EFFECTS OF SOIL-PILE-STRUCTURE INTERACTION ON THE NON-LINEAR SEISMIC RESPONSE OF COUPLED WALL-FRAME SYSTEMS

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ABSTRACT :

The aim of this paper is to study the effects of Soil-Structure Interaction (SSI) on the seismic response of coupled wall-frame structures on pile foundations considering the non-linear behaviour of the superstructure. According to the substructure method, the kinematic interaction analysis is performed by means of a numerical procedure obtaining the dynamic impedance functions and the foundation input motion both necessary to perform inertial interaction analysis. A suitable Lumped Parameter Model (LPM) is defined to reproduce in the time domain the frequency-dependent impedances. The vertical, horizontal and rotational components of the soil-foundation impedance functions, as well as the roto-translational coupling one, are taken into account. The non-linear inertial interaction analysis is performed by using a finite element model of the superstructure. With reference to a 6-storey 4-bay wall-frame structure, designed according to the Displacement-Based Design and founded on a soft soil deposit, comparisons are made between the compliance base and the fixed base models. The effects of the soil-structure interaction, in terms of structural displacements, base shear, ductility demand and evolution of dissipative mechanisms, are evaluated.

KEYWORDS: coupled wall-frame structures, dissipative mechanisms, inertial interaction, kinematic interaction, lumped parameter models, soil-pile-structure interaction

1. INTRODUCTION

The coupled wall-frame systems are efficient seismic resistant structures able to combine the advantages of the frames and the walls by assuring the development of an effective dissipative mechanism under severe earthquakes and the limitation of displacements under weak motions. In such systems the walls are interested by significant bending moments requiring suitable degree of the base restraint. Pile foundations are generally considered to be rigid enough to prevent the wall rocking and a fixed base model is generally supposed to be conservative. Actually a more realistic evaluation of the seismic behaviour of coupled wall-frame systems should account not only for the non-linear behaviour of the superstructure but also for soil-structure interaction since the distribution of the ductility demand and the stress resultants may result modified with respect to a conventional fixed base approach.

The authors have already investigated the effects of soil-structure interaction on the seismic response of this type of structures with reference to the Damageability Limit State (DLS) by assuming a linear behaviour for both the soil-foundation and the superstructure (Carbonari et al. 2008a-b). It was found that a fixed base model is poor and does not catch the behaviour of such structures especially for soft soil deposits for which an increase of the interstorey drift and a reduction of the shear resisted by the wall are obtained with respect to the fixed base model.

The aim of this paper is to extend the investigation to the non-linear field for the superstructure and evaluate the effects of soil-structure interaction on the dissipative mechanisms of the structure subjected to strong earthquakes. Soil-structure interaction analysis is performed by means of the substructure method which separately considers the soil-foundation system (kinematic interaction analysis) and the superstructure on deformable restraints (inertial interaction analysis). In a first step, the kinematic interaction analysis, performed in the frequency domain according to the procedure developed by Dezi et al. (2007), allows obtaining the

soil-foundation impedance functions and the foundation input motion necessary to perform the second step analysis. Artificial accelerograms matching the elastic response spectra suggested by EN 1998-1 (2004) are used to represent the seismic incoming motion. Due to the non-linear behaviour of the superstructure, the inertial interaction analysis is performed in the time domain. To this purpose, the soil-foundation impedance functions evaluated in the frequency domain are reproduced in the time domain thanks to suitable Lumped Parameter Models developed to take into account the behaviour of the soil-foundation system under generic roto-translational motions. The non linear dynamic analysis of the compliance base structure is performed by means of a non-linear finite element model with concentrated plastic hinges.
The effects of the soil-structure interaction are evaluated with reference to a realistic case study, constituted by a 6-storey 4-bay wall-frame structure, designed according to the Displacement Based Design (DDBD) (Priestley et al. 2007), founded on piles and considering a soft soil deposit. The results obtained with the compliance base model are compared with those achieved with the fixed base model focusing on the effects of the soil-foundation deformability on the lateral displacements, base stress resultants, ductility demand and evolution of dissipative mechanisms of the superstructure.

2. ANALYSIS METHODOLOGY

Soil-structure interaction is studied according to the substructure method by performing separate analyses for the soil-foundation system (kinematic interaction) and for the superstructure on compliance base (inertial interaction). To take into account the superstructure non linear behaviour the inertial interaction analysis is carried out in the time domain. Suitable Lumped Parameter Models (Wolf, 1994) are introduced in order to transform the frequency-dependent impedance functions in the time domain. In the following sections a brief description of the adopted numerical procedures are reported.

2.1. Kinematic interaction analysis

The kinematic interaction analysis is usually performed in the frequency domain under the assumption of linear behaviour for soil and foundation piles. The foundation piles are modelled with beam elements embedded in a soil constituted by independent horizontal infinite layers capable of describing in-plane and out-plane wave propagations. The numerical procedure proposed by Dezi et al. (2007) is used to study the soil-foundation kinematic analysis considering the pile-to-pile interaction and the radiation damping. For pile groups, a rigid cap is considered and a master node is introduced at the centroid of the pile cap.
The transient seismic free field motion along the piles is obtained by means of a signal deconvolution, according to a one-dimensional ground response analysis, starting from artificial accelerograms defined at the outcropping soil.
The results of this first step of analysis consist of the frequency-dependent impedances for each foundation (the single pile for columns and the pile group for the wall) and the foundation input motion. It is worth noticing that due to the roto-translational coupling of the pile groups, the impedance function matrix contains coupled terms and the foundation input motion is characterized by translational and rotational components.
In the analysis of the fixed base structure, the artificial accelerograms are directly used as base input motions.

2.2. Lumped Parameter Model

In the frequency domain the impedances are complex functions characterized by a real and an imaginary part that are both frequency-dependent. When the non-linear behaviour of the superstructure has to be considered, a time domain analysis is required. In this case the frequency-dependent behaviour of the soil-foundation may be conveniently taken into account by means of Lumped Parameter Models (Wolf, 1994).
In this work a 3 *dof* LPM able to describe the coupled roto-translational behaviour of the soil foundation system is introduced (Figure 1). The three degrees of freedom are constituted by the horizontal and vertical translations x and z, respectively, as well as the rotation φ measured at the external node. The model is characterized by the following parameters: the translational (m_x, m_z) and rotational (I) masses, lumped at the centroid of the pile rigid caps, the elastic (k_x, k_z, k_r) and the viscous (c_x, c_z, c_r) constants defining relevant spring-dashpot elements and an additional eccentric mass m_t connected to a spring-dashpot element (k_t, c_t).

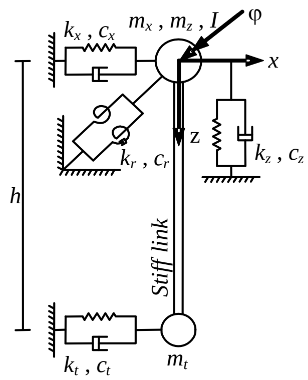


Figure 1 Lumped parameter model for the soil-foundation system

The second mass is introduced to catch the coupling between translation x and rotation φ. The impedances of such system are frequency quadratic functions for the real part and linear functions for the imaginary part and may be represented in the form of impedance matrix

$$Z(\omega)=\begin{bmatrix} k_x+k_t-\omega^2(m_x+m_t) & k_t h-\omega^2 m_t h & 0 \\ k_t h-\omega^2 m_t h & k_r+k_t h^2-\omega^2 I & 0 \\ 0 & 0 & k_z-\omega^2 m_z \end{bmatrix}+i\omega\begin{bmatrix} c_x+c_t & c_t h & 0 \\ c_t h & c_r+c_t h^2 & 0 \\ 0 & 0 & c_z \end{bmatrix} \quad (2.1)$$

The thirteen constants comparing in (2.1) and defining the LPM are calibrated with a least mean squares procedure in order to achieve the better approximation, for a frequency range of interest, of the real and imaginary parts of the foundation impedances evaluated in the previous step for the column and wall foundations. Each foundation (columns and wall) has a relevant LPM, depending on the layout of the piles, that is assembled to the superstructure so that the three displacements previously defined coincide with the relevant degrees of freedom of the column or the wall base.

2.3. Inertial interaction analysis

The inertial interaction is obtained by means of a non linear dynamic analysis of the structure on compliance base characterized by LPMs previously described. The foundation input motion, obtained by means of the kinematic interaction analysis, is characterised by the translational and rotational components, associated to the three degrees of freedom of the master nodes placed at the centroid of the pile rigid caps.

3. SEISMIC RESPONSE OF COUPLED WALL-FRAME SYSTEMS

The seismic response of coupled wall-frame systems is investigated with reference to a case study for which both compliance and fixed base models are considered. A 6-storey 4-bay structure extracted from a regular building with interstorey height of 3.20 m is considered (Figure 2*a*). The structure is designed according to the Direct Displacement-Based Design (DDBD) (Priestley et al., 2007) by imposing the displacement configuration at the ULS and assuring the strength hierarchy thanks to a suitable capacity design. Concrete C25/30 and steel grade B450C are considered in the design; reinforcement ratios fulfil minimum values prescribed by EN 1998-1 (2004). The wall-frame system is founded on piles having diameter ϕ = 0.6 m; single pile foundations are considered for columns and a 3x2 pile group for the wall. The foundation caps are connected by tie-beams. Figure 2*b* shows the foundation geometries for wall and columns considered in the numerical applications. A soil profile constituted by a soft layer 12 m thick overlying a stiffer stratum is considered. The surface layer is characterized by shear wave velocity V_{s1} = 100 m/s and density ρ_1 = 1.5 Mg/m³ while the stiffer layer has shear wave velocity V_{s2} = 400 m/s and density ρ_2 = 1.8 Mg/m³. A constant material damping ratio ξ = 5 % and a constant Poisson's ratio ν = 0.4 are assumed for the soils.

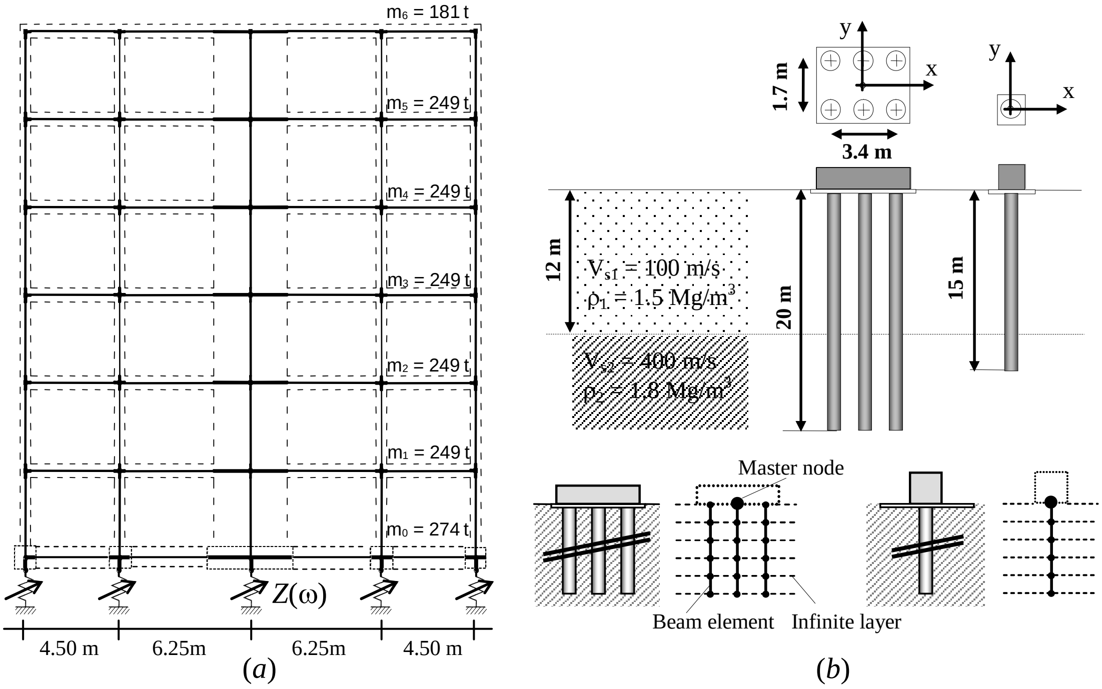


Figure 2 (*a*) Coupled wall-frame system; (*b*) geometry and model of the foundations and soil properties

3.1. Soil-foundation model

The kinematic interaction is performed by means of the numerical procedure proposed by the authors (Dezi et al., 2007). Each pile is modelled by 1 m long finite elements to provide a suitable level of accuracy (Figure 2*b*). The incoming seismic action at the outcropping soil is represented trough three artificial accelerograms matching the EN 1998-1 (2004) elastic response spectrum for soil type D and *PGA* 0.25*g*.

Results of this stage of the analysis are the foundation input motions and the frequency-dependent impedances of the soil-foundation systems necessary for the subsequent inertial interaction. The foundation input motion is characterised by the translational and rotational components, associated to the three degrees of freedom of the master nodes placed at the centroid of the pile rigid caps. The dynamic behaviour of the foundations is accounted for by means of the LPM described in the previous section by calibrating the constant parameters in order to obtain the better approximation of the real and imaginary parts of the soil foundation impedances in the range 0-20 *Hz*. Figure 3 shows the non-zero components of the impedance matrix of the wall foundation; the dash-dotted lines represent the frequency dependent soil-foundation impedances, while the continuous lines are the LPM approximated solutions.

3.2. Structural model

The non-linear dynamic analysis of the structure is performed by means of *SeismoStruct* (SeismoSoft, 2007). Two different cases are considered: a Fixed Base (FB) model fully restrained at the base nodes of columns and wall and a Compliance Base (CB) model obtained by assembling at the base nodes the LPMs previously defined accounting for the dynamic properties of the soil-foundation systems. In the FB model the fully restraint at the column base is justified by the presence of a stiff tie-beam at foundation level.

Frame and wall members are modelled by beam elements for which the non-linear mechanic behaviour is accounted for introducing zero-length links (plastic hinges) placed at the ends of rigid links necessary to simulate the beam-to-column joints and the real width of the wall. The in-plane rigidity of the floor is included by increasing the axial stiffness of the beam elements.

The masses associated to the different floors of the system are shown in Figure 2*a*. The masses at the ground level are also considered since they affect the structural dynamics in the soil-structure interaction analysis.

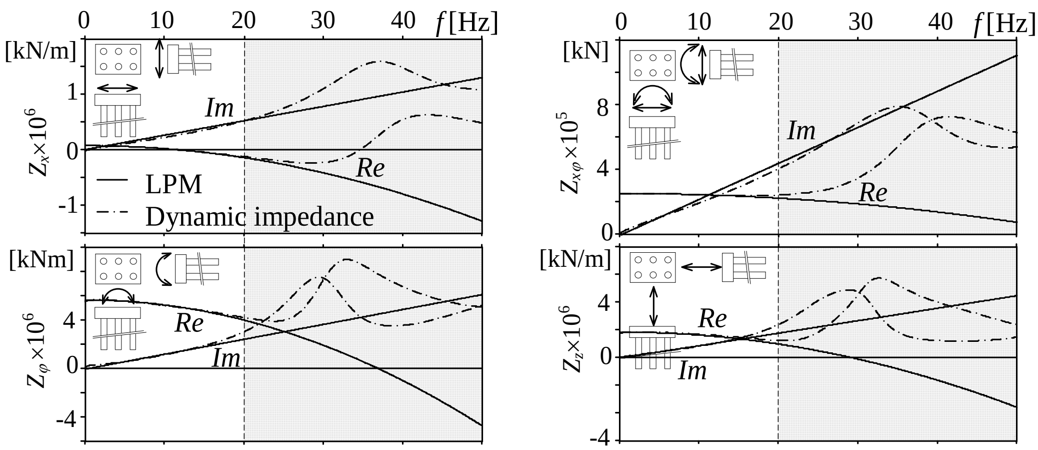


Figure 3 Wall foundation impedances

Stiffnesses of beams and columns are determined by means of a sectional moment-curvature analysis in order to catch the smeared cracking of the elements. The same sectional analyses are used to define the moment-rotation relationships for the plastic hinges. Mander's law (Mander et al., 1988) and King's law (Montejo and Kowalsky) are considered for the confined and unconfined concrete and for the reinforcements, respectively. In the definition of the non-linear links the elastic contribution of the beam section in the zone of the plastic hinge as well as the strain penetration and the tension stiffening are taken into account. The Takeda's hysteretic model (Takeda et al., 1970) is considered to simulate the non-linear cyclic behaviour of the links. 5% structural damping is introduced in terms of tangent stiffness proportional damping (Priestley and Grant, 2005).

3.3. Main results

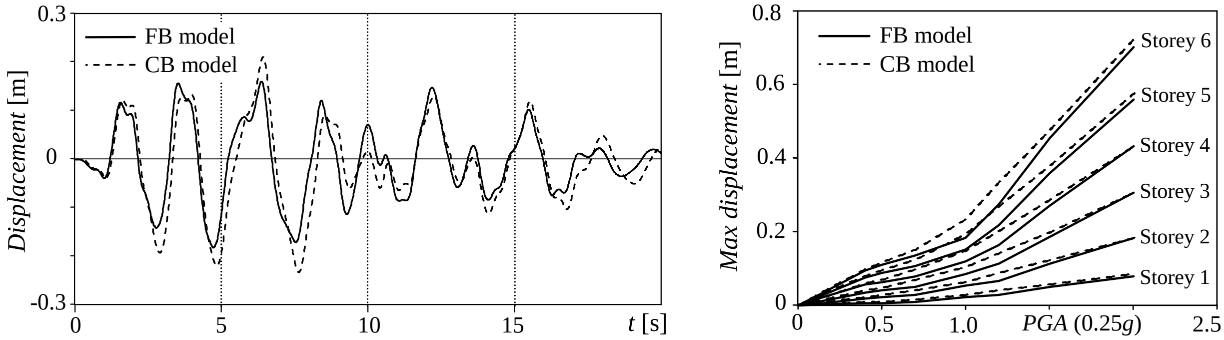
In this section the effects of the soil-structure interaction on the non-linear seismic response of the analysed coupled wall-frame system are discussed by comparing the main results with those obtained by the fixed base model.

3.3.1 Displacements

Figure 4*a* shows the time history of the displacement at a control point placed on the last level obtained with one accelerogram for *PGA* = 0.25*g*. As expected, the soil-structure interaction leads to an important increase of the global structure deformability and the displacements obtained on the compliance base model are 25% higher than those of the fixed base one. This result may be of a certain practical interest because neglecting the effects of soil-structure interaction may lead to a non conservative evaluation of the seismic gaps between adjacent structures.

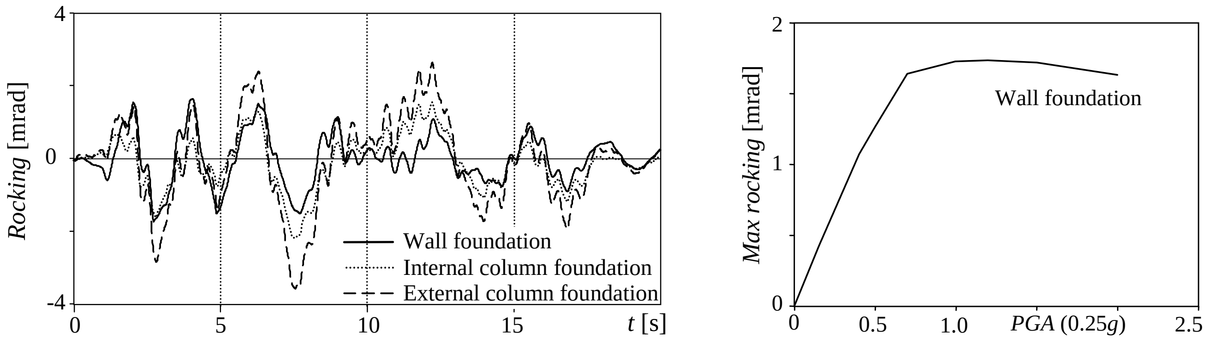
Figure 4*b* shows the maximum displacements at each floor obtained with an incremental dynamic analysis. The maximum differences with respect to the FB structure are obtained for *PGAs* nearby the design value of 0.25*g* where a change of the curve slope is also evident as a consequence of the incipient plasticization of the structure. At higher *PGA* values the differences between the FB structure and the CB one become less significant.

Figure 5*a* shows the time histories of the rocking of the wall and columns foundations obtained from the CB structure analysis. Different amplitudes of the wall and column foundation rocking are evident; in particular the external column foundation undergoes rotations that are greater compared with that of the internal one and this is due to the different degree of the restraint exerted by the rigid tie-beams at the foundation level. The foundation rocking revealed to play a key role in the seismic response of the coupled wall-frame structure increasing the storey displacements and the interstorey drifts. This is important in the seismic damageability analysis (DLSs) since it may induce the early failure of the non-structural elements (Carbonari et al. 2008a-b). Figure 5*b* shows the maximum wall foundation rocking obtained with an incremental dynamic analysis. An almost linear relationship with the seismic intensity is evident for *PGAs* less than the 70% of the design value. For higher seismic intensities the curve presents a plateau that is due to the plasticization at the base of the wall that limits the moment resisted by the foundation.



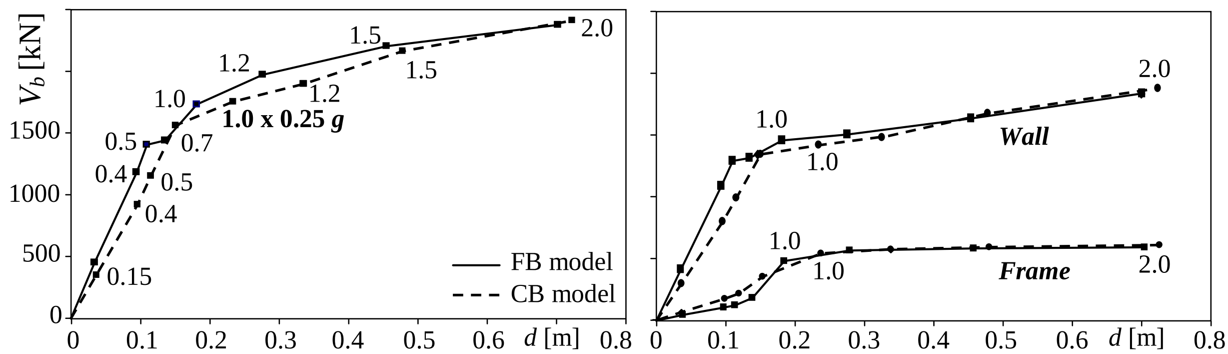
(*a*) (*b*)

Figure 4 (*a*) Time history of the displacement at the last storey; (*b*) storey displacements



(*a*) (*b*)

Figure 5 (*a*) Time history of the foundation rocking; (*b*) maximum wall foundation rocking



(*a*) (*b*)

Figure 6 (*a*) Capacity curves of the FB and CB structures; (*b*) maximum wall and frame base shears

3.3.2 Dissipative mechanisms

Capacity curves of the structure, reporting maximum base shear vs. maximum displacement of the control point at the last level, are constructed by incremental dynamic analyses (Figure 6*a*) showing the global behaviour of the structure subjected to different seismic action levels. *PGA* values, normalised with respect to the design value 0.25*g* are reported in the diagrams. It may be remarked that in the elastic range the CB structure is characterized by a minor global stiffness while, as expected, at ultimate conditions the total seismic base shears resisted by the FB and the CB structures converge to the same value. Differences are evident in the elastic-early plastic behaviour where the deformability of the soil-foundation system affects the initial structural stiffness and the evolution of the dissipative mechanism. Figure 6*b* shows the maximum base shear resisted by the wall and the frame under increasing seismic actions. In the elastic range the soil-structure interaction leads to a decrease of the wall base shear and an increase of the frame base shear as a consequence of the wall rocking. This effect ends when the plastic hinge at the wall base activates producing a migration of the shear force from the wall to the frame. It is worth noticing that the shear migration to the frame is more gradual than in the case of the FB structure as can be observed from the dashed curve of Figure 6*b*. The progress in the dissipating mechanisms for the FB structure and for the CB structure is shown in Figure 7. For the case study, the capacity design is able to guarantee the formation of a proper dissipating mechanism with hinges located at the beam ends and at the base of the wall and columns for both fixed and compliance base models. Soil-structure interaction influences the evolution of the dissipative mechanism by delaying the formation of the hinge at the wall base and anticipating the formation of those at the frame lower levels. It is worth noticing the early formation of the plastic hinge at the base of the interior columns in the case of the compliance base model. As expected the stiffness of the tie-beams at the foundation level plays an important role in the formation process of the plastic hinges in columns.

3.3.3 Ductility demand

Figure 8 shows the hysteretic cycles of the plastic hinges of the beam localised near the wall at the first storey (A) and at the last storey (B) and the plastic hinge at the base of the wall (C) for the FB and the CB structures Comparisons among the results obtained with the FB model show that the soil-structure interaction induces an increment of the rotation demand to the hinges at the beam ends but not at the wall base. It is worth noticing that this should not be a problem if beams are well designed with sufficient ductility to withstand the increment of the rotation demand that is of the same order of the wall rocking. Furthermore it is worth noticing that a major amount of energy is dissipated by the structure.

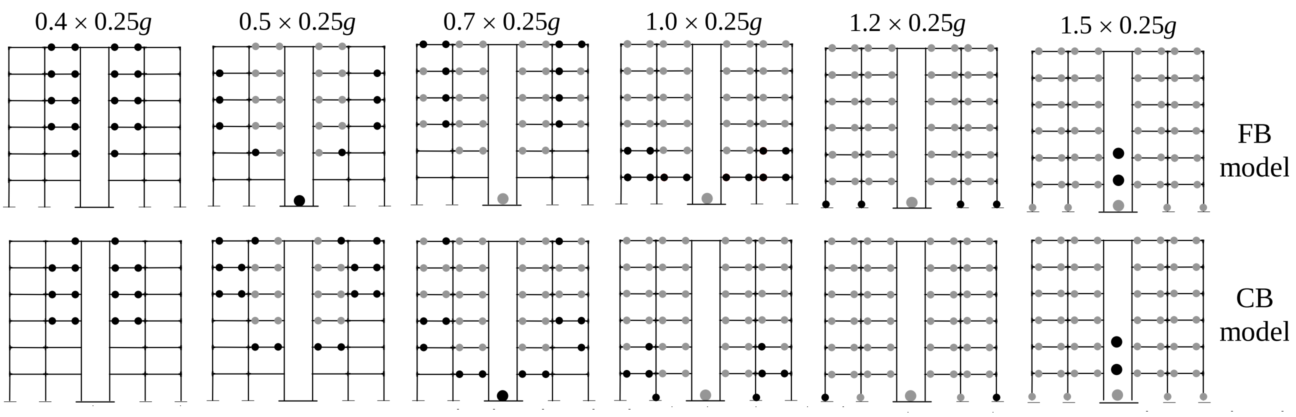


Figure 7 Evolution of the dissipative mechanism

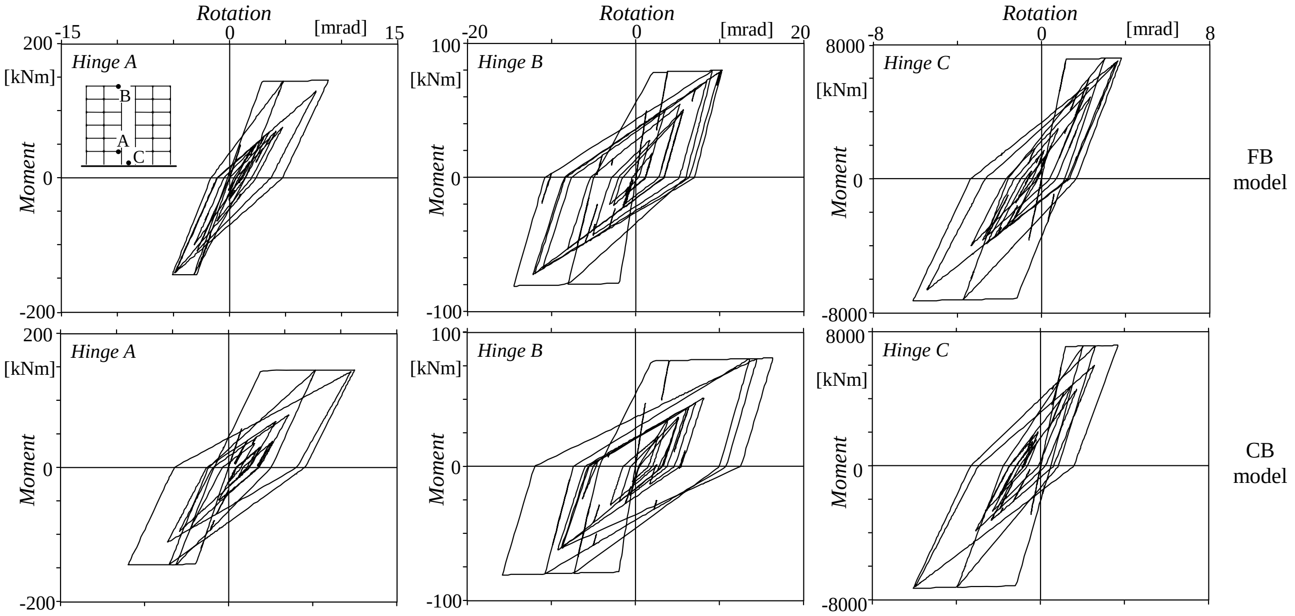


Figure 8 Hysteretic cycles of three different plastic hinges

4. CONCLUSIONS

The effects of soil-structure interaction in the seismic response of a coupled wall-frame structure have been investigated accounting for the non-linear behaviour of the structure. Lumped Parameter Models have been introduced to reproduce in the time domain the frequency dependent impedance functions of the soil-foundation systems including the roto-traslational coupling component. Analyses have been performed with respect to a soft soil profile for which the soil-structure interaction effects are expected to be significant. From the comparison with a fixed base model the following remarks may be drown:

- SSI increases the lateral deformability of the whole structure. The horizontal displacements become sensibly higher with respect to a fixed base model with consequences in the design of seismic gaps between adjacent buildings. Interstorey drifts also augment by leading to a possible early damage of the structure subjected to weak seismic motions (Damageability Limit State);
- in the elastic range the shear distribution depends on the effective stiffness of the components and is significantly affected by the soil foundation deformability; SSI reduces the shear at the wall base and increases that at the frame base. At ultimate, no significant differences are observed between the fixed base and the compliance base models, since plastic hinge development produces a shear redistribution between wall and columns;
- SSI accelerates the formation of the plastic hinges in the lower level beams and delays the wall base one;
- SSI increases the beam ductility demand, especially at higher levels, due to the foundation rocking. On the other hand, for the wall base the maximum rotation of the hinge is not affected by SSI.

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