

Influence of Infill Masonry Walls in the Seismic Response of Buildings: From Field Observations to Laboratory Research



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Abstract The seismic performance of infill masonry walls is a topic of growing importance due to the significant number of collapses observed through the recent earthquakes. Nowadays is recognized by the scientific community the influence of these elements in the structural response of reinforced concrete structures subjected to seismic actions. The infills out-of-plane (OOP) behaviour depends on a series of variables and there is a lack of experimental data to understand and predict their expected seismic performance. There is a need of data to calibrate numerical models and to understand the effect of each variable such as type of masonry, boarder constrains, slenderness, previous in-plane damage and insufficient support width in the infills OOP capacity. The present chapter pretends to overview some considerations regarding the performance assessment of infills OOP performance such as based on experimental tests and numerical modelling results. Additionally, a brief literature and international codes recommendations review on this topic will be presented and discusses and will help to understand the importance of the infills seismic behaviour on the performance assessment of reinforced concrete structures.

Keywords Calibration · Experimental tests · Numerical modelling
Seismic behaviour

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1 Introduction

In recent years, increased interest is denoted in studying the infill masonry (IM) walls' influence in the seismic response of existing buildings, which can be favourable or not, depending on several phenomena, detailing aspects and mechanical properties, namely the relative stiffness and strength between frames and masonry walls, the type or lack of connection between masonry and surrounding structures, etc. (Hermanns et al. 2014).

From surveys on damaged and collapsed RC buildings in recent earthquakes many buildings having suffered severe damage or collapse exhibited poor performance due to IM panels. It is observed that in-plane (IP) behaviour of IM can prevent the development of out-of-plane (OOP) strength mechanisms by arching effect. By contrast, in most cases the major damages were found in non-structural elements, particularly in clay IM, including diagonal cracking, OOP collapse or detachment of surrounding RC frames (the latter taking place in early earthquake instants) due to absence of or deficient connection to that frames. These damage types often require high investments, either for the repair process or for demolition and reconstruction, resulting in economic problems related with interdiction of building use.

Different authors (Furtado et al. 2016a, b) reported that the OOP performance and capacity of IM walls can be strongly influenced by the following issues: connection between the panel and surrounding RC frames; connection between the internal and external leaves (in the case of two-leaf IM walls); insufficient support width due to constructive procedures adopted for thermal bridges' prevention and, last but not the least, the existence of previous in-plane damages. Moreover, IM walls OOP collapse can also introduce plan and/or height vertical stiffness irregularity which can induce formation of mechanisms such soft-storey or torsion, likely to originate building collapse. Considering the lack of experimental studies conducted during the last years regarding this behaviour and considering the common use of hollow clay bricks with horizontal perforation in Portugal, it is of utmost importance to validate some proposed retrofitting strategies in the literature and develop new ones to improve the OOP behaviour and prevent the collapse of these IM walls' types.

For assessing RC frame structures, the nonlinear behaviour induced by earthquakes demands and the influence of IM walls should be considered. Different modelling techniques can be found in the literature (Asteris et al. 2011a, b, 2013) for the response simulation of infilled frames, from refined micro-models to simplified macro-models, the former involving high discretization level of the IM panel and the later relying in simplifications aiming at representing the IM global behaviour with a few structural elements and mechanical parameters. In many cases, for non-linear analysis of complex structures under earthquake action, it is not suitable to adopt refined models. Thus, for simulating the response of frame structures with IM walls, taking into account the interaction between them, the adoption of simplified models is unavoidable (Smyrou et al. 2011; Rodrigues et al. 2010). This is

further confirmed in FEMA356 (2000) which recommends assessing the structural response of buildings, considering the infill panels represented by an equivalent diagonal strut model.

The main objective of the present chapter is to highlight the seismic behaviour of IM walls, particularly when subjected to OOP loadings. Firstly, a brief review of code provisions regarding the IM walls seismic performance will be presented, discussing the most important issues on this field. Then, the major results of an experimental campaign of quasi-static OOP tests on full-scale IM walls that was carried out on the Laboratory of Earthquake and Structural Engineering (LESE) will be presented and discussed. Finally, a simplified numerical approach to simulate the combined IP and OOP behaviour of IM walls subjected to earthquakes will be presented.

2 Code Provisions and Recommendations on IM Walls Seismic Performance

Some international codes recommend various formulations for the analysis of IM walls for both in-plane and out-of-plane directions. For instance, FEMA274 (1997) specifies that masonry infill panels shall be represented as equivalent diagonal struts and may be placed concentrically across the diagonals, or eccentrically to directly evaluate the infill effects on the columns. It specifies strength requirements for column members adjacent to infill panels. The shear force demand may be limited by the moment capacity of the column with reduced length. EC8 (2003) specifies that the period of the structure used to evaluate base-shear stress shall be the average between periods for the bare frame and for the elastic infilled frame. Frame member actions are then determined by modelling the frame without the struts. Irregular infill arrangement (in plan and elevation) is addressed with important recommendations to avoid the formation of soft-storeys and torsion-effects. Moreover, designing techniques are suggested to account for irregularities, such as increase of accidental eccentricities use of three-dimensional analysis. Regarding irregularities in elevation, if a refined model is not used, the code suggests the computation of a magnification factor to increase the seismic actions on columns (only).

Regarding the lateral load shared between infill walls and frame, EC8 (2003) does not make a reference to the infill walls, considering only that the frame system should resist totally the vertical loads, and to have a 65% base-shear capacity—50% as a minimum for other types of structure—of the total lateral loading on the building. For the serviceability limit state, it is recommended the control of lateral deformation between storeys (drifts, δ_r). For buildings with brittle non-structural elements, it should be limited to $0.005 h = \mu$. For buildings with ductile non-structural elements the drift is limited to $0.0075 h = \mu$, or $0.010 h = \mu$ if the building does not have non-structural elements (h is the height of the storey, and μ

is the reduction coefficient ranging from 0.4 to 0.5, depending on the importance class). Due to the nature of the infill masonry walls, non-structural and brittle elements, the limit to use should be $0.005 h = \mu$.

There are some international codes that provides some recommendations on the OOP capacity of infills, as well as the indications given about the design demand acting on them. For example the Italian Building Code (NTC08 2008) gives some indications regarding OOP seismic action on infill considered as non-structural elements, but no provision aimed at determining their OOP capacity; FEMA306 (1998) provides some recommendations on infills OOP strength, but no indication on their maximum displacement capacity.

Eurocode 6 (2005), in Sect. 6.3.2, proposes an expression (Eq. 1) to calculate the lateral strength of masonry walls in which arching action can occur; this relationship can be extended, eventually, to infill panels:

$$q = f_d \left(\frac{t}{l_a} \right)^2 \quad (1)$$

In this relationship f_d is the design compressive strength of masonry in the direction of arching thrust while l_a is the panel dimension in the same direction. The maximum OOP load is the one that equilibrates the maximum thrust that can form in the masonry wall thickness determined from:

$$N_{ad} = 1.5f_d \left(\frac{t}{10} \right) \quad (2)$$

FEMA 273 (1996) and FEMA 356 (2000) provide some indications concerning the ultimate OOP displacement of infills with reference to different Limit States. A 2% OOP drift is set as maximum displacement for Immediate Occupancy Limit State: this drift value corresponds with the opening of visible cracks on the panel surface; with reference to the Life Safety Limit State a 3% OOP drift is fixed as limit displacement: this drift value corresponds with high possibility of detachment and expulsion of at least part of the infill. FEMA356 (2010) sets a 5% OOP drift as maximum displacement at Collapse Prevention. These indications are effective for both new and existing buildings.

FEMA273 (1996) lists the conditions that allow considering arching action in the assessment of infills OOP strength, such as the effectiveness of the infill connection to the surrounding frame, its columns and beams stiffness and strength, and the panel slenderness. Among these statements, the one referring to the infills boundary conditions seems to be the most significant. In fact, the analysis of the experimental database presented in Sect. 4 shows that, even for panels with slenderness greater than the value proposed as upper limit for the arching action effectiveness, the best strength prediction was provided by relationships based on that resistant mechanism. Under the above-mentioned conditions, it is possible to express the lateral strength of the infill as Eq. 3:

$$Q = \frac{0.7 \times f_m \times \lambda_2}{(h/t)} \quad (3)$$

in which f_m is the lower bound of the compressive strength of masonry calculated by dividing by 1.6 (by 1.3 according to FEMA356 2000) its average compressive strength; λ_2 is a slenderness parameter. FEMA274 (1997) points out that the previous expression is the relationship by Angel simplified to evaluate a lower bound of the infills lateral strength. To compute it, FEMA306 (1998) provide Angel's relationship without modifications. This means that FEMA306 (1998) consider OOP strength reduction due to IP damage explicitly, even if it is not stated how to set the IP drift at which the OOP strength should be assessed.

3 Experimental Characterization of the Infills OOP Seismic Behaviour

3.1 Introduction

The experimental testing of infilled frames to OOP loadings started in 1988 with Moghaddam et al. (1988) with four steel infilled frames on a biaxial shake table test. Two small and two larger walls were tested with the aim of evaluate the effect of use reinforcement bars on the horizontal bed joints. The authors concluded that the infills' presence reduced the displacements of the frame even after the infills' cracking. It was observed that the reinforcement reduced the vulnerability of the panel by the improvement of the arching mechanism phenomena. Dawe and Seah (1989) tested 9 full-scale concrete infill walls subjected to uniform normal pressure applied through airbags. The authors concluded that the IM walls ultimate loads increased for larger panel thicknesses; however, for smaller panel length and height it was observed a strength reduction. No significant influence of the openings on the IM panels OOP strength. Finally, the authors remarked that the horizontal reinforcement bars provided higher OOP ductility. Angel et al. (1994) carried out an experimental campaign composed by combined IP and OOP tests of RC frames infilled with brick and concrete blocks. The strategy was to submit first the specimens to IP demands and to cause different levels of damage, and then the specimens were subjected to OOP monotonic distributed loadings applied by airbags. From these tests, the authors concluded that the panels' OOP strength depends highly of the slenderness ratio, masonry compressive strength but not from the tensile strength. The authors observed that cyclic loadings within the elastic region of the panel did not affect the stiffness of the panel. The IP shear demand combined with panel gravity load increase slightly the initial OOP stiffness but the OOP strength is not affected. Nevertheless, the previous IP cracking reduced the OOP strength for slender panels. Repairing techniques were tested and it was observed that the increase of the damaged panels OOP strength was achieved. Calvi and

Bolognini (2001) carried out OOP tests with and without previous IP damage, with and without reinforcement. Four OOP loading tests were carried out with the aim of assess the OOP vulnerability of traditional and slightly IM walls for different level of damages induced by IP action. It was observed that the panel' state of damage play an important role on the OOP response of the panel, and that higher levels of damage increase the OOP vulnerability of the panel.

From the analysis of the experimental efforts made throughout the different studies on the literature, the following main conclusions can be drawn:

- High panel slenderness can result on poor OOP performances, since it is observed that the arching mechanism developed after the maximum strength do not occur for panels with high slenderness;
- Previous IP damage reduce the OOP initial stiffness, strength and can lead to fragile OOP expulsions. This effect is due to the loss of the boarder constrains that were modified, since the detachment of the panel from the surrounding frame occurred and a rigid body behaviour occurs when subjected to OOP loadings;
- The masonry compression strength revealed to be more important to the formation of the arching mechanism than the tensile strength.

At the Laboratory of Earthquake and Structural Engineering (LESE) an experimental campaign composed by five full-scale IM walls was carried out with main purpose of analyze any effect of the panel support width, axial load on columns and previous IP damage (Furtado et al. 2016a, b). The test results will be presented and discussed in terms of damage observed throughout the tests and the cracking pattern.

3.2 Specimens' Detailing and Testing Campaign Description

The specimen dimensions are $0.15 \times 4.80 \times 3.30$ m respectively thickness, length and height, with columns sections 0.30×0.30 m and the top and bottom beams 0.30×0.50 m. For the RC frame specimen construction, three different bar diameters were used, from the same lot, namely $\phi 6$ mm, $\phi 10$ mm and $\phi 16$ mm. Five IM walls (M1, M2, M3, M4 and M5) were built with hollow clay horizontal brick, as frequently adopted in the Southern Europe and particularly in Portugal. The mortar adopted was an industrial pre-dosed M5 class ("Ciarga" type) with the following composition. No plaster was adopted in both panels.

The main characteristics of each specimen are summarized below:

- M1: One-leaf panel (thickness: 150 mm), aligned with the external face of the support beam, monotonic test with 300 kN in the top of each RC column, totally width supported;

- M2: One-leaf panel (thickness: 150 mm), aligned with the external face of the support beam, cyclic test with no axial load in the top of each RC column, totally width supported;
- M3: Double-leaf panel, composed by one external leaf with thickness of 150 mm and an internal leaf of 110 mm. Subjected to a previous in-plane test until reach 0.5%. After the in-plane test the internal panel was removed and the external one was subjected to OOP cyclic test with no axial load in the top of each RC column;
- M4: One-leaf panel (thickness: 150 mm), aligned with the external face of the support beam, cyclic test with axial load in the top of each RC column of 270 kN (constant throughout all the test), panel width totally supported;
- M5: One-leaf panel (thickness: 150 mm), aligned with the external face of the support beam, cyclic test with no axial load in the top of each RC column, 2/3 width supported on the bottom beam;

The contact between both specimens and the surrounding columns and the bottom beam is provided by approximately 1 cm layer of mortar. Regarding the contact between the top beam, half-brick and mortar are used to fill the gap that resulted from the IM wall construction.

3.3 Test Setup

The OOP test consisted on the application of a uniform distributed pressure, throughout the entire panel under tested, through nylon airbags. With this procedure, it is pretended to mobilize the entire infill panel considering all the distributed inertia forces that results from a seismic excitation. The uniform load applied through all the infill panel is reacted against a self-equilibrated steel structure composed by five vertical and four horizontal alignments that are rigidly connected to the RC frame with steel re-bars in twelve previous drilled holes (Fig. 1). Between the self-equilibrated steel structure and the RC frame it was inserted twelve load cells that allow the monitoring of the forces transmitted along the experimental test. In front of the self-equilibrated steel structure it was placed a wooden platform to resist the airbags pressure and transfer it to the structure and to the tested panel. The self-equilibrated system uses the RC frame bending stiffness and strength to react to the OOP forces developed from the application of the pressure on the panel. This OOP test setup can be adaptable to specimens with different geometries, different types of masonry materials and existence of openings. As disadvantage is the impossibility of perform complete cyclic tests. With this test setup only charge-discharge loadings can be carried out. In the top of each column, the axial load was applied through hydraulic jacks inserted between a steel cap placed on the top of the columns and an upper HEB 200 steel shape, which, in turn, was connected to the foundation steel shape resorting to a pair of high-strength rods per column. Hinged connections were adopted between these rods and the top and foundation steel shapes.

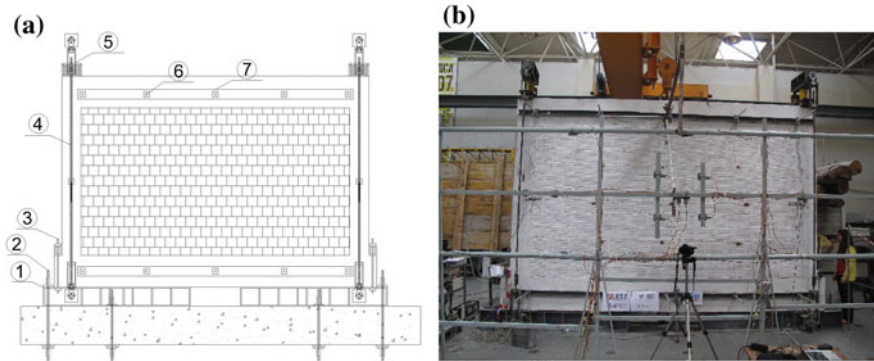
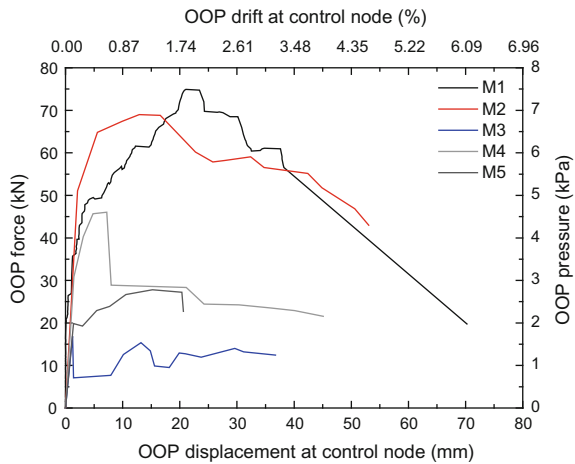


Fig. 1 OOP test setup: **a** front view schematic layout, **b** general front view. 0—strong floor, 1—foundation steel shape, 2—high-strength rods ($\varnothing 30$ mm) fixing the foundation steel shape to the reaction slab, 3—steel rod ($\varnothing 20$ mm) connecting the RC frame to the foundation steel shape, 4—vertical high-strength rods ($\varnothing 30$ mm) to apply axial load, 5—steel cap, 6—steel rods ($\varnothing 20$ mm) connecting the RC frame and the reaction structure, 7—distributing load plate

Fig. 2 Comparative analysis of global results: global force displacement envelope



3.4 Test Results

From the force-displacement envelopes illustrated in Fig. 2 the following observations can be performed:

- Comparing the specimens M1 and M4 it can be observed that the first obtained 50% higher OOP capacity. Besides the same axial load in the top of the columns

and being subjected to different loading solicitations (monotonic and cyclic respectively) the main differences that justify the OOP capacity can be associated to the mortar properties of each test;

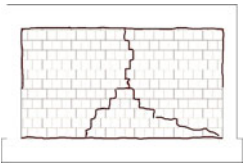
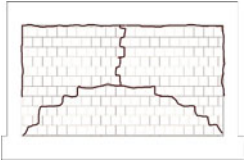
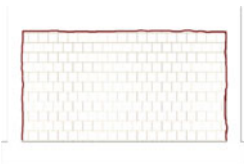
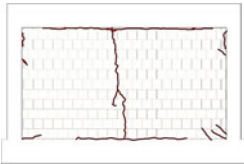
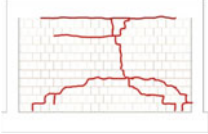
- Regarding the tests M2 and M5, both tested cyclically and without axial load in the columns it can be observed that M5 reached 2 times and half lower OOP force than the M2 justified by the support conditions of the panel;
- From the tests it was observed that the M1 obtained less 7.4% initial stiffness than the M4 specimen, and the test M2 obtained 58.6% higher initial stiffness than M5.

In Table 1 is summarized the maximum strength (F_{max}), OOP displacement at the maximum strength ($D_{OOP,Fmax}$), initial stiffness (K_i) and the failure mode observed on each specimen.

The variables that affected more the OOP response of the specimens were the previous damage and the reduction of the width support of the panel. The previous damage which is more representative of the real behaviour of IM panels during an earthquake, revealed to be the most vulnerable condition. The previous detachment of the panel from the surrounding frame leads to fragile OOP ruptures and reduced about 60% the OOP strength capacity of the panel. From the test results it was observed that the reduction of the panel width support leads to OOP instability of the panel. The OOP strength was significantly reduced, however the arching mechanism provided the sufficient capacity to not occur the panel collapse. The application of the axial load on the top of the columns modified the cracking pattern observed and a vertical cracking was observed followed by the detachment of the panel from the top and bottom beams. On the tests with axial load on the top of the columns, a pronounced strength degradation was observed after reached the maximum ones. From the comparative study it was verified that the monotonic test with application of the axial load on the columns seems to define an envelope of the cyclic tests. These combined variables reached on higher OOP strength capacity, for larger OOP displacements.

Regarding the influence of the axial on the columns prior to carrying out the tests for OOP, revealed a decrease of the OOP stiffness of the panel although combined with an increase in load bearing capacity of the same panels. This increase of the confinement, resulted in a different failure mode when compared with the tests without axial load in the columns. The application the axial load on the top of the columns, without increasing the same during the test leads to the panel acquire a markedly brittle failure. The definition of the stiffness degradation curve, allows to visualize that such non-structural elements begins to lose rigidity to share OOP when the request starts to be transmitted to the panels. This loss of stiffness will be accentuated depending on the behavior exhibited by the panel, that is, the case presents a slow and gradual failure or an instantaneous rupture/collapse.

Table 1 Comparative study: initial stiffness, OOP displacement at maximum strength, OOP maximum strength and failure mode

Specimen	K_i (kN/m)	D_{OOP} , F_{max} (mm)	F_{max} (kN)	Failure mode	Cracking pattern
M1	13,481	21.1	75.9	Vertical cracking at the middle of the panel and detachment between the panel and the top and bottom beams	
M2	26,088	16.6	69.8	Trilinear cracking, extending to the bottom corners of the panel. Deflection pronounced at middle height of the panel	
M3	12,507	1.3	17.9	It was not observed any visible cracking. However, and due to the detachment of the right, left and top borders of the panel from the RC elements, the panel behaved as a rigid body	
M4	14,583	7.2	46.0	Vertical cracking slightly to the left of the central alignment of the panel	
M5	10,797	15.2	27.8	Trilinear cracking, slightly to the right of the central alignment of the panel until the mid-height and then extending to the bottom corners of the panel	

4 Numerical Simulation of the IM Walls Seismic Behaviour

4.1 Introduction

Recent advances have been achieved regarding modelling approaches to infill masonry walls to simulate their real contribution to the structural seismic behaviour. Different modelling approaches are available in the literature, such as detailed modelling strategies where the panel is discretized into numerous elements to consider the local effects in detail (Asteris et al. 2013) and simplified macro-models based on strut model concepts (Asteris et al. 2011a, b). The main advantage of the first group is the fact that the local effects related to cracking, crushing, and contact interaction can be captured and simulated; however, this approach requires several parameters and involves high computational effort. Simplified macro-models use the concept of simulating the IM wall through an equivalent strut, and these models have recently been extended to multi-strut models to consider different infill panel behaviours. In 1960, Polyakov proposed (1960) the concept of the equivalent strut, which was later modified by Holmes. Different proposals that simulate the cyclic infill panels' in-plane behaviour with good accuracy can be found in the literature (Rodrigues et al. 2010; Crisafulli and Carr 2007). Recently, some authors provided strut models with the capacity to represent the combined in-plane and out-of-plane behaviour, for example, Kadysiewsko and Modalam's proposal (Kadysiewsko and Modalam 2009), which modelled an IM wall through one diagonal beam element pinned at the edges and provided with a lumped mass in the centre that was active only in the out-of-plane direction. This section pretends to describe a simplified numerical model that was developed to represent the IM walls combined IP and OOP behaviour.

4.2 Simplified Macro-modelling Approach

The numerical modelling approach proposed here is a simplified strut macro-model that is an upgrade of the equivalent bi-diagonal compression strut model proposed by Rodrigues et al. (2010) and later upgraded and implemented in OpenSees (Furtado et al. 2016a, b). The model considers the interaction of the masonry panel behaviour in both directions; the occurrence of panel damage in one direction affects its behaviour in the other direction. Each panel is numerically simulated by four support strut elements, with rigid behaviour, and a central strut element, where the non-linear hysteretic behaviour is concentrated (Fig. 3a). This simplified macro-model can be applied in OpenSees (Mckenna et al. 2000) in association with the available OpenSees materials, sections, and element commands. The infill model was composed of four elastic beam columns for the diagonal elements and one nonlinear beam column element for the central element (Fig. 3) with six

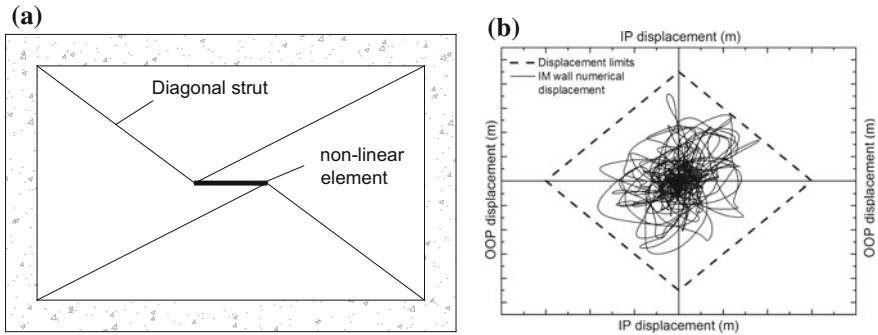


Fig. 3 IM walls simplified numerical modelling approach: **a** schematic layout, **b** IP and OOP interaction

degrees of freedom. The idealization of the central element's non-linear behaviour is characterized through a multi-linear curve, defined by eight parameters, representing: (a) cracking; (b) yielding; (c) maximum strength, corresponding to the beginning of crushing; and (d) residual strength. The Pinching4 uniaxial material model was used to represent the hysteretic behaviour of the infill panel and was attributed to the central element. This uniaxial material is used to construct a material that represents a “pinched” load-deformation response and exhibits degradation under cyclic loading. Cyclic degradation of strength and stiffness occurs in three ways: by unloading stiffness degradation, reloading stiffness degradation, and strength degradation. The model has the main advantage of considering the combined IP and OOP behaviour of the panel. The central element is joined to the diagonal struts through two nodes in which the OOP mass is lumped. The OOP behaviour is assumed to be elastic-plastic with strength and stiffness calculated according to Kadysiewski and Mosalam's approach. Also in this case, part of the model is an algorithm that removes the elements representative of the infill from the structural model if its IP and OOP displacement history exceeds an interaction domain in terms of ultimate displacement. The domain is linear and assumes that, for the undamaged panel, the maximum in-plane drift is equal to 1.5% while the maximum OOP drift is equal to 3% (Fig. 3b).

4.3 Seismic Assessment of a 8 Storey Infilled RC Structure Considering IM Walls OOP Behaviour

With the aim of evaluate the influence of the evaluate the seismic assessment of RC buildings with different considerations regarding the IM walls modelling subjected to seismic actions, one eight-storey building was studied. The building has plant

dimensions of 20 m × 15 m, which consists of 4 × 5 m modules, with a storey height of 3 m. The building was designed by the Portuguese Laboratory of Civil Engineering (LNEC) as part of a study on the seismic design of buildings, in accordance with the existing code rules in Portugal. A 3D model was generated in the computer software OpenSees (Mckenna et al. 2000). A set of three building configurations was selected according to the IM modelling strategies adopted: (i) bare frame model (BF) which does not consider the presence of the IM walls; (ii) in-plane model (IP) which considers the presence of the IM walls in the external perimeter of the building, and only the IP behaviour is considered; (iii) OOP model (IP_OOP) which considers the presence of the IM walls in the external perimeter of the building and both the IP and OOP behaviour interaction. The 3D models were subjected to incremental dynamic analysis (IDA) to develop fragility curves according to the Monte Carlo proposal.

Limit state criteria based on the maximum inter-storey drift were selected for the present study. To determine the inter-storey drift limits, a set of 6 values proposed by Rosseto and Elnashai (2005) was fixed, as described in Table 2.

From the resulting vulnerability curves, it can be observed that the difference between the performance of the three numerical models namely for the moderate, extensive and collapse damage states (Fig. 4). As concerns moderate damage, it is observed that the BF model is the most vulnerable model and that the IM walls with only IP behaviour protect the building. Extensive damage occurs for lower peak ground acceleration values (<0.4 g) more quickly for the BF model and at >0.4 g for the IP_OOP model.

Finally, it is observed that the OOP behaviour of the IM walls is critical in terms of the collapse damage state. In fact, clear differences are observed between the IP_OOP model compared with the BF and IP models. This fact increases the need to consider the OOP behaviour of the IM walls in the seismic safety assessment of the existing buildings, and consequently in the numerical models.

Table 2 Inter-storey drift limits for infilled RC frames according to Rosseto and Elnashai’s (2005) proposals

Damage state	Inter-storey drift (%)
Slight	0.05
Light	0.08
Moderate	0.30
Extensive	1.15
Partial collapse	2.80
Collapse	>4.40

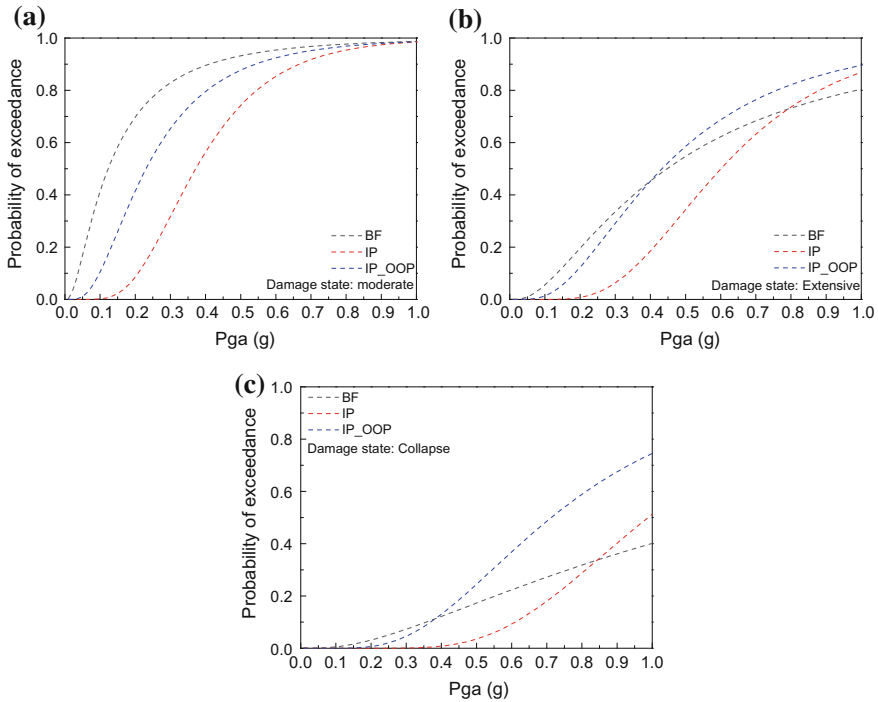


Fig. 4 Vulnerability curves for **a** moderate, **b** extensive and **c** collapse damage states

5 Conclusion

This chapter presents a research work regarding the IM walls seismic behaviour, such experimental and numerical, and their interaction with the RC structures. From this chapter the main conclusions that can be achieved is that in the assessment of existing buildings, and in the design of new buildings:

- Consideration of the masonry infill walls in the structural design (based on simple checking rules/procedures after the structural design) should be enforced;
- Attention should be given to the stiffness differences between the 1st storey and the upper storeys (storey height, dimensions and position of openings, distribution of masonry infill walls);

From the test results, and from numerical analysis of the RC building studied, with the simplified macro-model that simulates the IP and OOP behavior of IM walls, it can be concluded:

- The large IP shear demands that IM walls are subjected to are likely to increase their out-of-plane vulnerability;

- The OOP collapse of infills can result in serious human and material consequences, as observed in recent earthquakes.

So, there is a need to consider the OOP behavior of IM walls in the seismic safety assessment of existing RC structures.

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