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Influence of Soil-Structure Interaction on the Design of Seismic Retrofitting Interventions for Existing Reinforced Concrete Structures

Romeo Tomeo[a], Antonio Bilotta[a], Dimitris Pitilakis[b], Emidio Nigro[a]
a Department of Structures for Engineering and Architecture, University of Naples Federico II, 21, Via Claudio, 80125, Naples, Italy
b Department of Civil Engineering , Aristotle University of Thessaloniki, 54124, Thessaloniki, Greece

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ABSTRACT
In the present paper we show the results of non-linear dynamic analyses performed on 2D Reinforced Concrete (RC) Moment Resisting Frames (MRFs) designed for vertical loads only, in order to investigate the influence of Soil-Structure Interaction (SSI) on the design of seismic retrofitting interventions for existing buildings. The analyses show that, for flexible structures such as MRFs, SSI can imply, with respect to an analysis of the fixed-base structure, a reduction of the seismic demand in terms of maximum inter-storey drift ratios and maximum base shear. This results in a significant saving in terms of quantity of needed reinforcement, and thus in economic terms in the design process of seismic retrofitting interventions. On the contrary, for stiffer structural systems like mixed systems with both frames and shear walls (usually used to increase the capacity in terms of force of the building), modelling of SSI can produce, with respect to the fixed-base configuration, an increase of the seismic demand. In this case, the usual practice to consider the structure fixed at the base turns to be un-conservative and unsafe for the evaluation of the seismic demand.

1 INTRODUCTION

The rehabilitation of seismically vulnerable buildings is an important problem in earthquake engineering. In recent decades, the goal of building rehabilitation and strengthening has gained research attention and numerous techniques have been developed (Kaplan et al., 2011).

Among these techniques, fiber-reinforced polymers (FRP) products or stiffening shear walls are commonly used in practice to seismically retrofit existing reinforced concrete (RC) moment resisting frames (MRFs).

FRP composites materials are very attractive for use in civil engineering applications due to their high strength-to-weight and stiffness-to-weight ratios, corrosion resistance, light weight and potentially high durability. Their application has recently increased in the rehabilitation of concrete structures, mainly due to their tailorable performance characteristics, ease of application and low life cycle costs (Van Den Einde et al., 2003).

Usually, FRP products are used to increase the ductility capacity of the structural elements, as well as to avoid the possible formation of brittle shear failures (Priestley & Seible, 1995), and consequently to improve the global performance of the structure.

Shear walls are often used, instead, to increase the lateral load capacity as well as the global stiffness of the structural system .

However, it is worth noting that these interventions are usually designed, in engineering practice, assuming that the structure is fixed at the base, completely ignoring, thus, the Soil-Structure Interaction (SSI) effects.

In reality, flexibility of the supporting soil medium allows some movement of the foundation. This decreases the overall stiffness of the building frames resulting in a subsequent increase in the natural periods of the system and, thus, the overall response is altered (Dutta et al., 2004).

Moreover, nonlinear behaviour at the soil–foundation interface due to mobilization of the ultimate capacity and the associated energy dissipation, particularly in an intense earthquake event, may be utilized to reduce the force and ductility demands of a structure, provided that the potential consequences such as excessive settlement are tackled carefully (Raychowdhury, 2011).

For these reasons, a more precise knowledge of the expected structural seismic response could allow to reduce the cost of the structure and to improve the earthquake engineering practice (Saez et. al, 2011).

In this paper we present the results of non-linear time-history analyses performed for a 4 floors 2D RC-MRF designed for vertical loads only, with the aim of investigating the impact of SSI effects on the structural response and, thus, on the vulnerability assessment and on the design strategy of seismic retrofitting interventions.

We performed the analyses for: (i) the "bare" frame, (ii) the frame strengthened by means of a RC shear wall and (iv) the frame strengthened by means of FRP wraps (increase of confinement).

We modelled SSI by means of a "direct" approach, in which the soil, the foundation and the structure are analysed in a single step. This kind of approach was preferred over a simpler "sub-structures" approach inasmuch it allows to take implicitly into account the frequency variability of the foundation stiffness and damping.

The study shows that, on one hand, SSI effects can be substantially beneficial in the case of a "bare" MRF, inasmuch they involve a reduction, with respect to a fixed base model, of the seismic demand both in terms of maximum base shear and of maximum inter-story drift ratio. Moreover, SSI effects can imply a reduction of the amount of FRP needed to increase the seismic performances of the structure.

On the other hand, the study shows that for stiffer structural systems SSI can have a detrimental effect. So, the designers should carefully evaluate the possibility to take into account SSI effects both in the phase of vulnerability assessment and in the design process of a retrofitting intervention that can lead to an increase of the overall stiffness of the structure.

2 STRUCTURE, SOIL AND RECORDS

We selected a 4 floors Reinforced Concrete (RC) 2D Moment Resisting Frame (MRF) as reference structure for dynamic analyses.

In order to simulate the behaviour of a structure designed without seismic provisions, we designed the building for gravity loads only (according to the Italian Ministerial Decree of the 30^{th} of May 1972) by means of elastic calculations based on the allowable stress method.

For concrete we assumed, in the design phase, a characteristic cubic compression resistance equal to R_c = 25 MPa, while for steel rebar we assumed a tension resistance equal to 380 MPa (steel grade A38).

We assumed the floor loads equal to 7.84 kN/m^2 for the all the floors except for the last one, for which we considered a load of 5.84 kN/m^2.

The elevation layout of the building, as well as the section geometries and the reinforcing details are reported in Figure 1and Table 1.

We defined the dimensions of the simple footings (area of 3.20m x 1.75m for internal footings and of 1.75m x 1.75m for external footings) assuming a maximum stress for the soil of 200 kN/m^2, in order to avoid reaching the bearing capacity for vertical loads.

The structure has a first period of vibration, in case of fixed base, of 0.97 s.

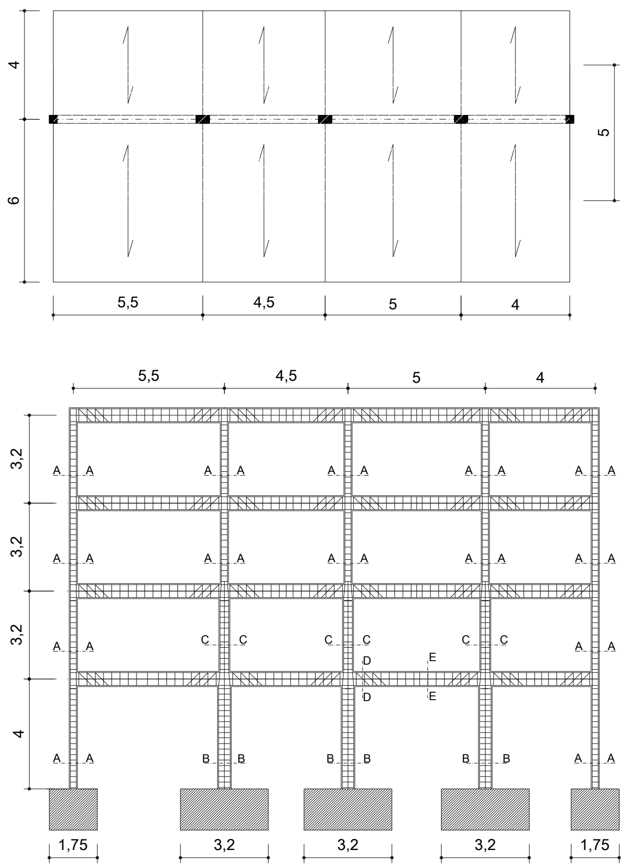

Figure 1 – Reference structure

For the analyses we referenced a medium clay sortable as soil type C according to EC8, in order to obtain significant SSI effects.

In Table 2, the soil properties considered in the study are reported, with the indication of the plasticity index used to choose an appropriate shear modulus reduction curve and a damping curve from those proposed by Darendeli (2001) for clays at a confining pressure of p_0' =1 atm.

Table 1 – Geometrical dimensions and reinforcements

Sez.	Element	B [m]	H [m]	Reinforcements	
				Long.	Trasv.
A-A	Column	0.30	0.30	4ϕ12	ϕ8/20cm
B-B	Column	0.30	0.50	6ϕ14	ϕ8/20cm
C-C	Column	0.30	0.40	6ϕ12	ϕ8/20cm
D-D	Beam	0.30	0.55	5ϕ20 (M-) 2ϕ20 (M+)	ϕ8/30cm
E-E	Beam	0.30	0.55	2ϕ20 (M-) 5ϕ20 (M+)	ϕ8/30cm

Table 2 – Mechanical properties of soil

	Soil type C
Height of the soil deposit	30 m
Type of soil	Clay
Plasticity Index	15%
Shear wave velocity (V_{s0})	250 m/s
Density (ρ)	2.0 t/m^3
Cohesion (c)	65 kPa

For the dynamic analyses, we chose a set of seven accelerograms by means of the software Rexel (Iervolino et al., 2009), compatible, on average, with the Eurocode 8 type 1 spectrum (high seismicity zone).

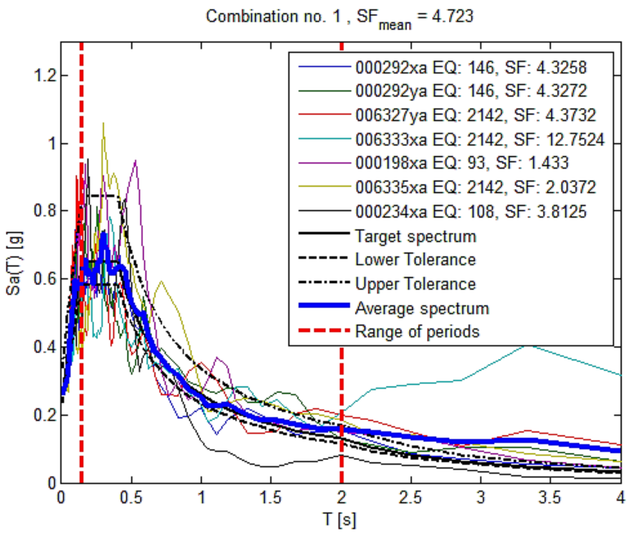

Figure 2 – Records and their compatibility

The records refer to outcrop conditions, recorded at site conditions classified as rock according to EC8 (soil type A) with moment magnitude (M_w) and epicentral distance (R) that range between 5.0 < M_w < 7.0 and 0 < R < 30 km respectively. We checked the compatibility of the records with the response spectrum in the period range from 0.15 s < T < 2.0 s (see Figure 2).

3 RETROFITTING INTERVENTIONS AND NUMERICAL MODELLING

Two possible seismic retrofitting interventions were assumed for the reference structure (see Figure 3):

- a retrofit by means of a RC shear wall of height 5.5 m and thickness of 20 cm, reinforced with longitudinal rebars of 20 mm;
- a retrofit with 2 layers (thickness of 0.66 mm) of CFRP wrap of the nodal zones in order to obtain a greater concrete confinement and, thus, an increase of the ductility of the plastic hinges.

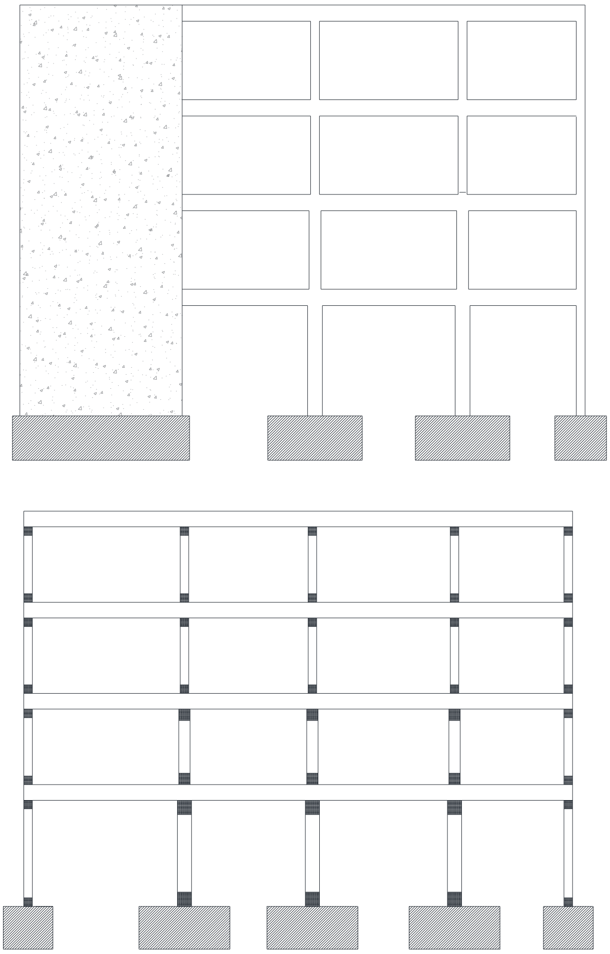

Figure 3 – Seismic retrofitting interventions

Concerning the intervention with FRP, the increase of the resistance and of the ultimate strain on compressed concrete were evaluated based on the formulations provided in the document CNR-DT 200 R1/2013:

$$\frac{f_{ccd}}{f_{cd}} = 1 + 2.6\left(\frac{f_{1,eff}}{f_{cd}}\right)^{2/3} \quad (1)$$

$$\varepsilon_{ccu} = 0.0035 + 0.015\sqrt{\frac{f_{1,eff}}{f_{cd}}} \quad (2)$$

where f_{cd} is the design resistance on unconfined compressed concrete and $f_{1,eff}$ is the effective confinement stress, dependent by the shape of the transversal section of the column and of the realization technique of the intervention.

Based on the dimensions of the columns of the building, and assuming per the CFRP E_f = 270000 MPa and $\varepsilon_{fd,rid}$ = 4‰, it was assumed: f_{ccd} = 28 MPa e ε_{ccu} = 0.0085.

We implemented the numerical models with the OpenSees software (Mazzoni et al., 2009).

As concerns the structural modelling, we adopted lumped mass models, and we modelled the non-linear structural behaviour by means of a lumped plasticity approach.

Moreover, we took into account the possible development of brittle shear failures by means of springs, working in series beam elements, that are activated when the shear force in the column reaches its ultimate value, defined based on what suggested by Sezen & Moehle (2004).

To account for the viscous damping mobilized during the dynamic response of the structure, we assigned tangent stiffness proportional damping (Priestley & Grant, 2005) with a damping ratio of 5%.

The RC shear wall was modelled with a ‘beamWithHinges’ element at the first floor and with ‘ElasticBeamColumn’ elements at the upper floors.

The non–linear behaviour of the wall was modelled by means of a fiber plastic hinge at the base of the frame element at the first floor. The length of the plastic hinge was assumed equal to 0.3 l_w, with l_w equal to the height of the wall, as suggested by Paulay & Priestley (1992).

The connection between the wall and the beams was modelled by means of rigid links.

Concerning the building retrofitted with CFRP, the constitutive law of confined concrete in the plastic hinge regions was suitably modified based on the values of resistance and ultimate strain previously defined.

Concerning the modelling of SSI, we implemented a complete FEM model. In particular, we modelled a 2D soil deposit (plane strain conditions) with homogeneous mechanical properties and with a bedrock placed at a depth of 30 m under the surface. We incorporated the soil nonlinearity in the model by means of an elastic-isotropic material with an elastic modulus properly reduced to take into account the shear strain amplitude (as suggested by FEMA 440, 2005) and viscous damping employed in the frequency-dependent Rayleigh form (Rayleigh & Lindsay, 1945).

This kind of modelling, in fact, is generally preferred because it facilitates dynamic analyses, although the damping in the soil is of hysteretic type and frequency independent.

Further details about the model (shown in Figure 4) are reported in Tomeo et al. (2017).

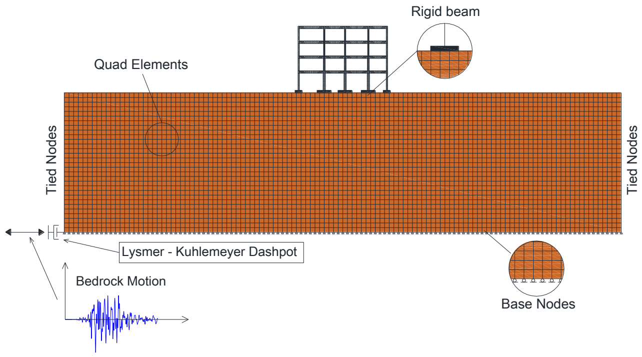

Figure 4 – OpenSees complete 2D FEM model

4 RESULTS OF THE NUMERICAL ANALYSES

In the next we show the results of the analyses performed for:

- the “bare” frame;
- the frame retrofitted by means of the RC shear wall;
- the frame retrofitted by means of the CFRP wrap.

The analyses were performed for both the fixed base model and the complete FEM model.

In order to obtain a significant comparison, in the fixed base model we took into account the site effects applying at the base of the models the free field motion (FFM) obtained by means of a 1-D wave propagation analysis of a soil column that has the same properties and the same constitutive law assumed for the soil in the complete FEM model.

In all the analyses, in order to investigate the structural behaviour from the linear field to the collapse, we scaled the records to eight different values of peak acceleration at the bedrock: 0.05g, 0.075 g, 0.10 g, 0.125 g, 0.15 g, 0.20g, 0.25 g, 0.30g.

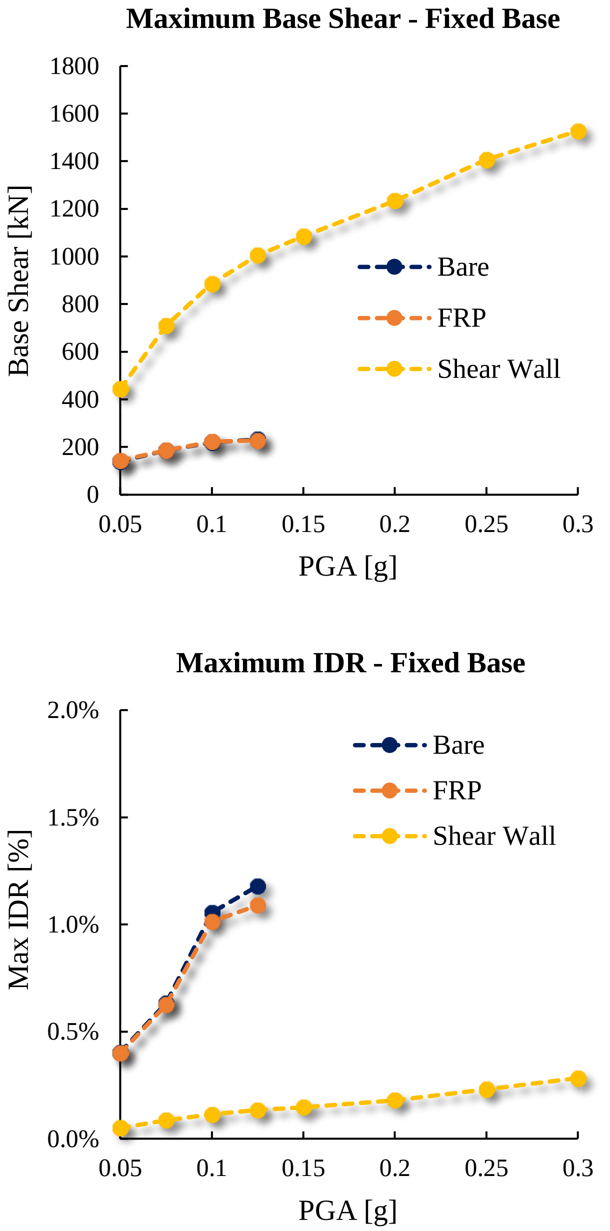

Figure 5 – Results for fixed base models

We chose the maximum base shear, V_{max}, and the maximum inter-story drift ratio, IDR_{max}, as synthetic engineering demand parameters.

In Figure 5and Figure 6 the results obtained for the models on fixed base and with SSI are shown, respectively.

The graphs represent the average, given the PGA, of the results obtained for all the records.

However, it's worth noting that the graphs were obtained considering only the values of the seismic demand not corresponding to a structural damage and, thus, sometimes the average do not correspond to an average of 7 results. Moreover, the curves were stopped to a PGA level for which at least 3 values of the structural response were obtained.

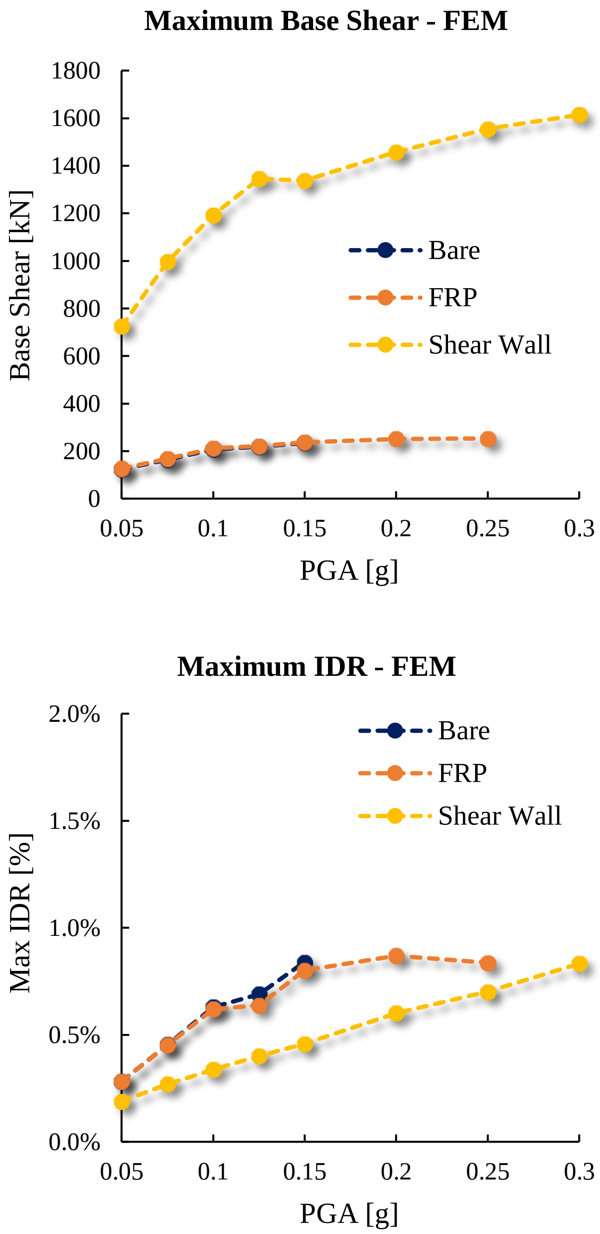

Figure 6 – Results for complete FEM models

Concerning the "bare" frame (blue curves), it can be noted that:

– the modelling of SSI with a FEM model allows to obtain reductions of the seismic demand, with respect to the fixed base model, up to 11% in terms of maximum base shear and up to 40% in terms of maximum *IDR*;
– the strong reduction of the maximum *IDR* can strongly affect the safety checks at the Damage Limit State (DLS); assuming, for example, a maximum allowable *IDR* of 0.5% (as suggested by Italian standards for constructions), the modelling of SSI lead to an increase of the maximum PGA that the structure is able to withstand without damaging, from 0.05 g to 0.075 g;

– the modelling of SSI can have, however, an important effect on the safety checks at the Collapse Limit State (CLS) too; assuming a conventional value of the maximum *IDR* of 1% (as suggested by Ghobarah 2004 for non-ductile frames), it can be observed that for the fixed base condition the structure is able to withstand, without collapsing, a maximum PGA of 0.1g, while with SSI the structure suffers a maximum *IDR* lower than 1% even for a PGA of 0.15 g.

For the building strengthened with the RC wall (yellow curves) the results show that the modelling of SSI could lead to an increase, with respect to the fixed base model, of the estimated seismic demand both in terms of maximum base shear and in terms of maximum IDR.

As shown in Figure 7, the maximum base shear at the base of the wall can increase up to 50% taking into account SSI.

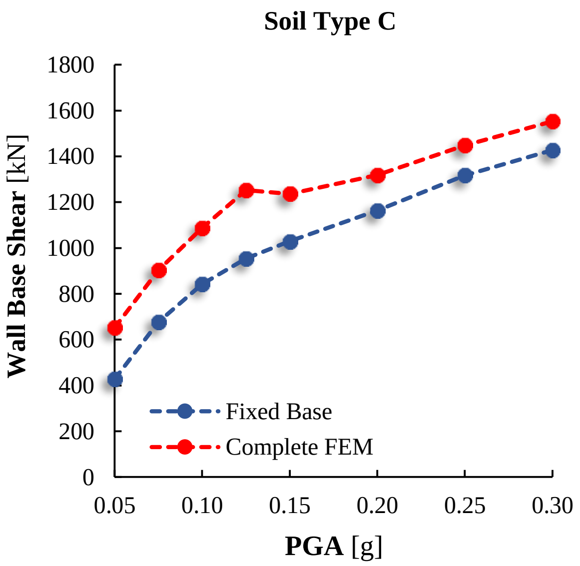

Figure 7 – Building retrofitted with RC wall: maximum shear at the base of the wall

The design of this kind of interventions could be strongly un-safe.

These results can be explained through some considerations related to easier linear analyses (i.e. analyses performed assuming linear structural behaviour).

In Figure 8, the acceleration recorded at the top of the structure is plotted for the fixed base model and for the complete FEM model.

Dividing the Fourier spectrum of the top acceleration by that of the free field motion, it is possible to obtain the transfer functions (see Figure 9) of the two systems and then the values of the fundamental vibration period.

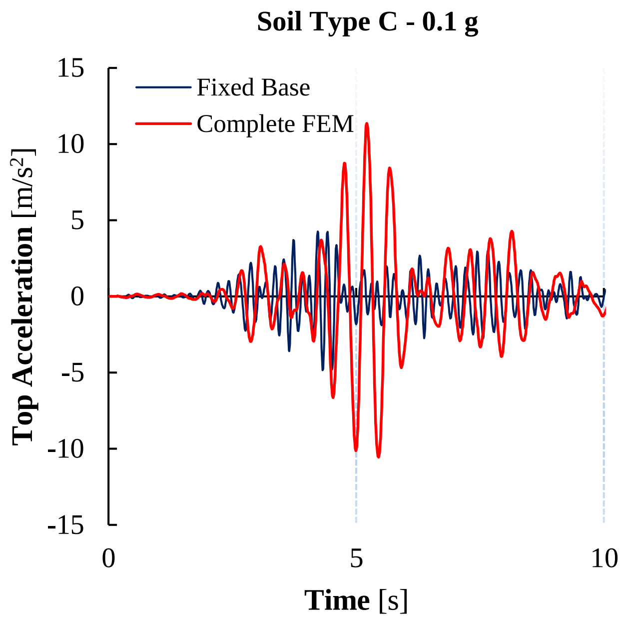

Figure 8 – Building retrofitted with RC wall: acceleration on the top for fixed base model and complete FEM model (linear analysis)

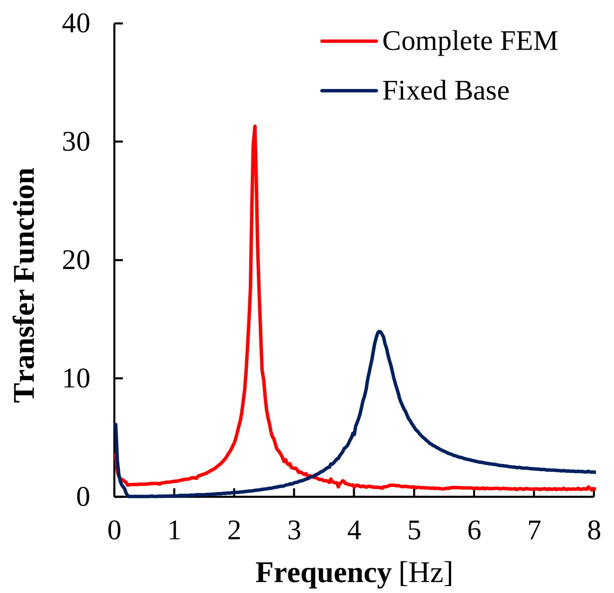

Figure 9 - Building retrofitted with RC wall: transfer functions for fixed base model and complete FEM model

As can be noted, the initial elastic period of the fixed-base structure is equal to T = 0.22 s (f = 4.55 Hz) while the modelling of SSI lead to an increase of the fundamental period up to a value of T = 0.43 s (f = 2.34 Hz).

However, for stiff structural systems, the fundamental period is usually on the ascending branch of the response spectrum and such an increase of period can imply a strong increase of structural demand (as shown in Figure 10).

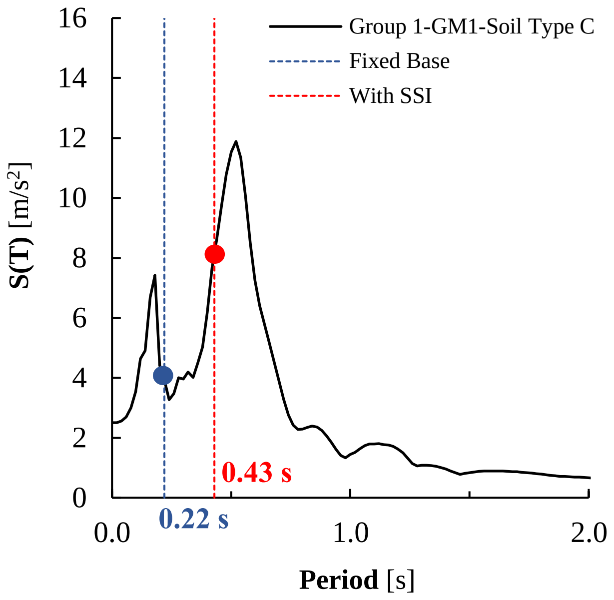

Figure 10 – Building with RC wall: spectral acceleration for fixed base model and complete FEM model

Another important issue regards the repartition of the seismic shear between the wall and the frame. As shown in Figure 11, a fixed base model leads to an overestimation, with respect to an SSI model, of the seismic shear rate affecting the wall, and this effect seems to be more relevant for seismic events of low to medium intensity.

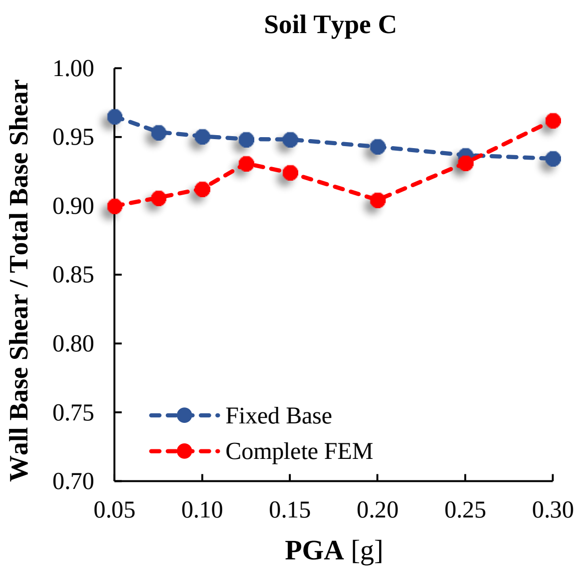

Figure 11 – Repartition of the total base shear between wall and frame

Concerning the building retrofitted with CFRP (orange curves), the analyses show that, assuming that the structure is fixed at the base, the supposed intervention with CFRP wrap does not allow to obtain a significant improvement of the seismic performance of the frame. However, taking into account SSI, the CFRP allows to increase the maximum PGA that the frame is able to withstand, from 0.15g (“bare” frame) to 0.25g.

This result can be justified based on the results obtained for the “bare” frame. The reduction of the maximum *IDR* due to SSI, for the same PGA level, leads to a lower ductility demand and, thus, the structure is able to withstand a further increase of PGA before to collapse.

A potentially ineffective intervention in the case of fixed base structure, proves to be effective when the soil-structure interaction is taken into account.

So, an accurate modelling of SSI effects could lead to a significant save of CFRP reinforcement and, thus, of money.

5 CONCLUSIONS

In this paper, we showed the results of some non-linear dynamic analyses performed for a 4 floors RC moment resisting frame designed for vertical loads only.

The analyses were performed for the “bare” frame and assuming that the building is seismically retrofitted by means of two typical interventions: by means of a RC shear wall and by means of wrapping with CFRP of the plastic hinge zones, in order to obtain an increase of ductility.

The analyses showed that for the “bare” frame, the soil-structure interaction can imply, with respect to a fixed base model, a reduction of the structural demand, especially in terms of maximum inter-storey drift ratio.

The analyses performed for the frame retrofitted with CFRP showed that an accurate modelling of SSI could justify a reduction, with respect to a common fixed base assumption, of the amount of reinforcement necessary to obtain a given increase of the structural performance.

Finally, the analyses performed for the building retrofitted with a RC wall showed that particular caution should be paid in the case of strengthening interventions that can cause a strong increase of the global stiffness of the structure. In this case, indeed, SSI can imply a strong increase of the seismic demand, both in terms of maximum base shear and maximum inter-storey drift. Moreover, because of SSI, the wall tends to absorb a lower amount of the total base shear, with variations, with respect to a fixed base model, in any case lower of 5%.

The results of the analyses showed that a common fixed base model can be, in this case, strongly un-conservative and un-safe for the design of this kind of retrofitting intervention.

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