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MASONRY INFILLS AND EARTHQUAKES

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ABSTRACT
Masonry infills are building elements with high cost that often suffer cracking due to movement of the supports and thermohygral behaviour, with serious consequences in comfort and efficiency, particularly in the case of enclosure walls. The new seismic code in Europe (Eurocode 8) clearly defines the structural designer as responsible for the safety of masonry infills. In this context, there is a clear need of adequate provisions to reduce the seismic vulnerability in regions of moderate and high seismicity, which are not clearly prescribed in the code. A research program involving monotonic and shaking table testing of masonry infills is being prepared at University of Minho and National Laboratory of Civil Engineering. The reasons for the program and a preliminary analysis of the prototype are addressed here.

KEYWORDS: masonry infills, earthquake damage, push-over analysis, shaking table.

INTRODUCTION
Masonry walls are still the most used enclosure system in Europe and they are subjected to several functional and structural requirements. A comfortable and safe internal environment must be provided, namely with respect to temperature, noise, moisture and fire. Furthermore, even non-load-bearing walls have to be checked against wind loads, seismic actions and accidental loads. Masonry clay walls are made of natural materials that ensure a healthy indoor environment and exhibit very good durability properties. Besides, experience (and calculations) has proven the positive role of masonry walls in the seismic behaviour of buildings, provided that severe in-plane and in-elevation irregularities are prevented. Actually, masonry walls offer substantial extra bearing capacity (not taken into account in the design), as well as extra stiffness, which results in reduced deformations of the entire structure.

Masonry walls, including rendering, can represent around 13 to 17 % of the total value of the building construction market [1]. The total European market value for walls can then be estimated at very significant value around 45 and 60 billion euro (Construction Investment at EU 27 – 2006 was 1196 billion euro [2] and the building market can be conservatively estimated at

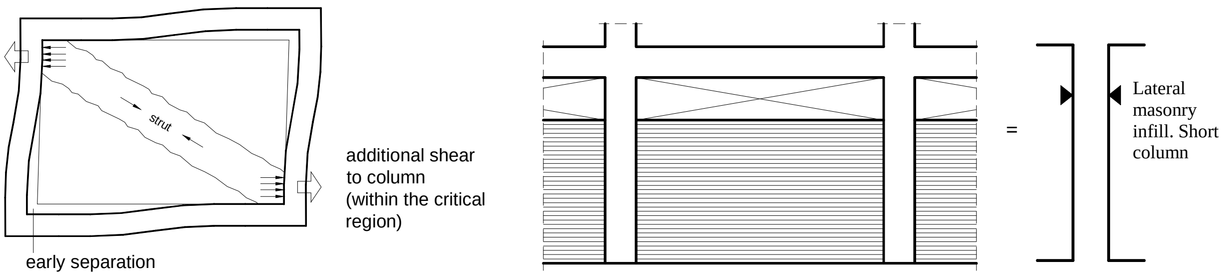
30% of the total market). Additionally, earthquake induced damage in masonry walls has major consequences, both social and economical. Such damage can be classified as follows: (a) Out-of-plane collapse, see Figure 1, may cause injuries or even casualties, and it certainly requires very high post-earthquake reconstruction costs; (b) In-plane damage leads also to high post-earthquake reconstruction and repair costs, even for low or no damage in the structural skeleton. For a recent earthquake in Greece (Parnitha, Magnitude 5.9, September 1999), the Hellenic Association of Schools accounted 60% of the repair costs due to damage in masonry infills, associated finishings and installations (water, electricity, etc.). An older statistical study carried out by insurance companies [3] refers even higher costs (up to 80% of the total value of the building) for repairing and reconstructing non-structural elements, including masonry infills, finishings, gypsum ceiling boards, windows, doors and installations. Therefore, the market value of masonry walls and the unbearable costs of post-earthquake repair and rebuilt, justifies the need of further investment in the study of novel possibilities for the construction of masonry enclosure walls. In this context, technological advances for the industry are a must, or it faces the risk of losing a significant market.

Figure 1: Collapse and Fall of Masonry Enclosures Due to an Earthquake

EARTHQUAKE CODES IN EUROPE

Recent earthquake codes in Europe [4-6] require the safety assessment of non-structural elements (parapets, antennas, mechanical equipment, veneer masonry walls, infill walls, etc.) and their supports, when their collapse entails risks for people or for the main structure, or the equipment is affected in a critical way. The models for structural analysis should take into consideration the importance and hazard of the non-structural elements.

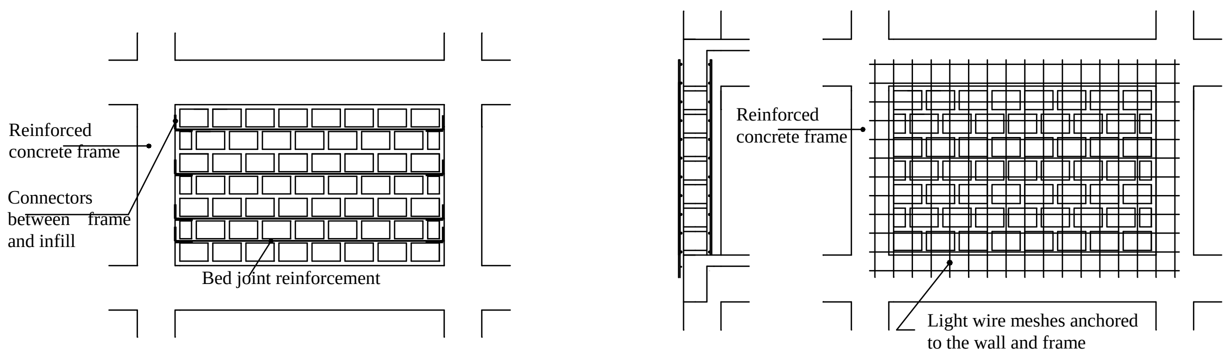
The following aspects should be considered in the seismic design of structures with non load-bearing masonry infills: (a) consequences of irregularity in plan; (b) consequences of irregularity in elevation; (c) high uncertainties related to the behaviour of the infills, namely the variability of their mechanical properties and of their attachment to the surrounding frame, their possible modification during the use of the building, as well as their non-uniform degree of damage suffered during the earthquake itself; (d) possible adverse local effects due to the frame-infill-interaction, see Figure 2. When the masonry infills are not regularly distributed, these irregularities must be taken into account or the accidental eccentricity must be increased. If there are considerable irregularities in elevation (e.g. drastic reduction of infills in one or more storeys compared to the others), the seismic action effects in the vertical elements of the respective storeys shall be increased [4].



(a) (b)

Figure 2: Local Adverse Effects in the Interaction Frame-Infill: (a) Additional Shear to Column; (b) Short Column Due to Long Openings.

According to Eurocode 8 [4], appropriate measures should be taken to avoid brittle failure and premature disintegration of the infill walls, as well as the partial or total out-of-plane collapse of slender masonry panels. Typical examples of measures include bed joint reinforcement, see Figure 3a, light wire meshes well anchored to the wall, see Figure 3b, and concrete posts and belts across the panels and through the full thickness of the wall. If there are large openings or perforations in any of the infill panels, their edges should be trimmed with belts and posts.



(a) (b)

Figure 3: Solutions for Damage Control in Masonry Infills Without Openings: (a) Masonry with Bed Joint Reinforcement and Connectors; (b) Anchored Light Wire Meshes.

PREVIOUS RESEARCH

Masonry infills received much attention in the past, particularly with respect to the possibility of carrying out adequate structural analyses. The analysis of frames infilled with masonry subjected to in-plane lateral actions is a statically undetermined problem, involving stress redistributions between the frame and panel, often simulated using an equivalent compression strut concept to replace the action of the masonry panels [7-9]. In this system, the moderate stiffness of the frame interacts with the very high in-plane stiffness of the masonry panel. Moreover, the confinement effect provided by the frame allows the panel to support loads and to attain strains much larger than those obtained in the isolated panel. Before ultimate load, the panel usually separates from the frame, cracks diagonally and, possibly, crushes at the compressed toes. Therefore, the process is highly non-linear and exhibits considerable complexity.

The progress of the finite element method allowed a better understanding of the complex behaviour of infilled frames [10]. Several difficulties were evident from the simulations, namely the issues of modelling the separation between frame and panel, of the bond strength and friction of the connection between frame and panel, and of the mechanical constitutive behaviour of masonry itself. Later [11] it was found that the critical stresses for the masonry panel are located in the centre and are mostly associated with tensile and shear failure. In this case, the frame-panel interaction was modelled by using double nodes and normal springs at the interfaces, with contact/separation modelled in a simplified way. This work was further extended with non-linear behaviour of the panel and frame, by many authors. Presently, most of the research is devoted to the behaviour under seismic loading and retrofitting, namely for an equivalent strut approach with a smooth hysteretic model [12], for weakly reinforced masonry [13] and for masonry retrofitted with fibre-reinforced polymer laminates [14].

The experimental and numerical results using hollow clay bricks similar to the ones adopted in Portugal [13] show that the presence of some kind of light reinforcement increases significantly the performance of the response, and that non-reinforced masonry infills cannot fulfil the code in zones of moderate to high seismicity, see Figure 4 and 5. In Figure 5, the highest failure loads are for the infills tested out-of-plane without previous in plane damage, whereas the lowest failure loads are for panels tested with previous in plane damage. It is noted that: (a) in well designed reinforced concrete frames, major damage or out of plane expulsion of the infill occurs before significant damage occurs to the frame; (b) out of plane expulsion of the masonry infill should be considered as an ultimate limit state due to the risk to human life, even if the reinforced concrete frame is far from collapse; (c) The out of plane seismic loading in masonry infills increases with the height but in plane damage in buildings tends to concentrate in the lower storeys, meaning that the critical situation for expulsion of the infill might occur at an intermediate or lower storey, as observed in real earthquakes.

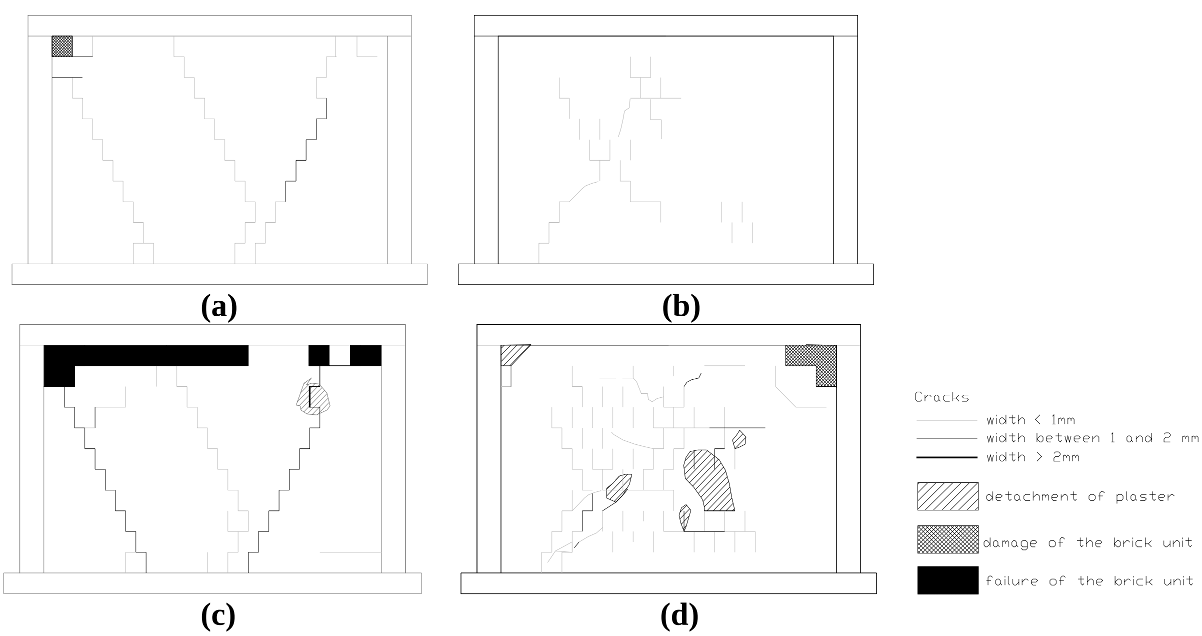
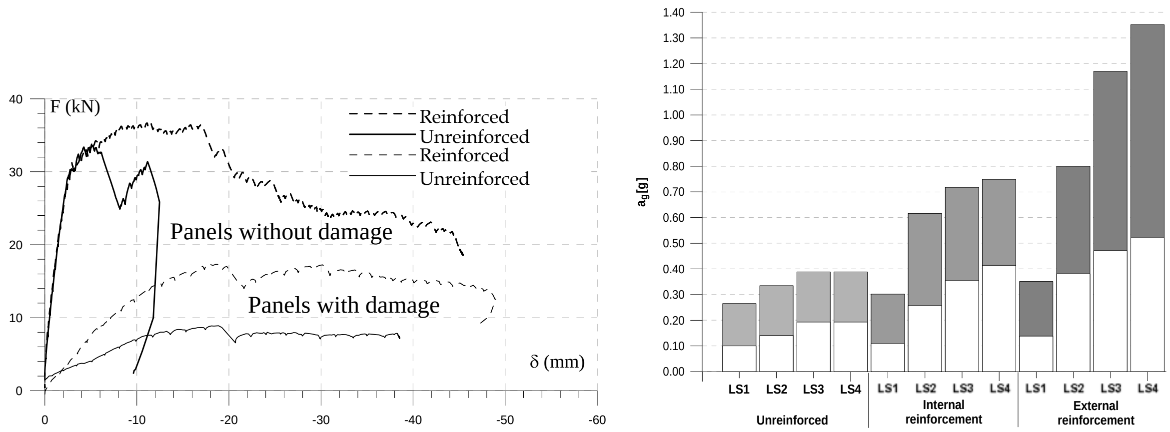


Figure 4: Observed Damage of Masonry Infills Without (a,c) and With (b,d) Light Bed Joint Reinforcement for a Drift of: (a,b) 0.2% e (c,d) 0.4% [13].



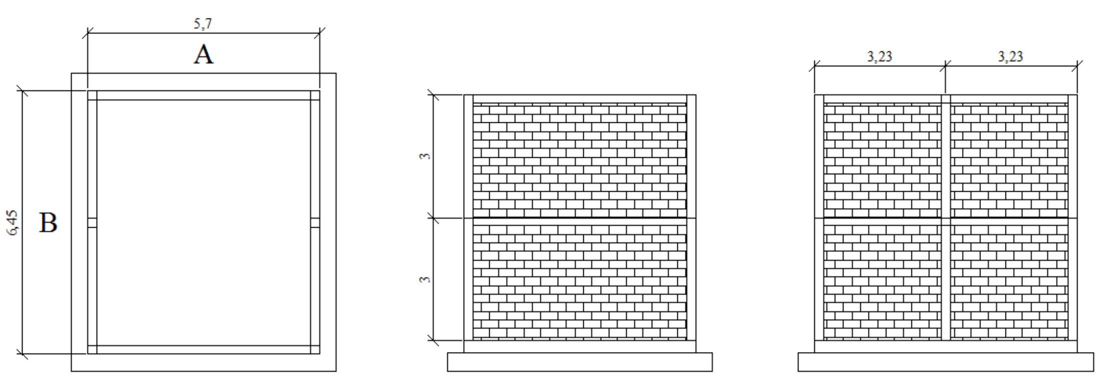
(a) (b)

Figure 5: (a) Test Comparison Between Force-Displacement Diagrams for Reinforced and Unreinforced Out of Plane Tests, for Panels Without Damage and With Damage Induced by a Drift of 0.4 %. (b) Numerical Minimum and Maximum Peak Ground Acceleration Required to Reach a Given Limit State LS, Depending on Building Type and Storeys (LS1 – Undamaged; LS2 – Operational; LS3 – Life Safety; LS4 – Collapse) [12].

DEFINITION OF THE EXPERIMENTAL PROGRAM

The objective of the current testing program is to test, in a shaking table, three typical reinforced concrete framed buildings with masonry enclosures: Model A replicates the traditionally designed buildings until today, with double leaf masonry enclosures of horizontally perforated clay units; Models B and C replicate the buildings occurring in the immediate future, designed according to Eurocode 8 [4] and with a single leaf masonry enclosure. The possibilities for execution are the use of vertically perforated clay units that fulfil the more stringent thermal requirements or the use of the traditional weak horizontally perforated clay units with addition of an external insulation system.

The prototype buildings have the geometry shown in Figure 6. These are two-storey buildings with one span in one direction and two spans in another direction, with in-plane dimensions of 6.45 (B) x 5.70 (A) m^2 and 6.00 m height, scaled 1:1.5. Details of the walls are given in Figure 7, including the double leaf wall and the lightly reinforced single leaf wall with a steel mesh or bed joint reinforcement, and also with or without connections to the reinforced concrete columns.



(a) (b) (c)

Figure 6: Prototype Buildings to Be Considered in the Testing Program.

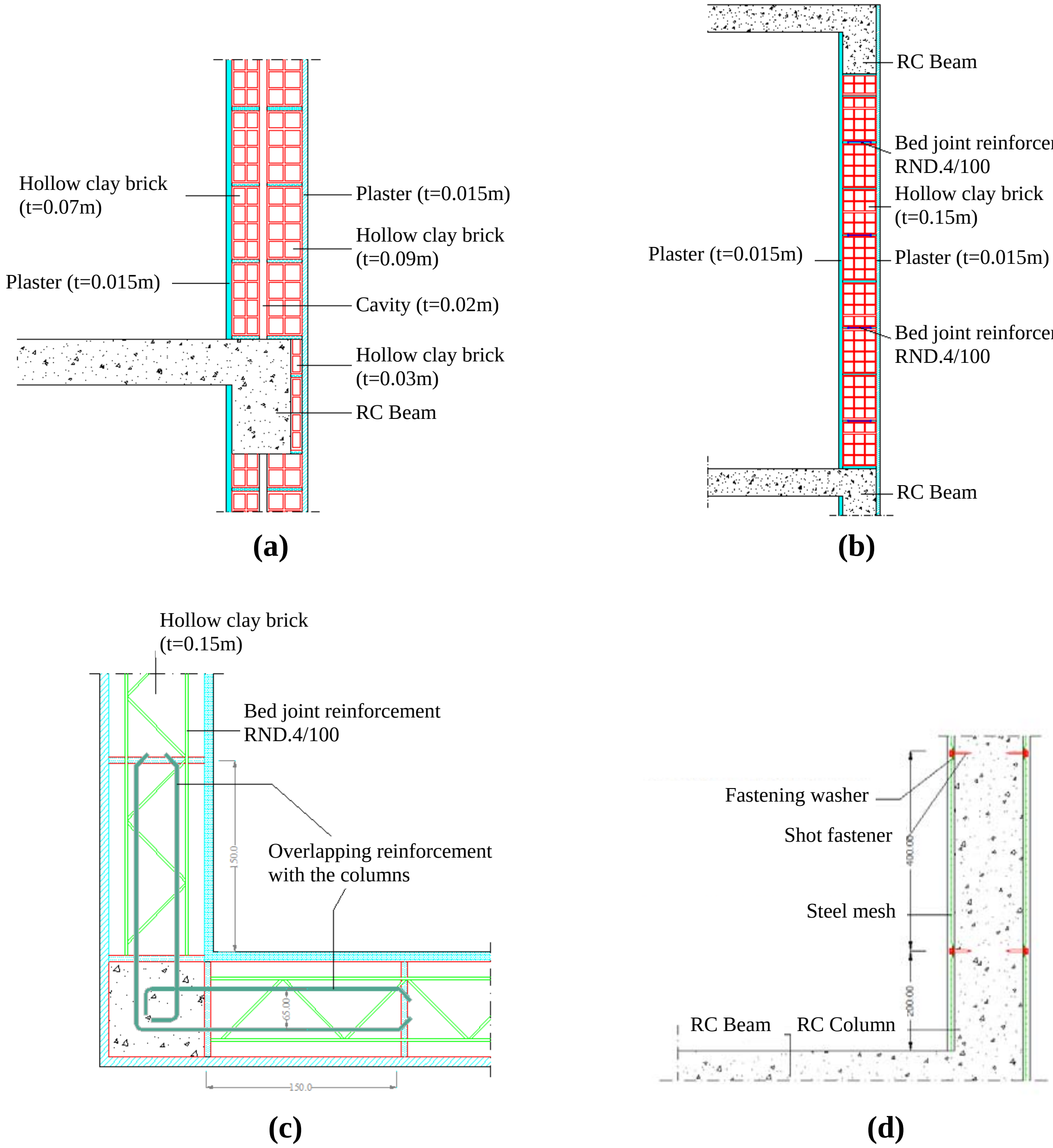
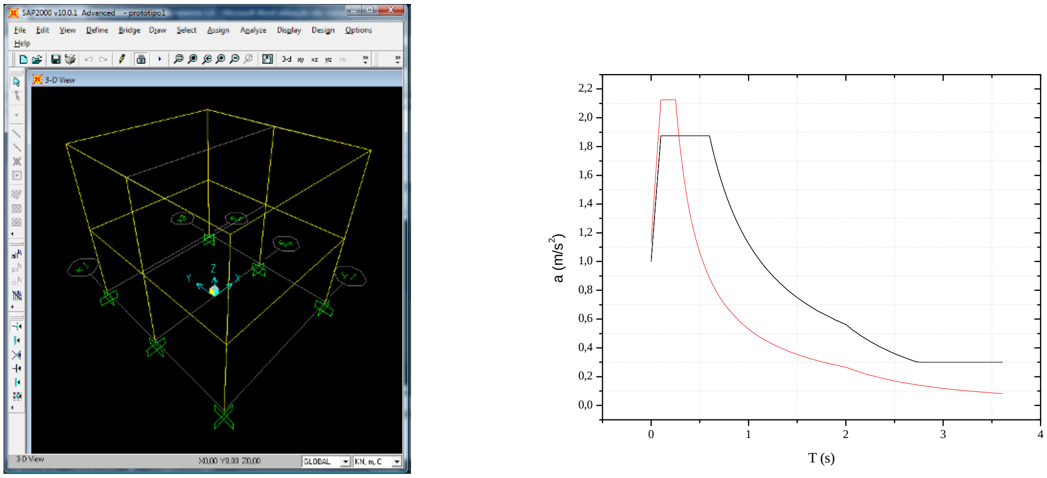


Figure 7: Details of Prototype Buildings (Note the Scale 1:1.5): (a) Typical Existing Cavity Wall Buildings; (b) Planned New Buildings With External Thermal Insulation or New Thermal Units Vertically Perforated (With Bed Joint Reinforcement); (c) Detail of the Connection Between Masonry Infill and Column for Bed Joint Reinforcement; (d) Detail of The Connection Between Masonry Infill and Column for Plaster Reinforced With a Steel Mesh.

DESIGN AND ASSESSMENT OF THE BUILDINGS

The buildings were designed as usual in practice, considering the dead weight and the live loads. Modal superposition was adopted for the seismic analysis. Figure 8 illustrates the adopted structural analysis model (SAP2000® from Computers & Structures, Inc. [15]) and the adopted design response spectrum for the building according to Eurocode 8 (a near field and a far field earthquake are shown in the plot.



(a) **(b)**

Figure 8: Design Aspects: (a) Structural Model; (b) Design Response Spectrum for New Buildings, According to Eurocode 8.

In the tests, the design earthquake will be applied in two steps, at a level of 50% and 100%, so that damage does not increase due to successive random base excitations. But, it is well known that masonry infills significantly change the response of reinforced concrete buildings. The objective of the tests is also to quantify different damage levels, in the usual context of "Collapse Prevention", "Life Safety", "Immediate Occupancy" and "Operational Level", and limitation of interstory drift, see Figure 9.

	Collapse Prevention Level (5-E)	Life Safety Level (3-C)	Immediate Occupancy Level (1-B)	Operational Level (1-A)
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.

(a)

Interstory drift limit	*Immediate occupancy*	*Damage control*	*Life safety*	*Structural stability*
Maximum total drift	0.01	0.01 – 0.02	0.02	$0.33(V_i/P_i)$
Maximum inelastic drift	0.005	0.005 – 0.015	No limit	No limit

(b)

Figure 9: Performance Levels: (a) Definition From FEMA-356 [16]; (b) Drift Limit From ATC-40 [17].

For the purpose of defining the loading steps and the expected performance, two numerical models were considered, one without the masonry infills and one with the masonry infills. For the model without the masonry infills, modelling is rather straightforward and can be defined fully automatically in SAP2000. For each bar, two hinges have been defined in the ends. In the case of the beams, the hinges only consider bending, whereas the axial force is also considered in the columns. The lateral loads have been considered uniformly distributed, i.e. proportional to the mass, or proportional to the first elastic mode [4].

The model with the masonry infills is far less straightforward, and an example of a possible modelling strategy and associated mechanical properties is given in [18]. Additional diagonal struts (solely with compression failure) are added as diagonal bars to the previous model, where the thickness of the bar is defined according to [19] and the strength of the bar is defined according to Eurocode 6 [20]. The performance levels, with the exception of “Operation Level” were assigned to each singular point in the stress-strain diagram and two hinges were again considered at the ends of each diagonal.

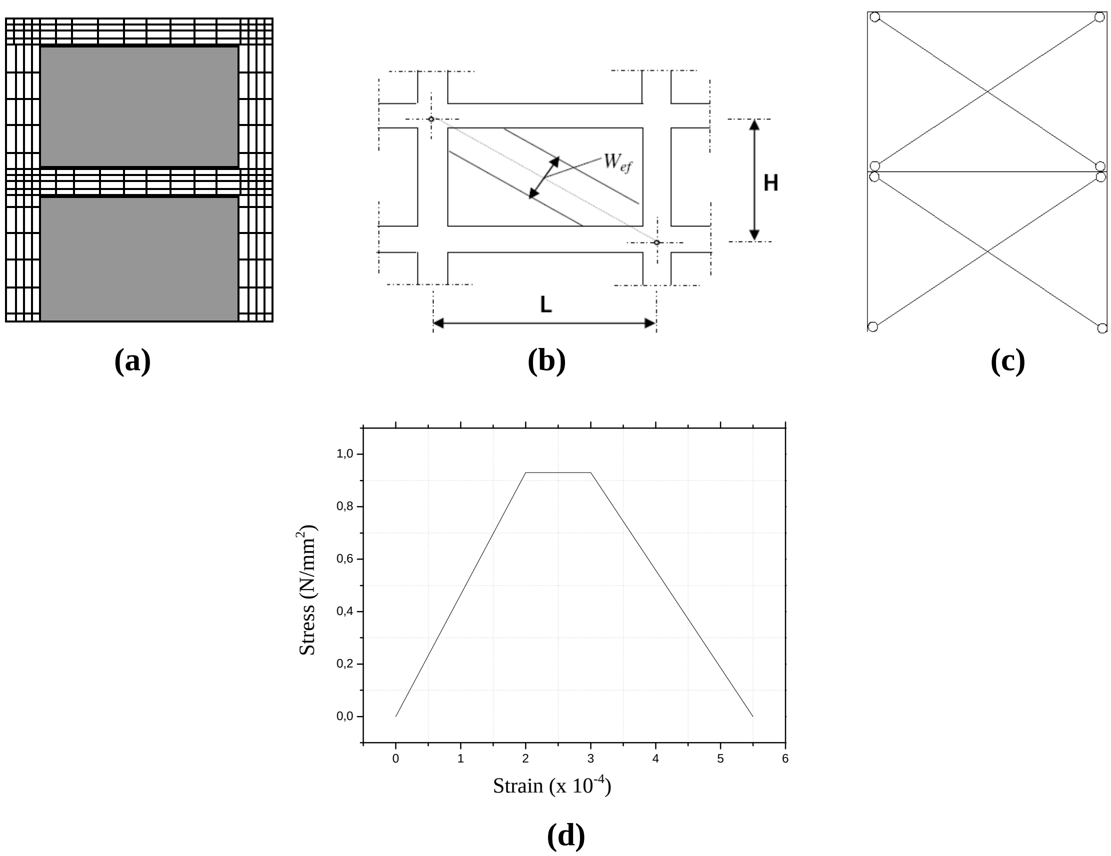


Figure 10: Modelling of Reinforced Concrete Frame Buildings: (a) Model with Masonry Panels; (b) Definition of an Equivalent Diagonal; (c) Model With Diagonal Bars; (d) Adopted Stress-Strain Relationship for the Diagonals.

The results of the analysis are shown in Figure 11, where it can be seen that the failure mode and capacity curve for the model with and without the walls are different. The model with the masonry infills concentrates the damage and failure in the lower level, whereas the model without the masonry fill exhibits also damage in the elements of the second storey. Out-of-plane failure of the walls is not considered in the model at this stage, which is a key issue to define the performance level, as discussed above. It is also shown that the capacity of the structure (and the associated performance level) increases about 20% due to the presence of the masonry infills and the stiffness of the response increases enormously, with a reduction of the peak displacement of about 60%.

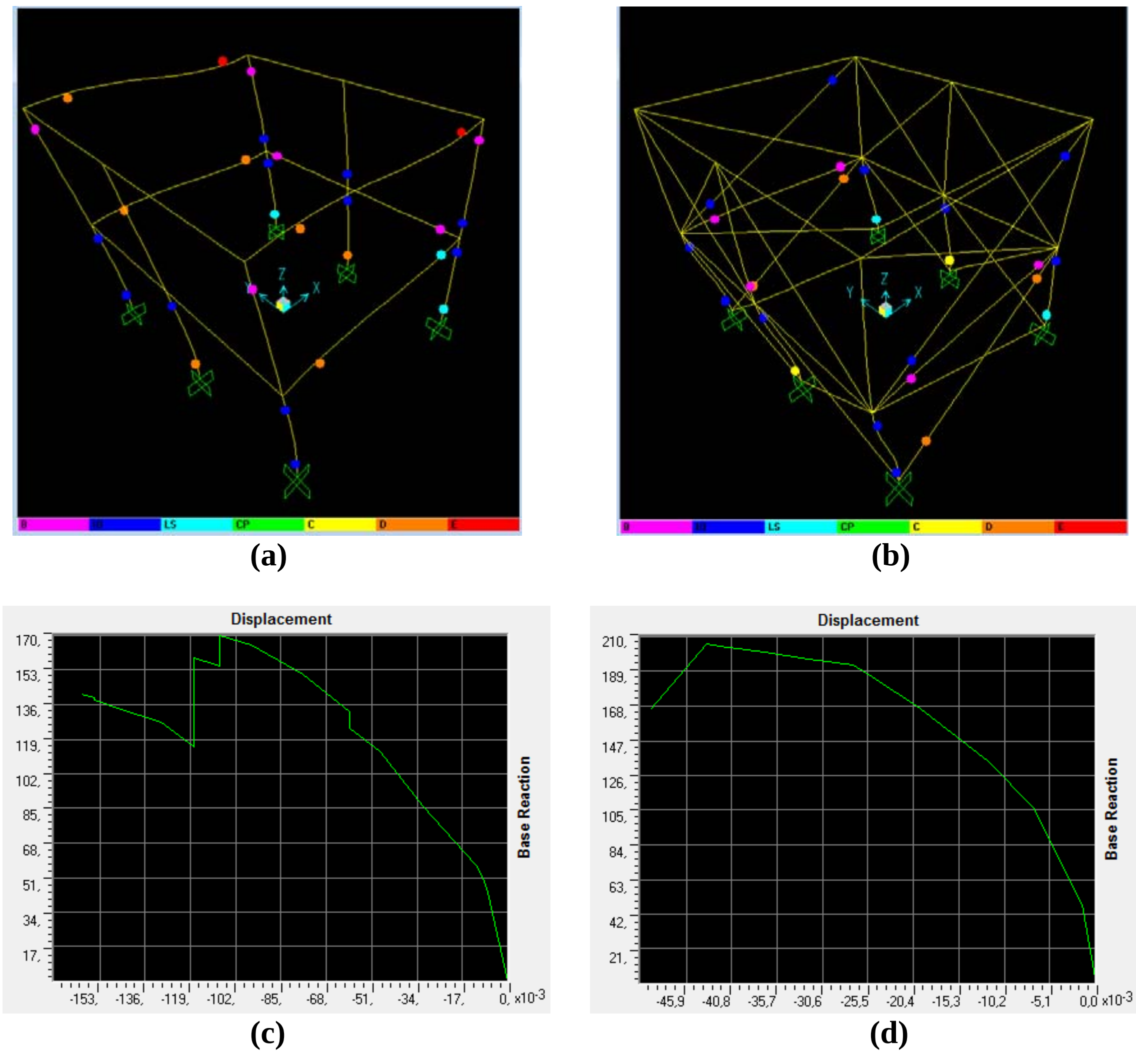


Figure 11: Results of Push-Over Analysis: Failure Mode (a) With and Without (b) Masonry Infills; Capacity Curve (c) With and Without (d) Masonry Infills.

CONCLUSIONS

The paper addresses the issue of masonry panels infilled in reinforced concrete frame buildings under seismic loading, the vulnerability of the panels and the measures to reduce their vulnerability. Subsequently, a shaking table testing program to be carried out together with the National Laboratory of Civil Engineering is detailed and the preliminary structural analysis to define the loading steps is discussed. For this purpose, a push-over analysis of the reinforced concrete building, with and without the masonry infills, is discussed.

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