INFLUENCE OF INFILL PANELS ON THE SEISMIC BEHAVIOUR OF A R/C FRAME DESIGNED ACCORDING TO MODERN BUILDINGS CODES

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ABSTRACT

It has been broadly shown that presence of infill panels as closing elements of R/C frame buildings has a significant influence on global structural behaviour. Nevertheless, infill elements are not usually considered in the modelling process during the design phase.
The present work investigates the effect of infill masonry walls on the dynamic characteristics of a R/C MRF building, designed according to a modern seismic building code, and on its seismic performance at different levels of seismic intensity.
An analytical investigation is carried out through eigenvalue analysis on both bare and infilled structure, in order to calibrate the elastic properties of the concrete and infills according to in situ tests; nonlinear static analyses are also performed to characterize the inelastic behaviour.
The infill system considerably affects the behaviour of the examined structure, in agreement with earlier studies related to very simple and usually ”unrealistic” structures. This result becomes more reliable due to the consistency between the results of the eigenvalue analysis and the experimental dynamic data.

Keywords: Reinforced Concrete, infills, seismic load design

1. INTRODUCTION

The strength and the energy dissipation capacity of such structural systems can drastically change due to the insertion of masonry elements within the RC frame. Nevertheless, infill elements are not usually considered in the modelling process during the design phase, rather focusing the attention on the flexure- controlled behaviour of RC members.
In this first Section, a review of main numerical studies from literature is presented first.
Then the results of modal analyses are presented, focusing the attention on the differences between the bare model and the infilled one. Moreover, the calibration process of the infilled model on the in situ dynamic identification results is shortly showed.
Then nonlinear static analyses are performed, both in the case of the bare structure and in the case of the infilled one, thus showing the influence of infill elements on seismic capacity at Near Collapse Limit State.

2. PREVIOUS NUMERICAL STUDIES ON SEISMIC BEHAVIOUR OF CONTEMPORARY SEISMIC DESIGNED RC BUILDINGS WITH INFILLS

Observation of damage to buildings after severe earthquakes (e.g., Kocaeli 1999, L’Aquila 2009) has clearly shown the significant influence of infill walls on the seismic behaviour of existing RC buildings. The interaction between infill panels and RC structural elements under seismic action develops at global level, leading to an increase in lateral stiffness and base shear capacity, but also at local level, potentially leading to brittle failure mechanisms in surrounding elements such as columns or beam-column joints. Moreover, infills irregularly placed throughout the structure, in plan and/or in elevation, can lead to a detrimental localization of inelastic displacement, resulting in less ductile and potentially catastrophic collapse mechanisms. However, such influence can be not negligible at all also for new buildings, designed for seismic loads according to Capacity Design principles, as recognized by modern seismic code prescriptions (e.g., EC8-part I (CEN, 2004a) – Sections 4.3.6 and 5.9). In the following, a brief review of main literature studies on the influence of infills on the seismic behaviour of contemporary seismic load designed RC structures is reported.

Fardis and Panagiotakos (1997b) presented a comprehensive study based on nonlinear dynamic analyses carried out on different structural systems, including RC structures designed for seismic loads according to Eurocode 8, complying with the weak beam/strong column principle, with different number of storeys (4, 8 or 12) and infill configuration (Bare, Fully infilled or Pilotis). Infill influence was deemed beneficial on seismic response of these frames, except for very brittle or irregularly distributed infills. However, the detrimental effect of an irregular infill distribution was more pronounced at ground motion intensities much higher than that of the design motion. In medium-high rise reinforced concrete frame buildings, the presence of infills and even irregularities in their arrangement in elevation had a very small effect on the global and local seismic response, also due to the low shear strength of the infills in comparison to the total strength and base shear of the building.

Kappos (1998) carried out nonlinear dynamic analyses on a case study ten-storey infilled RC frame designed according to Eurocode 8, in intermediate ductility class and for a PGA equal to 0.25g, considering different infill configurations (Bare, Fully infilled and Pilotis) and different strength values for infill. Under the same seismic action, the best performance (in terms of lowest interstorey drift demand) was shown by the Fully infilled frame, and the worst behaviour was shown by the Pilotis frame.

Negro and Colombo (1997) carried out a numerical-experimental comparison with the results of the pseudo-dynamic tests on a full-scale four-storey RC building designed according to Eurocodes 2 and 8, in ductility class High – complying the weak beam/strong column principle – for a PGA equal to 0.3g, without infills, with uniformly distributed infills and with infills in all storey but the first, respectively (Negro and Verzeletti, 1996). Authors highlighted the detrimental effect of localization of ductility demand due to an irregular infill distribution, as expected. However, authors pointed out that a detrimental effect on seismic behaviour may be expected also from a uniform infill distribution since storey-level sidesway mechanisms take place after the failure of the panels at that storey, thus leading to a similar effect of localization of ductility demand. This effect may or may not be counterbalanced by the beneficial effect due to the increase in stiffness, strength and energy dissipation capacity provided by infills.

In Dolšek and Fajfar (2001) nonlinear dynamic analyses were carried out on numerical models representing two different structures, including a “contemporary” structure designed for a base shear coefficient equal to 0.15 and complying with Capacity Design principles (e.g., weak beam/strong column condition), representing the test structure reported in (Negro and Verzeletti, 1996). The structure was uniformly infilled, considering two different cases for infill mechanical characteristics. Results of nonlinear dynamic analyses highlighted that uniformly distributed infills led to a beneficial reduction in displacement demand up to a certain intensity of ground motion, but if this threshold (that increased with the infill strength) was exceeded a concentration of displacement demand was observed at the bottom of the structure when weak infills were considered.

Another study assessing the seismic performance of RC frames with infills was proposed by Dymiotis et al. (2001). The considered frame was designed according to Eurocode 8 (CEN, 1995), for the intermediate ductility class and a design PGA equal to 0.25g. Uniformly infilled, Pilotis and Bare configurations were considered. Authors observed that the Pilotis frame was more vulnerable than the Uniformly infilled frame, at both Serviceability (S-) and Ultimate (U-) Limit State (LS). Compared with the Bare frame, the Uniformly infilled frame was less vulnerable at SLS but more vulnerable at ULS. The Pilotis frame resulted as the most vulnerable system at both SLS and ULS.

3. CASE STUDY BUILDING

The examined building is part of a more complex residential construction, made up of eight reinforced concrete structures, divided by structural joints, sited in a high seismic zone in Southern Italy. This building, still under construction, was designed according to a seismic code in force in Italy before the 2009 L'Aquila earthquake. The structure has a rectangular shaped plan and it has two underground storeys and seven storeys above the ground level with average 3.15m interstorey height and a plan area equal to about $390m^2$.

The structure is a framed system in the longitudinal direction (X) and as a dual system in the transversal direction (Y), due to the presence of two RC walls.

Three infill elements typologies are used in the structure: concrete blocks walls in the underground storeys, the hollow brick infill in the higher levels and the light hollow brick panels for internal partitions. The general layout of infills is not regular, neither in elevation (see Figure 2) nor in plan.

Columns have the same dimensions at all levels ($70x70cm^2$ and $40x80cm^2$) and geometrical longitudinal reinforcement ratios is about 1.2% for all of them. Moreover, internal beams are flat in the seven storeys above the ground level and the longitudinal reinforcement ratio is in the range of 1.4% - 2.4%. External beams have base of 40cm and height of 70cm or 60cm $(\rho \cong 1.0\%)$.

The geometrical properties (width, thickness and height) of the masonry infill panels are directly observed during a structural survey. In order to estimate the mechanical properties of such infill walls, some considerations are made taking into account experimental data available in the current literature, actual standards for construction and dynamic identification results, described in the following.

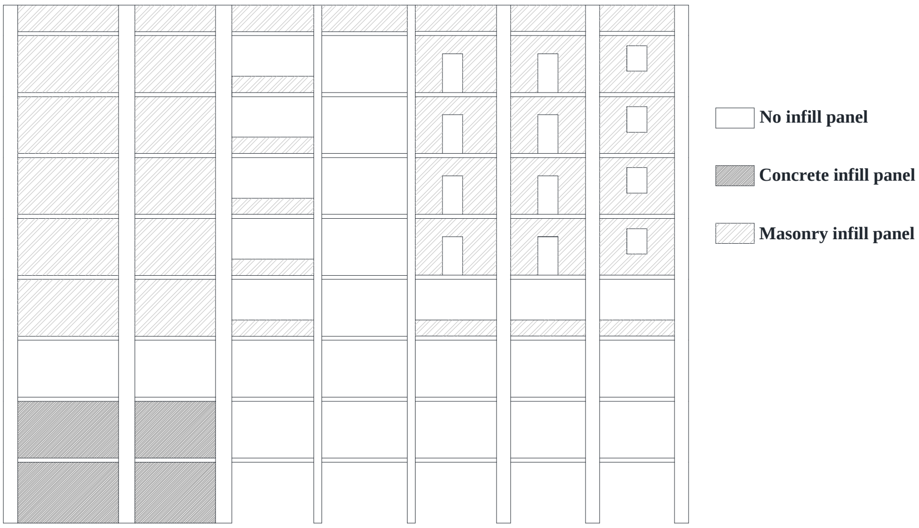

Figure 1. Infills distribution in an external X frame

4. LINEAR ANALYSES

A preliminary experimental dynamic identification test is executed in order to obtain reliable elastic analytical model of the investigated structure, as much as possible representative of the effective condition of the building and of its response under seismic actions. This test allows the individuation of the dynamic properties of the building in operational conditions and consequently the definition of a numerical model validated through the execution of updating procedures (Rainieri, 2009).

The investigated structure has been instrumented at different levels with a couple of force balance accelerometers in the two main directions of the building in two opposite corners. These accelerometers are selected for monitoring operational conditions due to their higher sensitivity. The placement of sensors on the building has been selected in order to get both the translational and torsional modes of the structure.

The identification of the modal parameters of the structure was performed according to different OMA (operational modal analysis) techniques, giving the results in terms of natural frequencies, damping ratios (Table 1) and modal shape at two monitored levels (Fig. 3).

Table 1. Experimental frequencies of the first three natural modes

Mode	Type	Natural frequency	Damping ratio
[-]	[-]	[Hz]	[%]
I	Translational (short side)	2.64	0.5
II	Translational (long side) + Torsional	4.01 (3.92 - 4.40)	1.6
III	Torsional	4.3 (4.2 - 4.4)	1.7

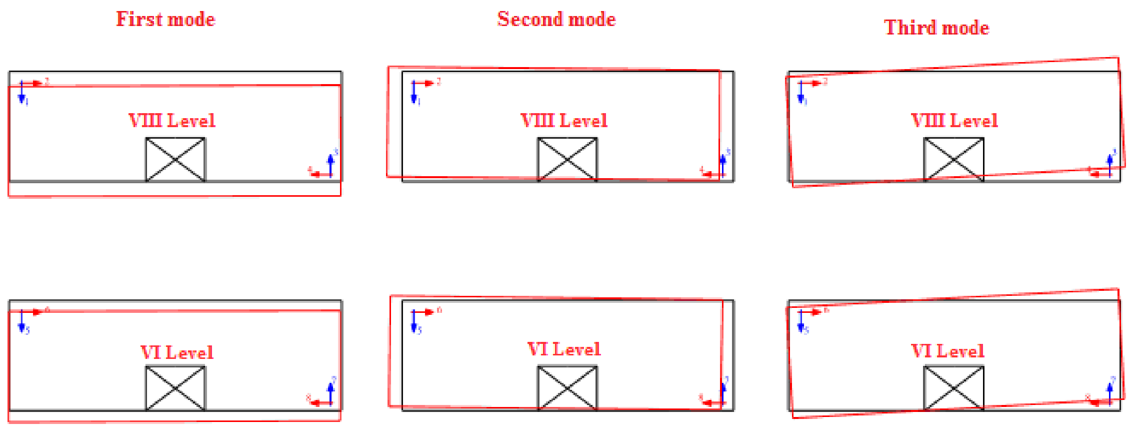

Figure 2. Experimental modal shapes of the first three natural modes

4.2 Linear model

Two different numerical finite element elastic models are developed in order to analytically reproduce the dynamic response of the case study, individuated through the execution of an in situ identification.
A first preliminary model neglects infills, introducing only their mass; a second model is characterized by equivalent diagonal struts considering the influence of their openings. The two models considers the design value of the concrete elastic modulus equal to $32308MPa$ (corresponding to the C28/35 class) and the shear modulus of infills equal to $1500MPa$, according to code suggestions.
The resistant structure (beam, columns and structural concrete walls) is modelled with beam elastic elements. With regards to the second linear model, the infill panels are modelled as an equivalent diagonal strut; according to the model proposed by Panagiotakos and Fardis (1996), the elastic lateral stiffness of the panel is evaluated as $\frac{G_w \cdot A_w}{h_w}$ (Fardis, 1997). Moreover, the influence of openings is taken into account through a coefficient (Eqn. 2.1) linearly dependent on the opening ratio ($A_{openings} / A_{panel}$), based on the experimental results reported in Kakaletsis and Karayannis (2009):

$$\lambda_{openings} = \max\left(0;1-1.8 \cdot \frac{A_{openings}}{A_{panel}}\right) \tag{2.1}$$

Hence, stiffness of the panels are multiplied by $\lambda_{openings}$. Comparing the modal results of the two described models, it is found that infills as well as the presence of openings cannot be neglected. The first bare model results are very far from the vibration periods of the experimental tests, while taking into account the presence of the masonry infills and the actual distribution of the openings the periods becomes more similar to the dynamic identification values (Table 2). In Table 2 the two models results are presented in terms of modal shape types too.

Table 2. Experimental frequencies of the first three natural modes

Mode [-]	I model - No infills Type [-]	Analytical frequency [Hz]	Frequency scatter [%]	III model - Infills w openings Type [-]	Analytical frequency [Hz]	Frequency scatter [%]
I	Translational (long side)	1.38	-47.8%	Translational (short side)	2.47	-6.4%
II	Torsional	1.72	-57.2%	Translational (long side)	3.23	-19.3%
III	Translational (short side)	1.78	-58.7%	Torsional	3.63	-15.7%

Chosen the linear model, the mechanical characteristic of concrete and masonry, is investigated through an optimization process in order to closely match the experimental dynamic properties. The selected updating parameters are the elastic modulus of concrete and the shear modulus of masonry. By adopting a fine increment for the values of modules of masonry and concrete and considering all possible combinations, a total number of 451 modal analyses are performed. The linear models (Fig. 4) have been automatically generated and analysed starting from a basic model by mean of the “PBEE toolbox” software (Dolšek, 2010), combining MATLAB® with OpenSees (McKenna et al., 2004).

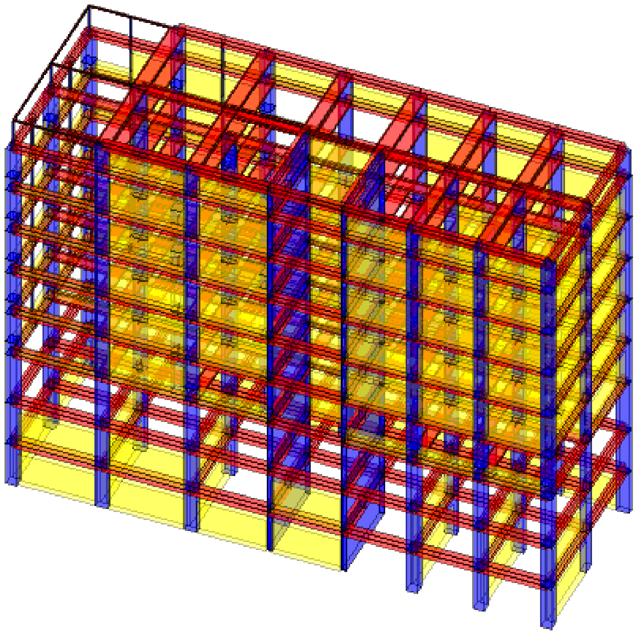

Figure 3. 3D infilled linear model of the investigated structure

Widely used objective functions are defined in terms of scatter between analytical and numerical values of natural frequencies (Eqn. 2.2.), or taking into account also mode shape correlation (Rainieri et al., 2012):

$$J_f = \frac{100}{N_m} \cdot \sum_{i=1}^{N_m} \left| \frac{f_i^e - f_i^a}{f_i^e} \right| \tag{2.2}$$

The results is given in the Fig. 5, in which the objective function is plotted versus the investigated mechanical properties,. The minimization process gives for the concrete a value of $43500MPa$ and for the masonry a value of $2000MPa$. The masonry shear modulus is similar to the mechanical characteristics suggested by the code and the concrete elastic modulus is in line with the modulus evaluated from the median compressive strength of in situ tests.

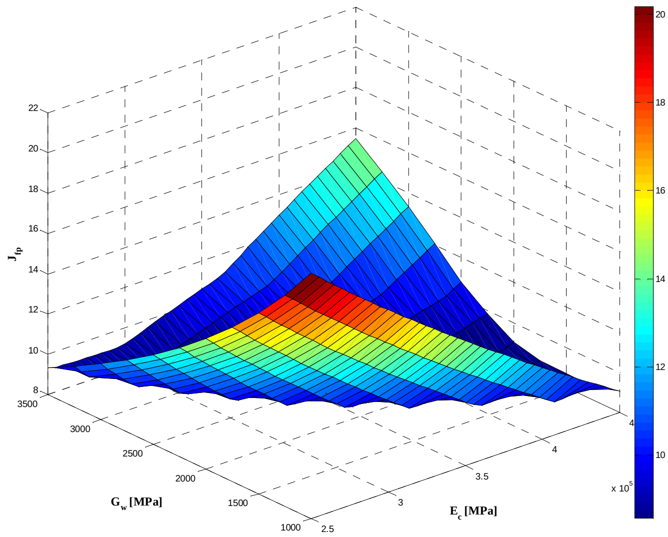

Figure 4. Objective function versus Gw and Ec

The results of the modal analysis with these found values are showed in Table 3 in terms of frequencies scatter and modal shape agreement.
In the present case of study, the ambient dynamic identification has allowed the investigation of the infill influence as well as the optimal values of the material properties.

Table 3. Participating masses and frequencies scatter of the first tree modes

Mode [-]	**Participating mass** **Ux** [%]	**Uy** [%]	**Rz** [%]	**Type** [-]	**Analytical frequency** [Hz]	**Frequency scatter** [%]
I	0.16	94.2	4.3	Translational (short side)	2.86	8.0
II	93.8	1.6	3.5	Translational (long side)	3.74	6.7
III	2.4	52.0	35.0	Torsional	4.19	0.23

5. NONLINEAR MODEL

Pushover analyses are performed on the described structure, both with and without infills. A lumped plasticity model is used in the OpenSees-based analysis platform “PBEE toolbox” (Dolšek, 2010), combining MATLAB® with OpenSees (McKenna et al., 2004), to simulate the inelastic behaviour of columns and beams. The constitutive law of the moment-rotation end springs has three characteristic branches: a linear elastic first branch up to the cracking point, a linear branch from cracking to yielding and a perfectly plastic post-yielding branch up to the ultimate point. Section moment and curvature at cracking and yielding are calculated by means of a fiber section analysis, for an axial load value corresponding to gravity loads. Chord rotation at yielding ad ultimate are calculate according to the formulas proposed in (Fardis, 2007 – Eqs. 2.20a and 3.27a).
Infill panels are modelled by means of equivalent struts. The adopted model for the envelope curve of the force-displacement relationship is the model proposed by Panagiotakos and Fardis (1996). The

monotonic envelope of the lateral force-displacement curve is given by four branches: the first branch corresponds to the linear elastic behaviour up to the first cracking, and the stiffness is given by

$$K_{el} = \frac{G_w \cdot A_w}{h_w} \tag{2.5}$$

where: A_w is the cross-sectional area of the infill panel, G_w is the elastic shear modulus of the infill material and h_w is the clear height of the infill panel. The shear cracking strength is given by $F_{cr} = \tau_{cr} \cdot A_w$, where τ_{cr} is the shear cracking stress.

The second branch follows the first cracking, up to the point of maximum strength. The maximum strength is given by $F_{\max} = 1.3 \cdot F_{cr}$ and the corresponding displacement is evaluated assuming that the secant stiffness up to this point is given by Mainstone's formula (1971), that is, assuming an equivalent strut width given by:

$$b_w = 0.0175 \cdot (\lambda_h \cdot h_w)^{-0.4} \cdot d_w, \tag{2.6}$$

where d_w is the clear diagonal length of the infill panel and coefficient λ_h is given by:

$$\lambda_h = \sqrt[4]{\frac{E_w \cdot t_w \cdot \sin 2\vartheta}{4 \cdot E_c \cdot I_p \cdot h_w}} \tag{2.7}$$

The third branch is the post-capping degrading branch, up to the residual strength. Its stiffness depends on the elastic stiffness through the parameter α: $K_{soft} = -\alpha \cdot K_{el}$ the value of this parameter has to be arbitrarily assumed. However, the authors give some indication [1971]: the range of values for a should be between 0.005 and 0.1, although a value of 0.1 is unrealistically high (very brittle infill), while a value of 0.01 may be more realistic yet still conservative (well-constructed infill); the fourth branch is the horizontal branch corresponding to the residual strength. This strength is given by $F_{res} = \beta \cdot F_{\max}$ with β is between 0.05 and 0.1. Hence, the ratio between post-capping degrading stiffness and elastic stiffness (parameter α) is assumed equal to 0.01, whereas the ratio between residual strength and maximum strength (parameter β) is assumed equal to 0.01.

Nonlinear Static Push-Over (SPO) analyses are performed on the studied building both in X and Y direction, according to the N2 method (Fajfar, 2000). A lateral load pattern proportional to the displacement shape of the first mode is used. Lateral response is evaluated in terms of base shear-top displacement relationship.

Seismic capacity at Near Collapse (NC) Limit State is evaluated, corresponding to the attainment of the chord rotation at ultimate in the first RC member.

When infill failure leads to a significant strength degradation of the lateral response, a multi-linearization of the pushover curve is carried out by applying the equal energy rule respectively between the initial point and the maximum resistance point and between the maximum resistance point and the point corresponding to the first RC element conventional collapse, as shown in the following. When the lateral response is not characterized by a strength degradation an elasto-plastic bi-linearization is carried out by applying the equal energy rule between the initial point and the maximum resistance point.

Then, the IN2 curves (Dolšek and Fajfar, 2004b) for the equivalent SDOF systems are obtained by assuming as Intensity Measure both the elastic spectral acceleration at the period of the equivalent SDOF system ($S_{ae}(T_{eff})$) and the Peak Ground Acceleration (PGA). IN2 curves in terms of $S_{ae}(T_{eff})$ are evaluated based on the R-μ-T relationships given in (Dolšek and Fajfar, 2004a) or in (Vidic et al., 1994) for degrading or non-degrading response, respectively. Then, the corresponding IN2 curves in terms of PGA are evaluated, too. To this end, the elastic demand spectra adopted in Italian code (DM 14/1/2008) – provided in (INGV-DPC S1, 2007) – for the site of interest are used. Soil type C and 1st topographic category are assumed. Hence, $S_{ae}(T_{eff})$ and PGA capacities at NC Limit State are

evaluated, defined as the $S_{ae}(T_{eff})$ and the PGA of the elastic demand spectrum under which the displacement demand is equal to the displacement capacity at NC Limit State, both with and without considering infill elements and both in X and Y direction.
Results are illustrated in the following. Values of seismic capacity at NC in terms of $S_{ae}(T_{eff})$ and PGA are compared with the design values $S_{ae}(T_1)$ and PGA, both considering or not the infill presence.

5.1. Analysis of results

Pushover and IN2 curves for bare and infilled models in X direction are presented below (Fig. 5).

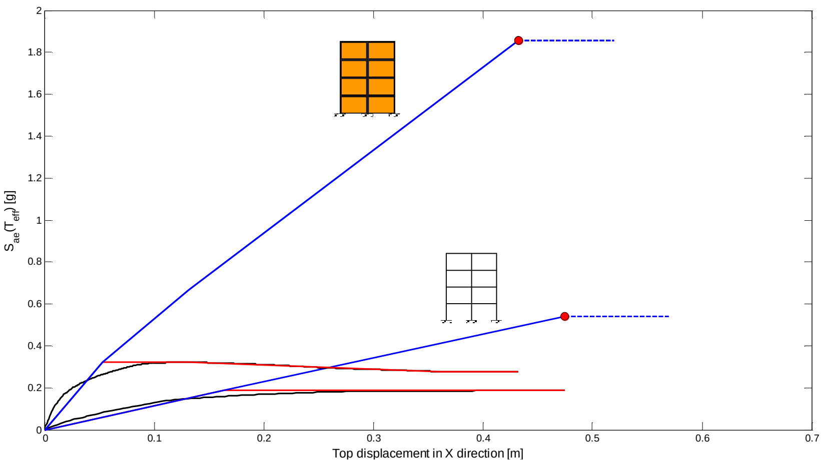

Figure 5. Pushover (black), linearized backbone (red) and IN2 (blue) curves for bare and infilled models in X direction (NC limit state reported as red circle)

A global collapse mechanism is observed in the bare structure, whereas in the infilled structure the collapse mechanism involves all of the storeys except the two storeys below the ground level, where concrete infill panels are present. However, displacement capacity at NC Limit State is not significantly affected by infill presence. On the other hand, presence of infills leads to a significant increase in global stiffness and strength (that is, decrease in effective period T_{eff} and increase in inelastic acceleration capacity $C_{s,max}$, see Table 4), thus leading to a high increase in $S_{ae}(T_{eff})$ at NC Limit State.

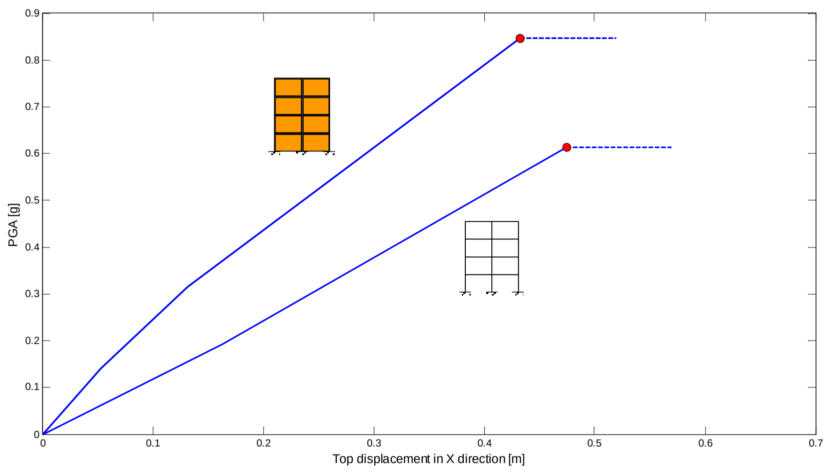

Figure 6. IN2 (blue) curves for bare and infilled models in X direction (NC limit state reported as red circle)

Fig. 6 reports IN2 curves for bare and infilled models in X direction in terms of PGA. Seismic capacity at NC Limit State is still higher for the infilled model. Nevertheless, the ratio between PGA capacities of the infilled and bare models is lower than the corresponding ratio in terms of PGA, due to the difference in T_{eff}.

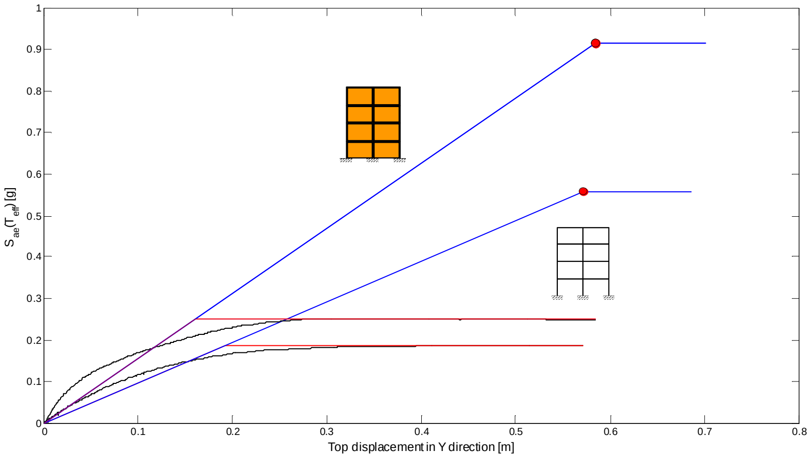

Figure 7. Pushover (black), linearized backbone (red) and IN2 (blue) curves for bare and infilled models in Y direction (NC limit state reported as red circle)

In Y direction, similarly to X direction, presence of infills does not significantly affect the displacement capacity at NC Limit State. Moreover, a significantly lower percentage of infill walls is present in this direction, thus leading to a lower difference between the bare and the infilled models, both in terms of global stiffness and strength (see Table 4).

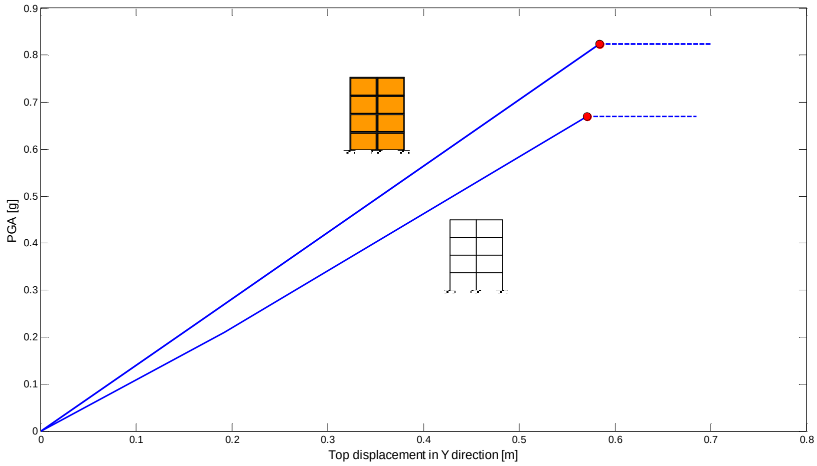

Figure 8. IN2 (blue) curves for bare and infilled models in X direction (NC limit state reported as red circle)

In terms of PGA, the difference in T_{eff} leads, again, to a lower difference between seismic capacity at NC Limit State for the bare and the infilled models, compared with the capacities expressed in terms of $S_{ae}(T_{eff})$.

Table 4. Main parameters of pushover analyses

		T_{eff} [sec]	$C_{s,max}$ [g]	$C_{s,min}$ [g]	S_{dy} [m]	$\mu_{collapse}$ [-]	$S_{ae}(T_{eff})_{collapse}$ [g]	$PGA_{collapse}$ [g]
Bare	X	1.88	0.187	/	0.164	2.9	0.542	0.614
	Y	2.03	0.187	/	0.192	2.98	0.559	0.670
Infilled	X	0.818	0.321	0.278	0.0533	8.12	1.86	0.847
	Y	1.6	0.252	/	0.161	3.64	0.916	0.824

6. CONCLUSIONS

The influence of infills on the seismic capacity of a real RC building, designed according to a contemporary seismic code, was investigated. Data provided by an in-situ dynamic identification allowed to calibrate a linear structural model, including mechanical characteristics of concrete and infill materials. Based on these results, seismic capacity at Near Collapse Limit State was assessed by means of non linear static pushover analyses. The beneficial influence of infills, in terms of stiffness and strength increase, was highlighted; moreover, no significant detrimental decrease of displacement capacity was observed, since collapse mechanisms involving almost all of the storeys were anyhow observed, in both models and both directions. However, it is observed that a flexure-controlled behaviour of RC members was assumed, and no possible shear failure mechanism due to local interaction between infill panels and RC members was taken into account. Nevertheless, it is likely to assume that such mechanisms may be avoided, or at least significantly limited, adopting the seismic details prescribed by modern seismic codes according to Capacity Design principles, such as transverse reinforcement (i) in beam-column joints and (ii) at the ends of the columns (with proper spacing). Although these prescriptions are aimed (i) at avoiding joint failure due to flexural forces from beams and columns and (ii) at providing higher ductility in critical (plastic hinge) region, respectively, they can also avoid brittle failure mechanisms due to forces from local interaction with infill panels.

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