Seismic response analysis of the *Two Towers* subsoil in Bologna (Italy)

Daniela Boldini [i)], Gaetano Falcone [ii)] and Guido Gottardi [iii)]

i) Associate Professor, DICMA, Sapienza University of Rome, via Eudossiana 18, 00184 Rome, Italy.
ii) Research Associate, DICEA, Università degli Studi di Napoli Federico II, via Claudio 21, Napoli 80125, Italy.
iii) Full Professor, DICAM, University of Bologna, viale del Risorgimento 2, 40136 Bologna, Italy.

ABSTRACT

The paper summarises the main findings of a parametric numerical investigation focusing on the seismic response of the foundation soil below the *Two Towers*, the historic medieval symbol of Bologna (Italy), dated back to the XII century. The fine-grained strata were recently characterised by an experimental field campaign down to 40 m below the ground level (b.g.l.), providing a shear wave velocity profile not exceeding 400 m/s. The seismic geotechnical model was then extrapolated down to either 150 m or 230 m b.g.l., where the presence of a rigid bedrock was assumed on the basis of geological considerations, with modest effects on the final outcome though. Analyses were run in the frequency domain with the code MARTA, using an equivalent linear visco-elastic law for the soil, and in the time domain with the finite element code PLAXIS, using instead the hardening soil model with small strain stiffness. The seven selected seismic inputs were scaled to a peak ground acceleration equal to 0.191 g, in order to be representative of the earthquake reference hazard scenario of the site for a return period of 712 years. Results at the tower embedded foundation depth (*i.e.* 5 m b.g.l.) clearly indicate that, while the peak ground acceleration is approximately coincident with the input, a significant seismic amplification can be observed at the fundamental periods of the *Two Towers* (about 1.35 s for the Garisenda and 3.00 s for the Asinelli). Larger values of spectral accelerations and maximum shear strains, also exceeding 0.1% in the shallower layers, are typically predicted by PLAXIS, thus highlighting the importance of adopting a more advanced constitutive formulation in the context of strong earthquake excitations.

Keywords: *Two Towers*, historic heritage, seismic response analysis, equivalent linear model, HSsmall.

1 THE TWO TOWERS

The *Garisenda Tower* and the *Asinelli Tower* (Fig. 1), also widely known as the *Two Towers*, are the best preserved and famous medieval towers in the city of Bologna (Northern Italy). Standing one close to the other, right in the heart of the city centre, the *Two Towers* are delicate remains of the old towered city, which counted more than 75 towers in the 12th century.

At present, the *Garisenda Tower* is 48.16 m high and 7.4 m wide at the base, with an inclination of its straight axis of 3.914° toward east and 0.6651° toward south. At the end of its construction, the tower was 60 m high, but already in 1353 the city government decided to demolish the top part, due to its dangerous leaning. *Asinelli* is 97.20 m high and 8.7 m wide at the base (Bertolini et al. 2022).

Fundamental vibration periods are about 1.35 s for the *Garisenda Tower* and 3.00 s for the *Asinelli Tower* (Baraccani et al., 2020).

The subsoil is mainly made of fine-grained alluvial deposits, well beyond the maximum investigated depth. From geophysical data, a consistent gravel layer is possibly located at around 150 m depth, whereas a proper bedrock, made of slightly cemented yellow sandstone, could be locally found at about 230 m depth (Giacomelli et al., 2023).

A rather extensive geotechnical investigation campaign was carried out in 2016 with the aim of integrating the available data and defining a reliable geotechnical model of the foundation subsoil, which consists of a relatively homogeneous alternation of silty-clays and clayey-silts, down to the investigated depth of 100 m (Bertolini et al. 2022). A down-hole test to 40 m depth was also carried out in a borehole located close to the *Two Towers* (Fig. 2), together with the execution of three resonant column and torsional shear tests (Fig. 3) for the determination of relevant soil seismic parameters.

On the basis of such available geotechnical data, a preliminary study on the local seismic response of the *Two Towers* subsoil was therefore carried out and reported herein. Two different numerical approaches were adopted characterised by a different level of complexity. Performing reliable seismic response and soil-structure interaction analyses is in fact possible if the constitutive models of the soil strata are conveniently calibrated (*e.g.* Amorosi et al. 2010, Régnier et al. 2016).

Fig. 1. The *Two Towers* of Bologna.

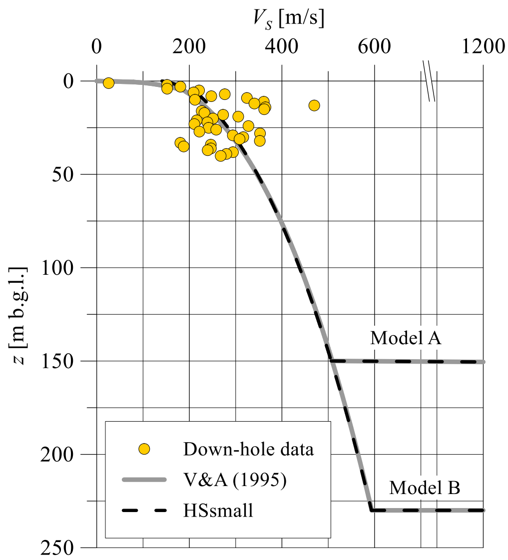

Fig. 2. Down-hole test data ($V_{S,30}$ = 238 m/s) and V_S profiles derived from the Viggiani & Atkinson expression (indicated as V&A in the legend) and the HSsmall model.

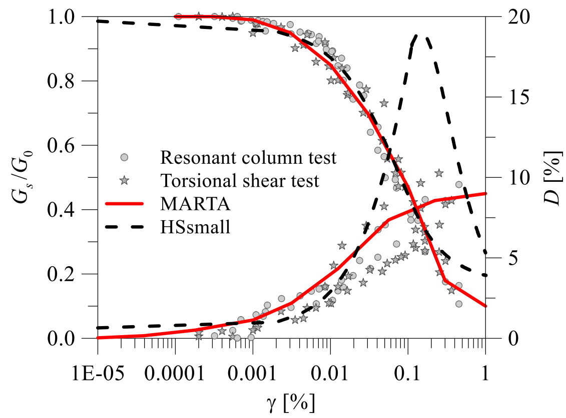

Fig. 3. Comparison between laboratory data and curves of the shear modulus decay and damping ratio adopted in MARTA and derived from the HSsmall formulation.

2 THE GEOTECHNICAL MODELS

Due to the uncertainties regarding the bedrock depth, two geotechnical models were considered, named Model A and Model B in Fig. 2. In Model A the base of the model was located at 150 m b.g.l., while in Model B at 230 m b.g.l.; in both cases, a shear wave velocity V_S of 1200 m/s for the bedrock was assumed.

The water table was set at 5 m b.g.l. and hydraulic conditions were assumed hydrostatic. Considering the relatively limited heterogeneity of the investigated soil, the subsoil conditions were assumed to be homogeneous with respect to the parameters provided in Table 1. Here, the overconsolidation ratio *R* is given in terms of mean effective stress, corresponding to an *OCR* value of 3.9. The earth coefficient at rest K_0 was computed as $K_0 = (1-\sin\varphi')\cdot OCR^{0.5}$.

The profile of shear wave velocity with depth was defined considering the relationship proposed by Viggiani & Atkinson (1995) for the small-strain shear stiffness G_0:

$$\frac{G_0}{p_r} = A\left(\frac{p'}{p_r}\right)^n R^m \qquad (1)$$

where p_r is a reference pressure taken equal to 1 kPa, p' is the mean effective stress, *A*, *n* and *m* are parameters that depend on the plasticity index I_P and *R* is the already defined overconsolidation ratio. Considering an I_P value of about 17, the following values were obtained from Viggiani & Atkinson (1995): A=1800, n=0.75 and m=0.22. It can be observed from Fig. 2 that the corresponding shear wave velocity profile, obtained as $V_S=(G_0/\rho)^{0.5}$, fits very well the available down-hole data.

Table 1. Values of the physical and mechanical properties and state variables assumed for the foundation soils.

Parameter	Symbol and unit	Value
Plasticity index	I_P [%]	17
Unit weight of volume	γ [kN/m³]	18.0
Overconsolidation ratio	*R* [-]	2.6
Effective cohesion	*c'* [kPa]	29
Effective friction angle	ϕ' [°]	29
Earth coefficient at rest	K_0 [-]	1.0

3 THE SEISMIC SCENARIO

The reference seismic hazard scenario was selected following the Italian hazard map available at http://zonesismiche.mi.ingv.it/ (Stucchi et al. 2004). In particular, considering a 10% probability of exceedance in 75 years, corresponding to a return period T_R of 712 years, a peak ground acceleration *PGA* of 0.191 g was estimated for a rigid outcrop at the *Two Towers* site (latitude = 44.495 and longitude = 11.43). The resulting *PGA* is compatible with the maximum possible effect of the nearby seismogenetic sources. In particular, the most relevant one is the Casalecchio di Reno fault, characterised by a moment magnitude of $M_w = 5.5$ and a distance lower than 20 km. The resulting acceleration spectrum is plotted in red in Fig. 4.

A set of 7 accelerograms were selected within the European Strong Motion Database (ESD) using the software Rexel (Iervolino et al. 2009) and imposing the following conditions: average matching of the target response spectrum in the period range $T = 0.15$ s – 3.00 s, moment magnitude $M_w = 4.0 - 6.5$ and epicentral distance $ED < 20$ km (Table 2 and Fig. 4).

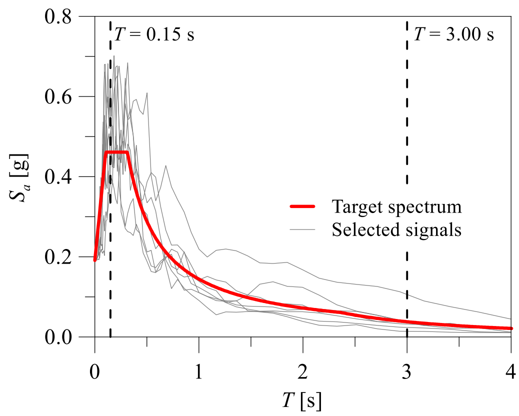

Fig. 4. Target spectrum and selected earthquake signals.

Table 2. Characteristics of the selected signals (*AS* = aftershock).

ID	Earthquake	Date	M_W	*ED* [km]
7142	Bingol	01/05/2023	6.3	14
1243	Izmit (AS)	13/09/1999	5.8	15
383	Lazio Abruzzo (AS)	11/05/1984	5.5	14
473	Vrancea	31/05/1990	6.3	7
5655	NE of Banja Luka	13/08/1981	5.7	10
6335	South Iceland (AS)	21/06/2000	6.4	15
6326	South Iceland (AS)	21/06/2000	6.4	14

4 NUMERICAL MODELS

The numerical simulations of the seismic site response were run using different constitutive schematisations for the soil based on the equivalent linear (EQL in the following) and non-linear (NL in the following) approaches (Michetti 2020, Cristalli 2021). A preliminary comparison of the two approaches for low-input motion intensity (PGA = 0.001 g) was carried out for the sake of validation (e.g., Régnier et al. 2016).

4.1 Equivalent-linear approach

The first set of analyses were performed with the code MARTA (Callisto & Ricci 2019), freely available at https://luigicallisto.site.uniroma1.it/attivita.

The input data in terms of shear wave velocity profile and decay curves are plotted in Figs. 2 and 3, respectively. In detail, the $V_S(z)$ profiles for the two models were discretised with steps of 1 m. The selected signals were applied at the outcrop and $V_S = 1200$ m/s was considered for the deformable bedrock at the base of models.

4.2 Non-linear approach

The non-linear numerical simulations were carried out with the Finite Element code PLAXIS (Brinkgreve et al. 2022). They included the initialization of the stress state using the gravity loading option, a second static phase consisting in a plastic nil phase in which no changes were applied to solve large out-of-balance forces eventually occurred and to restore equilibrium, and a third dynamic phase during which the seismic input motion was applied assuming undrained conditions. Standard boundary conditions were considered in the static stages (*i.e.*, no horizontal displacement along the vertical sides and totally fixed displacements at the base of the model), while free-field boundary conditions along the vertical sides and a compliant base at the bottom of the model were implemented during the dynamic phase (Joyner & Chen 1975, Falcone et al. 2018, Brinkgreve et al. 2022). The free field conditions inhibit the reflection of the seismic waves at the vertical boundaries. In order to get the signal at the bedrock, which is an output of the analysis in the case of compliant base, a visco-linear-elastic 1 m thick layer was added at the base of the numerical model ($D = 0.5\%$ and stiffness derived from $V_S = 1200$ m/s). The dynamic simulations were carried out with a time step equal to that of the input signals. The mesh element size was set to be lower than the maximum value given by:

$$d_{max} = \frac{V_S}{6 f_{max}} \quad \text{with } f_{max} = 15 \text{ Hz} \tag{2}$$

The PLAXIS simulations were performed using the isotropic elasto-plastic hardening constitutive model named Hardening Soil model with small strain stiffness (Schanz et al. 1999, di Lernia et al. 2019), herein named HSsmall. HSsmall allows to describe the hysteretic behaviour of soils at small strains, introducing in the model formulation the initial shear modulus, G_0, and the reduction of the secant shear modulus, G_S, and the corresponding increase in the damping ratio, D, with increasing values of shear strain, γ. The irreversible soil response is accounted for by the HSsmall by an isotropic hardening elasto-plastic formulation characterised by two yield surfaces: a shear hardening yield surface, whose evolution depends on the deviatoric plastic strains, and a cap yield surface, introduced to bound the elastic region for compressive stress paths, which expands as a function of the plastic volumetric strains.

The HSsmall parameters (Table 3) were calibrated ensuring the best fitting of site investigations ($V_S(z)$) and laboratory data ($G_S(\gamma)/G_0$ and $D(\gamma)$ curves) as shown in Figs. 2 and 3. Additional damping, based on the Rayleigh formulation, was also added for very small amplitude shear strain levels. Rayleigh controlling frequencies equal to 3 and 6 Hz and 0.5% target damping ratio were selected.

Table 3. HSsmall model parameters.

Parameter	Description	Value
c' [kPa]	Effective cohesion	29
φ' [°]	Effective friction angle	29
ψ [°]	Dilatancy angle	0
E_{50}^{ref} [kPa]	Reference secant stiffness in standard drained triaxial test	11806
E_{oed}^{ref} kPa]	Reference tangent stiffness for primary oedometer loading test	11806
E_{ur}^{ref} [kPa]	Reference unloading/reloading stiffness at engineering strains	35417
ν_{ur} [-]	Poisson's ratio for unloading/reloading	0.25
G_0^{ref} [kPa]	Reference shear modulus at very small strains	85000
m [-]	Power for the stress-level dependency of stiffness	0.79
$\gamma_{0.7}$ [%]	Shear strain at which $G_S = 0.722 \cdot G_0^{ref}$	0.032
p'_{ref} [kPa]	Reference stress for stiffness	100
R [-]	Failure ratio	0.9
σ_{tens} [kPa]	Tensile strength	0

5 NUMERICAL RESULTS

Results are presented in terms of profiles of maximum shear strain with depth, $\gamma_{max}(z)$, pseudo acceleration response spectra, $Sa(T)$, at 5 m b.g.l. (*i.e.*, the tower foundation depth) and amplification factors, $AF(T)$, given by:

$$AF(T) = \frac{Sa(T, z = 5 \text{ m b.g.l.})}{Sa(T, \text{outcrop})} \tag{3}$$

where $Sa(T,$ outcrop) is the response spectra of the reference motion. In addition, differences between EQL and NL results are highlighted using the residuals of maximum shear strain and acceleration response spectra. In particular, these latter were quantified by mean of:

$$\Delta P[\%] = \frac{P_{EQL} - P_{NL}}{P_{NL}} \cdot 100 \tag{4}$$

where P is γ_{max} or Sa. Hence, ΔP lower and higher than 0% is referred to EQL results lower and higher than the NL ones, respectively.

Figs. 5a and c refer to the profiles of mean γ_{max} with depth for Models A and B, respectively, where the value at each depth was obtained as the mean value for the seven input motions. Figs. 5b and d show the box plots of the residuals quantified by means of Equation (4) at each examined depth. EQL mean $\gamma_{max}(z)$ profile is generally characterised by lower values than the NL one, being the maximum value of residual equal to about - 25% in the upper portion of the analysed soil columns.

It is worth noting that the variability of the residuals (*i.e.*, the size of the box plots in Figs. 5b and d) enlighten the variability of the EQL results when compared to the NL ones for each one of the 7 input motions. In detail, the highest differences are for thickest box, hence for z lower than 40 m b.g.l. and z in the range 100-90 m b.g.l. referring to Model A and z lower than 50 m b.g.l. and z in the range 230-170 m b.g.l. for Model B.

At the foundation depth, the two numerical approaches are however consistent, both in terms of γ_{max} and response spectra (Figs. 6a and b), showing in this latter case only slightly lower spectral accelerations in the case of the EQL analyses in comparison to the NL ones (see Figs. 6c-d).

Differences for the two models A and B are highlighted in Fig. 7. Model B mean shear strains are generally lower than those of Model A, but the two mean response spectra are quite similar at the foundation level of the *Two Towers*.

Finally, amplification for T equal to 1.35 and 3.00 s, *i.e.* the fundamental periods of the *Garisenda* and *Asinelli Towers*, respectively, are listed in Table 4. Overall, the amplification factors obtained by the EQL analyses are lower than the NL-based ones. Moreover, Model A amplification is higher and lower than Model B one for T equal to 1.35 and 3.00 s, respectively, as expected, since the higher the thickness of the soft soil deposit the higher the fundamental period of the same deposit. In fact, considering time-averaged V_S equal to 346 and 398 m/s for Models A and B, respectively, the fundamental periods are 1.7 and 2.3 s, quantified by:

$$T = \frac{V_S}{4H} \tag{5}$$

As a matter of fact, the latter equation is referred to linear elastic homogenous soil deposit but allows to preliminary assess amplification phenomena.

Table 4. Amplification factors for T equal to 1.35 s and 3.00 s.

		Model A		Model B	
		NL	EQL	NL	EQL
1.35 s	minimum	1.87	1.87	1.69	1.70
	mean	2.63	2.34	2.19	2.02
	maximum	3.05	2.71	2.61	2.40
3.00 s	minimum	1.55	1.60	1.87	1.68
	mean	1.92	1.82	2.17	1.99
	maximum	2.29	2.11	2.55	2.20

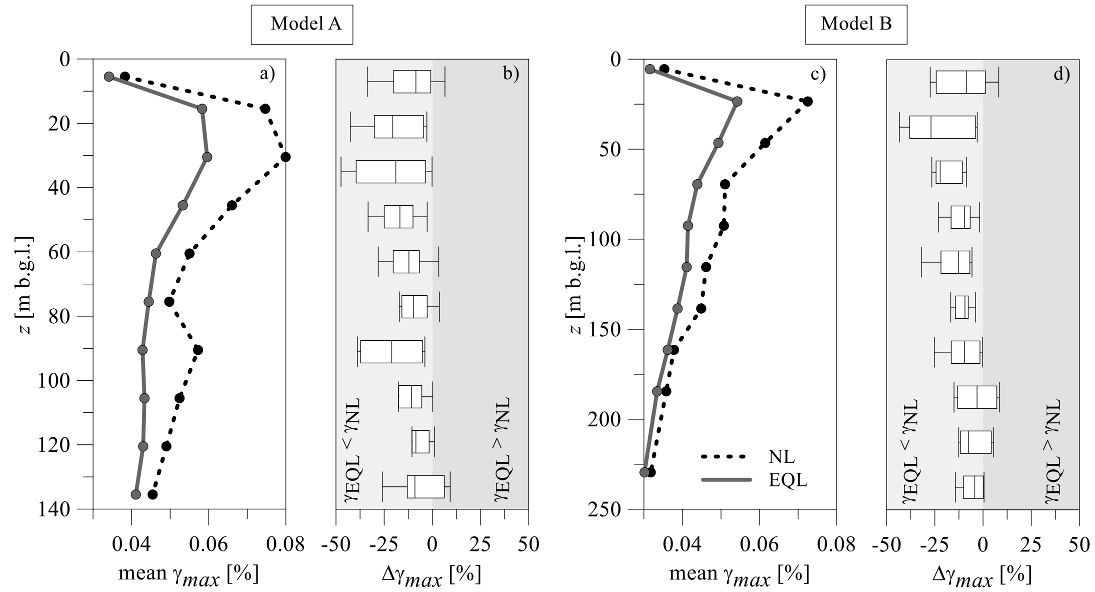

Fig. 5. Profile of mean values of $\gamma_{max}(z)$ obtained by means of NL and EQL approaches and boxplots of $\Delta\gamma_{max}$.

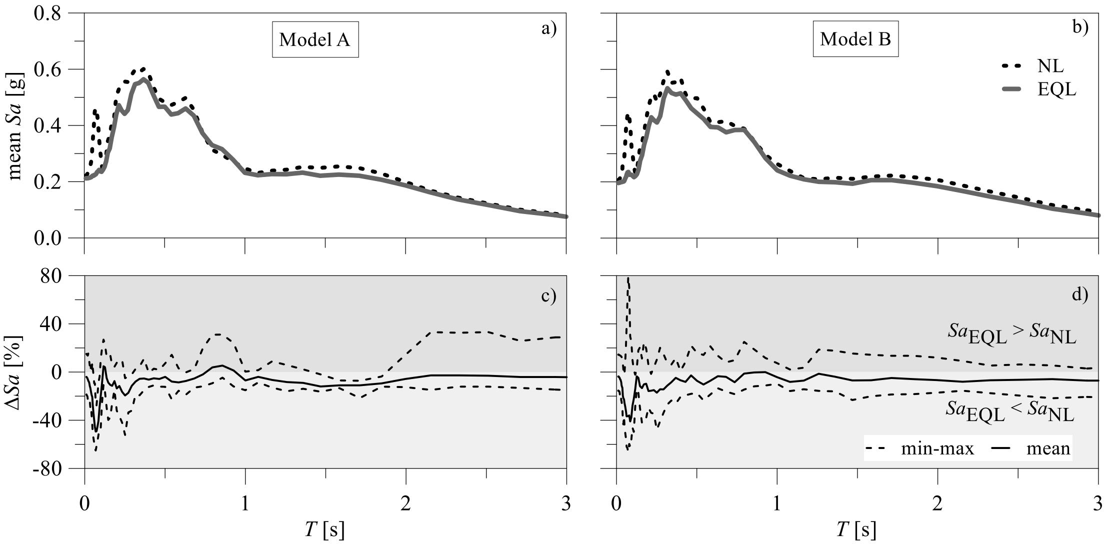

Fig. 6. Mean pseudo-acceleration response spectra obtained by means of NL and EQL approaches and residuals $\Delta Sa(T)$.

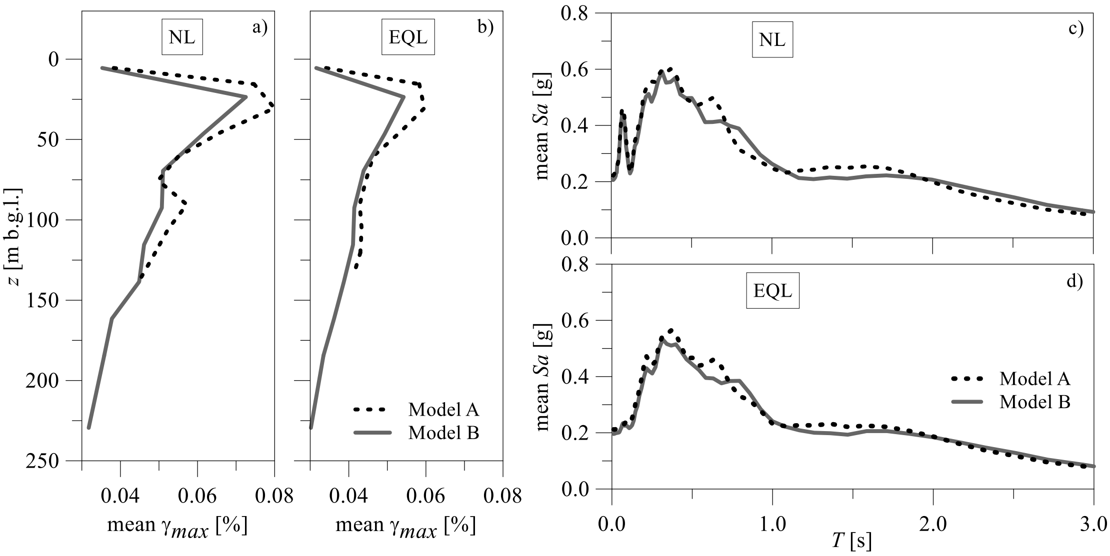

Fig. 7. Comparison between results based on Models A and B in terms of a-b) profile of mean values of γ_{max} with depth and c-d) mean response spectra calculated at the tower foundation depth.

6 CONCLUSIONS

Seismic site response of the *Two Towers* subsoil was conducted applying the equivalent linear elastic and non-linear approaches. The input for the analyses included extensive site-specific and laboratory data, which enabled to define a shear wave velocity profile with depth, along with the curves depicting the decay of the shear secant stiffness and the corresponding increase in the damping ratio. However, the experimental campaign for seismic purposes could reach only a depth of 40 meters b.g.l.. Consequently, two distinct numerical models (A and B) were introduced, differing for the depth of the bedrock, respectively located at 150 meters and 230 meters b.g.l.. A selection of 7 input motions was made from the European Strong Motion Database to match the target response spectrum, defined considering the reference seismic hazard for a return period of 712 years.

In relation to the examined case study, the main conclusions are as follows:

- the results obtained using the EQL approach yielded lower values compared to those from the NL approach, primarily evident in terms of maximum shear strain profiles at varying depths;
- the peak ground acceleration at the foundation depth (5 m b.g.l.) closely corresponded with that of the input motion;
- even though the peak spectral acceleration at the foundation depth was observed for periods lower than 1.0 s, a significant seismic amplification (*i.e.* the ratio of the signal at the foundation depth to that at the outcrop) was observed in correspondence of the fundamental periods of the *Two Towers* (about 1.35 s for the Garisenda and 3.00 s for the Asinelli);
- for a spectral period of 1.35 s, the response spectrum at the foundation depth (5 m b.g.l.) for Model A slightly exceeded that of Model B, while the reverse was observed for a spectral period of 3.00 s.

The current main limitations of the study are the lack of data of the shear wave velocity profile at greater depth and the actual deformability of the bedrock. In fact, for this specific case, it was shown that the position of the bedrock does not seem to strongly affect the ground motion at the tower foundation depth.

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