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The effects of construction practices on the seismic performance of RC frames with masonry infills

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Abstract. A number of construction practices, implemented during the design process of a reinforced concrete (RC) structural system, may have significant consequences on the behaviour of the structural system in the case of earthquake loading. Although a number of provisions are imposed by the contemporary Greek national design codes for the seismic design of RC structures, in order to reduce the consequences, the influence of the construction practices on the seismic behaviour of the structural system remains significant. The objective of this work is to perform a comparative study in order to examine the influence of three, often encountered, construction practices namely weak ground storey, short and floating columns and two combinations on the seismic performance of the structural system with respect to the structural capacity and the maximum interstorey drifts in three earthquake hazard levels.

Keywords: construction practice; Greek national design codes; capacity spectrum method; hazard levels.

1. Introduction

The behaviour of a building during an earthquake depends critically on its overall shape, size and geometry. A wide range of structural damages observed during past earthquakes across the world has been very educative in identifying construction features related to the shape, size and geometry of the structure that must be avoided. The earthquake forces developed at the storeys of a building need to be brought down to the ground by the shortest path; any deviation or discontinuity in this load transfer path results in poor performance. Buildings that have fewer columns or they are fully infilled in some of the storeys or they have partially infilled storeys tend to be more vulnerable to earthquake loading. Buildings with columns that hang or float on beams at an intermediate storey and do not extend up to the foundation show also poor structural performance against earthquake. A large number of multi-storey reinforced concrete (RC) buildings collapsed in past earthquakes, due to

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construction features like weak stories, short columns, strong beams-weak columns, large and heavy overhangs and others (Dolšek and Fajfar 2001, Ghobarah *et al.* 2006, Watanabe 1997, Mitchell *et al.* 1996).

Most of the RC buildings in Greece are up to four or six storeys, while the most usual construction practice is the weak ground storey. In a number of works (Lee and Woo 2002, Negro and Verzeletti 1996) it has been studied the effect of weak ground storeys on the seismic perfor-mance of RC frames, where it was recognized that the masonry infill contribute to the large increase in the stiffness and strength of the global structure whereas earthquake inertia forces are also increased. Short columns is a second construction practice that is often encountered in RC buildings, mainly industrial. Short columns at the ground storey of the structures are prone to brittle shear failure which may result in severe damages or even collapse because of the poor ductility during earthquakes (Li 2005, Guevara and Garcia 2005). Floating columns is the third construction practice that is examined in this work and they are mainly implemented in multi-storey buildings. It has been seen (Jain 2001) that RC buildings with floating columns are relatively vulnerable against earthquake.

The majority of the RC buildings are constructed with masonry infill walls. However, the combination of masonry infill with the framed structure is most often neglected during the design procedure, assuming that the contribution on the structural performance is always positive. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength and ductility of the structure. In the past a number of studies have been devoted on the seismic behaviour of RC frames with masonry infill (Madan *et al.* 1997, Perera *et al.* 2004, Dolšek and Fajfar 2005) and a number of analytical models for masonry infills have been developed for a rational approach of the behaviour of the masonry infills.

In this work a parametric study is performed with respect to the vertical layout of the building where a number of construction features are examined, in order to quantify their influence on the performance of the structure in three earthquake hazard levels (50, 10 and 2 percent in 50 years) versus the fully infilled structural design. In particular three construction practices often encountered in Greece are examined: (i) weak ground storey, (ii) short columns and (iii) floating or hanging columns, while two combinations of these three construction practices are also studied.

2. Construction practices

2.1 Weak ground storey

RC building structures have become very popular during the last decades in urban Greece. Many such buildings constructed in recent times have a special construction feature; the ground storey is left open for the purpose of parking. Such buildings are also called weak ground storey buildings, while the weak storey is also called soft storey or *pilotis*.

2.1.1 Behaviour of weak ground storey buildings

Weak ground storey buildings have shown poor performance during past earthquakes across the world (Dolšek and Fajfar 2001, Ghobarah *et al.* 2006); while a significant number of them have collapsed. The fully infilled upper storeys are much stiffer than the open ground storey. Thus, the upper storeys deform almost together, and the maximum interstorey drift occurs in the weak ground storey. Consequently, the columns in the open ground storey are severely stressed. If the columns do not have the required strength to resist or do not have adequate ductility, they might severely damaged which may lead to the collapse of the building. Summarizing a weak ground storey building has two characteristics: (i) The maximum interstorey drift is encountered on the ground storey. (ii) The total horizontal earthquake force caring capacity of the ground storey is significantly smaller than that of the fully infilled storeys above.

2.1.2 Seismic design provisions

The Greek national seismic code (EAK 2000) imposes special design provisions in the case of weak ground storey. In particular, EAK 2000 specifies higher design forces for the open ground storey compared to the rest of the storeys. Beams and columns in the open ground storey are required to be designed for 2.5 times the forces obtained from the case of a fully infilled storey. Additional design provisions are considered for avoiding the formation of floor mechanism, for this reason the strong column-weak beam concept (Paulay 1986, Priestley and Calvi 1991) is imposed in order to avoid the formation of plastic hinges at both ends of the columns in the same storey. This concept is considered in EAK 2000 through the capacity design where it is stated that the sum of the resisting moments of the columns, taking into account the axial forces, should be greater than the sum of the resisting moments of all adjacent beams for each (positive or negative direction of the earthquake action). Therefore, the capacity design is satisfied if the columns are designed for the following moment

$$M_{CD,c} = \alpha_{CD} M_{Ec} \tag{1}$$

$$\alpha_{CD} = \gamma_{Rd} \frac{\left|\sum M_{Rd}\right|}{\left|\sum M_{Eb}\right|} \tag{2}$$

where a_{CD} is the magnification factor of the node, ΣM_{Rd} is the sum of the resisting moments of the adjacent beams, ΣM_{Ed} is the sum of the resisting moments of the columns, $\gamma_{Rd} = 1.40$ is the overstrength factor and M_{Ec} is the maximum moment of the column taken from the combination

$$S_d = G_k \pm E_d + \sum \psi_{2,\,i} Q_{k,\,i} \tag{3}$$

where G_k denotes the characteristic value of the permanent action, E_d is the design value of the seismic action, $Q_{k,i}$ refers to the characteristic value of the variable action i and $\psi_{2,i}$ is the combination coefficient for the quasi-permanent action i, here taken equal to 0.30.

2.2 Short columns

During past earthquakes, RC buildings having short columns suffered from damages (Watanabe 1997, Mitchell *et al* 1996), due to the concentration of large shear forces. The short columns are stiffer compared to the regular size columns attracting larger earthquake forces. If a short column is not adequately designed for such a large force, it can suffer significant damage during an earthquake. Short columns are characterized by a small value of the shear span ration α

$$\alpha = \frac{M_{sd}}{V_{sd} h} \leq 2.5 \tag{4}$$

where M_{sd} and V_{sd} are the maximum moment and shear force values obtained from the combination of Eq. (4) while h is the column depth.

2.2.1 Behaviour of buildings with short columns

Generally speaking the failure modes of short columns can be classified into two cases: (i) shear failure, occurs when $\alpha \leq 1.50$ and (ii) sliding failure when $1.5\text{-}2.0 \leq 2.5$. According to the ultimate strength model developed by Tegos and Penelis (1988) the total shear capacity of a short column is estimated by superposition of the three partial mechanisms: (i) truss mechanism, (ii) rhombic truss mechanism of the inclined bars and (iii) compression parallelogram formed by the compressive stress part in concrete.

2.2.2 Seismic design provisions

Short columns should be avoided, to the extent possible, when it is not possible this construction feature should be addressed in design stage. The Greek national design code requires that: (i) the entire height of the column to be considered as a critical region. (ii) special confining reinforcement to be provided over the full height of columns that are likely to sustain short column effect. (iii) When $\alpha \leq 1.5$ bidiagonal reinforcement and ties should be added to the longitudinal and the transverse reinforcement.

2.3 Floating columns

Floating columns is also an often encountered construction practice, that it should be avoided because it leads to the overload of the beams. The joists between beams and floating columns are considered as particularly critical since their stability influence the general stability of the building. The Greek national design codes do not prohibit the implementation of floating columns but if however they are used, the vertical component of earthquake should also be considered.

3. Capacity spectrum method

The purpose of the Nonlinear Static Procedure (NSP), is to assess structural performance in terms of strength and deformation capacity globally as well as in element level. The procedure is terminated as soon as a collapse mechanism is formed or a predefined target displacement is reached. In order to determine the target displacement two alternative procedures have been proposed, the Capacity Spectrum Method (CSM) (Freeman 1998) and the method of FEMA-273 (1997). CSM compares the capacity of a structure to resist lateral forces with the demands of seismic excitation in a graphical representation allowing a visual evaluation of how the structure will perform when subjected to the ground motion represented by the response spectrum used. Therefore, the response spectrum represents the demand capacity while the capacity curve represents the available supply of bearing capacity of the structure. Both curves are converted and plotted against an acceleration-displacement graph making easy the evaluation of the point of equal demand and capacity, also known as performance point.

4. Definition of seismic response spectra

The most common approach for the definition of the seismic action is the use of design code response spectrum. This is a general approach which is easy to implement. However if a more realistic design is required the use of spectra derived from natural earthquake records is more appropriate. Based on a previous work of the first author (Lagaros *et al.* 2006) three sets of natural records, i.e. sets with their longitudinal and transverse components, are used. These records have been selected from the database of Somerville and Collins (2002). The basic characteristics of these records are provided in Tables 1, 2 and 3 corresponding to the three hazard levels, 50, 10 and 2 percent in 50 years, respectively. It can be seen that each record corresponds to different earthquake magnitudes and soil properties. The records are scaled to the same PGA in order to ensure compatibility between the records, according to the hazard curves for Greece obtained from the work of Papazachos *et al.* (1993) (Table 4). The response spectrum for each scaled record and for each component is shown in Figs. 1, 2 and 3 for the three hazard levels. It has been observed that the response spectra follow the lognormal distribution (Chintanapakdee and Chopra 2003). Therefore the median spectra, also shown in Figs. 1, 2 and 3, are calculated from the above set of spectra using the following expression

$$\hat{x} = \exp\left[\frac{\sum_{i=1}^{n} \ln(R_{d,i}(T))}{n}\right] \tag{5}$$

where $R_{d,i}(T)$ is the response spectrum value for period equal to T of the i-th record ($i = 1,\ldots, n$). $n = 22$ for the 50% in 50 years hazard level, $n = 19$ for the 10% in 50 years and $n = 6$ for the 2% in 50 years hazard level.

Table 1 Natural records representing the 50% in 50 year hazard level

Earthquake	Station	Distance	Site
Honeydew (PT) 17 August 1991	Cape Mendocino	20	rock
	Petrolia	17	soil
Cape Mendocino (CM) 25 April 1992	Bunker Hill	8.8	rock
	Butler Valley	37	rock
	Centerville	16	soil
	Eureka College	21	soil
	Eureka School	24	soil
	Ferndale	14	soil
	Fortuna	13	soil
	Loleta	17	soil
	Rio Dell	13	soil
Cape Mendocino (C1) aftershock, 26 April 1992 0741GMT	Bunker Hill	27	rock
	Centerville	27	soil
	Eureka College	46	soil
	Eureka School	48	soil
	Ferndale	34	soil
	Fortuna	43	soil
	Loleta	41	soil
Cape Mendocino (C2) aftershock, 4/26/92 1118GMT	Bunker Hill	27	rock
	Centerville	28	soil
	Ferndale	34	soil
	Fortuna	43	soil

Table 2 Natural records representing the 10% in 50 year hazard level

Earthquake	Station	Distance	Site
Tabas (TB) 16 September 1978	Dayhook	14	rock
	Tabas	1.1	rock
Cape Mendocino (CM) 25 April 1992	Cape Mendocino	6.9	rock
	Petrolia	8.1	soil
Chi-Chi (CC), Taiwan 20 September 1999	TCU052	1.4	soil
	TCU065	5.0	soil
	TCU067	2.4	soil
	TCU068	0.2	soil
	TCU071	2.9	soil
	TCU072	5.9	soil
	TCU074	12.2	soil
	TCU075	5.6	soil
	TCU076	5.1	soil
	TCU078	6.9	soil
	TCU079	9.3	soil
	TCU089	7.0	rock
	TCU101	4.9	soil
	TCU102	3.8	soil
	TCU129	3.9	soil

Table 3 Natural records representing the 2% in 50 year hazard level

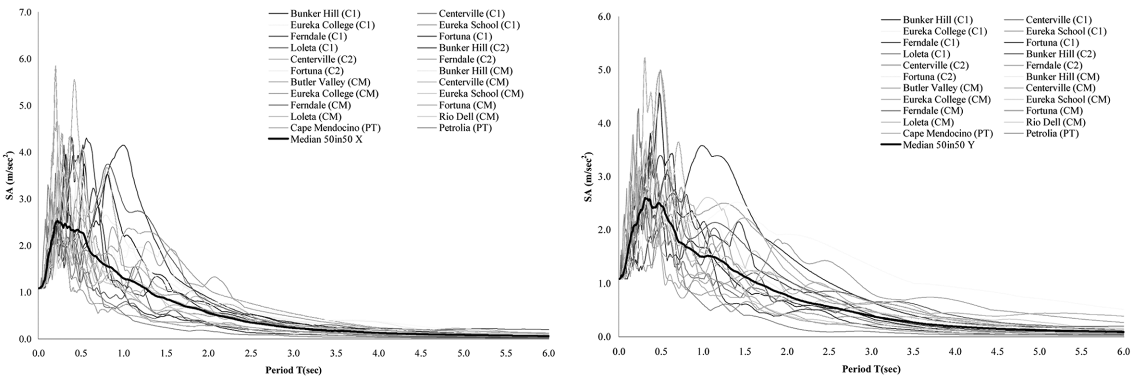
Earthquake	Station	Distance	Site
Valparaiso (VL), Chile 3 May 1985	Vina del Mar	30	soil
	Zapaller	30	rock
Michoacan (MI), Mexico 19 September 1985	Caleta de Campos	12	rock
	La Union	22	rock
	La Villita	18	rock
	Zihuatenejo	21	rock

Table 4 Seismic hazard levels (Papazachos *et al.* 1993)

Event	Recurrence Interval	Probability of Exceedance	PGA (g)
Frequent	21 years	90% in 50 years	0.06
Occasional	72 years	50% in 50 years	0.11
Rare	475 years	10% in 50 years	0.31
Very Rare	2475 years	2% in 50 years	0.78

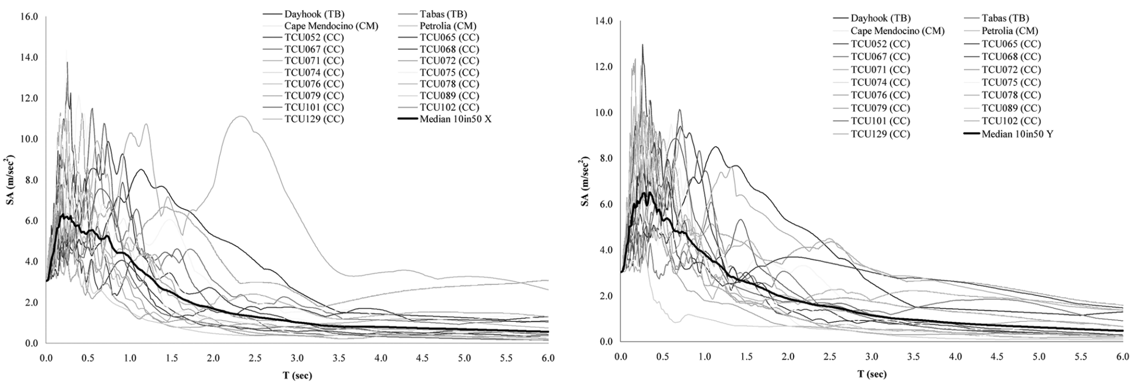
5. Description of the models

The 3D RC building having four storeys, shown in Fig. 4, is considered for the parametric study performed in this work. The cross section for all columns is 45×45 cm^2 and 30×60 cm^2 for all



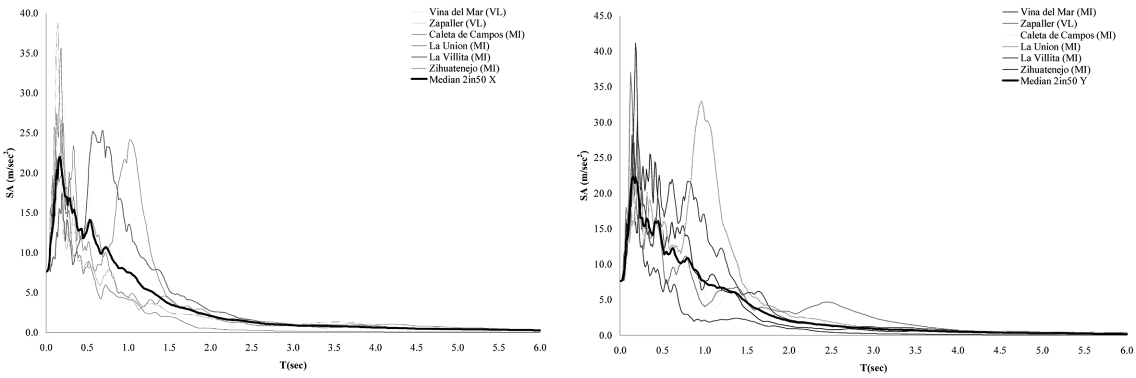
(a) (b)

Fig. 1 Natural records response spectra and their median (Hazard level/component) (a) 50in50/Longitudinal(x), (b) 50in50/Transverse(y)



(a) (b)

Fig. 2 Natural records response spectra and their median (Hazard level/component) (a) 10in50/Longitudinal(x), (b) 10in50/Transverse(y)



(a) (b)

Fig. 3 Natural records response spectra and their median (Hazard level/component) (a) 2in50/Longitudinal(x), (b) 2in50/Transverse(y)

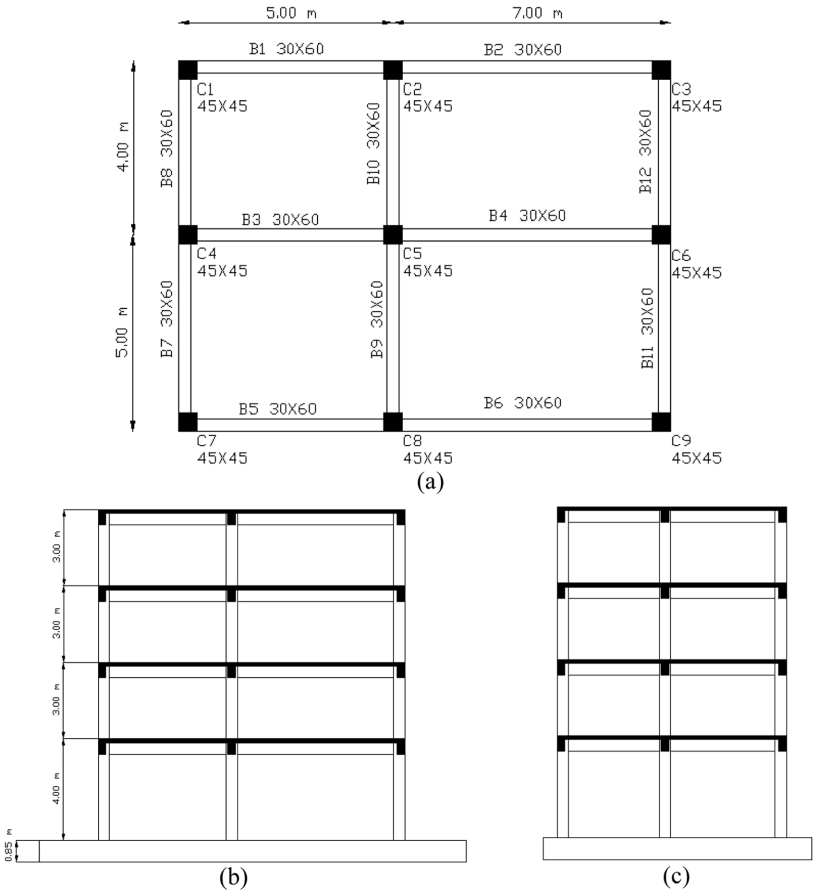


Fig. 4 Geometry of the three storey 3D building (a) layout, (b) front view, (c) side view

beams. The lateral design forces were derived from the design response spectrum i.e., 5%-damped elastic spectrum divided by the behaviour factor q at the fundamental period of the building. Concrete of class C16/20 (nominal cylindrical strength of 16 MPa) and class S500 steel (nominal yield stress of 500 MPa) are assumed. The infill walls consist of 30 cm × 20 cm × 15 cm horizontally perforated bricks with compressive strength equal to 3.0 MPa and modulus of elasticity equal to 2,250 MPa. The base shear is obtained from the response spectrum for soil type A (stiff soil $\theta = 1$, with characteristic periods $T_1 = 0.10$s and $T_2 = 0.40$s) and PGA equal to 0.31 g, corresponding to zone III for the 10/50 hazard level. Greece is divided into three zones of equal seismic hazard. Papazachos *et al.* (1993) have presented a semi-probabilistic approach to the seismic hazard assessment of Greece resulting to the hazard curves for all zones. The city of Athens which is the location where the six designs will be built belong to zone III. Moreover, the importance factor γ_I was taken equal to 1 (importance category Σ2), while damping correction factor is equal to 1.0, since a damping ratio of 5% has been considered (as it is suggested by EAK 2000 for RC structures) while a behaviour factor q equal to 3.5 is considered. In ELSA laboratory (Negro and Verzeletti 1996, Negro and Colombo 1997) models, similar to the ones examined in this study, have been tested.

The slab thickness is equal to 15 cm and is considered to contribute to the moment of inertia of the beams with an effective flange width. In addition to the self weight of the beams and the slab, a distributed dead load of 2 kN/m^2 due to floor finishing and partitions is considered, while live load with nominal value of 1.5 kN/m^2 is also imposed. In the combination of gravity loads ("persistent design situation") nominal dead and live loads are multiplied with load factors of 1.35 and 1.5, respectively. Following EAK 2000, in the seismic design combination, dead loads are considered with their nominal value while live loads with 30% of the nominal value. EAK 2000 belongs to the category of the prescriptive building design codes versus the relatively new concept of Performance-Based Design (PBD) for structures (FEMA-273 1997, Marano *et al.* 2007).

The model employed in this study for simulating the masonry infill panels is based on the one developed by Perera *et al.* (2004). According to this model the contribution of the masonry infill panel to the response of the infilled frame is modeled by a system of two diagonal masonry compression struts. The two struts are considered ineffective in tension since the tensile strength of masonry is negligible. The combination of both diagonal struts provides the lateral load resisting mechanism for the opposite lateral directions of loading. Each strut element is modeled as a simple longitudinal inelastic spring whose behaviour is described in terms of the axial force-axial deformation relation of the strut using the notion and principles of continuum damage mechanics. In the work by Perera *et al.* (2004) the following relation was obtained

$$N = K_0(1-d)\delta^e = K_0(1-d)(\delta - \delta^p) \tag{6}$$

where N is the axial force of the strut, δ, δ^e and δ^p are the total, elastic and plastic shortenings of the strut, respectively. K_0 is the initial stiffness before cracking and d is the internal damage variable representing the degradation of the infill. More details about the damage model of the masonry infill used in this work can be found in the work by Perera *et al.* (2004).

For all test cases, a centreline model was formed based on the OpenSEES (McKenna and Fenves 2001) simulation platform. The members are modelled using the force-based fiber beam-column element while the same material properties are used for all the members of the six structures. Soil-structure interaction was not considered and the base of the columns at the ground floor is assumed to be fixed.

In this work six different test cases have been examined: (a) fully infilled model that corresponds to the design where all circumferential frames in all storeys are considered fully infilled (see model of Fig. 5(a)), (b) weak ground storey model, where no masonry infill are present in the ground storey (see model of Fig. 5(b)), (c) short columns model, where transverse frames C3-B12-C6-B11-C9 and C1-B8-C4-B7-C7 are fully infilled (see model of Fig. 5(a)) and longitudinal frames C1-B1-C2-B2-C3 and C7-B5-C8-B6-C9 are partially infilled in the ground level (see model of Fig. 5(c)), (d) floating columns model, where floating columns are considered in the second and third storey of frames C1-B1-C2-B2-C3 and C7-B5-C8-B6-C9 (see model of Fig. 5(d)) while frames C1-B8-C4-B7-C7 and C3-B12-C6-B11-C9 are fully infilled (see model of Fig. 5(a)), (e) combination A model, which is produced combining the two models ii, iv (see model of Fig. 5(e)) and (f) combination B model, produced combining the two models iii, iv (see model of Fig. 5(f)). All models were designed to meet the EKOS 2000 and EAK 2000 requirements, implementing all the provisions suggested by the codes in order to alleviate the effect of the construction practices on the structural performance. The weight of the steel reinforcement and the concrete volume required for the six models are given in Table 5. As it can be seen the concrete volume is the same since the cross

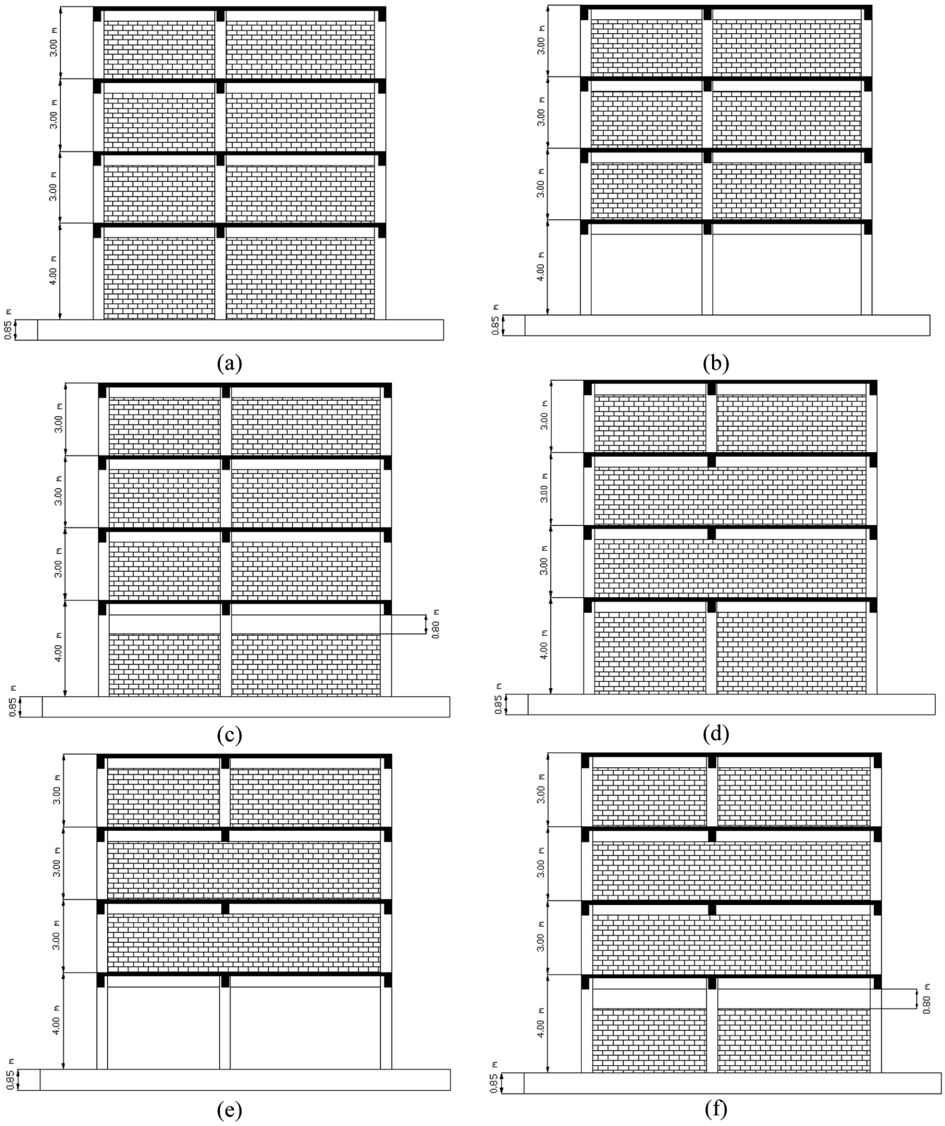


Fig. 5 The models for the C7-B5-C8-B6-C9 frame (a) fully infilled, (b) weak ground storey, (c) short columns, (d) floating columns, (e) combination A, (f) combination B

Table 5 Comparison of steel and concrete quantities in the six models

Model	Columns		Beams	
	Steel (kg.)	Concrete (m^3)	Steel (kg.)	Concrete (m^3)
Fully infilled	5868	23.7	5495	39.5
Weak ground storey	6516	23.7	5794	39.5
Short columns	6210	23.7	5495	39.5
Floating columns	5544	23.7	5539	39.5
Combination A	6408	23.7	6016	39.5
Combination B	5382	23.7	5950	39.5

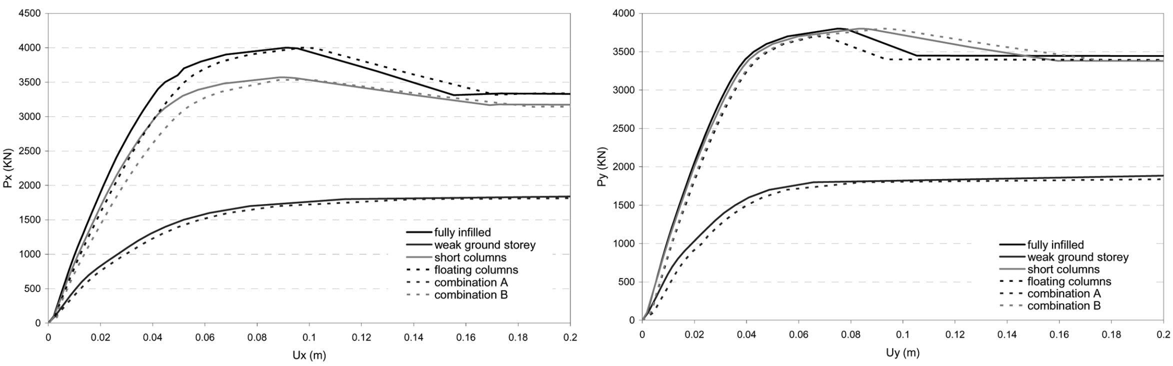
section for all columns and beams is equal to 45 × 45 cm^2 and 30 × 60 cm^2, respectively, all for all six designs. The difference on the reinforcement of the columns and the beams is due to the implementation of the code provisions for the various construction practices. As it can be seen from Table 5 the fully infilled design requires less reinforcement than any other design required, while the weak ground storey design requires the most reinforcement.

6. Numerical study

The parametric study resulted in six different designs depending on the models described in the previous section. The main objective of this work is to assess the performance of various construction practices in three earthquake hazard levels. This is achieved by comparing the six designs with respect to their capacity and the maximum interstorey drift in three earthquake hazard levels, in particular the 50, 10 and 2 percent in 50 years hazard levels. Capacity spectrum method is used for calculating the performance point E(Sa, Sd) for each model. The performance point *E* represents the largest required plastic deformation of the top storey for the hazard level in question.

Figs. 6(a) and 6(b) depict the capacity curves for each model in lognitudinal-x and transverse-y directions, respectively. Pushover is limited with regard to evaluation of the simultaneous response to ground shaking in different directions. In this work the recommendation of FEMA-350 is employed where multidirectional excitation effects are accounted for by combining 100% of the response due to loading in the longitudinal direction with 30% of the response due to loading in the transverse direction, and by combining 30% of the response in the longitudinal direction with 100% of the response in the transverse direction.

It can be seen that the curves for both directions can be classified in three groups. The capacity curves of weak ground storey and combination A designs belong to the first group having the poorest performance. The capacity curves of short columns and combination B designs belong to the second group with the second poorest performance, while fully infilled and floating columns designs belong to the third group with the best performance. In the longitudinal direction x the capacity of the designs of the first group is close to 1,800 kN while those of the second and third



(a) (b)

Fig. 6 Capacity curves: (a) Longitudinal-x, (b) Transverse-y groups is close to 3,600 and 4,100 kN, respectively. Due to the strength degradation in the infill walls the overall capacity of the second and third groups of designs is reduced to 3,200 kN. The trend is different in the transverse direction y where the capacity of the designs of the first group remains close to the 1,800 kN while those of both second and third groups is close to 4,000 kN. Accordingly, due to the strength degradation in the infill walls, the overall capacity of the second and third groups of designs is reduced to 3,500 kN. The performance of the designs of second and third groups is the same due to the same configuration of the models of the two groups in this direction. It can be observed that the presence of the weak ground storey in the two designs of the first group reduces the capacity of the structure by almost 50%. On the other hand the

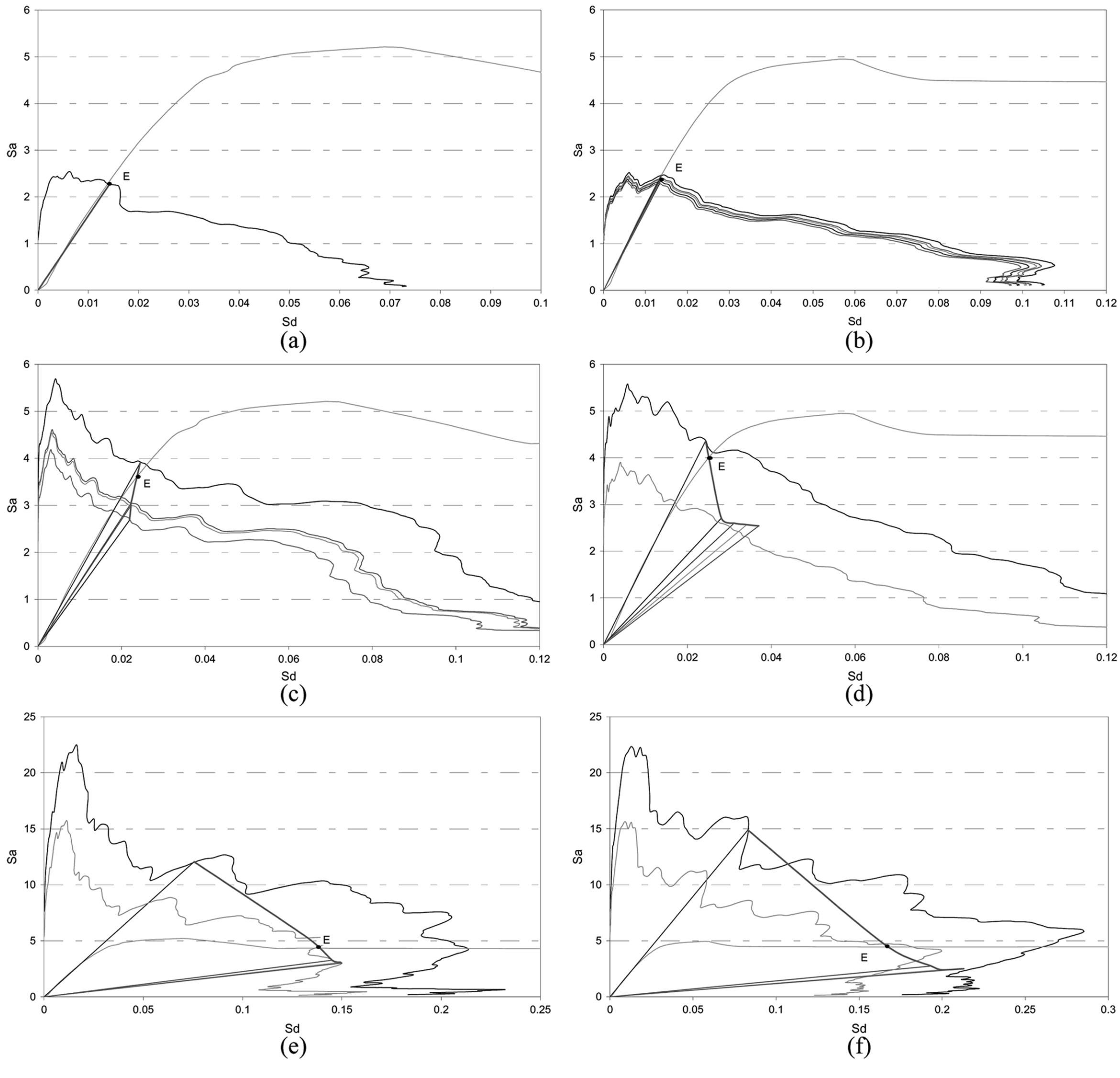


Fig. 7 Fully infilled model - Demand spectra versus capacity curves: 50/50 (a) Longitudinal(x), (b) Transverse(y), 10/50 (c) Longitudinal(x), (d) Transverse(y), 2/50 (e) Longitudinal(x), (f) Transverse(y)

structural capacity of the designs of the second group drops by 10% in the longitudinal direction compared to the designs of the first group due to the existence of short columns in this direction only.

Figs. 7 to 12 depict the capacity curves and the calculated performance points for the six models in each hazard level in the longitudinal and transverse directions where both response spectrum and capacity curves were converted and plotted against an acceleration-displacement graph. In particular the steps of the CSM are depicted in order to define the target displacement and then calculate the maximum interstorey drift for this target displacement. As it can be seen from Figs. 7(a) and 7(b),

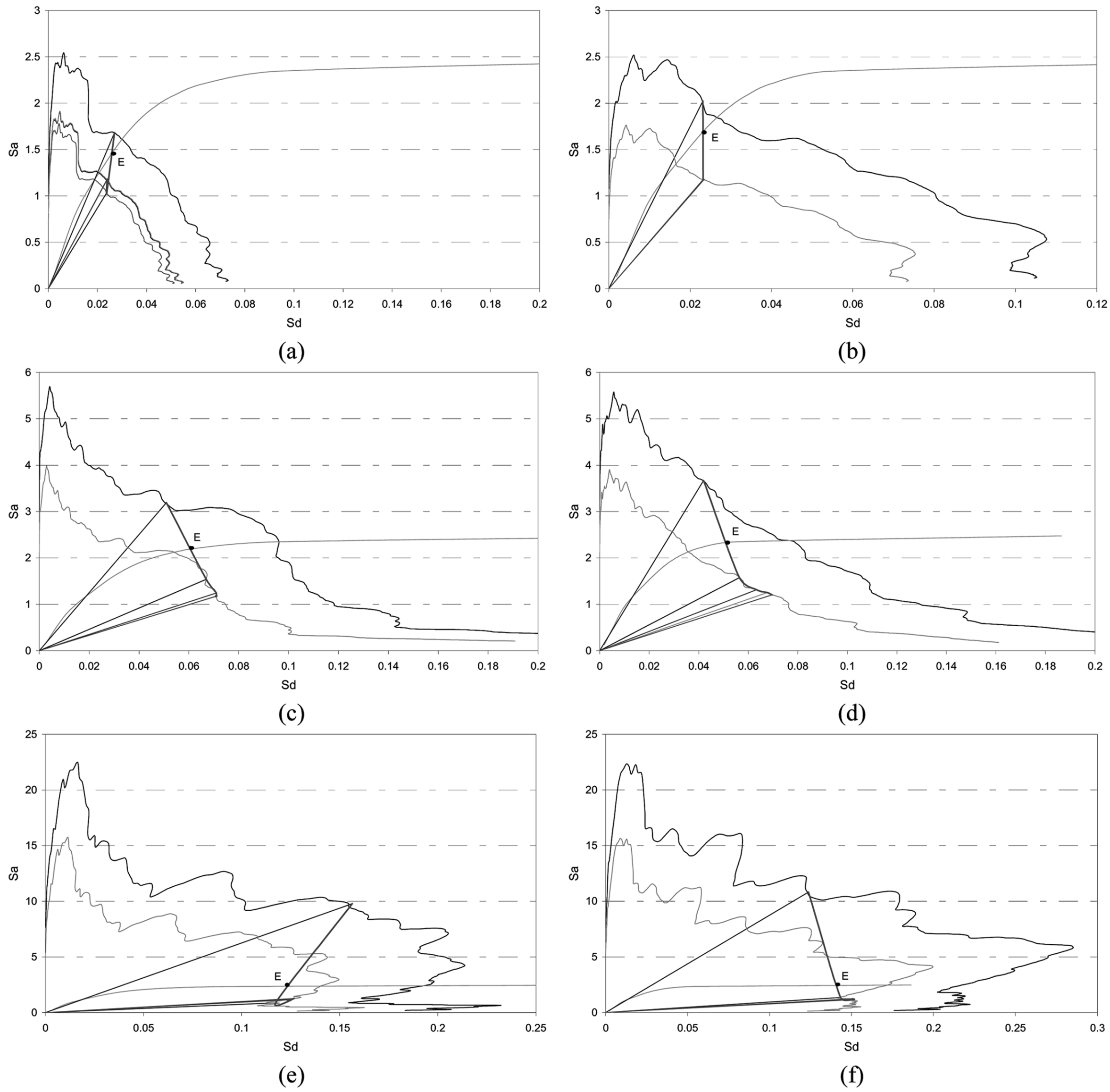


Fig. 8 Weak ground storey model - Demand spectra versus capacity curves: 50/50 (a) Longitudinal(x), (b) Transverse(y), 10/50 (c) Longitudinal(x), (d) Transverse(y), 2/50 (e) Longitudinal(x), (f) Transverse(y)

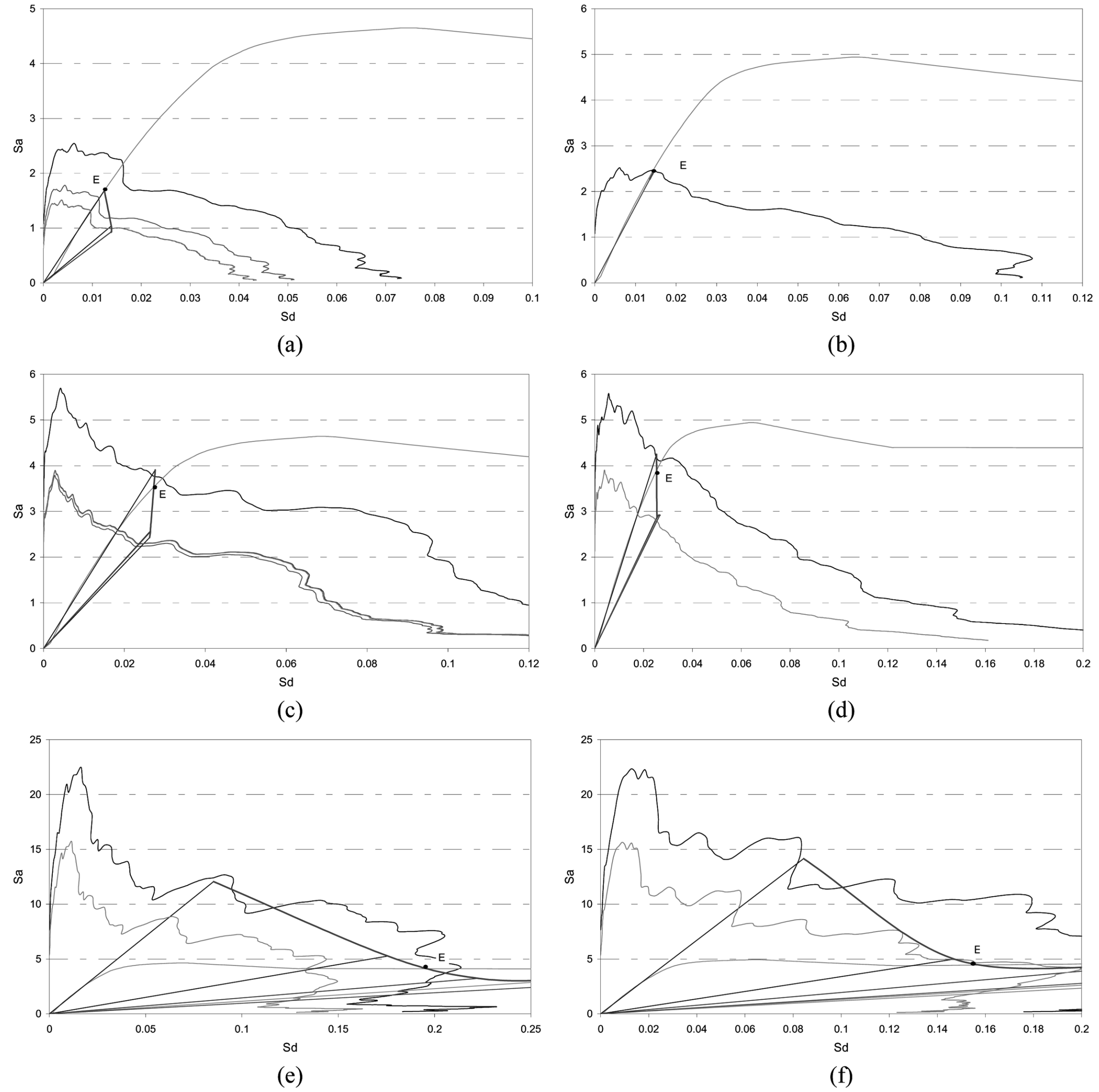


Fig. 9 Short columns model - Demand spectra versus capacity curves: 50/50 (a) Longitudinal(x), (b) Transverse(y), 10/50 (c) Longitudinal(x), (d) Transverse(y), 2/50 (e) Longitudinal(x), (f) Transverse(y)

the fully infilled design behaves elastically for the 50% in 50 years hazard level both in longitudinal and transverse directions. Figs. 7(c), 7(d), and 7(e), 7(f) depict the performance points for the 10% and 2% in 50 years hazard levels in both longitudinal and transverse directions for the fully infilled design, respectively. Correspondingly, in Figs. 8 to 12 the performance points for the other five models are provided. Worth mentioning that in the cases where the strength degradation of the infill walls have significant influence on the structural performance (i.e., full infilled, short columns, floating columns and combination B) performance points for the 50/50 and 10/50 hazard levels are

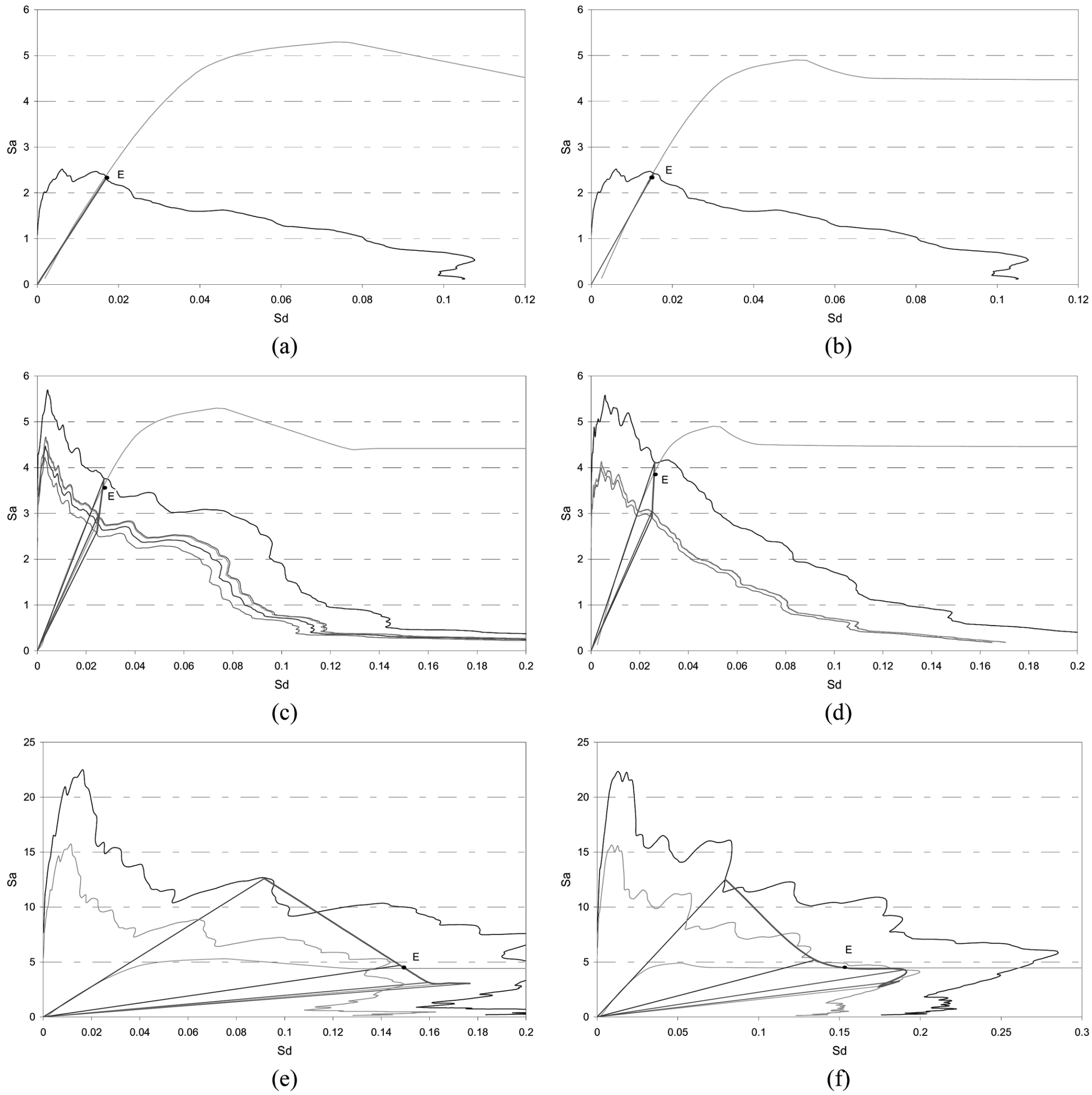


Fig. 10 Floating columns model - Demand spectra versus capacity curves: 50/50 (a) Longitudinal(x), (b) Transverse(y), 10/50 (c) Longitudinal(x), (d) Transverse(y), 2/50 (e) Longitudinal(x), (f) Transverse(y)

encountered before strength degradation while the performance point for the 2/50 is encountered after strength degradation. All the performance points are presented graphically in Figs. 13(a) and 13(b) for the both directions.

Figs. 14(a) to 14(c) show the maximum interstorey drift distribution along the height of the building for the three hazard levels. The drift limits, $\Delta_{tar} = 0.4\%$ for the 50% in 50 years hazard level, $\Delta_{tar} = 1.8\%$ for the 10% in 50 years hazard level and $\Delta_{tar} = 3.0\%$ for the 2% in 50 years hazard level. These drift limit values are based on the work of Ghoborah (2004) for ductile RC moment resisting

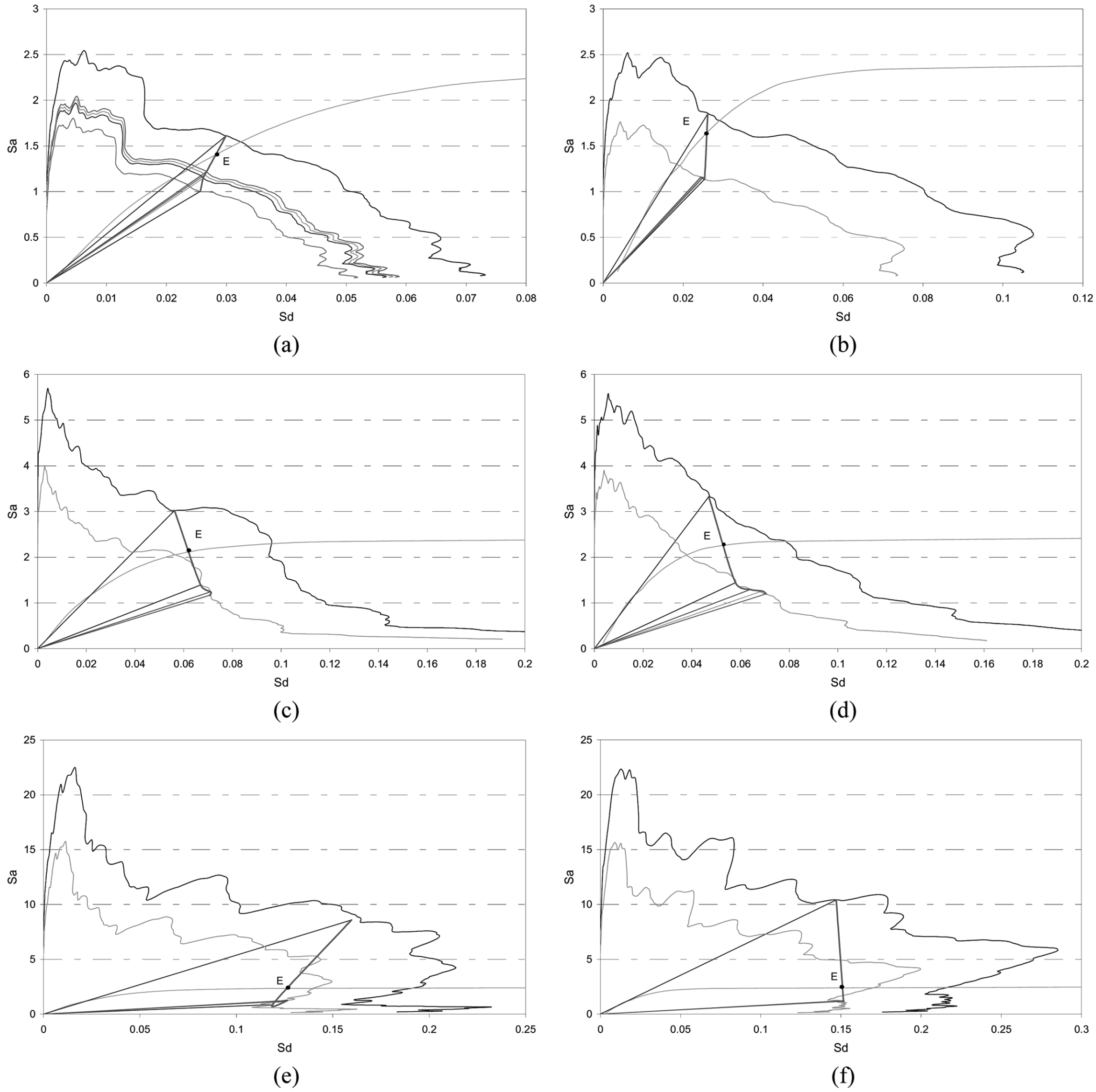


Fig. 11 Combination A model - Demand spectra versus capacity curves: 50/50 (a) Longitudinal(x), (b) Transverse(y), 10/50 (c) Longitudinal(x), (d) Transverse(y), 2/50 (e) Longitudinal(x), (f) Transverse(y)

frames and they are shown with a gay bold line in Figs. 14(a) to 14(c). For the case of a RC moment resisting frames with infill walls the drift limits are 0.2%, 0.7% and 0.8% for the three hazard levels (Ghobarah 2004), respectively, which are shown with a black bold line in Figs. 14(a) to 14(c). As it can be seen all weak ground storey and short columns designs exceed significantly the drift limits in all three hazard levels, on the other hand the fully infilled and floating columns designs fulfill the drift limit requirements, i.e., the MRF with infills drift limits, for the 50 and 10 percent hazard levels.

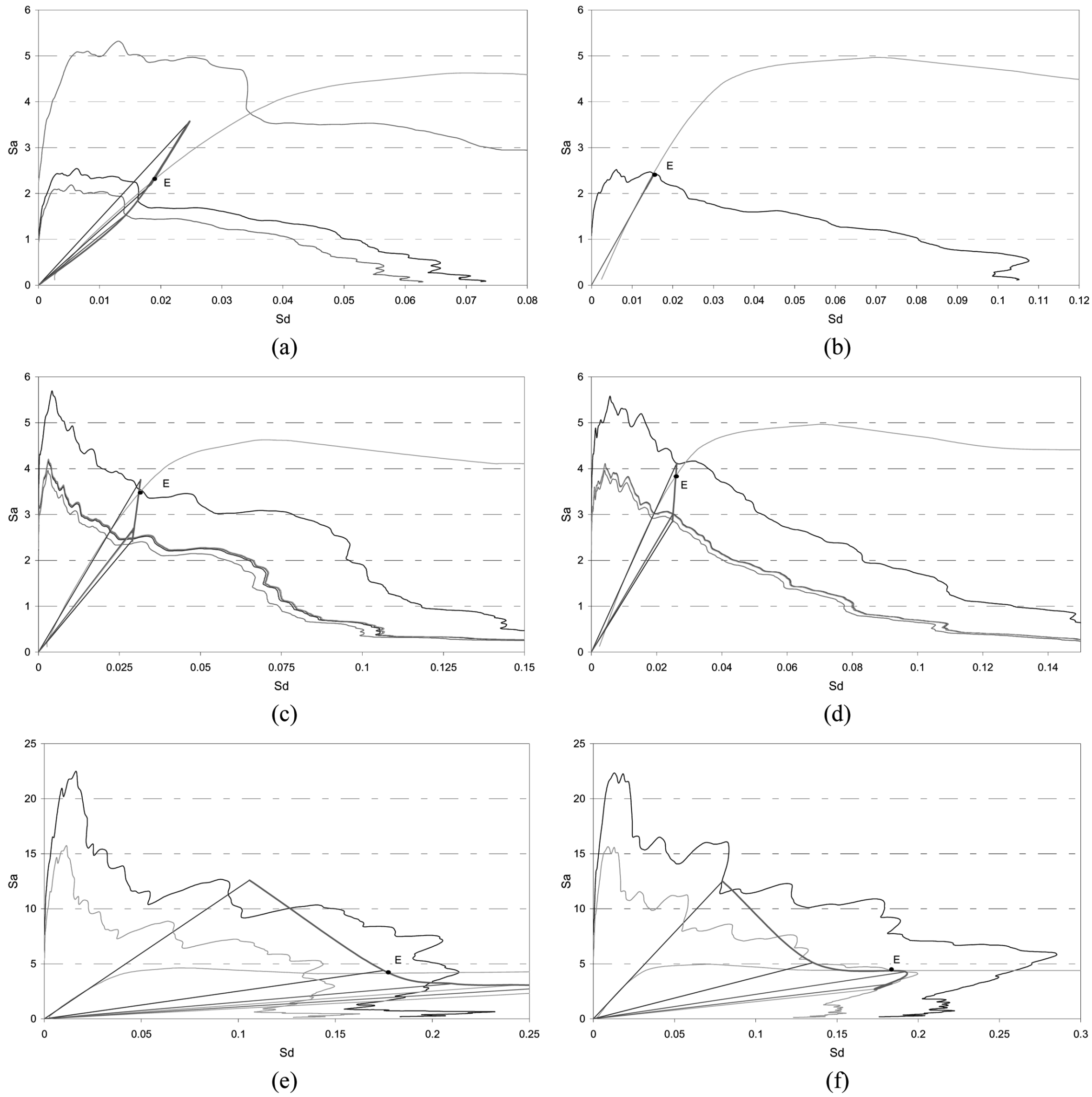
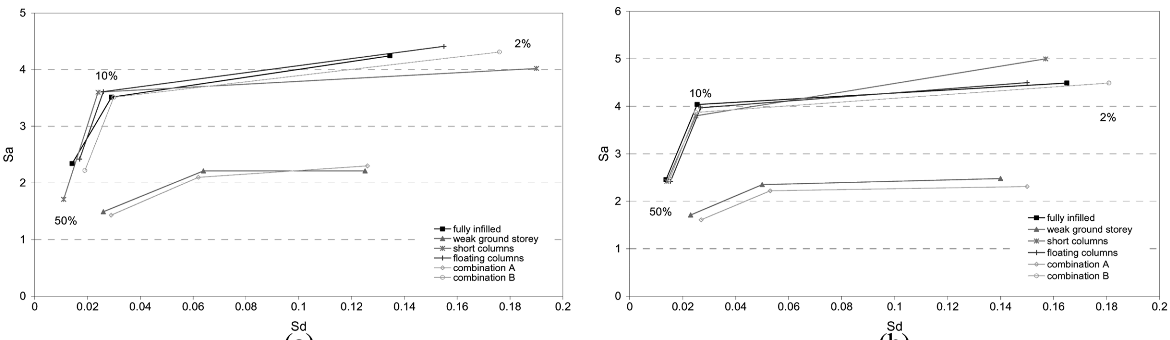


Fig. 12 Combination B model - Demand spectra versus capacity curves: 50/50 (a) Longitudinal(x), (b) Transverse(y), 10/50 (c) Longitudinal(x), (d) Transverse(y), 2/50 (e) Longitudinal(x), (f) Transverse(y)

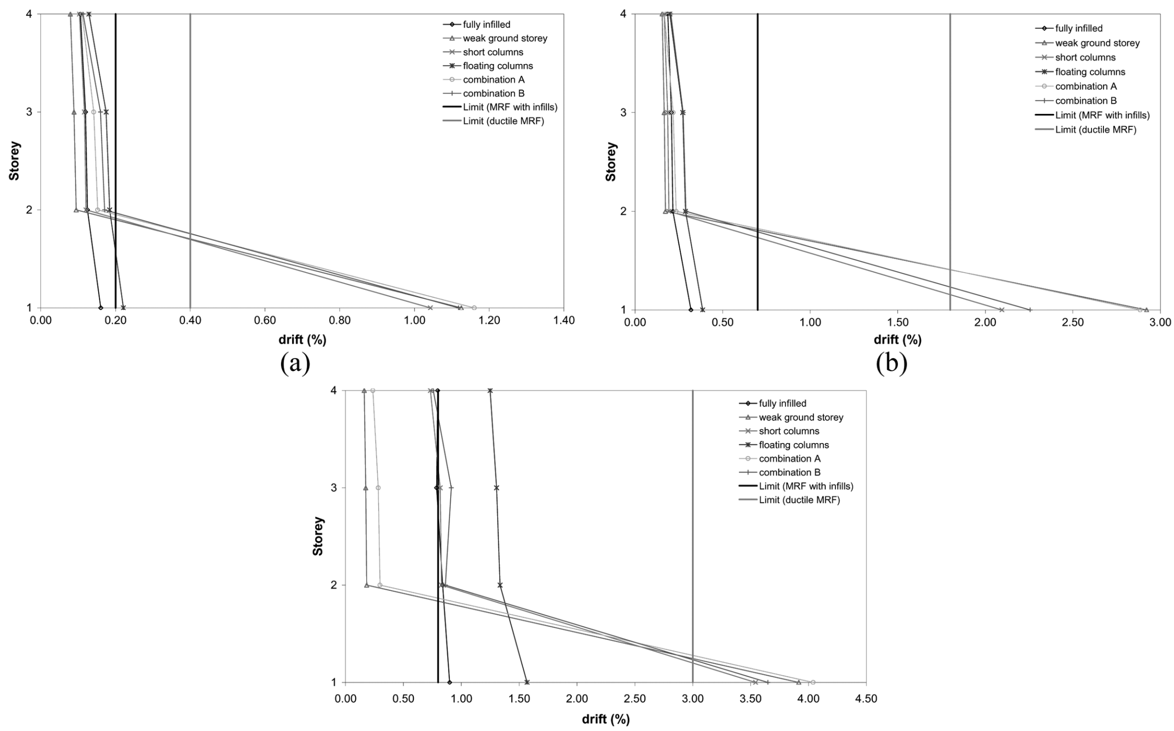
7. Conclusions

The main purpose of the present study was to assess the seismic performance of multi-storey RC buildings designed based on modern codes and in particular the contemporary Greek national design codes. For this purpose a parametric study has been performed considering six models where a number of construction features have been implemented through the provisions imposed by the Greek national design codes. In particular three construction practices often encountered in Greece are



(a) (b)

Fig. 13 Graphical representation of the performance points in both directions



(a) (b)

(c)

Fig. 14 Drift profiles: (a) 50/50 hazard level, (b) 10/50 hazard level, (c) 2/50 hazard level

examined: (i) weak ground storey, (ii) short columns and (iii) floating or hanging columns. In addition two combinations of the three construction practices are also studied. These models are assessed with reference to their performance against three earthquake hazard levels. Through the parametric study, it was found that the performance of a multi-storey fully infilled RC building was superior to all construction practices examined. In particular the fully infilled model was the only one that fulfilled the drift limit requirement for the occasional and rare hazard levels while it slightly exceeded drift limit for the very rare hazard level. In terms of the structural capacity the weak ground storey and short column designs were found to be 50% worst versus the fully infilled design. Although these conclusions cannot be generalized, it is an indication that for the test example considered the Greek national design codes failed to take into account the weak ground storey, the short and the floating columns construction practices.

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