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The role of soil-structure interaction in the inelastic performance of multi-storey buildings

Christos Giarlelis, George Mylonakis
Department of Civil Engineering, University of Patras, 26500 Rio, Greece
email: giarlelis@alumni.rice.edu, mylo@upatras.gr

ABSTRACT: A comparative evaluation study is reported of the response of a group of 3-storey reinforced-concrete building structures designed according to EC8 and EC2 code provisions subjected to a wide set of seismic excitations. The buildings under consideration employ different structural systems and have been designed as fixed-based according to current design practices. A comprehensive set of non-linear time-history analyses were performed using seismic excitations on soft soil, taking into account the soil structure interaction effect by means of a set of multi-axial springs and dashpots attached at the base of the structures. The results highlight the role of soil in the structural response and show that soil-structure interaction should always be considered in design as it leads to significant redistributions of internal member loads.

KEY WORDS: Soft soil, Soil-structure interaction, Inelastic seismic response, Multi-storey buildings.

1 INTRODUCTION

Seismic design practice may involve methods and beliefs that are not always accurate. An example is the role of soft soil conditions and the inclusion of soil-structure interaction effects (SSI) in the analysis.

Despite numerous studies that have been carried over the years there is still controversy regarding the role of SSI in the seismic performance of structures founded on soft soil. In fact, SSI has been traditionally considered to be beneficial for seismic response.

Taking into account SSI effects is not emphasized in design code provisions. Eurocode 8 partly accounts for soft soil conditions by intoducing a soil factor parameter in the design spectrum. In design practice, neglecting SSI effects is being suggested as a conservative simplification that would facilitate analyses and at the same time lead to improved safety margins. This belief addresses the usually beneficial increase of the period of the structure that would lead to a lower seismic demand but fails to consider the redistribution of internal member loading. It should be mentioned that the above practice is applied for soils of categories A-D, according to EC8 classification, but usually it is not applied when very poor soil conditions or liquefiable soils are present. In those cases, that would require pile foundations or soil treatment, SSI is often considered but only as a means to better design the foundations and not, in most cases, out of a belief that accounting for it would affect noticeably the structure.

However these misconceptions often arise and are discussed after earthquake events. One of these events was the Lefkada earthquake of 14 August 2003, where a number of stiff, low-rise reinforced concrete structures were badly damaged. A study conducted then by the authors (2006, 2010) highlighted the interplay of soil, foundation and superstructure in modifying seismic demand. That study triggered the current investigation of the role of soil-structure interaction in the inelastic performance of multi-storey buildings.

2 BUILDINGS UNDER CONSIDERATION

A group of 3-storey reinforced-concrete building structures were selected as representative case studies. The lateral load resisting system consists either of frames (building 3C) or dual system with main variation in the structural walls size (buildings 3SW and 3BW). As shown in the typical floor layouts, presented in Figure 1, buildings have dimensions of 24.50 m x 15.50 m in plan, consist of 20 vertical elements (columns or walls) bearing beams with a typical span of 6m. Member dimensions are shown in floor layouts. Slabs are 18 cm thick in all floors. Storey height is 3m so the total building height is 9m. Concrete of class C25/30 and steel B500c are considered.

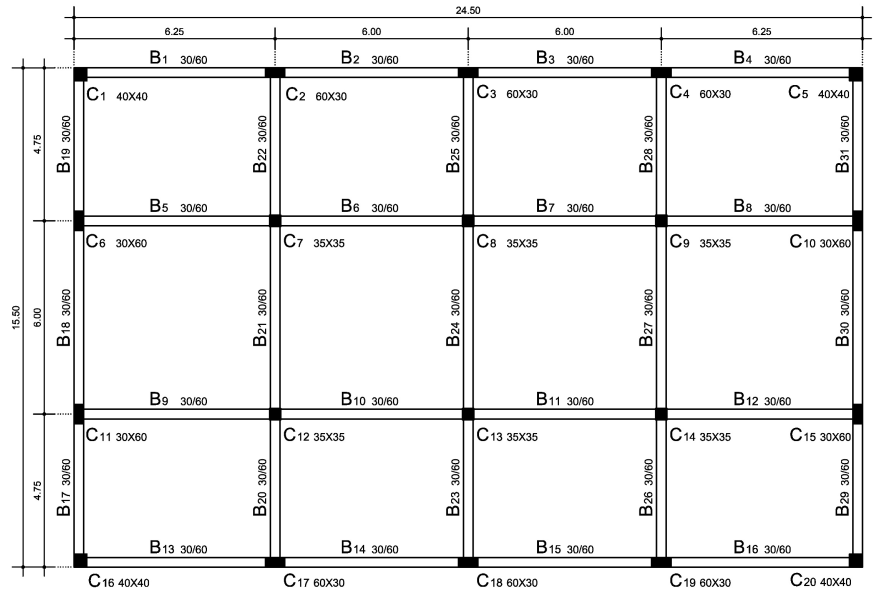
Regarding dead loads, both the self-weight of the structural members and the additional weights from insulation and flooring were considered. Live loads were taken as 2KN/m^2, as required for residential and certain types of office buildings.

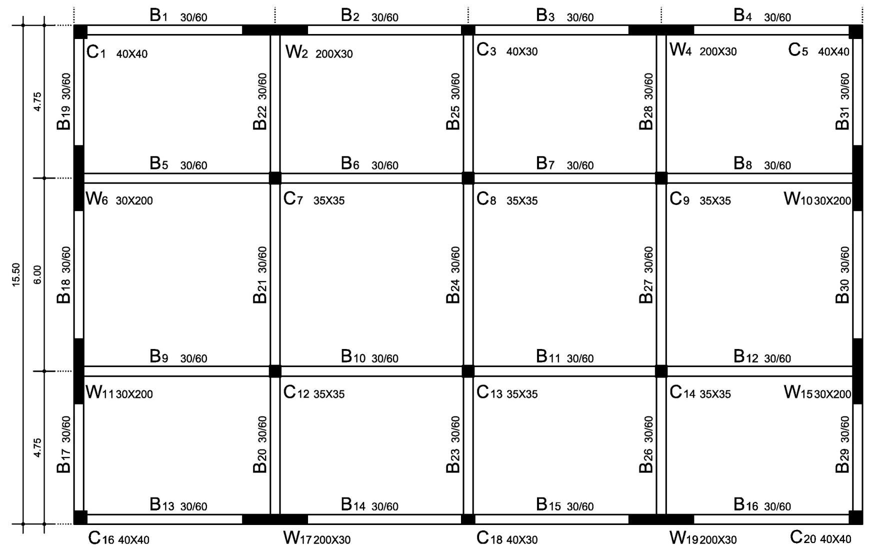
The structures were designed according to EC8 and EC2 code provisions. Seismic action was considered using the following parameters:

- Design ground acceleration α_g = 0.16 (corresponding to Zone I structures in Greece and to an importance factor, γ_I = 1),
- Soil factor S=1.15 for ground type C,
- Damping correction factor η = 1 (for ζ=5%),
- Behavior factor q = 3.

In Figure 2 the corresponding elastic response spectrum is presented. For the seismic load case, the accompanying gravity load combination used was G+0.3Q (where G and Q denote dead and live loads respectively).

For the design of the buildings, all vertical elements were considered fixed at their base following the aforementioned widespread rule of practice. The foundation consists of spread footings with connecting beams under each vertical element.





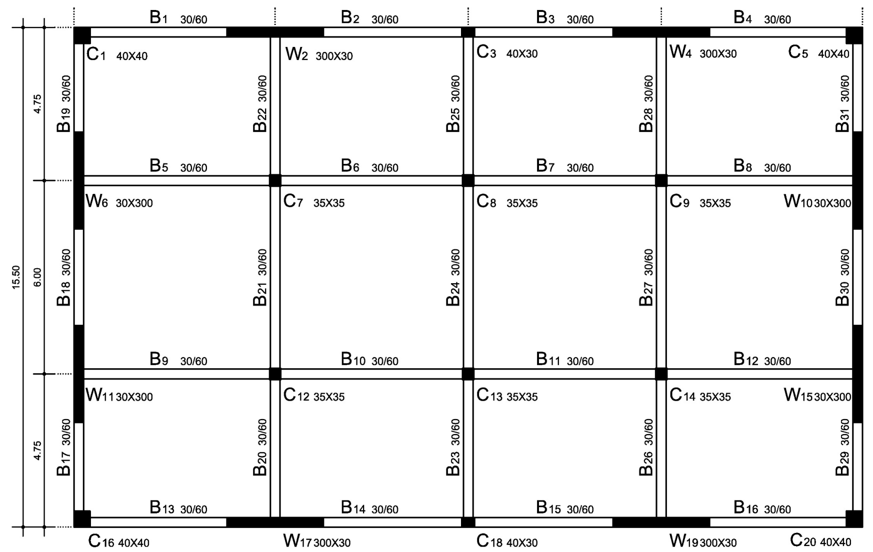


Figure 1. Three storey R/C building structures typical floor layout. From top to bottom buildings 3C, 3SW and 3BW.

TABLE 1. Peak values of recent strong ground motion recordings in Greece: Peak ground acceleration (PGA), Peak ground velocity (PGV), Peak ground displacement (PGD), Housner Intensity (I_H), Arias Intensity (I_A), Bracketed duration (D_B), Destructive potential (D_p). Records from Ambraseys et al (2000)

Earthquake Code	Station	Event/ Date	Magnitude (M_s)	Epicentral Distance (km)	Distance from fault (km)	Ground Conditions	Instrument Orientation		PGA (g)	PGV (cm/s)	PGD (cm)	I_H (cm)	I_A (m/s)	D_b (s)	D_P (cm/s^2)
EQ1	Lefkada OTE	Ionian sea 11/04/1973	5.7	15	11	Soft Soil	L	N-S	0.53	57.02	12.02	168.1	1.36	7.85	0.053
							T	E-W	0.26	25.51	4.77	76.76	0.5	8.40	0.017
EQ2	Thessaloniki City Hotel	Thessaloniki 06/20/1978	6.4	29	17	Soft Soil	L	N-S	0.14	11.79	1.73	41.1	0.17	6.13	0.003
							T	E-W	0.15	15.47	2.39	41.03	0.21	6.35	0.004
EQ3	Korinthos OTE	Alkionides 02/24/1981	6.7	20	13	Soft Soil	L	N-S	0.23	22.36	5.08	94.97	0.64	17.11	0.022
							T	E-W	0.31	22.64	5.40	94.45	0.82	13.11	0.025
EQ4	Pyrgos Agro Bank	Pyrgos 03/26/1993	5.3	10	5	Soft Soil	L	N-S	0.15	9.31	0.97	20.82	0.10	4.48	0.001
							T	E-W	0.43	18.74	1.78	39.68	0.33	4.88	0.003
EQ5	Lefkada OTE	Ionian sea 14/08/2003	6.4	14	14	Soft Soil	L	N65E	0.34	28.4	-	129.1	2.03	15.2	5.1
							T	N335W	0.42	35.2	-	126.6	4.08	10.6	1.2

TABLE 2. Three-storey models used in the analyses.

	NAME	DESCRIPTION
1	3Cst	3 storeys, columns only, medium member stiffness, fixed
2	3CstE	3 storeys, columns only, medium member stiffness, elast. supported
3	3Cfl	3 storeys, columns only, small member stiffness, fixed
4	3CflE	3 storeys, columns only, small member stiffness, elast. supported
5	3SWst	3 storeys, small walls & columns , medium member stiffness, fixed
6	3SWstE	3 storeys, small walls& columns, medium member stiffness, elast. supported
7	3SWfl	3 storeys, small walls & columns, small member stiffness, fixed
8	3SWflE	3 storeys, small walls & columns, small member stiffness, elast. supported
9	3BWst	3 storeys, big walls & columns, medium member stiffness, fixed
10	3BWstE	3 storeys, big walls & columns, medium member stiffness, elast. supported
11	3BWfl	3 storeys, big walls & columns, small member stiffness, fixed
12	3BWflE	3 storeys, big walls & columns, small member stiffness, elast. supported

3 STRUCTURAL RESPONSE INVESTIGATION

The structures's behavior considering or not SSI effects was thoroughly investigated by a series of elastic and inelastic analyses on different models as described below.

3.1 Structures modeling

Each structure is treated as a space frame subjected to combined gravitational and earthquake loading. The beam-column or beam-wall system is modeled with inelastic beam elements located along the centroidal axes of the members. Slabs are considered undeformed in their own plane (diaphragms).
The analysis considers cracked properties for the members. Two different sets of properties are used: (a) properties close to EC8 suggestions with stiffness values equal to 50% of nominal (uncracked) values for beams, 50% of uncracked stiffness for walls and 70% for columns (to be referred hereafter as "medium stiffness" case). (b) Stiffness values at 20%, 20% and 30% of the uncracked for beams walls and columns respectively ("soft stiffness" case) as evaluated from the moment-curvature diagrams of the members.
Regarding the supports, two alternatives are considered: (a) fixed-base conditions and (b) flexible-base conditions. The simulations are performed using the computer codes ETABS 9.72 and Rauomoko 3D (Version 2005) which employ concentrated plasticity models.

3.2 Foundation modeling

Closed-form solutions of dynamic stiffness of spread footings have been derived by regression analysis based on finite- and boundary-element data. The validity of these expressions has been verified by several investigators over the years. Modeling the footings is accomplished using the expressions published by Mylonakis et al. (2006). The dynamic stiffness of the foundation is the product of a dynamic stiffness coefficient times the static stiffness coefficient calculated as:

$$K_z = \frac{2GL}{1-\nu}\left(0.73 + 1.54\chi^{0.75}\right) \tag{1}$$

$$K_y = \frac{2GL}{2-\nu}\left(2 + 2.5\chi^{0.85}\right) \tag{2}$$

$$K_x = K_y - \frac{0.2}{0.75-\nu} GL\left(1 - \frac{B}{L}\right) \tag{3}$$

$$K_{rx} = \frac{G}{1-\nu} I_{bx}^{\;0.75}\left(\frac{L}{B}\right)^{0.25}\left(2.4 + 0.5\frac{B}{L}\right) \tag{4}$$

$$K_{ry} = \frac{G}{1-\nu} I_{by}^{\;0.75}\left[3\left(\frac{L}{B}\right)^{0.15}\right] \tag{5}$$

Where:
- B and L are semi-width and semi length of the circumscribed rectangle.
- A_b, I_{bx}, I_{by} are area and moments of inertia about x and y axes, of the actual soil-foundation contact surface.
- Finally, G and n are the shear modulus and Poisson's ratio, respectively while χ is given by the expression:

$$\chi = \frac{A_b}{4L^2} \tag{6}$$

A footing-soil-footing interaction reduction factor of 30% was also considered after taking into account the size of the footings and the distance between them.
For Soil Category C, the lower bound of corresponding shear wave velocity is $V_{s,30}$ = 180m/s (a value on the limit between Soil Categories C and D). Using this value spring values were calculated for all footings.
Depending on the footing size, values found vary from:
$K_z = 110E^3 \sim 600E^3$ KN/m, K_x, $K_y = 150E^3 \sim 500E^3$KN/m
K_{rx} , $K_{ry} = 100E^3 \sim 5500E^3$ KN m/rad

3.3 Input records for time-history analyses

A series of records from recent distructive earthquakes in Greece were used for the time-history inelastic analyses (Table 2). All these records are recorded on soft soil and come from surface earthquakes with small epicentral distances.

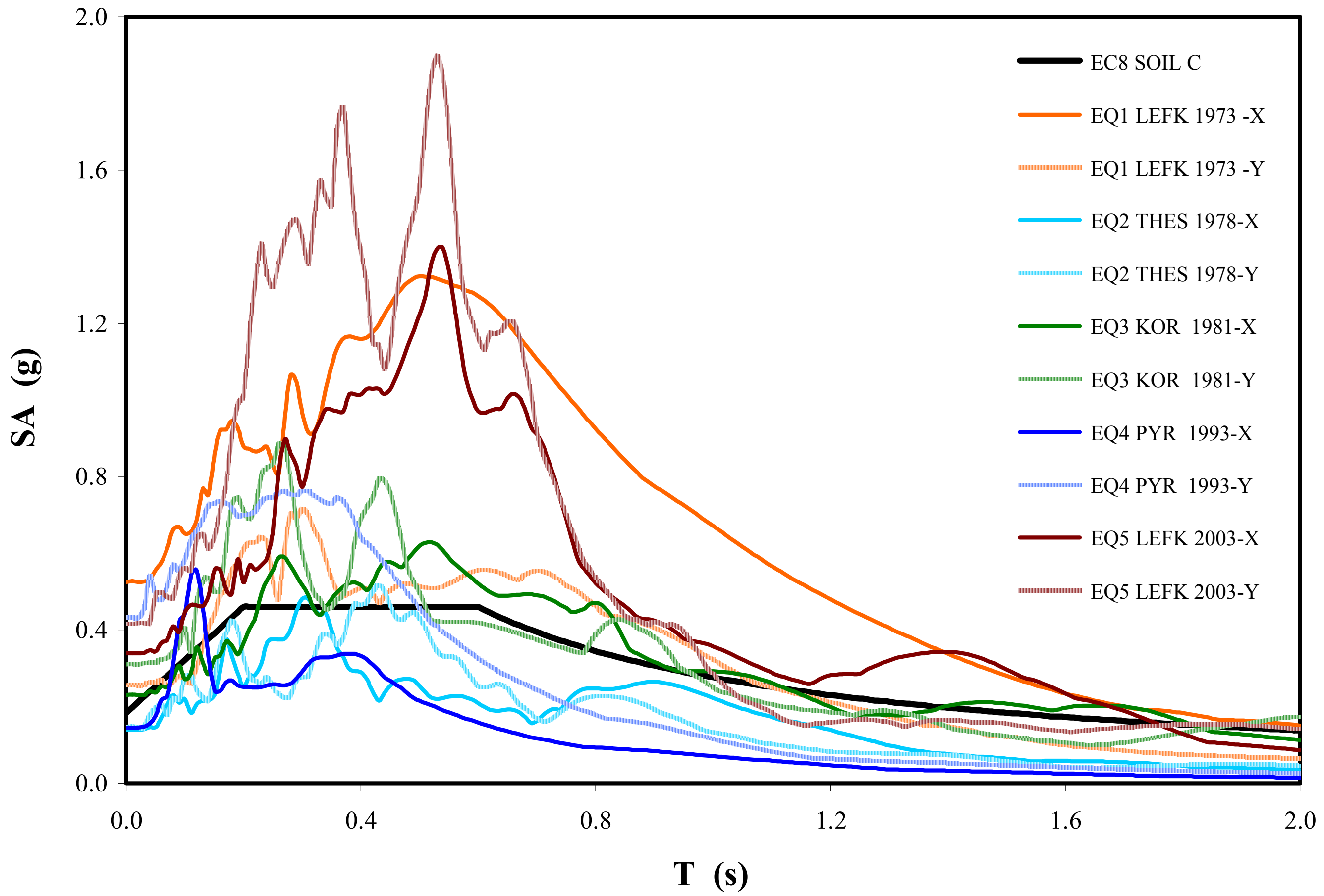


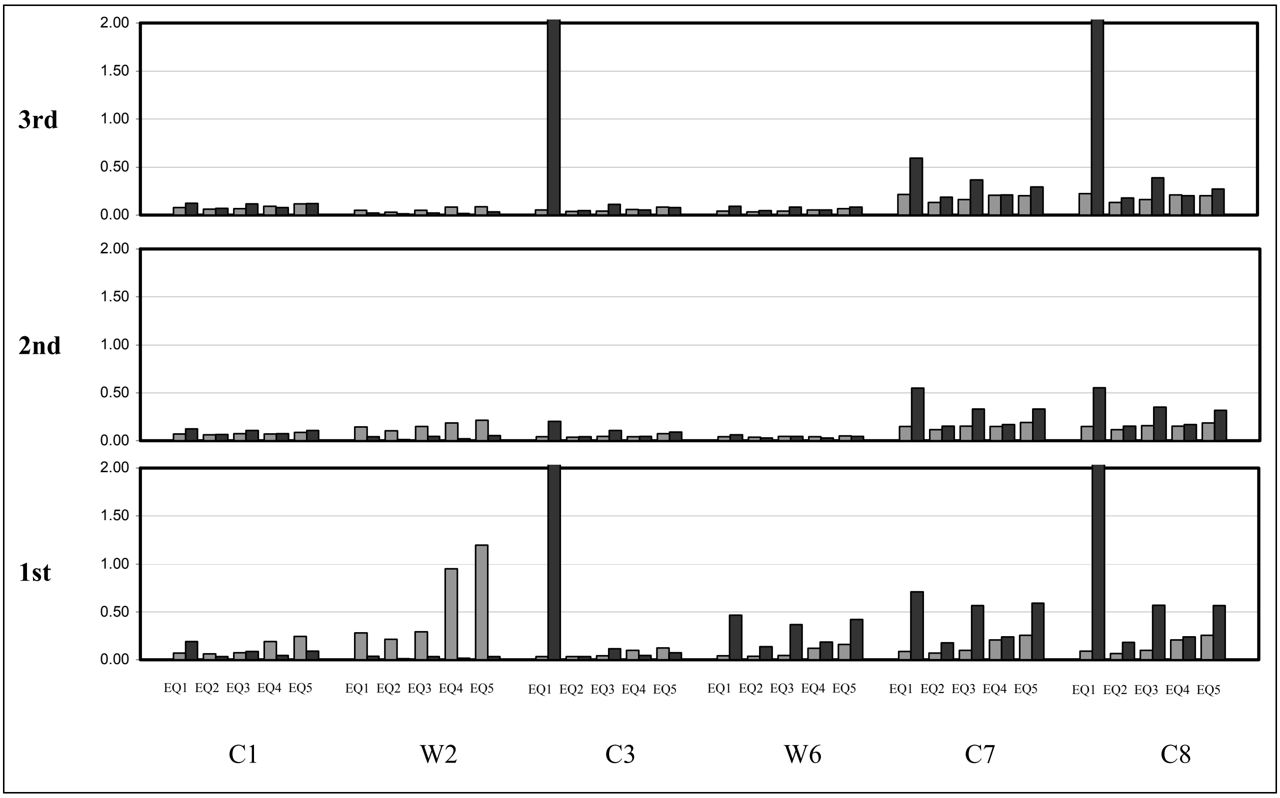
Figure 2. Response spectra of the set of seismic excitations of Table 1, used in the analyses.

Table 3. Periods (s) of 3-storey structures with *medium* member stiffness: fixed based vs elastically supported models

Mode	3CstE	3Cst	3SWstE	3SWst	3BWstE	3BWst
1	0.59	0.57	0.44	0.39	0.39	0.28
2	0.58	0.56	0.43	0.38	0.36	0.28
3	0.19	0.18	0.11	0.09	0.09	0.06
4	0.18	0.18	0.11	0.09	0.09	0.06

Table 4. Periods (s) of 3-storey structures with *small* member stiffness: fixed based vs elastically supported models

Mode	3CflE	3Cfl	3SWflE	3SWfl	3BWflE	3BWfl
1	0.87	0.85	0.62	0.59	0.51	0.43
2	0.84	0.83	0.61	0.57	0.48	0.42
3	0.28	0.28	0.15	0.14	0.12	0.09
4	0.27	0.26	0.15	0.13	0.11	0.09



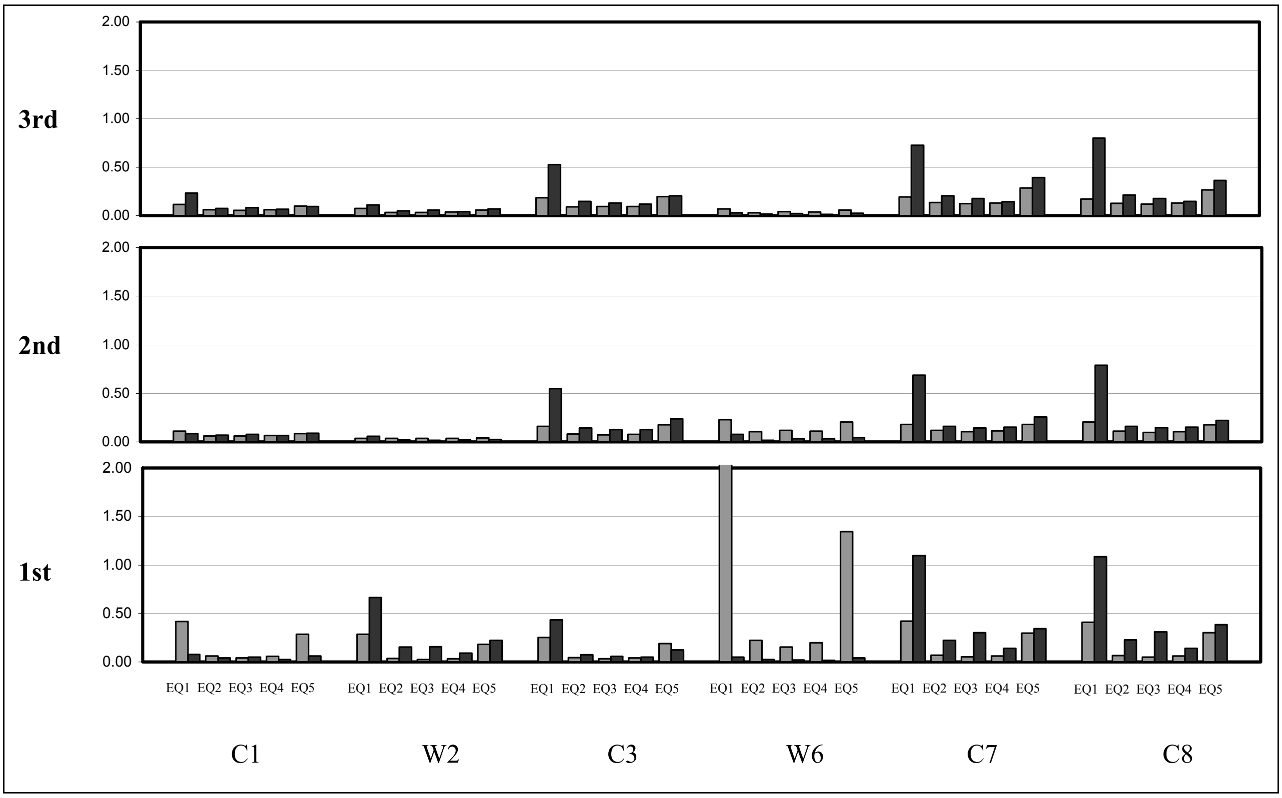


Figure 3, 4. Maximum ductility demand in the horizontal and perpendicular direction respectively of building 3BW, from ground motions EQ1-EQ5 on the bottom of column (C) and wall (W) elements, for each storey level, considering (dark grey) and not considering (light grey) SSI.

3.4 *Results*

A total of 12 models were investigated (Table 2), using three configurations of the structural system, two configurations for the member properties and two conditions regarding the foundation. Dynamic analysis considers the natural modes of the structure encompassing 100% of the total effective mass in each direction. The periods of the first 4 modes are presented in Tables 3 and 4 showing as expected and increase as the structure gets softer whether this is due to member stiffness or SSI. This increase is obviously more pronounced for the 3-storey structure with big structural walls, 3BW.

A series of elastic response spectrum analyses were performed that showed a redistribution of moments in all elements. This is again more pronounced in the initially stiffer models.

For instance for both the models 3SW and 3BW (buildings with structural walls) there is a high reduction of end moments at the foundation level on the structural walls on their longitudinal direction (strong axis) of the order of 70% combined with an increase of the order of 200% of the moments on their weak direction. Also there an increase by the same amount approximately on the end moments of the inner columns.

Regarding shear forces results show also a high redistribution showing similar decrease and increase of 50% and 200% respectively on the same members.

Finally differences in axial forces are present but these are not that pronounced with a decrease and increase factor of 20% and 25% respectively however combining this with changes in moment values may become critical.

Results from inelastic time-history analyses show more clear the effect of this redistribution of internal forces.

In Figures 3 and 4, maximum ductility demand in the horizontal and perpendicular direction respectively, of building 3BW with medium member stiffness, from ground motions EQ1-EQ5 on the bottom of column and wall elements, for each storey level, considering and not considering SSI is presented. It is obvious that for column elements, ductility demand becomes higher when SSI is considered; in one case there is even a failure of an inner column (column C8).

4 CONCLUSIONS

A comparative evaluation study is reported of the response of a group of 3-storey reinforced-concrete building structures designed according to EC8 and EC2 code provisions. Elastic and inelastic analyses were performed accounting or not for soil structure interaction.

The main conclusion is that despite the widespread rule of practice that soil-structure interaction is beneficial and that it is not necessary to be considered in the analysis, results show that this may not be the case. Despite the usually beneficial increase of the period of the structure that would lead to a lower seismic demand, the redistribution of internal member loading may increase the local seismic demand leading even to failures.

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