Seismic Evaluation of Infilled RC Structures with Nonlinear Static Analysis Procedures

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SUMMARY:
Nonlinear static procedures are applied for the evaluation of the seismic capacity of reinforced concrete frames with and without infills. The effect of the infills on the capacity curves and on the performance points of reinforced concrete (RC) structures is investigated. Two types of infilled frames are considered: (a) fully infilled and (b) infilled frame without infills at the base floor (pilotis type building). Moreover, two different configurations of the base floor morphology are studied: (i) all the floors of the frame have equal heights and (ii) the height at the base floor is greater than the one of the other stories. The performance point of all the examined structures is estimated through the Capacity Spectrum Method (ATC-40) and through the Coefficient Method (FEMA356). Results in terms of overall demands, failure modes, capacity curves, interstory drifts and ductility requirements are considered and presented. From the results it can be deduced that the estimated values of the performance points of the examined structures are not significantly influenced by the presence of the infills. The influence of the infills on the global and local response of the structures differs depending on the performance level at which the seismic assessment is performed.

Keywords: nonlinear static analysis, seismic assessment, masonry infilled RC frames

1. INTRODUCTION

Damages in reinforced concrete (RC) buildings during the recent earthquakes indicated that the interaction between masonry infills and bare frame can lead to unexpected effects on the seismic response of the structure such as shear failure in columns, damages in joint regions and soft-storey mechanisms (Karayannis et al., 2011). Furthermore the damage distribution over the structure is completely changed due to the presents of the infills. Thus, over the last five decades many analytical researches have been performed for the investigation of the influence of the mansory infills on the seismic response of the RC structures. The aim of this study is to investigate the effect of the masonry infills on different performance levels of the RC structures, using nonlinear pushover analyses procedures.

It is well known that the nonlinear static analysis procedures are applied for the estimation of the seismic performance assessment of RC structures. For this purpose the nonlinear pushover analysis is first performed in order to derive the capacity curve of the structure. The seismic response of the structure is then evaluated by comparing the demands to the available capacities at the various performance levels of interest. The most commonly used procedures for the estimation of the performance point are the Capacity Spectrum Method (ATC,1996), the Coefficient Method (FEMA,2000), and the N2 Method (Fajfar, 2000).

The effect of the infill panels on the capacities and on the performance points of the RC structures is studied herein considering three different structural models: (a) bare frame, (b) fully infilled frame and (c) infilled frame without infills at the base floor (pilotis type frame).

Moreover, each of these models is examined for two different configurations of the first storey level morphology;

- Case A: frame structure with equal interstory heights,
- Case B: frame structure where all interstory heights are equal but the base story that exhibits higher interstory height.

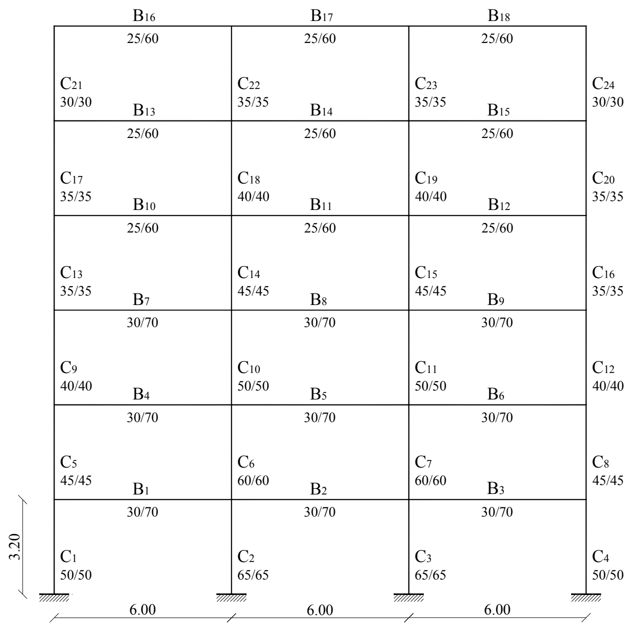
The performance point of the examined RC structures with and without infills is estimated based on two methods; Capacity Spectrum Method (ATC-40) and Coefficient Method (FEMA 356).

For the needs of this study special purpose inelastic elements-models are adopted for the simulation of the RC beams and columns and the infilled panels. In fact, for the simulation of the behaviour of the infills the equivalent diagonal strut model is used taking into account the actual conditions of the effective lateral confinement of the masonry by the reinforced concrete frame. The analyses are performed using the program Drain-2Dx (Prakash et al., 1993).

Results in terms of overall demands, failure modes, capacity curves, interstorey drifts and ductility requirements are presented. Moreover, results about the maximum plastic rotations of beams and columns of the structures are presented including the local effect of the infills.

2. DESIGN OF THE STRUCTURE

The examined RC structure is a 6-story frame building structure designed according to the Greek codes that are very close to Eurocodes 2 & 8. The mass of the structure is taken equal to M = (G+0.3Q) (where, G gravity loads and Q live loads). The design base shear force of the examined 6-story structure was equal to V = (0.3g/q)M where, q is the behaviour factor of the structure equal to 3.5. Reduced values of member moments of inertia (I_{ef}) were considered in the design to account for the cracking; for beams $I_{ef}=0.5I_g$ (where I_g the moment of inertia of the gross section) and for the columns $I_{ef}=0.9I_g$. Critical for the dimensioning of the columns proved to be in most of the cases the code provision regarding the axial load ratio limitation $v_d \leq 0.65$ and in a few cases the code requirements for minimum dimensions. Structural geometry and reinforcement of the columns of the 6-story frame are shown in Fig. 1.



External columns		***Internal columns***	
reinforcement		*reinforcement*	
$C_{1\ \&\ 4}$	10Ø18 up	$C_{2\ \&\ 3}$	12Ø20up
	8Ø20+6Ø18 dn		8Ø20+6Ø18dn
$C_{5\ \&\ 8}$	8Ø18up	$C_{6\ \&\ 7}$	10Ø20up
	4Ø20+6Ø18dn		12Ø20dn
$C_{9\ \&\ 12}$	6Ø20+2Ø18	$C_{10\ \&\ 11}$	8Ø20+4Ø18
$C_{13\ \&\ 16}$	10Ø18up	$C_{14\ \&\ 15}$	10Ø20up
	2Ø20+8Ø18 dn		4Ø20+8Ø18dn
$C_{17\ \&\ 20}$	4Ø18+10Ø16 up	$C_{18\ \&\ 19}$	12Ø20+4Ø18up
	8Ø16dn		8Ø18dn
$C_{21\ \&\ 24}$	8Ø14	$C_{22\ \&\ 23}$	10Ø14

Figure 1. Structural system and column reinforcements of 6-storey RC frame.

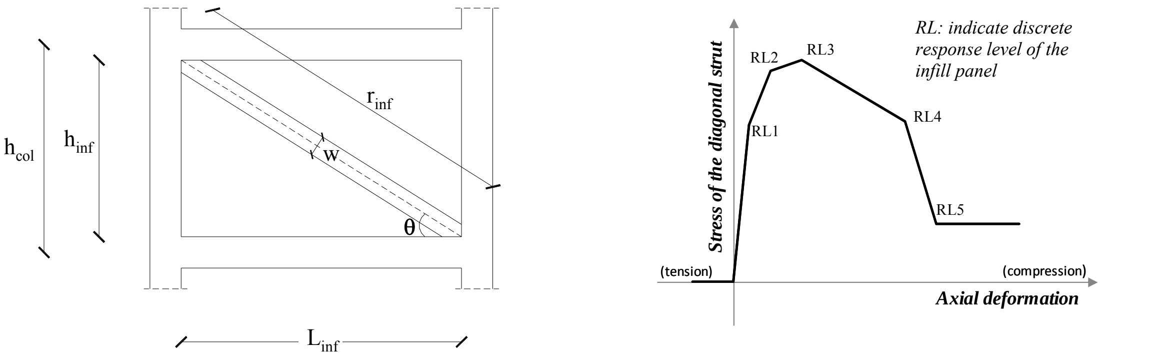
3. MODELLING ASSUMPTIONS

3.1 Simulation of the beams and columns

The structural system consists of beams and columns. The structure is modeled as a 2D assemblage of non-linear elements connected at nodes. The mass is lumped at the nodes and each node has three degrees of freedom. The finite element mesh utilizes a one-dimensional element for each structural member. Two types of one-dimensional beam-column elements were used. The first one is the common lumped plasticity beam - column element and it was used for the modeling of the beams. With this element-model the inelastic behaviour is concentrated in zero-length "plastic hinges" at the element's ends. For the modeling of the columns a different type of element is adopted. That was the "distributed plasticity" special purpose element. This type of element is accounting for the spread of inelastic behaviour both over the cross-sections and along the deformable region of the member length. Moreover, this element performs numerical integration of the virtual work along the length of the member using data deduced from cross-section analysis at pre-selected locations. Thus, the deformable part of the element is divided into a number of segments and the behaviour of each segment is monitored at the centre cross-section (control section) of it. The cross-section analysis that is performed at the control sections is based on the fibre model. This fibre model accounts rationally for axial – moment (P-M) interaction.

3.2 Simulation of the infill panels

For the simulation of the local response of the masonry infill panel the equivalent diagonal strut model is used. For this purpose a special purpose beam-column element is used for the modeling of the infills (Karayannis et al 2005). This element accounts for more accurate definition of the response properties of infilled masonry since it includes degrading branch (Fig. 2). Special attention has been given in the implementation of this element for the simulation of the infill panel in order to exhibit axial response only and not flexural one. An important problem in modeling the infill panel is the determination of the response characteristics of the diagonal strut model, taking into account the actual conditions of the effective lateral confinement of the masonry by the reinforced concrete frame. The actual properties of the infill panel have been approached using the experimental results by Karayannis et al (2005) and Kakaletsis & Karayannis (2009). After the assessment of the lateral resistance of the infill panel the characteristics needed for the diagonal strut model were determined. The effective width of the diagonal element was determined according to FEMA 273 (1997) & FEMA 306 (1999) recommendations that are mainly based on the Mainstone's formula (1971) (see also Fig. 2).



w : width of equivalent diagonal compression strut

Maximum compression strength $f'_m \approx 5,05$ MPa

a. Characteristics of the equivalent diagonal strut

b. Local response of the infill element

Figure 2. Simulation of the infill panel based on the diagonal strut model

4. NON-LINEAR STATIC PUSHOVER ANALYSIS

In this work, the inelastic pushover analyses are carried out in order to investigate the effect of the infill panels on the capacities and on the performance points of the RC structures. For this purpose three structural models have been studied: (a) bare frame, (b) fully infilled frame and (c) infilled frame without infills at the base floor (pilotis type frame). Each of these models is examined for two different configurations of the first storey level:

- Case A: frame structure with equal interstory heights,
- Case B: frame structure where all interstory heights are equal but the base story that exhibits higher interstory height.

Two different load patters are taken into account. The first load shape considered is the triangular one based on the design code and the second is the uniform load pattern. All the pushover analyses have been performed until the maximum top displacement reached the 1% of the total height of the structures (top drift equal to 1%h_{str}). However in this work results only for the case that triangular distribution of the inertia forces is imposed on the structures are presented (due to space limitations). Comparative presentation of the capacity curves in terms of global base shear-top displacement of the examined structural system Case A (with and without infills) is shown in Fig. 3. From the results of Fig. 3, an increase of the global stiffness and strength of the structure due to the presence of the masonry infill panels can be observed. Nevertheless, abrupt degradation points of the load carrying capacity of the infilled frames are observed that are attributed to the failure points of the infills. The results for the Case B structural system are presented in Fig.4.

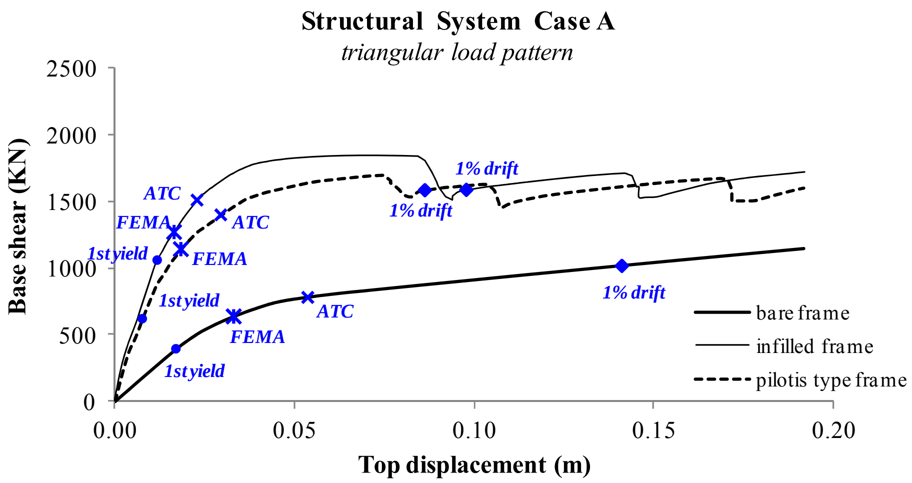


Figure 3. Capacity curves of the 6-story frame structures with equal interstory heights.

For the evaluation of the performance points of all the examined RC structures two different nonlinear static analysis procedures have been used: (a) the Capacity Spectrum Method (ATC-40) and (b) the Coefficient Method (FEMA 356). The results are presented in Table 4.1 for all the examined cases. It can be observed that the performance point of the structures according to FEMA is reached at a displacement smaller than the corresponding one estimated according to the provisions of ATC. The performance points of the structural systems Case A and Case B are also depicted in Figs. 3 & 4. The presence of the infills has as a result a decreasing of the target displacement at which the buildings have to respond within the acceptance criteria.

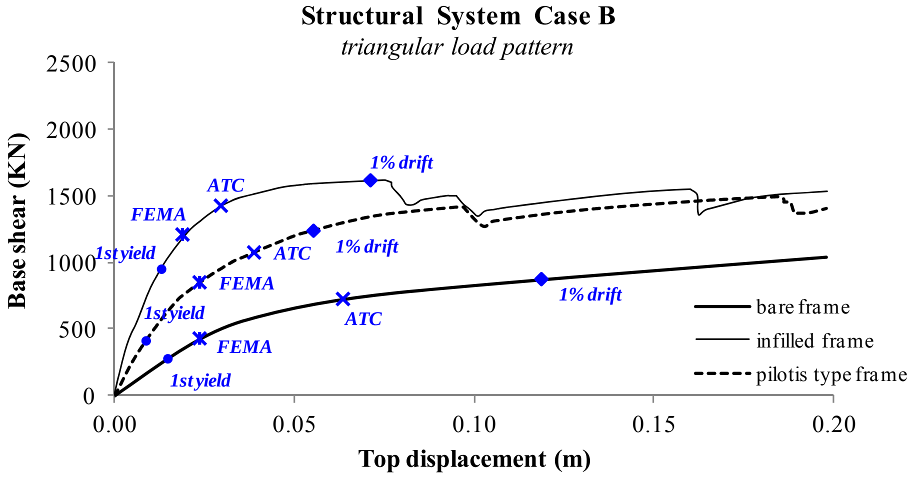


Figure 4. Capacity curves of the 6-story frame structures with the height at the base to be greater than the other equal interstory heights (Case B).

Table4.1 Target displacements and performance points of the examined 6-story frame structures

	Structural type Case A			
	FEMA 356		***ATC-40***	
	Target displacement (m)	V(kN)	Performance point (m)	V(kN)
bare frame	0.0332	634.49	0.0538	777.42
fully infilled frame	0.0166	1267.45	0.0230	1507.16
pilotis type frame	0.0185	1139.91	0.0296	1395.91
	Structural type Case B			
	FEMA 356		***ATC-40***	
	Target displacement (m)	V(kN)	Performance point (m)	V(kN)
bare frame	0.0238	426.34	0.0638	726.59
fully infilled frame	0.0181	1148.58	0.0297	1422.66
pilotis type frame	0.0238	847.76	0.0382	1063.19

The effect of the infills on the response of the RC structures is also investigated at the performance level of immediate occupancy (IO). This level corresponds to a maximum interstory drift equal to 1% of the story height (h_s) (ATC-40). The point at which each frame, with and without infills, reaches the IO level is shown in Figs. 3 and 4, for the examined Case A and Case B, respectively. It can be observed that the 6-story bare frame structure in Case A develops the 1% interstory drift at top displacement equal to 0.14m, while the 6-story bare frame in Case B develops the 1% interstory drift at top displacement equal to 0.12m. Nevertheless, it is observed that in cases that the local effect of the masonry infills is taken into account in the analyses of the structures the infilled frames reach the IO level (1%drift) at top displacements smaller than the corresponding ones of the bare frames (infilled frame: 0.098m (Case A) & 0.072m (Case B), and pilotis type frame: 0.086m (Case A) and 0.055m (Case B)) (Figs. 3&4). The infilled structures of Case A reach the performance level of immediate occupancy (IO level) after infills degrade their stiffness and strength whereas the infilled structures of Case B reach the performance level of IO before any infill panel collapses. Strength and stiffness degradation of all the infills (in all the examined cases) are observed after the structural performance points according to FEMA356 and ATC-40.
The maximum interstory drifts of the 6-story frame structure with equal interstory heights (Case A) are presented and compared with the corresponding responses of the infilled frames (fully infilled and

pilotis type); (a) at top drift $1\%h_{str}$ (Fig. 5) and (b) at the performance points as deduced based on ATC-40 and FEMA356 (Fig. 6).
The results in Fig. 5 indicate that the interstory drifts of the infilled structures are greater in comparison to the corresponding ones of the bare frame at the storey levels 1st to 4th. The maximum drifts are observed at the 3rd floor level of both infilled frames. In the case of the fully infilled frame the interstory drift at the 3rd floor level is equal to $1.98\%h_s$ which is very close to a value of $2\%h_s$ drift that corresponds to the Life Safety performance level (ATC-40). In the case of bare frame a maximum value of $1.34\%h_s$ interstory drift is observed at 4th storey. At the upper floor levels (5th-6th) the drifts of the infilled structures are very small compared with the corresponding ones of the bare frame of Case A.
At the performance points of the RC structures the presence of the infills resulted to a smaller interstory drift values compared to the corresponding values that are developed in the case of the bare frame building (Fig.6). Moreover, it is noted that high values of 1st level interstory drifts are observed for the cases of pilotis type frames compared with the ones of the other structures.

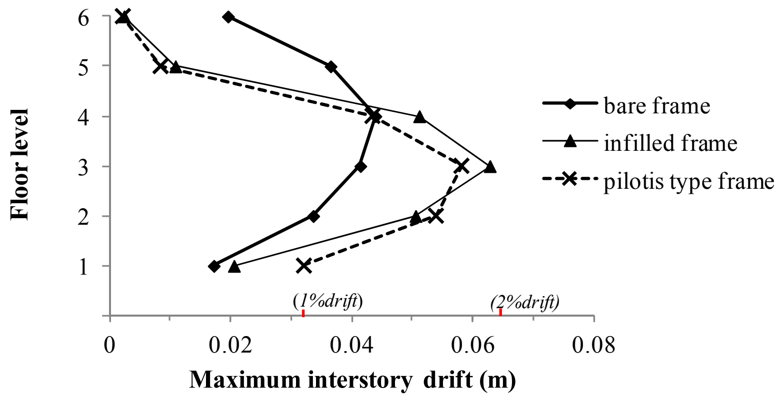


Figure 5. Maximum interstory drifts of the 6-story frame structures with equal interstory heights (Case A) at top drift equal to $1\%h_{str}$.

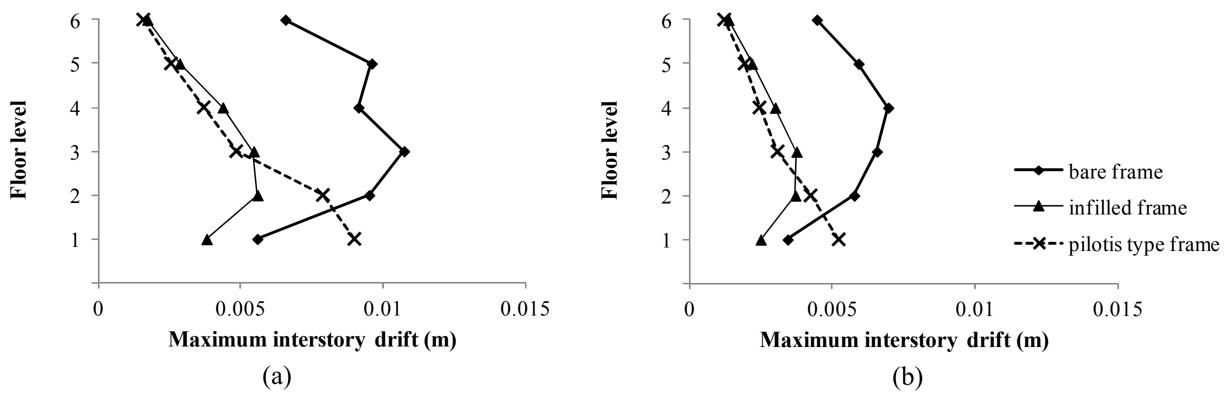
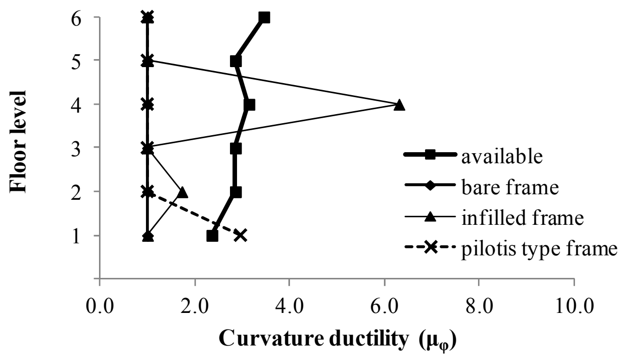


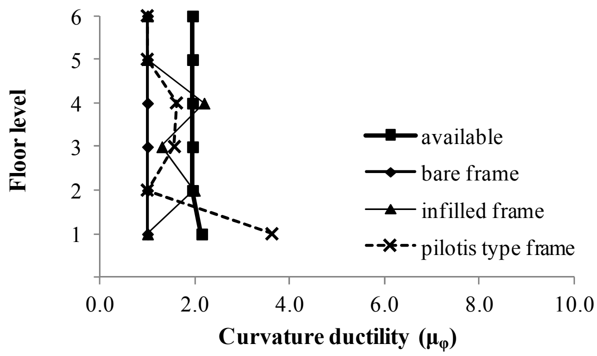
Figure 6. Maximum interstory drifts of the 6-story frame structures with equal interstory heights (Case A): a) at performance points to ATC40 and b) at target displacement to FEMA356.

The local inelastic demands of the beams and the columns of the 6-story RC structures are also studied in terms of curvature ductility (μ_φ) and plastic rotational hinge requirements (θ_p) as well (Figs. 7 & 8). The behaviour of the structural members is checked at four performance levels; (a) at top drift equal to 1% of the total height of the structures (h_{str}), (b) at interstory drift equal to 1% of the storey height (h_s) and (c) at the performance points of the structures according to ATC-40 & FEMA356.
In Fig. 7 comparative results of the maximum ductility requirements of the columns, between the three different types of infilled frames with structural system of Case A, are presented and compared with the available ductility capacity. These results demonstrate that at the point of top drift $1\%h_{str}$, a soft-

story mechanism is developed at the base floor in the case of the pilotis type frame and at the 4th floor in the case of the fully infilled frame, since the ductility requirements of these columns exceed the available ones. It is also noted that in the Case B structural system (and triangular load pattern) a soft-story mechanism is developed at the base floor level of both frames (fully infilled frame and pilotis type frame) at the point of top drift equal to $1\%h_{str}$. At the performance points to ATC and FEMA the columns remain in the elastic range.



a) local requirements of the internal columns



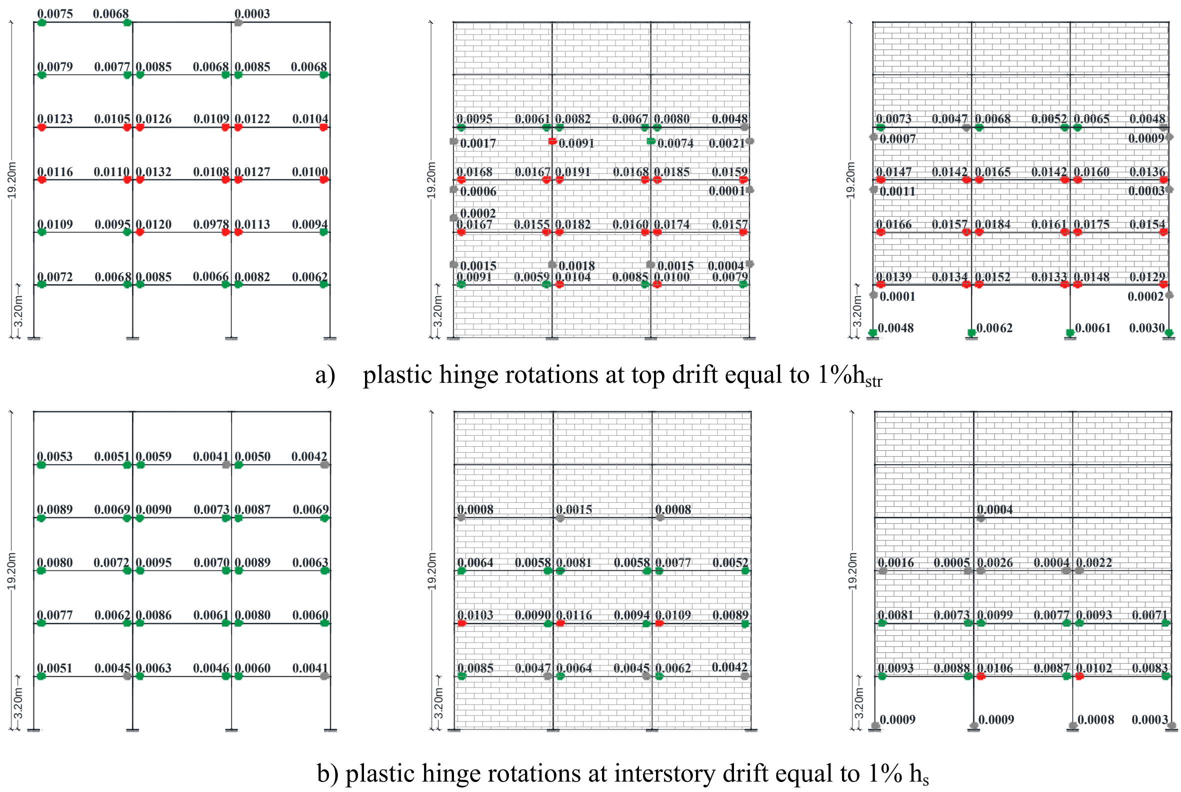
b) local requirements of the external columns

Figure 7. Curvature ductility requirements of the columns of the 6-story frame structures with equal interstory heights (Case A) at the point of top drift $1\%h_{str}$.

The collapse of the infills in the Case A led to the development of soft-story mechanisms. Moreover, based on the results shown in Fig. 8b it can be observed that when the performance of the structure is studied at an early stage of the response (e.g. at interstory drift $1\%h_s$) the development of a soft-story mechanism at the base of the pilotis type frame cannot be prevented.

The local inelastic responses of the infill panels of the fully infilled 6-story frame structure with equal interstory heights are presented in Fig. 9, in terms of compressive axial deformations vs. top displacement of the structure until top drift becomes equal to $1\%h_{str}$. In the same figure three different response levels of the infills are also shown: (a) deformation level (‰) at maximum strength (RL3), (b) deformation level (‰) with strength and stiffness degradation (RL4) and (c) ultimate deformation level (‰) – infill collapse (RL5) (see Fig.2b). In this way a direct comparison between demands and capacities of the mansory infills is provided while useful comments can be deduced about the seismic performance of the infills along the capacity curves of the structures.

As it can be observed in Fig. 9, the infill panels of 2nd, 3rd and 4th story levels of the Case A structure have failed since the developing demands for deformations exceed their ultimate capacity. Damages (strength and stiffness degradation) are also occurred at the 1st floor level of the examined building.



Acceptable limits of θ_p (in rad) at performance levels:

Beams

- SS (structural stability): $\theta_p = 0.02$
- LS (life safety): $\theta_p = 0.01$
- IO (immediate occupancy): $\theta_p = 0.005$
- $\theta_p < 5^0/_{00}$

Columns

- SS (structural stability): $\theta_p = 0.015$
- LS (life safety): $\theta_p = 0.0075$
- IO (immediate occupancy): $\theta_p = 0.0025$
- $\theta_p < 2.5^0/_{00}$

Figure 8. Maximum rotations of plastic hinges θ_p and corresponding performance levels in beams and columns of the 6-story frame structures with equal interstory heights (Case A); a) at top drift equal to 1% h_{str} and b) at interstory drift equal to 1% h_s.

Moreover, based on the results of Fig. 9 the infill panels that first develop the critical deformation demands are these of the 2nd and 3rd floor levels of the infilled frame.

In the case of pilotis type frame (Case A) the corresponding results of the seismic performance of the infill panels are presented in Fig. 10. Similar to the case of fully infilled structure, high deformation demands are developed at the infills of the 2nd, 3rd and 4th story level of the structure. Focused on the damage distribution of the infills along with the top displacement of the structure, it can be observed that these critical infills reached collapse in the following order; infill panels at 2nd, then at 3rd and finally at 4th floor level.

Moreover, in Table 4.2 information about the first time that each deformation level of the masonry infill is reached throughout the structure are given. These results are in terms of the corresponding top drift (%h_{str}) and are given for both examined cases of infilled frames of Case A.

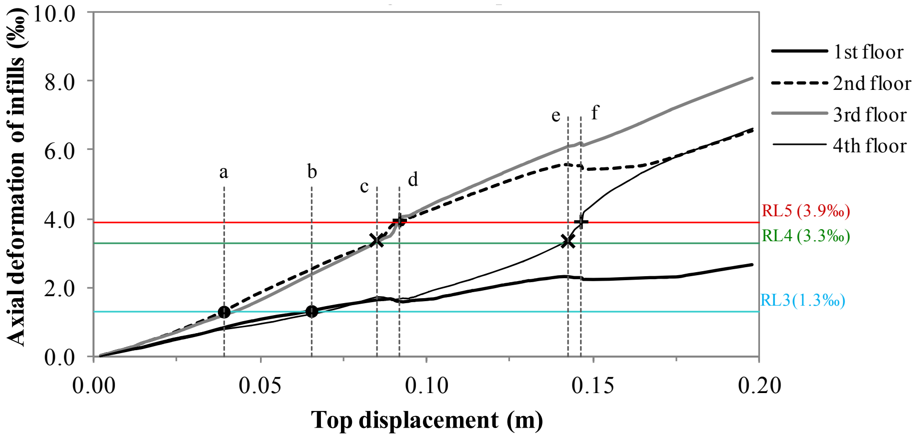


Figure 9. Local inelastic responses of the infill panels of the fully infilled 6-story frame structure with equal interstory heights.

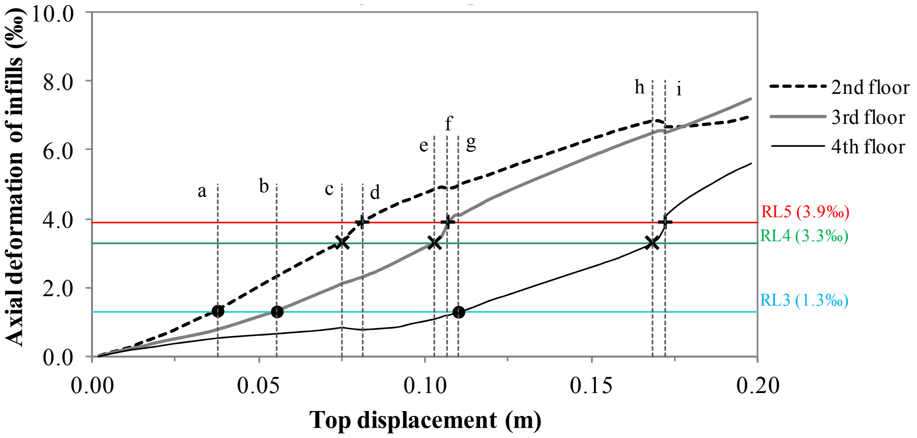


Figure 10. Local inelastic responses of the infill panels of the pilotis type 6-story frame structure with equal interstory heights.

Table4.2 Top drifts (%h_{str}) at different local response levels of the infills

	Structural type Case A			
response levels (RL)*	***first cracking (RL1)***	***RL3***	***RL4***	***RL5***
fully infilled frame	0.072	0.213	0.454	0.485
pilotis type frame	0.072	0.198	0.392	0.423

**level reached for the first time during the analysis*

5. CONCLUSIONS

The effect of the infills on the evaluation of the seismic capacity of RC frames structures is investigated based on extensive use of inelastic pushover analyses and the FEMA and ATC-40 recommendations. The effects of the infills on the seismic evaluation of RC 6-story frame structures, at four performance levels are presented: (a) at top drift equal to 1% of the total height of the structures (h_{str}), (b) at interstory drift equal to 1% of the storey height (h_s), (c) at the performance points of the structures according to ATC-40 and (d) at the performance points of the structures according to FEMA356.

The results of this study indicate that the global stiffness and strength of the RC structures are increased due to the presence of the infills as long as the seismic demand does not exceed the deformation capacity of the infills. When an infill collapses an abrupt decrease of the capacity of the structure is occurred.

The values of the performance points of the examined structures have not been significantly influenced by the presence of the infills. This is attributed to the fact that in all examined cases the infills failed after the structural performance point. Nevertheless, a decrease of the target displacement is observed in all the cases of infilled frames when compared to the ones of the corresponding bare frames.

The seismic performance of the infilled frame structures at the point of top drift 1%h_{str}, indicate that the presence of the infills lead to the development of a soft-story mechanism not only in the case of the pilotis type frame but also in the cases of fully infilled frames. However, when the performance of the structure is studied at an early state of response due to the infills there are no demands for inelastic behaviour of the members.

Thus, from the examined cases of this study it can be concluded that the influence of the infills on the global and local response of the structures differs depending on the performance level at which the seismic assessment is performed.

Moreover, although the development of a soft-story mechanism can be considered as typical for pilotis type infilled frames (infills missing at base story) the results of this study indicate that this mechanism can also be occurred in fully infilled frame structures (regular distribution of infills) (see also Dolšek and Fajfar, 2001, 2008). In the cases of pilotis type frames high values of base floor interstory drifts are observed in comparison with the corresponding ones of the other structures.

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