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Soil-structure interaction effects on the seismic performances of reinforced concrete moment resisting frames

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Abstract

The paper investigates the influence of Soil-Structure-Interaction (SSI) effects on the seismic performances of 2D reinforced concrete (RC) moment resisting frames (MRFs), which were investigated by means of non-linear dynamic analyses. The goal was pursued by means of a parametric study in which (1) the soil properties, (2) the modelling technique of the SSI effects, (3) the seismic design level of the structures were varied. The soil classes suggested by Eurocode 8 were taken as reference to define the mechanical properties of soil. As concerns the SSI modelling, both a sub-structures approach and a direct approach were considered. Finally, structures of 4 and 8 floors designed for vertical loads only or according to the Italian regulations for constructions (NTC-08) were considered. RC-MRFs founded on soft soils were considered, because SSI effects on the seismic response are expected higher. The study shows that SSI affects the seismic demand in terms of maximum base shear and maximum inter-story drift ratio with different significance depending on the modelling approach.

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Keywords: Soil Structure Interaction; Dynamic Analyses; RC Moment resisting frames

1. Introduction

In this paper, the results of a parametric study performed with the aim to investigate the effects of the Soil-Structure Interaction (SSI) on the seismic performances of reinforced concrete (RC) moment resisting frames (MRFs) are shown. Nonlinear dynamic analyses were performed varying (i) the soil properties, (ii) the modelling technique of SSI effects, (iii) the seismic

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design level of the structures. As concerns the soil, the different classes suggested by Eurocode 8 [1] were referenced, while as concerns the modelling technique of SSI effects, both a "direct" approach and a "sub-structures" were considered. Finally, structures of 4 and 8 floors designed for vertical loads only or according to the current Italian regulations for constructions [2] were considered. Before showing the results of the analyses, the reference structures are introduced, as well as the selected records and the mechanical properties of the soil deposits assumed for the numerical analyses. Moreover, some explanations about the numerical models implemented in OpenSees are provided.

2. Analyses

Four different 2D RC MRFs were selected as reference structures. They can be considered as inner frames of "in-plan" regular 3D buildings. The structures were designed according to different seismic code levels, in order to capture different periods of construction. Two of them were designed for gravity loads only (i.e. with no seismic provisions) according to [3], while two were designed with high level of seismic details according to the current Italian seismic code [2]. Table 1 summarizes the main characteristics of the reference structures, namely the total mass, the concrete and steel strength adopted in the numerical models and the fundamental elastic period. Further details about the structures analyzed in the study can be found in [4]. As concerns the soil properties, two types of clays (medium and soft), sortable respectively as soil type C and D according to [1] (see Table 2), were considered in order to obtain significant SSI effects [5].

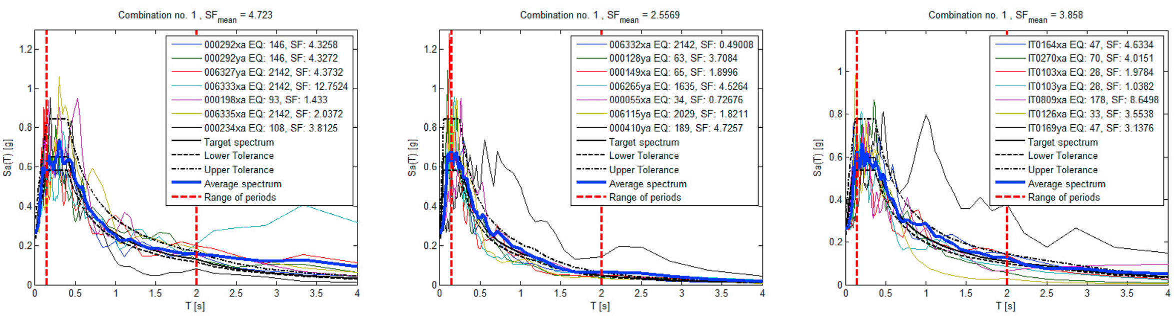
Table 1. Reference structures: main properties

Building	Total Mass (t)	f_c (MPa)	f_y (MPa)	$T_{1,fix}$ (s)
No Code - 4 Floors	290.6	17	380	0.97
No Code - 8 Floors	647.2	17	380	1.14
With Code - 4 Floors	292.9	25	450	0.65
With Code - 8 Floors	692.8	25	450	0.92

Table 2. Soil properties

	Soil Type C	Soil Type D
Height of the deposit	30 m	30 m
Type of Soil	Clay	Clay
Plasticity Index	15%	100%
Shear Wave Velocity (V_{s0})	250 m/s	160 m/s
Density (ρ)	2.0 t/m^3	1.6 t/m^3
Cohesion (c)	65 kPa	49 kPa

A set of 21 records was used for the dynamic analyses. In particular, three different sets of seven accelerograms were chosen by means of the software Rexel [6], respectively compatible on average with the Eurocode 8 type 1 spectrum (Group1), Eurocode 8 type 2 spectrum (Group 2) and Italian D.M. 14/01/2008 (Group 3). All the records refer to 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. The compatibility with the response spectra was checked in the period range from $0.15 \text{ s} < T < 2.0$ s (see Fig.1).



Group 1 – EC8 type 1 spectrum b) Group 2 – EC8 Type 2 spectrum c) NTC 08 spectrum

Fig. 1. Selected records: compatibility

3. Numerical modelling in OpenSees

The numerical models were implemented with the OpenSees software [7]. As concerns the structural modelling, lumped mass models were adopted, in which the structural mass is concentrated in the nodes of the computational model. Beams and columns were modelled as *'beamWithHinges'* elements, which consider plasticity to be concentrated over specified hinge lengths at the element ends. In order to better taking into account the influence of axial load on the flexural behavior of the columns, the plastic hinges were defined as fiber hinges. Concerning the constitutive laws, the uniaxial *'Concrete01'* material [8] was used for concrete while the uniaxial *'Steel01'* material was used for the steel. The length of the plastic hinges was defined based on [9]. Brittle shear failures were taken into account by means of springs working in series with the *'beamWithHinges'* elements. The work of *Elwood* [10] was referenced but re-adapted. The shear springs were defined in OpenSees by means *'zero-length'* elements with an initial stiffness defined as $K_v = GA_g / H$ where G is the shear modulus of concrete, A_g is the cross section of the element and H its length. The force-displacement rule of the spring is defined in terms of force-drift between the end nodes of the beam-column element and is described by means of a *'Limit State Material'*, that allows to define a threshold beyond which the spring has a softening behavior and a residual capacity. In this study, in order to simulate a complete and sudden loss of shear capacity, no residual capacity and an infinite degrading slope were assigned to shear springs. Moreover, the threshold for the model suggested in [10] is defined based on a specific value of drift, but in this study it was defined by the shear resistance of the element according to the formulation suggested by *Sezen & Moehle* [11]. To account for the viscous damping mobilized during the dynamic response of the structures, tangent stiffness proportional damping [12] was assigned with a damping ratio of 5%.

In the logic of a sub-structures approach, the Beam on Nonlinear Winkler Foundation (BNWF) model, implemented in OpenSees by *Raychowdhury and Hutchinson* [13], can be used to model the inertial interaction. The model consists of an *'elasticBeamColumn'* element that captures the structural footing behavior and independent *'zero-length'* elements that model the soil-footing behavior. One-dimensional uniaxial springs are used to simulate the vertical load displacement behavior, horizontal passive load-displacement behavior against the side of a footing, and horizontal shear-sliding behavior at the base of a footing. Moment-rotation behavior is captured by distributing vertical springs along the base of the footing. A limitation of the approach relates to its one-dimensional nature: a spring responds only to loads acting parallel to its axis, so loads acting in a perpendicular direction have no effect on the response of the spring. In this study, the spring stiffnesses and the corresponding radiation damping coefficients were calibrated based on the formulations suggested by *Pais and Kausel* [14] considering the initial shear modulus of the soil. The horizontal passive load-displacement behavior was neglected, since it has a negligible effect on the structural response in case of shallow foundations with a small depth of embedment.

In order to compare the results obtainable by means of a modelling of the SSI with a sub-structures approach and with a direct approach, a complete FEM model was implemented. A homogeneous soil deposit with a bedrock lying at 30 m beneath the ground surface was modelled in two-dimensions using the plane strain formulation of the *'quad'* element. To account for the finite rigidity of the bedrock ($V_{s,bedrock} = 1000$ m/s), a *Lysmer-Kuhlemeyer* [15] dashpot was incorporated at the base of the soil profile. The dashpot was defined based on the viscous *'Uniaxial'* material model and the *'zeroLength'* element formulation, to connect two previously defined nodes. This material model requires a single input, the dashpot coefficient, that is defined as the product between the bedrock mass density, its shear wave velocity and the base area of the soil profile [16]. The out-of-plane thickness of the quad elements was set equal to $3B$, with B equal to the width of the foundation footings. This assumption was made in order to reproduce with the complete FEM model the initial elastic stiffness of the BNWF model. As the impedance functions proposed by *Pais and Kausel* were formulated for 3D foundations, it is likely that they result in higher foundation stiffness than the 2D solution would. The nodes placed at the same depth on the two opposite lateral boundaries were tied together in order to achieve a simple shear deformation pattern of the soil profile [17], while the nodes at the base of the soil deposit were restrained against vertical translation and constrained to have the same displacement in the horizontal direction. No detachment or sliding were allowed between the foundation and the soil and the foundations were modelled, for sake of simplicity, as elastic elements of infinite rigidity. The soil profile is excited at the base by a horizontal force time history, which is proportional (through the dashpot coefficient) to the known velocity time history of the ground motion [16, 18]. The dimensions of the soil grid were chosen to ensure free field and "quasi transparent" condition at the boundaries, while the those of the soil elements were set equal to 1.0m x 1.0m based upon the concept of resolving the propagation of the shear waves at or below a particular frequency (a maximum frequency of interest of 10 Hz was considered in the study) allowing an adequate number of elements to fit within the wavelength of the chosen shear wave [19]. The soil nonlinearity was modelled by means of an *'ElasticIsotropic'* material with a modulus properly reduced to take into account the shear strain amplitude (as suggested in [20]) and viscous damping employed in the frequency-dependent Rayleigh form [21, 22]. 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. The Rayleigh damping was assigned based on two target frequencies that were chose as the predominant frequencies of the soil deposit, f_1 and $f_2 = 5f_1$ [23]. This choice was made in order to simulate an almost constant damping at the frequency range of interest. The damping ratio ξ was determined, given an appropriate reduction curve (the curves provided in [24] for a confining pressure $p'_0 = 1$atm were referenced in the study) of the shear modulus for the soil and an expected value of the PGA at the site, based on the shear strain level corresponding to the shear modulus reduction factor provided by [20] (see [4] for more details).In Fig.2b, the complete FEM model implemented in OpenSees is illustrated.

4. Results

Prior to the parametric analyses, preliminary analyses showed that as the FE model of the soil is sufficiently large, no spurious reflections due to the structural oscillation were observed close to the FE lateral borders [4]. In the dynamic analyses performed for the complete FEM model, site effects and kinematic interaction are implicitly taken into account by the model. In order to obtain a significant comparison, for fixed base and BNWF models site effects were taken into account applying at the base of the models the free field motion (FFM) obtained by means of a preliminary 1-D wave propagation analysis of a soil column (see Fig.2a).

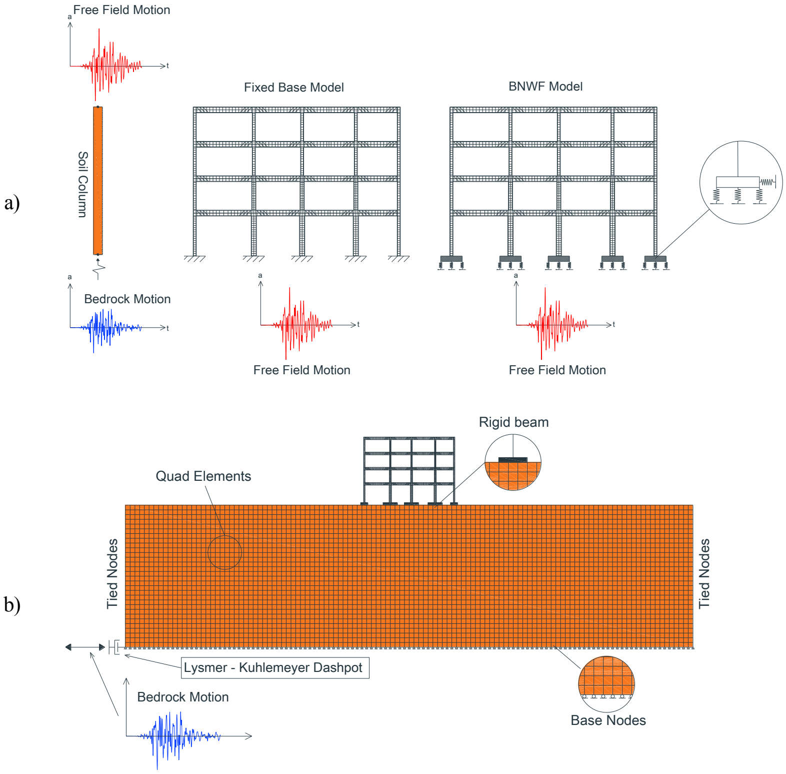


Fig. 2. Reference schemes for dynamic analyses: a) Fixed Base and BNWF models, b) Complete FEM model

In the BNWF model, the kinematic interaction was neglected; this assumption is commonly accepted in the case of shallow foundations resting on the surface of the half-space. In all the analyses, the records were scaled 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.

The maximum base shear, V_{max}, and the maximum inter-story drift ratio, IDR_{max}, were chosen as engineering demand parameters. In Fig.3, the results of the analyses obtained for a soil type D are shown. The results are reported in terms of ratio between the seismic demand obtained for the compliant base models and for the fixed base model. Given the PGA level, the points on the graphs were determined as the average of the values obtained for all the 21 signals. However, only the results obtained for PGA levels that did not cause the structural collapse were considered so, in some cases, the points on the graphs represent an average obtained on less than 21 values. As can be noted, the modelling technique of SSI can significantly affect the estimation of the seismic demand. The adoption of a refined Complete FEM model can lead to reductions in the estimation of the seismic demand, with respect to a fixed base model, up to 50% in terms of IDR_{max} and up to 25% in terms of V_{max}. A simplified modelling of SSI effects by means of a BNWF model leads to lower reductions in the estimated seismic demand (maximum reduction of 20% in terms of both V_{max} and IDR_{max}).

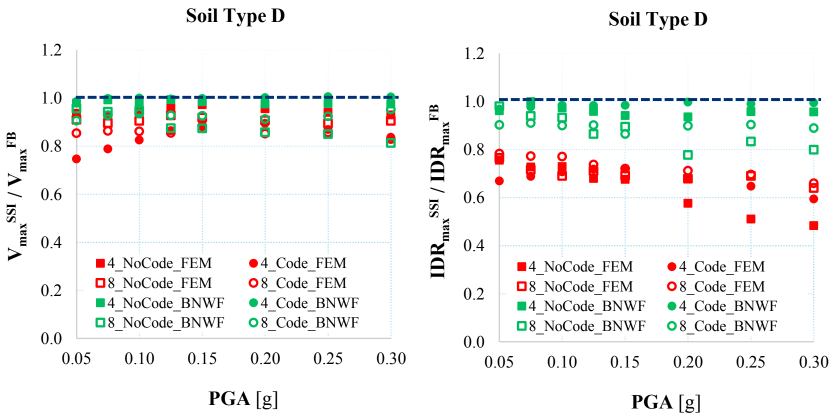


Fig. 3. Results of dynamic analyses for soil type D

As concerns the structures, SSI affects more the estimation of the seismic demand in terms of IDR_{max} for "No Code" buildings, while the reduction is higher in terms of V_{max} for "Code" buildings. In addition, the height of the structure plays an important role when SSI is modelled by means of a BNWF model: only in case of 8 floors buildings, whereas rocking effects are more significant, the model can lead to significant differences in the estimation of the seismic demand with respect to a fixed base model. In case of shorter buildings, the incapability of the BNWF model to accurately predict the sliding response of the foundation behavior [25] make it unable to adequately capture SSI effects. As concerns the soil properties, referring to a Complete FEM model, the SSI effects seem to be not strongly affected by the soil properties and only light differences in terms of seismic demand can be appreciated passing from a soil type C to a soil type D. Referring to a BNWF model, the SSI effects seem to be more affected by the soil properties, with appreciable differences in the estimated seismic demand only in case of very soft soils (soil Type D).

5. Conclusions

In this study, SSI effects on the seismic demand of reinforced concrete moment resisting frames were investigated by means of a parametric study in which: (i) the soil properties, (ii) the modelling technique of SSI effects and (iii) the seismic design level of the structures were varied. In particular, nonlinear dynamic analyses were performed.

The study shows that, based on the modelling approach adopted, SSI can affect more or less the estimation of the seismic demand with respect to a fixed-base model. The adoption of a refined complete FEM model can lead to reductions in the estimation of the seismic demand, with respect to a fixed-base model, up to 50% in terms of maximum inter-story drift ratio and up to 20% in terms maximum base shear. A simplified modelling of SSI effects by means of a Beam on Nonlinear Winkler Foundation (BNWF) model can affect the evaluation of the seismic demand only in case of 8 floors buildings founded on very soft soils, whereas rocking response tends to prevail over sliding response of the foundation. Anyway, the reductions with respect to a fixed-base model (up to 20% in terms of both maximum base shear and maximum inter-story drift ratio) are lower than those predicted by a complete FEM model. The difference between the two modelling approaches is related to the different characterization of the overall damping, as shown in Fig.4, in which the acceleration of a point on the top of the structure, and the corresponding Fourier spectrum, is plotted for a specific record. The BNWF model, because of the lack of coupling between vertical and lateral modes of foundation response and because of its incapability to take into account the frequency variability of foundation impedances, seems to under-estimate the energy dissipation due to SSI.

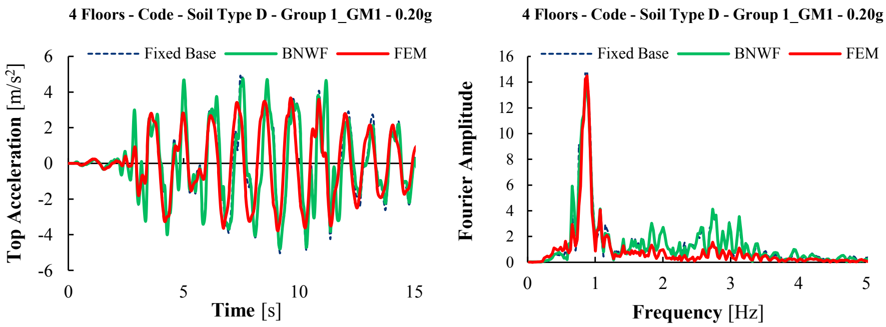


Fig. 4. Top acceleration and corresponding Fourier Spectrum

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