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Seismic Assessment of a URM Building and Effect of Floor Diaphragm Stiffness

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ABSTRACT: The seismic behaviour of a previously calibrated Finite Element Model of a typical vintage unreinforced masonry building from the end of XIX century found in Lisbon, Portugal, was investigated using Time History Analysis and Pushover Analysis. The response of the building was analysed using DIANA software package and strain maps were obtained in order to identify the global failure mode, potential local damage and localised failure modes of the subject building. A subsequent comparison between the results obtained with the two analysis methods determined the pros and cons of each method and assisted in establishing the confidence level when using Pushover Analysis in comparison with Time History Analysis. Finally, a parametric analysis was carried out, using both Time History Analysis and Pushover Analysis on models that featured floor diaphragms with different stiffness values, to investigate the effect of the floor diaphragms stiffness on the overall response of the building.

1 INTRODUCTION

Existing unreinforced masonry (URM) buildings are typically identified as earthquake prone due to multiple reasons, but only the seismic behaviour of masonry walls, floor diaphragms and their interaction are discussed herein. A common source of problems when URM buildings are subjected to seismic induced shaking is related to the configuration of the structure such as heavy stiff walls and lightweight flexible diaphragms. In addition, this type of construction typically features weakly connected structures with stiff and brittle behaviour that give little indication, if any, before collapsing, increasing the risk that these buildings pose for the society. An inherent disadvantage of masonry is the properties of the constituent materials used, such as the low tensile strength of the masonry units and the mortar poor bond strength, that combined with the geometrical irregularities commonly found on these buildings result in the non-linear behaviour typical of these structures when subjected to seismic loads. Multiple methods for modelling structures and replicating their seismic behaviour have been developed in the past, including the Finite Element Method (FEM) which was the modelling method used in the present research, utilising the software package DIANA V9.4 (TNO 2009).

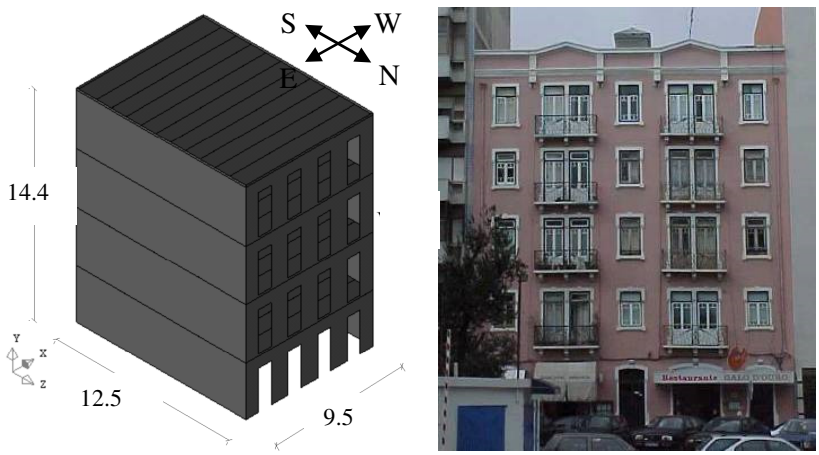
When assessing the effects that earthquakes have on buildings one of the most accurate method is Time History Analysis (THA), in which the accelerograms recorded from real earthquakes or artificially developed are applied as external actions on the structures. THA is time and computationally expensive and a specific ground motion compatible with the seismic hazard spectrum for the site where the building is located might be difficult to obtain. For these reasons simpler and more straightforward loading methods such as Pushover Analysis (PA) have been developed. In PA the seismic weight of the building is statically applied horizontally to the structure to match different load patterns such as proportional to the weight or proportional to the first mode of response. Alternatively, a pattern of displacements can be statically applied horizontally in order to calculate the strength and stiffness of the structure at different displacement levels. The results obtained using THA and PA methods for the subject building are compared herein with the aim of validating PA as a valid yet time effective method to predict the seismic behaviour of URM buildings. A parametric analysis in which several mechanical properties of the building were modified in order to assess the influence of these properties on the final response of the structure was undertaken as part of the research. The floor diaphragm stiffness was found to be the parameter with the largest influence on the final response of the building, which results are reported herein, with the rest of the parameters being discussed by del Rey Castillo (2012).

2 GAIOLEIRO BUILDING

The building under study is a common typology found in Lisbon, Portugal, called "Gaioleiro" that appeared profusely between the middle of the 19th century and the beginning of the 20th century motivated by the rapid growth of the city. The main characteristic of this building typology is thick, heavy and commonly weak URM loadbearing walls with lightweight and flexible timber floor diaphragms typically not connected to the walls, which results in independent walls not responding cohesively to lateral loads. "Gaioleiro" buildings typically consist of four or five storeys and commonly feature walls with different seismic weight depending on the opening configuration, which increase the torsional response of the building derived from geometrical irregularities. The lack of box-like behaviour and their irregular dynamic response are the two main reasons to consider "Gaioleiro" buildings as a potential risk in the event of an earthquake, which together with containing a large number of occupants and having significant architectural heritage value, justified the effort made to analyse, assess and understand the seismic behaviour of this building typology.

The subject building features four 0.51 m thick URM load-bearing walls, two solid walls in the North-South direction and two largely perforated walls in the East-West direction, with consistent materials characteristics. The soil corresponding to the Lisbon area is Type A Rock with a damping ratio of 5%, as specified in the Portuguese Code (RSA 1984).

The footprint dimensions of the building are 12.5 m in the N-S direction and 9.5 m in the E-W direction, with four storeys of 3.6 m height each totalling 14.4 m. The perforated walls feature four openings per storey measuring 2.7 m high and 0.9 m wide each, see Figure 1 for (a) a general perspective of the building and (b) an example of a real building in Lisbon, Portugal.



(a) General perspective, (units in meters)

(b) Example of 'Gaioleiro' building (Mendes & Lourenco 2010)

Figure 1 – Perspective and real picture of the case study building.

The floor diaphragms consist of timber joists 0.3 m deep and 0.225 m wide spaced at 1.05 m sitting on the walls (simply supported) with Medium Density Fibreboard (MDF) panels 0.036 m thick. A reduced scale model of the subject building was tested at the triaxial shaking table of the National Laboratory of Civil Engineering (LNEC) in Lisbon, see (Candeias et al. 2004)

The material properties specified in Table 1 and Table 2 were used as input, being these values obtained from the materials used in the reduced scale model. The FEM model was created using eight-node quadrilateral isoparametric curved shell CQ40S elements for the walls and MDF floor panels, based on quadratic interpolation with a Gauss integration scheme over the area using four integration points, and a Simpson integration scheme with five points over the thickness. CL18B three-node, three-dimensional class-III beam elements were used for the timber joists. All the elements are based on the first-order shear deformation theory of plates, in which the shear deformation is considered. The variables were the translations and rotations of the nodes for the three directions while the structure boundary conditions assigned were fully fixed to the ground and the building was modelled as free-standing. Rayleigh damping was used to characterise the dynamic behaviour of the building, with the first coefficient related to mass equal to 0.761100 and the second coefficient related to stiffness equal to 0.002618. The FEM model involved 5816 elements (1080 beam elements and 4736 shell elements) with 15,176 nodes, resulting in 75,880 degrees of freedom and was previously calibrated by Mendes and Lourenco (2010) with the results obtained from the shaking table.

Table 1: Linear material properties used as input for DIANA

Material	Young's Modulus (GPa)	Poisson ratio	Density (kg/m^3)
Timber joists	12	0.3	580
MDF floor panels	140	0.3	760
Masonry	1	0.2	2150

Table 2: Non-linear material properties used as input for DIANA

Material	Tensile strength* (MPa)	Fracture energy in tension (kPa/m)	Compressive strength** (MPa)	Fracture energy in compression (kPa/m)
Masonry	0.1	0.12	6.0	9.6

* With a post-cracked exponential behaviour

** With a post-cracked parabolic behaviour

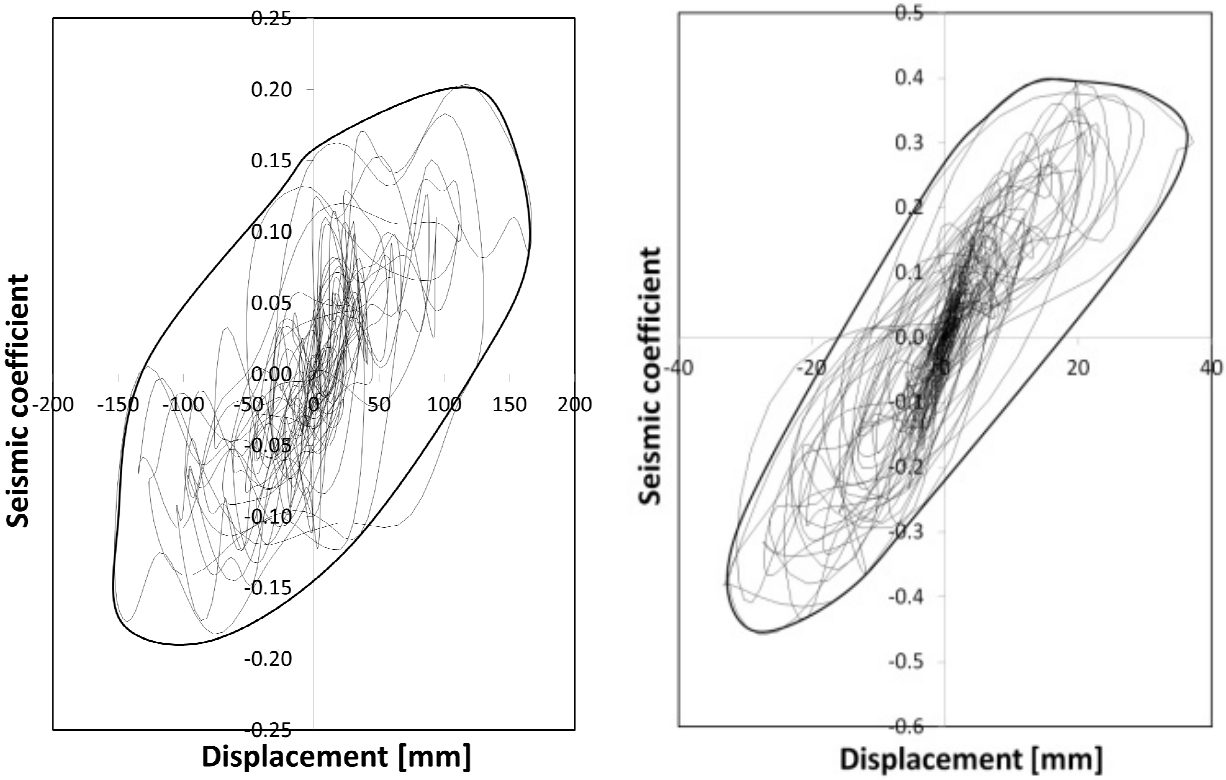
3 TIME HISTORY ANALYSIS (THA) OF THE REFERENCE MODEL

Two accelerograms were generated using the software SIMQKE_GR (Gelfi 2006), with a baseline correction filter using SeismoSignal (Seismosoft 2004). The accelerograms were based on the response spectrum (Type 1) defined by Eurocode 8 (2004), one was generated for the E-W direction and another one for the N-S direction. The soil type chosen was type A, corresponding to region of Lisbon and the selected damping ratio was 5%. The accelerograms are obtained from the Portuguese National Annex of Eurocode 8 (2004). Vertical motion was not considered in the shaking table experimental study at (Candeias et al. 2004) and hence this direction was not considered herein either. Damping ratio was obtained in a previous stage of the research campaign, see (Mendes & Lourenco 2010), simulating Rayleigh viscous damping, with constants α=2.18 and β=0.00044 determined from the results obtained in the dynamic identification tests, see Table 3. The accelerograms had a total duration of 36 seconds that was divided into 6000 steps, but measurements were only recorded for every second load step to shorten analysis times. The HHT integration method was used (also called the alpha method) with a linear iteration method as this type of calculation fits better with loading-unloading systems. Energy convergence criteria were used and the tolerance was established as 0.001.

The results obtained with the accelerograms showed that an earthquake with the characteristics expected for the Lisbon area would not cause collapse of the subject building, as previously reported in detail by del Rey Castillo (2012). Therefore, the peak ground acceleration (PGA) was increased three times in an attempt to obtain the maximum base shear capacity of the structure. The seismic coefficient was defined as the ratio of the horizontal base shear divided by the weight of the building, and this value was obtained and plotted versus the displacement at the top of the solid wall (see Figure 2) and at the top of the façade (see Figure 3) for the entire duration of the earthquake. The structure in the E-W direction reached a maximum seismic coefficient of 0.2 corresponding to a maximum displacement of 150 mm at the top of the building, while in the N-S direction the maximum seismic coefficient was equal to 0.4 with a displacement of approximately 40 mm.

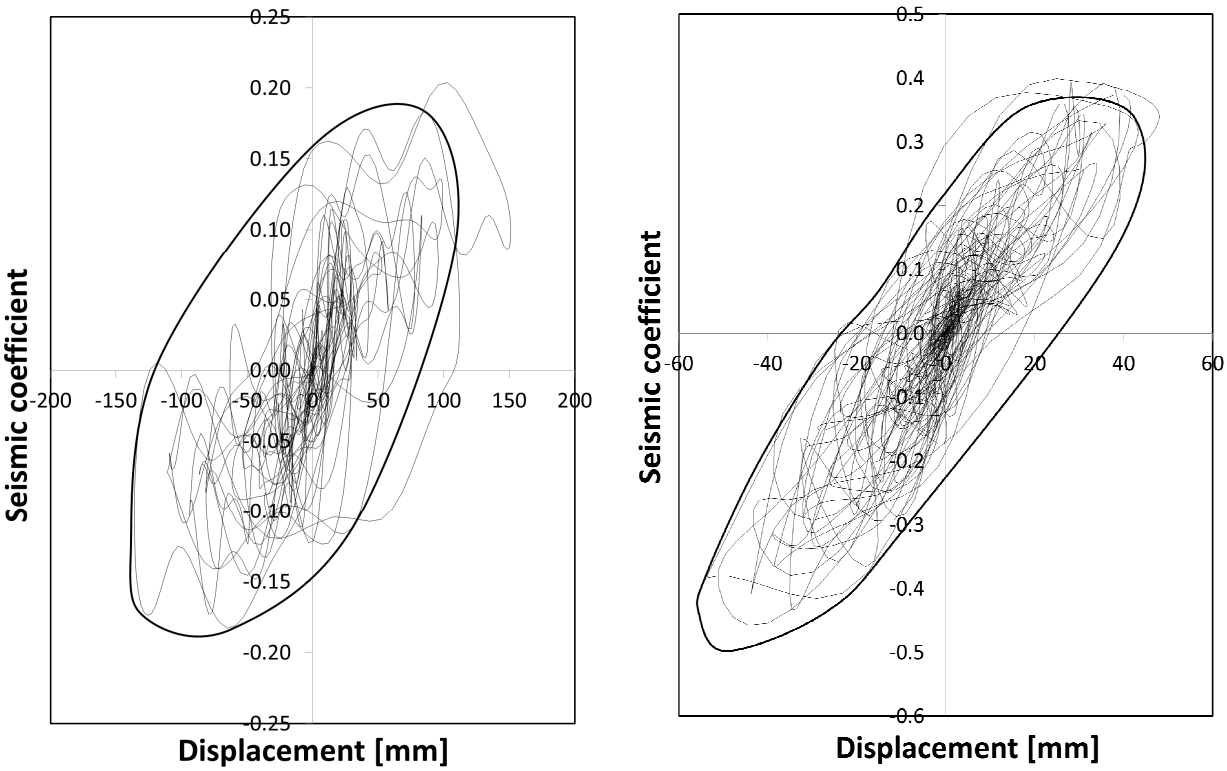
Table 3: Values of damping ratio obtained during the experimental campaign

	Frequency (Hz)	Modal damping ratio (%)
First translational mode	4.8	4.3
First torsional mode	9.2	2.5



(a) E-W direction *(b) N-S direction*

Figure 2 – History and envelopes of displacement-seismic coefficient for the solid wall.

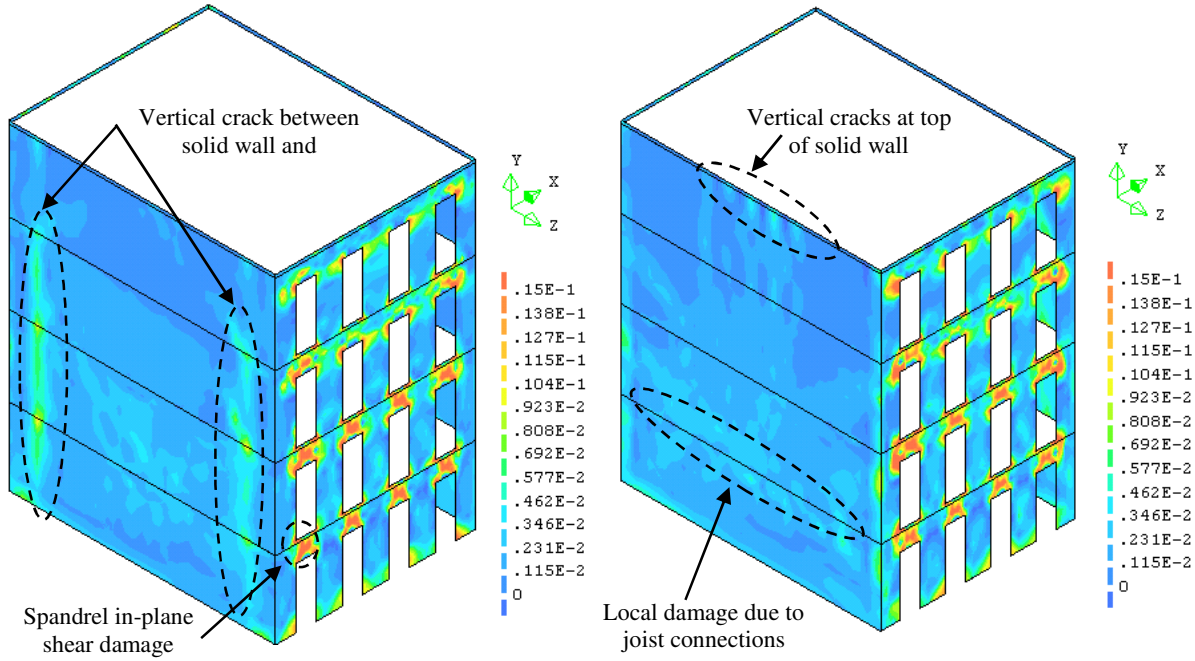


(a) E-W direction *(b) N-S direction*

Figure 3 – Histories and envelope of displacement-seismic coefficient for the perforated wall.

Based on the results, the expected failure of the building was found to be related to diagonal cracking of the spandrels due to in-plane actions in the perforated walls, particularly in the ground and first floor levels, as diagnosed with the help of the plot of the maximum values of the principal tensile strains (see

Figure 4). Lower levels of strains were identified in the spandrels of the upper levels, being attributed to the smaller seismic weight involved at these levels. Vertical cracks extending along the sides of the solid wall in the vicinity to the building corner were identified due to a weak wall-to-wall connection, together with some minor vertical cracks at the top of the solid wall. Localised damage was identified due to the presence of timber joists at the first floor level.



(a) Interior surface of the walls *(b) Exterior surface of the walls*

Figure 4 - North-East view of the maximum values of principal tensile strain of the walls.

4 PUSHOVER ANALYSIS (PA) OF THE REFERENCE MODEL

Non-linear static analysis commonly known as Pushover Analysis was conducted to assess the seismic behaviour of the subject building, loading the structure in two independent orthogonal directions (E-W and N-S). The capacity curves were obtained and are reported herein to relate the applied horizontal load to the expected displacement of the structure at the control point, which was set at the top of the walls at mid-span. Mass proportional and first mode proportional loading patterns were applied to the structure, using force based and displacement based methods. The best match between the results obtained with THA and those obtained with PA was using the mass proportional pattern combined with the force based method, as reported by del Rey Castillo in detail (2012). Therefore this methodology was followed, applying at each step a force equal to 2.5% of the self-weight of the structure but when the steps did not converge the load was modified to different values, never being larger than 5% of the self-weight. Arc-length displacement control was used, combined with the spherical path method. The Newton-Raphson regular iteration method was used with the same convergence criteria as in the THA.

4.1 *E-W direction*

The PA in the E-W direction was performed by applying the load towards the West direction and the seismic coefficient was used to plot the capacity curve (see Figure 5) as when plotting the envelopes for THA. Three branches were identified in the curve, corresponding to three different structural behaviours with the first branch representing a quasi-elastic behaviour, an almost linear relationship between applied load and displacements could be observed. The second branch corresponded to a softer portion of the curves, where the building would suffer larger levels of damage than in the previous branch for similar applied load increments, and with cracks opening in the walls. The last branch was characterised by large displacements corresponding to small load increments, but this branch was unrealistic as the building might have partially collapsed at some point between branches two and three. The last step shown (step 19) was identified as the ultimate state of the structure, with the building heavily damaged

and potentially completely collapsed. The displacements and seismic coefficients for steps 16 or 17 were in agreement with the results from THA, see Figure 2 and Figure 3, which means that the input load for the THA could have been increased further in order to reach the ultimate state.

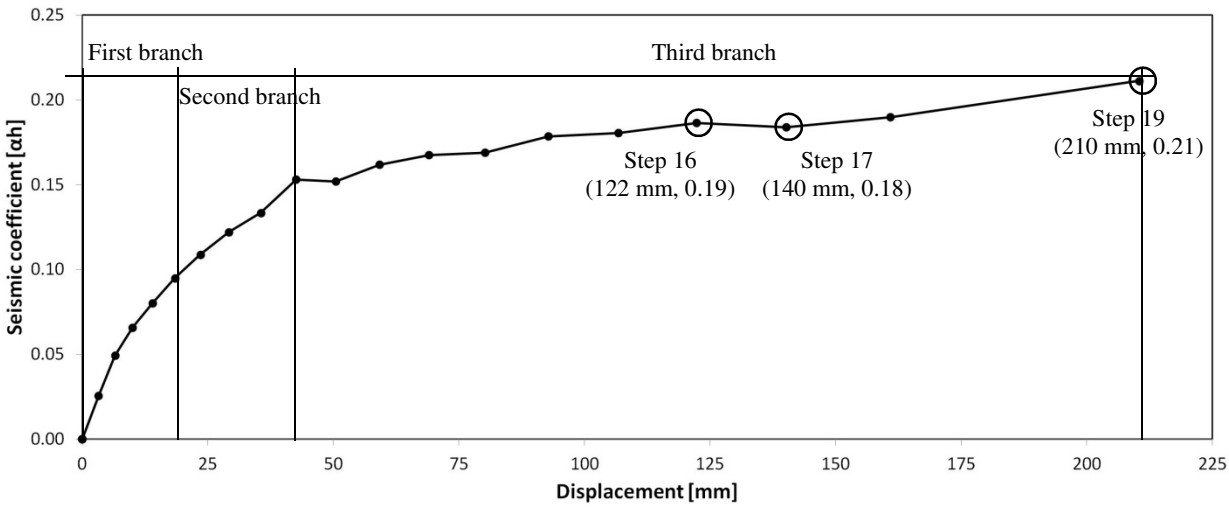


Figure 5 - Capacity curve for the E-W direction of the reference model

The principal tensile strains obtained from PA (shown in Figure 6) were compared with the principal tensile strains obtained from THA for the E-W direction, see Figure 4. The damage patterns agreed for both PA and THA, with the most significant damage identified in the spandrels, especially in the ground and first floor levels, and less significant damage found in the form of a vertical crack in the solid walls. However, PA was not able to identify the local damage due to the presence of the joists in the first floor levels. The displacements obtained for step 19 with PA were larger than those obtained through THA, which explained the larger strains obtained with PA. The strains were asymmetrical, as the PA was performed in one direction only.

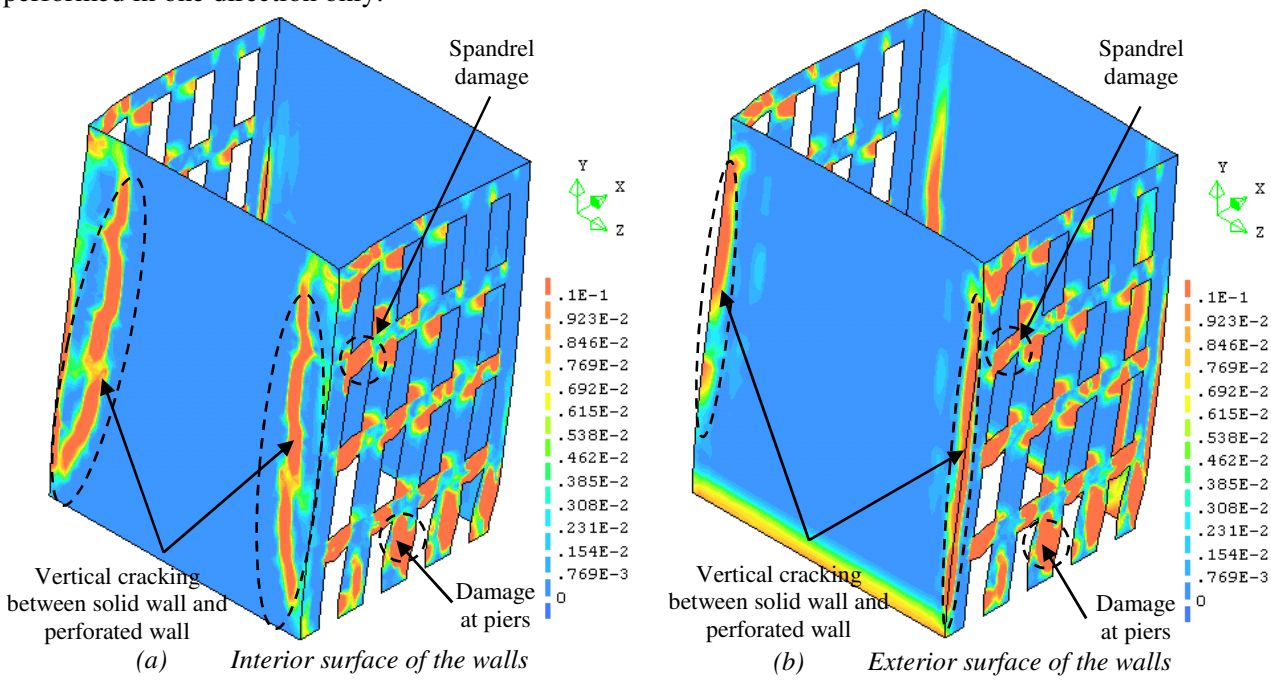


Figure 6 - North-east perspective of the principal tensile strain state of the walls at step number 19

4.2 *N-S direction*

The PA in the N-S direction was performed by applying the load towards the North direction and the seismic coefficient was used to plot the capacity curve. Corresponding branches with similar characteristics as for the E-W direction were identified for the N-S direction. The last step shown (step 31) was identified as the ultimate state of the structure, with the building heavily damaged and collapsed. As for the E-W direction the results from THA correlated with results for steps previous to step 31, see Figure 7, but in this case the seismic coefficient and displacement were not in direct accordance with those obtained with THA. As for the E-W direction, in this direction the input load for the THA could have been increased further in order to identify the ultimate state.

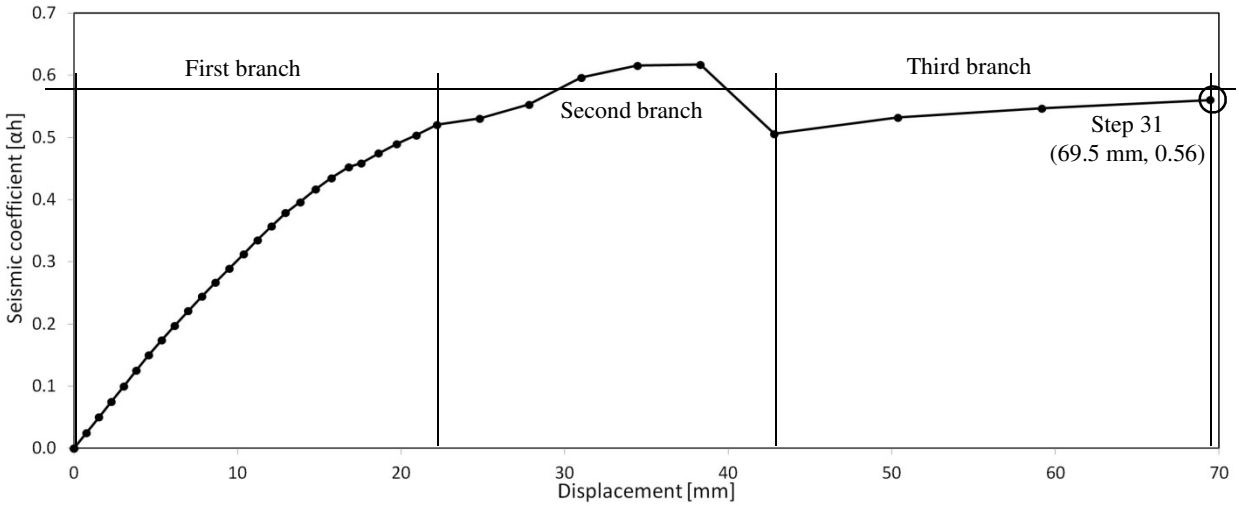


Figure 7 - Capacity curve for the North-South direction of the reference model

The principal tensile strains obtained with PA for the N-S direction and shown in Figure 8 were comparable with the principal tensile strains envelope obtained with the THA, see Figure 4. The vertical crack between the solid walls and the perforated walls was identified based on the results from PA, but not the vertical cracks at the top of the solid walls. In addition a crack appeared diagonally crossing the solid wall that was not identified with the THA. The damage pattern observed in Figure 8 was not symmetrical because the load was applied in one direction only, which might explain the difference when compared with the results obtained using THA. The displacements obtained for step 19 with PA were larger than those obtained using THA, which explains the larger strains reached with PA.

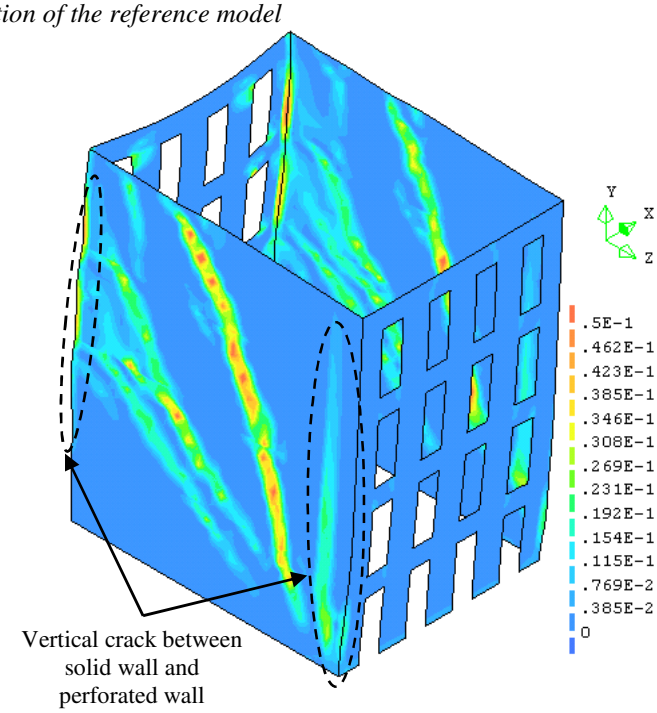


Figure 8 - North-east perspective of the principal tensile strain state at step 37

5 INFLUENCE OF FLOOR DIAPHRAGM STIFFNESS

The parametric analysis undertaken during the research, see (del Rey Castillo 2012), identified the floor diaphragm stiffness as the material characteristic with the largest influence on the seismic response of the building. The capacity of the floor diaphragm to tie together the four URM walls and provide box-like behaviour, was found to be paramount in order to improve the seismic performance of the structure.

The parameter used to investigate this property was the elastic modulus of the MDF floor panels, having an original value of 140 GPa and also being divided by 10 and multiplied by 10, 100 and 1000 to investigate the sensitivity of this parameter on the results. As for the reference model, THA and PA were undertaken with the new values of floor stiffness and the revised results were compared with the original results. Regarding THA, the envelopes of the seismic coefficient versus out-of-plane displacement at the top of the walls are reported in Figure 9 but the in-plane behaviour of the walls has not been presented because of the negligible difference in the response of the building. The results from the 0.1xE model gave significantly larger displacements than for the reference model, indicating that box-like behaviour was not achieved. As the stiffness of the MDF floor panels was increased the displacements observed at the top of the walls decreased for similar values of seismic coefficient. The box-like behaviour achieved with the stiffest floor diaphragms improved the response of the building significantly, reducing the out-of-displacement at the top of walls in mid-span approximately from 150 mm to 50 mm in the solid walls and from 40 mm to 10 mm in the perforated walls. The reason for the better performance of the structure is the ability of the diaphragm to join the four walls together distributing the loads between them and improving the cohesively response of the building.

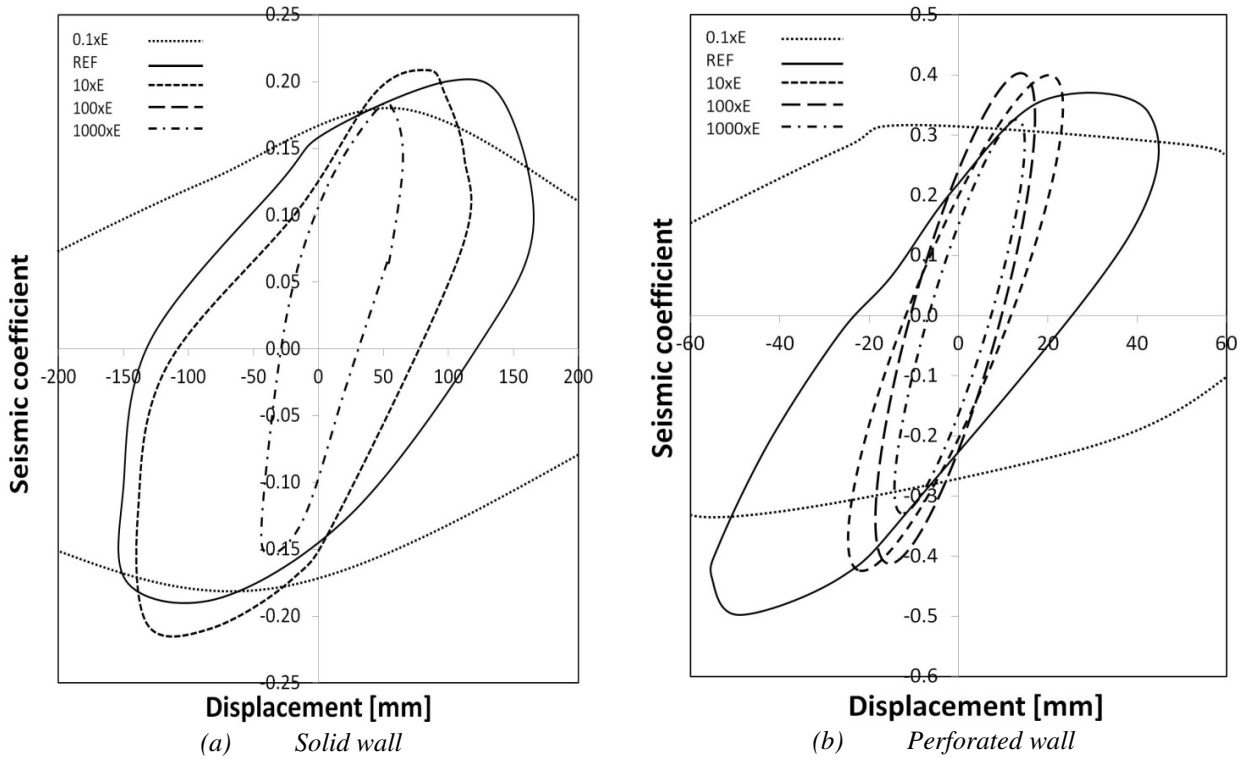


Figure 9- Comparison of envelope displacement-seismic coefficient with different MDF stiffness values for out-of-plane behaviour

The capacity curves obtained with PA were plotted to compare the response of the building featuring floor diaphragms with different stiffness values, see Figure 10 for the E-W direction and Figure 11 for the N-S direction. The results were in accordance with those obtained using THA, with larger capacities corresponding to stiffer floor diaphragms and with a weak structure when the floor diaphragms were flexible, being the influence of the stiffness in the third branch of the curves not as relevant as it was for the first and second branches. The N-S direction was less sensitive to this parameter due to the stiff solid walls, which govern the behaviour in the N-S direction. In addition, the diaphragm in the N-S direction is stiffer than in the E-W direction because of the smaller span-to-width ratio in the N-S direction compared to the span-to-width ratio in the E-W direction. Consequently, the near-rigid diaphragm condition was achieved with smaller values of stiffness in the N-S direction (10xE) than in the E-W direction (100xE).

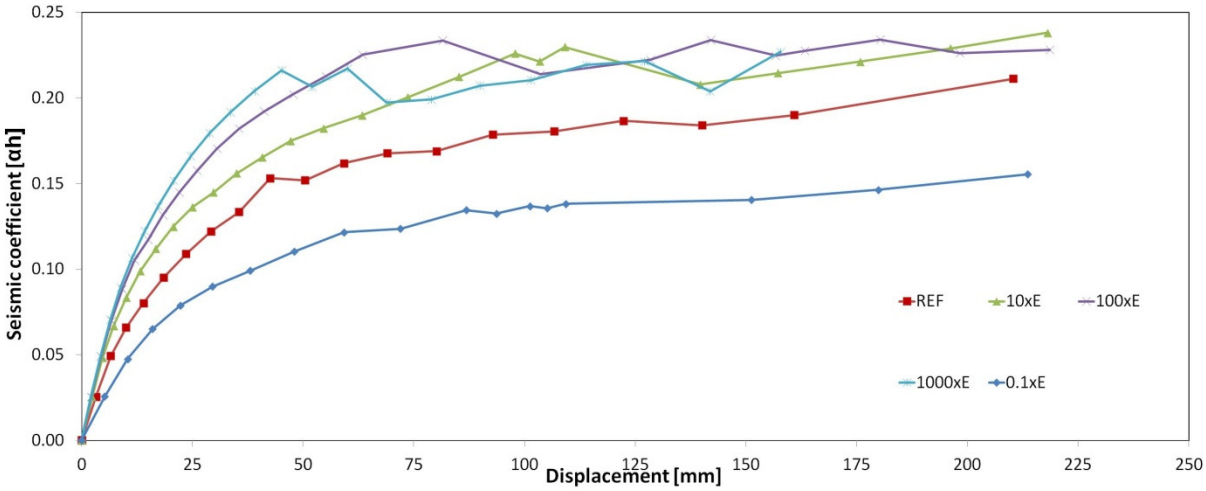


Figure 10 - Comparison of capacity curves with different MDF stiffness values in the E-W direction

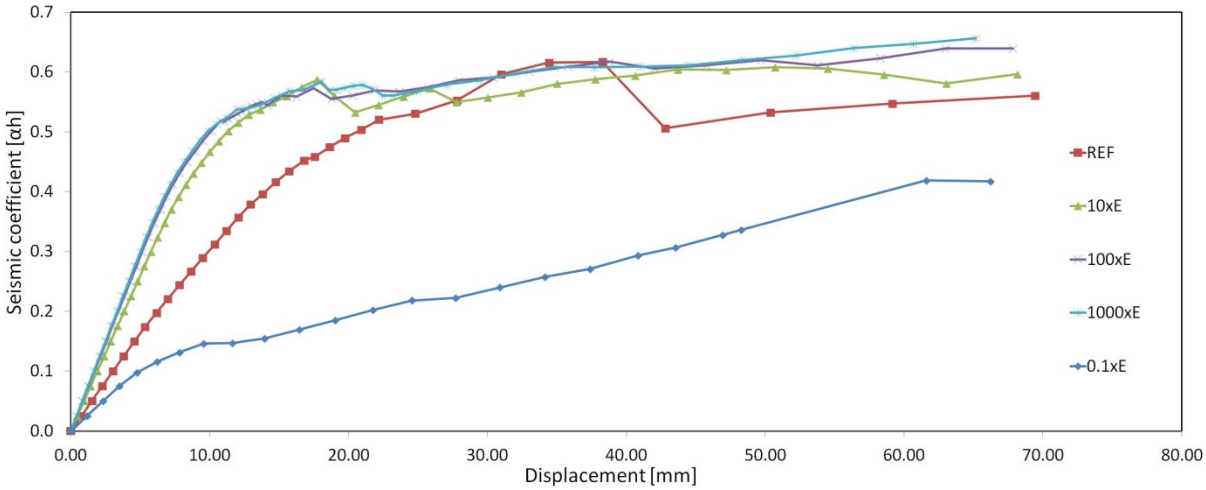
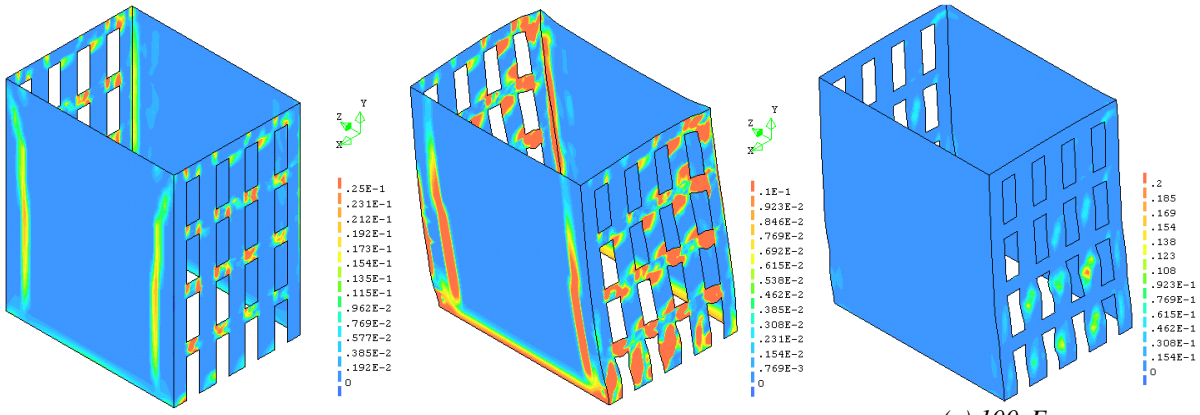


Figure 11 - Comparison of capacity curves with different MDF stiffness values in the N-S direction

The ultimate principal tensile strains are shown in Figure 12 for (a) 0.1xE, (b) reference model and (c) 1000xE. The model with the least stiff floors behaved in the same way as the reference model, with damage in the spandrels and vertical cracks between the solid wall and the perforated wall. On the contrary, the model with the stiff floor behaved differently, shifting the failure mode of the building from failure in the spandrels to failure in the piers of the second floor, reaching larger strain levels which corresponded to larger applied loads.



(a) 0.1xE *(b) reference* *(c) 100xE*

Figure 12 - South-west perspective showing the inner side of the walls at the ultimate principal tensile strains

6 CONCLUSION

The THA method was shown to be more precise than PA, providing a good representation of dynamic loading on the building and being more robust. This advantage of THA over PA is especially noticeable when irregular buildings are under study as these buildings might give different results depending on the direction on which the structure is loaded. However, THA is time and computationally consuming, and the results obtained are extensive and not always easy to interpret, especially in regards to the definition of the building collapse point. The results obtained using PA were mostly adequate in comparison with the results obtained using THA, in terms of maximum capacity, displacement levels and failure modes. Identification of the governing failure mode based on the PA results was possible, while the local damage or localised failures were not diagnosed with the results from PA. In addition, the out-of-plane failure on the fourth level piers observed in the reduced scale model was not identified using either THA or PA, see (Candeias et al. 2004; Mendes & Lourenco 2010) for more details.

PA was carried out independently in two orthogonal directions, in contrast with THA that was performed under biaxial excitation. The results obtained using PA are therefore sensitive to the direction in which the building was loaded, which may cause some problems when interpreting the results, as stated in Section 4 Pushover analysis (PA) of the reference model. The loading pattern applied has an important influence on the results, and further investigation using more advance and precise loading patterns is recommended.

The results obtained with the parametric analysis on floor diaphragm stiffness demonstrate that this parameter is crucial to the seismic response of URM buildings. Stiff floor diaphragms can significantly increase the capacity of the building, reducing the displacements experienced and ultimately resulting in changed failure modes.

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