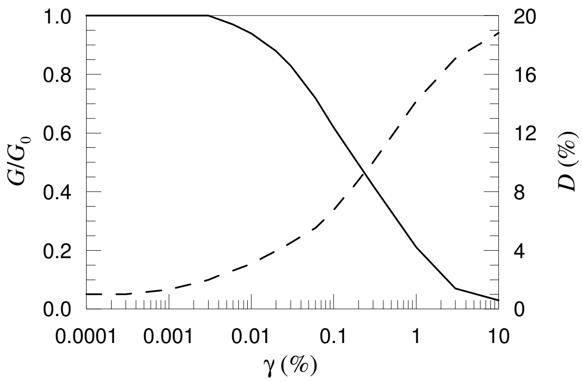


**Figure 5. Modulus reduction curve $G/G_0$ and variation of damping ratio $D$ with shear strain $\gamma$.**

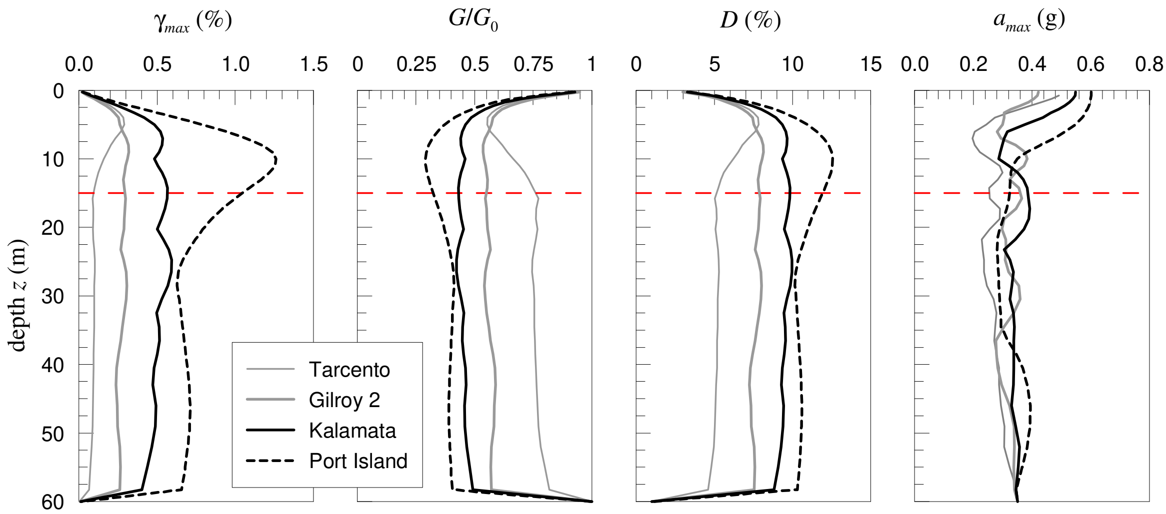


**Figure 6. Results of the 1D ground response analyses performed with EERA**

## ANALYTICAL SOLUTIONS

Here, the closed-form solutions summarised in Wang (1993) to predict the transverse response of the tunnel are adopted, according to what suggested by Hashash *et al.* (2005). These solutions take explicitly into account the soil-structure interaction effect under both no-slip and full-slip conditions. They are based on the following assumptions:

- the ground is an infinite, elastic, homogeneous and isotropic medium;
- the tunnel and the lining are circular and the lining thickness is small in comparison to the tunnel diameter.

Seismic actions are considered as external static forces acting on the tunnel lining, induced by the ground distortion related to a vertically propagating shear wave. The resulting ovalisation of the tunnel lining is assumed to occur under plane strain conditions.

According to Wang (1993), the flexibility ratio $F$ is the most important parameter to quantify the ability of the lining to resist against the distortion imposed by the ground:

$$F = \frac{E_u \left(1 - \nu_l^2\right) r^3}{6 E_l I \left(1 + \nu_u\right)} \tag{2}$$

where $E_u$ and $\nu_u$ indicate the mobilised soil Young's modulus (evaluated with reference to the previously calculated mobilised shear modulus $G$) and the Poisson's ratio (assumed equal to 0.5) in undrained conditions, respectively. $t$ and $I$ the thickness and the moment of inertia of the tunnel lining, respectively.
For the investigated cases $F$ ranges between 0.966 and 2.271, i.e. the flexural stiffness of the lining is between one and two times the flexural stiffness of the excavated soil material inside the tunnel cavity. In these cases no relevant slippage between the soil and the tunnel lining is expected.
Table 3 summarizes the increments in the hoop force and bending moment of the tunnel lining computed for both full-slip and no-slip conditions. Increments in the hoop force, as expected, are significantly higher in the no-slip case. Increments in the bending moment coincide, irrespectively of the different slippage conditions assumed in the analyses.

**Table 3. Increments of hoop force and bending moment according to Wang (1993).**

| **Earthquake** | *F* | **full-slip conditions** | | **no-slip conditions** | |
|---|---|---|---|---|---|
| | | $\Delta N_{\mathrm{max},min}$ (kN/m) | $\Delta M_{max,min}$ (kNm/m) | $\Delta N_{max,min}$ (kN/m) | $\Delta M_{max,min}$ (kNm/m) |
| Tarcento | 2.271 | ±32.2 | ±162.4 | ±136.8 | ±162.4 |
| Gilroy 2 | 1.641 | ±90.8 | ±458.4 | ±329.5 | ±458.4 |
| Kalamata | 1.287 | ±157.9 | ±797.4 | ±517.8 | ±797.4 |
| Port Island | 0.966 | ±254.9 | ±1287.5 | ±754.9 | ±1287.5 |

## 2D FE NUMERICAL MODELLING

The coupled numerical analyses were performed with the Finite Element code PLAXIS 2D (2003), a two-dimensional (plane strain and axisymmetric) code that implements the coupled Biot dynamic equations (Biot 1941) adopting the so-called *u-p* simplification (where $u$ is the skeleton displacement and $p$ the pore pressure), assuming as negligible the fluid acceleration relative to the solid skeleton.
The code adopts the Generalized Newmark method (Katona and Zienkiewicz 1985) for the time integration under dynamic conditions. In this case the following standard values of the Newmark's constants were selected in all the analyses illustrated in this paper: $\alpha_N = 0.3025$ and $\beta_N = 0.6000$. Those values ensure that the algorithm is unconditionally stable, while being dissipative mainly for the high-frequency modes.
In the dynamic solution the code allows to introduce frequency dependent viscous damping by means of the Rayleigh formulation, the damping matrix being defined as follows:

$$[C] = \alpha_{\mathrm{R}} [M] + \beta_{\mathrm{R}} [K] \tag{3}$$

where $[M]$ and $[K]$ are the mass and the stiffness matrix of the system, respectively. The coefficients $\alpha_R$ and $\beta_R$ are obtained considering the following relationship with the damping ratio $D$ (e.g. Clough and Penzien 1993):

$$\begin{Bmatrix} \alpha_R \\ \beta_R \end{Bmatrix} = \frac{2D}{\omega_n + \omega_m} \begin{Bmatrix} \omega_n \omega_m \\ 1 \end{Bmatrix} \tag{4}$$

where $\omega_n$ e $\omega_m$ are the angular frequencies related to the frequency interval $f_n \div f_m$ in which the viscous damping is equal to or lower than $D$.

The boundary conditions adopted for the static stages of the analyses were the standard ones: nodes at the bottom of the mesh were fixed in both vertical and horizontal directions, while those along the lateral sides were only fixed in the horizontal direction. In the dynamic analyses the bottom of the mesh was assumed to be rigid and the lateral sides were characterised by the viscous boundaries proposed by Lysmer and Kuhlemeyer (1969), with parameters $a = 1.0$ and $b = 0.25$.

In order to perform a comparative analysis with the EERA results, a linear visco-elastic constitutive model for the soil was first selected in the dynamic stage of the analyses, coupling a linear isotropic elastic model and the Rayleigh viscous formulation. Plasticity was then added, leading to a non-associated visco-elasto-plastic constitutive assumption characterised by a Mohr-Coulomb yield criterion and a null dilatancy angle.

In this paper, a recently developed calibration procedure of the visco-elastic parameters to be assumed in the dynamic FE analyses is adopted (Amorosi *et al*. 2010). $G$ and $D$ profiles are set in such a way to match the corresponding profiles resulting from the free-field 1D analysis performed by the code EERA, based on the assumption of equivalent-linear visco-elastic soil behaviour. Adopting the proposed procedure, a reasonably good matching between the EERA and the FE viscoelastic analysis results was achieved at each depth for all the investigated cases, both in terms of frequency response and acceleration time histories (see Amorosi *et al.* 2010 for further details).

The structural elements adopted to simulate the tunnel lining were assumed to be linear visco-elastic plates, formulated according to the Mindlin theory (e.g. Bathe 1982). Impervious interface elements were also introduced to model the interaction between the lining and the soil, according to the formulation summarised in the manual of the code. In particular, the interface was characterised by values of the shear strength parameters equal to those of the surrounding soil: such an assumption can be considered as corresponding to the no-slip condition of the Wang's solutions.

The mesh employed in the present study is reported in Fig. 7: it is characterised by a width equal to 8 times its height, in order to minimize the influence of boundary conditions on the computed results (Amorosi *et al*. 2010). The domain was discretised in a total number of 2431 15-node plane strain triangular elements.

In the central part of the mesh, where the tunnel is located, the characteristic dimension of the elements $h$ always satisfies the condition:

$$h \le h_{max} = V_S /(6 \div 7) f_{max} \tag{5}$$

where $V_S$ is the shear wave velocity and $f_{max}$ is the maximum frequency of the seismic signal (equal to 12 Hz).

The domain was partitioned into 20 horizontal layers to account for variable stiffness and damping parameters with depth. A detail of the mesh around the tunnel is shown in Fig. 8.

All the analyses were carried out performing a set of initial static stages, to simulate the tunnel excavation, the installation of the lining and the subsequent consolidation stage, followed by the dynamic stage, during which the seismic signal was applied at the bottom of the mesh, and a final static post-seismic consolidation stage.

In particular, the simulation of the tunnel excavation was performed in undrained conditions by imposing a volumetric contraction of the tunnel section corresponding to a volume loss of 0.4 %. This value was selected as representative of a satisfactory performance of the tunnel excavation stage for a shallow tunnel in clayey material. The following installation of the tunnel lining was also carried out under

undrained conditions, while in the subsequent consolidation stage the previously cumulated excess pore water pressures were allowed to dissipate, leading to the pre-seismic reference state of the system. The dynamic stage was then carried out under undrained conditions, adopting a time step equal to 0.01 s or lower (0.05 s for Tarcento earthquake), corresponding to the time interval at which the seismic signal is recorded. A post-seismic consolidation analysis was finally performed, to evaluate the long term effects of the seismic event on the tunnel section.
The elastic soil shear stiffness moduli $G$ assumed in the static stages of the analyses were selected scaling down the corresponding initial values $G_0$ according to the normalised modulus reduction curves shown in Fig. 5, assuming an average mobilised shear strain level equal to 0.1%.
All the static stages of the analyses were also characterised by the assumption of elasto-plastic behaviour of the soil, irrespectively of the hypotheses holding for the dynamic stages. This was aimed at reproducing the same pre-seismic conditions for all the dynamic analyses.

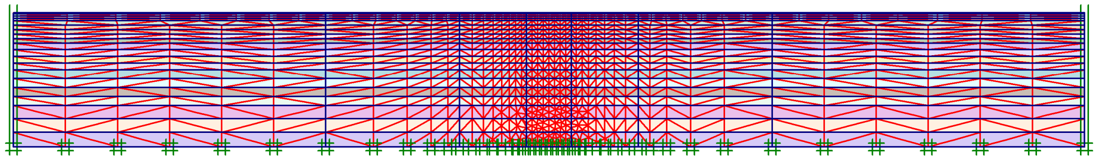

**Figure 7. Mesh employed in the FE analyses.**

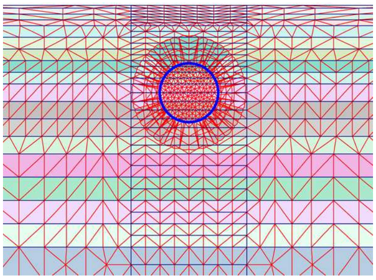

**Figure 8. Detail of the mesh around the tunnel section.**

## FE NUMERICAL ANALYSES OF THE TUNNEL

### Visco-elastic analyses

The distribution of the predicted hoop force $N$ and bending moment $M$ prior to and after the earthquake, as well as their minimum and maximum envelopes during the seismic event, are shown in Fig. 9a,b for the analysis considering the Gilory 2 earthquake, selected as representative of the case under study. The results are reported as a function of the angle $\theta$, also shown in Fig. 9, and defined positive in counter-wise direction.
These results can be compared to the load increments predicted by the analytical solutions mentioned before. Results indicate a good agreement between the visco-elastic FE solution, characterised by maximum increments, evaluated with respect to the static conditions, of hoop force $|\Delta N|_{max}$ = 336.2 kN/m and bending moment $|\Delta M|_{max}$ = 509.3 kNm/m, and the corresponding maximum increments predicted by the Wang's solutions for the no-slip case (Table 3). These latter are only slightly lower than the numerical results both in terms of hoop force (- 2.0 %) and bending moment (- 10.0 %). It is worth remarking that the two solutions compared above are based on substantially different approaches: the analytical results rely on a quasi-static analysis of the problem, while the dynamic FE solution includes more realistic features like the time dependent kinematic soil-structure interaction. Nonetheless, the results compare nicely in this visco-elastic case, as it is shown in Fig. 10 for all the investigated earthquakes.

**Visco-elasto-plastic analyses**

Adding soil plasticity to the FE analysis (Fig. 9c,d for the case of Gilroy 2 earthquake) significantly modifies the stress distribution in the lining both qualitatively and quantitatively. In fact, the behaviour during the earthquake is characterised by reduced peaks and more uniformly distributed loads in the tunnel lining as compared to the visco-elastic case. The highest increments with respect to the static conditions are in this case equal to $|\Delta N|_{max}$ = 379.5 kN/m (which is larger than the one recorded in the visco-elastic case) and $|\Delta M|_{max}$ = 431.1 kNm/m. This pattern is consistent with what observed by Shahrour and Khoshnoudian (2003) for plasticity based dynamic analyses of shallow tunnels in soft soils. More important, permanent increments of hoop force and bending moment are predicted at the end of the seismic event, as a consequence of the irreversible deformation cumulated by the soil during the earthquake. In particular, the permanent increment of hoop force with respect to the initial static conditions, entirely in compression, is characterised by a maximum value of $\Delta N$ = - 305.8 kN/m at the tunnel crown, while that of bending moment $\Delta M$ varies between – 265.9 kNm/m and + 263.4 kNm/m.
The results of the visco-elasto-plastic analysis indicate that the irreversible soil behaviour significantly modifies the tunnel actions and displacements both during the earthquake and, more importantly, after it. In fact, although the simple perfectly plastic constitutive assumption adopted, a considerable amount of plastic strain cumulate in the soil during the dynamic analyses, leading to a corresponding permanent modification of the effective stress distribution around the tunnel lining. This is also confirmed by the final deformed configuration.

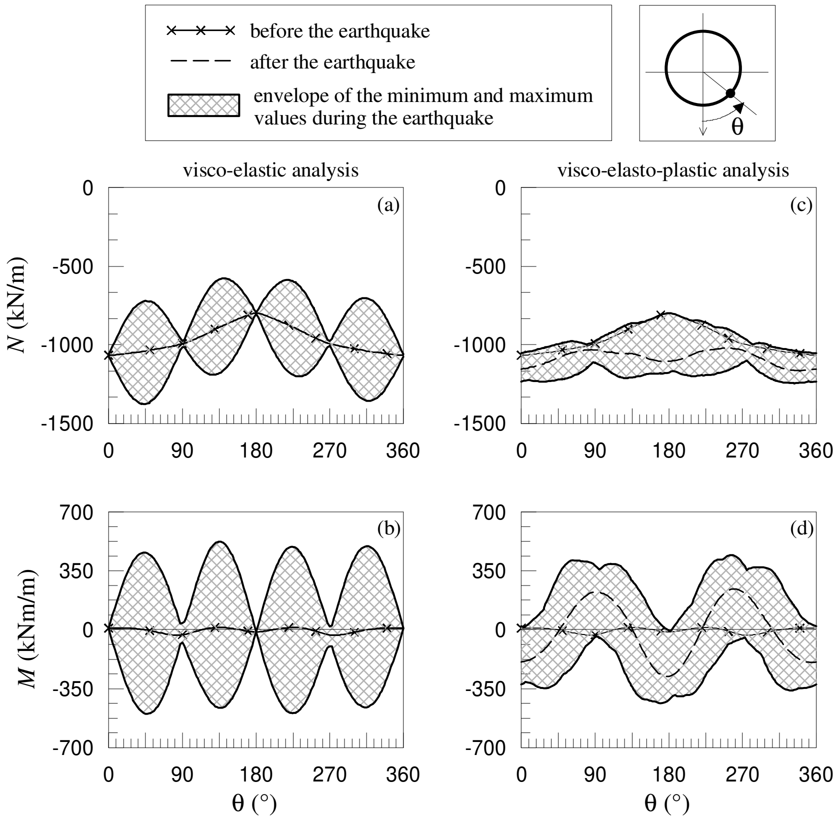


**Figure 9. Visco-elastic and visco-elasto-plastic analyses for Gilroy 2 earthquake: distribution of hoop force and bending moment before and after the seismic event and their maximum envelope.**

**Quasi-static analyses**

Quasi-static analyses were performed substituting the dynamic stage of the FE dynamic analyses with a static stage in which the mesh is subjected to simple shear conditions according to the schematisation described in Kontoe *et al.* (2008). This condition is obtained by: 1) restricting the vertical displacements along all mesh boundaries and the horizontal ones along the bottom boundary, 2) applying a uniform horizontal displacement $u_{top}$ along the top boundary; 3) applying a triangular horizontal displacement distribution, with $u_h = u_{top}$ at the top and $u_h = 0$ at the base, along the lateral boundaries. The relation between the applied $u_{top}$ and the corresponding shear strain in the soil $\gamma$ is given by:

$$u_{top} = \gamma H \tag{6}$$

where $H = 60$ m is the depth of the mesh. In order to apply simple shear conditions in the mesh, this stage is characterised by uniform soil properties with depth, which were set equal to the ones adopted in the dynamic stage of the FE analyses at the tunnel depth.

In Fig. 10 a comparison between increments of hoop force and bending moment according to Wang (1993) and results of quasi-static analyses is provided for all the investigated visco-elastic cases. These values were calculated applying in the quasi-static analyses a value of $\gamma$ equal to the maximum shear strain $\gamma_{max}$ computed in the 1D ground response analyses at the tunnel depth (Fig. 6). The two approaches provide very similar results, especially in terms of bending moments.

Quasi-static analyses adopting a plastic constitutive model for the soil are currently under way.

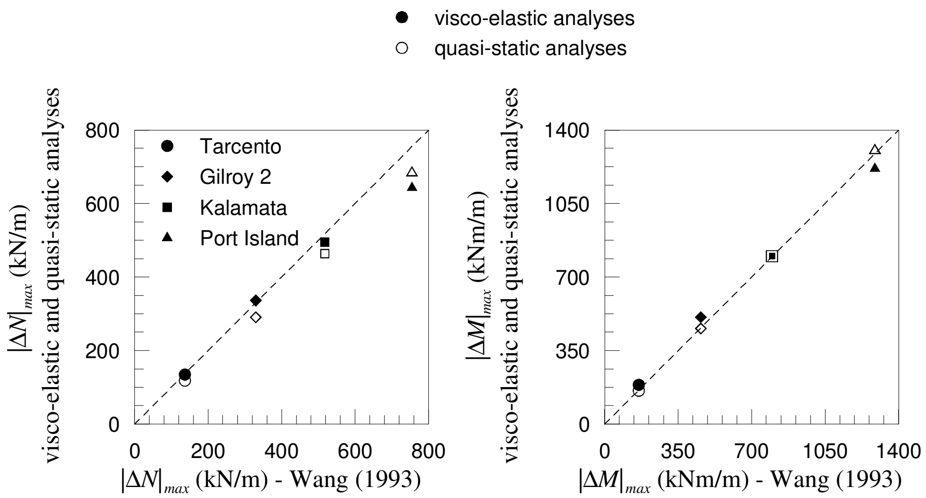


**Figure 10. Comparison between increments of hoop force and bending moment according to Wang (1993) and results of visco-elastic and quasi-static analyses.**

## CONCLUSIONS

This paper presents the results of a set of analyses aimed at studying the seismic response of a shallow tunnel built in a soft clayey deposit. Three different approaches were adopted to evaluate the increments of seismic-induced loads in the transverse direction of the tunnel lining in terms of hoop force and bending moment. The simplest uncoupled approach consists of two separated steps: 1D site response analyses followed by the application of the computed ground shaking parameters as input data for selected analytical solutions proposed in the literature. It is based on a number of simplified hypotheses concerning the behaviour of the soil and the tunnel lining and their interaction, but has the advantage of generating straightforwardly reliable results without the need of employing sophisticated numerical procedures. The second approach requires the execution of fully dynamic analyses using a non-linear Finite Element program, based on visco-elastic or visco-elasto-plastic constitutive assumptions. The comparison between uncoupled and FE visco-elastic solutions proved to be satisfactory: differences between the two approaches resulted to be less than 15% in all the investigated cases both in terms of maximum increments in hoop force and bending moment acting in the lining. It is it is worth remarking, however, that the EERA results should not be considered as the right term of comparison when modelling strong motion earthquakes, as those selected for this study. In fact, intense shaking results in relatively large and partly irreversible strains, making the plasticity-based time domain approach more realistic. FE results accounting for soil plasticity introduced new ingredients in the analysis of soil-tunnel interaction in dynamic conditions, i.e. different distribution and magnitude of the seismic-induced $N$ and $M$ and permanent increments of loads at the end of the seismic event. These features should be carefully considered in the design of underground structures in seismic areas, leading to the selection of the appropriate design philosophy to be adopted (i.e. design for strength versus design for ductility). The last approach proposed in the paper, based on quasi-static application of simple shear strain conditions to the FE model, represents a first attempt to improve the previous uncoupled scheme accounting for a more realistic non-linear interaction between the soil and the tunnel lining. This latter method proved to be effective in the visco-elastic case, while requires further investigation for the visco-elasto-plastic soil constitutive hypothesis.

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