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Evaluation of the seismic behavior of multi-propped shallow underground structures embedded in granular soils: A comparison between coupled and decoupled approaches

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ABSTRACT: The evaluation of the seismic behavior of underground structures represents one of the most actual seismic geotechnical and structural engineering research topics about the study of the complex phenomena of soil-structure interaction.

In the last decades, different types of simplified and numerical approaches have been developed for the evaluation of the seismic behavior of these structures, especially after the recent strong earthquakes where they have been subjected to significant damages. In the same way, in the last few years, the International Codes are beginning to pay attention to the concepts of their seismic design. Despite the significant development of knowledge still remain open several uncertainties of the correct reproduction of the underground structures behavior under seismic load.

The paper presents the main results of the comparison between coupled and decoupled approach used for the evaluation of the seismic behavior of multi-propped shallow underground structure embedded in granular soils, considering four homogeneous soil profiles characterized by a different value of shear wave velocity and five natural accelerograms included within the European Strong Motion Database. The results of the analysis show the influence of the soil characteristics related to the seismic signal parameters on the seismic response of the structure in terms of maximum bending moment acting on the concrete retaining walls.

1 INTRODUCTION

For a long period, the underground structures were considered less vulnerable to earthquake compared to above-ground constructions. This assumption has been denied as a result of damage observed following strong motion earthquakes in the last 30 years and in particular the 1995 Kobe (Japan), the 1999 Chi-Chi (Taiwan) and the 1999 Kocaeli (Turkey) earthquakes (Hashash et al., 2010), when their seismic design was not adequate.

An effective seismic design requires a very good knowledge of the behavior of this type of structures under seismic loads. Respect to above-ground structures, in facts, the underground structures exhibit a complex soil-structure interaction phenomenon that contribute to create uncertainty on the evaluation of their seismic behavior.

The goal of this study is to further improve the knowledge of the underground structure seismic response, considering a multi-propped underground structure embedded in granular soils through the application of decoupled and coupled approaches. The decoupled approach consists in the evaluation of the ground deformation without taking into consideration the presence of the structure (i.e. 1D free field condition) and in the application of such deformations to the structure computing the structure response parameter (bending moment, in this work). The coupled approach, instead, was carried out by the fully dynamic nonlinear analysis, considering a numerical model implemented in FLAC^{2D} which represents both structure and soil.

2 SENSITIVITY ANALYSIS

As mentioned before, the behavior of the underground structures under seismic action is strictly related to the response of the surrounding soil to a given seismic input. In particular, if the fundamental frequencies of the soil profile are near to the frequencies of the seismic input characterized by the maximum energy content, the occurrence of the resonance effects is possible (Tropeano & Soccodato, 2014; Soccodato & Tropeano, 2015).

Different types of approach were developed for the evaluation of the seismic behavior of the underground structures that consider the soil-structure interaction effects (coupled approach) or not (decoupled approach). This fact can lead to obtaining different values of internal actions on the structural elements.

To evaluate the seismic behavior, a sensitivity analysis, based on the performance-based design approach, was conducted considering the variation of shear modulus of the homogeneous soil which the structure is embedded. The model used is composed by the presence of 31.6 m of dry coarse-grained soil profile overlying 51.3 m of sandstone and the bedrock.

2.1 Soil profiles

Four different soil profiles have been defined to have a shear wave velocity equal to 360 m/s, 450 m/s, 600 m/s and 750 m/s. Table 1 lists the main mechanical properties of the soil profiles considered in this analysis which are obtained considering the mechanical properties of the Lima (Perù) outback soil (Zucca, 2019).

In particular, the profiles consist of a layer of dry coarse-grained soil, 31.6 m in thickness, resting upon a rigid bedrock. An elastic perfectly plastic model with Mohr-Coulomb yield locus, characterized by mechanical properties listed on Table 1 was adopted. The soil hysteretic behavior was modeled using the shear modulus decay curves given by Seed and Idriss (1970). The hysteretic damping is, however, computed by applying the generalized Masing criteria implemented in the computer code used in this study. The dilatancy, considered in this study, is equal to zero.

2.2 Structure

A representative section of the shallow multi-propped underground structure, considered in this study, is shown in Figure 1.

The principal structural elements that characterized the structure are the concrete retaining walls, 1 m thick, and a series of concrete circular columns, 1.2 m diameter, positioned in a

Table 1. Mechanical properties of soil profiles (γ = unit weight, c' = cohesion, ϕ' = friction angle, ν = Poisson ratio).

Soil	γ [kN/m ³]	model	c' [kPa]	ϕ' [°]	ν [-]	V_S [m/s]
coarse-grained soil	21	elasto-plastic	40	39	0.25	360 ÷ 750
sandstone	22	elastic	-	-	0.38	1750

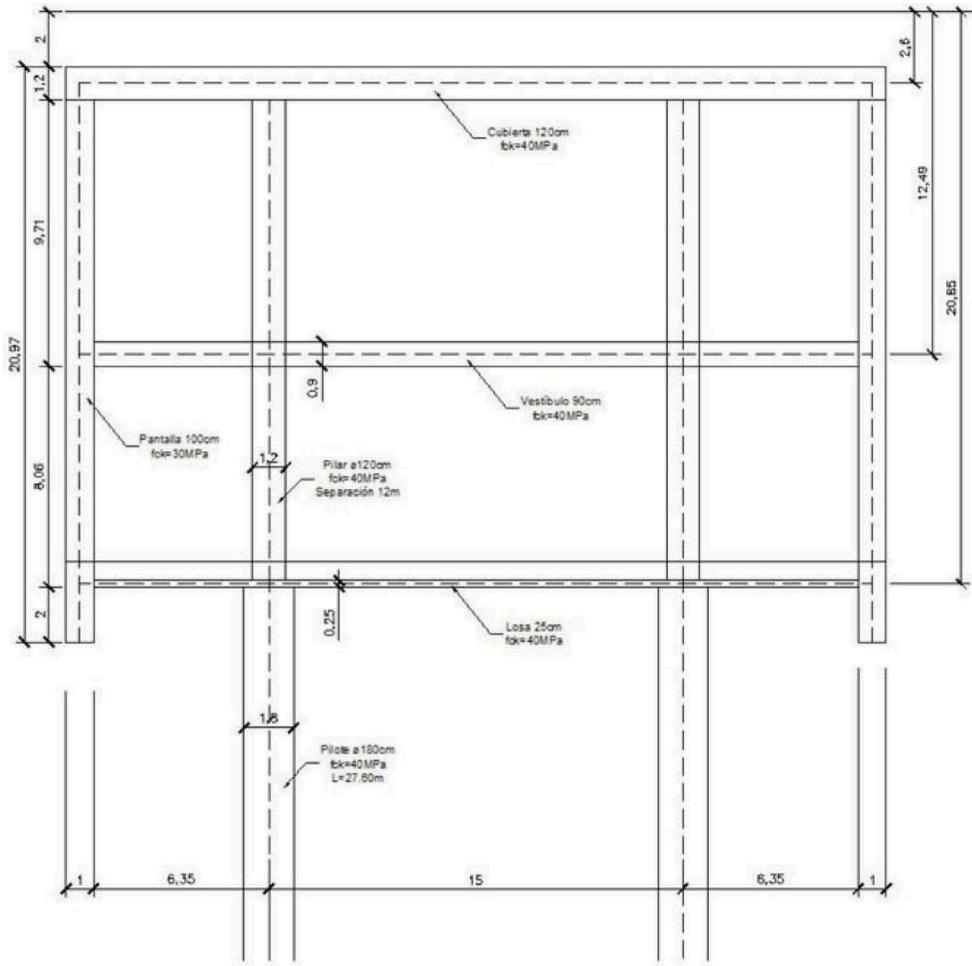


Figure 1. Representative section of the structure.

regular grid 14.70×12.00 m. The foundation of the columns consists of circular concrete piles, 1.8 m diameter and 9 m deep. The materials mechanical properties of these elements are listed in Table 2.

2.3 Seismic inputs

For the seismic analysis, five acceleration times histories recorded during different European seismic events (Greece, Amatrice, L'Aquila, Friuli and Montenegro), included within the European Strong-Motion Database (Luzi et al., 2016) have been used. The principal

Table 2. Mechanical properties of the structural elements.

Structural elements	Columns	Pile	Retaining Walls	Slabs
concrete (f_{ck}) [MPa]	40	40	30	40
steel (f_{yk}) [MPa]	420	420	420	420

Table 3. Characteristics of the seismic inputs.

Event	M_w [-]	Station	PGA [g]	I_A [cm/s]
Greece	6.5	AIGA	0.52	117.1
Amatrice	6.5	AMT	0.53	156.4
L'Aquila	6.1	AQG	0.49	132.4
Friuli	6.0	FRC	0.35	84.5
Montenegro	6.9	PETO	0.45	455.7

characteristics of the seismic inputs (PGA = peak ground acceleration, I_A = Arias Intensity) are listed in Table 3. The accelerograms have been selected for event magnitude between 6 and 7, peak ground acceleration value between 0.4 g and 0.6 g and in such a way the other integral ground motion parameters have maximum variability.

In Figure 2 the seismic signals in frequency domain are shown. Note that the peaks of the Fourier transform of the different seismic inputs are localized near the frequencies range that include the first fundamental frequency of the four soil profiles. Only for the Montenegro seismic input, all the main picks are found at lower frequencies compared to the first fundamental frequency that characterized the different soil profiles.

2.4 Numerical modelling

The dynamic analyses were carried out, under plane strain conditions, through the finite difference code FLAC (Itasca, 2007). The numerical model and the relative computational grid is shown in the Figure 3.

An elastic perfectly plastic model with Mohr Coulomb yield locus for the soil, characterized by the mechanical properties listed in Table 1, was adopted while for the sandstone a simple elastic constitutive law was chosen (Zucca et al., 2017). The shear stiffness at small strain, G_0 , is calculated as a function of the shear waves velocity value, V_s .

According to Kuhlemeyer & Lysmer (1973) criterion, the maximum size of the computation mesh elements has been fixed to allow the correct propagation of harmonics with 18 Hz,

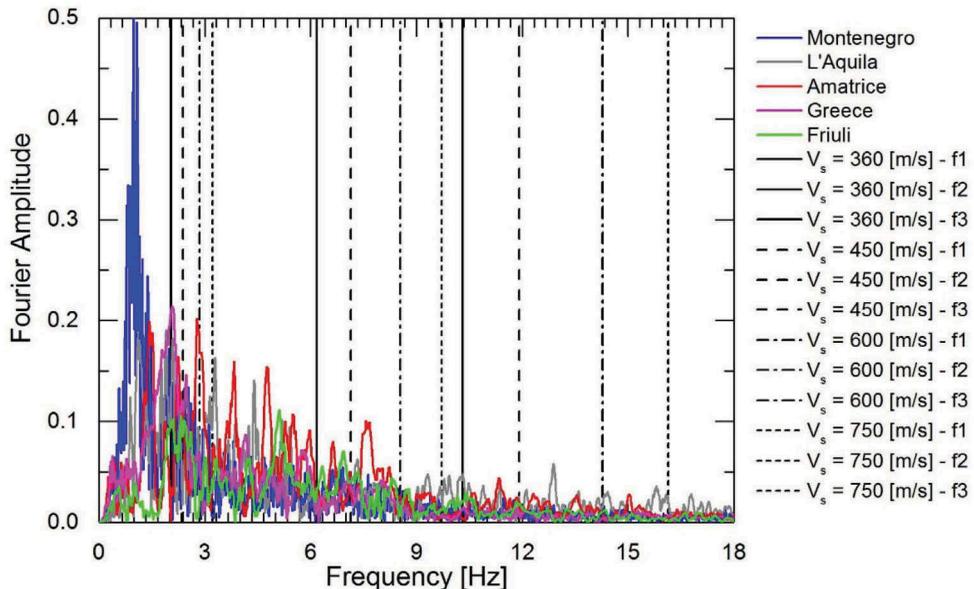


Figure 2. Fourier Transforms of the seismic inputs.

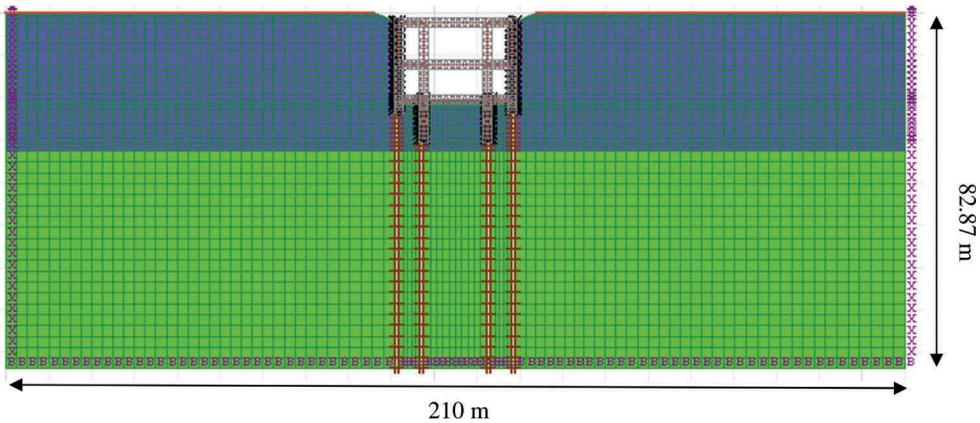


Figure 3. Model geometry and computation grid.

which is the maximum frequency of the seismic inputs used in this work. The formulation to optimize the size of the mesh is given in Pagliaroli et al. (2007).

To minimize reflection effects on vertical lateral boundaries of the computational grid, free field boundary conditions available in FLAC 7.0 library have been used. The contact between soil and walls was modeled by using elastic-perfectly plastic interface elements, with a friction angle equal to 20° .

2.5 System response

The response of model appears significantly influenced by different overlapping effects:

1. different stiffness of the soil profiles, due to the different values of shear wave velocity;
2. non-linear behaviour of the soil;
3. geometry of the system (2D effects);
4. soil-structure interaction effects.

The comparison between the parameters of the seismic inputs and those obtained at surface level enables an evaluation of system response in relation to the first two effects mentioned above.

The results of the free field analysis (i.e. in 1D condition for the scheme considered in this study), in terms of acceleration ratio (ratio of maximum acceleration at ground level, PGA_{S-1D} and maximum acceleration of the seismic input, PGA), are showed for each soil profiles in the Figure 4.

The soil response, at the different seismic inputs, is related to the system vibration modes excited by the signal. In fact, the seismic response of the soil column is meaningfully influenced by the soil motion and straining, in particular for the markedly non-linear behaviour of the soil.

As mentioned before, the seismic inputs have resonance effects with the first vibration mode of the different soil profiles, leading to an increase of the shear strain in the deeper soil layers and, consequently, additional damping according to the strongly non-linear behaviour of the soil (Socodato & Tropeano, 2015). For these reasons, the free field analyses exhibit larger amplification of the peak acceleration. Only for the unscaled accelerograms, the free field analyses show a reduction of the peak acceleration due to characteristics of the inputs which amplify the damping effects due to the nonlinear behaviour of the soil.

The effects 3 and 4 are evaluated considering the ratio between the maximum acceleration at the 2D model surface, PGA_{S-2D} , and the peak acceleration at the surface obtained in free field condition, PGA_{S-1D} . The results are summarized in the Figure 5.

The geometry of the system and the presence of the structure generate an amplification of the seismic motion behind the walls and at the center of the excavation for the focusing

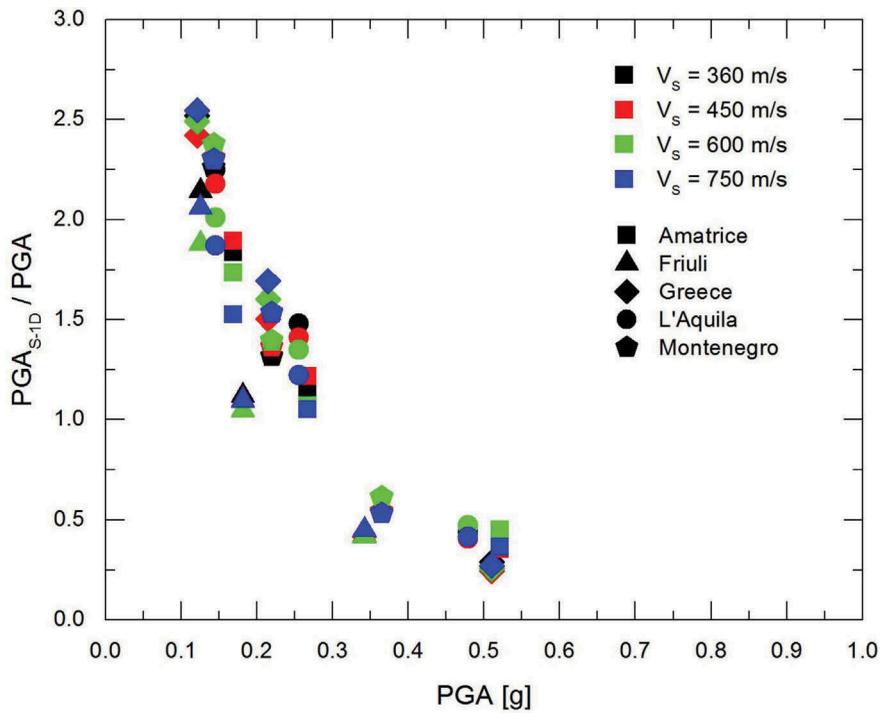


Figure 4. 1D response factor.

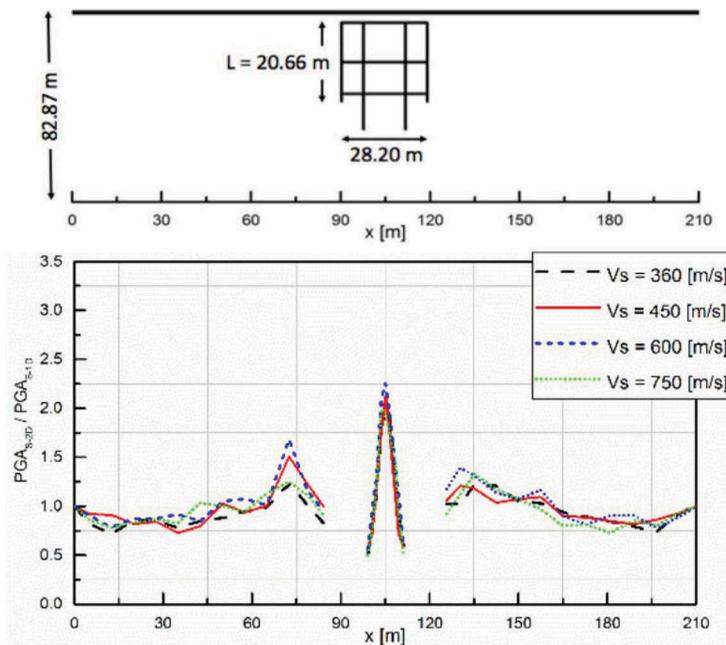


Figure 5. 2D response factor for L'Aquila seismic input.

phenomena of the waves and seismic motion attenuation in front of the retaining walls due to the diffraction phenomena of the waves. The presence of the structure, furthermore, due to its high stiffness, produces additional reflections for the soil-structure interaction effects. The interaction of reflected and incident wave fields modifies the shaking amplitude that depends on the phase shift of the two signals. The geometrical amplification and the phase shift are closely related to the frequency content of the signal that changes due to the non-linear behavior of the soil (Kuhlemeyer & Lysmer, 1973; Soccodato & Tropeano, 2015).

2.6 Decoupled approach

The decoupled approach, considered in this study, consists in the evaluation of the ground deformation without taking into consideration the presence of the structure (i.e. in free field condition) and in the application of such deformations (valued at the depth of the structure) to the structure, according to the scheme reported in Wang (1993), to obtain the bending moment acting on the retaining walls.

The seismic response of the soil profiles and, consequently, the displacements are evaluated at the roof and the base of the structure with two different analyses approaches. The 1D response analyses have been performed with DEEPSOIL (Hashash et al., 2016) adopting an equivalent linear formulation and fully non-linear approach. The results obtained through both approaches analyses are shown in Figure 6.

The displacements obtained by the equivalent linear analysis (DA-EL) shows a trend resulting from the first vibration mode shape of the soil column. These results are in according to the considerations expressed in the previous paragraph 2.3, i.e. the seismic signals mainly excite the first fundamental frequency of the soil profiles.

The second approach (DA-NL), used to obtain the displacements at the depth of the base and at the roof of the underground structure, is to consider the free-field displacements

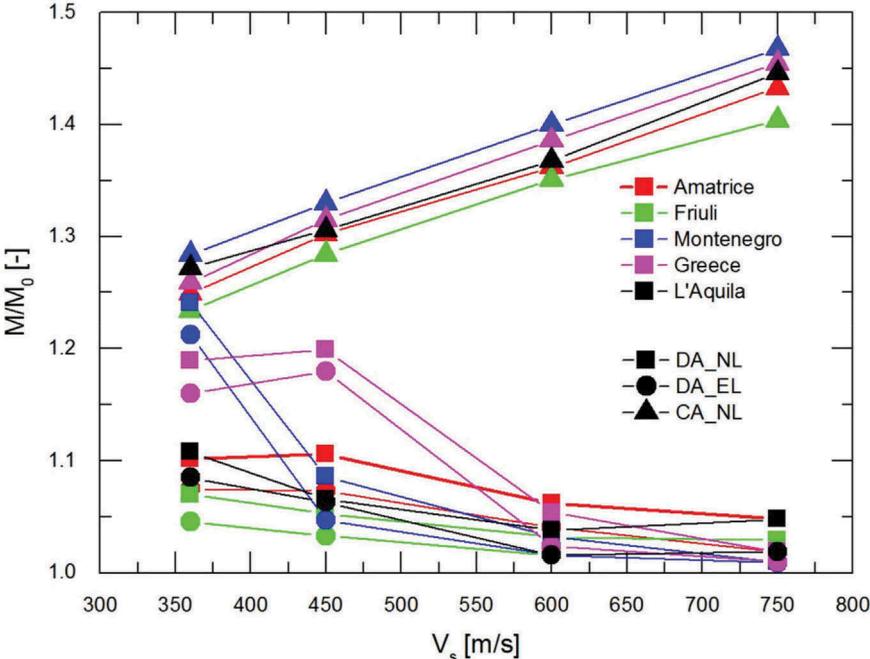


Figure 6. Comparison of the results (M = maximum moment occurred during the seismic action; M_0 = static moment).

resulting from the numerical model, described in the previous paragraph 2.4, through the execution of a fully dynamic non-linear analysis.

The comparison of the results (Figure 6) of the two different methods showed that the relative displacements obtained by the non-linear analysis are greater than the relative displacements obtained by the equivalent linear analysis. Consequently, the increase of the bending moment acting on the retaining walls follows this trend. The results show, also, that the difference of the values of the bending moment acting on the section in correspondence to the central slab obtained by non-linear analysis, DA-NL, and that obtained by equivalent linear analysis, DA-EL, lies between 2% and 4% for all the selected accelerograms. This limited value is due to the incidence of the value of the moment obtained under static conditions on the value of the final moment compared to the increment due to the seismic action.

2.7 Coupled approach

The coupled approach is performed through one Finite Difference Model (FDM), implemented in FLAC 7.0 (Itasca, 2007) software, representing both structure and soil. The model characteristics are described in the previous paragraph 2.4. The evaluation of the seismic behaviour of the underground structure described above is carried out considering static and dynamic loads. In the static phase, the boundary conditions are the following: vertical supports in the base nodes to restrain the vertical displacements and horizontal supports in the lateral nodes of the mesh to permit vertical soil settlements. The external loads considered are: the dead load on the sides of the station equal to 50 kPa due to the presence of existing buildings and streets, the dead load in correspondence to the station equal to 20 kPa due to the presence of the streets and the self-weight of the structure and the soil.

The static condition is determined by performing a construction stage analysis, accounting for all the main phases involved in the construction of the structure.

The dynamic analysis is performed, after the final step of the construction stage, by using fully non-linear dynamic analysis in time domain considering the five different seismic inputs.

The results obtained with coupled approach (labelled as CA-NL) and are shown in Figure 6.

3 RESULTS

Figure 6 shows the summary of the results obtained by the different approaches for all the seismic inputs.

The results of the equivalent linear analysis (DA-EL) indicated that the maximum increase of bending moment during the seismic action, in the section in correspondence to the central slab, is equal to 110 kN m for Montenegro seismic input. Bending moment increase is almost negligible in cases where the soil profile is characterized by shear waves velocity equal to $V_S = 600$ m/s and $V_S = 750$ m/s.

Considering the fully non-linear 1D dynamic analyses results (DA_NL), the maximum increment of bending moment occurred in the profile characterized by $V_S = 360$ m/s for Montenegro earthquake and it is equal to 125 kN m. The difference of the results obtained by the equivalent linear analysis and that resulted from non-linear analysis were between 2% and 4% for all the accelerograms. As mentioned before, this limited value is due to the incidence of the value of the moment obtained under static conditions on the value of the post-seismic moment compared to the increment due to the seismic action.

The results of the coupled approach (CA-NL) showed that for all the accelerograms the increment due to the seismic action was higher for the soil profiles characterized by $V_S = 600$ m/s and 750 m/s in contrast to the decoupled approach results. The maximum increment, also in this case, occurred for Montenegro seismic signal and was equal to 150 kN m for the soil profile characterized by $V_S = 750$ m/s.

The trend of the maximum value of the bending moment, obtained during the seismic event, decrease of the maximum value when the soil column stiffness increases.

4 CONCLUSIONS

The values, in terms of maximum bending moment acting on the section in correspondence to the central slab during the seismic event, obtained by the coupled approach are always greater than the values obtained by the decoupled approach.

For all the selected accelerograms these differences are moderate for the soil profiles characterized by a low stiffness, confirming the validity of the decoupled approaches, but tending to increase for the soil profiles characterized by $V_S = 600$ m/s and 750 m/s where the decoupled approach appears underestimate the internal actions on the structural elements.

Furthermore, the results obtained are affected by the different assumptions which characterize the methods. In particular, the decoupled approach tends to lose the criterion of the contemporaneity of the actions because they are considered only the maximum displacement that occurs during a seismic event.

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