Nonlinear static and dynamic assessment of the seismic performance of RC buildings considering the out-of-plane collapse of URM infills

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ABSTRACT
In recent times, a significant interest in modelling the out-of-plane response of unreinforced masonry infills is growing. Such a response cannot be adequately assessed if the influence on it of the in-plane damage is neglected. In fact, past and recent experimental studies showed that the damage induced by the in-plane action can significantly affect the strength, stiffness and displacement capacity of infills.
In this study, a recently-proposed modelling strategy is applied to account for the out-of-plane response and collapse of infills also considering the in-plane/out-of-plane interaction. Non-linear incremental dynamic analyses are performed on infilled reinforced concrete structures designed to Eurocodes to show how the out-of-plane collapse of infills and the in-plane/out-of-plane interaction effects can modify the seismic response of the case-study buildings. In addition, the seismic base acceleration and the in-plane displacement demand at which the first out-of-plane collapse of infills occurs are discussed and evaluated. The same evaluation is performed, in a simplified form, via non-linear static analyses. The results of both approaches are compared and discussed.

1 INTRODUCTION

In recent years, a growing interest is arising on the experimental, theoretical and numerical assessment of unreinforced masonry (URM) infills' out-of-plane (OOP) response under seismic actions. The main issues investigated are i) the pure OOP response of URM infills and ii) the effects on the OOP response of the damage due to in-plane (IP) seismic action (IP/OOP interaction). For which concerns the first topic, experimental and theoretical studies were presented, especially in past years (McDowell et al. 1956, Dawe and Seah 1989, Bashandy et al. 1995, Flanagan and Bennett 1999a). For what concerns the second topic, experimental, analytical and numerical studies were presented, especially in recent years (Angel et al. 1994, Calvi and Bolognini 2001, Kadysiewski and Mosalam 2009, Mosalam and Günay 2015, Furtado et al. 2016, Di Domenico et al. 2018, Ricci et al. 2018a-b-c, Di Domenico et al. 2019a-b,Verderame et al. 2019).

It is well-known that the OOP collapse of URM infills can occur also with a very ruinous and abrupt overturning phenomenon that, above all, can harm human life safety. Hence, it seems appropriate considering the attainment of the OOP collapse of URM infills the same as the attainment of Life Safety Limit State (LS). That being said, it is worth to remember that (Eurocode 8 2004) requires the verification of nonstructural components against the seismic action within the construction assessment at LS (section 2.2.2(6)P).

Presently, Eurocode 8 proposes a simplified floor spectrum for the calculation of the seismic acceleration/force demand acting on acceleration-sensitive nonstructural components in section 4.3.5. A specific formulation for the calculation of the OOP strength of URM infill walls is not provided. However, the formulation proposed in (Eurocode 6 2005) for the lateral resistance of one-way spanning masonry walls accounting for one-way arching action may be used for this aim. Therefore, it is clear that current codes propose a force-based safety check of URM infills with respect to OOP seismic actions. Within this approach, the assumption of a behaviour/response modification factor is allowed to reduce the elastic seismic force demand calculated by means of the proposed floor response spectra. More specifically, the behaviour factor proposed by Eurocode 8 is equal to 2.

It is also worth to mention that IP damage affects the OOP response of URM infills by reducing the OOP strength, stiffness and ductility capacity. In addition, the OOP damage reduces the IP strength and stiffness (Angel et al. 1994, Calvi and Bolognini 2001, Guidi et al. 2013, Hak et al. 2014, Furtado et al. 2016, Ricci et al. 2018a-b-c). This phenomenon is known as "IP/OOP interaction". European provisions do not account for this highly detrimental phenomenon

In this study, 16 Reinforced Concrete (RC) buildings designed to Eurocodes are infilled by two infill layouts different for geometric and mechanical properties. As above stated, the OOP collapse of infills can be associated, according to Eurocode 8, with the attainment of LS. So, the seismic demand corresponding to the first OOP collapse of an infill in a building can be assumed as the seismic capacity of the same building on the non-structural side. The aim of this work is calculating and comparing the capacity Peak Ground Acceleration (PGA) of the case-study buildings with respect to the OOP collapse of infill walls by applying three different approaches.

The first one (Level 1 approach) is based on linear analysis and consists on the application of current Eurocode provisions for the evaluation of the OOP capacity and demand.

The second one (Level 2 approach) is based on nonlinear static analysis and allows accounting – at least partially – for the IP/OOP interaction effect.

The third one (Level 3 approach) is based on nonlinear dynamic analysis. This approach is, of course, the most complete and refined one. A recently-proposed infill wall macro-model is applied to model URM infills by accounting for their OOP response and for the IP/OOP interaction effects.

For each approach, mean values of the PGA demand at which the first OOP collapse of infills occurs and the parameters having a potential influence on it, namely the number of storeys and the design PGA of the construction, are presented and discussed.

2 DESCRIPTION AND MODELLING OF THE CASE-STUDY BUILDINGS

16 Reinforced Concrete (RC) moment-resisting frames different for the number of storeys (equal to 2, 4, 6 or 8) and for design PGA at LS (equal to 0.05, 0.15, 0.25 or 0.35 g) are considered. The case-study buildings are provided of 5 and 3 bays in the X and Z direction, respectively. All bays spans are 4.5 m long while the inter-storey height is always equal to 3 m. Each building has been designed for gravity and seismic loads by applying the Response Spectrum Analysis method according to (Eurocode 2 2004) and Eurocode 8. The materials used for the building design are class C28/35 concrete and reinforcing steel with characteristic yielding stress equal to 450 N/mm^2. The buildings were designed on a stiff and horizontal type A soil. Eurocode 8 Type 1 elastic spectrum, which is recommended for high-seismicity zones, was used to evaluate horizontal seismic actions. A behaviour factor equal to 4.68 was applied in the design process. All buildings resulted regular in plan while not regular elevation due to a non-gradual stiffness reduction along their height. P-δ effects resulted negligible. A lateral deformability verification at Damage Limitation Limit State (DL) was performed under a seismic action defined by applying a scaling factor equal to 0.4 times the response spectrum at LS (Hak et al. 2012). The longitudinal and transverse reinforcements for beams and columns were determined according to force demands assessed through RSA and by applying capacity design rules. A sketch of the case-study buildings is reported in Figure 1.

Two infill layouts are considered. The first is constituted by a two-leaf (thickness: 80+120 mm) URM 'weak' infill wall (weak layout, WL), , the second is constituted by a one-leaf (thickness: 300 mm) URM 'strong' infill wall (strong layout, SL). The mechanical properties of these infills are those calculated for the masonry wallets tested by (Calvi and Bolognini 2001) for the WL and those by (Guidi et al. 2013) for the SL. Note that the value of masonry shear strength of Guidi et al.'s specimens is not provided, so it is set to 0.30 N/mm^2 according to Table 3.4 of Eurocode 6.

Dependently on the approach adopted, the RC elements' non-linearity was differently modelled in OpenSees (McKenna et al. 2000). Within Level 1 approach, no explicit modelling of RC elements is necessary. Within Level 2 approach, the RC elements non-linearity is modelled by using a trilinear moment-chord rotation backbone provided with the cracking point and perfectly plastic after yielding point. These points are determined using a section analysis and by applying the dispositions about yielding chord rotation given by the Annex A of (Eurocode 8, part 3 2005). Within Level 3 approach, the RC elements non-linearity is modelled by using ModIMKPeakOriented Material with response parameters determined according to (Haselton et al. 2008) and with the introduction of the cracking point.

For what concerns infill walls' IP response, within Level 1 approach, no explicit modelling is necessary. Within Level 2 and 3 approaches, each infill wall is introduced in the structural model by using a couple of equivalent struts whose non-linear behaviour is modelled based on (Panagiotakos and Fardis 1996) proposal. According to this modelling approach, the slope of the softening branch of the force-displacement IP behaviour relationship is a fraction α of the infill initial elastic stiffness, while the infill residual strength is herein set to zero. In (Fardis 1996) it is suggested to set α to a value between -1.5% and -5%. For the 80-, 120- and 200-mm thick leaves α is set to -1.6% while for the 300-mm thick leaf it is set to -3.6%. These values yield to predictions of the softening stiffness and ultimate IP displacement in good accordance with the experimental evidences shown by (Calvi and Bolognini 2001) (specimen 2) for α=-1.6% and by (Guidi et al. 2013) (specimen URM-U) for α=-3.6%.

For what concerns infill walls' OOP response, within Level 1 and 2 approaches, no explicit modelling is necessary. Within Level 3 approaches, the OOP behaviour of IP-undamaged infills is modelled by using the lumped-plasticity empirical-based modelling strategy proposed by (Ricci et al. 2018a) in the updated version described in (Di Domenico 2018). This modelling strategy consists in defining for IP-undamaged infills a trilinear elastic-cracked-plastic OOP backbone. In addition, the IP damage effects on infills' OOP behaviour and vice-versa modelled by means of supplementary IP and OOP backbones that mutually-neutralize themselves or activate based on the IP and OOP displacement demands calculated step-by-step during the nonlinear time-history analysis. For the definition of the IP and OOP degraded backbones relationships are proposed to calculate the coordinates of the characteristic points of the IP-undamaged OOP backbone. A peculiarity of this modelling strategy is the flexibility with respect to the definition of the IP and OOP damaged and undamaged backbones, for which whichever material model, hysteretic rule and degradation rule can be used. Based on the expected first-mode deformed shape, the mass participating to the first OOP vibration mode of each leaf was set to 66% of the infill total mass.

3 DESCRIPTION OF THE ASSESSMENT PROCEDURES

Three different approaches are applied to calculate the PGA demand at which the first OOP collapse is expected to occur for each case-study building. A summary of the characteristics of each approach is reported in Table 1. The approach is more refined and reliable when passing from "Level 1", which is completely Eurocode-based (and, so, e.g., not-accounting for the IP/OOP interaction effects) to "Level 3", which is based on literature proposals for modelling the seismic response of RC elements and infills, both in the IP and in the OOP direction, and the IP/OOP interaction in a nonlinear dynamic framework.

A more detailed description is reported in the following sub-sections.

3.1 Level 1 approach

For the application of Level 1 approach, the OOP strength of infills, F_{Rd}, is calculated by applying Eurocode 6 formulation for masonry walls under uniformly-distributed lateral load reported in section 6.3.2, herein extended to infill walls (Equation 1).

$$F_{Rd} = f_d \left(\frac{t}{l_a}\right)^2 wh \tag{1}$$

In Equation 1, t is the infill thickness, w is the infill width, h is the infill height, f_d is the design compressive strength of masonry in the vertical direction and la is the height of the infill calculated as distance between the confining beams' centrelines.

For each case-study bare frame, the OOP force demand, F_{Ed}, acting on the infills at each storey is assessed by applying Equation 2, which is proposed in section 4.3.5 of Eurocode 8.

Table 1. Comparison of different approaches for the evaluation of the PGA corresponding to the first OOP collapse.

	Level 1	Level 2	Level 3
Analysis type	Linear	Nonlinear static	Nonlinear dynamic
Definition of a structural model	No	Yes	Yes
RC elements' response	Not modelled	Elastic-cracked-plastic	Elastic-cracked-hardening-softening
Infill walls' IP response	Not modelled	Modelled	Modelled
Infill walls' OOP response	Not modelled	Not modelled	Modelled
OOP response modified by IP damage	No	Yes (strength only)	Yes
IP response modified by OOP damage	No	No	Yes

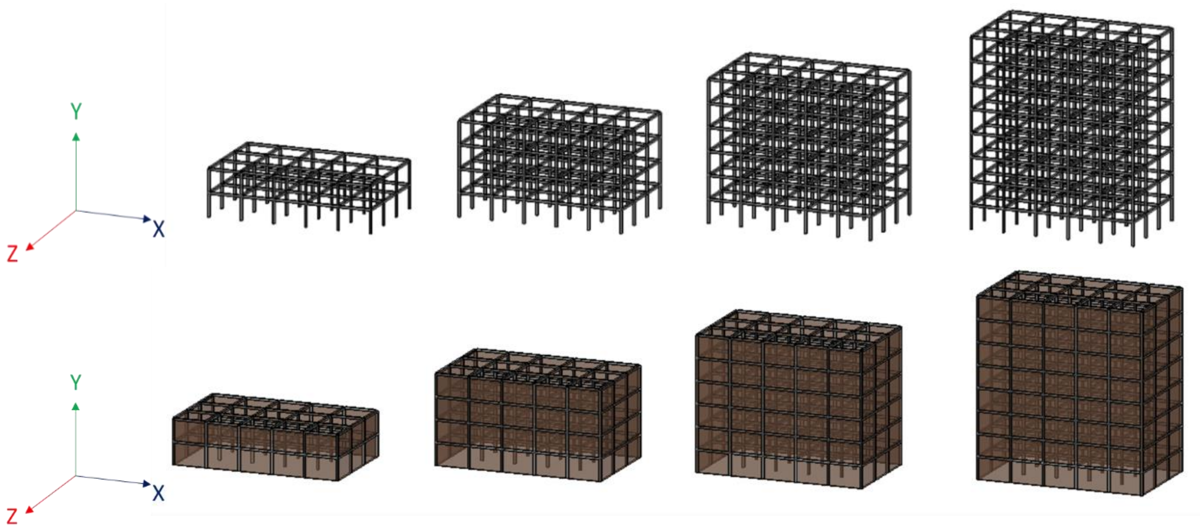


Figure 1. Sketch of bare and infilled case-study buildings.

$$F_{Ed} = \frac{S_a W_a \gamma_a}{q_a} \quad (2)$$

In Equation 2, W_a is the weight of the infill participating to its first out-of-plane vibration mode, γ_a is the importance factor of the infill, assumed equal to 1 according to section 4.3.5.3 of Eurocode 8, q_a is the behaviour factor of the infill, assumed equal to 2, as suggested for exterior walls in section 4.3.5.4 of Eurocode 8. S_a is the seismic coefficient, which is equal to the PSA acting on the infill in the OOP direction divided by gravity acceleration, g, and is calculated as shown in Equation 3.

$$S_a = \alpha S \left[\frac{3(1+z/H)}{1+(1-T_a/T_1)^2} - 0.5\right] \quad (3)$$

In Equation 3, α is the design acceleration on type A soil, ag, divided by the acceleration of gravity g, S is the soil factor, z is the height of the infill barycentre above the building base, H is the total height of the building, T_1 is the fundamental vibration period of the building in the relevant direction, i.e., in our case, the design fundamental vibration period of the building in the OOP direction, calculated for the bare frame model with halved-inertia for the structural elements' section. T_a is the infill vibration period in the OOP direction. T_a is calculated by using the classical formulation for a single-degree of freedom system, with mass equal to the infill mass participating to the first OOP vibration mode (assumed as the 66% of the infill total mass), and stiffness calculated as for an elastic plate pinned along all edges according to the formulation by (Timoshenko and Woinowsky-Krieger 1959). With some manipulation, Equations 2 and 3 can be written as Equations 4 and 5, respectively.

$$F_{Ed} = \frac{PSA}{g} \frac{W_a \gamma_a}{q_a} \quad (4)$$

$$PSA = PGA \left[\frac{3(1+z/H)}{1+(1-T_a/T_1)^2} - 0.5\right] \quad (5)$$

Level 1 approach consists in:

1. Calculating, for each infill layout, the OOP strength by applying Equation 1.
2. Calculating, for each case study building and in each horizontal direction, the maximum demand acting on infills – which

always occurs at the last floor – by using Equation 4 and matching it to the capacity calculated using Equation 1 in order to define the PSA_c.
3. Calculating PGA_c from PSA_c using Equation 5.

3.2 Level 2 approach

For the application of Level 2 approach, literature formulations for OOP capacity and demand are applied.

In this study, for the prediction of the IP-undamaged infill OOP strength under seismic load, the mechanical model by (Dawe and Seah 1989) is applied for thin infills (WL). In fact, based on experimental data, (Di Domenico et al. 2019b) showed that Dawe and Seah's mechanical model, which allows the calculation of the entire OOP force-displacement response of the infill, is the most effective in predicting the OOP strength of thin URM infills.

Experimental values of the OOP strength of IP-undamaged URM thick and robust infills (SL) are not provided in the literature. For this reason, it seems conservative to propose for this type of infills the application of Eurocode 6 formulation. The effectiveness of this formulation was not assessed on experimental data, as above explained, but it is certainly conservative, as it neglects the contribution to strength of horizontal arching action and is derived based on an application of the lower bound theorem of limit analysis (Di Domenico et al. 2019b). It should be noted, in addition, that Eurocode 6 formulation is dedicated to infills under uniformly-distributed load. Within the application of Level 2 approach, this formulation is adapted to the seismic load shape as reported in (Di Domenico et al. 2019b), i.e., by multiplying Equation 1 times a coefficient equal to 0.85. As above stated, the IP damage reduces the OOP strength of infills. R, the OOP strength degradation factor due to the IP damage, is calculated by applying the empirical relationship derived in (Ricci et al. 2018c) and reported in Equation 6. This formulation is based on experimental tests' results for URM infills in RC frames. The IP damage is represented by the maximum IP IDR demand, expressed in percentage, at given vertical slenderness (i.e., height-to-thickness) ratio, h/t.

$$R(IDR) = \frac{F_{Rd}(IDR|h/t)}{F_{Rd}(IDR=0)} = \min(1; [1.21 - 0.05\min(20.4; h/t)](IDR)^{-0.89}) \qquad (6)$$

The seismic demand on infills is obtained by multiplying the demand PSA times the infill mass participating to the first OOP vibration mode, equal, also in this case, to the 66% of the infill total mass. Equation 3 proposed by Eurocode 8 does not account for the effects of the non-linear behaviour of the primary structure on floor acceleration demands while at LS the RC structure is supposed to have already experienced a significant non-linearity. Also for this reason, Equation 3 may overestimate floor accelerations (Pinkawa et al. 2014, Petrone et al. 2015). For this reason, within Level 2 approach, the OOP acceleration demand will be calculated by using the floor spectra proposed for inelastic Multi-Degree of Freedom (MDOF) systems by (Vukobratović and Fajfar 2017). Vukobratović and Fajfar formulation of the PSA demand differs from Eurocode 8 proposal mainly for two aspects. First, it accounts for the effects of higher vibration modes, which are neglected in Eurocode 8 formulation: for this reason, for a given PGA the acceleration demand may be not monotonically increasing along the building height. Second, it accounts for the inelastic structural behaviour due to the excitation of the first vibration mode using the PSA reduction factor R_μ, which in this work is obtained from the SPO2IDA tool (Vamvatsikos and Cornell 2006). For this reason, for a given floor, the acceleration demand grows up with decreasing rate as PGA increases. The ductility of the non-structural element is considered by assigning to it an equivalent damping ratio, i.e., directly when calculating the PSA demand. For this reason, the force acting on infills is calculated without the application a posteriori of a behaviour factor. According to (Vukobratović and Fajfar 2017), such equivalent damping ratio is fixed to 10%.

For simplicity, the procedure is described with reference to a 4-storey building.

1. For each infill layout, the OOP strength, F_{Rd}, is calculated by applying Dawe and Seah's model (for WL) or Eurocode 6 formulation (for SL).
2. For each case-study infilled building, a static pushover (SPO) analysis is performed in the IP direction to obtain a base shear (V_b) vs roof displacement (Δ_{TOP}) curve. The loading path used to carry out SPO analyses is proportional to the force distribution along the frame height associated with the first vibration mode in the considered IP direction.
3. The SPO V_b-Δ_{TOP} curve is then multi-linearized according to the piecewise procedure described for elastic-hardening-

negative systems in (De Luca et al. 2013). Note that the application of the above procedure results in an effective fundamental period assigned to each case-study building equal to its elastic fundamental vibration period.

4. For each case-study building, the 50th percentile IDA curve is associated with each SPO curve by applying the SPO2IDA tool. This allows defining an elastic PSA vs Δ curve. The introduction of each elastic PSA in the Eurocode 8 Type I spectrum allows passing from elastic PSA to elastic PGA vs Δ curve. Using the SPO analysis results, with each Δ it is possible to associate the IDR for each storey and to define PGA vs IDR curves for each storey (Figure 2).
5. With each IDR demand, for each storey, it is possible to associate the degraded strength of the infills at that storey, by means of Equation 6, and trace a PSA_c vs PGA_{IP} curve (Figure 2).
6. It is assumed that the PGA acting in the OOP direction is equal to the PGA acting in the IP direction. For each PGA_{OOP} value, the PSA demand, PSA_d, in the OOP direction is calculated by means of Vukobratović and Fajfar floor spectrum and demand PSA vs PGA_{OOP} curves for each storey can be defined (Figure 3a).
7. The lower PGA at which the PSA_c vs PGA_{IP} and the demand PSA vs PGA_{OOP} curves intersect is the PGA_c accounting for the IP/OOP interaction associated with the considered building (Figure 3b). For each case-study infilled building and for each infill layout the effective PGA_c is the minimum between the one calculated assuming X and Z as the IP direction, clearly. The IDR distribution associated with PGA_c is the collapse IDR distribution assessed by accounting for IP/OOP interaction.

The effects of OOP actions on the IP response of infills that were experimentally observed by some authors (Flanagan and Bennett 1999b) are neglected. This approach, given the overestimation of the infilled structure stiffness, leads to a non-conservative underestimation of the infills IP displacement and, so, of their OOP capacity reduction due to interaction. Moreover, the infill OOP stiffness reduction due to IP actions is neglected together with the consequent T_a elongation.

3.3 Level 3 approach

Within Level 3 approach, the PGA capacity of URM infills with respect to the OOP failure is determined by means of nonlinear time-history incremental dynamic analysis.

Ten ground motions were selected among the records of seven different European earthquakes collected in the Engineering Strong-Motion (ESM) Database (Luzi et al. 2016) in order to perform Incremental Dynamic Analyses (IDAs) (Vamvatsikos and Cornell 2002). Significant characteristics of the selected ground motions are reported in Table 3.

The selection of records was performed searching among the bidirectional registration of stations based on Eurocode 8 type A soils, consistently with the design soil type. Consistently with the choice of using Eurocode 8 Type I design spectrum, only earthquakes with magnitude between 5.5 and 7 and only registration of stations with epicentral distance between 10 and 30 km were considered. Both horizontal components of the selected records were simultaneously matched to the 5%-damped Eurocode 8 design spectrum at Life Safety Limit State by using wavelets through the RspMatchBi software (Grant 2010).

IDAs were performed by scaling each selected and matched record for a set of pre-determined scale factors ranging from 0.067 to 10. This allowed performing the IDAs for 32 values of PGA roughly equal in both directions and ranging from 0.010 g to 1.50 g.

The analyses were carried out by applying mass- and tangent stiffness-proportional Rayleigh damping rules for two control vibration modes. A “global” and a “local” mode were selected as control modes. For instance, the first control mode corresponds the first natural frequency of the infilled structure, while the second control mode corresponds to the mode associated to the frequency closer to the infill natural frequency in the OOP direction. The assigned damping ratio is equal to 5% both for the first global and for the second local control mode: the last choice is due to the lack of exhaustive studies on this topic, which is worth to be investigated in the future.

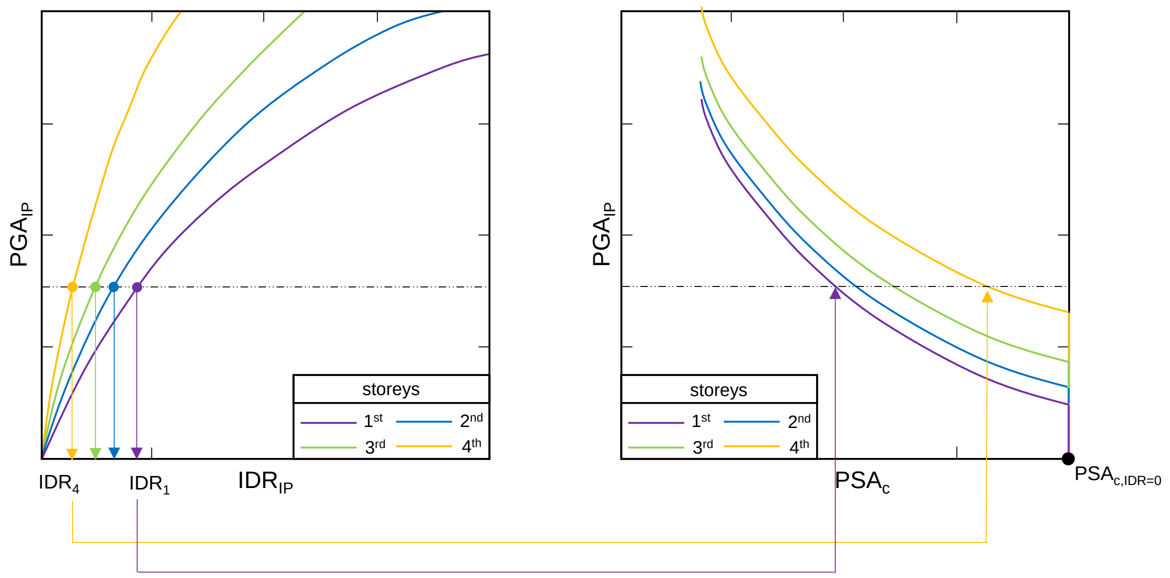
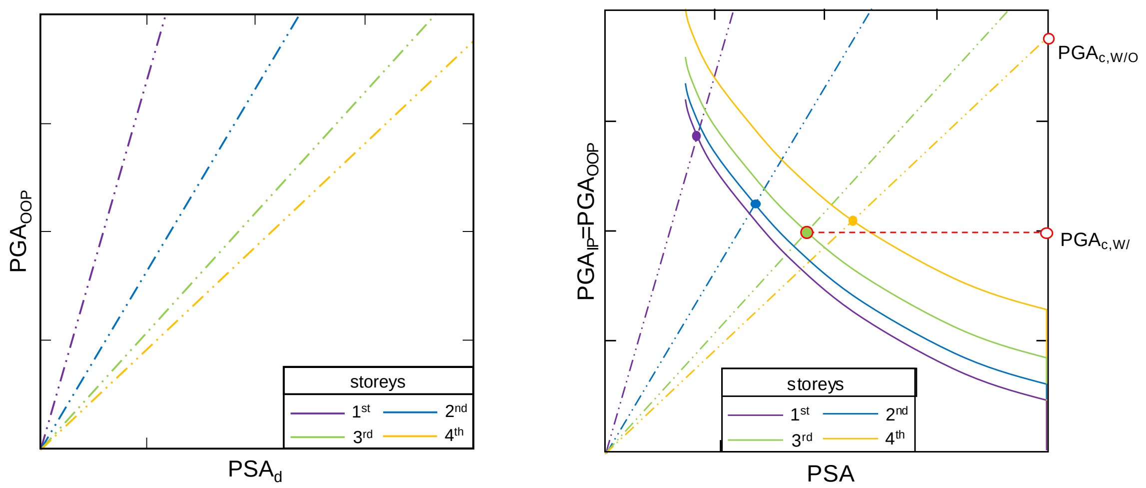


Figure 2. Level 2 approach schematic representation: definition of IP displacement demand as a function of the IP PGA and definition of the degraded OOP strength of infills corresponding to that IP PGA.



(a) (b)

Figure 3. Level 2 approach schematic representation: definition of OOP demand (a) and matching of OOP capacity and demand (b).

4 RESULTS AND DISCUSSION

The values of the capacity PGA, PGA_c, with respect to the first OOP collapse are reported for each case-study building and for each assessment approach in Table 2. The value reported for Level 3 approach is an average value of the ten PGA_c values determined for each record adopted for nonlinear time-history analyses.

Generally, it is expected that simplified and/or code-based approaches should provide conservative results. Actually, it is observed that when passing from a simplified and code-based approach (Level 1) to a refined and literature-based approach (Level 3) the values of PGA_c for all case-study buildings reduce. This occurs for two main reasons:

1. The significant reduction of PGAc observed when passing from Level 1 to Level 2 approach is manly due to the IP/OOP interaction effects, which are neglected within the current Eurocode framework;
2. The reduction of PGA_c observed when passing from Level 2 to Level 3 approach is mainly due to a sort of "underestimation" of the IP/OOP interaction effects. In fact, within Level 2 approach, the effect of the OOP damage on the IP strength and stiffness of infills is neglected. This yields to an overestimation of the lateral stiffness of buildings and, so, to an underestimation of the IP displacement demand and, hence, to an underestimation of the OOP strength reduction due to the IP/OOP interaction effects.

As also shown in Figure 4, the PGAc reduces at increasing number of storeys of buildings and increases at increasing design PGA of the building. Many reasons may be related to these trends, also concerning the parameters affecting the OOP seismic demand acting on infills and, more in general, floor response spectra, as explained in (Di Domenico 2018) and (Ricci et al. 2019). However, it is worth observing that a higher lateral deformability is expected for taller buildings as well as for buildings designed for lower values of PGA. Hence, in these cases, also a higher proneness to the IP/OOP interaction effect is expected for infills and, hence, a lower value of PGA_c with respect to the OOP collapse.

For WL infills, the capacity PGA obtained for mid- and high-rise buildings in mid- and high-seismicity zones may be lower than the design PGA at LS. This means that these buildings are safe on the structural side but not safe on the non-structural side. In fact, Eurocode 8 prescribes the safety check at LS of non-structural elements in section 2.2.2(6)P.

For SL infills, the capacity PGA obtained within the application of Level 1 approach is clearly without any physical meaning. In general, such a result only means that, according to a totally Eurocode-based approach, the OOP collapse of such a type of infills is practically impossible. Actually, the PGA capacity obtained by applying Level 2 and 3 approaches is significantly lower than that obtained by using Level 1 approach but, at the same time, significantly higher than whichever value of PGA expected even for very strong earthquakes. In other words, the OOP collapse of SL infills seems to be actually improbable, independently on the assessment procedure adopted.

It is observed that the results of Level 2 and 3 approaches are in quite good accordance for WL infills, while a greater gap between them is observed for SL infills. This may be due to potential non-negligible difference between the floor acceleration demand predicted by Vukobratović and Fajfar spectrum (used within the application of Level 2 approach) and that actually observed through nonlinear dynamic analyses. Such differences may be relatively small in low PGA ranges (as those associated with the WL capacity) and relatively large in high PGA ranges (as those associated with the SL capacity). In the last case, in fact, the structure is expected to experience large inelastic demands, whose influence on the modal contributions to the floor acceleration demand may be not completely caught by the closed-form floor spectrum adopted within the application of Level 2 approach.

Within Level 2 and Level 3 approaches, it is also possible to assess the average IP IDR demand at which the first OOP collapse occurs. With reference to WL infills, such an average value, for all case study buildings, is equal to 0.26% according to Level 3 approach and to 0.44% according to Level 2 approach. Also in this case, Level 2 approach is not conservative, most likely due to the same "underestimation" of the IP/OOP interaction effects which yields to an unconservative evaluation of the capacity PGA.

Table 2. Values of the PGA corresponding to the first OOP collapse according to different assessment approaches.

	WL			SL		
	Level 1	Level 2	Level 3	Level 1	Level 2	Level 3
2P05	1.26 g	0.333 g	0.243 g	30.8 g	1.36 g	0.705 g
2P15	1.26 g	0.334 g	0.243 g	30.7 g	1.36 g	0.758 g
2P25	1.26 g	0.334 g	0.243 g	30.7 g	1.37 g	0.730 g
2P35	1.19 g	0.397 g	0.253 g	30.5 g	1.41 g	0.730 g
4P05	1.31 g	0.324 g	0.210 g	30.9 g	1.23 g	0.705 g
4P15	1.25 g	0.320 g	0.235 g	30.9 g	1.28 g	0.715 g
4P25	1.21 g	0.319 g	0.235 g	30.8 g	1.27 g	0.785 g
4P35	1.15 g	0.300 g	0.250 g	30.6 g	1.29 g	0.715 g
6P05	1.31 g	0.282 g	0.165 g	31.6 g	1.25 g	0.560 g
6P15	1.30 g	0.260 g	0.198 g	31.3 g	1.25 g	0.600 g
6P25	1.28 g	0.278 g	0.205 g	31.1 g	1.28 g	0.605 g
6P35	1.23 g	0.280 g	0.223 g	30.8 g	1.23 g	0.645 g
8P05	1.35 g	0.220 g	0.153 g	33.6 g	1.13 g	0.495 g
8P15	1.35 g	0.202 g	0.178 g	33.6 g	1.18 g	0.505 g
8P25	1.33 g	0.222 g	0.198 g	33.6 g	1.21 g	0.625 g
8P35	1.29 g	0.224 g	0.218 g	33.2 g	1.19 g	0.705 g

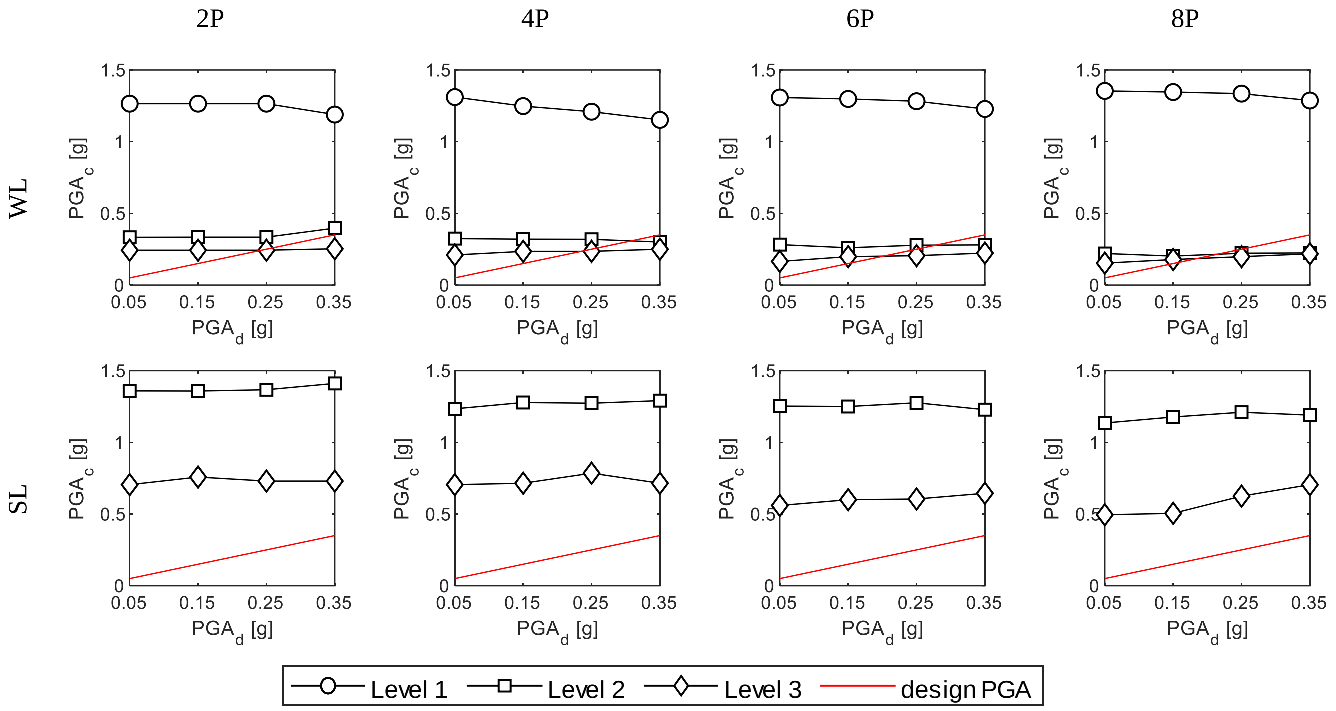


Figure 4. Values of the PGA corresponding to the first OOP collapse according to different assessment approaches.

5 CONCLUSIONS

In this paper, different approaches are applied for the assessment of the PGA demand at which the first OOP collapse of the URM infills of RC case-study buildings. The case-study buildings are different for number of storeys (ranging from 2 to 8) and for design PGA at LS (ranging from 0.05 g to 0.35 g). Two different infill layouts are considered: a two-leaf "weak" infill layout (WL) and a one-leaf "strong" infill layout (SL).

The first approach applied, named "Level 1 approach" is totally Eurocode-based. It consists in the application of Eurocode provisions for the evaluation and force-based comparison of the OOP capacity and of the OOP demand acting on infills. It does not account for the detrimental effects of the so-called IP/OOP interaction, i.e., the OOP strength, stiffness and ductility capacity reduction due to the IP damage and the IP strength and stiffness reduction due to the OOP damage. Despite being a totally code-conforming approach, for the above reasons, it is expected to provide unconservative results in terms of PGA capacity with respect to the first OOP collapse of infills.

The second approach applied, named "Level 2 approach" is based on a recently proposed procedure consisting in the evaluation of the IP displacement demand through nonlinear static analyses and on the evaluation of the OOP capacity and demand by means of refined literature formulations. Such an approach partially accounts for the IP/OOP interaction effects as far as the OOP strength reduction due to the IP damage is concerned.

The third approach applied, named "Level 3 approach" is based on the results of nonlinear time-history analyses. A refined modelling of both RC elements and infill walls is used. Namely, all the IP/OOP interaction effects are reproduced during the analyses.

First, the analyses results revealed that, independently on the assessment approach adopted, SL infills are not expected to collapse for OOP actions even for very strong and rare seismic events. Focusing on WL infills, it was observed that the current Eurocode approach may yield to a significant overestimation of the OOP seismic capacity of URM infills. Actually, based on the results of Level 2 and 3 approaches, which are in good accordance, the OOP collapse of infills is expected for buildings in mid- and high-seismicity zones. Such a result is not obtained by applying an Eurocode-based approach. This also means that RC buildings designed to Eurocodes in mid- and high- seismicity zones are safe on the structural side but not on the nonstructural side.

Based on the above results, improvements and suggestions for a simplified, code-oriented and conservative safety assessment of URM infills under OOP actions have been proposed in recent

(Di Domenico 2018, Ricci et al. 2019), namely concerning the use of simplified but robust formulations for the calculation of the OOP strength, the use of the IDR distribution calculated for bare frame models for the assessment of the OOP strength reduction due to the IP/OOP interaction and/or the use of a behaviour factor equal to 1 for a correct safety assessment of slender URM infills.

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