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Seismic assessment of low ductile RC structures: buildings from before the modern seismic codes

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Abstract

Purpose – When a numerous amount of buildings was built in reinforced concrete, in a period when the regulations did not have the design philosophy for the occurrence of earthquakes, it is of extreme importance to carry out full and effective structural assessments, specially considering and comparing bare frame and infilled structure. The paper aims to discuss these issues.

Design/methodology/approach – Among several possibilities to make the evaluation as, simplified, linear analysis and static non-linear analysis, the non-linear dynamic can provide the most accurate numerical behaviour compared to the real one. The time-history non-linear analyses are developed on the software SeismoStruct for different levels of intensity. Local verifications are then applied separately from both Eurocode and Italian Code.

Findings – The application of validated models for the analysis of real buildings allows a complete seismic assessment. The level of uncertainty increases integrating particularities regarding the infill masonry walls. The paper shows important global and local seismic safety for these complex typology of buildings.

Originality/value – A representative common concrete structure without seismic provisions is first analysed and discussed in terms of global behaviour, deformations and progression of forces. The case study structure is considered both as bare structure and with integrated infill panels. It is also discussed in a local level, about brittle and ductile mechanisms, and extra comparisons between different interpretations of different standards. The case study structure is considered both as bare structure and with integrated infill panels.

Keywords 3D modelling, Dynamic and non-linear analysis, Infill masonry, RC building, Seismic assessment

Paper type Case study

1. Introduction

Accounting for seismic action in the design and construction of buildings has always been a delicate matter in engineering. The unpredictable nature of these natural phenomena, both in terms of occurrence and intensity, has posed tough challenges. There are several large-scale earthquakes reported along the history of numerous regions of the country. In Portugal, earthquakes such as the one in 1755 that devastated Lisbon raise particular concern. Even if seismic activity in Portugal is not particularly constant or intense, the risks for human and economic losses are high (Silva, 2013).

In spite of some areas of the world being more exposed to these actions, there is still a lot of research on the development of procedures to protect human lives and material goods. With the elaboration of recent technical codes for this purpose, such as the

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European Design Codes, Eurocode 2 (CEN, 2004) and Eurocode 8 (CEN, 2003), the Italian Code (NTC08, 2008), or, for example, older codes such as the Portuguese Design Code RSA (RSA, 1983), a lot of research has been made to design new efficient structures towards seismic actions. Nowadays, all over Europe, a large part of the building stock has not been designed according to modern seismic engineering principles.

With the changing of philosophy towards seismic actions, structures became successively more ductile (Ricci, 2010) compared with the old ones, which have a clear different behaviour. The main vulnerabilities of these buildings – apart from material quality – to sustain seismic actions were mainly connected to constructive deficiencies of stirrups and hoops; inadequate shear capacity of the joints; no method/strategies to avoid formation of mechanisms such as strong-beam-weak-column and short-column; and often had large irregularities in plan and in elevation from both structural and non-structural elements, which can lead to torsion effects or soft-storey mechanisms.

The main objective and contribution of this work was the perform of a complete assessment – usually studies are limited to global or local levels – of a representative pre-1970s structure, with non-linear analysis applying a time-history seismic action, and with full 3D model. The conclusions may serve as careful description of the risks involved in these structures, not only in Portugal but in other countries where the typology is similar.

2. Study case description

The building used to perform the analysis was built in Portugal during the 1950s. It is representative of the type of construction in concrete, until the later design codes had been applied as RSCCS (1958), RSEP (1961) or RSA (1983) that began to include seismic recommendations for designing. Thus, the study case is an example of a typical building that were not designed for earthquake demands, in terms of characteristics and vulnerabilities. According to some statistics available in INE (2013), an average of 170,000 of the current 900,000 (19 per cent) buildings dated before 1960, which survived until today, were built of reinforced concrete with the vulnerabilities already pointed. The vulnerability of this kind of structure assumes a greater potential since most of the buildings with more than two storeys are RC framed, and the tallest buildings are located in a high percentage in Lisbon, which is an area with a high seismic risk (Silva, 2013).

The building was designed in 1954 and built in 1955. It has a rectangular implantation of 26.20 per 9.90 square metres and 18 metres of height. Half of the top columns have indirect supports on beams instead continuity to the foundations, as identified in Figure 1(b). (Structure identified on Figure 1 with some frame dimensions. Complete details in Dias-Oliveira (2013).)

The vertical access of the building is made by exterior stairs, located on a framed structure separated from the building, only linked through simply supported beams. This extra block is formed by a concrete wall in all its height on one of the sides, linked by beams and columns to the structure. The stairs are independent from the structure because they are unlinked. It also has an inside escalator unlinked from the structure, with an old system of pushing it up directly from the ground.

Only the confined infill walls were considered for the modelling. It has an exterior wall almost in its entire perimeter and another two interior walls from the second to the fourth storey. (Visual location for the infill panels are addressed on Figure 1(a). For more details consider Dias-Oliveira (2013).)

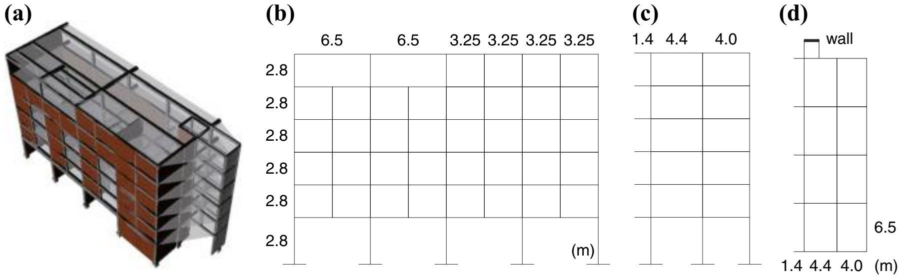


Figure 1. Model on SeismoStruct

Notes: (a) With infill masonry walls. Frame scheme with dimensions for (b) longitudinal direction (c) transversal direction and in (d) plan

3. Modelling and assumptions

3.1 Modelling

The dynamic analysis were performed with the software SeismoStruct (SeismoSoft, 2012). The elements were defined through a force-based formulation, distributing the inelasticity on a finite length hinge zone close to the fixed length of the element, where the non-linear behaviour is formulated as a plastic hinge. This formulation has its advantage on the amount of time consumption since it performs the fibre integration on the two fixed-end parts of the element, providing more control of the plastic hinge length ("spread of inelasticity").

3.1.1 Material properties. The existing materials' properties were considered according to the design project, without extraction of samples. The concrete behaviour was formulated with a model developed by Mander and Martinez (Mander *et al.*, 1988; Martnez-Rueda and Elnashai, 1997), and the steel by a model by Menegotto and Filippou (Menegotto and Pinto, 1973; Filippou *et al.*, 1983).

Eurocode 8 CEN (2005) gives specific factors (named knowledge levels) to reduce the designing values depending on how well the material properties, geometry and details are known. No *in-situ* tests were performed and neither a full information regarding the details were available. That leads to a decreased of the level of knowledge, however the work is intended to analyse the building with strong excitation demands, thus it was decided not to reduce the strength of the materials as it would be done for a full safety assessment project.

The concrete has a characteristic compressive strength (f_{ck}) of 16 MPa. The design strength, calculated by Eurocode 2 (CEN, 2004) is $f_{cm}=f_{ck}+6=22$ MPa. The tensile strength (f_{ct}) is 1.9 MPa and the strain at peak stress (ε_c) is 0.002 m/m. Two slightly different concrete materials were defined for all sections regarding confinement of the core and cover. Using Mander *et al.*'s (1988) recommendations and the available technical drawings of the amount of stirrups, the result for confinement factor was taken as 1.005. This value could eventually be greater, but there is no information on how the hoops were performed to decrease the conservativeness of it. Therefore, the concrete was considered to have a less ductile behaviour to avoid potentially inaccurate results. A concrete cover of 30 mm was used.

The applied steel was smooth bars with yield strength (f_{syk}) of 235 MPa, elastic modulus (E_{sm}) of 200 GPa and strain hardening parameter, (μ) of 0.005, a conservative value. The rest of the defined parameters are the recommended ones, namely the transition curve of the initial shape (R_0) which is defined as 20, and the used fracture/buckling strain (ε_{ult}) was 0.2, which is a conservative value for this kind of steel, hot rolled, with low strength and high ductility. Tensile tests on that type of steel show an average extension of 24 per cent. The yield strength which was used could have been increased to at least 300 MPa, and buckling strain to 24 per cent, but due to uncertainties regarding the slippage[1], and because the steel has less impact on the global response compared to the concrete (until the initiation cracking phase), it was decided to integrate more conservative values.

3.1.2 Loads and masses. The assigned loads were calculated according to the load combination of Eurocode 8 (CEN, 2003), and load values taken from technical tables, design information and previous studies (Freitas, 2008). The final used loads for both buildings for permanent and variable loads are defined according to Table I.

3.1.3 Structural elements. The elements were modelled according to a force-based formulation with concentrated inelasticity within a fixed-end length of the elements. The plastic hinge lengths were defined according to conclusions taken from (Varum, 2003). A proposed formulation by Paulay and Priestley (1992) was used for typical beam and column proportions and smooth bars, where the effective plastic hinge length can be estimated as approximately 25 per cent of the height of the beams and 25 per cent of the height of the strong-axis on columns, counting from the face of the adjacent element.

The exhaustive information of the cross-sections can be found in Milheiro *et al.* (2008). As generalized view, the building has square columns varying dimensions of about 0.2/0.3/0.4 m, and rectangular beams varying dimensions from 0.25/0.35 m (width) to 0.4/0.8 m (height; example on Figure 1(d)).

3.1.4 RC frames conditionals. The exterior block, where the vertical access stairs are placed, has a concrete wall on extremity. It was modelled with 20 cm thickness and 2.5 metres wide along the building height, with constructive reinforced steel bars.

The exterior and interior stairs were not modelled, since they unattached, causing no increase of stiffness on the building. Thus, no extra elements were modelled, and the respective zone was considered as being a hole with the correspondent permanent loads, supported by the beams where they are resting.

The slabs were built of hollow bricks with 17 cm height with a RC compression depth of 5 cm. They were modelled as rigid diaphragm constraints, restricting degrees of freedom on the horizontal plane, linking the nodes from each floor, unifying its behaviour. The modelling strategy was adopted since all slabs are limited by beams, and supported on the Eurocode's suggestion of modelling solid slabs with 7 cm of thickness with rigid diaphragms.

3.1.5 Infill panels. The infill panels were modelled by Crisafulli's (1997) model. The definition of the properties on the modelling, used a similar procedure/properties to Smyrou (2006) infill panel work for the framework of the ICONS research programme (Carvalho *et al.*, 1999). It is assumed that the used infills are representative of the ones

Table I. Permanent loads, overloads and reduction factors

	Weight (kN/m^2)	Location	Q (kN/m^2)	ψ_2 (EC0)
Waffle slab (storeys)	3.18	Habitation	2.0	0.2
Thick slab (banconies)	2.88	Balcony	2.0	0.2
Finishing	0.60	Roof	0.3	0.0
Interior walls	1.00			

also applied on the construction in Portugal. Some parameters and intervals are available on Table II, and the complete location of the infills can be found on (Dias-Oliveira, 2013). The assumed thickness for the infill walls was 0.15 m.

3.2 Earthquake loading

Artificial accelerograms implemented on the modelling were created for a medium/high risk for Europe used on the ICONS programme. The implemented accelerograms have a return period (RP) of 73, 170, 475 (demonstrated in Figure 2(a)), 975 and 2,000 years. The used scenarios are in accordance to Portugal's risk, in which the assessed buildings are located. The loadings were applied in both transversal and longitudinal directions of the buildings. The Figure 2(b) shows the peak ground motion of the used RPs.

4. Response and assessment at the global level

The global analysis of a building serves the purpose of understanding how the structure behaves. Knowing more generalized information about it, it is possible to expose some deficiencies, such as the tendency for the formation of soft-storey and other mechanisms, torsion effects, evaluation of weaknesses by floor or direction, distribution of loads, deformations, etc.

4.1 Calibration of natural frequencies

To get an accurate behaviour of the model, it is important to calibrate with the experimental frequencies taken from the building. With them, it is possible to have a notion if there is not a significant error on the modelling, reducing the discrepancy between the virtual and real structure. The data were extracted from the experimental results available in Milheiro's *et al.* (2008) work is available on the Table III.

The calibration took as an assumption the reduction of stiffness on some panels, due to some conditions. The main façades of the building have openings for windows and balcony doors, which naturally reduces the stiffness of the panels. For the respective panels, a reduction of 50 per cent on the compressive strength was adopted as a simplified ratio. This ratio has shown to be accurate also for other performed calibrations (Dias-Oliveira, 2013). The reduction of the compressive strength has effects on the elastic modulus and on the vertical separation between struts, which were accounted. On the rest of the infills, as for the panels without openings on the façades, and as for the interior walls, the properties obtained from the results of the empirical proposed formulas were maintained. Table III shows the final errors between the experimental natural frequencies and the ones computed by the modelling. Since usually the first modes are the most important in demand and response, the calibration was considered satisfactory.

Strut curve		*Shear curve*	
$E_m = (1.3,5.1)$ GPa	Initial young modulus	$\tau_0 = 700$ kPa	Shear bond
$f_m = (1.3,5.1)$ MPa	Compressive strength	$\mu = 0.7$	Friction coefficient
$f_t = 0.5$ MPa	Tensile strength	$\tau_{max} = 1{,}200$ kPa	Maximum strength
Other parameters			
$A_1 = (0.15,0.35)\ \mathrm{m}^2$	Strut area 1	$A_2 = 40\% A_1\ \mathrm{m}^2$	Strut area 1
d = 5%	Out-plane failure Drift	$x_0 = (3,10)\%\ L_{panel}$	Horizontal offset
$h_z = (16,24)\%\ h_{panel}$	Equivalent contact length	$y_0 = (4.5,6)\%\ h_{panel}$	Vertical offset

Table II. Some parameters on modelling of the infill panels

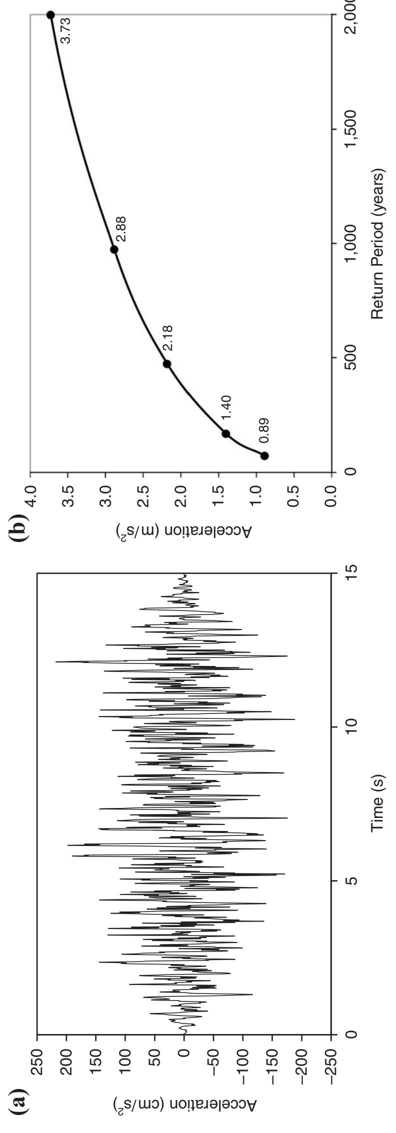


Figure 2.
Earthquake's (a) ICONS's accelerogram for a return period of 475 years and (b) hazard curves for the moderate-high European scenario

4.2 Modal shapes

The participations for the six first shape modes are summarized in Table IV.

The results show that the bare framed structure is more flexible in the transversal direction even with the participation of the concrete wall. Although, the first mode has a participation of 77 per cent in transversal translations, the wall, due to its stiff nature decentralized from the centre of mass of the building, create torsion effects measured by a participation of 7 per cent. It is interesting to note that due to the slender architecture, this mode has also a participation of 13 per cent in rotational mass, showing less (global) transversal flexural rigidity. The second mode has mainly a translation participation of 88 per cent on the longitudinal direction with a lot less rotation participation compared with the first mode. The third mode has a torsion shape, with 70 per cent of mass participation of the rotation on the vertical axis.

Regarding the incorporation of the infills on the model, it has an increase of 2.5 times of the frequencies, increasing them to values between 3.0 and 3.7 Hz for the first two modes. It is interesting to verify that the integration of infills create a flip on the modes: the first with infills has a mass participation of 91 per cent of longitudinal translations and the second mode has 70 per cent of translation on the transversal direction. Due to the architecture, the translations on this direction take a key role, so it continues to be verified that there is a torsion effect, with mass participation of 11 per cent of rotational on the vertical axis, later shown due to the infill panels instead of the concrete wall. The third mode has its correspondent frequency increased, due to the global increase of stiffness, but no big change of rotation mass participation is verified. Despite this, the translation on transverse has a higher participation, increased from 4 to 12 per cent.

The modifications of frequencies, mass participation and flip on the mode shapes, illustrates the great importance of considering the infills on the modelling.

4.3 Displacement profiles and drifts

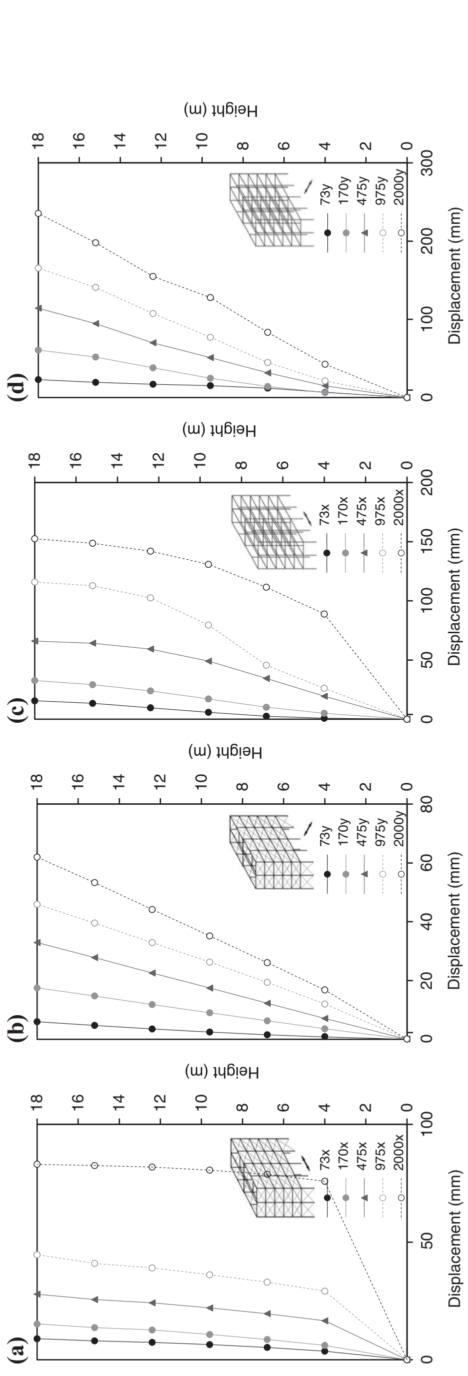
Some of the deformations, by earthquake, and drift progressions, for a RP of 475 years, can be checked on Figures 3 and 4. The Figure 4 plot the temporal instant in which the highest top displacement occur.

Table III. Experimental frequencies and numerical errors (of infilled structure)

Experimental		Error (%)	Experimental		Error (%)
Mode 1	2.178 Hz	−3.0	Mode 4	4.000 Hz	31.1
Mode 2	2.343 Hz	3.3	Mode 5	4.469 Hz	28.5
Mode 3	2.999 Hz	1.0	Mode 6	6.002 Hz	14.3

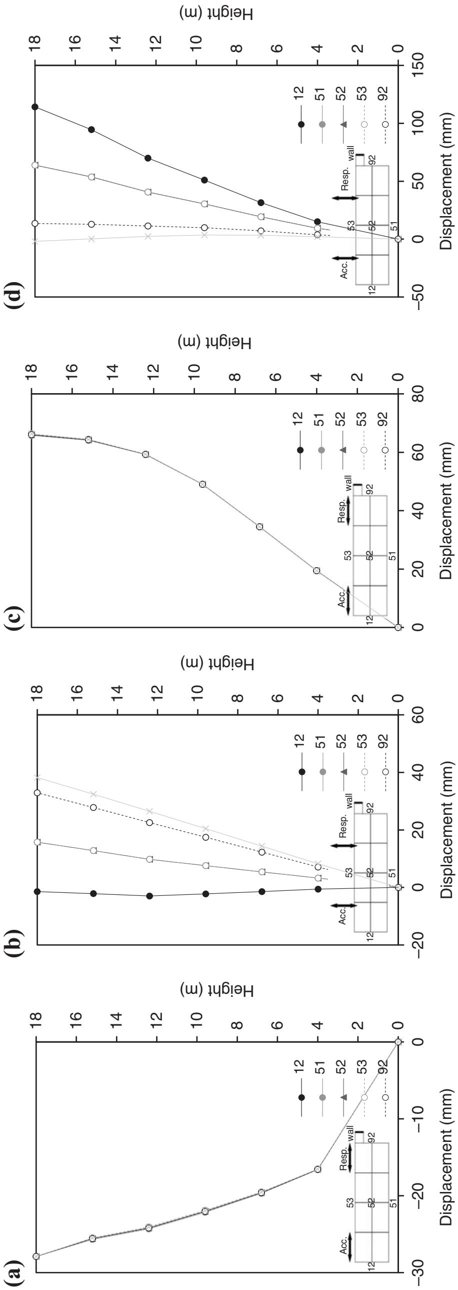
Table IV. Frequencies comparison with and without infill panels

	Frequency (Hz)		Change		Participation				
	Infill	No Inf.	Hz		Infill		No Inf.		
Mode 1	2.96	1.25	1.71	1.36	91.88%	(Ux)	76.52%	(Uy)	Switch
Mode 2	3.70	1.36	2.34	1.72	68.31%	(Uy)	88.40%	(Ux)	Switch
Mode 3	4.76	1.73	3.03	1.75	70.59%	(Rz)	70.26%	(Rz)	0.34%
Mode 4	6.28	3.50	2.78	0.80	0.11%	(Uz)	36.98%	(Rx)	Switch
Mode 5	7.14	3.98	3.16	0.79	0.01%	(Ry)	20.44%	(Ry)	−20.43%
Mode 6	7.33	5.96	1.36	0.23	0.04%	(Uz)	4.38%	(Rx)	Switch



Notes: (a) Longitudinal and (b) transversal earthquakes with infill panels, and (c) longitudinal and (d) transversal earthquakes without infill panels

Figure 3.
Lateral displacement profile for maximum top displacement



Notes: (a) Longitudinal earthquake and response, and (b) transversal earthquake and response; and without infill panels for (c) longitudinal earthquake and response, and (d) transversal earthquake and response

Figure 4.
Displacements of different locations of columns with infill panels for the earthquake with a return period of 475 years

4.3.1 Displacements with infill panels. The displacement profiles show that the building with the infill panels integrated presents regular translations in its height for both demand directions, until an earthquake with a RP of 475 years. It shows that the building has a similar stiffness on all storeys, or, in other words, until degradation of the infills, the building is highly controlled by them (Figures 3(a)-(b) and 4(a)-(b)).

As expected from the gap between the centre of inertia and the centre of mass on the transversal direction, the rotational effects are evident for a transverse earthquake. The deformations are mainly controlled by the infills, as confirmed by Figure 4(b) because is possible to observe that the façade “12”, which is totally infilled and without openings, controls the shape of deformations being more static than the other columns.

Overlaying the displacements for each level of excitations, is possible to compare the evolution and the formation of soft-storey on the ground storey for a longitudinal earthquake, described in Figure 3(a). It begins to show the tendency on the earthquake with a RP of 475 years. Its tendency (on the first storey) is due to the taller columns directly affecting the lateral strength, and also due to fewer infills compared with the other storeys. The drift on the first storey increases from 0.4 to 2 per cent from 475 years to 2,000 years of RP. With the exception of this storey the other drifts are 0.1 per cent, corresponding roughly to three millimetres for 2.8 metres of column height, assumed to be a safe limit. On the transversal direction, it shows a linear increase of deformations without softening of any of the storeys. The drift is slightly larger on the first storey, a little more than 0.4 per cent for a RP of 2,000 years.

The issue with the torsion effects, Figure 4(b), from a global point of view, should be solved by a calibration or retrofit of some infills, or by reduction of irregularities.

4.3.2 Displacements of bare frame structure. Regarding the behaviour without infill panels, Figures 3(c)-(d) and 4(c)-(d), the difference of the deformation shapes and magnitudes of displacements are evident. Due to a less stiff structure, provided just by the concrete frames, the drifts are less constant in its height for a transverse earthquake, having higher rotations on the plane (Rz). For a 975 years of RP, the structure seems to be forming a soft-storey on the third floor, due to reduction of cross-section’s area of columns. However, continuing to higher accelerations, the height of the first storey becomes important, where the higher axial load together with the top displacement induces higher moments, leading to more bending and then formation of a soft-storey at ground level. The analysis on the other direction shows once more the presence of torsion on the building. Without infills, the displacements are less variable near the wall, meaning a whip on the opposite side of the building, in other words, the stiffness on this direction is controlled by the concrete wall and the columns which supports the interior stairs.

4.3.3 Comparison with recommended limits. The top displacements without infills are about three times higher than with infills, while as showed on Figure 5(a) (plotted for the middle structural column), the inter-storey drifts do not have such difference. Comparing the drifts with the limits recommended by FEMA-356 (FEMA, 2000) and VISION2000 (SEAOC, 2005) (vd. Figure 5(a)) it is concluded that the values are bellow the limitations. These limits, defined for different types of construction techniques and different demand levels, were considered only as good practice boundaries.

The building has a rotation at the top of 0.16 and 0.31 degrees, respectively with and without infills for a RP of 2,000 years, which increases almost linearly for excitations on the transverse direction. The top rotation on the other direction is roughly 0.01 degrees, not a noteworthy value. The inter-storey rotations (vd. Figure 5(b)), which better represent the torsion demand, are generally 50 per cent more intense on the first storey

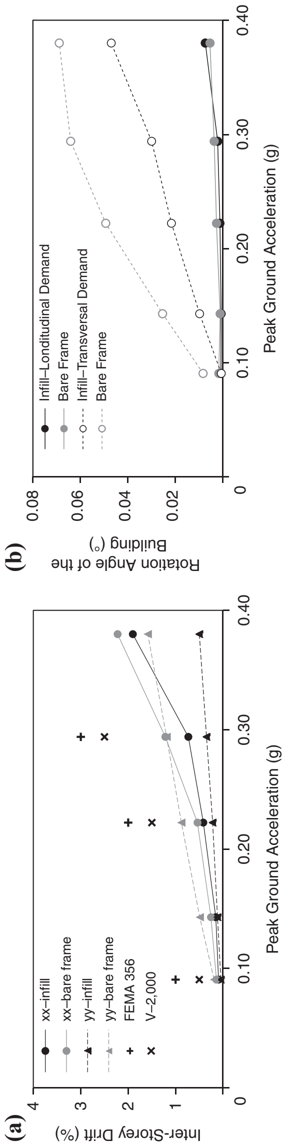


Figure 5.
Larger inter-storey drifts for central columns (a), and maximum inter-storey rotation (b), for different intensities of earthquake and for both bare frame and infilled structure

compared with the other storeys with infills and are similar in height for the bare structure. For the transversal earthquake, the building has an inter-storey rotation of 0.045 and 0.07 degrees, with and without infills respectively, for a RP of 2,000 years.

According to the damage indexes (DI) proposed in Rossetto and Elnashai (2003):

$$DI_{HRC} = 34.89 \ln (ISD_{\max\%}) + 39.39, \tag{1}$$

and:

$$DI_{HRC} = 22.49 \ln (ISD_{\max\%}) + 66.88, \tag{2}$$

respectively for non-ductile moment resistant frames and for infilled frames, the damage level can be assessed qualitatively. Table V has the summary of DI with the respective damage scale (DS) for some considered RPs. For a RP of 975 years the level of damage is within the moderate DS, corresponding to the observation of flexural/shear cracking and yielding in a limited number for the bare frame structure, and increased brick crushing on connections, start of structural damage and diagonal shear cracking on exterior frames for the infilled structure.

4.4 Global force demands

4.4.1 Foundations. The total weight of the building is 11,200 kN. The vertical loads on the building, for an earthquake with a RP of 475 years, changes the total compression in 1,000 and 3,500 kN respectively for bare and infilled structure. It shows a difference of three times the loading on the foundations and soil, therefore, great increase of soil stress demand.

Figure 6 summarizes the base-shear levels. The infilled building has a maximum base-shear capacity of 3,500 kN achieved for 0.22 g on the longitudinal direction. On the transversal direction, it has a continuous growth of base-shear stresses, which shows

		Bare frame		Infilled Frame	
RP	Direction	DI	DS	DI	DS
73	xx	0	None	14	Light
	yy	0	None	0	None
475	xx	18	Light	47	Moderate
	yy	32	Light	33	Light
975	xx	44	Moderate	60	Moderate
	yy	43	Moderate	43	Moderate

Table V. Damage index and scale for the computed inter-storey drifts with and without infill walls

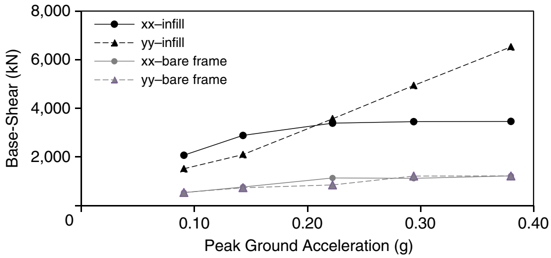


Figure 6. Base-shear variations with and without infill panels for each earthquake

that even for earthquakes with a PGA higher than 0.38 g, the infills still have capacity left, greater than 6,500 kN. The bare frame structure has a base-shear capacity of 1,000 kN for both directions, achieved for 0.29 g. Therefore, is evident that the infill panels are increasing the global shear capacity of the building, therefore, the foundations should be prepared for it.

4.4.2 Columns' compressive load variation. The variations of axial loads are analysed for comparison of earthquakes with a RP of 73 and 975 years. Figure 7 shows the variations in height for different column localizations and earthquake directions. Figure 7(a)-(b) refers to infill panels, and Figure 7(c)-(d) refers to bare frame structure, comparing columns located in the interior, corners and façades (longitudinal and transversal).

The variation on the axial solicitations is higher for the corner columns than for other locations. The columns located on the interior of the building have significantly fewer variations, even if the translations are similar to others – the tilt of the building leans more on extremity elements. As the eccentricity increases, so the variations. The façade columns have from 50 to 100 per cent fewer variations than corner ones. The torsion effects on the transverse earthquake induces an evident increase in the variations, where the same columns can go up to twice load variations.

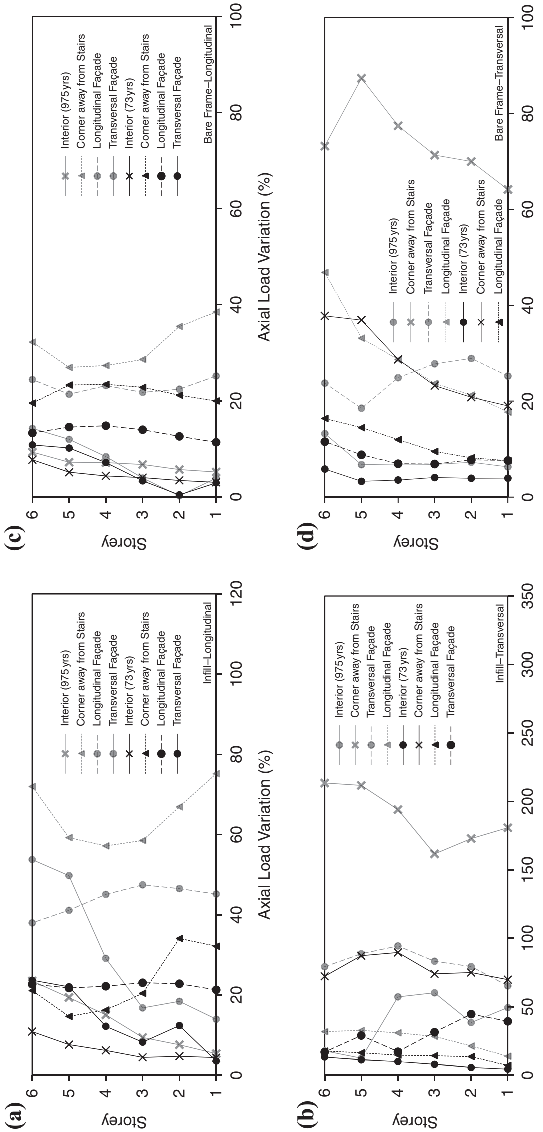
The corner columns have the biggest variations in an average range of 100 per cent, followed by the façade columns with 50 per cent and interior with less than 20 per cent for a strong earthquake with a RP of 975 years. For an earthquake with a RP of 73 years, the variations are more concentrated and do not exceed 20 per cent for both framed and infilled structures.

It was verified variations that have led to tractions on some columns. In general, the variations are similar for both compressive and "traction" growths. It is noteworthy that columns surrounding infill walls got two times higher variations compared to the columns without walls.

4.5 Shear-drift

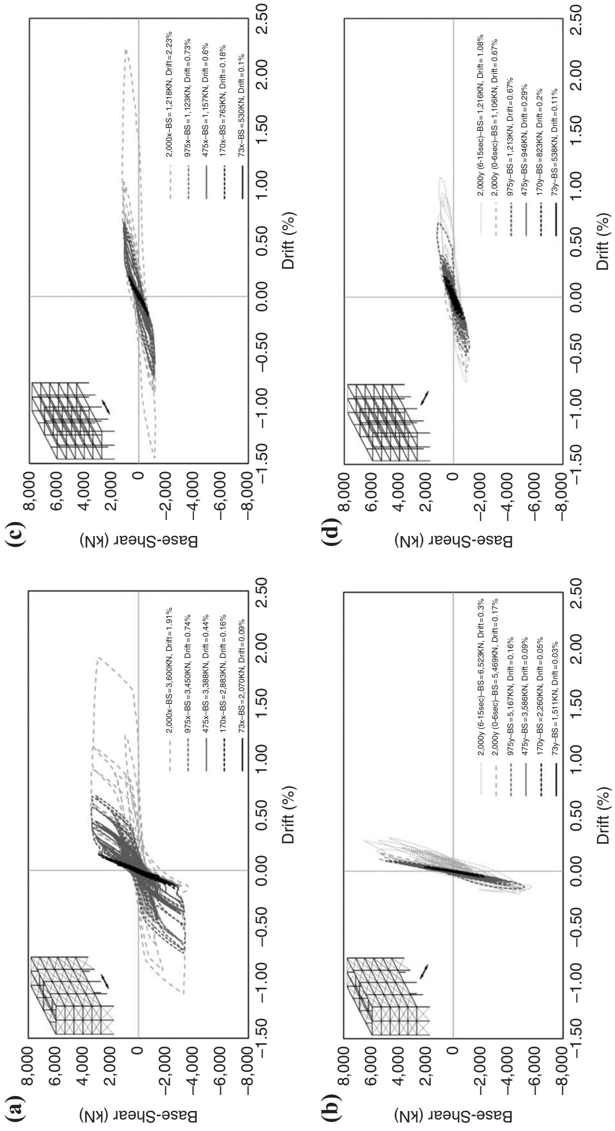
In this section, the base-shear vs drift is plotted, which helps understanding of the progression of the forces towards deformations and also the ductility moved by the structure in the two directions. Figure 8(a)-(b) is related to the floor shear-drift for the structure with infill panels and Figure 8(c)-(d) is related to the structure without infills.

First, for the structure with infills, Figure 8(a)-(b), as expected from the early deformation results, both the highest stresses and deformations are located at the bottom floor. Starting from the infilled structure, once again the stiffness of the structure is proved to be higher on the transverse direction where the slope of shear-drift is higher. As shown before, the soft-storey is happening on the longitudinal direction, where Figure 8(a) shows a mobilization of non-linear progression. The longitudinal direction, with a maximum base-shear capacity achieved near the 4,000 kN, is less stiff by the infills, showing more ductility compared with the transverse direction. In the transverse direction, due to high stiff elements such as those infill walls on the extremities and the concrete wall, the structure deals with more forces, up to base-shear 6,000 kN, and smaller deformations. Discarding the influence of the concrete wall in terms of torsion effects, without measuring local effects, these infills are protecting the structure globally, absorbing a great amount of forces which is not going to affect the structural elements. For a RP of 2,000 years, a



Notes: (a) A longitudinal and (b) a transversal earthquakes, and for bare frame structure, on (c) a longitudinal and (d) a transversal earthquakes

Figure 7.
Comparison between axial stress variation on columns in different places for with infill panels



Notes: (a) Longitudinal direction and on (b) transversal direction; and without infill panels on (c) longitudinal direction and on (d) transversal direction

Figure 8.
Base-Shear-Drift with infill panels for earthquake and response

reduction of stiffness for the transverse direction and reduction of stresses is visible at the base level, due to yielding of infills and transference of energy to the wall.

Discarding the infill panels, the stiffness of both directions and global behaviour at the base level is very similar, both for deformations and base-shear capacity. The structure starts to yield for an earthquake of 475 years of RP. For an earthquake of 2,000 years RP, the level of viewable deformation starts to increase exponentially, which leads to the conclusion that the global capacity is yet to be achieved, but not far from an unsafe perspective. The amount of shear stress at the foundations is, at least three times lower than for the structure with infills, which is a significant difference.

Figure 9 compares the modelling with and without infills in more detail. The main conclusions are: for the longitudinal direction there are similar drifts but higher stresses with infills; and for the transversal direction there are higher stresses but more than two times fewer drifts. The stiff panels together with the concrete wall provide strong elements which restrict the deformations.

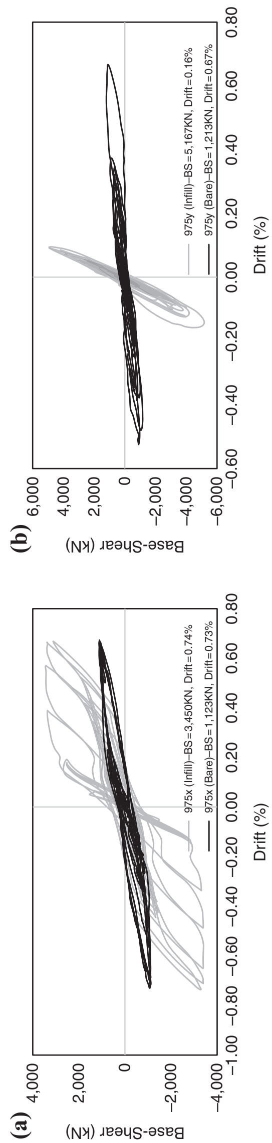
Figure 10, for a 975 year RP, without infill panels, is showing small differences between the storeys, which have a similar shear-drift relationship slope with higher demands at the bottom floors. On the transversal direction the top floors show a slight reduction of stress levels and increase of drifts, which means a reduction of stiffness due to the lack of influence of the concrete wall along the height.

5. Assessment at the section level

The safety verifications at the section level are only presented for the bare frame structure, and for an earthquake with 975 years of RP, corresponding to the ultimate capacity of safety level. The verifications for the infilled frame were not performed due to the simplifications of the used model, since the panels are connected to the beam-column joints making it difficult to properly predict the bending moments and shear forces in the surrounding frame (Smyrou, 2006).

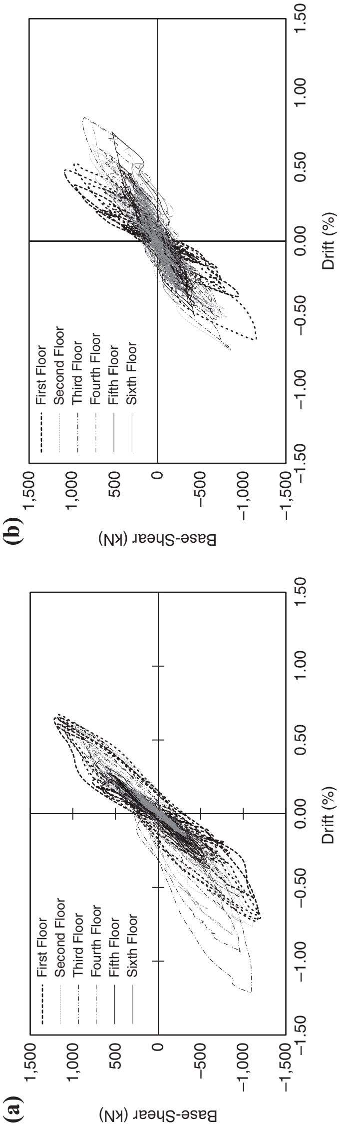
5.1 Ductile mechanism

In each temporal instant of an earthquake, conditions change, and the capacity of the elements evolves accordingly. Axial, shear and moment stresses, deformations and degradation change during the earthquake, causes direct impact on the material properties and on its final resistance. Therefore, to compute the moment and chord-rotation capacity and to plot it, it is vital to decide which are the expectable envelope limits, which are not clearly defined in the code. Two main approaches for the verifications regarding the axial loads on elements were tested. The first shows the verifications for static axial loads, from the gravitational seismic combined with a shear span considered to be equal to half the length of each element (L/2). The second shows the verification approach for maximum and minimum axial loads, extracted from the analysis, corresponding, respectively to high and less stress on the elements, and its correspondent moment/shear forces to calculate the shear span (M/V). The empirical and the theoretical approaches of the EC8 (CEN, 2009) are also compared, with the same assumptions. The theoretical approach is used for comparison, although it is not recommended for non-seismic designed structures. The empirical formula was calibrated taking into account the cyclic degradations on the elements, and reduction factors were used to decrease verification limits for structures with seismic deficiencies.



Notes: (a) Longitudinal direction and (b) transversal direction

Figure 9.
Comparison of base-shear-drift with and without infill panels for an earthquake with a return period of 975 years



Notes: (a) Longitudinal earthquake and (b) transversal earthquake

Figure 10.
Shear-Drift progression by floor, on the centre column for a return period of 975 years

5.1.1 Verification of safety. Figure 11 presents the percentage of the elements with non-verified safety for chord-rotation, when subjected to the earthquake in both longitudinal (xx) and transversal (yy) directions.

Different verifications were performed to check the limits which are allowed by the indeterminate information on the code. The results are divided by groups with the envelope combination of all the elements which did not pass on the verifications, for both empirical and theoretical approaches, and also for merged. The first individual group took the axial loads of the seismic combination, and shear span equal to "L/2". This approach is important because it is one of the most simplified verifications, and when compared with the others, the fastest one. The drawback is the lower conservative results. The next two groups refer to maximum and minimum axial loads taken from the analysis, again, considering the shear span equal to "L/2". In the last two groups, the difference from the latter is on the shear span which were done according to the formulation moment/shear, "M/V". The moment and shear forces which were used are the exact ones acting on the elements for the instant with the maximum and minimum axial loads.

The axial load on the elements, mainly on columns, takes an important part, since the axial load affect the behaviour of the element on deformation capacity.

In general, the results do not match, then may be recommended to perform the assessment from the combination of the various approaches. If such extensive analyses are not possible, the most conservative approach should be used, namely the combination of both earthquakes, using the theoretical approach, with maximum compressive stress and "L/2" shear span.

5.1.2 Location of deficiencies. The chord-rotation deficiencies mainly appear up to the third storey but are more evident on the second and third storeys. The progression of the non-verified elements are described in Figure 12, which as an example, is only presented for the transversal direction. It shows a higher demand on columns than beams for both earthquakes, and also that the building still holds a lot of capacity to sustain deformations, with 50 per cent of spare capacity for roughly 90 per cent of the columns. The progression on columns confirms that the distribution of capacity left is higher on the bottom storeys than the top storeys, which can be explained by the differences on the axial load demands.

5.1.3 Ductility on chord-rotation. The ductility of the elements regarding the chord-rotation deformations is schematically presented on Figure 13. The assumed limit for this evaluation was the theoretical formulation of the EC8 to compute the yielded

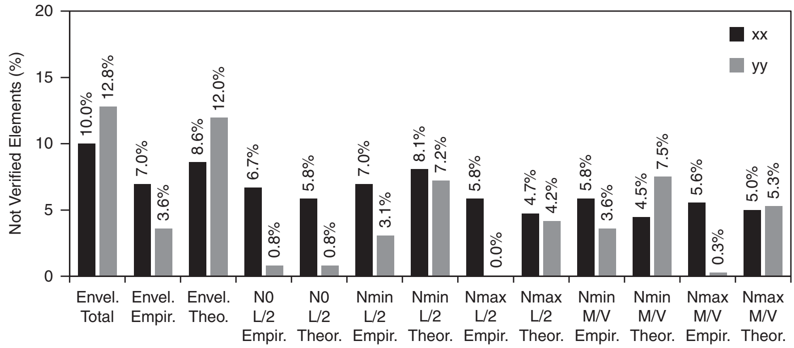
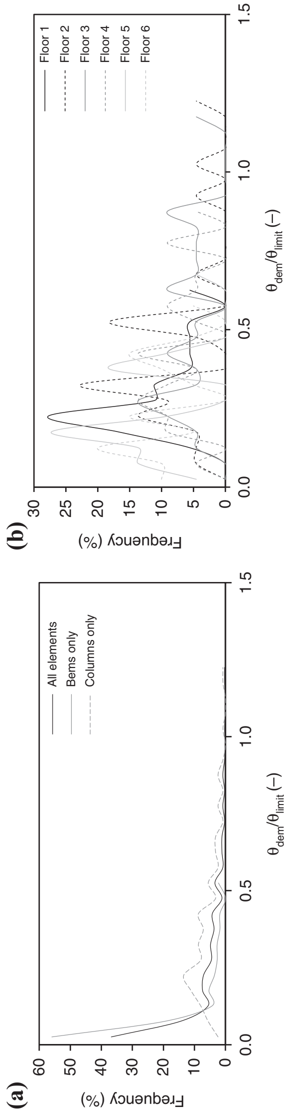
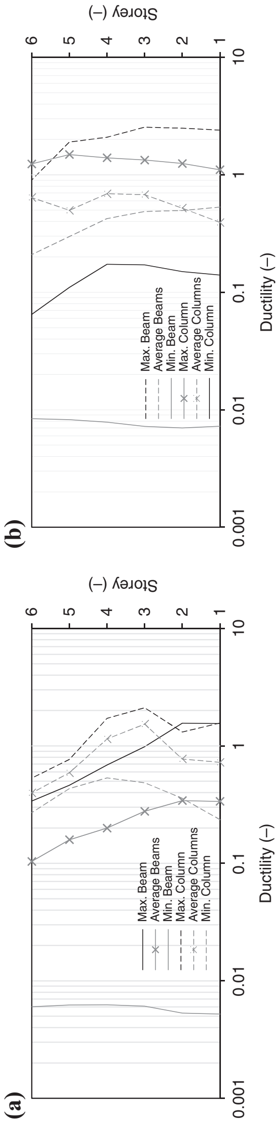


Figure 11.
Elements failing in chord-rotation limitation



Notes: (a) All elements and for (b) columns in different storeys

Figure 12.
Level of safety
on a transversal
earthquake



Notes: Minimum, average and maximum ductility by floor for (a) longitudinal and (b) transversal direction

Figure 13.
Ductility of beams and columns, regarding the chord-rotation

chord-rotation, referring to the parameters of seismic combination with shear span equal to "L/2". It is separated by groups of beams and columns, and by storey, for the two demand directions. The different lines show the variations regarding the minimum, maximum and average ductility mobilized per storey. On average, only the columns on the third and fourth storeys for longitudinal direction are performing in their plastic behaviour. In general, the elements do not have a high ductile response, excepting only a few groups of columns and beams. Regarding the maximum ductility, all storeys have some beams and columns which did perform in plastic performance and still maintained the safety limits. The chord-rotation demands are lower for the top storeys, showing a good level of safety in terms of deformations for those levels.

5.2 Brittle mechanism

A brittle failure is dangerous because it does not allow the redistribution of stresses for the equilibrium. The verification of this type of failure was performed according to the Eurocode 8, which separates contribution of axial load, concrete strength and transverse steel strength. The formulations are based on empirical calibrations for new constructions, therefore, for assessment purposes of existing buildings, the formulation may not be well calibrated.

5.2.1 Verification of safety. The results are described on Table VI, showing vulnerability to shear stress, both on columns and beams. The beams have more shear deficiencies on the transversal earthquake than on longitudinal, and are spread along the floors. A concentration of vulnerabilities on a specific zone of a floor is not verified, but it prevails on the first three floors. In terms of vertical elements, for a longitudinal earthquake, the central columns on the two exterior transversal façades have a high shear demand in their height until the fifth floor, which makes those elements unsafe. For the transversal earthquake, the deficiencies are more concentrated near and on the opposite façade to the concrete wall.

In general, the elements have a lack of less than 15 per cent of shear capacity for longitudinal earthquakes. For transversal ones, the percentage is higher where some elements surpass 50 per cent of the safety limit for shear stress. Without the safety factors, using the average properties of the materials, the percentage of non-verified elements drops to 2 per cent and 10 per cent for longitudinal and transversal earthquakes, respectively. It is noteworthy that the information for the transversal reinforcement is scarse, and therefore, some results may have not been accurate, since the reinforcement strongly affects the verifications.

5.3 Joint shear strength

The shear on joints is another very important factor in the assessment of structures and may be one of the most important issues on seismic demand. Together with the

Earthquake	Longitudinal		Transversal	
Direction	22	33	22	33
Total (%)	0.0	16.1	4.8	15.9
Beams (%)	0.0	14.5	0.0	24.2
Columns (%)	0.0	19.0	13.5	0.8

Note: With stirrups of 8ϕ//.20m or 8ϕ//20cm or 8ϕ//200mm

Table VI. Unsafe elements in shear demand without infill panels ineffectiveness of the joint projected by the old codes, where the longitudinal reinforcement is not properly tied, lacking reinforcement and the slippage of the smooth bars can be potentially harmful. This matter is addressed in (CEN, 2004) and well organized in a support concrete manual (IStructE, 2006).

To perform this local assessment, the shear capacity on joints was performed according to the formulations of EC8-1 (CEN, 2003) and the Italian Code (NTC08, 2008). The major difference between both approaches is that the Italian Code is specifically prepared to verify this failure on existing buildings, in other words, for structures without seismic provisions. It separates the maximum diagonal compression and tensile stress in the joint core which needs to be compared with the concrete strength.

5.3.1 Verification of safety. The results are summarized in Table VII. The analyses were made considering two different envelopes, one with the higher compressive load, with its respective shear, and also with the inverse. The formulation by the Italian Code, for the compressive strength of the diagonal strut, is slightly more conservative and is higher using the maximum shear stress as a parameter. The safety verifications from both codes are not very different on these results, by the ratio of strength and capacity. The higher conservative character is adequate to the verifications on joints and type of structure because of being prepared for old structures.

Regarding the joint failure on the tensile diagonal strut, the results show a very conservative verification. According to Paulay and Priestley's (1992), even with the joint cracked, the joint panel and reinforcement retains capacity to transfer shear forces, therefore, the joint failure should be only considered by the compressed strut crush. It is also specified that, for high axial loads, the compressed crushing should be verified before the tensile cracking.

The joints that are failing on compressive crush are from three interior columns located on the first and second storey, and from three columns of the longitudinal façade which have no indirect columns. From the 110 nodes, no more than ten joints are failing. Without the use of the safety factor, the achieved verification would be 100 per cent safe.

6. Conclusions

The modelling of existing structures lack formulations to compute the influence of slippage, so as a consequence, there is no accurate way to compute real rotations of the fixed-ends on global structure modelling. Analysing at an element level, the contribution of the slippage on the fixed-end rotation may have an impact up to 90 per cent on the total deformation of the element, according to (Verderame *et al.*, 2008a, b). Some approaches, such as reduction of the plastic hinge, introduction of spring elements on nodes or calibration of steel properties to increase flexibility, are some strategies that may be used but yet unreliable.

Table VII. Joints failing in shear demand according to EC8 and NTC8 for diagonal compressive and tensile(*) strength

Earthquake	Longitudinal			Transversal		
	Compressive		Tensile	Compressive		Tensile
Code	EC8	NTC8	NTC8*	EC8	NTC8	NTC8
Nmax (%)	2.7	3.6	66.4	2.7	2.7	70.0
Vmax (%)	2.7	6.4	62.7	2.7	6.4	61.8

The infill masonry panels have great influence on the structure, regarding its locations in height and in plan, affect of modal shapes and changes completely the participations. Therefore, in case of irregularities, their influence have to be addressed. The presence of stiff elements, such as concrete walls or as a set of secondary elements (such as those for supporting the stairs), have a big impact on the global response due to rotation effects, a consequence of the modification of uncentred stiffness and mass.

The axial stress on columns can achieve high levels of variation. Briefly, for strong earthquakes, on average it can happen in a range up to 100 per cent for corner columns, 50 per cent for façade columns and less than 20 per cent for interior columns. For columns limiting the infill panels, these limits can be increased somewhere up to two times higher variations compared with bare frame structure.

In terms of local safety verifications, regarding chord-rotations, there are a lot of different ways to compute the limits and there is no individual one which can be considered as the most conservative. The results tend to be more conservative for the theoretical approach than the empirical one. The theoretical approach is more complex to apply, but guarantees, at least, slightly more conservative results which seem to be adjustable. Regarding shear strength, the building show deficiencies for both beams, columns and also on joints. The verification of safety on a small amount of joints is not attained, a very dangerous type failure which should be avoided at any cost.

The numerical results – which were a limited portion of study cases – showed that structures with low ductility present a few vulnerabilities in terms of safety levels and damage limitation, for medium-strong seismic intensity (EC8). The conclusions are valid for good quality of materials, construction, and for isolated buildings.

Note

1. The programme SeismoStruct has no current integrated model to compute the effect of steel slippage. To overcome it, is necessary to have an accurate definition of the properties of the materials and the plastic hinges.

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