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FEM modelling of the seismic behavior of a tunnel-soil-aboveground building system: a case history in Catania (Italy)

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Abstract

In the design of tunnels it is extremely important to assess the possible damage to the tunnel and to the aboveground structures in order to provide adequate mitigation measures, above all in high seismic hazard areas. Great attention has been devoted to separated tunnel-soil interaction analysis and soil-aboveground structure interaction analysis. Unfortunately, studies involving tunnel plus soil plus aboveground structures (full-coupled analysis) are still very rare. The present paper deals with the seismic response of a tunnel-soil-aboveground building system. The effects of the tunnel on the response of the soil and/or of the building and vice versa have been analysed by means of a full-coupled FEM modelling. A cross-section of the recently-built underground underground in Catania (Italy), including an aboveground building has been analysed and considering the expected scenario earthquake. Results are reported in terms of amplification ratios, frequency ratios, as well as dynamic bending moments in the tunnel. The main goal of the paper is to highlight the combined effect of tunnel and aboveground structure.

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Keywords: Full-coupled analysis; clay; soil non linearity; amplification ratios; seismic tunnel lining forces

1. Introduction

The issue to evaluate the seismic behaviour of tunnel-soil-aboveground structure systems by means of full-coupled analysis is extremely important [1,2], above all in an area characterized by high seismic hazard such as Catania (Italy)

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area [3,4,5]. Full-coupled analyses allow us to evaluate the "real" inputs that hit aboveground an underground structures. Commonly, the seismic design of structures is performed using design spectra provided by national technical codes, often leading to significant underestimations.

This paper deals with the seismic response of a full-coupled tunnel-soil-aboveground structure system and studies the effects of the tunnel on the response of the soil and aboveground structure and vice versa through a 2D FEM model. The seismic response analysis has been carried out for a particular cross section relating to the Milo-Cibali segment belonging to Borgo-Nesima network of Catania underground. Seven synthetic accelerograms have been adopted at the bedrock, assuming the source to be along the Hyblean-Maltese fault in order to refer to the scenario earthquake that occurred in eastern Sicily in 1693.

Isotropic viscoelastic-linear behaviour has been assumed for all the materials involved; nevertheless, in order to take into account the soil non-linearity, the degradation of G and the increasing of D with the strain level have been considered for the soil as suggested by EC8 (2003) [6]. Results are presented in terms of amplification ratios and frequency ratios, as well as dynamic bending moments in the tunnel. Comparison with the simplified approaches of Wang [7] and Penzien [8] are also presented.

2. The case history

2.1. Description of the aboveground and underground structures

The underground network in Catania will serve the city centre in order to connect it with the north-west suburban area up to Catania airport and with the commercial area of Misterbianco.

In this paper, the seismic response analysis has been carried out for a particular cross section relating to the Milo-Cibali segment belonging to Borgo-Nesima network. On the analyzed section an aboveground structure made of reinforced concrete resting on the soil surface is present. It has the following mechanical properties: E_b=28500 MPa, ν_b=0.2, γ_b=25 kN/m^3, D_b=5%. The building is composed of four aboveground floors, each one characterized by a height of 3 meter. It has shallow foundations. In the FEM modelling a frame of the building 10 meters wide has been considered.

The tunnel is made of reinforced concrete (E_l=28500 MPa, ν_l=0.2, γ_l=25 kN/m^3, D_l=5%.) and it is 11 m wide and 7.2 m high (Fig. 1); it is located at 18 m below ground level.

2.2. Geological and geotechnical properties of the soil

Two geotechnical investigation surveys have been performed (July 1999, December 2005- January 2006); in particular, 7 and 10 boreholes have been carried out in 1999 and in 2005-2006, respectively, due to the great role played by geotechnical properties in local site response [9]. Detailed information has been also obtained by the laboratory tests (grain size determination, Atterberg limits, triaxial test, oedometric test), and in situ tests (SPT).

The soil profile of the analysed section is characterised by the following stratigraphy (Fig. 2a): anthropic layers (RL+Ret), silty clays (*ALg*) and clays (*Aa*). The sub-lithotypes *ALg* and *Aa* belong to the *PSa* lithotype. Table 1 shows geotechnical properties of the lithotypes found. The bedrock has been found at a depth equal to 38 m. The soil is type C according to the Italian code NTC08 [10].

Table 1. Geotechnical properties of the lythotypes located in the examined cross section of the Catania underground network.

Geotechnical properties	*PSa*
γ [kN/m3]	20
E'0 [MPa]	300 [z≤10m] 300+50(z-10) [z>10]

2.3. Inputs

Seven synthetic accelerograms have been adopted at the bedrock (Fig. 1), assuming the source to be along the Hyblean-Maltese fault [11]. This synthetic seismograms have been obtained by generating a seismic ground motion scenario at the bedrock, such as the 1693 earthquake [12,13,14], characterized by a *PGA* of 0.207g, corresponding to a return period of 475 years (10% probability of exceedance in 50 years) in the current Italian seismic code.

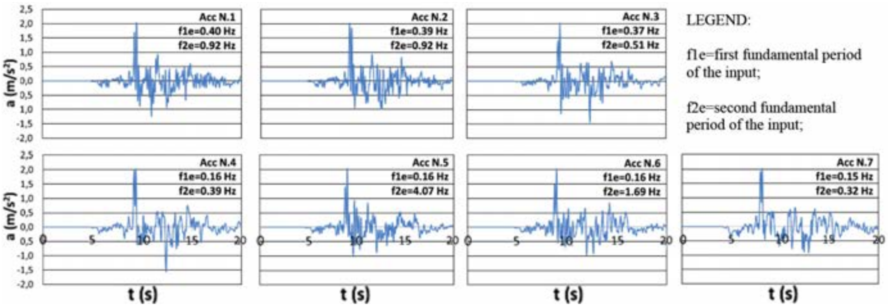

Fig. 1. Synthetic accelerograms for the scenario earthquake in Catania, adopted as seismic inputs at the bedrock.

3. FEM modelling

The study of the dynamic tunnel-soil-aboveground structure interaction has been performed by a 2D plain strain finite element modelling with ADINA code [15], widely used in dynamic analyses [16,17].

The soil has been modeled by 2D 9-nodes solid elements, by means of a visco-elastic constitutive model. In order to take into account the soil non-linearity, extremely important in soil mechanics [18,19], as suggested by EC8 [6] a degraded shear modulus $G_s = 0.36\ G_{s0}$ and a damping ratio D = 10% have been considered, because the estimated ground acceleration $S_s \cdot PHA_{input}$ according to NTC08 [10] is $0.207g \cdot 1.4 = 0.29g$ (Table 2). The soil has been divided into 15 horizontal layers. By moving from the soil surface, the first layer has a thickness of 10 m, followed by 14 layers, each of a thickness equal to 2 m with the soil characteristics shown in Table 1.

Table 2. Values of increasing of D and degradation of Vs and G (EC8, 2003).

Ground acceleration ratio, $S_s \cdot PHA_{input}$ [g]	Damping ratio	$V_s/V_{s,max}$	G_s/G_{s0}
0.10	0.03	0.90(±0.07)	0.80(±0.10)
0.20	0.06	0.70(±0.15)	0.50(±0.20)
0.30	0.10	0.60(±0.15)	0.36(±0.20)

NOTE: Through the ± one standard deviation ranges, the designer can introduce different amounts of conservatism, depending on such factors as stiffness and layering of the soil profile. Values of $V_s/V_{s,max}$ and G/G_{s0} above the average could, for example, be used for sitffer profiles, and values of $V_s/V_{s,max}$ and G/G_{s0} below the average could be used for softer profiles.

The tunnel and the building have been modelled by 2-node beam elements adopting a linear visco-elastic constitutive model and using the usual values for the reinforced concrete (see section 2.1).

As regards the boundary conditions, vertical displacements have been permitted to the vertical boundaries of the soil deposit, and in addition, the two vertical boundaries have been forced to have equal displacements in the horizontal direction [20]. At the base of the deposit, only horizontal displacements have been permitted. Moreover, to limit artificial phenomena of reflection and refraction of seismic waves at the boundaries, the vertical boundaries of the soil deposit have been located at a distance equal to 7 times B, where B is the building width. The height of the soil deposit is 38 m according to in situ tests (Fig. 2).

The element size adopted has been chosen in order to ensure the following criteria: i) efficient reproduction of all the waveforms of the whole frequency range under study (e.g. following the principle that the element size must be 1/6 ÷ 1/8 of the minimum wave length); ii) efficient modeling simulation of the soil close to the structure (therefore, a finer discretization near the structure and the tunnel has been used). According to a no slip hypothesis, a solid connection between the soil and the tunnel has been assumed.

In addition to the system described above, in order to analyze the influence of the tunnel and the aboveground structure on the seismic response, three other configurations have been modeled: one represents the free-field conditions; the second one includes only the tunnel; the last one includes only the aboveground structure.

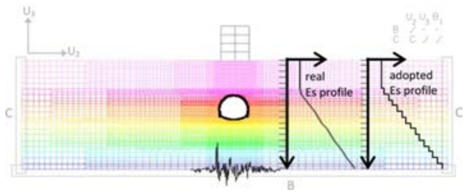

Fig. 2. FEM model with the boundary and seismic loading conditions. The different soil layer colours refer to the Young modulus profile adopted.

The Rayleigh damping factors α and β have been computed according to the following relations [21,22]: $\alpha = D \cdot \omega$; $\beta = D / \omega$, where D is the damping ratio and ω the pulsation.

All the analyses have been performed in two steps: in the first step a static analysis has been performed (static step) considering static vertical distributed loads and concentrated forces on the building, due to design surcharges; while in the second step the earthquake input motion has been applied at the bottom of the model (dynamic step). Moreover, both for the static and for the dynamic steps a “mass proportional load” has been applied to the whole system, in order to take into account the gravity loads.

4. Results

The seismic response of the investigated full-coupled system is presented in this paper in terms of amplification ratios and frequency content (Fig. 3), as well as in terms of bending moments in the tunnel (Fig. 4).

Figure 3a shows the comparison between the seismic response obtained by the 4 configurations analyzed in this paper (see Section 3) and by using only the input No. 3. The seismic response has been evaluated along the alignment A-B-C-D, where point A is the node localized at ground level, points B and C the nodes localized at the top and bottom of the underground structure, respectively, and finally point D is the node localized at the bedrock. This figure shows the amplification ratios profile (*Ra-z*) from the bedrock to the ground level, in which *Ra* is the ratio between the maximum acceleration at the depth z and at the bedrock, respectively. It is possible to observe in the free field conditions an amplification ratio (red line) equal to 2.26 measured at the ground level, which is the highest value among the analysed configurations. The presence of the only building determines a lower value (2.13) than the value obtained by the free field configuration. A similar result has been obtained for the configuration with only the tunnel obtaining a value of the *Ra* = 2.12 at soil surface and a de-amplification from C to B equal to *Ra*=0.9.

Thus, the tunnel has a beneficial effect on the seismic response. The beneficial effect of the underground structure is caused by the presence of the empty space in which the seismic waves can’t propagate. Considering the seismic behaviour of the whole soil-tunnel-aboveground structure system, a further reduction of the seismic input from bedrock to ground level (*Ra* = 1.94).

Figure 3b shows the amplification ratios Ra at the ground level for the whole soil-tunnel-aboveground structure system using all the seven synthetic accelerograms (see Section 2.3). The values of amplification ratios obtained vary between 1.57 and 2.35, which are significantly greater than that one evaluated according to NTC 2008 [10] for soil type C, equal to 1.4.

It’s obvious a significant underestimation of the amplification by the technical code. Figure 3c shows the ratios between the input first frequency and the soil frequency for the 4 configurations analyzed. In particular, the values between 0.5 and 1.5 (blue line) are ratios that indicates the frequencies for which a resonance phenomenon is possible.

This is confirmed for the Acc2 that is chacterized by a ratio equal to 1.2 which, by being within the probable resonance band, shows the highest Ra value.

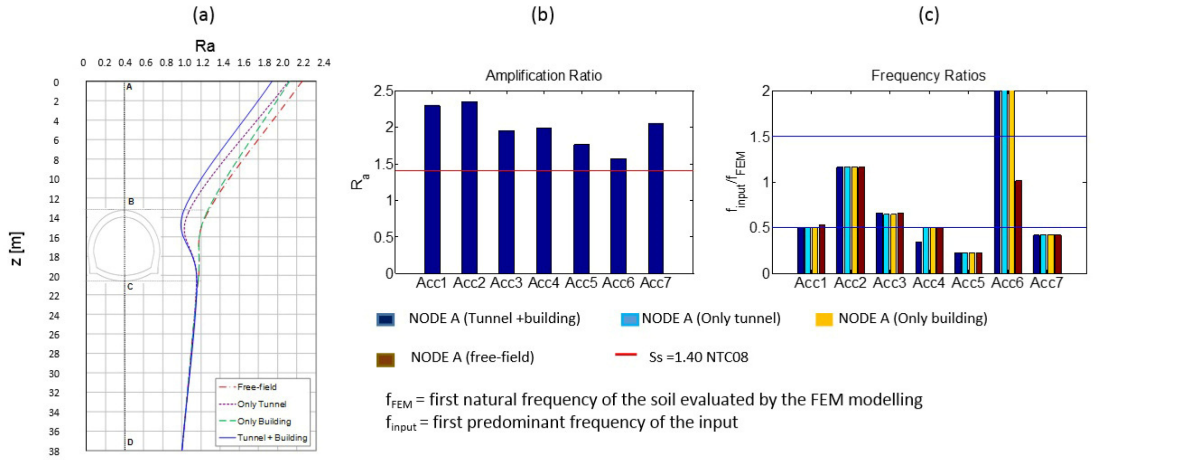

Fig.3. (a) The case-history seismic response: amplification ratio profiles (Input accelerogram No. 3); (b) The case-history seismic response: amplification ratio (all input accelerograms); (c) Probable resonance occurrence investigation.

Figure 4 shows the dynamic bending moment M along the perimeter of the tunnel lining using all the seven accelerograms and the full-coupled configuration. In particular, this figure shows the comparison between the numerical results using a no-slip conditions and the results obtained through the analytical full-slip and no-slip solutions proposed by Wang [7] and those obtained through analytical no-slip conditions proposed by Penzien [8]. The numerical results agree quite well with the analytical ones, although the analytical solutions concern circular tunnels while the analyzed tunnel has a horseshoe section.

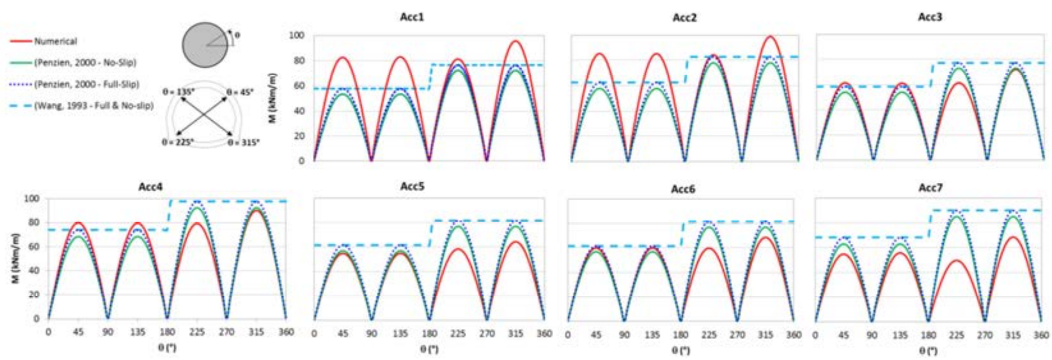

Fig. 4. Dynamic bending moment along the perimeter of the tunnel.

5. Conclusions

This paper deals with the analysis of the seismic behaviour of the tunnel-soil aboveground structure system (full-coupled analyses). In particular, a cross section relating to the Milo-Cibali segment belonging to Catania (Italy) underground has been analyzed. Seven synthetic accelerograms have been adopted at the bedrock, assuming the source to be along the Hyblean-Maltese fault in order to refer to the scenario earthquake that occurred in eastern Sicily in

1693. Results have been presented in terms of amplification ratios and frequency content, as well as dynamic bending moments in the tunnel.

The presence of the tunnel only in the soil deposit causes a de-amplification across the tunnel, and in turn a reduction in the amplification ratio *Ra* from the bedrock to the soil surface. So, the tunnel has a beneficial effect on the urban area. Also the presence of only the building causes a reduction in *Ra* compared with the free-field conditions, representing the building an important surcharge. The combined presence of the tunnel and the building causes a further reduction in *Ra* at the soil surface, compared with the previous cases It should be noted that in any case *Ra* is significantly greater than that one evaluated according to NTC, 2008, but lower than that one evaluated in commonly-performed free-field conditions. This result shows clearly the importance of full-coupled analysis up to the investigated tunnel depth.

As regards the seismic bending moments in the tunnel lining, the comparison between numerical and analytical results is certainly very satisfactory, above all considering that the analytical solutions [7,8] are for circular tunnels while the tunnel under investigation has a "horseshoe section".

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