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ASSESSMENT OF 3D BUILDINGS' SEISMIC DAMAGE CONSIDERING KINEMATIC AND INERTIAL SOIL-STRUCTURE-INTERACTION EFFECTS

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ABSTRACT

The present paper investigates the response of low and mid-rise 3D reinforced concrete frame buildings accounting for both kinematic and nonlinear inertial Soil-Structure-Interaction (SSI). The buildings are designed according to the guidelines of Eurocodes. Accounting for inertial SSI, the foundation soil flexibility is modelled through the nonlinear Beam on Winkler Foundation concept (BNWF) as is integrated in the software platform OpenSees which was used herein. The kinematic interaction is considered through state-of-the-art analytical solutions of transfer functions relating the translational and the corresponding rotational components of the Foundation Input Motion (FIM) with the free field, ground surface seismic excitation. Two approaches of SSI are examined, namely one considering both kinematic and inertial SSI (complete SSI) and another considering only the inertial part of SSI with the free field, ground surface time history as seismic input. The two buildings are subjected to 19 earthquake acceleration time histories (9 far fault and 10 near fault) recorded at soft soil sites (Eurocode 8 ground category C) and scaled at two performance levels (SD and NC). Comparison in terms of Maximum Interstorey Drift Ratio (MIDR) is made among the two SSI approaches and the fixed base approach and the results are reported. It is concluded that the effect of kinematic interaction is important even for small foundation embedment depth. Also, other important aspects that may determine whether SSI is favorable or unfavorable for the structural response are the performance level considered and the proximity of the site to the fault.

Keywords: Soil–structure–interaction; Vulnerability assessment; Seismic damage; R/C buildings

1. INTRODUCTION

Seismic Soil-Structure-Interaction (SSI) refers to how the flexibility of the supporting soil affects the response of a structure and vice versa, that is how the response of the supporting soil is affected by the inertial forces which are transferred to it by the structure. Although the interaction process occurs in both ways simultaneously, it is usually more convenient to study it in two distinct consecutive phases referred as kinematic and inertial interaction (Kausel et al. 1978, Wolf and von Arx 1978). The kinematic interaction refers to the effect of the seismic incident waves on a system which comprises of the foundation and the surrounding soil. In this phase, the system is considered to be massless. The result of the kinematic interaction analysis is the foundation input motion. The inertial interaction follows with the seismic analysis of the system including the foundation, the surrounding soil and the superstructure imposed to the foundation input motion. Extended research has been conducted over the last decades in order to investigate the effect of soil flexibility on the response of Single or Multiple Degree of Freedom systems (Roy et al. 2012, Rodriguez and Montez 2000, Matinmanesh and Saleh 2011, Nakhaei and Ghannad 2008, Lin and Miranda 2008, Mahsuli and Ghannad 2009, Ghandil and Behnamfar 2017) or in order to develop tools towards this direction (Gazetas 1991, Harden and Hutchinson 2009, Paolucci et al. 2013). In addition to this, special attention has been given to the kinematic component of SSI and the estimation of the foundation input motion by investigating the basic contributions which are referred as base slab averaging and embedment effects. However, whether the effect of SSI is favorable or not is not straightforward (Mylonakis and Gazetas 2000). Many parameters related to both the superstructure and the foundation system should be defined so that an estimation of the SSI effect is obtained (Veletsos and Nair 1975).
In this paper, the response of low and mid-rise 3D reinforced concrete buildings, accounting for both kinematic

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and inertial Soil-Structure-Interaction (SSI), is investigated. The buildings' set includes frame systems as defined per EN1998-1 (2005). The foundation system of each building consists of individual footings and link beams. The buildings were designed according to the EN1992-1-1 (2005) and EN1998-1 (2005) provisions accounting for foundation flexibility. The buildings under investigation were also investigated in the past, through Nonlinear Time history Analyses, in order to highlight the effect of nonlinear SSI on their damage index in comparison with analyses where no SSI consideration was taken at all. These analyses included only consideration of inertial SSI that is the effect of foundation flexibility on the response of the superstructure. This was made through nonlinear modeling of the footing with the utilization of the BNWF (Beam on Nonlinear Winkler Foundation) approach (Harden and Hutchinson 2009). The earthquake excitation included a set of free field, translational earthquake acceleration records, covering a wide range of characteristics, applied at the base of the buildings.
In the present study, the free field earthquake acceleration records are modified in the frequency domain to account for embedment effects and corresponding rotational components of earthquake excitation are obtained due to kinematic SSI. The modification and calculation of excitation components is made through the state-of-the-art methods used in practice and described in the corresponding literature (Mylonakis et al. 2006). After modification of the translational excitation records and calculation of the rotational components in the frequency domain, the resulting earthquake excitation is transformed back to the time domain to obtain the foundation input motion to be used in the Nonlinear Time History Analyses (NTHA). The comparison to be presented mainly refers to the damage index of the buildings which is represented by the Maximum Interstorey Drift Ratio (MIDR). The comparison is twofold since it is made both between the approach considering complete SSI (kinematic and inertial) and the approach which considers only inertial SSI and between the complete SSI approach and the no SSI approach as well. Thus, quantification of the SSI components effect is possible for the present set of realistic R/C buildings layouts designed according to the modern building codes.

2. MODELING OF NONLINEAR SOIL-STRUCTURE INTERACTION

2.1. Inertial soil-structure Interaction

The Beam on Nonlinear Winkler Foundation (BNWF) model, included in Harden and Hutchinson (2009), was implemented to model the foundation. The BNWF model is a classic Winkler spring model which captures some basic characteristics of foundation response by using a number of springs with appropriate constitutive laws and elements. A representation of BNWF along with the associated nomenclature is shown in Figure 1. The basic expressions used to calculate the properties of the foundation model are presented in Table 1.

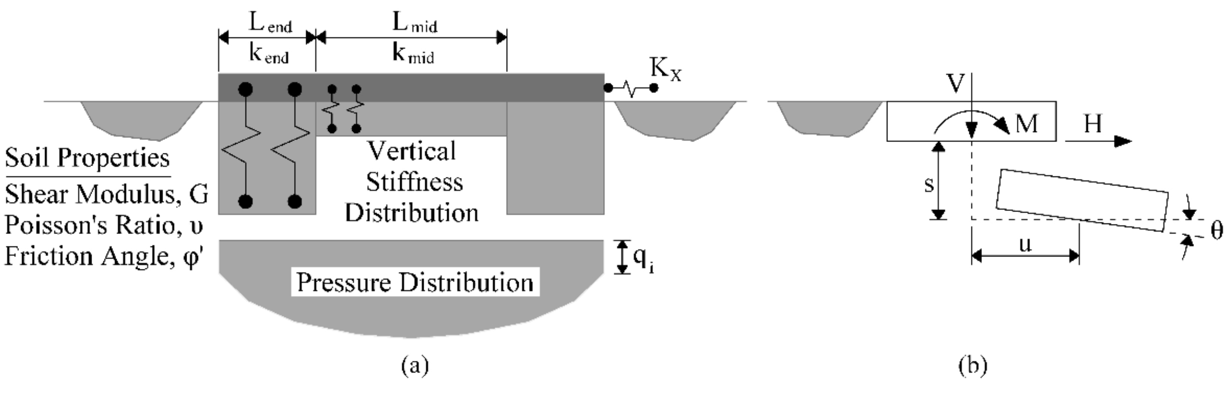


Figure 1. (a) Graphical representation of BNWF model and associated study parameters and (b) nomenclature for displacements and loads (Source: Harden and Hutchinson 2009)

Table 1. Basic expressions used for the calculation of the BNWF foundation model (Harden and Hutchinson 2009).

Model Component	Associated property	Expression
Vertical springs	Elastic stiffness	$K_Z = K_{z,Gazetas} e^{-1.374\ln(Q_{ULT}/G_{int}A)-3.5}$
	End Length	$L_e = \frac{L}{2}\left(1-(1-C_{R-V}^K)^{\frac{1}{3}}\right)$ $C_{R-V,x}^K = \frac{K_{\theta y} - \frac{K_Z}{A} I_y}{K_{\theta y}}$ $\quad C_{R-V,y}^K = \frac{K_{\theta x} - \frac{K_Z}{A} I_x}{K_{\theta x}}$
	Mid region subgrade reaction modulus	$k_{mid} = K_Z/A$
	Edge region subgrade reaction modulus	$k_{end,x} = k_{mid} + \frac{K_{\theta y}}{I_y} C_{R-V}^K$ $\quad k_{end,y} = k_{mid} + \frac{K_{\theta x}}{I_x} C_{R-V}^K$
	Bearing capacity	$q(x) = cN_c + q_b N_q D + q_\gamma(x)$ $q_\gamma(x) = \frac{1}{2}\gamma L N_\gamma \left[F_{qi} + \frac{6}{L^2}(1-F_{qi})\left(\frac{L^2}{4} - x^2\right)\right]$ $F_{qi} = \frac{q_{edge}}{q_{ave}} = \frac{1}{2}\left(\tan\varphi' + \frac{1}{F_{SV}}\right)$
	Seismic bearing capacity reduction factor	If $F_{SV}<4.0$, FQ=1.0 If $4.0 \leq F_{SV} \leq 6.0$, FQ= $(FQ^U + FQ^L)/2$ If $F_{SV} > 6.0$, FQ= FQ^L.
Horizontal springs	Elastic stiffness	$K_X = K_{x,Gazetas}(0.312\, F_{SV} + 1.134)$
	Friction resistance (pure frictional soil)	$V_{fr} = N \tan(\varphi_{cs})$
	Friction resistance (pure frictional soil)	$\varphi_{cs} = \varphi' - 3(Dr(10 - \ln(\sigma_v'/p_a)) - 1)$
	Cohesion resistance (pure cohesive soil)	$V_c = \left(\frac{2}{3}c\right) A_{foot}$
Viscous dampers	Radiation Damping (Gazetas 1991)	$C_Z=\rho V_{La} A_{foot}$ $\quad C_x=C_y=\rho\, V_s A_{foot}$ $\quad C_{rx}=C_{ry}=0$

The BNWF model (proposed by Harden and Hutchinson (2009)) was calibrated on a number of centrifuge tests which included footings lying on frictional or cohesive soil subjected to seismic excitations with various intensities. The BNWF was developed utilizing hysteretic models of the open source software platform OpenSees (McKenna and Fenves 2004). Except for the nonlinear springs assigned, viscous dampers were placed below each footing to model radiation damping. The viscous dampers properties were calculated by the expressions of Gazetas (1991) and Mylonakis et al. (2006). The value of the dynamic coefficient which multiplied the static stiffness and damping coefficients was calculated at the fundamental period of each case fixed base building.

The R/C buildings, which were examined in the present paper (see section 3), were designed and analyzed considering soft sand as foundation soil. The basic properties of the soil considered are shown in Table 2. In order to calculate the properties of each footing, the soil Shear Modulus at small strains (G_{max}) calculated by the shear wave velocity and the soil density was reduced by a factor depending on the Peak Ground Acceleration of the seismic motion, as suggested by EN1998-5 (Table 4.1 - §4.2.3(3)) (2005).

Table 2. Soil properties considered in the present study.

Soil property	Symbol, units	Values
Shear wave velocity	$V_{s,max}$ (m/s)	180
Modulus of subgrade reaction (used for design)	K_s (kN/m^3)	(Sand: 70000)
Unit weight	γ_s(kN/m^3)	20
Friction angle (for sands only)	φ (°)	28
G/G_{max} (for PGA=0.276g)	-	0.39
G/G_{max} (for PGA=0.475g)	-	0.15

2.2 Kinematic soil-structure interaction

The kinematic soil – structure – interaction component was addressed herein according to the recommendation of Mylonakis et al. (2006) who adopted the expressions originally published by Elsabee et al. (1977). According to them, for embedded foundations subjected to vertical and oblique shear waves, the translational and rotational component of the Foundation Input Motion (FIM) are calculated in the frequency domain as:

$$U_{FIM} = U_{free-field} \cdot I_U(\omega) \tag{1a}$$

$$I_U(\omega) = \begin{cases} \cos\left(\dfrac{\pi f}{2f_D}\right), & f \le \dfrac{2}{3} f_D \\ 0.5, & f > \dfrac{2}{3} f_D \end{cases} \tag{1b}$$

$$\Phi_{FIM} = \frac{U_{free-field}}{B} \cdot I_\Phi(\omega) \tag{2a}$$

$$I_\Phi(\omega) = \begin{cases} 0.2 \cdot \left[1 - \cos\left(\dfrac{\pi f}{2f_D}\right)\right], & f \le f_D \\ 0.2, & f > f_D \end{cases} \tag{2b}$$

In the above expressions, f is the frequency of oscillation, $f_D = V_s/(4D)$ is the fundamental frequency of a hypothetical soil layer of thickness D, and B is the halfwidth of the foundation. The transfer functions $I_U(\omega)$ and $I_\Phi(\omega)$, are defined in the frequency domain. As such, the earthquake time histories obtained on the ground surface of the free field are transformed in the frequency domain through Fast Fourier Transformation (FFT). Their Fourier Amplitude is multiplied by the transfer functions $I_U(\omega)$ and $I_\Phi(\omega)/B$ and the FIM (consisting of horizontal translation U_{FIM} and rotation Φ_{FIM}) is obtained in the frequency domain. Subsequently, both U_{FIM} and Φ_{FIM} are transformed back to the time domain through inverse FFT to obtain the time history of the FIM.

3. DATA OF THE NUMERICAL APPLICATIONS

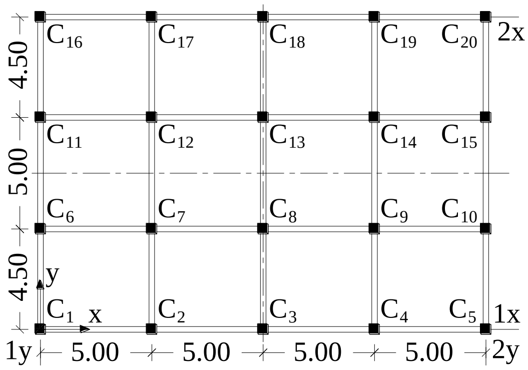
In this section the data of the numerical applications which were used to demonstrate the influence of kinematic and inertial soil-structure interaction on the R/C buildings' seismic damage level are presented.

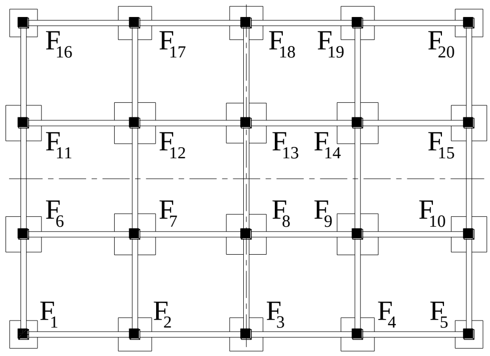
3.1 Description of the examined R/C buildings

For the purposes of the present investigation, two double-symmetric (a 3-storey: "SFxy-3" and a 7-storey: "SFxy-7") R/C buildings, with data supplied in Table 3 and Figure 2 are studied. The buildings have structural system that consists of members in two perpendicular directions (axes x and y, Figure 2). More specifically, the buildings have structural system which is classified to frame systems according to classification of EN1998-1 (§5.2.2.1). Moreover, both buildings are regular in plan and in elevation according to the criteria set by EN1998-1 (§4.2.3).

Table 3. Common design data for the two examined R/C buildings.

Ductility class	**Concrete**	**Steel**	**Slab thickness**	**Slab vertical loads**	**Design spectrum (EN1998-1)**
Medium (DCM)	*C20/25* $E_c=3{\cdot}10^7 kN/m^2$ $\nu=0.2$ $w=25kN/m^3$	*S500B* $E_s=2{\cdot}10^8 kN/m^2$ $\nu=0.3$ $w=78.5kN/m^3$	3-storey building:15cm 7-storey building:16cm	Dead: $G=1kN/m^2$ Live: $Q=2kN/m^2$	*Reference PGA*: $a_{gR}=0.24g$ *Importance class*: II $\rightarrow \gamma_I=1$ *Ground type*: C

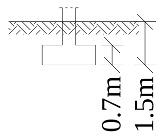




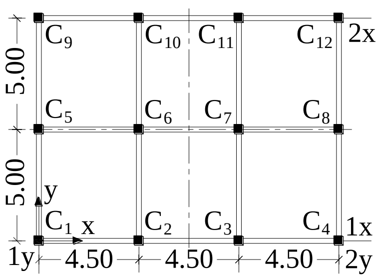
Storey	Columns	Beams
$1^{st}, 2^{nd}$	0.45/0.45	0.25/0.65
3^{rd}	0.40/0.40	0.25/0.60
4^{th}	0.35/0.35	0.25/0.55
5^{th}	0.35/0.35	0.25/0.55
6^{th}	0.30/0.30	0.25/0.50
7^{th}	0.30/0.30	0.20/0.45

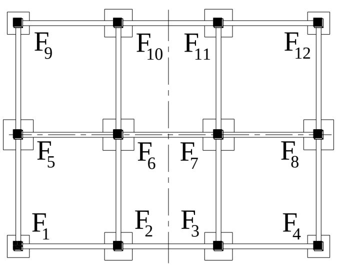
Footings	
F_1, F_5, F_{16}, F_{20}	1.50x1.50
$F_2, F_3, F_4, F_{17}, F_{18}, F_{19}$	1.75x1.75
$F_6, F_{10}, F_{11}, F_{15}$	2.00x2.00
F_7, F_9, F_{12}, F_{14}	2.20x2.20
F_8, F_{13}	2.20x2.20

Foundation beams: 25/60



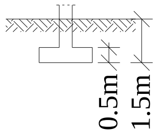
(a)





Storey	Columns	Beams
$1^{st}, 2^{nd}$	0.30/0.30 C_6, C_7: 0.35/0.35	0.25/0.50
3^{rd}	0.25/0.25	0.20/0.45

Footings	
F_1, F_4, F_9, F_{12}	1.00x1.00
F_2, F_3, F_{10}, F_{11}	1.25x1.25
F_5, F_8	1.35x1.35
F_6, F_7	1.40x1.40



Foundation beams: 25/60

(b)

Figure 2. Plan-view, dimensions of the R/C members and foundation system of buildings SFxy-3 (a) and SFxy-7 (b)

3.2 Elastic modeling and design of the structural members

For the buildings' modeling all basic recommendations of EN1998-1 (§4.3.1), such as the diaphragmatic behavior of the slabs, the rigid zones in the joint regions of beams/columns and the values of flexural and shear stiffness corresponding to cracked R/C elements were taken into consideration. The footings were considered and modelled as fully rigid bodies resting on Winkler-type elastic subgrade. The elastic support of the footings was modelled using a bed of vertical springs, which contribute to their vertical as well as their rotational stiffness (see e.g. Avramidis et al. 2016). The foundation beams were modeled using the usual beam elements. The buildings were analysed for static vertical loads as well as for earthquake loads (taking into consideration the accidental torsion effects) using the modal response spectrum analysis, as defined in EN1998-1. The characteristics of the design spectrum used for the analyses are given in Table 3. The R/C structural elements were designed following the provisions of EN1992-1-1 and EN1998-1. Consequently, a capacity design at frame joints was carried out. It should also be noted that the choice of the dimensions of the structural elements' cross-sections as well as of their reinforcement was made bearing in mind the optimum exploitation of the structural materials strength (steel and concrete). Therefore, the Capacity Ratios CRs (where CR=Design value of internal force/Design strength) of all the critical cross-sections due to bending and shear are close to 1.0.

The dimensions of footings were selected using as criterion the soil's bearing capacity. This capacity was calculated on the basis of equation (D.2) of Annex D of EN1997-1 (2005). In addition, the strength (bending and shear) capacities of the bodies of footings were checked using the provisions of EN1992-1-1. The professional program for R/C building analysis and design RAF (T.O.L. 2014) was employed in both the analysis and design.

3.3 Modeling of the nonlinear behavior

For the modeling of the R/C structural elements' nonlinear behavior, force-based beam-column elements were implemented in OpenSees (McKenna and Fenves 2004) using a 5-point Gauss – Lobato integration scheme. Fiber sections, describing exactly the reinforcement detailing were assigned to the beam-column elements whereas appropriate material constitutive laws were utilized for the steel rebar, the unconfined and confined concrete regions respectively.

3.4 Earthquake ground motions

A suite of 19 pairs of horizontal bidirectional earthquake ground motions (9 far-fault (Table 4) and 10 near-fault records (Table 5)) obtained from the PEER (2003) and the European strong motion database (2003) was used as input ground motion for the analyses of the buildings investigated. In order to define whether a ground motion was recorded in the near- or far-field the commonly used distance to the fault is adopted. More specifically, far-fault ground motions are considered the records at more than 15km from the fault trace, as the Uniform Building Code (UBC 1997) suggests. Similarly, the near-fault motions were recorded at less than 15km from the fault trace. At this point, it must be mentioned that there are also alternative criteria of distinguishing seismic motions into near- and far-fault records. Nevertheless, the above criterion was employed in the present paper, as it is one of the most common used criteria in relevant papers (e.g. Alavi and Krawinkler 2001, Iervolino and Cornell 2005). The seismic excitations, which have been chosen from worldwide well-known sites with strong seismic activity, were recorded on Soil Type C according to EN1998-1 and have magnitudes (M_s) between 5.7 and 7.6. The ground motion set employed is intended to cover a variety of conditions regarding tectonic environment (the chosen seismic records correspond to three different fault types: strike slip, normal, reverse), modified Mercalli intensity and distance to fault rupture, thus representing a wide range of intensities and frequency content.
The horizontal recorded accelerograms of each ground motion were transformed to the corresponding uncorrelated ones rotating them about the vertical axis by the angle θ_o (Equation 3) (Penzien and Watabe 1975). Then, the pairs of the uncorrelated accelerograms have been used as seismic input for the analyses of the structures, as ASCE/41-06 (2007) proposes. The characteristics of the input ground motions are shown in Tables 4 and 5 along with the correlation factor of the recorded components p, which is given by Equation 3:

$$p = \frac{\sigma_{xy}}{\left(\sigma_{xx} \cdot \sigma_{yy}\right)^{1/2}}, \ \tan\theta_o = \frac{2\sigma_{xy}}{\sigma_{xx} - \sigma_{yy}} \quad \text{with} \quad \sigma_{ij} = \frac{1}{t_{tot}} \cdot \left(\int_0^{t_{tot}} \alpha_i(t) \cdot \alpha_j(t) dt \right) \quad i = x, y \tag{3}$$

where $\alpha_x(t)$ and $\alpha_y(t)$ are the recorded ground accelerations along the two horizontal directions of the ground motion; σ_{xx}, σ_{yy} are the quadratic intensities of $\alpha_x(t)$ and $\alpha_y(t)$ respectively; σ_{xy} is the corresponding cross-term; t_{tot} is the duration of the motion.
Moreover, in order to account for two different seismic intensities corresponding to certain performance levels suggested by EN1998-3 (§2.1) (2005), the accelerograms were scaled to two different values of Peak Ground Acceleration (PGA). More specifically, the scaling factors adopted in the present study correspond to the following performance levels:
α) Performance level of Significant Damage (SD). According to EN1998-3 (§2.1(3)P) this performance level corresponds to a reference return period of 475 years, which, in turn, corresponds to $PGA=a_g \cdot S=0.276g$, where $a_g=0.24g$ and $S=1.15$ are the design ground acceleration and the soil factor respectively used for the elastic analysis of buildings (EN1998-1 (§3.2.1(3) and §2.1(4)) and the Greek National Annex). So, in this case the 19 pairs of accelerograms were scaled to PGA=0.276g.
b) Performance level of Near Collapse (NC). According to EN1998-3 (§2.1(3)P) this performance level corresponds to a reference return period of 2475 years, which, in turn, corresponds to $PGA=1.72 \cdot a_g \cdot S=0.475g$

(EN1998-1 (§3.2.1(3) and §2.1(4)) and the Greek National Annex). So, in this case the 19 pairs of accelerograms were scaled to PGA=0.475g.

Table 4. Far-fault ground motions recorded on Soil Type C according to EN1998-1.

No	Date	Earthquake name	Magnitude (M_s)	Station number	Distance to fault (Km)	Component (deg)	PGA (g)	Corr. factor p (%)
1	15/10/1979	Imperial Valley	6.9	6624	28.7	012/282	0.270/0.254	-18.63
2	18/10/1989	Loma Prieta	7.1	1028	28.2	090/180	0.247/0.215	-12.02
3	01/10/1987	Whittier Narrows	5.7	24303	25.2	000/090	0.221/0.124	-9.19
4	17/01/1994	Northridge	6.7	90063	25.4	177/267	0.357/0.206	-4.80
5	17/01/1994	Northridge	6.7	90091	30	020/110	0.474/0.439	-6.36
6	15/10/1979	Imperial Valley	6.9	6605	43.6	262/352	0.238/0.351	5.92
7	18/10/1989	Loma Prieta	7.1	57382	16.1	000/090	0.417/0.212	5.98
8	17/01/1994	Northridge	6.7	90054	30.9	155/245	0.465/0.322	-10.16
9	17/01/1994	Northridge	6.7	90022	36.9	090/180	0.290/0.264	-6.95

Table 5. Near-fault ground motions recorded on Soil Type C according to EN1998-1.

No	Date	Earthquake name	Magnitude (M_s)	Station number	Distance to fault (Km)	Component (deg)	PGA (g)	Corr. factor p (%)
1	17/01/1994	Northridge	6.7	90057	13	000/270	0.410/0.482	-14.63
2	17/01/1994	Northridge	6.7	90006	12.3	000/090	0.303/0.443	-3.28
3	01/10/1987	Whittier Narrows	5.7	90077	10.8	048/318	0.426/0.443	-8.09
4	25/04/1992	Cape Mendocino	7.1	89156	9.5	000/090	0.590/0.662	-11.48
5	20/09/1999	Chi-Chi, Taiwan	7.6	101	2.94	N/W	0.251/0.202	-32.97
6	20/09/1999	Chi-Chi, Taiwan	7.6	079	10.04	N/W	0.393/0.742	-7.17
7	20/09/1999	Chi-Chi, Taiwan	7.6	103	4.01	N/W	0.162/0.134	-10.49
8	20/09/1999	Chi-Chi, Taiwan	7.6	101	11.14	N/W	0.440/0.353	-3.68
9	13/03/1992	Erzincan, Turkey	-	95	2.0	N-S/E-W	0.515/0.496	29.84
10	18/10/1989	Loma Prieta	7.1	47380	12.7	000/090	0.367/0.322	26.38

4. ANALYSES RESULTS

The two examined R/C buildings presented were analyzed by Nonlinear Time-History Analysis (NTHA) for each one of the 19 earthquake ground motions taking into account the design vertical loads of the structures. The analyses were performed with the aid of the computer program OpenSees. Two SSI approaches were followed: one considering both kinematic and inertial SSI and one considering only the inertial part of SSI. As a consequence, a total of 152 NTHA (2 buildings x 2 performance levels x 2 SSI approaches x 19 earthquake records) were conducted in the present study.

For each ground motion, the damage state of the two buildings was determined. The seismic performance is expressed in the form of the Maximum Interstorey Drift Ratio (MIDR), which has been chosen, since it lumps the existing damage in all the cross-sections in a single value. So, it has been used by many researchers for the damage assessment of structures. The MIDR, which is generally considered an effective indicator of global structural and nonstructural damage of R/C buildings (Gunturi and Shah 1992, Naeim 2011) corresponds to the maximum drift among the perimeter frames. In order to demonstrate the effect of SSI on the damage state of the buildings, the index Soil-Structure-Interaction Damage Ratio (SSIDR) is defined as (Sotiriadis et al. 2017):

$$\mathrm{SSIDR} = \frac{\mathrm{MIDR}_{\mathrm{SSI}}}{\mathrm{MIDR}_{\mathrm{fixed-base}}} \tag{4}$$

The SSIDR values were calculated for each pair of earthquake recording and the average value of them is reported.

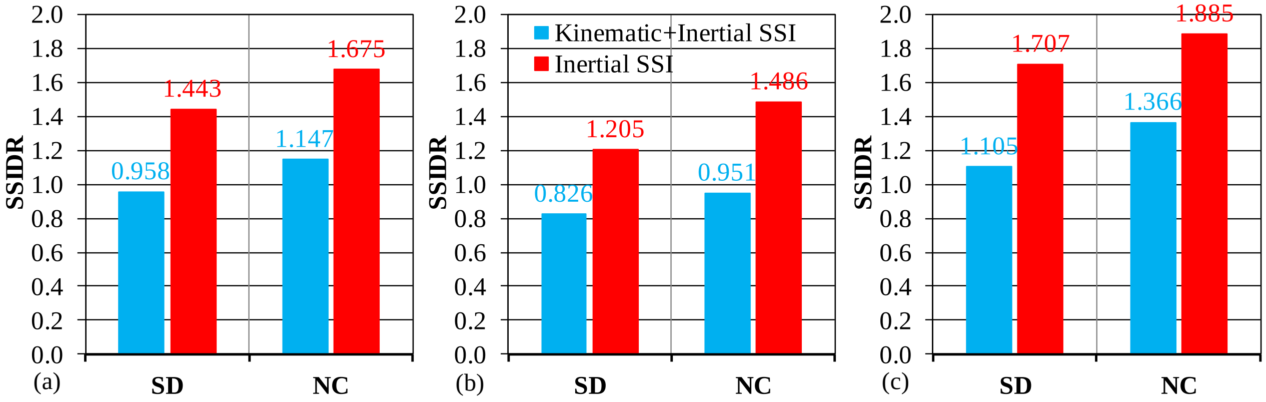


Figure 3. Average SSIDR values of SFxy-3 for all records (a), the Near Fault records (b), and the Far Fault records (c)

In Figure 3 the SSIDR values of the 3-storey frame building (SFxy-3) are shown. Looking at the average SSIDR values of all records, it is observed that consideration of both kinematic and inertial SSI leads to average SSIDR values slightly below unity for the SD performance level implying a favorable effect of SSI. On the other hand, for the NC performance level the average SSIDR is larger than unity implying a detrimental effect of SSI. The difference between the two cases lies, except for the larger seismic intensity for the NC performance level, in the fact that the soil in the latter case is much softer than the former due to the wave passage effects, as suggested in Table 2. Thus, proper estimation of such effects should be carefully assessed as superficial treatment of them may result in misleading outcome.

Furthermore, it is of high significance to note that when only inertial SSI effects are taken into account (together with the free field ground surface acceleration time history as seismic input), the average SSIDR values are highly increased with respect to unity for both SD and NC performance levels, implying a detrimental effect of SSI. The difference between this approach, which is quite common when SSI is considered, and the complete SSI approach (kinematic + inertial) is around 50% regarding the SSIDR and leads to completely different assessment outcome. When the resulting SSIDR values are distinguished according the record type, i.e. far fault or near fault, it is observed that the former lead to unfavorable effects of SSI whereas the opposite stands for the latter. In the case of the inertial SSI only, the SSIDR values are always larger than unity no matter the seismic record type, however, the far fault records give significantly higher values.

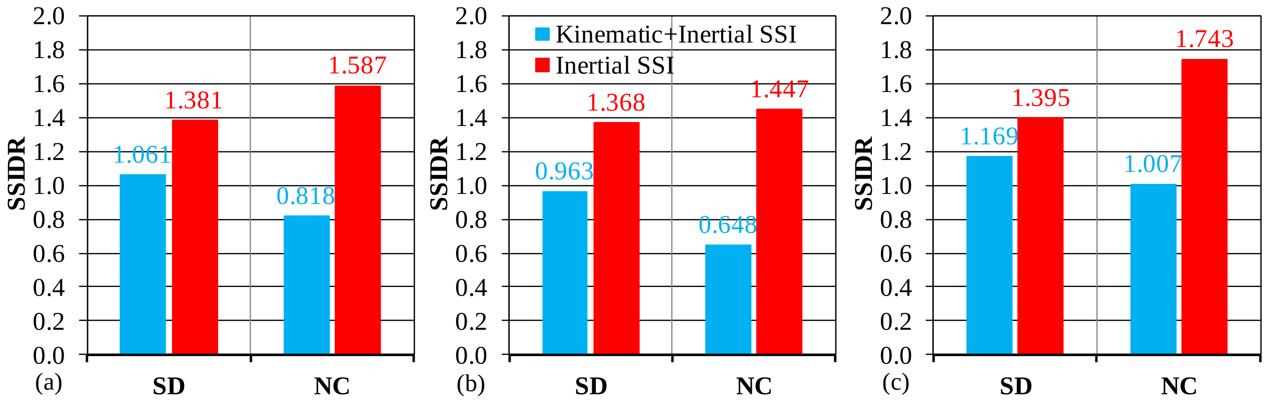


Figure 4. Average SSIDR values of SFxy-7 for all records (a), the Near Fault records (b), and the Far Fault records (c)

Figure 4 presents the SSIDR values of the 7-storey frame building (SFxy-7). The observations made for this case are similar to the ones referring to the 3-storey frame building with one exception. Regarding the NC performance level, the average SSIDR value when considering both kinematic and inertial SSI is below unity whereas for the SD performance level the SSIDR value is slightly above unity. This fact does not follow the trend shown in the 3-storey building case which was attributed to excessive softening of the soil due to wave passage effects. However, when only inertial SSI effects are taken into consideration the SSIDR values for the SD and NC performance support the trend observed in the 3-storey building case. What may lead to differentiation of the complete SSI case of the 7-storey frame building is the presence of rocking components

of the Foundation Input motion. Since the 7-storey building is much higher than the 3-storey building, the rocking components of the seismic action may impose to the foundation significant inelastic deformation. This is shown in Table 6 where the average residual foundation settlements are presented. Residual settlements are indicative of the inelastic response of the foundation soil. Although, the seismic input in the complete SSI approach is lower compared to the inertial SSI and the fixed base approach (see Figure 5), the footing residual settlements for the NC performance level is practically the same giving to the above perception some meaning regarding the effect of the rotational component of the Foundation Input motion. This phenomenon leads to a 94% difference between the average SSIDR values for the NC performance level whereas for the SD performance level this difference is limited to around 30%.
Regarding the comparison of SSIDR values between the two buildings, it is observed that for the SD performance level, the SSIDR value of the 7-storey building is increased with respect to the 3-storey building by about 10.3%. On the contrary, for the NC performance level the average SSIDR value of the 7-storey building is decreased with respect to the 3-storey building by about 29 %. When considering only inertial SSI the average SSIDR value of the 7-storey building in comparison to the 3-storey building is decreased by 4-5% for both performance levels.

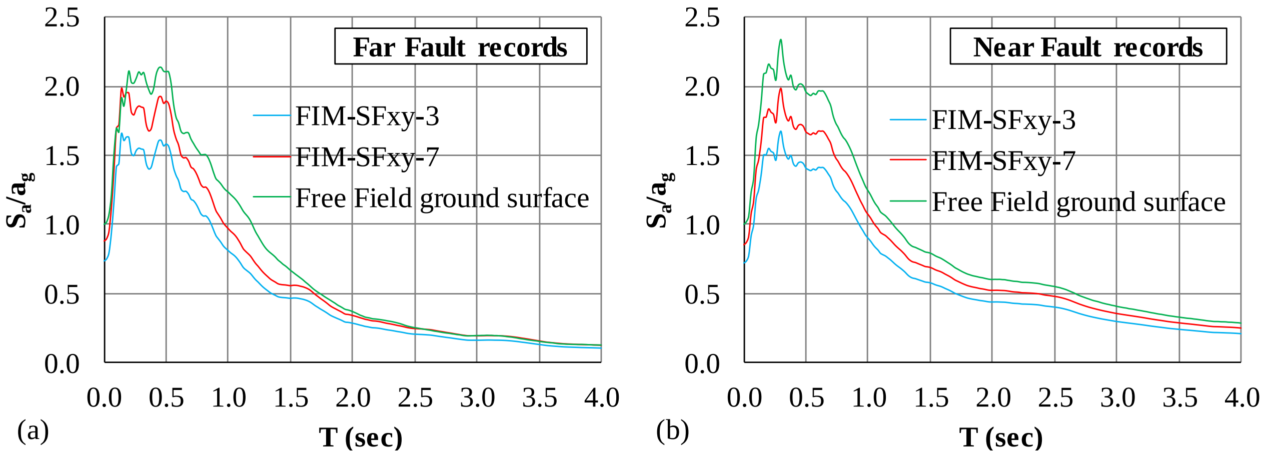


Figure 5. Average response spectrums of Far Fault records (a) and Near Fault records (b)

Figure 5 shows the average response spectrum of the free field ground motion and the Foundation Input Motions (FIM) for both the far fault and near fault records. The response spectrum [$S_{aFIM}(T)$] for the FIMs was calculated considering both the translational and the rocking component as:

$$S_{aFIM}(T) = S_{UFIM}(T) + S_{\Phi FIM}(T) \cdot H_{eff} \tag{5}$$

In the above equation, $S_{UFIM}(T)$ is the response spectrum of the translational component of the FIM, $S_{\Phi FIM}(T)$ is the response spectrum of the rocking component of the FIM and H_{eff} is the effective height of a SDOF oscillator simulating the actual building (it was taken $0.7 \cdot H_{tot}$). As one can observe, the average seismic input at the Foundation level due to kinematic SSI is decreased with respect to the free field seismic input even for low embedment depth as the one considered here. The reduced seismic input level partly explains the lower than unity values of SSIDR and the lower SSIDR values of the complete SSI approach compared to the inertial SSI approach.
The results reported herein, qualitatively agree with the conclusions made in Mahsuli and Ghannad (2009) and Ghandil and Behnamfar (2017). The former reported the ductility demands of SDOF systems including complete SSI and inertial SSI only. They noted that even for small embedment depths a slight increase of ductility demand may occur due to SSI (rocking component of the FIM) especially for soft foundation soils. However, they used an equivalent linear approach for the foundation soil response considering only the wave passage effects as the cause of soil inelasticity. In the study presented herein, foundation soil inelasticity can occur due to inertial loads transmitted by the superstructure as well. Ghandil and Behnamfar (2017) conducted SSI analyses, using the direct method, on multistory steel Moment Resisting Frames lying on soft soil and reported the ductility demands and interstorey drifts. The interstorey drifts reported for structures similar to

the ones presented herein compared to a fixed base approach analysis, agree with ones reported in the present study.

Table 6 presents the average residual settlements of the foundation ($S_{res,avg}$) for each analysis case. Residual settlements can qualitatively define the extent of foundation nonlinear response experienced during an earthquake (Nakano et al. 2004). As observed, for the SD performance level no significant residual settlements result for both the 3 and 7-storey building. This should be expected considering that the design of the buildings was based on such a performance level. However, occurrence of residual displacement indicates that, even instantly, the soil bearing capacity below the foundation was mobilized. Also, the residual settlements resulting from the Inertial SSI are larger than the corresponding ones coming from the complete SSI approach by 142% and 55.5 % for the 3 and 7-storey respectively. This difference seems reasonable considering the difference in base seismic excitation between the two approaches, as shown in Figure 5. Regarding the NC performance level, the foundation residual settlements resulting from both approaches are quite similar for both buildings. Considering the fact that the seismic input of the Inertial SSI approach is obviously larger than that of the complete SSI approach, one may imply that when the FIM is implemented instead of the ground surface free field motion, the rocking component greatly affects the response of the foundation and hence the entire structural system. In any case considered herein, the residual settlements observed do not indicate significant damage on the foundation (Nakano et al. 2004).

Table 6. Average residual settlements of foundation.

Building:	**SFxy-3**				**SFxy-7**			
Performance level:	**SD**		**NC**		**SD**		**NC**	
SSI effects:	**Kinematic + Inertial**	**Inertial**	**Kinematic + Inertial**	**Inertial**	**Kinematic + Inertial**	**Inertial**	**Kinematic + Inertial**	**Inertial SSI**
$S_{res,avg}$ **(m)**	0.0019	0.0046	0.027	0.026	0.0108	0.0168	0.033	0.0353

5. CONCLUSIONS

The present study investigates the effect of kinematic and nonlinear Soil-Structure-Interaction (SSI) on the seismic response of low and mid-rise frame buildings supported on soft soil ($V_{s,max}$=180 m/s). For this purpose, two double symmetric R/C buildings were studied. For the modeling of the buildings' foundation the Beam on Nonlinear Winkler Foundation model was implemented whereas well known analytical expressions to account the kinematic SSI were implemented. Two SSI approaches were followed: one considering both kinematic and inertial SSI and one accounting for only inertial SSI together with the free field ground surface records as seismic input. The buildings were subjected to 19 bidirectional earthquake strong motions (9 far-fault and 10 near-fault records), for which Nonlinear Time History Analyses (NTHA) were conducted. The accelerograms of each record were scaled to two different seismic intensities corresponding to certain performance levels suggested by EN1998-3 using appropriate scaling factors. The damage state of the buildings was expressed through the Maximum Interstorey Drift Ratio (MIDR). The comparative assessment of the results led to the following conclusions:

- The effect of the kinematic part of SSI is significant even for small embedment depth.
- The performance level considered is important as it may affect the implying role of Soil-Structure-Interaction (favorable or unfavorable).
- Accounting only for inertial nonlinear SSI leads to significantly increased SSIDR compared to the complete SSI approach and may cause a different assessment outcome.
- In this study, the selected Far fault records resulted in detrimental effects of SSI whereas the Near fault records resulted in beneficial effects giving SSIDR values below unity.
- The rocking components of the FIM, for the NC performance level, may lead to inelastic response of traditionally designed foundations of mid-rise buildings which results into limitation of the damage level on the superstructure.

Finally, it should be noted that the aim of the present paper is a pilot approach to the estimation of the seismic damage level of buildings taking into consideration the kinematic and inertial SSI effects. Therefore, the above conclusions refer to the systems studied above and are limited within the framework of the assumptions made. Such assumptions include the calculation of the shallow foundation impedances at a specific frequency (i.e. the fundamental frequency of the fixed base structures) instead of the entire frequency range of engineering interest, the ignorance of the interaction between the foundation link beams with the soil and the lack of consideration of the infill wall to the stiffness characteristics of the structures.

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