Influence of Infill Presence and Design Typology on Seismic Performance of RC Buildings: Sensitivity Analysis

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SUMMARY:
A growing attention has been addressed to the influence of infills on the seismic behavior of Reinforced Concrete (RC) buildings, also supported by the observation of damage to infilled RC buildings after severe earthquakes (e.g. L'Aquila 2009, Lorca 2011). In this paper, a numerical investigation on the influence of infills on the seismic behavior of four different case study buildings is carried out: four- and eight- storey buildings, designed for seismic loads according to the current Italian technical code or for gravity loads only according to an obsolete technical code, are considered. Seismic capacity at two Limit States (Damage Limitation and Near Collapse) is assessed through static push-over analyses, within the N2 spectral assessment framework. Different infill configurations are considered (Bare, Uniformly Infilled, Pilotis), and a sensitivity analysis is carried out, thus evaluating the influence of main material and capacity parameters on seismic response, depending on the number of storeys and the design typology.

Keywords: Reinforced Concrete, infills, design typology, Limit State, sensitivity analysis

1. INTRODUCTION

During last decades, post-earthquake damages (e.g., Kocaeli 1999, L'Aquila 2009), numerical and experimental studies showed that a growing attention must be addressed to the influence of infills on the seismic behaviour of Reinforced Concrete (RC) buildings. Currently, infills are generally considered in RC buildings as partition elements without any structural function, so neglecting their important influence on the increase in lateral stiffness and base shear, on the reduction in period of vibration, on possible brittle failure mechanisms in joints and columns due to local interaction between panels and the adjacent structural elements, on the building collapse mechanism.
In this paper, a numerical investigation on the influence of infills on the seismic behaviour of four case study buildings is carried out: four- and eight- storey buildings, designed for seismic loads according to the current Italian technical code or for gravity loads only according to an obsolete technical code, are considered. Seismic capacity at two different Limit States (Damage Limitation (DL) and Near Collapse (NC)) is assessed by means of Static Push-Over (SPO) analyses, within the N2 spectral assessment framework. Different infill configurations are considered (Bare, Uniformly infilled and Soft-storey infilled), and a sensitivity analysis is carried out, thus evaluating the influence of main material and capacity parameters on seismic response at different Limit States, depending on the number of storeys and the design typology.

2. CASE STUDY STRUCTURES

The case study structures are symmetric in plan, both in longitudinal (X) and in transverse (Y) direction, with five bays in longitudinal direction and three bays in transverse direction. Interstorey height is equal to 3.0 m, bay length is equal to 4.5 m. Slab way is always parallel to the transverse direction. So the four case study buildings are:
- two gravity load designed "GLD" buildings, a four-storey and an eight-storey building, defined by means of a simulated design procedure according to code prescriptions and design practices in

force in Italy between 1950s and 1970s (Regio Decreto Legge n. 2229, 16/11/1939; Verderame et al., 2010 a). The structural configuration follows the parallel plane frames system: gravity loads from slabs are carried only by frames in longitudinal direction. Beams in transverse direction are present only in the external frames. Element dimensions are calculated according to the allowable stresses method; the design value for maximum concrete compressive stress is assumed equal to 5.0 and 7.5 MPa for axial load and axial load combined with bending, respectively. Columns dimensions are calculated according only to the axial load, beam dimensions and reinforcement are determined from bending due to loads from slabs. Reinforcement in columns corresponds to the minimum amount of 0.8% of the section area, as prescribed by code (Regio Decreto Legge n. 2229, 16/11/1939). Reinforcing bars are smooth and their allowable design stress is equal to 160 MPa;

- two seismic load designed (SLD) buildings, a four-storey and an eight-storey building, designed for seismic loads according to the current Italian code (Decreto Ministeriale del 14/1/2008) in Ductility Class High. Beams in transverse direction now are present also in the internal frames. The principles of the Capacity Design are applied. C25/30 concrete (fcd = 14.17 MPa) and B450C steel (fyd = 391.3 MPa) are used. Mean values for materials strength are assumed equal to 36 MPa and 550 MPa for concrete and steel respectively (Cosenza et al., 2009a,b). They are located in a high seismic city in Southern Italy (Avellino, Lon: 14.793 Lat.: 40.915); soil type A (stiff soil) and 1st topographic category are assumed; the PGA used for the design at Significant Damage Limit State (SLV) – corresponding to an exceedance probability of 10% in 50 years and a return period of 475 years – is equal to 0.19g.

For each case study, three hypotheses are made about the infill configuration:

- Case 1: infill panels are uniformly distributed along the height (Uniformly Infilled frame).
- Case 2: first storey is bare and upper storeys are infilled (Pilotis frame).
- Case 3: no infill panel is present (Bare frame).

Infills panels, if present, are uniformly distributed in all the external frames of the building. Panel thickness is equal to 20 cm. Presence of openings is not taken into account.

Nonlinear response of RC elements is modelled by means of a lumped plasticity approach. A three-linear envelope is used for beams and columns, where characteristic points are cracking, yielding and ultimate. The behaviour is assumed linear elastic up to cracking and perfectly-plastic after yielding. Rotations at yielding and ultimate are evaluated through the formulations given in (Fardis, 2007). No reduction of ultimate rotation for the lack of seismic detailing is applied, due to the presence of smooth reinforcement (Verderame et al., 2010b). 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 (Panagiotakos and Fardis, 1996; Fardis, 1997). The ratio between post-capping degrading stiffness and elastic stiffness (parameter α) is assumed equal to 0.01. The ratio between residual strength and maximum strength (parameter β) is assumed equal to 0.01. First mode nonlinear SPO analyses are performed on the case study buildings both in X and Y direction. Structural modelling, numerical analyses and post-processing of damage data, including the 3D graphic visualization of the deformed shape, are performed through the "PBEE toolbox" software (Dolšek, 2010), combining MATLAB® with OpenSees (McKenna et al., 2004), modified in order to include also infill elements (Ricci, 2010; Celarec et al., 2011). A multi- or bi-linearization of the pushover curve is carried out depending on the degrading (due to infill failure) or not degrading base shear-top displacement relationship, respectively. Moreover, the procedure proposed in (Dolšek and Fajfar, 2005) to improve the accuracy of the displacement demand assessment in the case of low seismic demand is applied, by applying a specific R-□-T relationships for ductility lower than 1, as proposed by the authors.

Two Limit States (LSs) are defined: Damage Limitation (DL), corresponding to the displacement when the last infill in a storey reaches its maximum resistance thus starting to degrade (Dolšek and Fajfar, 2008a) or when the first yielding in RC members occurs, and Near Collapse (NC), corresponding to the first conventional collapse in RC members (i.e., the first RC member reaches its ultimate rotation). Potential brittle failure mechanisms are not taken into account in this paper.

IN2 curves (Dolšek and Fajfar, 2008b) 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). Due to the fact that the ratio between $S_{ae}(T_{eff})$ and PGA is not constant, the IN2 curves in terms of $S_{ae}(T_{eff})$ or in terms of PGA may have different shapes. Values of these seismic intensity parameters corresponding to characteristic values of displacement (ductility) demand (including the considered LSs) are calculated, 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. Elastic spectra are the Uniform Hazard Newmark-Hall demand spectra adopted in Italian code (Decreto Ministeriale del 14/1/2008) – provided by (INGV-DPC S1, 2007) – for a high seismic city in Southern Italy (Avellino, Lon.: 14.793 Lat.: 40.915). Soil type A (stiff soil) and 1st topographic category are assumed (no amplification for stratigraphic or topographic effects). Demand spectra are provided by (INGV-DPC S1, 2007) for a range of return periods from 30 to 2475 years. For intermediate values of seismic intensity, an interpolation procedure is proposed (Decreto Ministeriale del 14/1/2008). Nevertheless, in this study there is the need to extend elastic demand spectra above and below the extreme values, as in (Crowley et al., 2009). To this aim, the formulations proposed for the interpolation procedure are also used to extrapolate the above mentioned parameters out of the given range of values.

3. SENSITIVITY ANALYSIS

In order to evaluate the influence of material characteristics and element capacity on the seismic response of the case study structure, a sensitivity analysis is carried out (Celarec et al., 2011; Celarec et al., 2010). To this end, the following parameters are selected as Random Variables (RVs):

- Concrete compressive strength, f_c;
- Steel yield strength, f_y;
- Chord rotation at yielding in RC members, θ_y;
- Chord rotation at ultimate in RC members, θ_u;
- "Loads" of load-displacement relationship of the infill trusses, F_{infill};
- "Displacements" of load-displacement relationship of the infill trusses, D_{infill}.

The variable F_{infill} is a vector whose components are $[F_{cr};F_{max}]$, where F_{cr} and F_{max} are cracking and maximum strength of infills, respectively; similarly, the variable D_{infill} is the vector $[D_{cr};D_{max}]$, where D_{cr} and D_{max} are cracking and maximum displacement of infills, respectively. Ultimate strength and displacement of infills are obtained from F_{infill} and D_{infill} according to the adopted model (Fardis, 1997). Loads and displacements of the load-displacement relationship of the infill trusses can thereby vary independently of each other. The variability of F_{infill} and D_{infill} includes both mechanical and modeling variability, as explain below. A lognormal distribution is assumed for all of the RVs. Each distribution is defined through the central (median) value and the Coefficient of Variation (CoV) - see Table 1. For the concrete compressive strength, reference values come from a statistical analysis on the mechanical properties of concrete employed in Italy (Verderame et al., 2001; Cosenza et al., 2009a). For the steel yield strength, values are referred to Aq50 steel typology (the most widely spread in Italy during 1960s) for GLD structures, and to B450C for SLD structures (Verderame et al., 2011b; Cosenza et al., 2009b). The determination of infill material characteristics is affected by high uncertainties, and literature does not offer an enough large amount of experimental data.

Table 1. Summary of median and CoV values for the selected RVs

		Seismic Load Design			**Gravity Load Design**		
RV	*Distribution*	*Median Value*	*CoV*	*Reference*	*Median Value*	*CoV*	*Reference*
f_c	Lognormal	36.0 MPa	0.20	(Cosenza et al, 2009a)	25.0 MPa	0.31	(Verderame et al., 2001)
f_y	Lognormal	550.0 MPa	0.06	(Cosenza et al, 2009b)	369.7 MPa	0.08	(Verderame et al., 2011b)
θ_y	Lognormal	$1.015*\theta_{y,calculated}$	0.331	(Fardis, 2007)	$1.015*\theta_{y,calculated}$	0.331	(Fardis, 2007)
θ_u	Lognormal	$0.995*\theta_{u,calculated}$	0.409	(Fardis, 2007)	$0.995*\theta_{u,calculated}$	0.409	(Fardis, 2007)
F_{infill}	Lognormal	$[F_{cr};F_{max}]$	[0.30; 0.30]	(Fardis, 1997; Rossetto and Elnashi, 2005; Calvi et al., 2004)	$[F_{cr};F_{max}]$	[0.30; 0.30]	(Fardis, 1997; Rossetto and Elnashi, 2005; Calvi et al., 2004)
D_{infill}	Lognormal	$[D_{cr};D_{max}]$	[0.30; 0.70]	(Fardis, 1997; Rossetto and Elnashi, 2005; Calvi et al., 2004)	$[D_{cr};D_{max}]$	[0.30; 0.70]	(Fardis, 1997; Rossetto and Elnashi, 2005; Calvi et al., 2004)

In this study, a median value of 1240 MPa for the shear elastic modulus G_w is adopted, based on wallette tests carried out at the University of Pavia on specimens made up of hollow clay bricks with a void ratio of 42%, selected as representative of typical light non-structural masonry (Fardis, 1997).
Nevertheless, there are further infill mechanical characteristics to be determined in order to define, according to the adopted model, the load-displacement relationship of the infill trusses, namely the elastic Young's modulus E_w and the shear cracking stress τ_{cr}. A certain amount of correlation certainly exists between these parameters, although it is not easy at all to be determined. In this study, the ratio between E_w and G_w is assumed equal to 10/3 based on the proposal of the Italian code (Circolare del Ministero dei Lavori Pubblici n. 617 del 2/2/2009), whereas τ_{cr} is assumed as independent on G_w, thanks to the independence between the RVs F_{infill} and D_{infill}.
As far as the modeling of uncertainty in infill mechanical properties is concerned, both mechanical and modeling variability are considered based on experimental tests from literature and on experimental-to-predicted ratios obtained from the adopted infill model (Fardis, 1997). These two sources of variability are previously considered independently of one another, then the maximum CoVs for F_{cr}, F_{max}, D_{cr} and D_{max} are obtained (0.30, 0.30, 0.30 and 0.70, respectively): the variability of the parameter D_{max} is governed by model variability, whereas all of the other values of variability are given by variability of material properties. As far as rotations at yielding and ultimate in RC members are concerned, median and CoV values are evaluated starting from the values calculated through the formulations proposed in (Fardis, 2007) and using median and CoV values of the experimental-to-predicted ratio, as illustrated by the author.
A sensitivity analysis is carried out to investigate the influence of each variable on the seismic capacity of each case study structure. To this aim, two models are generated for each RV assuming median-minus-1.7-standard-deviation and median-plus-1.7-standard-deviation values for the considered variable, and median values for the remaining variables. In addition, another analysis is carried out assuming median values for all of the variables (Model#1).

3.1. Analysis of results

The seismic capacity can be defined in terms of $S_{ae}(T_{eff})$ or PGA. The seismic capacity expressed in term of PGA –for a certain LS– is defined as the PGA corresponding to the demand spectrum under which the displacement demand is equal to the displacement capacity for that LS. PGA capacity at a certain LS is represented by the ordinate of the IN2 curves (expressed in terms of PGA) corresponding to the displacement capacity of the equivalent SDoF system at that LS. In the following, obtained results are presented and discussed for Uniformly Infilled, Pilotis and Bare frames, in both longitudinal and transverse directions and at DL and NC LSs. The [top displacement, $S_{ae}(T_{eff})$] and [top displacement, PGA] points on IN2 curves corresponding to DL and NC LSs are reported as yellow and red circles, respectively. It is to be noted that the influence of each single variable, which will be illustrated through the sensitivity analysis, not only depends on the influence of the variable on the seismic response, but also depends on the dispersion assumed for that variable through the assigned CoV, which leads to consider values more or less distant from the central (median) value.
SPO (red) and IN2 (blue) curves in terms of $S_{ae}(T_{eff})$ and collapse mechanisms for Models#1 in both direction and for each infill configuration are shown in Figure 1 for the 4-storey SLD structure. Moreover, storeys involved in all of the collapse mechanisms for Models#1 in both X and Y direction and for each infill configuration of all the analyzed structures are reported in Table 2.
SPO and IN2 curves in terms of $S_{ae}(T_{eff})$ and collapse mechanisms can be obtained for all of the other case study structures, too and for each model generated through the only variation of one RV. IN2 curves in terms of PGA can also be obtained. Trough a comparison between Model#1 and the other models for each case, change in PGA capacity at both LSs respect to Model#1 due to variations of the assumed RVs can be calculated. It is worth underlining that collapse mechanism can be significantly changed when infill mechanical properties assume their upper or lower bound respect to their median values. Results of sensitivity analysis are reported in Figures 2 and 3 for GLD and SLD structures; the effect due to the variation of each RV is briefly discussed in Tables 3 and 4.

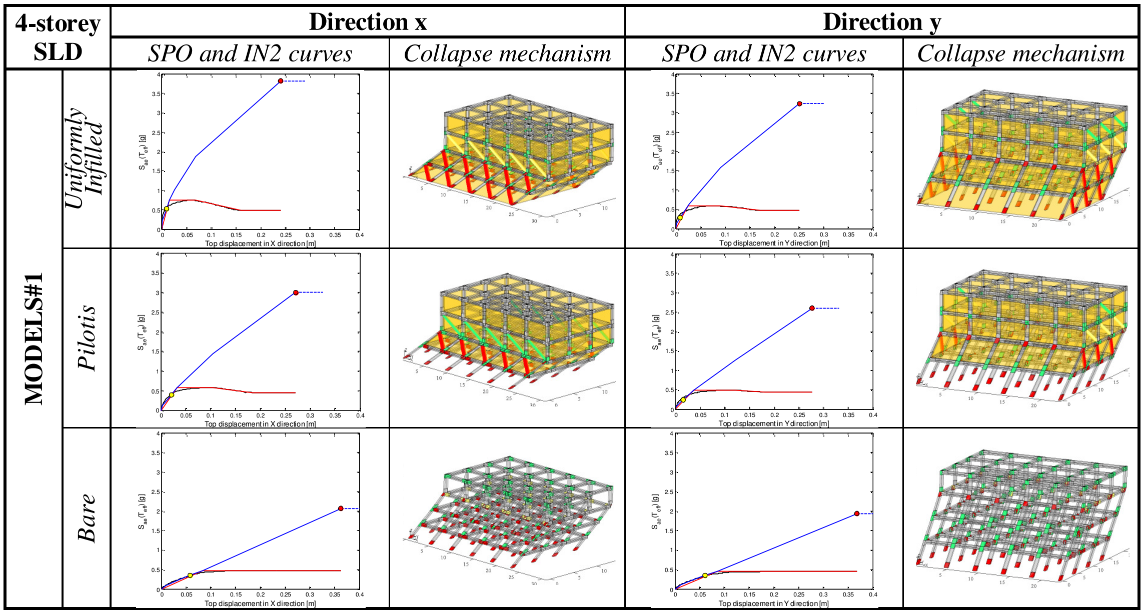

Figure 1. SPO, IN2 curves, deformed shape and element damage at NC for Models#1 (4-storey SLD)

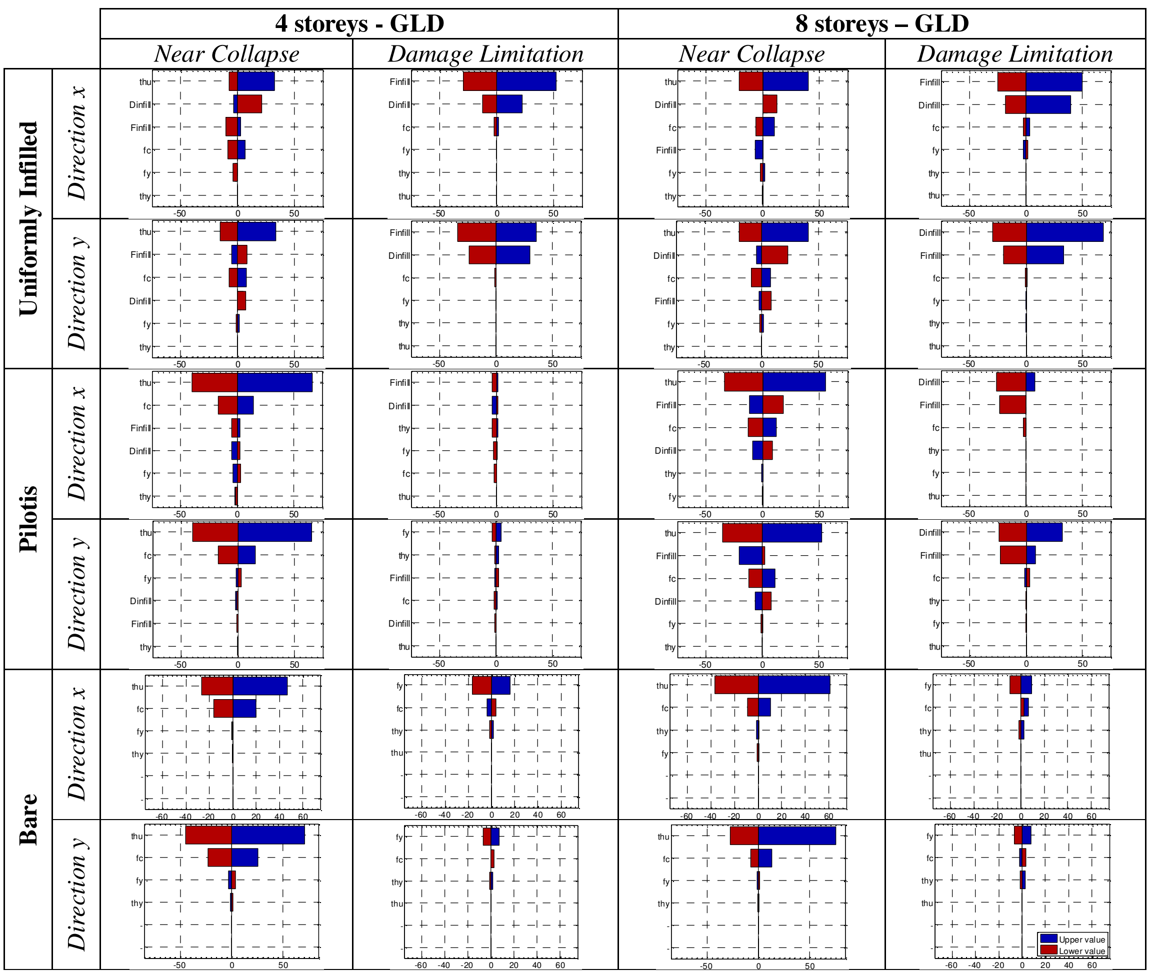

Figure 2. Change in PGA capacity (%) respect to Model#1 due to variations of the assumed RVs – GLD case study structures

Table 2. Storeys involved in the collapse mechanism for the Models#1 of each analyzed structure

Infill configuration	4-storey SLD		4-storey GLD		8-storey SLD		8-storey GLD	
	x	*y*	*x*	*y*	*x*	*y*	*x*	*y*
UI	1+2	1+2	1	2	1+2+3	1+2+3+4	2+3	3+4
P	1+2	1+2	1	1	1+2	1+2+3+4	1+2	1+2+3
B	1+2+3	1+2+3	3	global	1+2+3+4+5	1+2+3+4+5	2+3+4+5+6	global

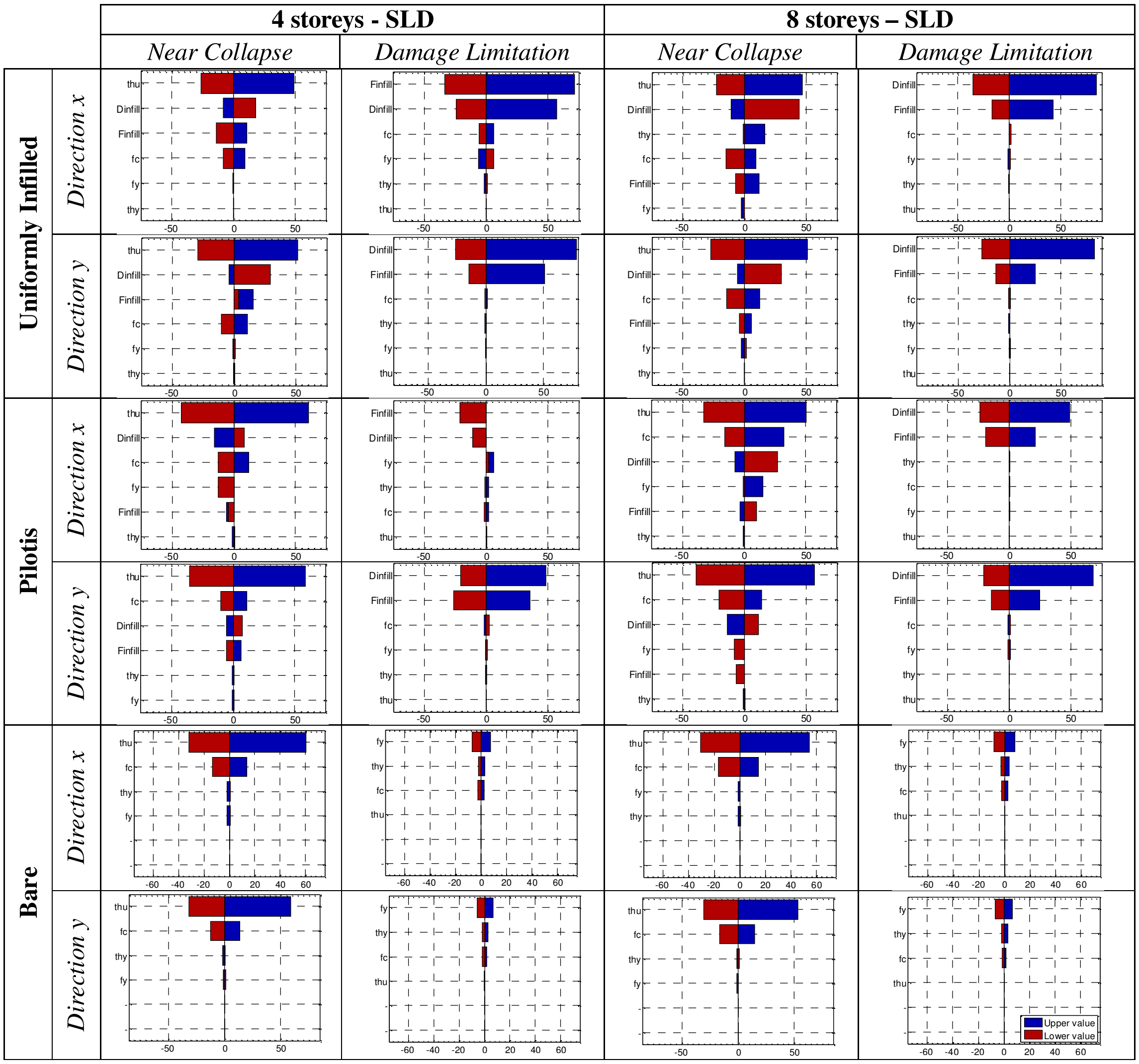

Figure 3. Change in PGA capacity (%) respect to Model#1 due to variations of the assumed RVs – SLD case study structures

Table 3. Effects and remarks about sensitivity analysis – GLD case study structures

			Gravity Load Design 4- and 8-storeys
Variable		*LS*	*Remarks*
RC parameters	θ_u	NC	parameter with the greatest influence in each case; if it increases, collapse ductility and PGA_{NC} increase.
		DL	no significant influence.
	θ_y	NC	no significant influence.
		DL	achievement of DL LS is generally due to infills, if they are present in the model; for bare configurations, an increase in θ_y produces an increase both in displacement capacity and T_{eff} of the equivalent SDoF, thus resulting in no change of PGA_{DL}.
	f_c	NC	when it increases, the axial load ratio in columns decreases and θ_u increases; consequently ductility at collapse and PGA_{NC} increase.
		DL	no significant influence.
	f_y	NC	no significant influence.
		DL	parameter with the greatest influence for bare configurations due to an increase in base shear strength C_s and displacement capacity Δ_{DL}; not important for infilled configurations when achievement of DL LS is due to infills.
Infills parameters	F_{infill}	NC	great influence on uniformly infilled configurations through the variation of collapse mechanism (i.e. 8-storey structures), of maximum strength $C_{s,max}$ and of T_{eff} ; its influence is smaller than θ_u and it is different depending on the case-study structure
		DL	its influence is higher for Uniformly Infilled configurations rather than for Pilotis ones; in both cases if it increases a beneficial decrease in T_{eff} is produced, thus resulting in an increase in PGA_{DL}
	D_{infill}	NC	except for variation of collapse mechanism (i.e., 8 storeys-Uniformly Infilled-longitudinal direction), when it increases the yielding displacement of the equivalent SDoF Sdy increases too, whereas the maximum strength $C_{s,max}$ and the displacement capacity Δ_{coll} do not change, then ductility at collapse and PGA_{NC} decrease
		DL	except for the 4 storeys-Pilotis-longitudinal direction case-study structure, if it increases T_{eff} increases and displacement capacity Δ_{DL} increases more than yielding displacement of the equivalent SDoF S_{dy}, and so collapse ductility and PGA_{DL} increase

Table 4. Effects and remarks about sensitivity analysis – SLD case study structures

			Seismic Load Design 4- and 8-storeys
Variable		*LS*	*Remarks*
RC parameters	θ_u	NC	parameter with the greatest influence in each case; if it increases, collapse ductility and PGA_{NC} increase.
		DL	no significant influence.
	θ_y	NC	it becomes important just in one case-study (i.e. 8 storeys-Uniformly Infilled-longitudinal direction) where, if it increases, a change in the collapse mechanism is produced and PGA_{NC} increases.
		DL	achievement of DL LS is generally due to infills, if they are present in the model; for Bare configurations, an increase in qy produces an increase both in displacement capacity and T_{eff} of the equivalent SDoF, thus resulting in no significant change in PGA_{DL}.
	f_c	NC	when it increases, the axial load ratio in columns decreases and θ_u increases; consequently ductility at collapse and PGA_{NC} increase.
		DL	only in one case (i.e. 4storeys-Uniformly Infilled-longitudinal direction), if it increases, an increase in base shear strength C_s is observed, thus leading to a beneficial decrease in T_{eff}.
	f_y	NC	important only for Pilotis configurations (i.e. 4- and 8- storeys-Pilotis-longitudinal direction, 8 storeys-Pilotis-transverse direction) where its change produces a variation of collapse mechanism.
		DL	parameter with the greatest influence for bare configurations due to an increase in base shear strength C_s and displacement capacity Δ_{DL}; not important for infilled configurations when achievement of DL LS is due to infills.
Infill parameters	F_{infill}	NC	except for variation of collapse mechanism, when it increases, the yielding displacement of the equivalent SDoF S_{dy} decreases and the maximum base shear strength $C_{s,max}$ increases.
		DL	except for variation of collapse mechanism in which displacement capacity Δ_{DL} decreases (i.e., 4 storeys-Pilotis-longitudinal direction), if it increases, an increase in base shear strength C_s and a decrease in S_{dy} are produced, thus leading to a higher ductility capacity and PGA_{DL}.
	D_{infill}	NC	when it increases, the yielding displacement of the equivalent SDoF S_{dy} increases too, whereas the maximum strength $C_{s,max}$ and the displacement capacity Δ_{coll} do not change, and then ductility at collapse and PGA_{NC} decrease.
		DL	except for variation of collapse mechanism (i.e. 4 storeys-Pilotis-longitudinal direction), if it increases T_{eff} increases and displacement capacity Δ_{DL} increases more than yielding displacement of the equivalent SDoF S_{dy}, then ductility at collapse and PGA_{DL} increase.

4. COMPARISONS AND REMARKS: INFILL CONFIGURATION, DESIGN TYPOLOGY AND NUMBER OF STOREYS

In this Section, the influence of the infill configuration, the design typology and the number of storeys on the seismic capacity of each case study building is evaluated.
To this aim, IN2 curves are compared, always referring to the models where median values are assumed for all of the variables. The comparison is carried out in both directions.

4.1. Influence of the infill configuration

First of all, a comparison between three different infill configurations can be carried out about four-storey SLD or GLD case study structures.
If the four-storey SLD structures are considered: *(i)* the Bare configuration shows the highest PGA capacity at DL LS (PGA_{DL}), both in longitudinal and transverse directions; *(ii)* Uniformly Infilled and Pilotis configurations have almost the same PGA_{DL}; *(iii)* the Uniformly Infilled and the Pilotis configurations show the highest and the lowest PGA capacity, respectively, at NC LS (PGA_{NC}) in both directions. If the four-storey GLD structures are considered, instead: *(i)* the Uniformly Infilled and the Pilotis configurations show the highest and the lowest PGA capacity, respectively, at DL LS (PGA_{DL}) in both directions; *(ii)* Uniformly Infilled configuration has the highest PGA_{NC}.
Similar conclusions can be drawn for eight-storey structures, thus leading to the following general remarks: *(i)* at NC LS, a beneficial effect on PGA capacity generally exists when a regular distribution of infill panels is considered, whereas a detrimental effect is shown by structures with an irregular infill distribution; *(ii)* at DL LS, the above mentioned beneficial effect is not observed for SLD structures; *(iii)* displacement capacity, as expected, is higher for Bare structures both in longitudinal and transverse directions at each LS; *(iv)* ratios between Uniformly Infilled or Pilotis PGA capacity and Bare PGA capacity are higher for GLD structures whose seismic performances are more affected by infill presence respect to SLD structures; *(v)* in SLD structures the presence of infills changes also significantly the collapse mechanism expected for the Bare configuration designed according to Capacity Design principles, thus influencing displacement, ductility and PGA capacities.

4.2. Influence of the design typology

A second kind of comparison can be carried out in order to show how the seismic capacity is affected by infills depending on the design typology of the structure.
At DL LS: *(i)* in the Uniformly Infilled configuration, GLD structures show a concentration of displacement demand at the first storey from early range of loading, resulting in a stiffening of the multi-linearized SPO curve and an increase in PGA_{DL} respect to SLD Uniformly Infilled structure; *(ii)* in Pilotis and Bare configurations, SLD structures show the highest displacement capacity and PGA_{DL}; *(iii)* in Pilotis configurations, in transverse direction, the absence of internal beams for GLD structures leads to a higher deformability and displacement capacity.
At NC LS: *(i)* SLD structures, respecting Capacity Design principles, show collapse mechanisms involving a greater number of storeys and a higher collapse ductility and PGA_{NC} respect to GDL structures; *(ii)* GLD Bare structure in transverse direction has a greater displacement capacity because in this case a global collapse mechanism is observed.
Similar conclusions can be drawn for eight-storey structures, thus leading to the following general remarks: *(i)* SLD structures generally show the best seismic performances at NC LS; *(ii)* as far as Uniformly Infilled configurations are concerned, GLD structures show the highest PGA_{DL}; *(iii)* exceptions to the above conclusions are the cases (for eight-storey structures) in which there is a concentration of displacement demand at bottom floors leading to an increase in PGA_{NC} due to a stiffening of the multi-linearized SPO curve (e.g., eight-storey Pilotis in longitudinal direction).

4.3. Influence of the number of storeys

A further analysis can be carried out about structural seismic capacity depending on the number of storeys. The same trends are shown by four- and eight-storey analyzed structures. It is worth noting that *(i)* eight-storey structures have a greater variability in collapse mechanisms depending on the infill configuration, input parameters and design typology and *(ii)* eight-storey structures are less affected by infill presence in terms of PGA_{DL}.

5. CONCLUSIONS

In this paper, the effect of main parameters influencing the seismic capacity of the case study structures has been investigated through a sensitivity analysis. Such analysis has shown that the ultimate rotational capacity of columns, directly influencing the displacement capacity, has the highest influence on seismic capacity at NC. At DL mechanical characteristics of infills have the highest influence on the response of the Uniformly Infilled frame, whereas for Pilotis and above all for Bare frames also steel yield strength has a relatively high influence. Presence of infills significantly influences the collapse mechanism even if the Bare structure is designed according to Capacity Design principles; nevertheless, seismic performances of GLD structures are more affected by infill presence respect to SLD structures. At NC, a beneficial effect on seismic capacity generally exists when a regular distribution of infill panels is considered, whereas a detrimental effect is shown by structures with an irregular infill distribution. The same trends are shown by four- and eight-storey structures, but eight-storey structures have a greater variability in collapse mechanisms, and they are less affected by infill presence. However, it should be pointed out that special attention should be addressed to the potential brittle failure mechanisms due to the local interaction between masonry infills and structural RC elements – which have not been accounted for herein – especially for existing RC buildings that have not been designed adopting general principles and detailing rules prescribed by modern seismic codes according to Capacity Design philosophy.

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