DAMAGES IN NON-STRUCTURAL ELEMENTS OF R.C. RESIDENTIAL BUILDINGS CAUSED BY THE 2011 LORCA EARTHQUAKE

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Abstract

This paper presents a study of the seismic response of four to six-storey RC Spanish residential buildings with a structure comprising reinforced concrete frames and interior and exterior unreinforced masonry infill walls and parapet walls. It is particularly focused on the building properties that influence non-structural component damage and collapse; due to that non-structural exterior unreinforced masonry infill walls and parapets are the cause of the majority of fatalities during some medium magnitude earthquakes. In the 2011 Lorca earthquake, Mw=5.2 only one building collapsed; however, there were nine fatalities as a result of falling debris.

Authors have studied 65 reports of buildings belonging to the aforementioned residential type, elaborated by experts just after the 2011 Lorca seismic event. Throughout the reports, the experts detail the damage suffered by structural and non-structural elements. Damage to non-structural elements could be caused by in-plane seismic loading–which commonly affects lower floors, by out-plane – higher floors- or by a combination of both types of seismic loading – intermediate floors-.

Data provided by these technicians along with theoretical research has permitted us to characterize statistically the urban, structural and non-structural features of this building type in the case of Lorca. Added to this, the degree of damage to the buildings has been compared with the age of the buildings and the changes in the seismic Spanish regulations; revealing that, the most recent regulations have reduced the vulnerability of this building type.

Furthermore, particularly vulnerable elements and Spanish construction techniques that are inadequate for seismic areas have been identified and documented, and alternative techniques have been proposed. Unreinforced masonry chimneys, single-leaf parapet walls without reinforcing elements or stone cladding without any type of mechanical fixation fall into this category.

Additionally, two actual Lorca buildings that suffered damages during this earthquake have been modelled using the Software SAP2000 according to plans, technical descriptions and graphical information available. In this way, their main and secondary modes of vibration and torsion effects have been studied. Moreover, a thorough analysis of the Floor Response Spectra has been carried out by means of a Time History Analysis with the Lorca earthquake accelerograms. This analysis aims to study the influence of the presence of a flexible ground floor, the consideration of the linear or non-linear structural behaviour and a comparison with the EN-1998 simplified predictions. This comparison illustrates the deficiencies of the European simplified predictions for short-period earthquakes, like the 2011 Lorca event.

Keywords: Spanish residential buildings, reinforced concrete structure, unreinforced masonry non-structural elements

1. Introduction

In medium seismicity areas like some regions of Europe, it is unusual for reinforced concrete (RC) structures to collapse when a medium magnitude earthquake occurs. This is due to robust earthquake resistance regulations being in place [1] and national standards that define strict criteria of quality for such constructions (e.g., [2, 3] in the Spanish case). Nevertheless, it is quite common for their non-structural elements (NSEs) to suffer damages or collapse that provoke the fall of debris onto the public roads, and consequently, a significant number of fatalities. Additionally, current regulations are not so rigorous in relation to NSEs due to this being a field of research still in development. An instance of this fact is the case of the 2011 Lorca earthquake since the nine fatalities that occurred during its quakes were caused by the fall of debris of non-structural elements –solely one RC building collapsed and it did not provoke any loss of life-.

For the aforementioned reasons, more than 300 reports documented by experts just after the 2011 Lorca earthquake have been analysed. In particular, those reports corresponding to four to six-storey RC buildings were studied in detail; as the majority of the fatalities occurred in the districts of Santiago (three), San Diego (two) and La Viña (four), of which this building type is predominant in the latter two. This research was carried out with three objectives: to identify the most common structural and non-structural features of this type of buildings, to determine the most hazardous non-structural elements within this building typology and to reach a better understanding of their behaviour.

Finally, this work presents the models of two buildings which are a prime example of the residential type studied according to several identified features. The Floor Response Spectra of both buildings has been obtained via Nonlinear Time History Analysis in the case of the Lorca earthquake, and their results have been compared with the predictions of the European regulation [1].

2. State of the art

In the last few decades, the research into the field of behaviour and damages of NSEs has attracted the interest of the scientific community. On the one hand, a significant number of reports and in situ studies are being carried out after the occurrence of different earthquakes [4]. On the other hand, several approaches are being investigated in order to idealise these elements, from simple analytical idealisations to complex numerical simulations. A significant part of such is focused on the characterization of non-structural unreinforced masonry elements, such as freestanding parapets or infill walls, as these elements are widespread in areas where there is a tradition of using masonry in residential buildings.

The behaviour and damage to the NSEs is usually classified as caused by in-plane loading, –which habitually has greater effects on lower floors-, by out-of-plane loading – on upper floors- or by a combination of both types of lateral loadings [5]. Additionally, the correct execution of the connections to the structural elements is a crucial aspect that should not be overlooked.

For its part, this work provides an analysis of the data collected after the Lorca earthquake aiming to characterise the features of the buildings and non-structural elements belonging to the previously aforementioned type. Moreover, the most frequent damages to non-structural elements have been described with the objective of detecting especially vulnerable elements and inadequate construction techniques. Results obtained may be extended to other cities of similar characteristics.

3. Methodology

Lorca is a city localised in south-eastern Spain, pertaining to the Autonomous Community of Murcia, comprising of around three km². In her work focused on this city, [6] indicates that the most common building types in this zone are one to two storey masonry buildings, and two to six-storey RC buildings with frame structures and non-structural unreinforced masonry elements. Furthermore, the author concludes that the RC buildings which suffered greater structural damage were: those buildings that are between four and six storeys in height, with characteristics that influence their seismic vulnerability such as vertical irregularity, protrusions in building façades (e.g., balconies), soft stories (partially enclosed) or located either within an urban expansion block with a central courtyard or in an urban history block.

In order to carry out more in-depth research of the damages shown by non-structural elements for this case study, the present authors have worked on a database of more than 300 reports elaborated by experts just after the seism of Lorca. This database includes buildings located all over Lorca which pertain to different types, whose unique common feature is that they were commissioned and elaborated by the same team of architects and engineers. Within this information, the visual reports of 65 four to six-storey RC or mixed buildings, and the architectural plans and reparation projects of 25 of them are found.

Thanks to this, throughout this work, the urban, structural and design features and the state of damage of the 65 buildings pertaining to the aforementioned building type have been statistically analysed. Furthermore, the main damages presented to them and possible improvements in the construction techniques of their elements have been documented. In relation to the graphical documentation of the reports, it is worth noting that inadequate construction techniques of non-structural elements represent a serious risk to life.

Additionally, a summary of the national regulations applicable to structural and non-structural elements is made herein. Seismic regulations in force during the periods of construction of the buildings analysed are: PGS-1 [7] (1968-1973); PDS-1 [8] (1974-1994), which includes a hazard map, some calculation methods and some mandatory and recommended construction rules; NCSE-94 [9] (1995-2001), a standard that represented a great advance by including a probabilistic seismic hazard map, expressed in terms of basic seismic acceleration; and NCSE-02 [2] (2002-currently).

This latest one was in force when the Lorca earthquake occurred. This regulation incorporates a novelty in the cases of high-intensity earthquakes: if they cause damages to buildings, an inspection and an evaluation of the severity of the damages must be done. Reports derived from compliance with this law constitute the data used for this study. Moreover, it establishes design rules and rigorous constructive prescriptions and details, especially for the design of the structure of masonry buildings and RC buildings. Lastly, it provides some criteria to prevent non-structural elements such as chimneys, infill and parapet masonry walls from detaching, in order to reduce the number of fatalities.

4. Results

According to the reports, the majority of the buildings pertaining to the type studied were built in the four decades leading up to the Lorca seism (note that the gathering of the data was carried out in the year 2011).

4.2 Characterisation of the features of the building type

The following features of the 65 four to six-storey RC residential buildings have been studied statistically:

-Urban features: the type of block in which they are located has been characterised as, distinguished between open block, isolated block, urban expansion block and urban history block. As can be seen in the histogram of Figure 1 the majority of the buildings pertaining to this type are located within an urban history block.

-Structural features: their structural system and their type of floor system: one-way joist floor system or two-way slab floor system (waffle slab).

-Design features: the total built area and the number of vertical communication cores are among the design features studied.

-Non-structural elements features: the number of buildings that present outer walls made of facing bricks has been studied. All the buildings documented have interior infill walls made of brick masonry. Furthermore, a widespread presence of suspended ceilings and cladding panels of different material have been noticed on the ground floor.

Lastly, the degree of damage to the buildings has been compared with some of their features, such as their age, vulnerability or structural system.

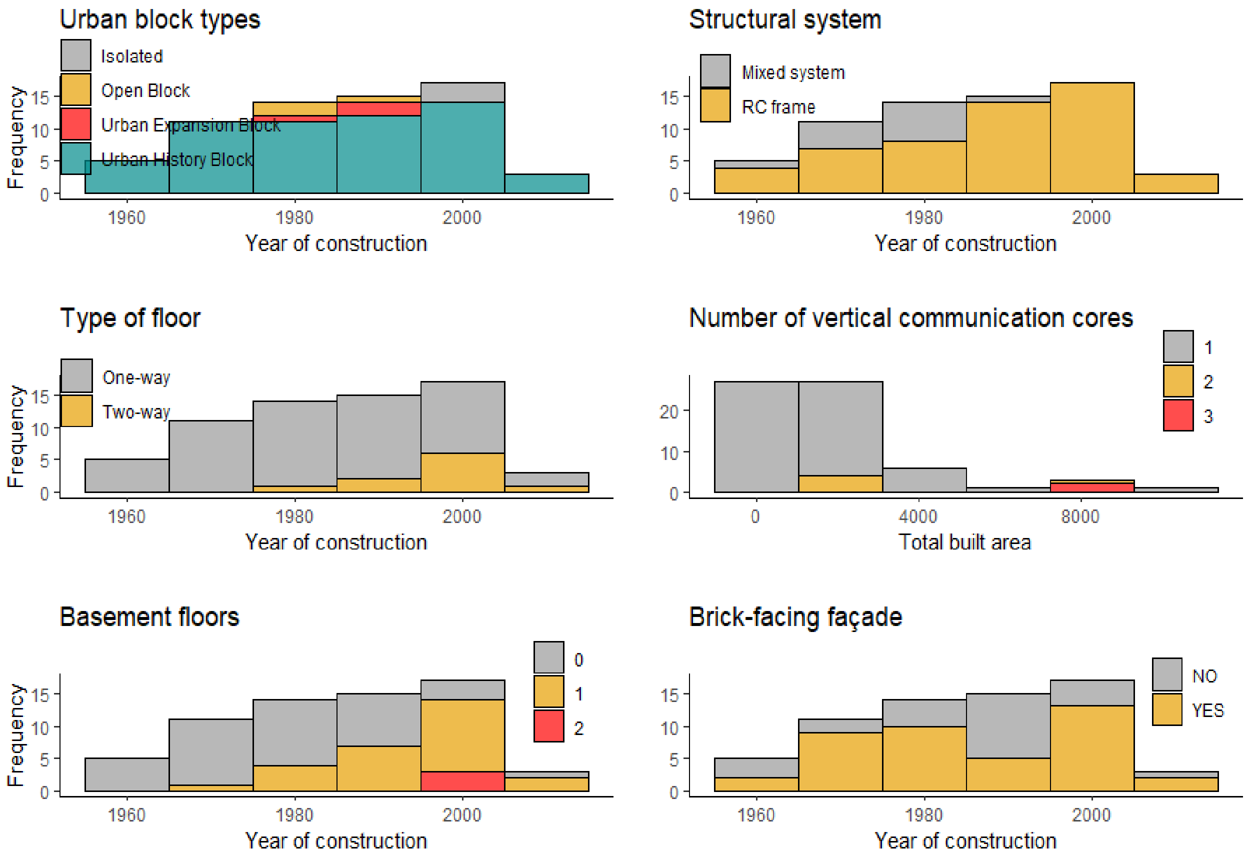

Fig. 1 – Urban, design and structural features of the four to six-storey RC buildings and features of their NSE

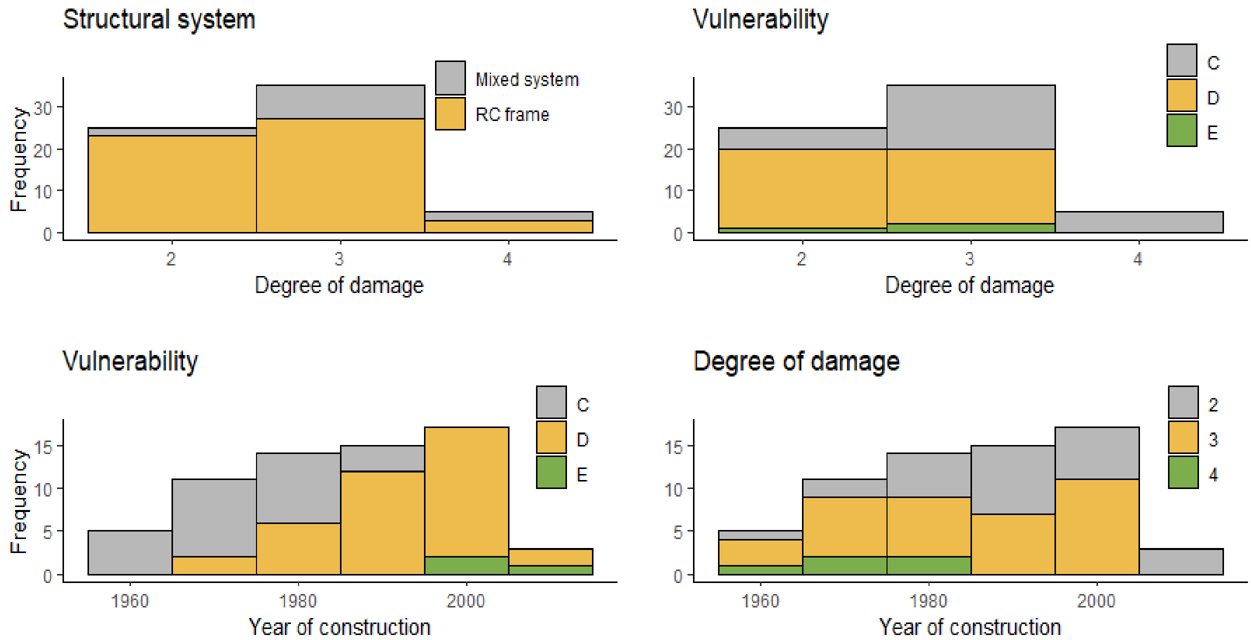

Fig. 2 – Vulnerability and degree of damage of the four to six-storey RC residential buildings

4.2 Damages identified and non-structural elements potentially more vulnerable

The most common damages to non-structural elements pertaining to this type of building during the Lorca earthquake were the following:

-Damages to unreinforced masonry infill walls located on vertical communication cores: a significant number of buildings presented damages to these elements. The main consequence associated with the damage to these elements is that they prevent the users of the building from being evacuated safely.

Fig. 3 – Damages to unreinforced masonry infill walls located on vertical communication cores

-Damages to non-structural elements located in the common areas of the ground floor: in the same vein as the damages to elements pertaining to the vertical communication cores, damages to these elements compromise the safety of the evacuations and in the cases of infill walls where cables and pipes run, their failure can prevent the proper functioning of the electricity and water installations. Suspended ceilings have also proved to be extremely vulnerable and hazardous elements. The majority of them show fissures and cracks after the seism, and a considerable number of them present detachments and local or global collapses.

Fig. 4 – Damages to non-structural elements located in the common areas of the ground floor

-Damages to interior and exterior claddings panels: the majority of the connections of the cladding panels to the exterior and interior walls were executed without any type of mechanical fixation (neither screws nor nails), trusting their adherence to the binding material, for example, cement. A type of connection that has demonstrated to be insufficient in the case of the Lorca earthquake. In some cases, the reparation projects prescribed the complete removal of the cladding panels and their substitution with a different type of panel, and in other cases, they prescribed an additional mechanical fixation for them.

Fig. 5 – Damages to interior and exterior claddings panels

-Damages to exterior infill walls (interior and exterior façades): on the one hand, an important number of façades has shown a lack of firm connection between the infill masonry walls and the structural elements. In these cases, not only the detached parts represented a risk to life, but also the remaining parts. Hence, the whole element has to be removed or fixed in a better way. On the other hand, these exterior IMWs are composed by two single-leaf brick masonry walls. However, just a tiny part of them includes elements of connection between both leaves, such as steel ties; which constitute a widespread and economical improvement to their resistance against lateral forces.

Fig. 6 – Damages to exterior infill walls (interior and exterior façades)

-Damages to interior infill walls: the most hazardous damages to interior infill walls have been found on the ground floor, where habitually commercial units can be found. Nevertheless, the reparation of fissures and cracks provoked by exceeding internal shear forces on interior walls and the restitution of the detached ceramic tiles in bathrooms constituted a significant part of the total budget of the reparation.

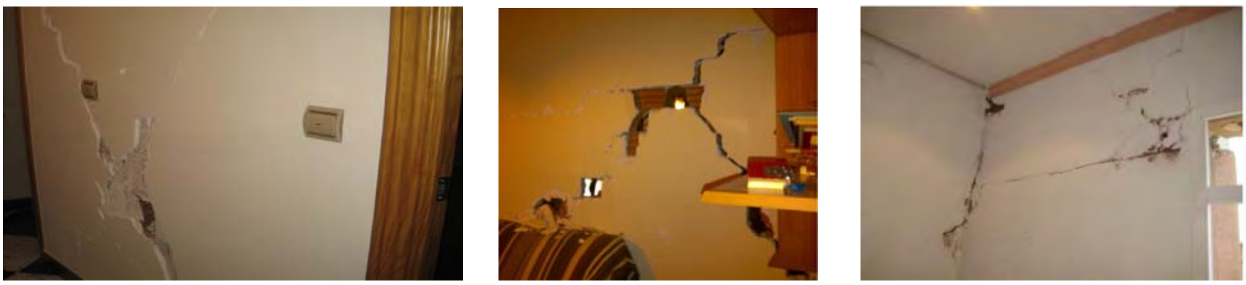

Fig. 7 – Damages to interior infill walls

-Damages to unreinforced masonry roof parapets: these elements have proved to be the most hazardous components of the buildings due to being the cause of the majority of the fatalities. Among the buildings studied, authors have found several complete collapses. Moreover, they present clay or stone coping pieces that, in the majority of the cases, are not firmly fixed to the parapet; resulting in an added risk. Additionally, some ledges have proved to be unable to withstand the Lorca earthquake.

Fig. 8 – Damages to unreinforced masonry roof parapets

-Damages to unreinforced masonry roof chimneys: a great deal of these elements, which are made of unreinforced masonry, have resulted in collapse or in such a state of degradation that had to be demolished.

Fig. 9 – Damages to unreinforced masonry roof chimneys

5. Discussion

The following patterns have been observed in the previous sections:

-From the urban point of view, the majority of these buildings constitute urban history blocks.

-From a structural point of view, the data documented by the experts show that from the 70s to the middle of the 80s, there was a strong tendency of employing a mixed structural system in construction, which consisted of steel beams and reinforced concrete columns. A structural system that none of the buildings constructed in the XXI century presents. Moreover, figure 1 shows an increasing propensity towards the use of two-way joist slab floor systems, facilitating this system to become as widespread as the one-way joist floor systems in the case of the most recent buildings. Additionally, throughout the development of this study, authors have verified that the presence of conventional RC beams is not significant, since the vast majority of the buildings present wide RC beams. Lastly, the number of basement floors has increased throughout time. At the time of the seism, the majority of the buildings presented one basement floor. Additionally, it has been observed that the majority of these buildings present a unique vertical communication core, even when they comprise a large area. Moreover, in quite a number of these cases, this influential element is localised at a considerable distance from the centre of mass of the building. A fact that represents an irregularity in terms of stiffness and may lead to the emergence of global torsions. Moreover, it is commonly agreed that the Spanish vertical communication cores possess greater stiffness than the rest of the structure. Hence, consequently, a significant number of them have resulted in being heavily damaged (their structural elements as well as their non-structural elements).

-In relation to the design aspects, according to reports, 63 % of the buildings have outer walls made of facing bricks, representing the majority since the 60s. A tendency that has shown a notable increase since the year 2000.

-In relation to the general degree of damage observed, figure 2 shows that the oldest buildings have been classified as more vulnerable to seismic forces (vulnerability types C and D according to EMS-98), and that they have experienced major damages after the event. All the buildings built before the 70s have suffered damages of a degree equal to or greater than 3 and that none of the buildings constructed in the last quarter of the century has suffered grade 4 damage. Furthermore, the buildings of type C vulnerability are the only ones that have reached grade 4 damage. This decrease in the experimental and theoretical vulnerability may mean that the construction techniques have improved or that the latest regulations have contributed to reducing substantially the vulnerability of the buildings, as they aimed to.

-In relation to other NSEs, the damages illustrated in figures 3 to 9 indicates that the construction techniques employed in the execution of the non-structural unreinforced masonry elements, cladding panel and suspended ceilings are inadequate in seismic-prone areas. To cite some instances, in the case of parapet masonry walls, the majority of them have been constructed as a half-brick single-leaf wall, which implies that their slenderness is excessive. The slenderness of a free-standing element of a determinate size is directly correlated with its predisposition to overturn. In the cases of the IMWs, it is worth noting that a substantial part of the reports mentions a generalised disconnection of the façade walls to the structure of the buildings, which constitute a defect in the construction techniques.

Future lines of investigation should develop a more in-depth statistical study of the damages to non-structural elements, in order to assess the evolution of their construction techniques.

6. Models

6.1 Buildings modelled

From all the four to six-storey RC residential buildings, two have been selected and modelled for a number of reasons: being representative of their type and a great deal of information being available about their materials, dimensions, original and reparation plans, photographs and definition of the damages suffered during the earthquake. Both case studies share several features: both are five-storey buildings with an RC frame structure. Furthermore, both show less stiffness in their ground-floors, which is a common deficiency detected in multi-storey family housing with commercial units on their ground floors. The lack of interior and exterior infill walls implies less stiffness in such floors. However, they also present significant differences in their structural features, such as age, geometry, column dimensions and positions, which allow the authors to evaluate and compare results.

Structural models of both buildings were recreated using the Software SAP2000 and according to the following criteria. Columns and beams were modelled as 1D unconfined RC frames with concentrated plasticity at their extremes (modelling parameters taken from [10]). Floors were modelled as 2D finite elements that behave as semi-rigid diaphragms. In addition, the authors have followed the criteria established by the contemporary regulations at the time of construction for the determination of their ultimate and service loads and steel reinforcement requirements of each RC element.

-Building A is representative of the 1970s residential type, and was designed according to outdated Spanish building regulations, [11] and [8], and is located in the corner of an urban expansion block with a central courtyard in the neighbourhood of La Viña. It is considered to be a vulnerability type D building and to have a medium level of seismic design. After the earthquake, it presented a Type 2 level of damage according to EMS-1998. Specifically, it showed slight generalised damage to ground floor and basement floor columns, and moderate damage to IMWs on all floors, particularly noticeable to the interior elements. The compressive strength of the concrete of this building is 17.5 MPa (cylindrical specimens) and its steel reinforcement is comprised of reinforcing bars of the type B-400S steel.

-Building B is located within an urban history block in the neighbourhood of San Diego, it was built in the 2000s and designed according to contemporary regulations [3] and [9]. This building presented type 3 damages according to EMS-1998. Specifically, it showed moderate structural damage manifested as cracks to several ground floor columns, and severe non-structural damage, manifested as several exterior and interior IMWs collapsed on the ground floor as well as cracks and fissures caused by internal shear forces on the IMWs of the ground and the first floors. Additionally, the first-floor north-facing façade presented a severe risk of collapse caused by pounding with the adjacent building, severe damages to the roof chimneys that led to their enforced demolition and the localised collapse of an unreinforced masonry parapet. Moreover, it presented a distinct soft storey ground floor. The compressive strength of the concrete of this building is 25 MPa (cylindrical specimens) and its steel reinforcement is of the type B-400S.

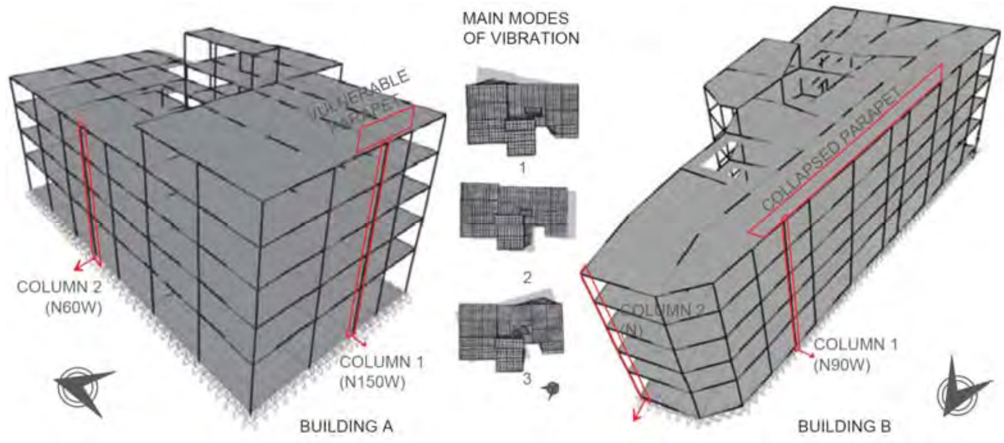

Fig. 10 – Models of Building A and Building B

Modal analysis of both models was carried out considering real inertia to be 125% of RC gross inertia (1.25 EI), in order to take into account a more than likely increase in the global stiffness of the buildings due to the presence of staircases and masonry infill walls. It reveals that the main modes of vibration of both buildings are 0,9 s, 0.8 s and 0.7 s (T_1, T_2 and T_3 respectively) when considering the secant stiffness.

Small differences in terms of fundamental natural periods have been found among these buildings. It is worth noting that the analysis of Building A shows that its most flexible modes of vibration are concentrated within these two ranges of periods: the fundamental one, between 1 and 0.8 s (from T_1 to T_3) and the second one, between 0.3 and 0.2s (from T_4 to T_8), which involve little mass, but that may be relevant as they are similar to the period of maximum acceleration during the Lorca event. The total weight in the seismic load combination was around 24300 kN for Building A and 32350 kN for Building B. Moreover, the modal results show that a global torsion is likely to occur due to the asymmetry of the buildings and to the eccentricity of their centres of mass in relation to their centres of rotation.

The accelerograms applied as inputs at the base of the buildings are the synthetic accelerograms elaborated by [12] at different points in the city of Lorca. The accelerograms corresponding to the nearest point to the locations of the buildings have been applied for each building. In order to recreate the actual seismic event as realistically as possible, the inputs in the two orthogonal horizontal directions have been rotated in line with the actual direction of the seism and applied.

6.2 Results in terms of floor response spectra and discussion

The floor response spectra (FRS) studied in this subsection include the comparison between linear and non-linear behaviour variations in terms of acceleration that represent the existence of a more flexible ground floor, and transformations floor to floor. 5% of damping of the NSEs has been accounted for the determination of the FRS. The predictions made by the simplified method of [1] were also analysed. These simplified European predictions describe the hazard by means of a single parameter, the value of the peak ground acceleration- agr - in the emplacement of reference. In these cases, the peak ground acceleration values have been taken from the new proposal for the Spanish hazard map [13] which constitutes an improvement over the previous one included in [2].

In both buildings, the selection of the points where the FRS are studied was based on a number of parameters related to the vulnerability of their non-structural elements, which depends on its position, elevation, length, distance to the centre of torsion, etc.

Figure 11 (graphs a) and b)) highlights the strong influence of the motion directionality in the seismic response of NSE, by contrasting the FRS in orthogonal directions of Building A. The results show that during the Lorca earthquake, directionality is especially relevant for short-period elements, whereas for intermediate and long-period elements the peak floor accelerations (PFA) are similar: 1.7 g for Linear Analysis and 0.9 g for Non-Linear Analysis).

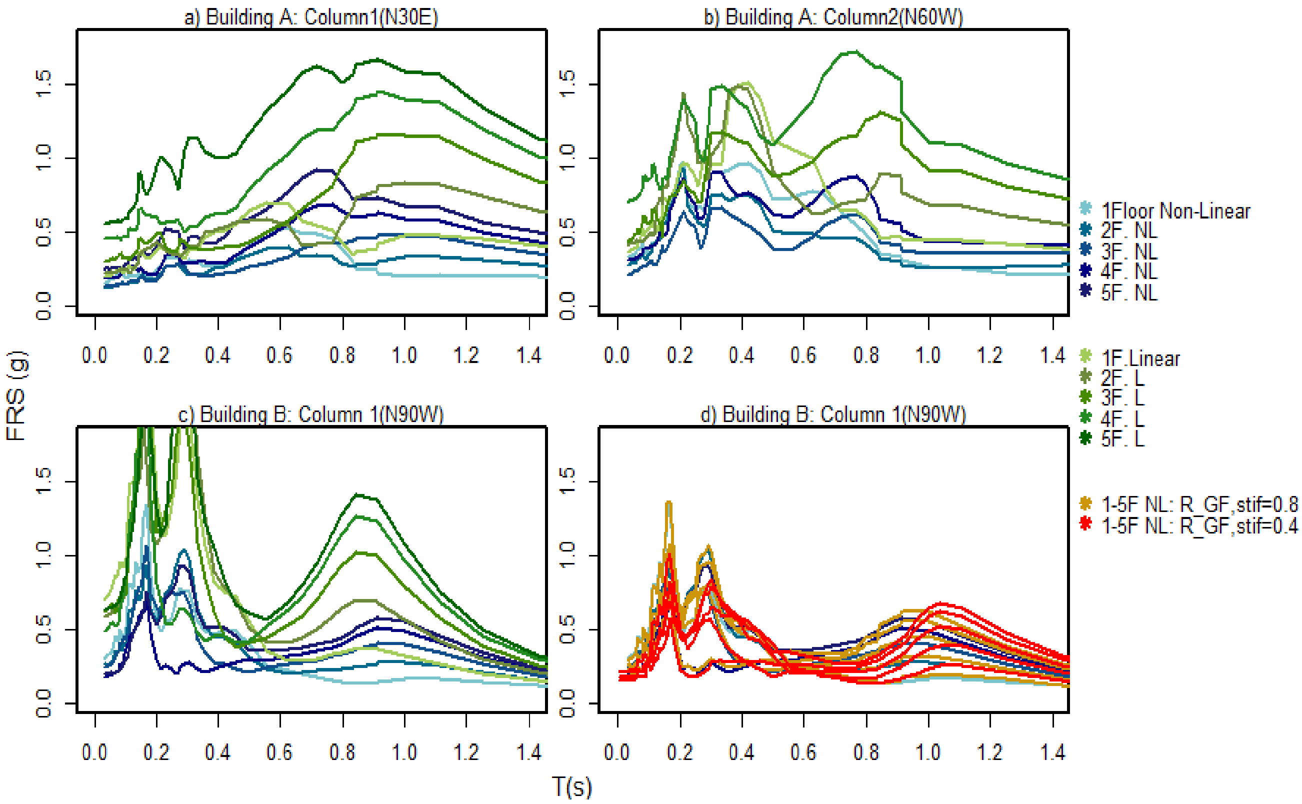

Fig. 11 – a) and b) Comparison between linear and non-Linear FRS in two points of Building A for two different directions of motion c) and d) Comparison of FRS of three different Ground Floor Stiffness Ratios $R_{GF,stif}$ = 1, 0.8 and 0.4.

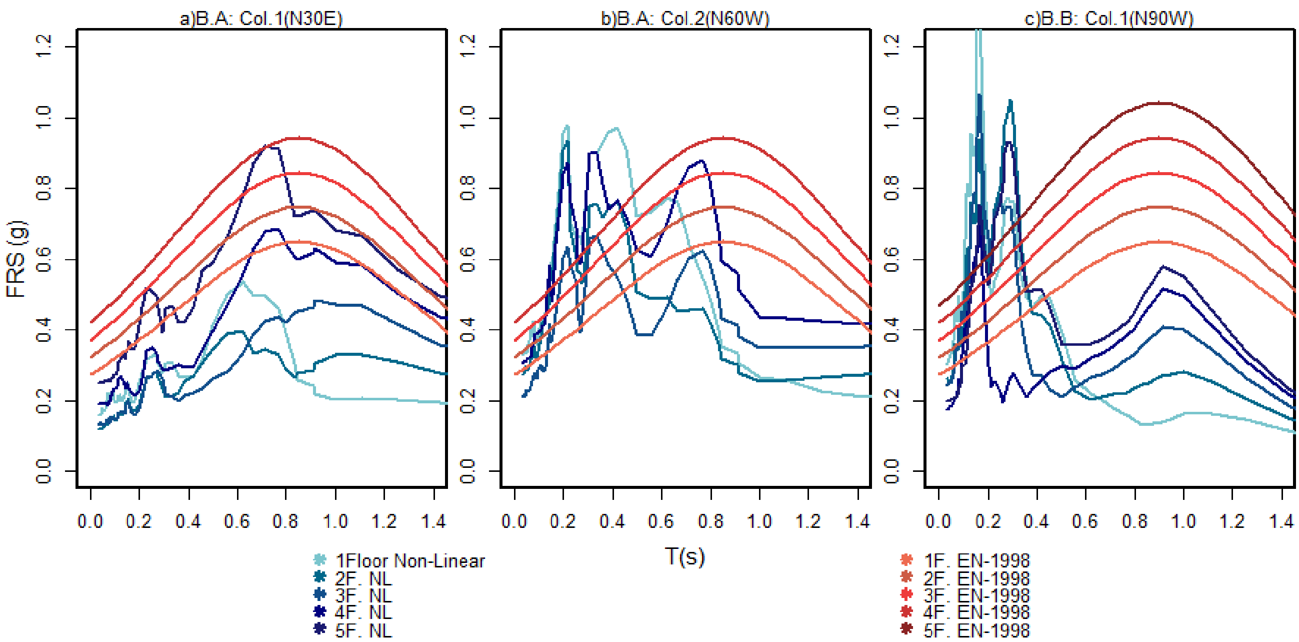

Fig. 12 – Comparison between non-Linear and EN-1998 FRS in two points of Building A for two different directions of motion

Additionally, in the same figure, graph c) contrasts the Linear and Non-Linear FRS of Building B, showing again the same substantial differences in the range of intermediate and long-periods (Linear Peak Floor Acceleration is more than twice the value of the non-linear acceleration).

Moreover, due to the characteristics of this particular seismic event, (high short-period content), the FRS of both buildings exhibited a change in tendency (increased and decreased) in terms of acceleration in the short-period content of the FRS of some floors. This situation may be considered as a localised effect derived from the existence of modes of vibration in that range of periods, even when they involve little mass, thereby stressing the importance of considering such modes of vibration as a modifying factor of the FRS.

Finally, the possibility of a soft storey was computed. This phenomenon is defined by the stiffness of the lateral force-resisting system in any storey being less than 70% of the stiffness of an adjacent storey (above or below) or less than 80% of the average stiffness of the three storeys (above or below) [14]. The soft storey is quantified by means of the relative storey stiffness and can be compared by considering different elastic modulus, moments of inertia or column lengths [15]. In this study, the moment of inertia of the structural elements located in the ground floor has been considered to be the most appropiate parameter of control for this phenomenon in Building B ($R_{GF,stif}$).

In graph 11 d), a comparison between the FRS of Building B considering different degrees of ground-floor stiffness in relation to the upper floors is made ($R_{GF,stif} = 1$, $R_{GF,stif} = 0.8$ and $R_{GF,stif} = 0.4$) allowing the authors to conclude that although this difference increases the acceleration slightly, it is not particularly relevant in global terms. However, it is commonly agreed that this phenomenon induces a crucial localised concentration of inter-storey drift.

The comparison between EN-1998 [1] predictions and the non-linear Time History Analysis FRS predictions in the same three points previously analysed (Building A: Columns 1 and 2 and Building B: Column 1) shows that, for this type of buildings and seismic spectra, there is a considerable lack of accuracy in the predictions of this simplified method. For the intermediate and long-period elements, the simplified predictions can be considered to be significantly overestimated for Building B, whereas the predictions are extremely consistant for the highest floors of Building A.

The predictions also underestimate FRS acceleration for short periods, where this situation can represent a risk to safety for rigid NSEs such as unreinforced masonry chimneys. In these buildings in particular, chimneys suffered significant damages: in Building A some chimneys resulted in partial collapse causing falling debris, and, in Building B several chimneys had to be demolished. This underestimation could be due to the fact that this particular earthquake presented higher acceleration in short-periods, in contrast to the fundamental periods of the building; and EN-1998 [1] predictions amplify mainly the accelerations in periods similar to those of the building.

7. Conclusions

Throughout the study, the main urban, structural and design features of the type that caused a major number of fatalities during the Lorca earthquake have been characterised. It has been ascertained that a prime example of the four to six-storey residential RC type would be typified by a building located in an urban history block, would have a RC framed structural system with one-way joist floor system and wide beams and would have one basement storey used as parking. Moreover, it would present solely one vertical communication core, a facing brick façade composed by two single-leaf layers separated by an air gap, single-leaf masonry interior infill walls, masonry parapets and several masonry chimneys.

The most hazardous NSEs of this building type would be: unreinforced masonry parapets and façade walls, infill walls located on staircases and suspended ceilings. Although others, like masonry chimneys, could be considered as more vulnerable. In relation to parapets walls, the importance of slenderness has been stressed: in spite of being the most widely used type of parapet, a 12 cm-thick single-leaf wall is inadequate in seismic-prone areas. Additionally, in relation to cladding panels and brick façades, the experience has demonstrated that a firm connection is crucial, and a mechanical fixation has been recommended.

The research findings of this study have provided some evidence that, for the analysed typology, it is crucial to consider the entrances to the structure in the non-linear range when characterising FRS. Furthermore, the results from the Lorca case study illustrate that [1] FRS proposal underestimates the actual demand for some frequencies, even after considering the increment of the peak ground acceleration that introduces the new proposed maps [13]. Moreover, the FRS obtained highlights the relevance of the modes of vibration that involve less mass. Future lines of research should include the modelisation of the IMWs.

9. Acknowledgements

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