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Limit states of RC structures with first floor irregularities

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Abstract. The seismic performance of reinforced concrete (RC) frame structures with irregularities leading to soft first floor is studied using capacity assessment procedures. The soft first story effect is investigated for the cases: (i) slab-column connections without beams at the first floor, (ii) tall first story height and (iii) pilotis type building (open ground story). The effects of the first floor irregularity on the RC frame structure performance stages at global and local level (limit states) are investigated. Assessment based on the Capacity Spectrum Method (ATC-40) and on the Coefficient Method (FEMA 356) is also examined. Results in terms of failure modes, capacity curves, interstory drifts, ductility requirements and infills behaviour are presented. From the results it can be deduced that the global capacity of the structures is decreased due to the considered first floor morphology irregularities in comparison to the capacities of the regular structure. An increase of the demands for interstory drift is observed at the first floor level due to the considered irregularities while the open ground floor structure (pilotis type) led to even higher values of interstory drift demands at the first story. In the cases of tall first story and slab-column connections without beams soft-story mechanisms have also been observed at the first floor. Rotational criteria (EC8-part3) showed that the structure with slab-column connections without beams exhibited the most critical response.

Keywords: limit states; performance levels; seismic assessment; soft story effect; RC frame structures; pilotis type frame

1. Introduction

The seismic performance of multistory RC frame buildings with vertical irregularities concentrated at the first floor is investigated, using capacity assessment procedures based on inelastic static pushover analysis.

In recent years the effect of the structural irregularities on the seismic behaviour of the structures has been the objective of numerous studies. A distinction between irregularities in plan and in elevation characterizes the scientific literature although usually a combination of both types is expected to coexist. However, a recent review of the research progress on the seismic response of plan and vertically irregular structures indicate that the research activity in vertically asymmetric structures is limited in comparison to the corresponding activity for in plan irregularities (De Stefano and Pintucchi 2008).

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Focusing on the vertical irregularities different distributions of mass, stiffness and/or strength along the height of the RC buildings have been studied in order to understand and clarify critical aspects for the seismic behaviour, analysis and design of these structures (e.g., Tremblay and Poncet 2005, Das and Nau 2003).

In this view, morphology of the first floor has a crucial role on the seismic performance of the RC buildings. The formation of a tall first story and discontinuities in beams and/or columns are the most common irregularities that can be observed at the first floor of the multistory RC frame structures. On the other hand these structures are typically infilled with masonry panels introducing this way additional irregularity into the structure due to reduction or total absence of infills in a particular story e.g., pilotis type buildings: structures with an open ground stories (parking space) and infills on the upper stories. Moreover, the most hazardous irregularities are found in the structures with a tall first story and/or an open ground story since the overall behaviour of this type of buildings is mainly governed by the response of the first floor. Further, the presence of an open ground story may cause (Murty *et al.* 2002): (a) soft story effect: increased deformation demand in the frame members of the open ground story because it may have smaller stiffness or (b) weak story effect: it may have lower lateral strength and cause a discontinuity in flow of lateral seismic shear in the open ground story.

Analysing the results of past earthquakes it can be observed that such irregularities at the first floor of the structures are responsible for severe damages and even collapses with a common type of structural irregularity failure to be that of soft/or weak story. For example, Kirac *et al.* (2011) reported that 59.67% of the damaged buildings at Kocaeli-Izmit earthquake (1999 Turkey) were due to first soft-story. The height of the soft story is pointed by the authors as a key parameter on the formation of a soft story mechanism. It was observed that about 50% higher height of one story comparing to the other stories led the structures to a soft story irregularity failure. Also, during 2001 Bhuj (India) earthquake 130 buildings that collapsed in Ahmedabad were of open ground story configuration while among those that did not collapse the damage was concentrated in the columns of the open ground story (Murty *et al.* 2002). On the contrary, several regular infilled frame structures with symmetrical distribution of infill in both elevation and plan performed reasonably well avoiding the collapse in the zone of highest intensity during the 1985 Mexico city earthquake.

Further, modern codes for the seismic design of RC structures also address problems associated with vertical irregularities such as first soft story. In general, in order to prevent the development of a plastic soft story mechanism, codes (e.g., Eurocode 8-CEN2004) recommend an increase in the resistance of the irregular story members depending upon the extent of irregularity, the type of structural system, etc. Details of how several codes approaching the seismic design of irregular infilled RC frames can be found in a recent work done by Kaushik *et al.* (2006).

The seismic inelastic response of the reinforced concrete structures with an open ground story as a result of the absence of infills has been the subject of many analytical researches (Karayannis *et al.* 2011, Korkmaz *et al.* 2007, etc). Also, experimental studies and retrofitting techniques for strengthening of buildings with a soft first story (pilotis type) have been reported in literature (e.g., Lee *et al.* 2011, Antonopoulos and Anagnostopoulos 2012).

Nevertheless studies focused on the effect of the first floor morphology due to tall story height or/and open ground story on the seismic assessment of multistory RC frames are limited.

In 2003, Das and Nau investigated the effectiveness of the equivalent lateral force procedure on the seismic design of vertically irregular RC buildings. For this purpose they included the following parameters of irregularity: (a) tall first-intermediate-top story, (b) heavy top-midheightbottom mass and (c) masonry infills; open first floor and partial infill. It was concluded that for the examined types of irregularity the restrictions on the applicability of the equivalent lateral force procedure are unnecessarily conservative.

Further, the work presented by Favvata *et al.* (2012) indicates that in multistory RC buildings the damage distribution not only is significantly changed by the presence of the infills but also it is depending on the performance level at which the seismic assessment is performed. Similarly, the results of the nonlinear analyses done by Celarec *et al.* (2012) indicate that at different limit states different remarks about the sensitivity of the response parameters to the modelling variables of infilled RC frames are computed.

In 2006, Repapis *et al.* presented a methodology for the evaluation of the structural overstrength, the global ductility and the available behaviour factor of existing RC buildings. For the estimation of these global performance characteristics different failure criteria were incorporated in the methodology in order to predict the failure mode of the structures. The proposed methodology was applied to a large number of typical existing structures, including vertically irregular RC frames. They concluded that buildings with a completely open floor or with column discontinuities in the ground floor exhibited the most unfavourable performance.

Based on the above, the aim of this study is to investigate the effect of the first floor morphology with vertical irregularities on the seismic response of the structures including the effect of the masonry infills using nonlinear pushover analysis procedures. It is well known that the nonlinear static analysis procedures are commonly applied for the estimation of the seismic assessment of RC structures. For this purpose the nonlinear pushover analysis is first performed in order to derive the capacity curve of the structure while different methods of performing the nonlinear static (pushover) analysis are provided by the seismic codes (Eurocode 8, ATC40, FEMA). 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 at a damage earthquake are the Capacity Spectrum Method (1996), the Coefficient Method (2000).

However, the accuracy of these procedures to evaluate the performance of the buildings although has been verified through a large number of studies in the case of regular structures there are restrictions in case of irregular buildings. The main restriction of these procedures is the inability to represent the torsional effects of plan irregular 3D structures. Thus, the interest of several researchers is focused on the development of pushover analysis methods capable to account the torsional behavior of plan asymmetric structures (Bhatt and Bento 2011). The effectiveness of the pushover analysis procedures to evaluate the response of vertically irregular structures is also reported as a field of interest. The results of the study done by Chintanapakdee and Chopra (2004) show that the modal pushover analyses may accurately estimate the seismic demands of irregular RC frames, except for the cases of structures with setbacks in the upper floor levels and structures with a strong first story or strong lower half.

In this study the examined RC structural systems are regular in plan without torsional effects either in plan or in vertical view, while only the first floor irregularity is under consideration. Thus, there are no severe restrictions of performing the pushover analyses for the seismic assessment of these structures. Moreover the investigation of the effectiveness of the pushover analysis procedure is out of the scope of this study.

This paper addresses the influence of the first floor morphology on the seismic performance of a six-story reinforced concrete frame structure. Three different configurations of the first floor are studied (Fig. 1), whereas infills are also considered as an additional parameter of structural

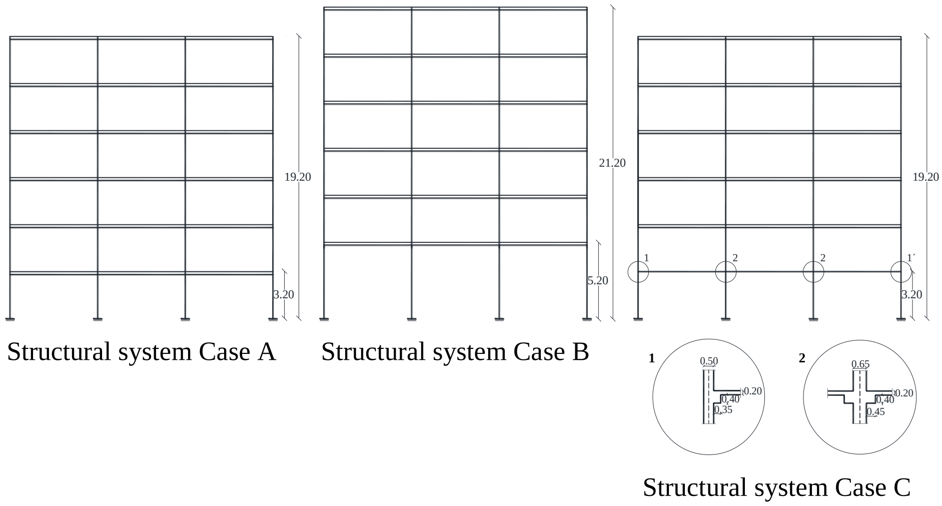


Fig. 1 First floor morphology configurations

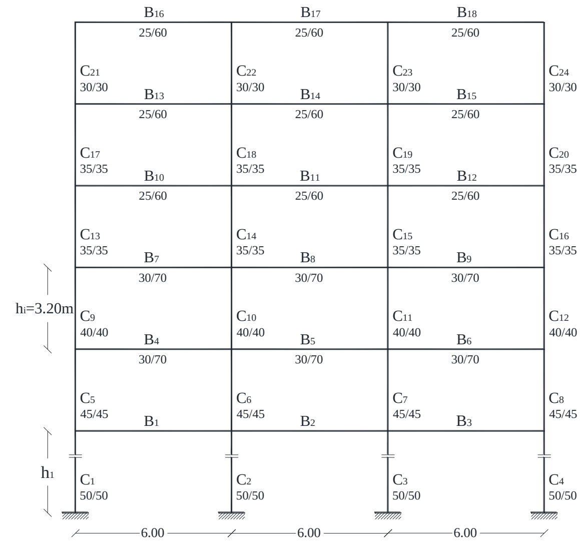
irregularity. Moreover, the effect of the first floor irregularity on the capacity of the multistory RC frame structure (with or without infills) to attain specific performance stages at the global level of the structure and at the local level of the structural members is also investigated and compared with the performance points of the structure estimated based on the Capacity Spectrum Method (ATC-40) and on the Coefficient (FEMA 356) Method.

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. The analyses are performed using the program Drain-2Dx (Prakash *et al.* 1993).

Results in terms of overall demands, failure modes, capacity curves, interstory drifts, ductility requirements and infills responses, are presented. The maximum plastic rotations demands of the beams and the columns are estimated and the attained limit state is identified. Comparative results between the local performance levels provided by FEMA and EC8-part3 are presented and commented. Finally results in terms of top drift displacement at which infills reach for first time specific response levels are presented in all the examined cases.

2. Design of the examined structures

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=594.69KN 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 and for the columns I_{ef}=0.9I_g (where I_g the moment of inertia of the gross section). 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. 2. It is noted that for the examined RC frame



External columns		*Internal columns*	
reinforcement		*reinforcement*	
$C_{1\ \&\ 4}$	10Ø 18 up 8Ø 20+6Ø 18 dn	$C_{2\ \&\ 3}$	12Ø 20up 8Ø 20+6Ø 18dn
$C_{5\ \&\ 8}$	8Ø 18up 4Ø 20+6Ø 18dn	$C_{6\ \&\ 7}$	10Ø 20up 12Ø 20dn
$C_{9\ \&\ 12}$	6Ø 20+2Ø 18	$C_{10\ \&\ 11}$	8Ø 20+4Ø 18
$C_{13\ \&\ 16}$	10Ø 18up 2Ø 20+8Ø 18 dn	$C_{14\ \&\ 15}$	10Ø 20up 4Ø 20+8Ø 18dn
$C_{17\ \&\ 20}$	4Ø 18+10Ø 16 up 8Ø 16dn	$C_{18\ \&\ 19}$	12Ø 20+4Ø 18up 8Ø 18dn
$C_{21\ \&\ 24}$	8Ø 14	$C_{22\ \&\ 23}$	10Ø 14

	Case A	**Case B**	**Case C**
B_1, B_2, B_3 [cm]	30/70	30/70	only slab d=20cm
h_i [m]	3.20	3.20	3.20
h_1 [m]	3.20	5.20	3.20

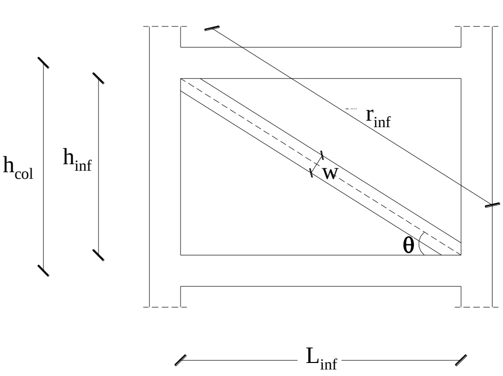
Fig. 2 Geometrical characteristics and column reinforcements of the examined 6-story RC frame structures

structure a strict code design procedure is followed. For this reason as it can also be observed in Fig. 2 some columns have different requirements for reinforcement in the up section compared to the ones of the bottom section.

3. Structural modelling

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 an 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 is 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 is 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 (Karayannis *et al*. 1994). 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.



w : width of equivalent diagonal compression strut

Floor level	r_{inf} *(m)*	*w(mm)*	*maximum axial compressive strength* $N_R = w \cdot t \cdot f'_m$ *(KN)*
1st	*6.128*	*731*	*369.15*
2nd	*6.019*	*689*	*348.03*
3rd	*6.087*	*655*	*330.77*
4th	*6.153*	*655*	*330.77*
5th	*6.197*	*630*	*318.15*
6th	*6.242*	*600*	*303.00*

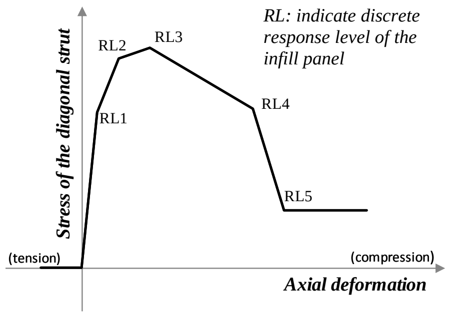
thickness of masonry infill t_{inf} *=10cm*

$$w = 0.175 \cdot (\lambda_1 \cdot h_{col})^{-0.4} r_{\inf} \qquad \text{(Mainstone 1971)}$$

$$\lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4EIh_{\inf}} \right]^{\frac{1}{4}}$$

E_{me}*: modulus of elasticity of masonry*
EI: flexural stiffness of columns

(a) Characteristics of the equivalent diagonal strut



Response level	*Stress of diagonal strut (MPa)*	*Axial deformation (‰)*
RL1	*3.57*	*0.3*
RL2	*4.79*	*0.7*
RL3	*5.05*	*1.3*
RL4	*3.65*	*3.3*
RL5	*1.32*	*3.9*

Maximum compression strength $f'_m \approx 5,05$ *MPa*
(Karayannis *et al.* 2005)

(b) Response of the infill element (see also Karayannis *et al.* 2005)

Fig. 3 Simulation of the infill panel based on the diagonal strut model and mechanical parameters

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. A special purpose element is used for the modeling of the infills (Karayannis *et al.* 2005). This element accounts for accurate definition of the response properties of infilled masonry since it includes degrading branch (Fig. 3). 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 properties. 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 surrounding reinforced concrete frame. The actual properties of the infill panel have been approached using the experimental results by Karayannis *et al.* (2005) and by Kakaletsis and Karayannis (2009). The mechanical properties of the infill panel are presented in Fig. 3. After the assessment of the lateral resistance of the infill panel the characteristics needed for the diagonal strut model are determined. The effective width of the diagonal element is determined according to FEMA 273 and FEMA 306 recommendations that are mainly based on the Mainstone's formula 1971 (see also Fig. 3).

4. Key assumptions for the seismic assessment

Examined structural systems

In this study the influence of the first floor morphology on the local and global demands and capacities of 6-story reinforced concrete frame structures is investigated. Three different configurations of the first floor morphology are examined (Figs. 1 and 2):

- *Case A:* all the floors of the frame have equal heights (all interstory heights equal to 3.2m and total height of the structure equal to 19.20m)
- *Case B:* the height at the first floor is greater than the one of the other stories (first floor height equal to 5.20m, the height of the other stories equal to 3.2m and the total height of the structure equal to 21.20m)
- *Case C:* frame structure without beams at the first floor (column - slab connection, Fig. 2) (all interstory heights equal to 3.2m and total height of the structure equal to 19.20m). As far as the authors know only once the influence of the floor slab on the seismic response of the structures with irregularities in elevation has been studied (Romão *et al.* 2004).

Masonry infills distribution

A parameter that can also change the first story morphology of a structure is the irregular distribution of the masonry infills. It is well known that the seismic performance of the RC frame structures is greatly influenced by the presence of the infills resulting in some cases to undesirable failure modes such as the development of a soft-story mechanism. This type of failure mechanism is typically occurred in frames in cases that infills are missing at first story level (pilotis type frame). Thus, the local response of the infills is also considered as a key parameter for the study of the seismic performance of the RC frame structures that include irregularities of the first story morphology. In this view, each of the above mentioned RC structural systems (Cases A, B and C) is studied as: (a) bare frame, (b) fully infilled frame (regular distribution of infills) and (c) infilled frame without infills at the first floor (pilotis type frame).

Nonlinear static pushover analyses

Nonlinear static analyses have been performed for the needs of this study adopting three different distributions of the seismic loading; (a) triangular load pattern, (b) uniform load pattern and (c) multimodal load pattern. The results obtained considering the uniform distribution were very close to the corresponding ones obtained using the multimodal distribution (see also Pereira *et al*. 2009). For this reason only the results provided by the analyses with triangular and uniform distributions are presented. 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}).

Structural performance levels under consideration

The effect of the first floor morphology on the attainment of the performance stages of the multistory RC frame structures (with and without infills) at global level and at the level of the structural members is also considered. At the level of the structure there is a lack of definition about performance levels in codes, thus different assumptions have been proposed in the literature

(Celarec *et al.* 2012, Tsonos 2007, 2010). In most cases displacement (or drift) and/or rotation criteria are used in order a specific structural performance level to be specified.

In terms of displacement's criteria the next four global level performance stages of interest are chosen to be examined:

(i) performance point as yielded by the Capacity Spectrum Method (ATC-40),

(ii) performance point (target displacement) according to Coefficient Method (FEMA 356),

(iii) the level of immediate occupancy that corresponds to a maximum interstory drift equal to 1% of the storey height (h_s) (ATC-40: IO level; 1% drift) and

(iv) top drift equal to 1% of the total height of the structures (h_{str}).

Rotational criteria are also taken into account and two more structural performance levels are defined. Assuming that the most critical column controls the global state of the structure the performance levels that are examined are:

(a) Significant Damage (SD) level when the rotation at any column exceeds the 75% of the ultimate rotation θ_u, and

(b) Near Collapse (NC) level when the rotation at any column exceeds the ultimate rotation θ_u.

In order to define the SD and NC global level the ultimate rotation (θ_u) of the critical each time column of the structure is evaluated based on the recommendations of Eurocode 8 - Part3 (EC8-part3). Thus, based on EC8-part3 the ultimate rotation θ_u is calculated by the following expression

$$\theta_u = \frac{1}{\gamma_{el}} 0.016 \cdot (0.3^{v}) \cdot \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225} \cdot \left(\frac{L_v}{h}\right)^{0.35} \cdot 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} \cdot (1.25^{100\rho_d}) \tag{1}$$

where γ_{el} is equal to 1.5, v is the normalized axial force, ω and ω' are the mechanical reinforcement ratio of the tension and compression longitudinal reinforcement, respectively, f_c and f_y are the concrete compressive strength and steel yield strength, ρ_{sx} is the ratio of the transverse steel parallel to the direction x of loading, α is the confinement effectiveness factor and ρ_d is the steel ratio of diagonal reinforcement in each diagonal direction.

Limit states of structural members

Acceptable limits at different performance levels are provided in modern codes for the structural members of the buildings. In this study the maximum acceptable rotations of the plastic hinges in beams and columns based on the recommendations of ATC-40 are considered in order to investigate the effect of the first floor morphology on the local response - performance level of the RC elements.

Further based on the requirements of EC8-part3 the limit states of near collapse (NC), significant damage (SD) and damage limitation (DL) are also defined for the RC members. The three limit states of DL, SD and NC for beams and columns in bending are reached when the developing rotation θ is equal to the yield rotation (θ_y), equal to the 75% of the ultimate rotation (θ_u) and equal to the 100% of the ultimate rotation (θ_u), respectively.

The EC8-part3 indicates that the yield rotation for the beams and the columns can be calculated as

$$\theta_y = \varphi_y \frac{L_v + \alpha_v z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_v}\right) + 0.13\varphi_y \frac{d_b f_y}{\sqrt{f_c}} \tag{2}$$

where, φ_y is the yield curvature, L_v is the shear span of the structural element, h is the section height, f_y is the steel yield stress, f_c is the concrete compressive strength and $\alpha_v z$ is the tension shift

of the bending moment diagram (it is equal to 0 if it is assumed that shear cracking does not precede flexural yielding at the end section). The first term in the above expression accounts for flexure, the second term for shear deformation and the third for anchorage slip of bars. In this study the bond conditions are assumed favorable, while the effect of the shear deformation on the results of this work is not considerable.
The rotation of plastic hinges θ_p is calculated by using the equation indicated in EC8-part3 as

$$\theta_p = \frac{1}{\gamma_{el}} 0.0145 \cdot (0.25^{v}) \cdot \left[\frac{\max(0.01; \omega')}{\max(0.01; \omega)}\right]^{0.3} \cdot f_c^{\ 0.2} \cdot \left(\frac{L_v}{h}\right)^{0.35} \cdot 25^{\left(\alpha \rho_{sx} \frac{f_{yw}}{f_c}\right)} \cdot (1.275^{100\rho_d}) \quad (3)$$

where, γ_{el} is equal to 1.8, v is the normalized axial force, ω and ω' are the mechanical reinforcement ratio of the tension and compression longitudinal reinforcement, respectively, f_c and f_y are the concrete compressive strength and steel yield strength, ρ_{sx} is the ratio of the transverse steel parallel to the direction x of loading, α is the confinement effectiveness factor and ρ_d is the steel ratio of diagonal reinforcement in each diagonal direction. Nevertheless, the length and the complex nature of expressions (1), (2), (3) rise equations for their practicability and do not really help towards an in depth understanding of the phenomenon they try to describe.

Thereafter comparisons in terms of rotations of plastic hinges between the acceptable criteria provided by ATC-40 and the limit states that are evaluated based on EC8-part3 is presented and commented. Moreover, the sensitivity of the results on the expected value of yield rotation of the beams and the columns is also examined.

Finally, it is well known that an important parameter that influences the local and global failure mechanisms of the RC structures is the beam – column joints capacity deterioration (Karayannis *et al*. 2011). In this study the joint damage effect has not been included and the response of the beam – column regions is assumed as rigid. Nevertheless, it has to be noted that a helpful tool for the assessment of the ultimate shear strength of the joints and therefore for the definition of the limit states of RC structures is the well established beam-column joint model by Tsonos (1999, 2002, 2007, 2010).

Performance points of the RC structures

The performance points (or target displacements) of the examined RC structures have been determined through the implementation of the nonlinear static analysis procedures of ATC-40 and FEMA356. Both procedures are globally established. In fact FEMA has been the basis for many similar assessment methods including the nonlinear static procedure recommended in EC8. Nevertheless, FEMA and ATC-40 are based on different concepts on the evaluation of the seismic displacement demand (performance point versus target point); this exactly is the reason for using these two methods in this study. It is pointed out that the ATC-40 method is based on the equivalent elastic spectrum concept for the determination of the seismic demand while FEMA is using coefficients for the modulation of the elastic demand to an inelastic one. For the purposes of this study inelastic pushover analyses are first carried out in order to derive the necessary capacity curve of each structure.

For the seismic assessment of the structures the elastic spectrum of EC8 for ground type B is adopted. Thus according to the EC8-part3 the demand is for Significant Damage limit state corresponding to ground motions with return periods of 475 years.

The estimated values of the performance points of all the examined frames with and without infills are presented in Table1. It can be observed that the performance point of the examined structures according to FEMA is reached at a higher displacement than the corresponding one estimated according to the provisions of ATC. In all the examined cases 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.

5. Seismic assessment of the structural systems - results

The seismic performance of a 6-story reinforced concrete frame structure in which all the floor levels have equal interstory heights (Case A) is investigated and compared with the corresponding response of a 6-story RC frame structure in which the floor height at the first story is greater than the one of the other stories (tall first story height: Case B) and with the corresponding response of a 6-story RC frame structure without beams at the first floor (column – slab connections: Case C). For the study of the first floor morphology effect on the seismic assessment of the structures the effect of the irregular distribution of the masonry infills along the height of the structure is also included. Two characteristic infilled structure modes are taken into account defined as: (a) fully infilled frame and (b) infilled frame without infills at the first floor (open ground floor: pilotis type frame). The case of bare frame is also taken into account. This way a set of nine 6-story structural systems-models is examined.

Nonlinear static analyses have been performed for the needs of this study and results based on triangular and uniform load pattern distributions are presented. In this study top drift equal to 1% is accepted as the end of pushover analyses.

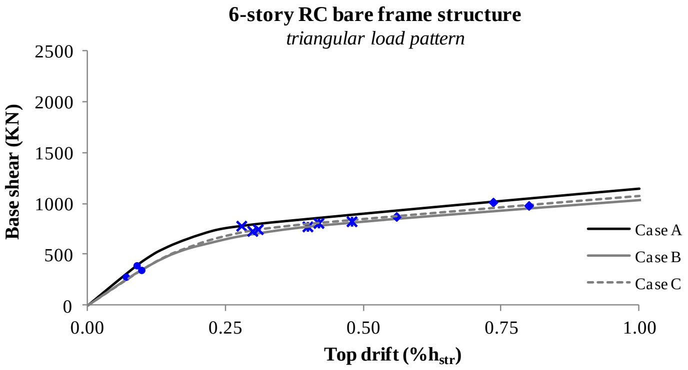
5.1 Effect of the first floor morphology on the global capacities

Comparative presentations of the capacity curves in terms of global base shear-top drift $\%h_{str}$ of the examined structural systems (Case A, Case B and Case C) with and without infills are shown in Fig. 4 for triangular consideration of the seismic load distribution. It can be observed that the global capacities of the 6-story frame structures with first floor morphology irregularities (Case B and Case C) are decreasing in comparison to the corresponding capacities of the frames with equal interstory heights in all the floor levels (Case A). The lower capacity on base shear was observed in the case of 6-story frame with first floor height greater than the one of the other stories (Case B; with and without infills). This reduction on the global responses of the 6-story RC frames due to the differences on the first floor morphology is more intense when infills are incorporated in the analysis model.

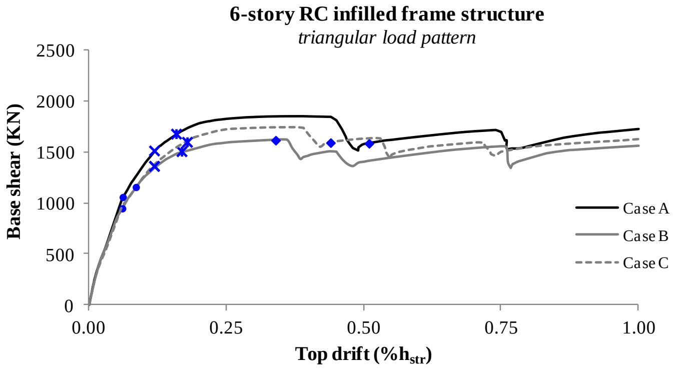
Abrupt degradation points of the load carrying capacity of the infilled frames are observed in the capacity curves that are attributed to failures of infills. The first floor irregularities cause changes on the sequence of infills collapse (failure point) and on the damage distribution of the beams and columns all over the structures. The influence of the first floor morphology on the local response of the infills and on the observed failure modes of the structures are presented and discussed in details in the following paragraphs.

The Fig. 4(c) is also shown that the different type of first floor morphology has as a result a reduction of the initial stiffness of the pilotis type structures in Cases B and C in comparison to the case that the only irregularity of the structure is the absent of infills at the first story (pilotis type–Case A). Nevertheless as it was expected an increase of the global stiffness and strength of the structures due to the presence of the masonry infill panels is observed in Fig. 4.

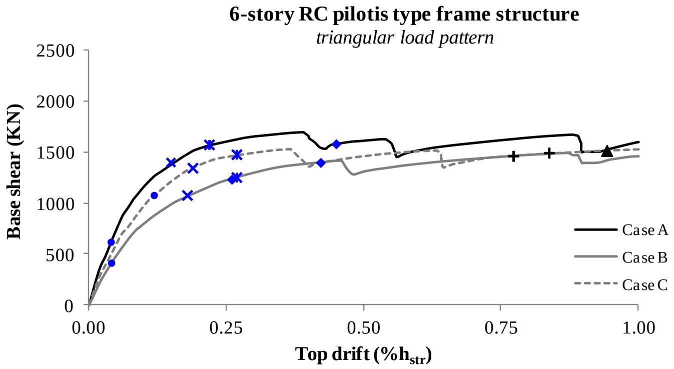
In Table 1 results about the effect of the first floor morphology on the attainment of the



(a) Bare frame structure



(b) Fully infilled frame structure



(c) Pilotis type frame structure (without infills at the first floor)

● *1st yield of any member*

<u>*Structural performance levels based on:*</u>

(i) Displacement criteria:
× performance point to ATC-40
⁎ target displacement (performance point) to FEMA356
◆ 1% interstory drift (ATC-40: Immediate occupancy level)

(ii) Rotation criteria:
+ Significant Damage level (EC8-part3)
▲ Near Collapse level (EC 8-part3)

Fig. 4 Influence of the first floor morphology on the capacity curves of the 6-story RC frame structural systems with and without infills (for triangular load pattern)

Table 1 Influence of the first floor morphology on the attainment of the predefined performance states of the multistory RC frame structure (with and without infills) at the global level. Results in terms of developing top drift (%h_{str}) and base shear to design base shear

		Structural System Case A						***Structural System Case B***						***Structural System Case C***					
		bare frame		***infilled frame***		***pilotis type***		***bare frame***		***infilled frame***		***pilotis type***		***bare frame***		***infilled frame***		***pilotis type***	
		TD (1) **[Δ_{roof}]**	***V/V_{des}*** (2)	***TD*** **[Δ_{roof}]**	***V/V_{des}***	***TD*** **[Δ_{roof}]**	***V/V_{des}***	***TD*** **[Δ_{roof}]**	***V/V_{des}***	***TD*** **[Δ_{roof}]**	***V/V_{des}***	***TD*** **[Δ_{roof}]**	***V/V_{des}***	***TD*** **[Δ_{roof}]**	***V/V_{des}***	***TD***	***V/V_{des}***	***TD*** **[Δ_{roof}]**	***V/V_{des}***
LP*	***PLs***+	***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**		***(%h_{str})*** **[cm]**	
TRIANGULAR	*FEMA*	0.40 [7.6]	1.29	0.16 [3.07]	2.81	0.22 [4.27]	2.64	0.42 [8.92]	1.35	0.17 [3.69]	2.52	0.27 [5.72]	2.10	0.48 [8.6]	1.38	0.18 [3.37]	2.68	0.27 [5.21]	2.48
	ATC 40	0.28 [5.38]	1.30	0.12 [2.30]	2.54	0.15 [2.96]	2.35	0.30 [6.38]	1.22	0.12 [2.61]	2.28	0.18 [3.9]	1.80	0.31 [5.95]	1.25	0.14 [2.66]	2.46	0.19 [3.56]	2.25
	1% interstory drift	0.73 [14.1]	1.70	0.51 [9.79]	2.66	0.45 [8.64]	2.66	0.56 [11.9]	1.47	0.34 [7.21]	2.72	0.26 [5.51]	2.07	0.80 [15.4]	1.65	0.44 [8.45]	2.67	0.42 [8.10]	2.35
	SD (0.75θ_u)	-	-	-	-	-	-	-	-	-	-	0.77 [16.4]	2.45	-	-	-	-	0.84 [16.1]	2.50
	NC (θ_u)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0.94 [18.1]	2.55
	1% top drift	1.00 [19.2]	1.92	1.00 [19.2]	2.89	1.00 [19.2]	2.69	1.00 [21.2]	1.80	1.00 [21.2]	2.61	1.00 [21.2]	2.44	1.00 [19.2]	1.80	1.00 [19.2]	2.72	1.00 [19.2]	2.57
UNIFORM	*FEMA*	0.41 [7.9]	1.79	0.16 [3.05]	3.25	0.23 [4.38]	3.00	0.42 [8.87]	1.54	0.19 [3.97]	2.70	0.27 [5.74]	2.22	0.46 [8.75]	1.70	0.19 [3.57]	3.12	0.28 [5.32]	2.78
	ATC 40	0.26 [4.99]	1.57	0.10 [1.92]	0.70	0.13 [2.5]	2.47	0.28 [5.94]	1.32	0.10 [2.18]	2.30	0.18 [3.82]	1.88	0.31 [5.95]	1.51	0.11 [2.18]	2.61	0.17 [3.26]	2.43
	1% interstory drift	0.74 [14.2]	2.14	0.43 [8.26]	3.36	0.42 [8.06]	3.00	0.44 [9.33]	1.57	0.29 [6.08]	2.87	0.24 [5.09]	2.13	0.66 [12.7]	1.91	0.41 [7.79]	3.02	0.41 [7.87]	2.64
	SD (0.75θ_u)	-	-	0.85 [16.4]	3.39	0.60 [11.5]	3.24	-	-	0.76 [16.1]	2.81	0.57 [12.1]	2.44	-	-	0.66 [12.7]	3.04	0.55 [10.6]	2.79
	NC (θ_u)	-	-	0.96 [18.4]	3.47	0.81 [15.6]	3.21	-	-	0.90 [19.1]	2.90	0.69 [14.6]	2.54	-	-	0.87 [16.7]	2.99	0.60 [11.5]	2.82
	1% top drift	1.00 [19.2]	2.40	1.00 [19.2]	3.49	1.00 [19.2]	3.34	1.00 [21.2]	2.07	1.00 [21.2]	2.73	1.00 [21.2]	2.73	1.00 [19.2]	2.20	1.00 [19.2]	3.04	1.00 [19.2]	2.84

** LP: Load pattern distribution of seismic force*

+PLs: Performances levels at global

SD: Significant damage level (EC8-part3)

NC: Near collapse level (EC8-part3)

(1) TD: Top drift

(2) V: Base shear, V_{des}: Design base shear

predefined performance states of the multistory RC frame structure (with and without infills) at the global level are presented in terms of developing top drift (%h_{str}) and ratio of base shear to design base shear. In Fig. 4 the structural performance levels are also depicted on the capacity curves of the structures. The point of the first yield of a structural member is also shown on the capacity curves of the 6-story frame structures (Fig. 4).

As it can be observed in Fig. 4 and in Table 1 the values of the performance points of the examined structures defined according to the nonlinear static analysis methods of ATC-40 and FEMA356 were not significantly influenced by the first floor morphology. Different values of performance points for the examined structural systems are mainly obtained in the cases that the initial stiffness of the structure was changed. This observation mainly holds for the case of the 6-story pilotis type frame structure where the initial stiffness is reduced (Case B and Case C) in comparison to the initial stiffness of the corresponding structural system with equal interstory heights (Case A).

Nevertheless, the presence of infills slightly changed the position of the performance points for all the examined structures. It is stressed that strength and stiffness degradation of infills in all the examined cases are observed after the structural performance points according to FEMA356 and ATC-40.

Moreover, Fig. 4 and Table 1 demonstrate that in the examined 6-story infilled frame structures (fully infilled and pilotis type) with structural systems the ones of Case A and Case C the structural performance level of immediate occupancy (IO level; 1%drift) is reached after the first abrupt degradation of the capacity curve (i.e. after some infills degraded their stiffness and strength). However the 6-story frame structure in the Case B (height at the first floor greater than the one of the other stories) has attained this performance level before the collapse of any infill panel.

The performance levels that are based on displacement criteria are attained at smaller top displacement in comparison to the ones that are obtained in the cases of bare frame structural systems due to the presence of the masonry infills.

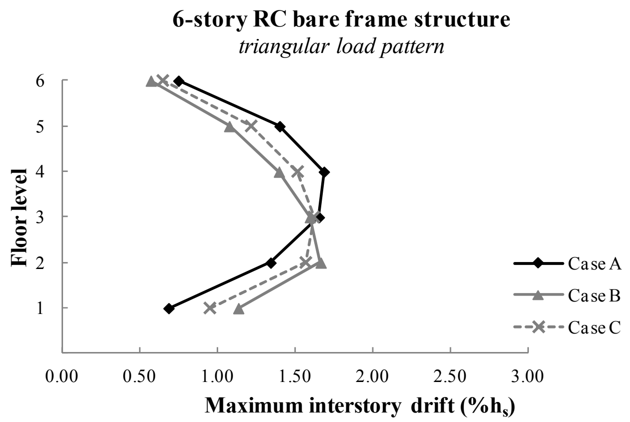
Interesting results are drawn when rotation criteria are used in order to define the structural performance levels of Significant Damage (SD) and Near Collapse (NC) of the multistory RC frame structures.

In all the examined cases when the 6-story RC frame was studied without infills the levels of SD and consequently the level of NC had not been reached for any of the three structural systems of Case A, Case B and Case C until the end of the pushover analysis.

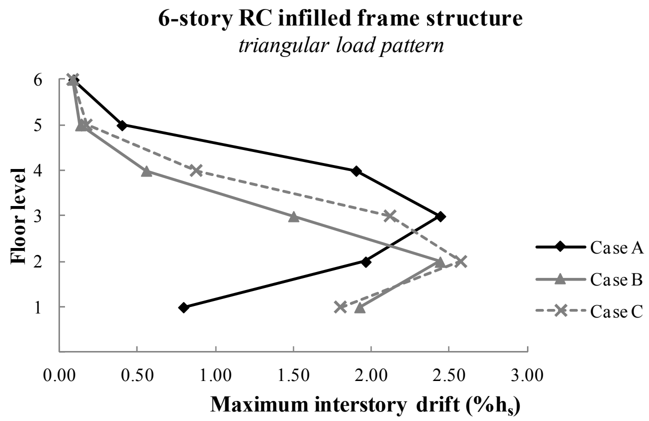
However, when the local response of the infills is incorporated in the analysis model of the multistory RC frame structure the seismic performance at the global level of the structures seems significantly influenced by the irregularities at the first floor. As it can be observed for the 6-story fully infilled frame without irregularities at the first floor (Case A) in Fig. 4(b) the capacity level of Significant Damage (SD) has not been reached until the end of the analysis (triangular seismic load distribution).

The additional irregularity at the first floor due to the pilotis type morphology has as a result the 6-story frames with structural systems these of Case B and Case C to reach the performance level of SD at top drifts to 0.77%h_{str} and 0.84%h_{str}, respectively. The performance level of Near Collapse NC is attained by the pilotis type structures of Case C at top drift equal to 0.94%h_{str} (see also Table 1).

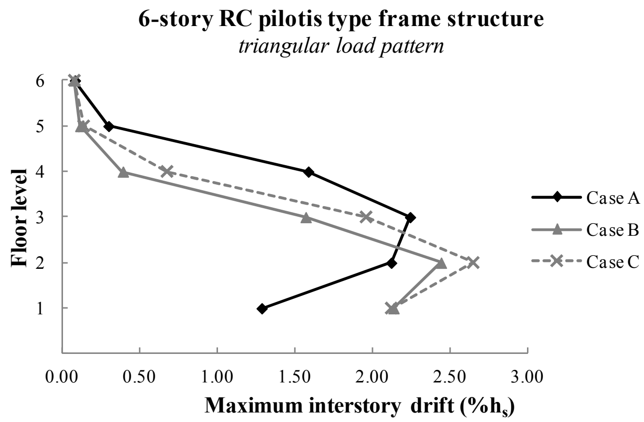
More critical has been proved to be the seismic performance of the infilled RC structures when the nonlinear static analyses are performed considering uniform load pattern of the seismic forces



(a) Bare frame structure



(b) Fully infilled frame structure



(c) Pilotis type frame structure (without infills at the first floor)

Fig. 5 Influence of the first floor morphology on the maximum interstory drifts of the 6-story RC frame structural systems with and without infills at top drift equal to 1%h_{str} (for triangular load pattern)

(Table1). The results of these analyses indicate that in all the examined cases of infilled (regular and pilotis type) structural systems (Case A, Case B and Case C) attain the performance levels of SD and NC at earlier stages in comparison to the corresponding results based on triangular load pattern.

The 6-story frame structure without beams at the first floor (Case C) exhibits the most critical global response in comparison to the other structural systems (Cases A & B), while the consideration of pilotis type first floor morphology yielded unfavorable capacity responses (Fig. 4, and Table 1).

5.2 Maximum interstory drift demands

The maximum interstory drifts of the 6-story RC frame structures with irregularities at the first floor morphology (Case B and Case C) are presented and compared with the corresponding responses of the frame structure of Case A with and without the effect of the masonry infills in Fig. 5 for triangular load pattern. These results comprise the interstory drift performances at top drift equal to 1% h_{str}.

It can be observed that the first floor irregularity causes an increase on the demands for interstory drift at the first story of the structures in comparison with the corresponding requirements of the structures without irregularities. Greater demands for interstory drifts at the first story are observed for the structure with tall first story height (Case B). In the case of structure with slab-column connections without beams at the first floor level (Case C) the interstory drifts of the 2nd floor level are the highest comparing to the ones of the other stories.

As expected the case that the open ground floor (pilotis) and the soft story simultaneously exist at the first floor exhibits the worst performance of all the examined cases in terms of maximum requirements for interstory drift at the first story (Fig. 5(c)).

A critical increase of the developed interstory drifts at the lower floor levels of the structures is observed due to the presence of the infills since the value of 2% interstory drift that corresponds to the Life Safety structural performance level (ATC-40) is exceeded. Nevertheless at the upper floor levels (5th-6th) of these structures the developed interstory drifts are very small compared with the corresponding drifts of the bare frames.

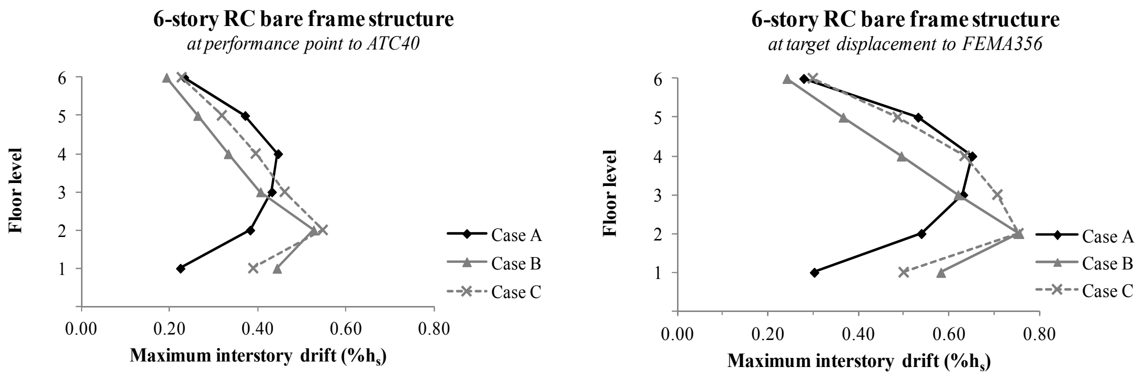
Similarly, the maximum interstory drift requirements at the performance points (ATC and FEMA) of the 6-story frame structures, before any infill collapses, are presented in Fig. 6, for the examined cases with triangular load pattern. An increase of the demands for interstory drift is observed at the first floor level due to the considered irregularities.

On the contrary to the afore-mentioned results, it can be observed that the regular distribution of the infills along the height of the structures resulted to smaller interstory drift values compared to the corresponding values that are developed in the case of the bare frame building (Figs. 6(a) and 6(b)). However, the form of an open ground floor structure (pilotis type) led to high values of interstory drifts at the first story (Fig. 6(c)).

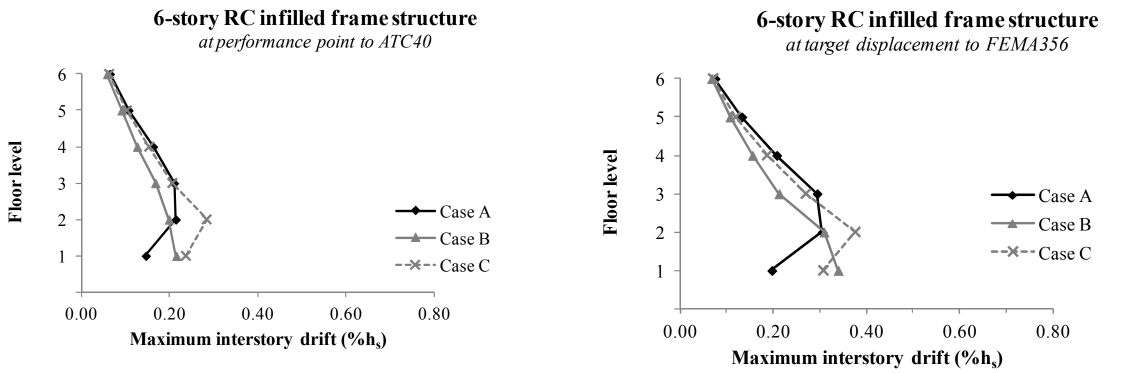
Finally, in all cases the nonlinear static analysis using uniform distribution of the seismic load yielded increased values for the lower floor levels in terms of maximum requirements for interstory drift.

5.3 Effect of the first floor morphology on the local responses

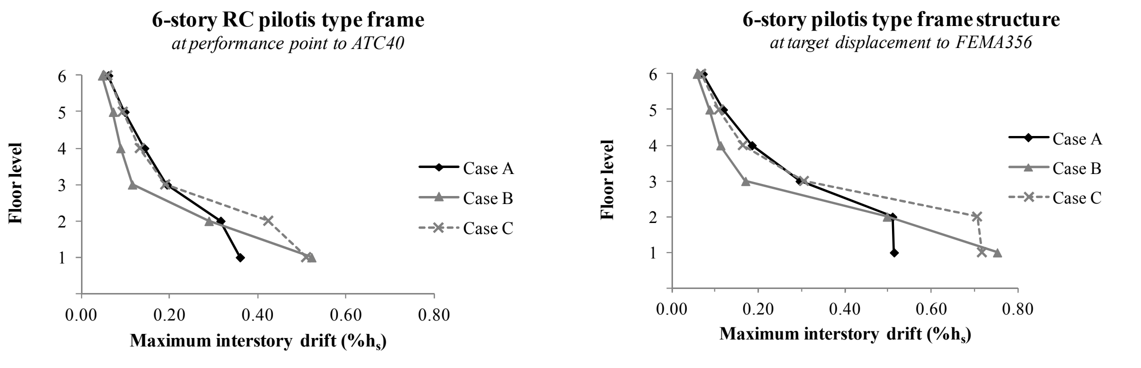
The influence of the first floor morphology irregularities on the inelastic demands of the columns and the beams of the 6-story RC structures is also studied.



(a) Bare frame structure



(b) Fully infilled frame structure

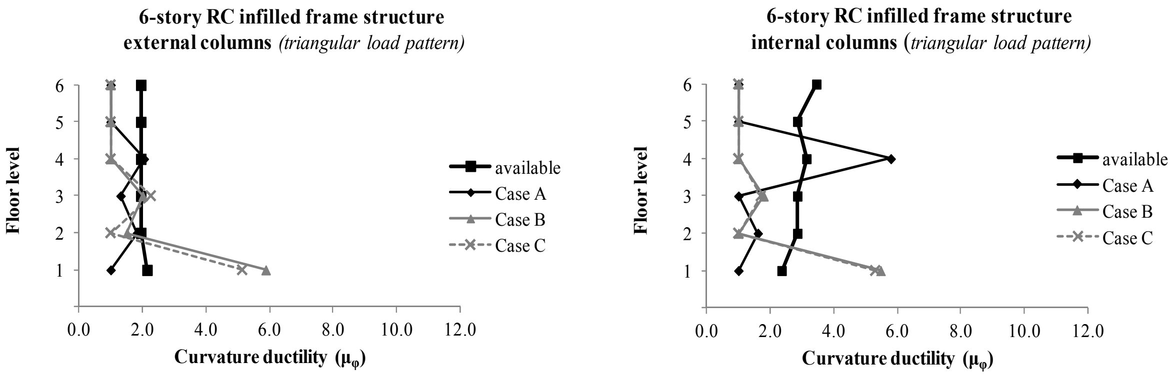


(c) Pilotis type frame structure (without infills at the first floor)

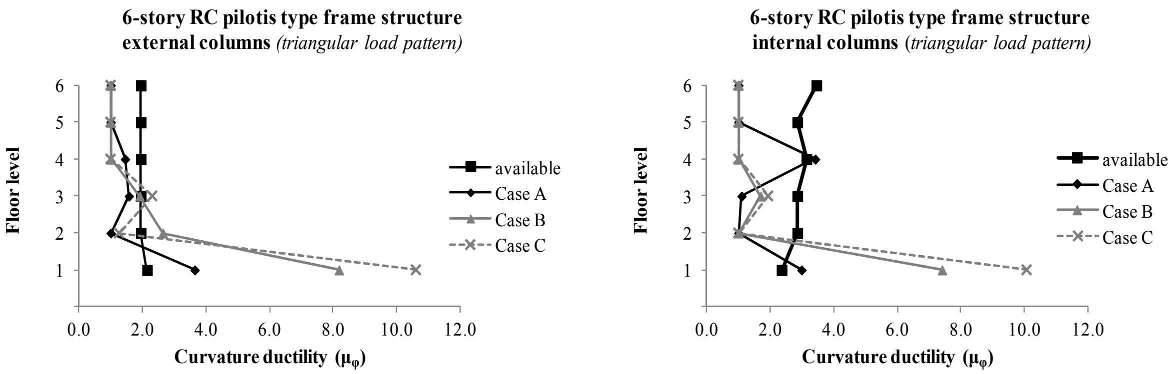
Fig. 6 Influence of the first floor morphology on the maximum interstory drifts of the 6-story RC frame structural systems with and without infills (for triangular load pattern); (i) at performance point to ATC40 and (ii) at target displacement to FEMA356

Maximum curvature ductility requirements of the columns

In Fig. 7 comparative results of the interstory maximum curvature ductility requirements of all the columns of the three examined structural systems (Case A, Case B and Case C) are presented,



(a) Fully infilled frame structure



(b) Pilotis type frame structure (without infills at the first floor)

Fig. 7 Influence of the first floor morphology on the curvature ductility requirements of the columns of the 6-story frame structures at the point of top drift 1%h_{str} (for triangular load pattern)

for triangular load pattern consideration. The maximum developing demands of columns are compared with the corresponding available capacities in terms of ductility. The results clearly demonstrate that at the point of top drift equal to 1%h_{str} the demands of the columns at the first floor are critically increased due to the first morphology irregularities in comparison to the corresponding demands of the same columns of the structures of Case A (case without first floor morphology irregularities).

It is also noted that in cases where the 6-story structure is studied without irregularities (Case A without infills) all the columns remain in the elastic range.

The first floor irregularity of Case A-pilotis type, Case B-types with infills and Case C-types with infills seems to have led to the development of soft-story mechanism at the base floor story (see Fig. 8).

Further the most critical type of first floor morphology for the seismic performance of the columns at the first story has been proved to be the one of Case C since in this type the demands for ductility in the columns are greater than the ones observed in the morphology cases of Case B and Case A.

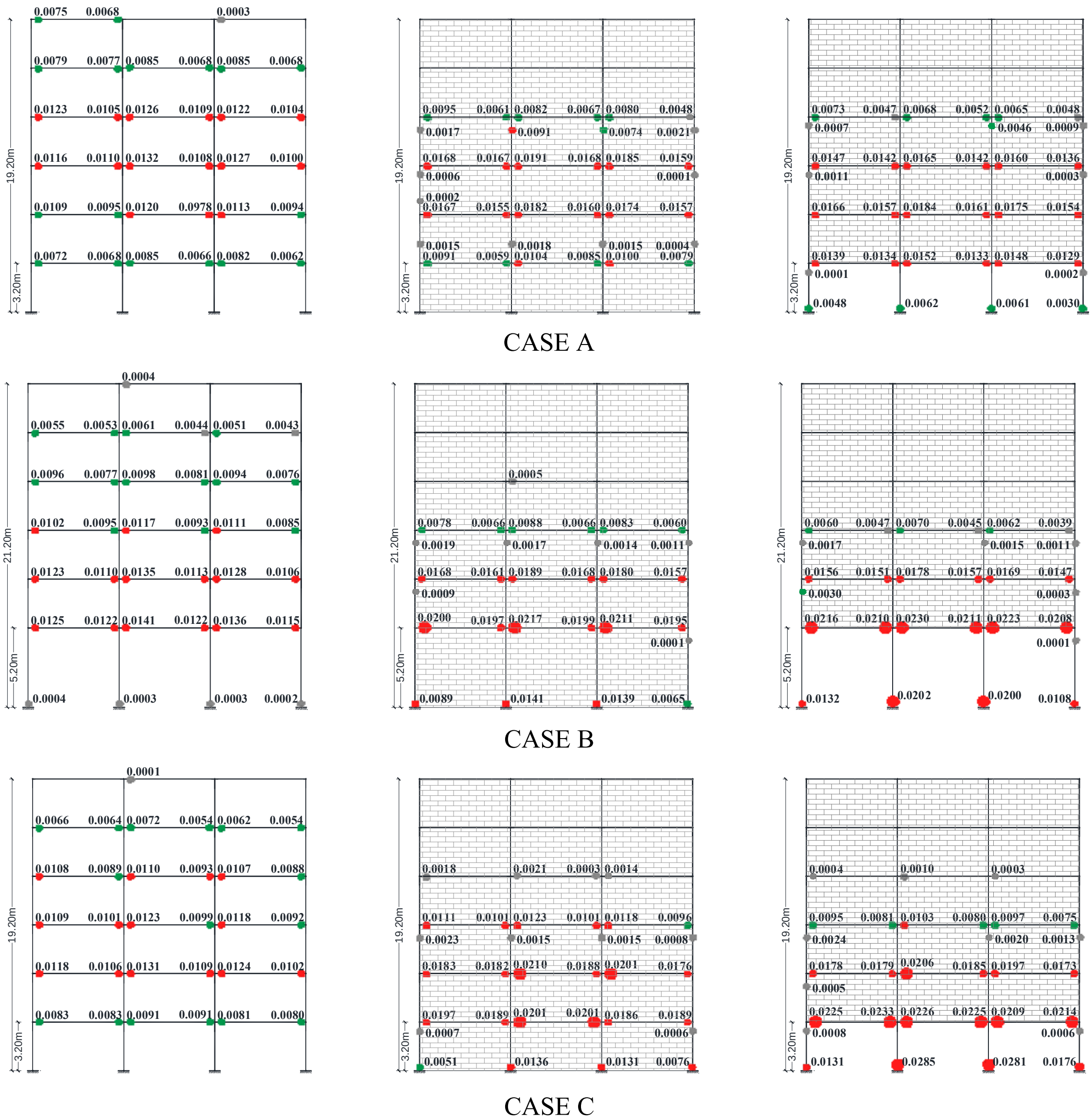
Failure modes – Limit states

In Fig. 8 the failure modes of all the examined structures (Cases A, B and C) with and without infills at the performance level of top drift equal to 1% h_{str} are presented for triangular load pattern. The maximum values of plastic hinge rotations θ_p and the corresponding limit states of the structures are also depicted in Fig. 8. The local limit states of structural stability (SS), life safety (LS) and immediate occupancy (IO) are defined based on the requirements of ATC-40. It is noted that the acceptable limits of beams are also adopted for slab zone at the first floor of the structural system of Case C (case without beams at the first floor).

The requirements for inelastic response of beams and columns are mainly concentrated at the floor levels where strength and stiffness degradation of the infills is also occurred. Moreover, although the development of soft-story mechanism can be considered frequent for pilotis type infilled frames the results of this study indicate that this mechanism can also be occurred in fully infilled frame structures.

The influence of infills on the plastic rotations that are developed at every deformation step of the analyses is also examined taking into account the acceptable rotation levels of the plastic hinges at the three limit states of SS, LS and IO (ATC-40). Moreover, the acceptable rotational limits of the most critical members of the examined structures are evaluated based on the recommendations of EC8-part3. In order to define the EC8-part3 limit states two different values of yield rotation (θ_y) for the beams and the columns are assumed. In particular, the first value of θ_{y1} is determined using the Eq. (2) (with the value of curvature φ_y to be estimated using a section analysis program), while the other value of yield rotation θ_{y2} is defined as the difference of θ_u minus θ_p, calculated using the aforementioned in section 4 Eqs. (1) and (3), respectively. Thereafter, the corresponding to EC8-part3 limit states are indicated as NC1, SD1 and DL1 for the case of using the yield rotation θ_{y1}, and NC2, SD2 and DL2 for the case of using the value of yield rotation θ_{y2}.

The acceptable plastic rotational limits that are given by ATC-40 and EC8-part3 are presented in Fig. 9 and are compared to the plastic rotational demands of the most critical elements of the frame of Case A at every deformation step of the analyses. Thus, in the case of adopting triangular load pattern distribution only the internal column at the 4th floor level (C14) of the fully infilled structure (Case A) exhibited inelastic behaviour and therefore the ATC-40 Life Safety (LS) limit state has been exceeded whereas the corresponding EC8-part3 Significant Damage (SD) limit state has not been reached (Fig. 9(a)2). Moreover, the beam of the 2nd storey level of the fully infilled frame structure of Case A developed the most critical demands for plastic rotation (Fig. 9(a1)). This beam (B8) almost reached the ATC-40 Structural Stability (SS) limit state whereas it has not reached the criteria to EC8-part3 Near Collapse (NC) limit state (Fig. 9(a1)). The requirements of the beam B8 for plastic rotation are increased in the case with the infills in comparison to the requirements that this element (B8) develops in the case of bare frame structure. Moreover, based on the results shown in Fig. 9(a1) it can be observed that the demands for plastic rotation of the beam B8 reached the ATC-40 LS limit state and the corresponding EC8-part3 SD limit state in the case of considering the local effect of the infills (cases of fully infilled and pilotis type frames) while in the case of bare frame only the ATC-40 LS limit state has been reached. Nevertheless, demands for plastic rotation of column C2 at the first floor of the infilled frame structures for uniform load pattern exceeds both the ATC-40 Structural Stability (SS) limit state and the corresponding EC8-part3 Near Collapse (NC) limit state (Fig. 9(b)). It is also noted that column C2 at the first floor of pilotis type frame structure attained the limit states of SS and NC earlier than the corresponding column of the fully infilled structure while the same column of the bare frame remained elastic throughout the analysis.



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

Beams

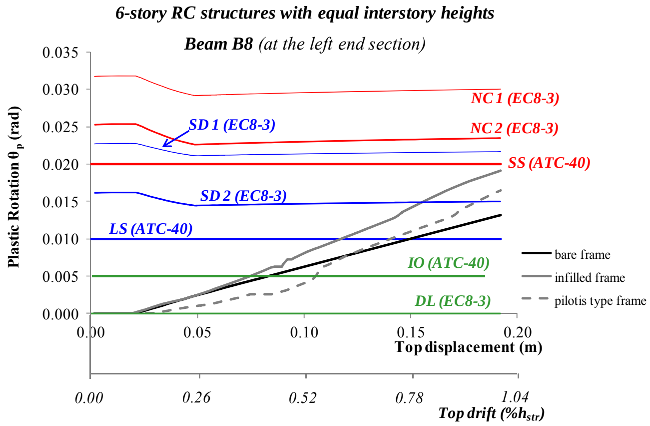
- SS (structural stability): $\theta_p = 0.02$
- LS (life safety): $\theta_p = 0.01$
- IO (immediate occupancy): $\theta_p = 0.005$
- $\theta_p < 5^0/_{00}$

Columns

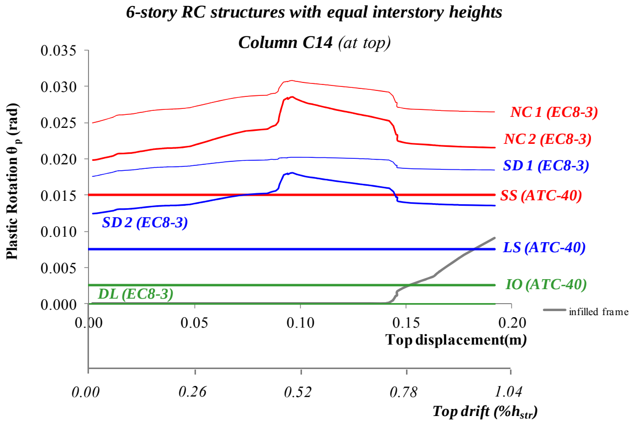
- SS (structural stability): $\theta_p = 0.015$
- LS (life safety): $\theta_p = 0.0075$
- IO (immediate occupancy): $\theta_p = 0.0025$
- $\theta_p < 2.5^0/_{00}$

Fig. 8 Maximum rotations of plastic hinges θ_p and corresponding performance levels in beams and columns of the 6-story frame structures at top drift equal to 1% h_{str} (for triangular load pattern)
*colour figure available online

It is stressed that the values of EC8-part3 limit states of NC and SD for the examined elements are greater than the corresponding values of limit states according to ATC-40. The demands for

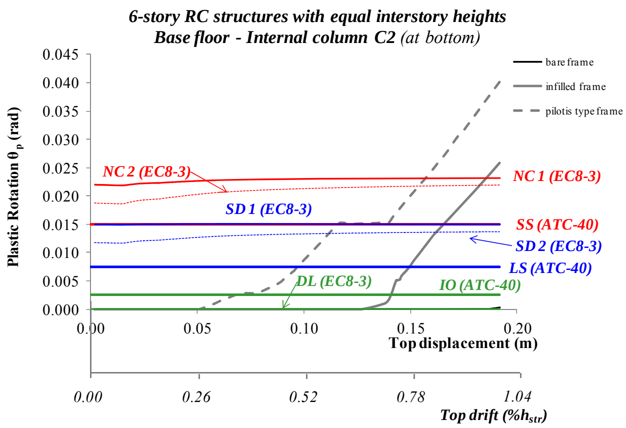


(1)



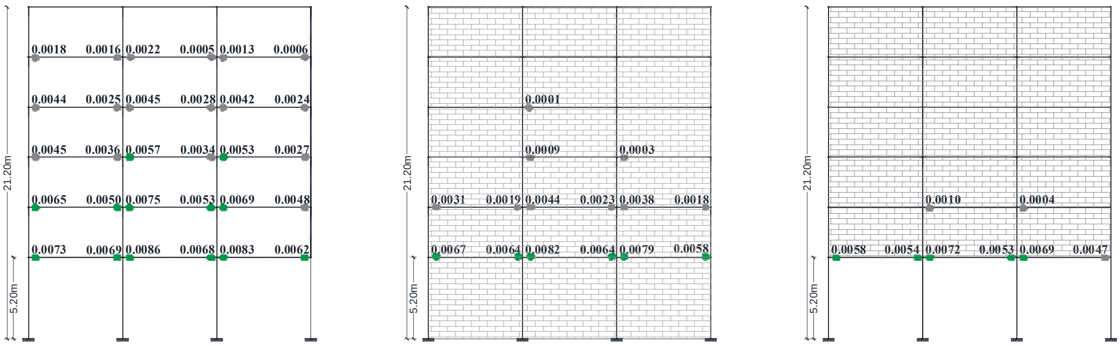
(2)

(a) Triangular load pattern distribution

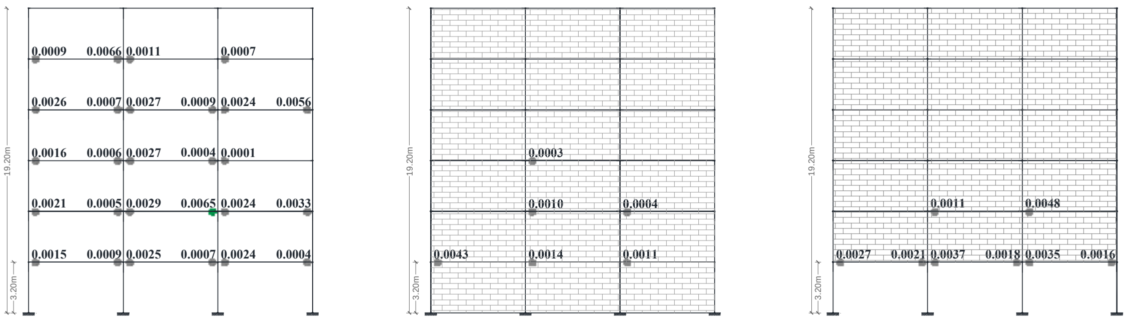


(b) Uniform load pattern distribution

Fig. 9 Limit States according to ATC-40 and to EC8-part3. Plastic hinge rotations θ_p developed in critical members of the 6-story RC structures with equal interstory heights (Case A) during the nonlinear static analyses (column C14, beam B8 for triangular load pattern and column C2 for uniform load pattern)
*colour figure available online



(a) 6-story frame structures with structural system that of CASE B; at interstory drift equal to 1% h_s



(b) 6-story frame structures with structural system that of CASE A; at performance point to ATC-40

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

<u>*Beams*</u>

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

<u>*Columns*</u>

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

Fig. 10 Contribution of the infills strength capacity - before collapsing - on the developed maximum requirements for plastic rotation θ_p of the beams and the columns of the 6-story frame structures (for triangular load pattern)
*colour figure available online

plastic rotation capacity are greater in the case of yield rotation θ_{y1} than in the case of yield rotation θ_{y2}, and the EC8-part3 limit states NC2 and SD2 are close to the corresponding SS and IO limit states of ATC-40 (Fig. 9).

Any irregularity (of the examined types) at first floor of the 6-story bare frame structure has as a result an increase of the flexural requirements of the beams and the columns at the lower floor levels in comparison to the corresponding requirements that are developed in the case of the bare frame structure of Case A (regular first floor morphology). As it has already been mentioned the most critical beam of the 6-story RC frame structures of Case A has been proved to be beam B8. The presence of the infills caused an increase of the demands for rotation of the plastic hinges.

Critical for the cases of pilotis type frame structures has been proven to be the beam B2 at the first floor. The column C2 at the first floor of the 6-story pilotis type frame structures with first floor irregularities (Case B and Case C) exhibits increased demands for rotation that exceeding the limit states of SS (ATC-40) and NC (EC8-part3). Although this column (C2) is also the most critical column of the pilotis type structure of Case A it is noted that the level of Life Safety (LS of ATC-40) or the level of Significant Damage (SD of EC8-part3) have not been reached.

Nevertheless, it is worth to be noted that when the performance of the structures is studied at an early state of the response where infills have not been collapsed yet the demands for inelastic response of the members are negligible or even zero due to the presence of the infills. Cases that yielded this type of seismic performance are:

(a) The 6-story infilled frames (fully infilled and pilotis type) with structural system that of Case B (height at the first story is greater than the one of the other stories) at the global level of 1%h_s interstory drift (see Figs. 10(a) and 4) and

(b) All the examined infilled frame structures (Cases A, B and C; fully infilled and pilotis type frames) at the performance points to ATC-40 and FEMA as it can be observed in Fig. 10(b) for the 6-story frame structure of Case A.

5.4 Effect of the first floor morphology on the local response of the infills

The local inelastic responses of the infill panels of the fully infilled 6-story frame structure with equal interstory heights (Case A) are presented in Fig. 11(a), in terms of compressive axial deformations vs. top drift (%h_{str}) ratio of the structure until the 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 ($^0/_{00}$) at maximum strength (RL3), (b) deformation level ($^0/_{00}$) at strength and stiffness degradation (RL4) and (c) ultimate deformation level ($^0/_{00}$) – infill collapse (RL5) (see Fig. 3(b)). This way a direct comparison between demands and capacities of the masonry infills is provided while useful comments can be deduced about the seismic performance of the infills along the capacity curves of the structures that are also presented in Fig. 11(b).

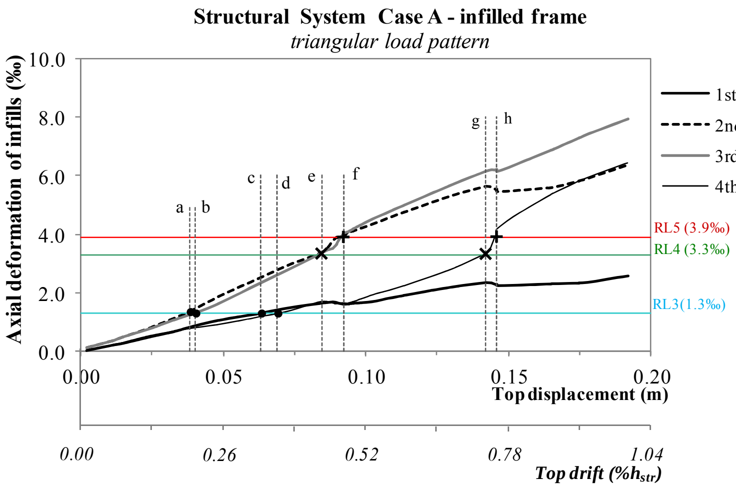
At the structural performance level of top drift 1% h_{str} as it can be observed in Fig. 11, the infill panels of 2nd, 3rd and 4th stories of the 6-story fully infilled structure have failed since the developing demands for deformations exceeded their ultimate capacity. Damages (strength and stiffness degradation) have also been occurred at the 1st floor level of the examined building.

It is noted that the failures of the infills of the 2nd and 3rd floor levels of the structure have already been occurred at the performance level of 1% interstory drift, while these infills at the performance points to ATC and FEMA only some first cracks have been developed.

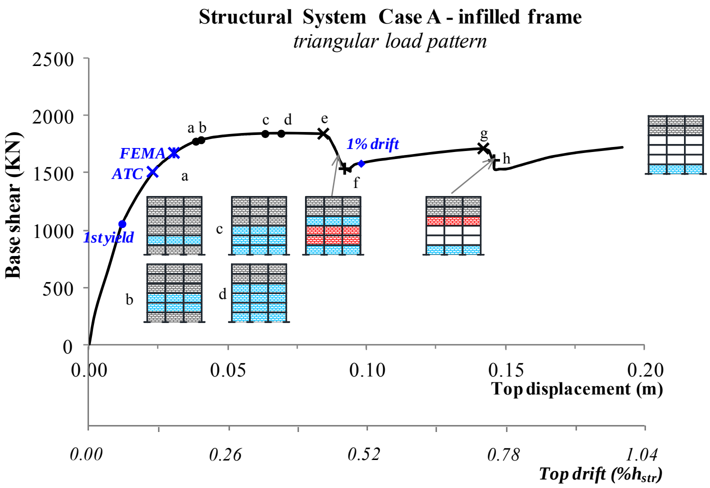
Similar remarks are obtained when uniform distribution of the seismic load is adopted for the analyses, although the observed damages on the infills have different distribution along the height of the structure.

Similarly to the case of the fully infilled structure in the case of pilotis type frame structure (Case A) high deformation demands are developed at the infills of the 2nd, 3rd and 4th story level of the structure (Fig. 13). 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. In particular, the infills of 2nd to 4th floor of the pilotis type frame have totally failed at the structural performance level of top drift 1% h_{str} since the developing demands for deformations exceed their ultimate capacity. Nevertheless, considering the pilotis structure at the performance level of 1% of the interstory drift only the infills of the 2nd floor have exceeded the local response limit RL5 that corresponds to ultimate deformation of infill collapse (RL5). Finally it is noted that at the performance points to ATC and FEMA only, some degree of cracking on the infills is occurred.

The results demonstrate that although the seismic response of the infills at the end of the



(a)



(b)

Colours of infills

infill intact - DS stress under RL1 (see Fig. 3)

DS deformation reaches deformation at top stress strength (RL3) - initation of strength degratation.

collapsing stage (totally colapsed - no longer exists) - DS deformation reaches the full collapsing deformation.

infill no longer exists

DS: diagonal strut

Fig. 11 Local inelastic responses of the infill panels of the fully infilled 6-story frame structure of Case A (for triangular load pattern)
*colour figure available online

nonlinear static analyses seem not to have been influenced by the type of the first floor morphology this observation cannot be obtained when the performance of the structure is

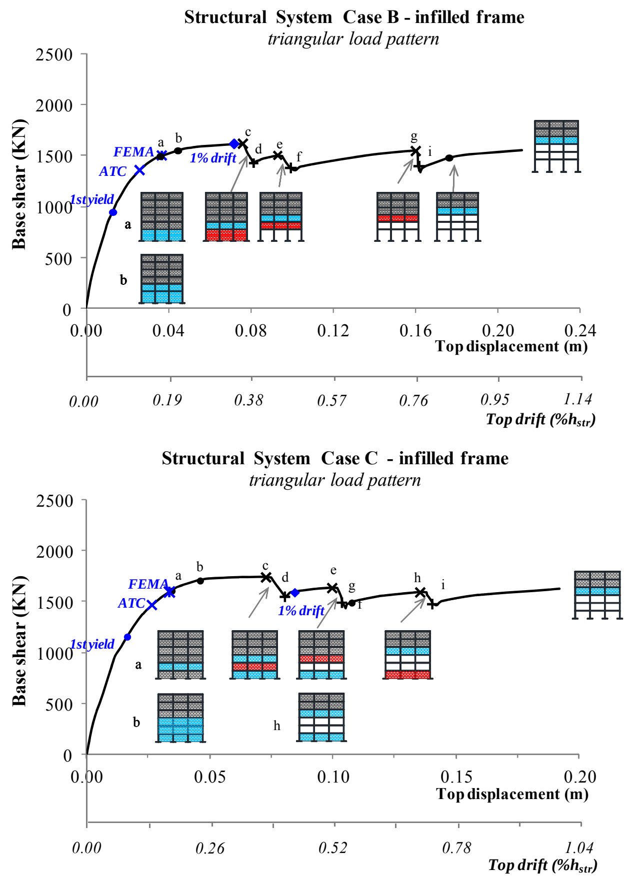


Fig. 12 Local inelastic responses of the infill panels of the fully infilled 6-story frame structures of Case B and Case C (for triangular load pattern) (see notation of Fig. 11 for colours of infills)
*colour figure available online

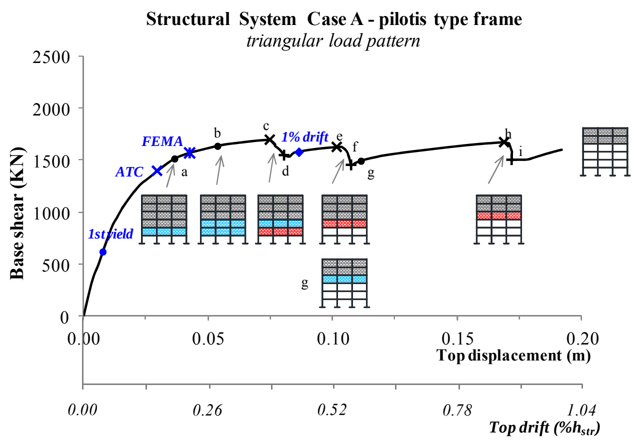
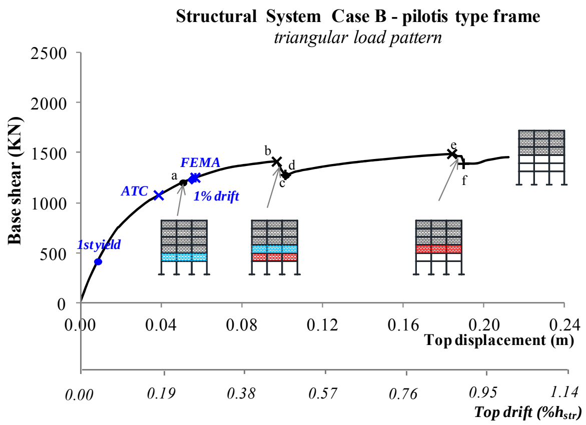


Fig. 13 Local inelastic responses of the infill panels of 6-story pilotis type frame structures of Case A, Case B and Case C (for triangular load pattern) (see notation of Fig. 11 for colours of infills)
*colour figure available online



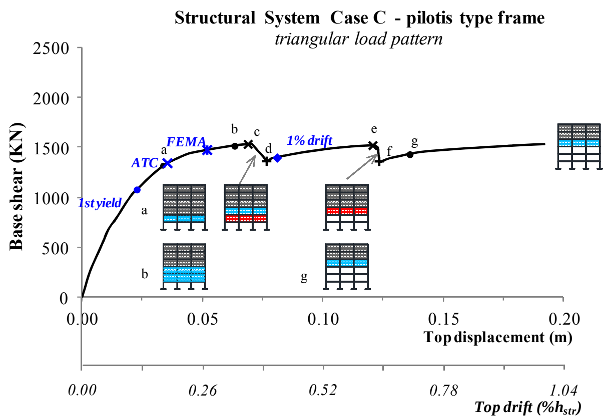


Fig. 13 Continued

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

	Structural type Case A							
	Triangular load pattern				*Uniform load pattern*			
response levels RL†	*RL1**	*RL3*	*RL4*	*RL5*	*RL1*	*RL3*	*RL4*	*RL5*
fully infilled frame	0.072	0.213	0.454	0.485	0.050	0.174	0.370	0.400
pilotis type frame	0.072	0.198	0.392	0.423	0.050	0.170	0.360	0.400
	Structural type Case B							
	Triangular load pattern				*Uniform load pattern*			
response levels RL†	*RL1**	*RL3*	*RL4*	*RL5*	*RL1*	*RL3*	*RL4*	*RL5*
fully infilled frame	0.060	0.170	0.360	0.385	0.054	0.150	0.320	0.340
pilotis type frame	0.088	0.240	0.460	0.480	0.100	0.053	0.460	0.490
	Structural type Case C							
	Triangular load pattern				*Uniform load pattern*			
response levels RL†	*RL1**	*RL3*	*RL4*	*RL5*	*RL1*	*RL3*	*RL4*	*RL5*
fully infilled frame	0.050	0.180	0.380	0.420	0.050	0.168	0.480	0.510
pilotis type frame	0.050	0.176	0.360	0.400	0.050	0.170	0.350	0.380

† *level reached for the first time during the analysis (see Fig. 3b)*
* *RL1; first cracking*

examined during the analyses process, as it has already been mentioned and detailed commented (see Figs. 11-13).

Information in terms of the corresponding top drift (%h_{str}) at the first time that each deformation level of the masonry infill is reached for each case of all the examined structural systems are given in Table 2.

6. Conclusions

The capacity of the 6-story frame structures with the first floor morphology irregularities of Case B (tall first story) and Case C (slab-column connections without beams) is decreased in comparison to the capacity of the frame with all the floor levels having equal interstory heights (Case A: regular frame). Further, the form of an open first floor structure (pilotis type) led to high values of interstory drifts at the first story. The first floor morphology of Case B exhibits the highest demands for interstory drift at the base floor story of all the examined cases.

The results indicate that when displacement criteria (ATC40) are adopted for the seismic assessment of the structures, the 6-story frame with the irregularity of tall first story (Case B) exhibits the most critical performance in comparison to the other two structural systems (Case A and Case C).

However, when rotational criteria (EC-part3) are used to evaluate the global performances, the 6-story frame structure of Case C (slab-column connections without beams) exhibits the most critical response in comparison to the other structural systems (Cases A and B). It is noted that in cases that the 6-story RC frame has been studied without infills the limit state of Significant Damage (SD) and consequently the state of Near Collapse (NC) have not been reached in any of the three structural systems (Case A, Case B and Case C).

Considering all the examined structures with infills in all story levels except for the first story level (pilotis or open ground story) the yielded results were quite different; the Case B structure reached only the SD performance level whereas the Case C structure attained the NC performance level.

Considering the load patterns adopted in the pushover analysis from the observed results it can be deduced that the use of uniform load pattern yielded more conservative results for infilled or pilotis frame structures compared to the corresponding results yielded from the use of triangular load pattern.

At local level the irregularities of Case B (tall first story) and Case C (slab-column connections without beams) caused changes to the failure mode of the structure leading to soft-story mechanisms at the first story. Thus, increase of the flexural requirements of the beams and the columns at the lower floor levels is observed due to the considered irregularities in comparison to the corresponding requirements that are developed in the case of the 6-story bare frame structure of Case A (regular first floor morphology). All the columns of the Case A structure remained in the elastic range.

The requirements for inelastic response of the beams and the columns are further increased when infills are taken into account in comparison to the corresponding demands in the case of bare frame 6-story structure. These flexural demands are concentrated at the floor levels where strength and stiffness degradation of the infills have been occurred.

The most critical type of first morphology for the seismic performance of the columns at the first story level has been proved to be the one of Case C since in this case demands for ductility of the columns are greater than the corresponding ones of Case B and Case A. Furthermore, in the case of the open ground story effect the demands of the columns at the first floor of Case C frame became even higher.

Although the development of soft-story mechanism can be considered frequent for pilotis type frames the results of this study indicated that this mechanism can also be occurred even in fully infilled frame structures (regular distribution of infills). Further, the results of this study indicated that the limit states provided by ATC-40 at the local level of the RC members (beam and columns) are more conservative compared with the limit states of EC8-part3. Finally, it is worth to be noted that when the performance of the structures is studied at an early state of the response where infills have not been collapsed yet the demands for inelastic deformations of the members are negligible due to the favor presence of infills.

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