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Preliminary considerations on the rocking behavior of foundations in precast industrial buildings

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Abstract

Recent research has shown the importance of considering the soil-structure interaction in evaluating the seismic response of structures. In the specific case of precast buildings, with not-interconnected footing foundations, it is important to evaluate the rocking behavior of the foundations in the case of an earthquake. In fact, after past Italian earthquakes (e.g., Emilia), this type of buildings has presented various damage compatible with the rocking of the foundations. The purpose of this paper is to assess the seismic performance of precast industrial structures considering the soil-structure interaction. Parametric studies of single column footing models were made to establish the influence of the rocking at the soil-foundation interface. A case study building was selected and non-linear static analyses of a 2D model including soil influence was carried out. The main results show that the rocking mechanism at the foundations of precast industrial buildings may influence the seismic performance of the super structure.

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Keywords: precast buildings; soil-structure interaction; foundation rocking

1. Introduction

The soil-structure interaction (SSI) is an important aspect of seismic assessment, although it is generally neglected particularly when it is believed that such interaction has a beneficial effect on the structural response. In fact, considering SSI, there is an increase in the fundamental period of vibration of the system (Veletsos and Meek 1974)

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and an increase in the overall damping of the system (Veletsos and Nair 1975) compared to the configuration with fixed-end conditions. With reference to the classic form of the acceleration response spectrum, these two effects result in a reduction in design forces. However, in the context of moderately flexible structures, the increase in the fundamental period of vibration could have adverse effects on the seismic demand (Mylonakis and Gazetas, 2000).

Considering specifically industrial precast concrete buildings, their typical configuration is represented by cantilever columns with not-interconnected footing at the base and connected at the top with simple supported beams; such conditions lead to consider this structural system as moderately flexible. The 2012 Emilia earthquake, in Italy, highlighted the main deficiencies of this type of structure which include the low performance of the connections between structural elements under seismic actions (Belleri et al. 2015a, Belleri et al. 2015b, Belleri 2017; Ercolino et al. 2016; Bournas et al. 2014; Magliulo et al. 2014; Minghini et al. 2016; Minghini and Tullini 2021; Palanci et al. 2017). Specifically, some columns showed some damage at the plastic hinge at the base and a possible lift of the foundation. Indeed, the foundation's rocking mechanism, in the context of the SSI, has been previously addressed in various structural configurations. Rosebrook and Kutter (2001a, 2001b), Deng and Kutter (2012) and Hakhamaneshi et al. (2012a; 2012b) established that the amount of settlement or uplift during rocking can be correlated to the contact area and the amplitude of footing rotation. Hakhamaneshi et al. (2013) used experimental data of monotonic and slow-cyclic loading tests of rocking foundations to develop a database of rocking foundation performance. In this paper the rocking of the footing in precast industrial buildings is investigated and compared to the state of the plastic hinge at the base of the column. The paper is organized as follows: section 2 presents the description of the 2D finite element model built in OpenSees software (McKenna et al. 2000) and presents the results of non-linear static analyses for a simple single-footing column system; section 3 focuses on the Finite Element-Boundary Integral Equation (FE-BIE) modeling technique (Tezzon et al. 2015) and presents its application to a case study structure, with a comparison with some of the results obtained in section 2.

2. Soil structure interaction and rocking foundation

This section introduces the finite element (FE) model built in OpenSees (McKenna et al. 2000) and its application to a single column with a rectangular footing. The FE model (**Error! Reference source not found.**a) consists of a half-space soil connected to the superstructure through zero-length elements with compression-only behavior.

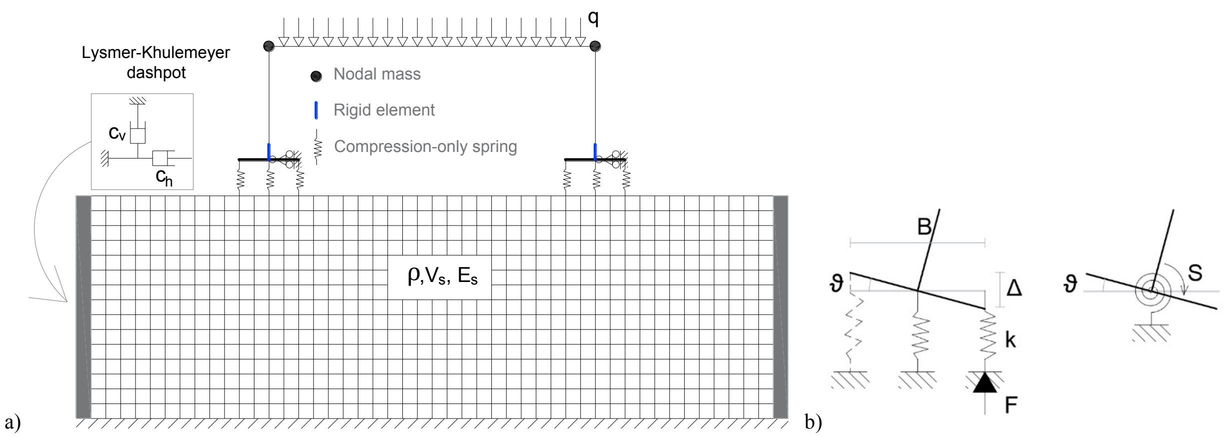


Fig. 1. a) FE model scheme. b) Different FE schemes for the rocking interface

The half-space soil was modeled through plane-stress quad elements of 1m width with elastic and isotropic behavior. In addition, the model allows assigning a non-linear hysteresis to the soil elements. The nodes at the base of the half-space have been constrained in both directions, while the nodes on the sides were bound to move together in the horizontal direction in accordance with the formulation proposed in Ezzatyazdi and Jahankhah (2014). In addition, vertical and horizontal Lysmer-khulemeyer dashpots (Lysmer and Kuhlemeyer 1969) were adopted to avoid waves reflections. The dashpots were modeled with zero-length elements and the viscous uniaxial material in OpenSees: the dashpot coefficient (c) was defined following the method in Joyner and Chen (1975), as the product of the mass density (ρ) and the shear wave velocity (v_s). The structural elements (footing, columns, and connecting beam) were modeled using elastic beam elements. The non-linear behavior of the column plastic hinge is included through a zero-length element with a multi-linear uniaxial material in moment-rotation terms.

The soil definition considered the assumption of a medium clay with the parameters reported in Table 1. The inelastic behavior was accounted for with the multi-yield-surface j2 plasticity model. This model was implemented in OpenSees with the following parameters: bulk modulus equal to 13889kN/m^2, shear modulus equal to 20833kN/m^2, initial yield stress equal to 65kN/m^2, final saturation yield stress equal to 600kN/m^2, exponential hardening parameter and linear hardening parameter both taken equal to 1.

Table 1. Soil parameters.

Parameter	Symbol	Value
Elastic modulus	E_s	25000 kN/m^2
Poisson ratio	v	0.2
Mass density	ρ	1800 kg/m^3
Shear wave velocity	v_s	200 m/s

The FE model also required the calibration of the initial stiffness (k) of the compression-only springs and the bending moment capacity of the footing, which was estimated in according with Gajan and Kutter (2008) in the hypothesis that, during rocking, the contact area is equal to half of the footing area. The spring stiffness (k) was estimated as:

$$k = \frac{S}{B^2} \tag{1}$$

This equation was derived by the equivalence between the two rocking interface modeling techniques (distributed springs, rotational spring) represented in Fig. 1b in which S is the initial rotational stiffness of the footing determined according with Gazetas (1991).

$$S = \frac{0.45GB^3}{1-\nu} \tag{2}$$

where B is the width of the footing (in the case of square footing) and G is the shear modulus of the supporting soil.

Considering the superstructure, a single column taken from a one-story precast portal frame was considered. The column had a height of 6.2m with a cross section 0.35mx0.35m and a rectangular footing (1.5mx1.5m) with thickness 0.5m. The total vertical load at the top of the column was 241.5kN. Given the footing geometry, the chosen soil extension was 48m for the width and 30m for the depth. Non-linear static analyses were carried out considering two different constraint conditions (i.e., fixed base and half-space soil) and two levels of inelasticity for the column plastic hinge (i.e., PH-1 and PH-2) defined according to the moment-rotation curve reported in Table 2.

Table 2. Points defining the piecewise moment-rotation curve of the column plastic hinge.

Plastic hinge	ϑ_i (rad), M_i (kN);
PH-1	0, 0; 0.00046, 24.48; 0.00333, 83; 0.0076, 86.63; 0.0182, 88.08
PH-2	0, 0; 0.0000035, 66.46; 0.003578, 210.48; 0.00586, 216.86; 0.0163, 223.6

The results of the nonlinear static analyses including second order (P-Delta) effects are reported in Fig. 2 and Fig. 3 for PH-1 and PH-2, respectively. Considering the PH-1 (Fig. 2) we note that the moment-rotation curve of the footing remains in the elastic range with no lifting of the footing. The three models achieve a similar ultimate shear capacity. While considering the PH-2 (Fig. 3), we note that the footing moment-rotation curve goes in the nonlinear range indicating a lift of the footing. As for the shear capacity, the models with elastic half-space and inelastic half-space achieve, respectively, 6% and 41% lower capacity than the fixed-base model. The greatest difference observed in the case of inelastic half-space can be attributed to the nonlinearity of the soil which is associated with a lower rotation capacity. A general comment can be made regarding the global stiffness of the system; in order of decreasing stiffness we have, as expected, the fixed base model, the model with the elastic half-space soil and finally the model with the inelastic half-space soil.

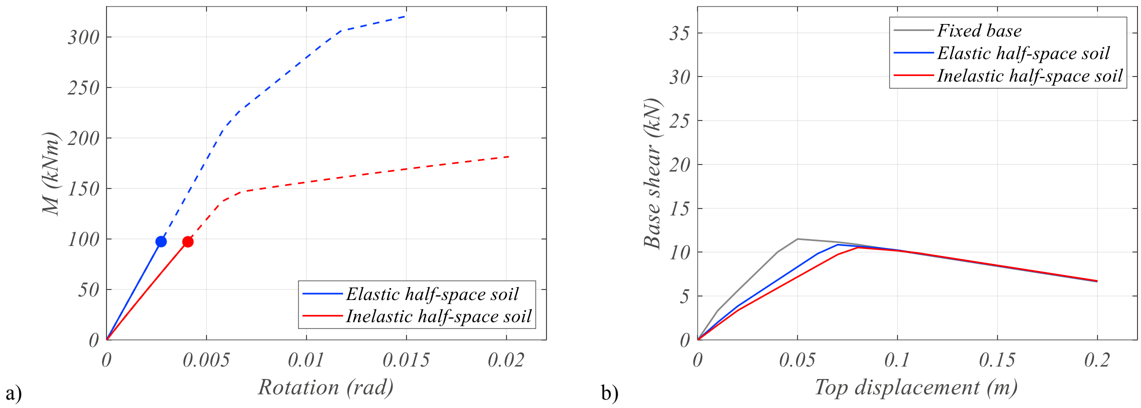


Fig. 2. Results for the case with PH-1: a) moment-rotation curve at the footing; b) capacity curves for the superstructure.
Note: solid circles represent the footing demand at the column capacity; the dashed curves refer to the case of an elastic superstructure.

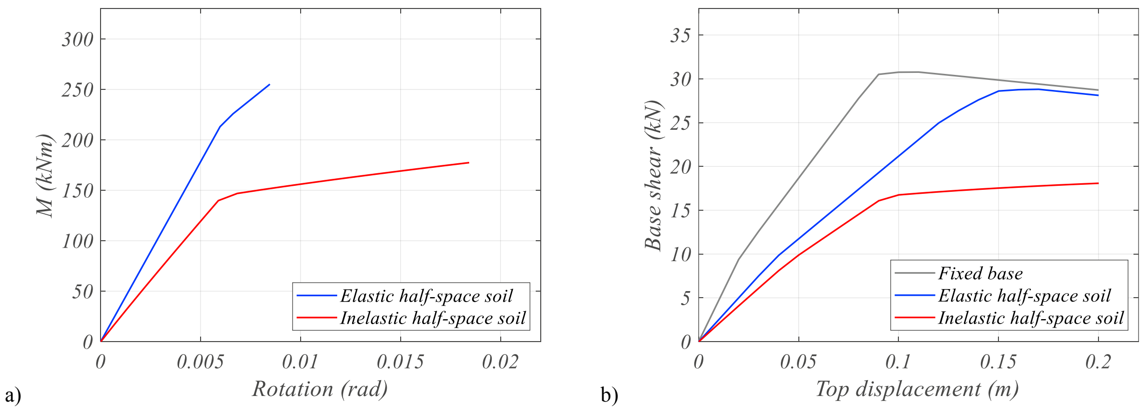


Fig. 3. Results for the case with PH-2: a) moment-rotation curve at the footing; b) capacity curves for the superstructure

3. FE-BIE model

The Boundary Element Method (BEM) has proved to be advantageous in reproducing the response of the elastic half-space, because only the boundary of the elastic substrate must be discretised (Ribeiro and Paiva 2015). However, the BEM coefficient matrix is non symmetric, so requiring a non-negligible computational effort to obtain the solution. Moreover, soil tractions arising at the substrate boundary are typically used as nodal reactions in the FE model of the foundation beam, leading to the lack of rotation continuity between beam and substrate.

An alternative computational method to reproduce the SSI is based on a mixed formulation coupling usual displacement based FEs, used for the structure, with a Boundary Integral Equation (FE-BIE). The BIE includes a suitable Green's function of the substrate and is evaluated analytically. In particular, regarding a two-dimensional half-space in plane state, Flamant and Cerruti's solutions are to be used (Kachanov et al. 2003). This method involves symmetric soil matrices and preserves the rotation continuity at the soil-foundation interface, resulting in strong computational advantages compared with BEM and, even more, with standard FEM. The interested reader is referred to Tezzon et al. (2015) and references cited herein for a comprehensive description of this method.

The application of the FE-BIE formulation to a case study is discussed in the following subsection.

3.1. Portal frame case study

A precast Reinforced Concrete (RC) portal frame with a beam pinned at the column top ends is considered (Fig. 4a). The columns have a square cross-section with the side of 500 mm and are longitudinally reinforced by 10 deformed 20 mm-diameter bars. The roof is assumed to be comprised of double-tee prestressed members spanning orthogonally to the frame plane, leading to a dead load of 31.5kN/m uniformly distributed along the beam. In the presence of gravitational loads only, the resulting compression at each of the column bases is of 290kN, whereas the total load at the substrate boundary is of 410kN for each of the footings. The soil is considered as a linear elastic half-plane whose properties are reported in Table 1.

A numerical model of the frame was developed using classical Hermitian beam elements based on Euler-Bernoulli's theory for columns and beam (B1 to B3 in Fig. 4b). At the column bases, rigid links were used for the connection with footing centroids. The footings (R1 and R2) were modeled as rigid flat punches in frictionless contact with an elastic half-plane, according with the formulation presented by Tezzon et al. (2015) in section 3.6 of their paper.

For columns, the material inelastic response was reproduced through a concentrated plasticity approach considering the formation of plastic hinges as analogous to the development of semi-rigid relative rotations at end joints. In particular, the semi-rigid joint model proposed by Shakourzadeh et al. (1999) and applied by Minghini et al. (2009, 2010) to fiber-reinforced plastic frames was adopted. Potential plastic hinges were placed, in each of the columns, at the top end section of the pocket (see Fig. 4b), where a suitable tri-linear moment-rotation behavior was defined. In addition, geometric nonlinearities were introduced in the form of a geometric stiffness matrix depending on column axial load. Each footing was discretized with $n_{\mathrm{el}} = 16$ FEs of equal size, and piecewise constant vertical tractions were adopted to reproduce the SSI.

A nonlinear static analysis of the portal frame was carried out controlling the column top horizontal displacement via the incremental algorithm proposed by Batoz and Dhatt (1979). Moreover, possible foundation uplifting was accounted for assuming a compression-only substrate boundary.

For comparison, two more alternative models were developed in STRAND7® (2004). For one of these models, fixed column bases were assumed defining rigid constraints at the top end section of each pocket foundation. For the other model, the SSI was accounted for by means of a compression-only, Winkler-type elastic support defined at the substrate boundary.

The response obtained from the FE-BIE model for one of the footings is reported in Fig. 5 corresponding to three different stages of the nonlinear analysis. It is worth noting that the slight footing rotation under purely vertical load (Fig. 5d) is due to the interaction between the two footings, which is an effect disregarded by Winkler's model. The vertical soil tractions are reported in Fig. 5g,h,i, where they are compared with those derived from a refined mesh with $n_{\mathrm{el}} = 32$ FEs. It can be observed that, among the three stages illustrated, uplifting is present for the case $\delta_2 = 200$ mm only (Fig. 5i). Moreover, the maximum traction increases as the mesh size decreases, but it tends to be restricted to the foundation tip. The refined mesh yields tractions smaller than 200 kPa everywhere, except for a very narrow portion of about 0.1 m.

The capacity curves obtained from fixed base and Winkler's soil models are reported in Fig. 6a, where the significant stiffness reduction due to SSI is evident. Conversely, the maximum shear capacity of the latter is only about 6% smaller than that of the former. The difference in ductility is a direct consequence of rocking. Both curves are stopped at a displacement providing the achievement of the ultimate plastic rotation at the column bases. However, for the fixed base model the plastic hinges were activated for a smaller displacement and attained earlier their capacity.

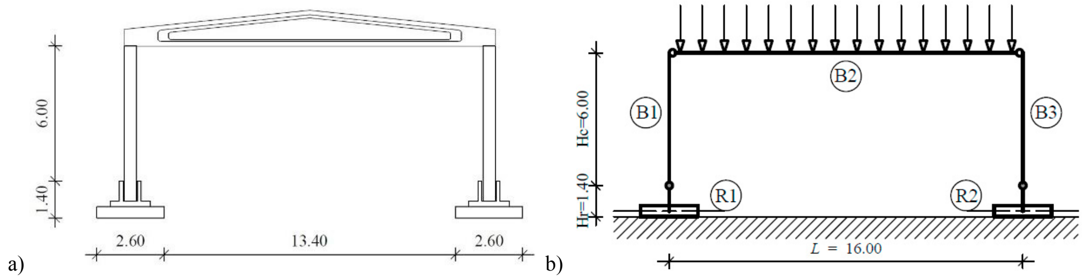


Fig. 4. Case study: (a) precast portal frame and (b) corresponding computational model.

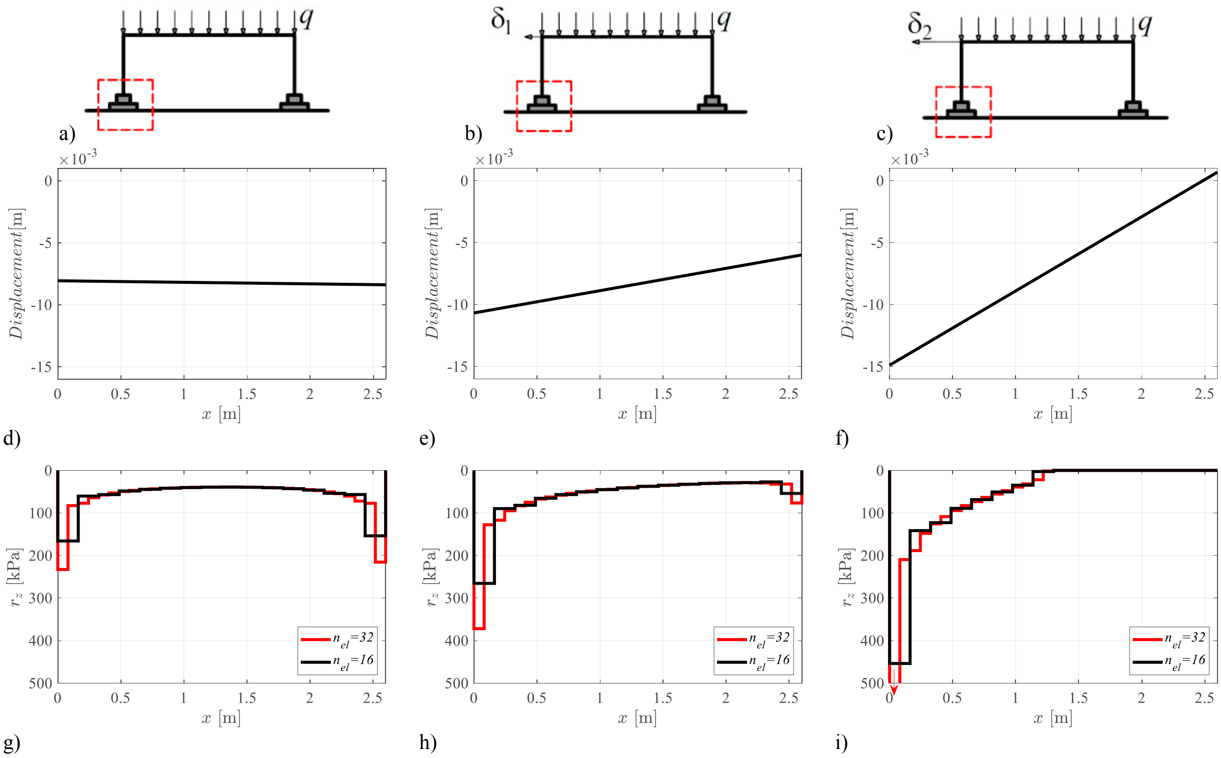
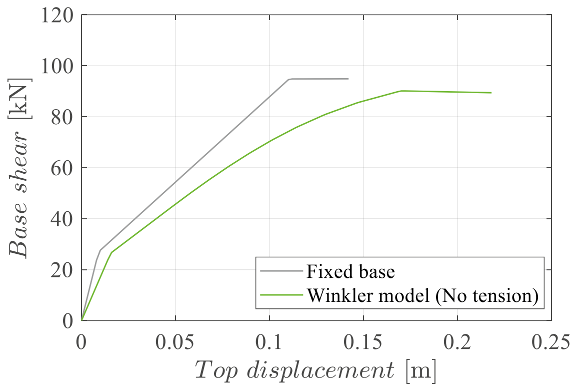


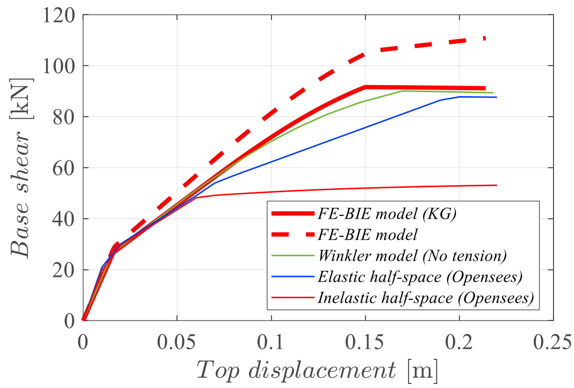
Fig. 5. Precast frame at three different stages of the nonlinear analysis with FE-BIE approach: (a, d, g) initial stage; horizontal displacement (b, e, h) δ_1 = 50 mm and (c, f, i) δ_2 = 200 mm; (d, e, f) deformed shape and (g, h, i) vertical soil tractions for the left footing.

The Winkler's solution is re-proposed in Fig. 6b, where it is compared with those obtained from the FE-BIE model (thick continuous line in red). The two curves are almost coincident except for the displacement interval between 0.10 and 0.17 m. This feature follows from the different distributions of soil tractions provided by the two models. In fact, the footing started to uplift for an axial load eccentricity of 0.167B for Winkler's model and of about 0.250B for the FE-BIE model, with B being the side length of the footing. The solution provided by the FE-BIE model neglecting geometric nonlinearities (thick dashed line in red in Fig. 6b) clearly indicates that second order effects must be accounted for in the analysis.

The thin blue curve in Fig. 6b refers to the OpenSees solution (see Sect. 2) for elastic half-plane and shows a base shear capacity substantially coincident with that predicted by FE-BIE and Winkler's soil models. However, a smaller capacity is provided by OpenSees within the displacement interval between 0.07 and 0.19 m, indicating premature foundation uplifting and the need for a tuning refinement of springs shown in Fig. 1. Finally, the red curve in Fig. 6b refers to the OpenSees solution for inelastic half-plane. In this case the base shear capacity resulted about 40% smaller than predicted by the elastic soil models.



a)



b)

Fig. 6. Capacity curves obtained from (a) fixed base and Winkler's soil models; (b) FE-BIE model with and without geometric nonlinearities, and OpenSees model with nonlinear soil.

4. Conclusion

This paper investigated the seismic performance of precast industrial buildings considering the SSI. An assessment based on finite element (FE) models and FE-boundary integral equations (FE-BIE) was carried out. Nonlinear static analyses were conducted on single column-footing assemblies by varying the capacity of the plastic hinge at the column base. The results showed that in some cases the uplifting of the footing occurs, i.e. when the capacity of the plastic hinge is greater than the rocking activation moment. A single-story precast portal frame was then analyzed: a comparative study was carried out with respect to the FE-BIE technique implemented in Matlab environment. From the performed analyses, the following preliminary conclusions can be drawn:

- the foundation rocking produces a significant stiffness reduction compared with the fixed base case; this could correspond to an important limitation to the seismic demand provided by usual FE models which neglect the SSI;
- the FE-BIE model provides an accurate approximation of the soil tractions, so allowing for the best simulation of uplifting conditions and global frame ductility;
- neglecting second order effects may result in wrong predictions of base shear capacity;
- the strong capacity reduction obtained from inelastic soil models suggests that retrofitting interventions on existing buildings should provide an adequate control of rocking motion to avoid irreversible rotations.

The previous considerations refer to a specific clay-type soil; further developments involve the extension to other types of soils, the improvement of the FE modeling of the foundation-soil interface in relation to both the number of springs and the related hysteresis, and the assessment through non-linear dynamic analyses.

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