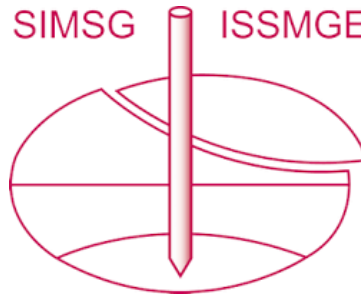


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Behaviour of coupled tunnel-soil-aboveground building systems in seismic conditions evaluated by means of parametric analyses

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ABSTRACT: In seismic areas, the realization of underground structures requires careful studies of dynamic interaction with the surrounding soil and with the existing aboveground buildings. Up to now, analyses involving at the same time tunnels-soil-aboveground buildings (full-coupled analyses) are still very rare. This paper reports the results of a series of parametric analyses by means of 2D FEM modeling, starting from a real case-history regarding the Catania (Italy) underground network. Different types of soils and different recorded accelerograms have been adopted, in order to investigate their effects on the response of the soil and of the tunnel. Equivalent linear visco-elastic constitutive models have been adopted for the soil, the tunnel and the structure. In order to take into the soil non-linearity, degraded shear moduli (G) and increased soil damping ratios (D) have been evaluated according to EC8 (2003). The models have been investigated in the time and frequency domains.

1 INTRODUCTION

A growing need for underground structures is more and more evident in order to solve the problems faced by transportation and utility networks. In seismic areas, it is extremely important to assess the possible damage produced by earthquakes to the tunnel and to the aboveground structures. Historically, underground structures have experienced a lower rate of damage than aboveground structures (Kawashima 2000). Nevertheless, recent studies have documented significant damage suffered by underground structures due to seismic events (Power et al. 1998; Hashash et al. 2001; Wang et al. 2001; Kontoe et al. 2008; Wang et al. 2009; Gazetas 2014). During an earthquake, the vibrations of aboveground structures may modify the dynamic response of tunnels (Lee & Karl 1992; De Barros & Luco 1993); at the same time, the presence of shallow tunnels may alter the response of aboveground structures. Most of the published papers consider only tunnel-soil systems (St. John & Zahrah 1987; AFPS/AFTES 2001; Hashash et al. 2005; Anastasopoulos et al. 2007, 2008, 2010; FHWA 2009; Lanzano et al. 2012; Abate et al. 2015), while a few consider tunnel-soil-aboveground structures (Luco & De Barros 1994; Kouretzis et al. 2007; Smerzini et al. 2009; Wang et al. 2013).

The present paper deals with parametric analyses performed by means of 2D FEM modeling and involving a full-coupled tunnel-soil-aboveground structure system. Different types of soils and different recorded accelerograms have been adopted in order to study their effects on the dynamic tunnel-soil-aboveground building interaction.

These 2-D FEM analyses have been performed considering the transversal direction of the tunnel, because the ovaling or racking deformations of tunnels are generally the most dangerous under seismic loading (Hashash et al. 2005; Ptilakis et al. 2014). Isotropic visco-elastic-linear behaviour has been assumed for all the material involved; but, in order to take into account soil non-linearity, the variations of the soil shear modulus and damping ratio with the expected PHA at the ground surface have been considered according to EC8 (2003).

Results are reported in terms of: ratios between the natural frequency of the system and the main frequency of the seismic input; amplification ratios along the symmetry axis of the building and the tunnel and along a parallel alignment in free-field conditions; bending moments in

the tunnel. Numerical bending moments have been also compared with those obtained using the closed-form solutions by Wang (1993) and Penzien (2000).

2 THE PARAMETRIC ANALYSES

The parametric analyses have been based on the underground network case-history of Catania (Italy) regarding a cross-section including an aboveground building (Abate & Massimino 2017a). This cross-section is characterized by: i) a reinforced concrete building 10 m wide with two equal spans in the direction under investigation, 12 m high, with four levels and shallow foundations (elastic modulus $E_b = 28500$ MPa, Poisson ratio $\nu_b = 0.2$, specific weight $\gamma_b = 25$ kN/m³; damping ratio $D_b = 5\%$; “b” is for building); ii) a reinforced concrete tunnel 11 m wide and 7.2 m high, with a horseshoe section, located 18 m below the ground surface (elastic modulus $E_l = 28500$ MPa, Poisson ratio $\nu_l = 0.2$ and damping ratio $D_l = 5\%$; (“l” is for lining); iii) soil classified as type C according to the NTC2018.

The parametric analyses have been carried out considering four different types of soil, classified as A, B, C, D according to NTC2018 (Table 1). Moreover, three seismic inputs have been adopted (Figure 1): they were different from each other for frequency content and have been scaled at the same two different peak ground accelerations a_g : 0.1g and 0.3g (Table 2).

As for the seismicity of the investigated area (Catania), the seismic parameters have been evaluated for a return period $T_r = 475$ years. Thus, the values of amplification factor S_s have been calculated according to NTC2018 for the four soil types, as shown in Table 3.

In order to take into account soil nonlinearity, the seismic geotechnical parameters have been modified in line with EC8 (2003), with reference to the expected peak acceleration at the ground surface (Table 4). So, equivalent visco-elastic analyses have been performed, appropriately degrading the shear modulus G and amplifying the damping ratio D . Actually, EC8 gives suggestions for soil type D; the Authors adopted proportionally similar assumptions for the other soil types.

Table 1. The adopted different soil types and the chosen values of shear waves velocities V_s .

Soil types	V_s^* (m/s)	V_s (m/s)
A	1000	>800
B	500	360-800
C	250	180-360
D	100	<180

V_s^* indicates the chosen values; V_s indicates the ranges provided by the NTC2018

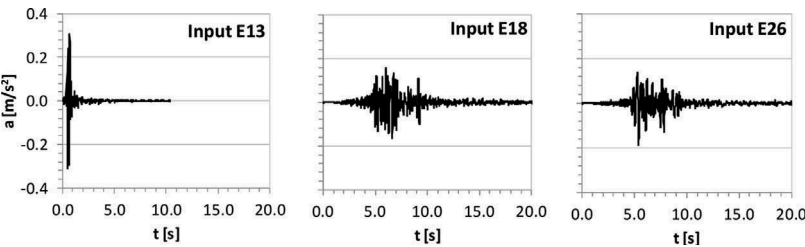


Figure 1. Adopted seismic inputs.

Table 2. Main properties of the adopted seismic inputs.

Seismic inputs	Country	Date	a_g (m/s ²)	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)
E13	Grecia (Timfristos)	14/06/86	3.02	12.70	6.86	8.26
E18	Italia (Friuli)	11/09/76	1.59	5.53	4.54	4.24
E26	Italia (Friuli)	11/09/76	1.83	2.95	3.26	2.12

f_1, f_2, f_3 are the first three fundamental frequencies of the inputs

Table 3. Values of S_s for the four different soil types.

Soil types	S_s
A	1.000
B	1.197
C	1.396
D	1.640

Table 4. Degradation of the seismic parameters for the equivalent visco-elastic parametric analyses, for the two different chosen values of the expected acceleration at the ground surface

Soil types	$a_g \cdot S_s$	$G/G_{\max}$	D	Model label*	$a_g \cdot S_s$	$G/G_{\max}$	D	Model label*
A	0.10	0.80	0.030	A80	0.30	0.36	0.10	A36
B	0.12	0.75	0.035	B75	0.36	0.30	0.10	B30
C	0.15	0.65	0.045	C65	0.45	0.23	0.10	C23
D	0.18	0.55	0.055	D55	0.54	0.16	0.10	D16

* “Model label” indicates the name adopted for the eight performed FEM models

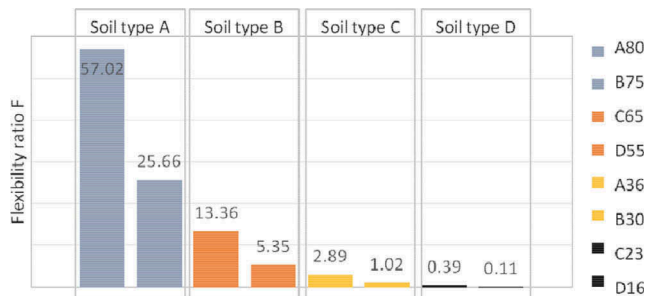


Figure 2. Flexibility ratios for the eight analyzed models.

In order to quantify the relative stiffness between tunnel and soil, the flexibility ratio F (Wang 1993), which is a measure of the flexural stiffness of the soil relative to the tunnel, has been calculated (Figure 2). For $F \rightarrow 0$ the tunnel is considered rigid and therefore no deformation will be; for $F < 1$ the tunnel is more rigid than the soil and the tunnel deformation will be lower than the soil deformation in free-field; for $F = 1$ the tunnel and the soil have the same level of stiffness and so the same level of deformation; for $F > 1$ the tunnel deformation is amplified (flexible tunnel). In the analysed cases, for soil types A, B and C (models “A80-A36-B75-B30-C65”) the tunnel has a flexible behaviour; for the soil type C soil with $G = 23\% G_{\max}$ (model “C23”) the flexibility of the tunnel coincides with that of the soil, instead for soil types D (models “D55-D16”) the tunnel is more rigid than the soil.

3 THE FEM MODELS CARRIED OUT FOR THE PARAMETRIC ANALYSES

In order to evaluate the seismic response of the tunnel-soil-building systems previously described, a finite element modeling by using the ADINA code (ADINA 2008) has been performed. Eight different FEM 2D models (Figure 3) have been developed, according to a different combination of the two parameters chosen for the analyses (soil type and seismic input). The FEM models have been labeled with reference to the soil type and to the degradation adopted for the G modulus as previously shown in Table 4.

The width of the considered soil deposit was fixed equal to 150 m ($> 4B$, where B is the width of the building), in order to avoid as much as possible boundary effects; the height of the soil deposit derives from geotechnical investigations according to which the bedrock was found at a depth of 38 m. The nodes of the soil vertical boundaries have been linked by “constraint equations” that

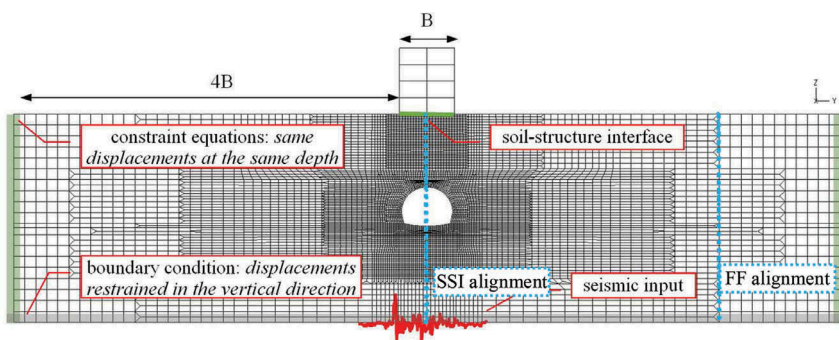


Figure 3. Typical adopted FEM model.

impose the same displacements at the same depth (Gajo & Muir Wood 1997; Abate et al. 2008, 2010; Grassi & Massimino 2009; Maugeri et al. 2012). All the nodes of the base of the mesh have been restrained in the vertical direction. The above-described accelerograms have been adopted at the base of the model (Table 2; Figures 1, 3). The tunnel and the building have been modeled by 2-node beam elements, adopting a linear visco-elastic constitutive model (Section 2). The soil has been modeled by 9-node solid rectangular elements and a linear-equivalent-visco-elastic behaviour (Section 2). The mesh element size has been chosen in order to ensure the following criteria: i) efficient reproduction of all the waveforms of the whole frequency range under study: $h \leq V_{s,\min}/6 \div 8 f_{\max}$ (Lanzo & Silvestri 1999); ii) a finer discretization near the tunnel. Tunnel-soil slip has not been allowed (Pitilakis et al. 2014). Special contact surfaces have been modeled between the foundations and the soil, considering a friction equal to $2/3 \varphi$, in order to model probable uplifting and/or sliding phenomena. The Rayleigh damping factors α_r and β_r have been computed according to the well-known relations (Chang et al. 2000; Lanzo et al. 2003): $\alpha_r = D \cdot \omega$ and $\beta_r = D/\omega$, being D the damping ratio and ω the angular frequency of the involved systems.

4 MAIN RESULTS

4.1 Evaluation of the frequencies of the system

Frequencies of the first two vibration modes of the full system have been evaluated for investigating probable resonance. Figure 4 shows the ratios between the input frequencies (f_{input}) and the system frequency (f_s). Ratios between 0.4 and 1.5 (black lines) indicate the frequencies for which resonance could occur (Sica et al. 2011; Abate & Massimino 2017b). It is possible to see that for many cases resonance could occur, especially for the second vibration mode.

4.2 Response in terms of accelerations

The first phase of the parametric analysis consisted in studying the amplification or de-amplification phenomena, in terms of amplification ratio R_a , that is the ratio between the maximum acceleration at the detected depth z and the maximum input acceleration at the bedrock. It has been firstly evaluated along the axis of symmetry of all the numerical models (SSI alignment) and secondly considering the free-field conditions (FF alignment), i.e. along a parallel alignment (Figure 3).

Figure 5 shows the comparison between the SSI alignment (continuous lines) and the FF alignment (dashed lines); moreover, the amplification ratios S_s at the foundation level evaluated according to NTC2018 are shown in the above-mentioned figures. Figure 5 shows that generally both amplification and de-amplification are possible from the bedrock to the tunnel. Then de-amplification of the input across the tunnel often occurs, i.e. the tunnel produces a beneficial effect. Finally, from the tunnel to the foundation level there is always an amplification. The beneficial effect of the tunnel is due to the absence of material inside the tunnel. In turn, the tunnel changes the path of seismic waves. In particular, for the A80 and B75 cases there is a strong amplification from the bedrock to the foundation level especially for the E18 and E26

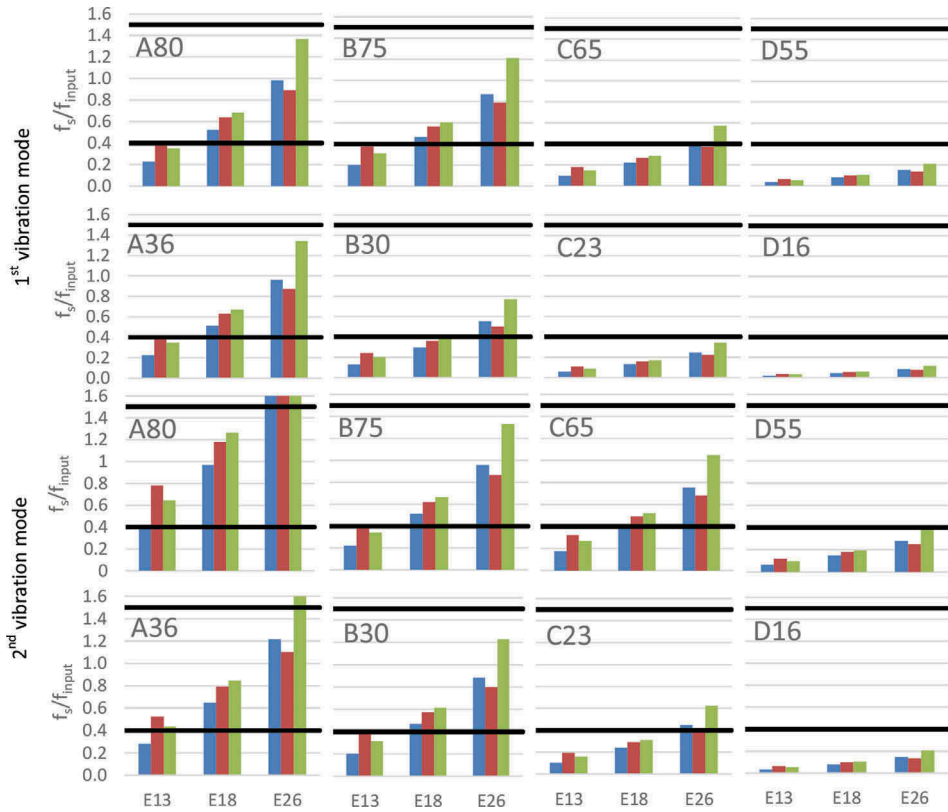


Figure 4. Natural frequency of the system (f_s) over main frequency of the seismic input (f_{input}).

inputs, due to the probable resonance (Figure 4). The A36 case shows lower R_a due to the greater values of D . The B30 case shows lower R_a values in comparison to previous cases, being far from possible resonance conditions (Figure 4). For the same reasons, in the remaining cases,

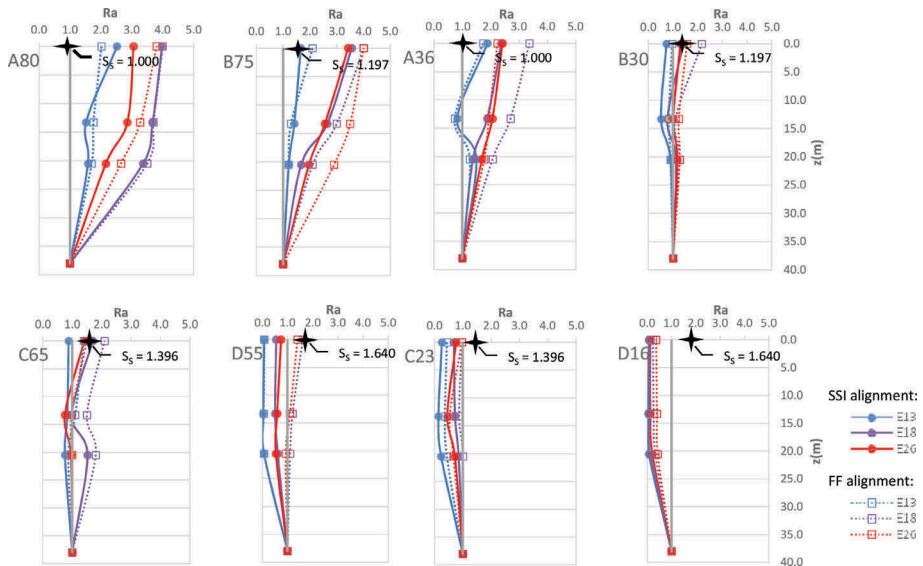


Figure 5. Amplification ratios by FEM analyses and by NTC2018 (S_s).

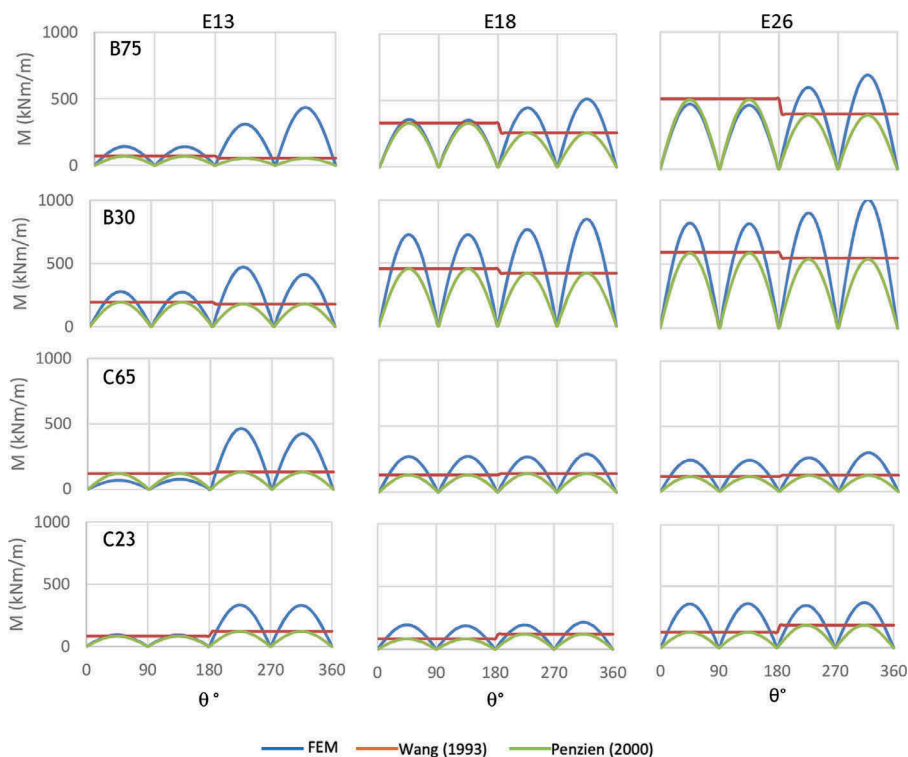


Figure 6. Comparison between numerical and analytical bending moments.

the inputs are generally de-amplified. Moreover DSSI (continuous lines) causes a reduction of R_a with respect to the free-field condition (dashed lines). Softer soils tend to de-amplify the seismic signal that reaches the foundation level; on the contrary, stiffer soils tend to amplify it; this apparently unusual result depends by the high soil damping D , whose effects are greater than those of G for softer soils. As regards the comparison with S_S , the Italian Technical Code underestimates the amplification at the foundations level for stiffer soils (A80-A36 and B75-B30) and then overestimates it for softer soils (C65-C23 and D55 -D16). These results clearly show the importance of a complete coupled analysis, for detecting the appropriate “real” design inputs.

4.3 Response in terms of tunnel bending moments

The parametric analysis also investigated the response of the system in terms of bending moments M in the tunnel lining, in the transverse section per unit of longitudinal dimension. The numerical values, which correspond to the maximum values of the time-history responses have been compared with those obtained through the analytical solutions proposed by Wang (1993) and Penzien (2000) for no-slip conditions, as functions of flexibility ratios F of the tunnel and the overburden pressure and at-rest coefficient of the earth pressure of the soil. The analytical solutions were developed for circular tunnels, so they have been adapted to investigate a “horseshoe tunnel section”, using two radii for $0^\circ < \theta < 180^\circ$ and for $180^\circ < \theta < 360^\circ$, respectively, where θ is the tunnel centre angle.

Figure 6 shows the comparisons between numerical and analytical bending moments for the most significant soil types B and C. It is possible to assert that there is always a good agreement between the numerical and the analytical bending moments for the upper arc ($0^\circ < \theta < 180^\circ$), instead for the inferior arc ($180^\circ < \theta < 360^\circ$) the numerical bending moments are greater than the analytical ones. Moreover, due to the probable resonant phenomena, the E18 and E26 inputs cause high bending moments for the cases characterized by flexible tunnel (see Figure 3) surrounded by soil types B. The bending moments increase when the qualities of the soil decrease; this is due to the flexibility ratio F that tends to decrease.

5 CONCLUSIONS

Parametric analyses have been performed in order to investigate the influence of soil type according to NTC2018 and of the frequency of the input motion in the seismic response of tunnel-soil-aboveground structure systems. The analyzed tunnel is characterized by a RC horse-shoe section; the aboveground structure is a RC building with two equal spans in the direction under investigation, four levels and shallow foundations. The achieved results highlight the importance of complete full-coupled analyses and they can be summarized as in the following:

- The presence of the tunnel and the aboveground structure causes a reduction of the amplification ratio from the bedrock to the foundation level with respect to the free-field condition.
- The tunnel produces a beneficial effect in the urban area, mainly for the soil types characterized by higher properties.
- For the investigated cases, softer soils tend to de-amplify the seismic signal that reaches the foundation level; on the contrary, stiffer soils tend to amplify it; this apparently unusual result depends by the high soil damping D , whose effects are greater than those of G for softer soils.
- As regards the tunnel bending moments, the comparison between numerical and analytical results is satisfactory for the upper arc ($0^\circ < \theta < 180^\circ$), instead for the inferior arc ($180^\circ < \theta < 360^\circ$) the numerical bending moments are greater than the analytical ones; this disagreement is due to the fact that the analytical solutions were developed for circular tunnels.

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