Influence of Infill Presence and Design Typology on Seismic Performance of RC Buildings: Fragility Analysis and Evaluation of Code Provisions at Damage Limitation Limit State

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SUMMARY:
The same case study structures of the companion paper (Ricci et al., 2012) are analyzed. Seismic capacity at different Limit States is assessed by means of Static Push-Over analyses, within the N2 spectral assessment framework. Fragility curves are obtained, through the application of a Response Surface Method. Seismic performance is also expressed in terms of failure probability, given a reference time period.
Italian code provisions for seismic assessment at Damage Limitation Limit State are discussed: code allows to take into account the presence of infills - if a bare numerical model, as usual, is used - by assuming a fictitious displacement capacity limit (e.g., 5‰ Interstorey Drift Ratio); code also allows to limit the maximum Interstorey Drift Ratio in a Reinforced Concrete structure to values related to masonry (e.g., 3‰ for unreinforced masonry) if infills are explicitly taken into account in the structural model. The conservatism of these provisions, depending on the number of storeys and the design typology, is analyzed.

Keywords: Reinforced Concrete, infills, Damage Limitation, Italian code provisions, fragility analysis

1. INTRODUCTION

The same case study structures of the companion paper are analyzed. Seismic capacity is assessed by means of Static Push-Over (SPO) analyses, within the N2 spectral assessment framework. Fragility curves are obtained, through the application of a Response Surface Method (RSM) and seismic performances are also expressed in terms of failure probability, given a reference time period.
Code provisions for seismic assessment at Damage Limitation (DL) Limit State are discussed: when assessing seismic capacity at DL, code (Decreto Ministeriale del 14/1/2008) allows to take into account the presence of infills - if a bare numerical model, as usual, is used - by assuming a fictitious displacement capacity limit (e.g., 5‰ Interstorey Drift Ratio). Moreover the Italian “Circolare Esplicativa” (Circolare del Ministero dei Lavori Pubblici n. 617 del 2/2/2009) allows to limit the maximum Interstorey Drift Ratio (IDR) in a Reinforced Concrete (RC) structure to values related to masonry (e.g., 3‰ for unreinforced masonry) if infills are explicitly taken into account in the numerical model (Section C 8.3). The conservatism of such a provision – depending on the number of storeys and the design typology - is analyzed by comparing seismic capacity at DL assessed according to this kind of procedure to seismic capacity assessed by adopting a "true" infilled model and a displacement capacity limit directly related to the damage of infill panels. So, three different Damage Limit States (LSs) are considered: “Damage Limitation” (DL) on Uniformly Infilled frames corresponding to the top displacement when the last infill in a storey reaches its maximum resistance thus starting to degrade (Ricci et al., 2012), “First 005” on Bare frames corresponding to the top displacement when an IDR equal to 5‰ is reached for the first time on the Bare model, and “First 003” on Uniformly Infilled frames corresponding to the top displacement when an IDR equal to 3‰ is reached for the first time on the Uniformly Infilled model.

2. FRAGILITY ANALYSIS

2.1. Methodology

First of all, the methodology used for the evaluation of fragility curves for the case study structures is illustrated.

A fragility curve represents a relationship between a seismic intensity parameter and the corresponding probability of exceedance of a given damage threshold (typically represented by a displacement capacity). The Peak Ground Acceleration PGA capacity – at 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. If PGA capacity is "observed" in a population of buildings, according to a frequentistic approach the cumulative frequency distribution of these observations provides the fragility curve (based on PGA seismic intensity measure) for that population of buildings and for that Limit State, based on the definitions themselves of fragility curve and PGA capacity. In this paper, such population of buildings is generated by a number of samplings of some Random Variables – which are input parameters to the determination of the PGA capacity (e.g., material characteristics or capacity parameters) – defined by Probability Density Functions describing the expected values and the corresponding variability, according to a Monte Carlo simulation technique. A stratified sampling of Random Variables is executed through the Latin Hypercube Sampling (LHS) technique (McKay et al., 1979), assuming a "median" sampling scheme (Vorechovsky and Novak, 2009). Nevertheless, it would be too computationally demanding to carry out a SPO analysis (for calculating the PGA capacity) for each sample of the chosen Random Variables. Hence, a RSM is applied (Pinto et al., 2004), assuming a second-order polynomial relationship between the PGA capacity, assumed as the scalar output variable, and the selected Random Variables, assumed as input variables. The design of *experiments* needed to determine such relationship is carried out according to the Central Composite Design (CCD) method. Hence, the number of experiments adds to $n=1+2k+2^k$, if k input variables are assumed. In our case, the input variables are the Random Variables selected for the sensitivity analysis (Ricci et al., 2012); in addition, the strength reduction factor R evaluated from R-μ-T relationship is assumed as a Random Variable, too. The estimate of the uncertainty in the evaluation of R – given a value of μ corresponding to the ductility capacity at a given LS – derives from the record-to-record variability observed in the results of the nonlinear dynamic analyses carried out on SDoF systems (with several records) to obtain such R-μ-T relationships. Hence, the strength reduction factor R is treated as a Random Variable: the value of R calculated by means of the given R-μ-T relationship is assumed as the median value, and the corresponding variability is taken into account by assuming the lognormal standard deviation β_R as a function of μ, depending on the characteristics of the SDoF backbone, from (Vamvatsikos and Cornell, 2006). It is to be noted that the assumption of R as a Random Variable does not imply the execution of further SPO analyses. In order to apply the illustrated procedure, the considered input variables are represented by the Random Variables normalized to their median values (see Table 1).

Table 1. Random Variables assumed to evaluated fragility curves

Variable	Distribution	Median Value	CoV (SLD)	CoV (GLD)
$\underline{f}_c$	Lognormal	1	0.20	0.31
$\underline{f}_y$	Lognormal	1	0.06	0.08
$\underline{\theta}_y$	Lognormal	1.015	0.331	0.331
$\underline{\theta}_u$	Lognormal	0.995	0.409	0.409
$\underline{F}_{infill}$	Lognormal	[1;1]	[0.30;0.30]	[0.30;0.30]
$\underline{D}_{infill}$	Lognormal	[1;1]	[0.30;0.70]	[0.30;0.70]
$\underline{R}$	Lognormal	1	$f(\mu)$	$f(\mu)$

Hence, the number of experiments adds to $n=1+2\cdot7+2^7=143$ (if infills are present, as in Uniformly Infilled and Pilotis frames) or $1+2\cdot5+2^5=43$ (if infills are not present, as in Bare frame) for each case study, in each direction. Note that the results of the SPO analyses carried out with the sets of values corresponding to the $2\cdot k$ "star points", whose position is assumed at a distance of 1.7 times the standard deviation from the centre of design (Liel et al., 2009), was illustrated in the sensitivity analysis (Ricci et a., 2012).

The resulting PGA capacity data allow to estimate the second-order polynomial relationship between the PGA capacity and the assumed Random Variables. Subsequently, a LHS of the k=7 considered Random Variables is carried out, thus obtaining m sets of values of these variables. In particular, m=1000 samplings are executed. The m×k obtained sampling matrix is used to estimate – through RSM – the corresponding m values of PGA capacity. This procedure is carried out 12 times for each case study structure (note that the same sampling matrix is always used; obviously in Bare frame only 5 of the 7 columns of the matrix are used). The corresponding cumulative frequency distributions of the obtained PGA capacity values provide the 12 fragility curves for Uniformly Infilled, Pilotis and Bare frames, in X and Y directions and at DL and NC LSs for each case study structure. Results are illustrated in the following. The comparison between the median values of the fragility curves reported herein has been actually already carried out through the observations reported in the companion paper (Ricci et al., 2012), when comparing the seismic capacities of Uniformly Infilled, Pilotis and Bare frames referring to Models#1 (median values for each Random Variable) for each case study. From a qualitative standpoint, the slope of the fragility curves – representing the variability associated with the seismic capacity – depends on the amount of variation in PGA capacity with the variation in each Random Variable, shown in the sensitivity analysis (Ricci et al., 2012). Lower this amount, lower the change in PGA capacity with the change in Random Variables, less sensitive the PGA capacity to the modelled uncertainties, steeper the fragility curve (as often happens, for instance, at DL Limit State). Moreover, further variability due to the variability of the strength reduction factor R affects all of the fragility curves, but to a different extent: larger the ductility capacity at the LS of interest, larger the variability of the corresponding strength reduction factor R, larger the increase in the variability of PGA capacity.

2.2. Analysis of results

4-storey GLD: As far as fragility curves at NC LS in X direction is concerned, a quite close median seismic capacity is noted between Uniformly Infilled and Pilotis frames, whereas the Bare frame results as the more vulnerable. Fragility curves at DL LS in X direction highlight the beneficial effect of uniformly distributed infills on the seismic capacity at this LS, that is, for relatively low seismic demand. Moreover, it is observed how in this case the detrimental effect of localization in displacement demand leads to a lower capacity of the Pilotis frame, compared with the remaining ones. Nevertheless, the relatively low slope of the fragility curve for the Uniformly Infilled frame reflects the particularly high influence of the uncertainty in mechanical properties of infill panels on the seismic capacity of this frame at DL. In Y direction the fragility curves at NC LS highlight that the best seismic performance is provided by the Uniformly Infilled frame. Moreover, also in Y direction the beneficial effect of the increase in stiffness and strength provided by uniformly distributed infills on the seismic capacity at DL LS is clearly shown.

8-storey GLD: As far as fragility curves at NC LS in X direction is concerned, a closer median seismic capacity respect to the previous case is noted between all the infilled configurations, thus highlighting the lower influence of infills on the 8-storey case study structures. Fragility curves at DL LS in X direction highlight the beneficial effect of uniformly distributed infills on the seismic capacity at this LS, similarly to the 4-storey GLD case study structure, whereas a quite close median seismic capacity is noted between Bare and Pilotis frames. Again, the relatively low slope of the fragility curve for the Uniformly Infilled frame reflects the particularly high influence of the uncertainty in mechanical properties of infill panels on the seismic capacity of this frame at DL. In Y direction, respect to the previous case, the fragility curves show that *(i)* at NC LS, seismic performance of Pilotis is better compared with the Bare frame and *(ii)* at DL LS, Bare frame is less vulnerable than the Uniformly Infilled frame.

4-storey SLD: Fragility curves at NC LS in X direction (see Figure 1) highlight the beneficial effect of uniformly distributed infills on the seismic capacity and the detrimental effect of localization in displacement demand leading to a lower capacity of the Pilotis frame, compared with the remaining ones. At DL LS, in both directions, seismic performance of Bare frame is better compared with the other infilled frames.

8-storey SLD: Fragility curves at NC LS in both directions (see Figure 1) highlight the beneficial effect of uniformly distributed infills on the seismic capacity whereas a quite close median seismic capacity is noted between Bare and Pilotis frames, at this LS. At DL LS, in both directions, seismic performance of Bare frame is better compared with the other infilled frames.

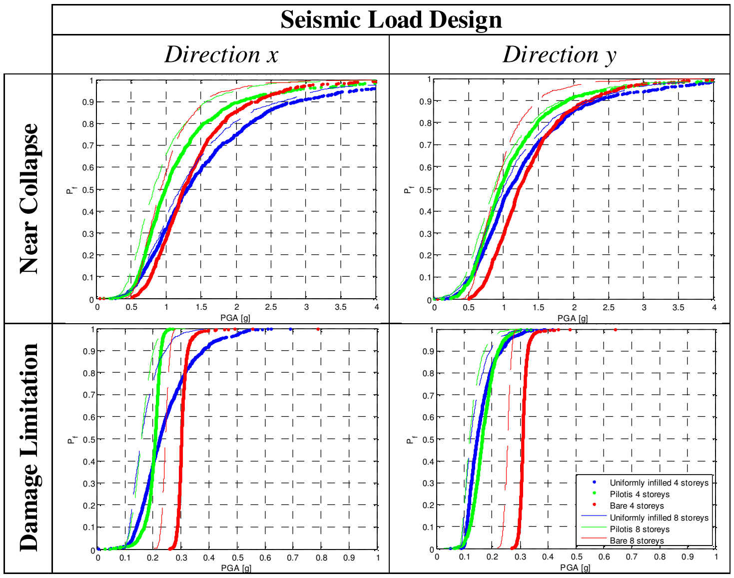


Figure 1. Fragility curves – SLD case study structures

Table 2. Estimated parameters of the cumulative lognormal distributions fitting the fragility curves

		Seismic Load Design								Gravity Load Design							
		4 storeys				*8 storeys*				*4 storeys*				*8 storeys*			
		DL		NC		DL		NC		DL		NC		DL		NC	
		$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}
UI	*x*	0.226	0.40	1.333	0.59	0.1641	0.29	1.243	0.56	0.394	0.32	0.904	0.57	0.234	0.39	1.073	0.50
	y	0.152	0.29	1.108	0.61	0.1308	0.28	0.999	0.67	0.254	0.34	0.848	0.58	0.173	0.33	0.763	0.71
P	*x*	0.199	0.18	1.052	0.52	0.1575	0.21	0.851	0.49	0.156	0.06	0.786	0.47	0.164	0.31	0.955	0.49
	y	0.164	0.25	0.980	0.55	0.1242	0.24	0.880	0.61	0.148	0.04	0.608	0.45	0.137	0.27	0.684	0.73
B	*x*	0.306	0.07	1.282	0.43	0.2454	0.05	0.955	0.38	0.218	0.13	0.571	0.34	0.169	0.10	0.911	0.40
	y	0.310	0.05	1.262	0.42	0.2548	0.04	0.920	0.37	0.160	0.06	0.639	0.49	0.236	0.06	0.641	0.39

Fragility curves can be fitted by lognormal cumulative distributions: parameters are reported in Table 2, with $\overline{\boldsymbol{PGA}}$ and β_{PGA} representing the estimated median (expressed in [g]) and logarithmic standard deviation of PGA capacity, respectively.
The latter provides an useful indication about the overall sensitivity of seismic capacity to the variability of the parameters mainly influencing the seismic response.

2.3. Evaluation of failure probability

A comparison between DL and NC LSs can be carried out also in terms of failure probability. The failure probability (P_f) of a structural system characterized by a resistance R under a seismic load S can be evaluated as

$$P_f = \int_0^{+\infty} f_S(S) F_R(S) dS \tag{2.3.1}$$

where $f_S(S)$ is the Probability Density Function (PDF) of the seismic intensity parameter and $F_R(S)$ is the probability that the resistance R is lower than a level S of seismic intensity. Hence, $F_R(S)$ is represented by a fragility curve, whereas the PDF of the seismic intensity S – in a given time window – is obtained from seismic hazard studies: based on the seismic hazard data provided by (INGV-DPC

S1, 2007) for the Italian territory, if the coordinates of the site of interest are given, PGA values corresponding to different return periods (T_R) can be determined. Hence, given a PGA value, the corresponding T_R(PGA) can be calculated. Finally, given a time window (V_R), the exceeding probability of the same PGA is given by the Poisson process:

$$P_{V_R}(PGA) = 1 - e^{-\frac{V_R}{T_R(PGA)}} \tag{2.3.2}$$

In the procedure described herein, PGA is assumed as seismic intensity parameter S, $F_R(S)$ is represented by the calculated fragility curves (assuming a linear interpolation between subsequent values of PGA) and $f_S(S)$ is derived from P_{VR}(PGA), by calculating the PDF of PGA corresponding to the Complementary Cumulative Distribution Function (CCDF) of PGA represented by P_{VR}(PGA). Hence, the failure probability P_f is calculated through Eqn. 2.3.1, by means of a numerical integration based on Simpson quadrature. Failure probabilities at DL and NC LSs are calculated for each frame, based on the fragility curves previously obtained and the seismic hazard described by the PGA exceeding probability in 50 years, obtained from (INGV-DPC S1, 2007) for the site of interest (Lon.: 14.793, Lat.: 40.915).

The failure probabilities P_f calculated for a time window of 50 years are reported in Figure 2 for each case study structure, showing a direct comparison between different infill configurations, number of storeys, considered directions and LSs.

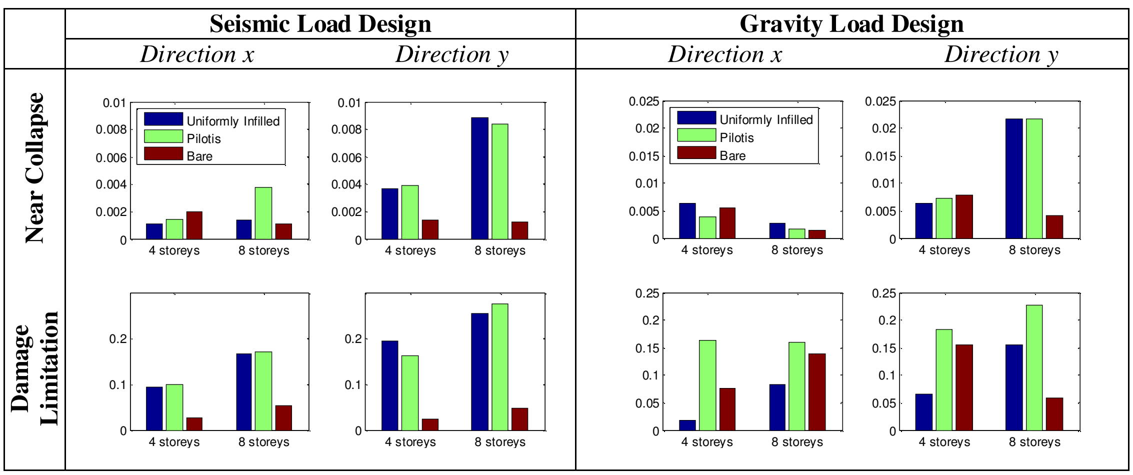


Figure 2. Failure probabilities P_f in 50 years for SLD and GLD case study structures

3. DAMAGE LIMITATION LIMIT STATE: ANALYSIS OF CODE PROVISIONS

When assessing seismic capacity at DL, Italian technical code allows to take into account the presence of infills - if a bare numerical model, as usual, is used - by assuming a fictitious displacement capacity limit - e.g., 5‰ IDR if infills are attached to the structure and they significantly influence the structural deformability (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2). Moreover the Italian "Circolare Esplicativa" (Circolare del Ministero dei Lavori Pubblici n. 617 del 2/2/2009 Section C 8.3) allows to limit the maximum Interstorey Drift Ratio in a RC structure to values related to masonry (e.g., 3‰ for unreinforced masonry) if infills are explicitly taken into account in the numerical model (C 8.3).

In order to evaluate the conservatism of such provisions two comparisons are performed:

1. one between seismic capacity at DL assessed on a bare model by limiting the maximum IDR to 5‰ ("First 005" LS) and seismic capacity at "Damage Limitation" – as defined in companion paper – assessed by adopting an infilled model and a "true" displacement capacity limit;

2. one between seismic capacity at DL assessed on a bare model by limiting the maximum IDR to 5‰ ("First 005" LS) and seismic capacity at DL assessed on an infilled model by limiting the maximum IDR to 3‰ ("First 003" LS).

For both comparisons and for each case study structure, IN2 curves for Models#1 (median values for all the Random Variables) in terms of $S_{ae}(T_{eff})$ or PGA can be obtained; points on IN2 curves corresponding to "Damage Limitation", "First 005" and "First 003" LSs are reported as yellow circle, square and diamond, respectively. Red circles are related to the NC LS, which is not analyzed in this section. In order to carry out the above described two comparisons, it is important to underline the effect of the infill presence on the capacity of the infilled model respect to the bare one: infill panels increase the effective stiffness of the infilled structure and, on the other hand, reduce its displacement capacity respect to the bare structure.

It can be observed that displacement capacity at "First 005" in Bare models is generally higher than that evaluated on the Uniformly Infilled ones at "First 003" or at "Damage Limitation" Limit States because of a higher deformability and a more uniform distribution of IDR demand.

Moreover, displacement capacity – and also strength C_s and PGA capacity – at "First 003" is generally higher than displacement capacity at "Damage Limitation" (both evaluated on the Uniformly Infilled models).

If the parameter F_{infill} increases with equal D_{infill} the displacement capacity at "Damage Limitation" LS, Δ_{DL}, does not significantly change, but an increase in effective stiffness is produced thus leading to an increase in $S_{ae}(T_{eff})$ (Figure 3 (b)) and also in PGA capacity, PGA_{DL}. If D_{infill} increases with equal F_{infill}, both the displacement capacity Δ_{DL} and the effective period increase, but the former effect prevails over the latter and $S_{ae}(T_{eff})$ (Figure 3 (a)), and consequently PGA_{DL}, increase, too. The parameter F_{infill} only has one beneficial effect on the sesmic capacity, whereas D_{infill} produces two opposite effects, one beneficial and one detrimental, thus leading to a higher influence of F_{infill} on seismic performance respect to D_{infill}.

It is worth to highlight that code provisions can be considered as conservative if PGA capacity estimated according to them is higher or equal to the "true" PGA capacity. Higher the PGA_{DL} evaluated on the Uniformly Infilled model, more conservative the code provision (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2); hence, higher F_{infill} or D_{infill}, more conservative the code provision.

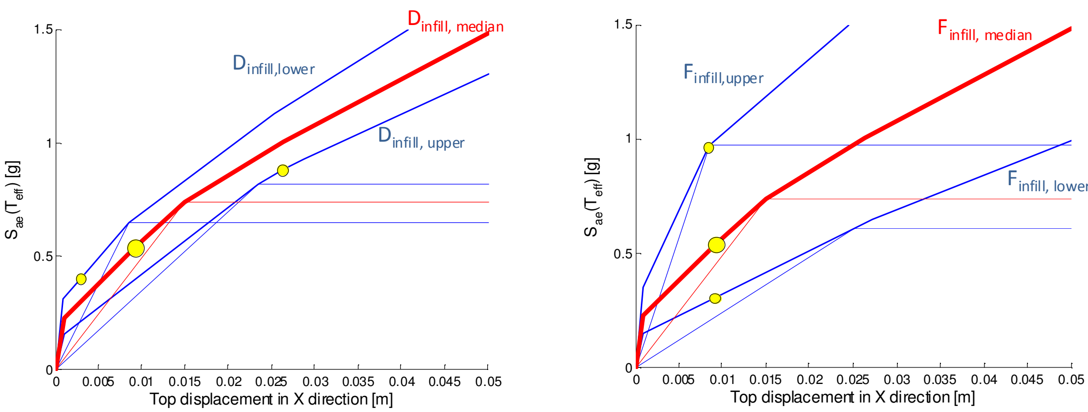
If Models#1 of the 4-storey SLD and the 8-storey GLD structures are considered, the PGA capacity at "Damage Limitation" LS of the Uniformly Infilled structure is higher than the PGA capacity at "First 005" LS on the Bare one in longitudinal direction (x), but not in the transverse one (y): in x direction, the presence of infills significantly increases the strength C_s (respect to the Bare model) and the beneficial increase in the effective stiffness prevails on the decrease in displacement capacity, thus leading to an increase in $S_{ae}(T_{eff})$ and PGA capacity at "Damage Limitation"; in y direction, this effect is less significant because of the lower percentage presence of infills.

If Models#1 of the 4-storey GLD case study is considered, the PGA capacity at "Damage Limitation" LS on the Uniformly Infilled structure is higher than the PGA capacity at "First 005" LS on the Bare one in both directions: in this case the strength C_s in both directions is much higher than the same strength in Bare model, due to the contribution of the infills. Opposite remarks are valid about Models#1of the 8-storey SLD structure.

Therefore, the fictiotious overestimation of capacity displacement suggested by Italian code when using a Bare numerical model, compared with the "real" capacity displacement (evaluated on the infilled configuration) can be counterbalanced by a reserve of strength due to infills - neglected in the Bare model –provided that infill strength contribution is high enough.

Results of such comparisons in terms of IN2 curves are reported in Figure 4 for 4-storey case study structures. Based on these comparisons and on the analysis of the 8-storey case study structures too, we can conclude that PGA capacity at "Damage Limitation" for Uniformly Infilled models is higher than PGA capacity at "First 005" for Bare models only if infill presence produces a considerable increase in strength – in terms of percentage contribution to the base shear respect to one due to RC members – such as Models#1 in longitudinal direction of 4-storey SLD structure, both directions of 4-storey GLD structure, longitudinal direction of 8-storey SLD structure. Therefore, in these cases, code provision (Section 7.3.7.2) can be defined as conservative.

It is worth calculating, for each case study, the “equivalent maximum IDR”, i.e. the maximum IDR demand on the Bare frame when the top displacement evaluated on its IN2 curve corresponds to the capacity PGA_{DL} of the Uniformly Infilled structure. Such “equivalent maximum IDR” is the IDR capacity that should be assumed when using a numerical Bare model in order to obtain a reliable estimate of PGA capacity at DL LS, that is, the same PGA capacity obtained using a “true” infilled model. According to code provisions, this value is equal to 5‰. In Figure 5 the “equivalent maximum IDR” is plotted on y axis for all the Models#1 and for Models corresponding to the only variation of the parameters F_{infill} or D_{infill} respect to their median values.



(a) (b)

Figure 3. Multi-linearized SPO and IN2 curves for Model#1 (red line) and for Upper and Lower values (blue lines) of D_{infill} (a) and F_{infill} (b) – 4-storey SLD, longitudinal direction

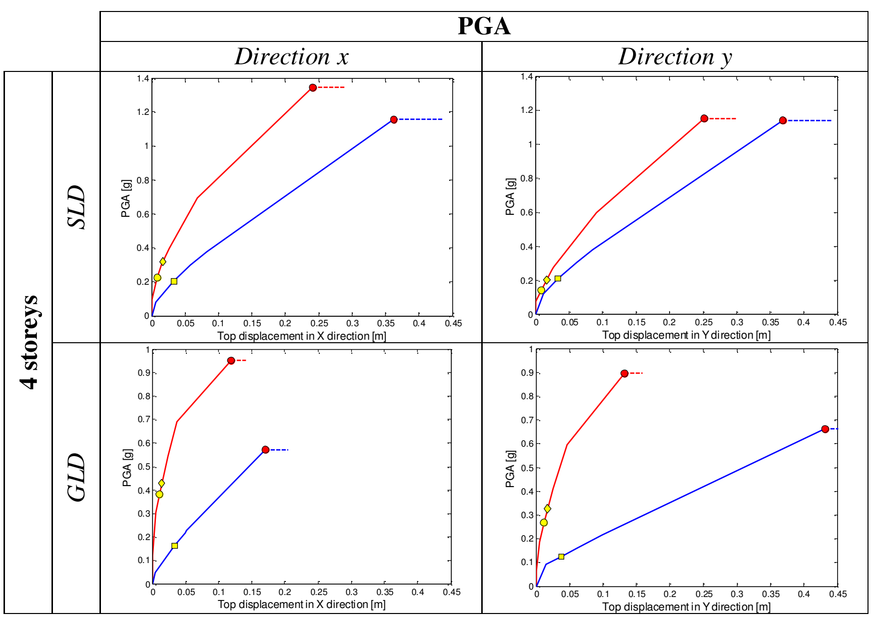


Figure 4. IN2 curves in terms of PGA for 4 storey case study structures – comparison between Uniformly Infilled (red) and Bare (blue) models

It can be observed that the “equivalent maximum IDR” is higher for GLD structures rather than for SLD ones, for 4-storey rather than for 8-storey structures, for the longitudinal direction rather than for the transverse one, and for upper values of the parameters F_{infill} or D_{infill} rather than for lower ones. This trend is tentatively expressed through the use of a parameter that takes into account both the strength and deformation capacity of infills, on a side, and the extent of the contribution of infill elements to the response of the infilled RC structure, on the other side. Such parameter is calculated as the ratio between the sum of the areas underneath the force–displacement envelope of the infills, summed up for all of the infill panels at first storey in the considered direction, and the plastic flexural shear capacity of RC columns at the first storey.

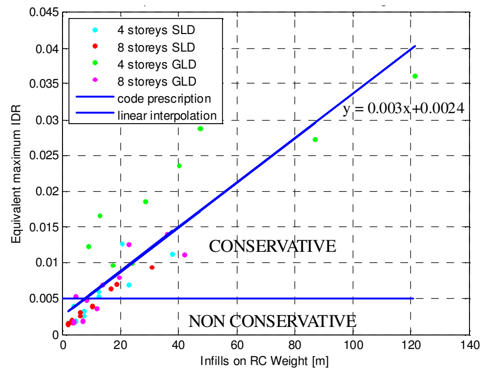


Figure 5. "Equivalent maximum IDR" versus "Infills-on-RC Weight" [kNm/kN]

This ratio will be referred to as "Infills-on-RC Weight" reported on x axis in [kNm/kN] in Figure 5. Figure 5 shows that the maximum IDR prescribed by code (i.e. 5‰) may be too conservative for some configurations (e.g., 4-storey GLD structure).

Moreover, it can be observed that the equivalence between "First 005" LS on Bare model and "First 003" LS on Uniformly Infilled model – that is implicitly assumed by Italian code (C 8.3) – is not generally confirmed by SPO analyses. It can also be observed that when code provision (Section 7.3.7.2) is conservative in the first comparison it is conservative in the second comparison, too.

3.1. Fragility curves

If fragility curves related to the described Damage Limitation LSs in both directions and for both design typologies and number of storeys are analyzed, a higher seismic vulnerability is always observed for "Damage Limitation" LS respect to "First 003" LS, thus highlighting that code provisions (C 8.3) are not conservative for these case study structures. If fragility curves related to SLD structures (Figures 6) are observed, it is more difficult to predict if seismic vulnerability is higher at "Damage Limitation" or at "First 005" LS; whereas 4-storey GLD structure, in both directions, is more vulnerable at "First 005" LS: in this case, code provisions (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2) can be defined as conservative. The relatively low slope of the fragility curve for the Uniformly Infilled frames reflects the particularly high influence of the uncertainty in mechanical properties of infill panels on the seismic capacity of these frames at Damage LSs.

Fragility curves at "First 005" related to Bare frames are instead very steep, because of the very low influence of the variability of RC parameters on the seismic capacity (it is worth highlighting that ultimate rotation capacity in RC members is not involved in Damage Limitation LS, as already shown in (Ricci et al., 2012)). Fragility curves can be fitted by lognormal cumulative distributions: parameters are reported in Table 3, with $\overline{\boldsymbol{PGA}}$ and β_{PGA} representing the estimated median (expressed in [g]) and logarithmic standard deviation of PGA capacity, respectively. The latter provides a useful indication about the overall sensitivity of seismic capacity to the variability of the parameters mainly influencing the seismic response.

3.2. Evaluation of failure probability

A comparison between "Damage Limitation", "First 003" and "First 005" Limit States can be carried out also in terms of failure probability P_f.

It is to be noted that the methodology for evaluating vulnerability curves in longitudinal or transverse direction has been previously described, whereas a single fragility curve for each case can be evaluated *independent* of the direction by assuming the PGA capacity, for each of the m=1000 sets of samplings, as the minimum value between longitudinal and transverse directions.

Hence, a fragility curves, *independent* of the direction, can be obtained as the cumulative frequency distribution of these PGA capacity values. Therefore, failure probability *independent* of the direction can be calculated, too. It is worth noting that it is more correct to analyze the conservatism of the code provisions through fragility curves and failure probabilities, which take into account the influence of variability and uncertainty on seismic performance. Code provisions can be considered as conservative if failure probability estimated according to them is higher or equal to the "true" failure probability. As shown in Figure 7, if a time window of 50 years is considered, code provisions (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2) appear to be conservative just in a few cases, while "Circolare Esplicativa" provisions (C 8.3) are never conservative.

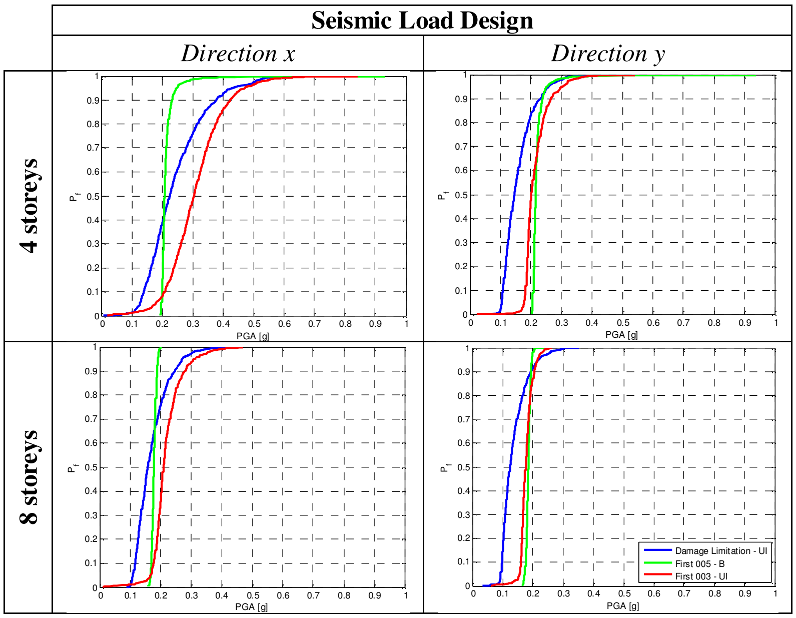


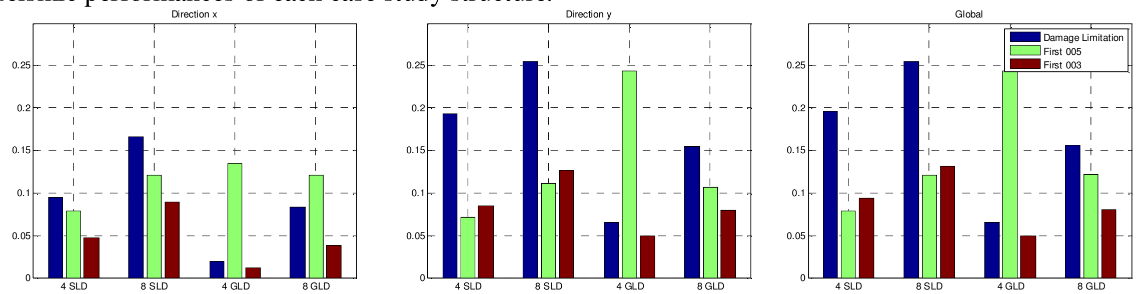
Figure 6. Fragility curves – SLD case study structures

Table 3. Estimated parameters of the cumulative lognormal distributions fitting the fragility curves

		Seismic Load Design				**Gravity Load Design**			
		4 storeys		*8 storeys*		*4 storeys*		*8 storeys*	
		$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}	$\overline{PGA}$	β_{PGA}
"Damage Limitation" - UI	*Direction x*	0.226	0.40	0.164	0.29	0.394	0.32	0.234	0.39
	Direction y	0.152	0.29	0.131	0.28	0.254	0.34	0.173	0.33
"First 005" - B	*Direction x*	0.213	0.11	0.178	0.04	0.171	0.10	0.179	0.07
	Direction y	0.221	0.10	0.185	0.04	0.129	0.08	0.188	0.04
"First 003" - UI	*Direction x*	0.295	0.35	0.213	0.28	0.432	0.27	0.289	0.22
	Direction y	0.212	0.19	0.177	0.12	0.286	0.36	0.219	0.19

4. CONCLUSIONS

In this paper, the same case study structures of the companion paper were analyzed. Fragility curves were obtained for each case study structure, through the application of a Response Surface Method. In order to apply this procedure, the considered input variables were represented by the same Random Variables already used for the sensitivity analysis in the companion paper in addition to the strength reduction factor R, whose variability was taken into account by assuming the logarithmic standard deviation β_R as a function of the ductility μ. Moreover, fragility curves were fitted by cumulative lognormal distributions. Analysis of seismic vulnerability and its dependence on the Random Variables' variability was performed in detail for each case study structure. Then, failure probabilities in the reference time period of 50 years were evaluated in order to underline the difference between seismic performances of each case study structure.



(a) (b) (c)

Figure 7. Failure probability for three DL Limit States – 50 years – direction x (a), direction y (b), global (c)

When assessing seismic capacity at DL, Italian technical code allows to take into account the presence of infills - if a bare numerical model, as usual, is used - by assuming a fictitious displacement capacity limit, e.g., 5‰ IDR if infills are attached to the structure and they significantly influence the structural deformability (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2). Moreover the Italian "Circolare esplicativa" allows to limit the maximum IDR in a RC structure to values related to masonry (e.g., 3‰ unreinforced masonry) if infills are explicitly taken into account in the numerical model (Circolare del Ministero dei Lavori Pubblici n. 617 del 2/2/2009 Section C 8.3). It has been observed that the displacement capacity assumed by the Italian code for bare numerical models is fictitiously higher than displacement capacity of the "real" (infilled) structure and this overestimation can be counterbalanced by a reserve of strength due to infills - neglected in the bare model –provided if infill strength is high enough: PGA capacity at "Damage Limitation" in Uniformly Infilled models is higher than PGA capacity at "First 005" in the Bare models only if infill panels produce a considerable increase in strength – in terms of percentage contribution to the base shear respect to the base shear strength provided by RC members. In these cases, code provisions (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2) can be defined as conservative. Moreover it is worth noting that it is more correct to analyze the conservatism of the code provisions through the analysis of fragility curves and failure probabilities, which take into account the variability influencing seismic capacity: code provisions (Decreto Ministeriale del 14/1/2008 Section 7.3.7.2) appear to be conservative only in a few cases, while "Circolare Esplicativa" provision (C 8.3) is never conservative.

REFERENCES

Circolare del Ministero dei Lavori Pubblici n. 617 del 2/2/2009. Istruzioni per l'applicazione delle "Nuove norme tecniche per le costruzioni" di cui al D.M. 14 gennaio 2008. G.U. n. 47 del 26/2/2009. (in Italian).

Decreto Ministeriale del 14/1/2008. Approvazione delle nuove norme tecniche per le costruzioni. G.U. n. 29 del 4/2/2008. (in Italian).

INGV-DPC S1 (2007). Progetto S1. Proseguimento della assistenza al DPC per il completamento e la gestione della mappa di pericolosità sismica prevista dall'Ordinanza PCM 3274 e progettazione di ulteriori sviluppi. Istituto Nazionale di Geofisica e Vulcanologia – Dipartimento della Protezione Civile, http://esse1.mi.ingv.it (in Italian)

Liel, A.B., Haselton, C.B., Deierlein, G.G., Baker, J.W. (2009). Incorporating modeling uncertainties in the assessment of seismic collapse risk of buildings. *Structural Safety*, **31(2)**, 197-211.

McKay, M.D., Conover, W.J., Beckman, R.J. (1979). A comparison of three methods for selecting values of input variables in the analysis of output from a computer code. *Technometrics*, **21(2)**, 239-245.

Pinto, P.E., Giannini, R., Franchin, P. (2004). Seismic reliability analysis of structures. IUSS Press, Pavia, Italy.

Ricci, P., De Risi, M.T., Verderame, G.M., Manfredi, G. (2012). Influence of infill presence and design typology on seismic performance of RC buildings: sensitivity analysis. *Proceedings of the 15th World Conference on Earthquake Engineering*, Lisbon, Portugal, September 24-28.

Vamvatsikos, D., Cornell, C.A. (2006). Direct estimation of the seismic demand and capacity of oscillators with multi-linear static pushovers through IDA. *Earthquake Engineering and Structural Dynamics*, **35(9)**, 1097-1117.

Vorechovsky, M., Novak, D. (2009). Correlation control in small-sample Monte Carlo type simulations I: A simulated annealing approach. *Probabilistic Engineering Mechanics*, **24(3)**, 452-462.