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Chapter 4

Earthquake Design and Assessment of Masonry Structures: Review and Applications

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Abstract

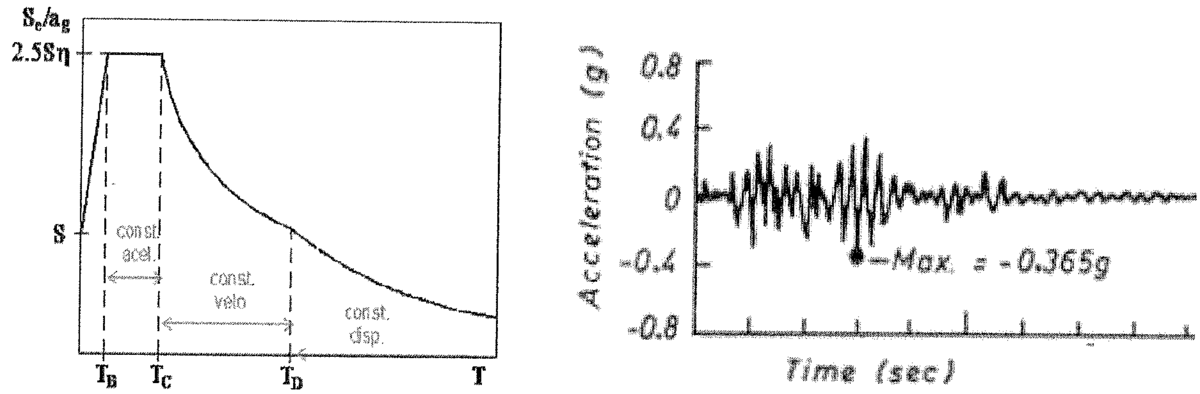
The seismic response of buildings is particularly difficult to characterize. This chapter provides a review of the seismic analysis of masonry structures, incorporating or not box-behaviour. The possibilities are discussed and selected results are presented. The knowledge seems to be consolidated in the case of box behaviour, but considerable efforts are still needed when box behaviour is not present. In this case, several comparisons are provided between push-over methods and time integration dynamic analysis.

Keywords: masonry structures, earthquake engineering, non-linear analysis, push-over methods.

1 Introduction

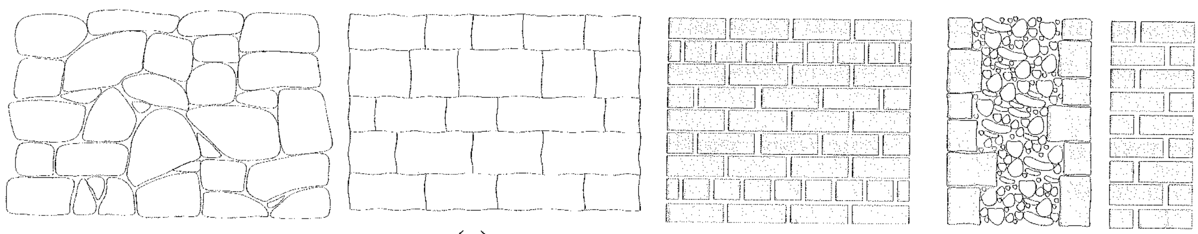
The seismic response of buildings is particularly difficult to characterize due to its nature, the low number of strong events in a given location, site effects, attenuation laws, the non-linear response of the structure, the relevance of execution defects, and many other factors. The seismic action (Figure 1) is usually defined in codes via elastic response spectra, which is the graphical representation of the maximum value of the response for a SDOF system S as a function of the period T. From the elastic response spectrum synthetic accelerograms can be generated, which provide the time history of accelerations at the foundations. Recorded accelerograms from real earthquakes also exist, their most relevant characteristics being the amplitude, the frequency contents and the duration.

This chapter presents a review on earthquake design and assessment of masonry structures, together with applications of different methodologies. Masonry is a heterogeneous material that consists of units and joints. Units are materials such as bricks, blocks, ashlars, adobes, irregular stones and others. Mortar can be clay, bitumen, chalk, lime/cement based mortar, glue or other. Some examples of masonry materials are shown in Figure 2. The huge number of possible combinations generated by the geometry, nature and arrangement of units as well as the characteristics of mortars raises doubts about the accuracy of the term "masonry". When the walls of ancient constructions are of small width, stone units could be of the full width (bond stone or through stone). If the walls are very thick, ashlars would only be used for the outer leaves and the inside can be filled with irregular stones or rubble, or more than one leaf of masonry can be used.



(a) (b)

Figure 1: Possible representation of the seismic action: (a) response spectrum; (b) synthetic or recorded accelerogram



(a) (b)

Figure 2: Different masonry bond: (a) bond in the wall face; (b) internal structure

Nevertheless, the mechanical behaviour of the different types of masonry has generally a common feature: low tensile strength. This property is so important that it has determined the shape of ancient and modern masonry constructions. In the case of seismic loading, it is certain that non-linear behaviour is triggered at early stages of loading and linear elastic analysis seems *not* to be an option. Therefore, the traditional design and assessment method of modal superposition, possibly with a 3-degree-of-freedom system per floor, is not applicable. The alternative options seem to be push-over methods, as recommended in most codes for earthquake safety assessment, or non-linear time integration methods, which is a complex and time consuming tool hardly available for practitioners.

Another very relevant property in the case of seismic loading is the presence of floors that provide diaphragmatic action and the so-called "box-behaviour". This possible feature holds for ancient and modern unreinforced masonry buildings, requiring different models of analysis, addressed next.

2 Masonry Structures with Box Behaviour

Modern masonry buildings usually adopt solutions for the slabs that provide considerable in-plane stiffness. This is done by using monolithic solutions for the floors, in concrete and steel, and also by establishing an effective connection between slabs and walls. Moreover, many existing buildings originally constructed with timber floors are capable of providing diaphragmatic actions or have been rehabilitated by stiffening the floors and by providing adequate connections.

The effect of floor diaphragms combined with the in-plane response of structural walls provides box behaviour to the building, which usually leads to good performance of the structure when subjected to earthquakes. The first assessment method for seismic analysis of masonry buildings was developed under this simple hypothesis. This early attempt was then the seed for more sophisticated methods recently developed. Next, a review is made on the development and application of recent analysis methods.

2.1 The POR Storey Mechanism Method and the Seismic Codes

As result of research studies in former Yugoslavia [1] and also as a consequence of the 1976 Friuli earthquake, the POR method was legally introduced in the Italian region of Friuli-Venezia Giulia, to assess the seismic performance of existing masonry buildings [2]. This method was formulated according to the following basic hypotheses: constant wall thickness in each floor; in-plane rigid floor diaphragm behaviour; only translation in the pier panel ends; elastic-perfectly plastic behaviour of panels, with a predefined ductility; and constant elastic stiffness of each pier.

In practice, it was through an annex of the Italian code Circolare n. 21745/1981 [3] that a seismic evaluation procedure for masonry buildings was first introduced in Italy. This method assumes that the dominant mechanism of collapse of the building is given by shear failure of the pier panels in a critical storey (storey mechanism) and the response in terms of shear force-horizontal displacement is assessed separately for each storey. Since the original version, several improvements were introduced, namely with consideration of the flexural failure mechanisms [4] and the definition of the effective pier height [5].

Despite the simplicity of the POR method, it was used until very recently in the analysis of existing buildings and in the design of new buildings, in Italy. Its limitations, namely the consideration of an independent storey mechanism, and the 2002 Molise earthquake, clearly stressed the need for methods that consider the overall response of the masonry structures.

Despite the recent introduction of the Eurocodes in Europe, Italy introduced recently its own earthquake codes for masonry buildings, namely the OPCM 3274/2003 and its revision OPCM 3431/2005 [6]. These codes are aligned with the philosophy of Eurocode 8 (EC8) [7], but collect aspects provided by the research, experience and application of design rules in Italy. Lagomarsino [8] identifies two aspects that seriously penalize the construction with structural masonry in the provisions given in the first version of OPCM, which corresponds in general to the current version of EC8: a low behaviour factor that renders unreinforced masonry impossible and in opposition with experimental findings; analysis methods and assessment criteria excessively complex and usually not available to practitioners. This section thus addresses adequate tools and provisions for unreinforced masonry structures subjected to seismic loading, with box behaviour.

2.2 Modern Analysis Methods

The POR method appeared in a context of the first steps in computational approaches, and severe simplified assumptions. Today, advanced tools for structural analysis allow a detailed modelling of the behaviour of structures. Software based on the finite element method (FEM) is the most evident example of this kind of tool. FEM is a popular approach for structural analysis software, usually considering isotropic and homogeneous materials, and continuum models. Discontinuum models can also be used but the computational effort is rather large, if applied at a full building level [9].

Methods inspired in the POR method and based on macro-elements have been developed, particularly in Italy. These methods seem the most appropriate for design and assessment of masonry buildings, given their widespread use in commercial software, the simplicity of modelling, the straightforward interpretation of results and the accuracy demonstrated in different validations.

For a correct simulation of the masonry panels failure mechanism and their behaviour different types of macro-elements have been developed, such as the formulations proposed by Gambarotta and Lagomarsino [10] and Magenes and Della Fontana [11] shown in Figure 3a-b, which are incorporated in the 3Muri [www.stadata.com] and ANDILWall/SAM II [www.crsoft.it/andilwall] computer codes, respectively. While the 3Muri formulation is based on the kinematic equilibrium of the macro-elements according to the panel degrees of freedom, the SAM II creates an equivalent frame idealization for a global analysis.

The 3Muri and SAM II computer codes perform the safety verification by a nonlinear static (pushover) analysis (Figure 4) that simulates the evolution of the structural condition during the earthquake, through application of incremental horizontal forces until collapse. The behaviour of the structure is represented by the so-called "capacity curve", which represents the value of the base shear versus the displacement of a control point (usually the mass centroid of the roof slab). Recently, Marques and Lourenço [12] carried out a benchmarking process in which the 3Muri and SAM II computer codes were compared. Good agreement of the results was obtained for a pushover analysis on two buildings.

An alternative general purpose software for push-over analysis is SAP2000 computer code [www.csiberkeley.com], based on a frame idealization. In this case, it is necessary to define the possible locations and types of plastic hinges that might develop along each element, to describe possible failure mechanisms (flexural or shear), as shown in Figure 3c [13].

In addition to 3Muri and SAM II, RAN [14] is another Italian macro-element method, which can be programmed in a worksheet. RAN allows a global nonlinear analysis of collapse (Figure 3d).

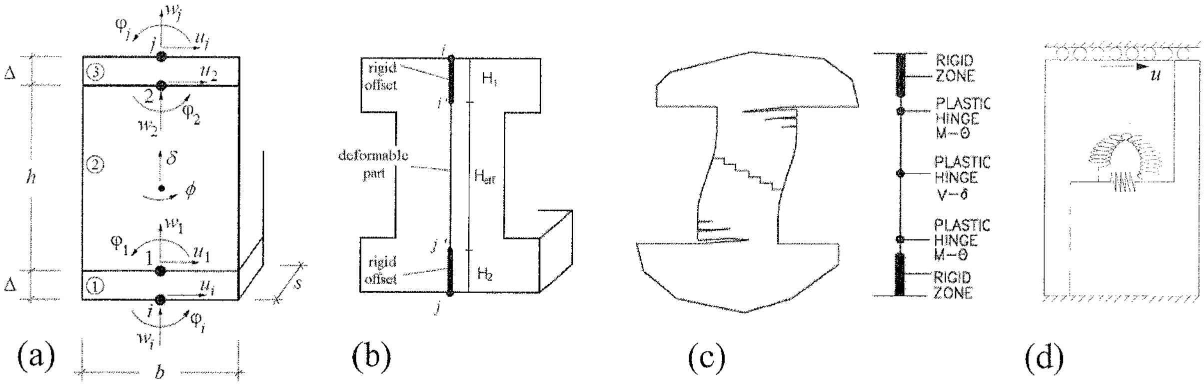


Figure 3: Macro-element in different methods: (a) 3Muri; (b) SAM II; (c) SAP2000 (d) RAN

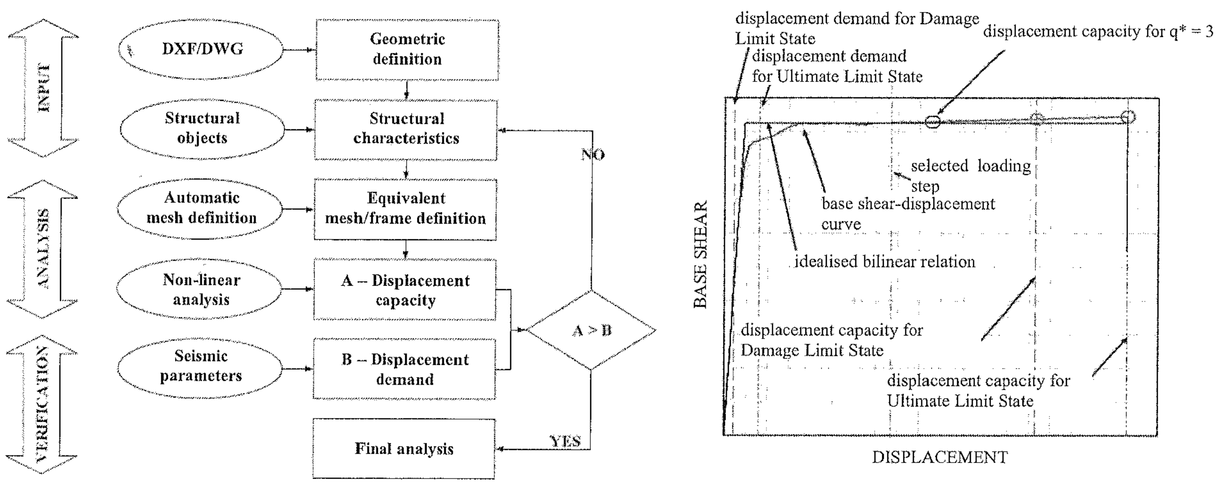


Figure 4: Seismic verification by nonlinear analysis: flowchart and capacity curve

Inspired by the RAN method hypothesis, a computer code was recently developed by Marques *et al.* [15]. Several buildings have been analysed and the corresponding maximum elastic base shear was similar to the one calculated with 3Muri and SAM II. However, the safety assessment with RAN is based on the comparison of the maximum elastic base shear predicted versus the seismic elastic force reduced by the behaviour factor. This force-based safety verification implies

2.3 The Role of Energy Dissipation Capacity

Despite the recent introduction of methodologies that allow consideration of the nonlinear reserve capacity of structures in displacements, namely by a pushover analysis, buildings are traditionally designed for earthquakes using force-based approaches and linear elastic analysis. The consequence is that safety assessment of existing structures is often incorrect and behaviour factors are required.

The behaviour factor q of a given structure is normally defined as the ratio between the F_y strength of an ideal bilinear system equivalent to the true nonlinear, and the maximum elastic base shear $F_{el,max}$. According to Magenes [16], after reaching the strength capacity (shear or flexural) for an element according to a linear elastic analysis, the deformation capacity into the nonlinear regime, even if limited in some cases, is sufficient to allow the system to sustain an increasing seismic load, due to the increase of forces on other structural elements. This force redistribution possibility is already accepted for framed structures in EC8, and for masonry structures in the Italian code OPCM 3431, for which the definition of the behaviour factor considers an *overstrength ratio (OSR)*. Thus, the definition of q should be [16], see also Figure 5c:

$$q = \frac{F_{el,\max}}{F_{el}} = \frac{F_{el,\max}}{F_y} \cdot \frac{F_y}{F_{el}} = q_0 \cdot \frac{F_y}{F_{el}} = q_0 \cdot OSR \qquad (1)$$

where F_{el} represents the base shear at which the first element would reach its strength capacity (shear or flexural) according to a linear elastic analysis.

In the case of EC8, a range of behaviour factor values is provided for masonry structures but, in each country, a maximum value of this factor can be defined in its National Annex. EC8 recommended values are the lower limits of the possible ranges. In the case of unreinforced masonry buildings, regular in elevation and with two or more storeys, the Italian code allows us to adopt a elastic force reduction factor value 2.4 times greater than that allowed by EC8 (Table 1).

Building configuration		EC8	OPCM 3431		
Type of construction	N. of storeys	Behaviour factor q (*)	Basic value q_0	*OSR*	Behaviour factor q
Unreinforced masonry building; regular in elevation	One	1,5 – 2,5	2,0	1,4	2,80
	Two or more			1,8	3,60
Unreinforced masonry building; non regular in elevation	One	1,5 – 2,0	1,5	1,4	2,10
	Two or more			1,8	2,70
Reinforced masonry building; regular in elevation	One	2,5 – 3,0	2,5	1,3	3,25
	Two or more			1,5	3,75
Reinforced masonry building; non regular in elevation	One	2,0 – 2,4	2,0	1,3	2,60
	Two or more			1,5	3,00

* The upper limit values of q for use in a country may be found in its National Annex. However, the EC8 recommended values are the lower limits of the ranges.

Table 1: Behaviour factor values for masonry buildings

In order to obtain additional information on the range of values for the behaviour factor distinct researches were performed at University of Pavia [16] and at ZAG in Slovenia [17]. Tomaževič [17] tested on a shaking table a series of models representing masonry buildings of two different structural configurations, typical for central Europe (a three-storey apartment house and a two-storey terraced house), and two different types of masonry materials (Figure 5a-b). Magenes [16] conducted also a numerical analysis using the SAM II method, where nine plan configurations of plain masonry buildings from one to three storeys were tested. Then, the OSR factor was computed from the capacity curve obtained for each building. Figure 5d reports the histogram of the values of OSR (F_y/F_{el}) that were obtained for the sample of two- and three-storey unreinforced masonry buildings.

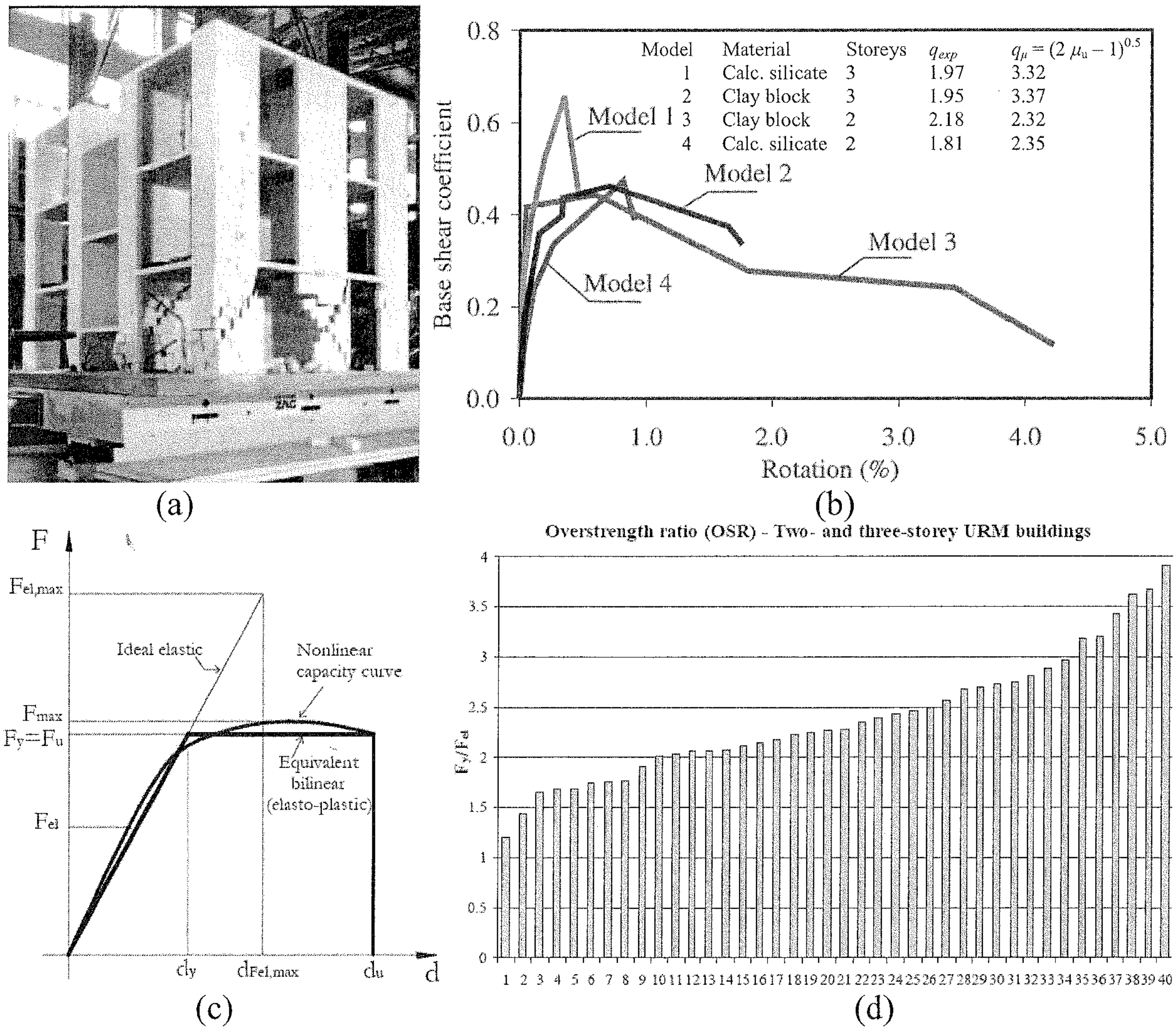


Figure 5: Research on the behaviour factor for plain masonry buildings: (a) shaking table test at ZAG on 1:5 model and (b) resistance curves of models of 2- and 3-storeys [17]; (c) reference parameters for definition of q_0 and *OSR* and (d) calculated OSR values for 2- and 3-storey buildings [16]

Based on the shaking table experimental results, particularly on the computed values of ductility and the structural behaviour factor q_μ calculated on the basis of damage-limitation requirements, and in the OSR values obtained from the numerical analysis, the need to adopt higher behaviour factor values seems clear, particularly in the case of EC8. This is addressed in detail in the next section.

2.4 Case Study

To discuss the possibilities of construction with unreinforced masonry in Portugal, the seismic safety of buildings with one up to three storeys, based on a pushover analysis carried out in the 3Muri computer code, is considered next. The building configurations studied are shown in Figure 6, namely a one-storey module, and two- and three-storey buildings for semi-detached houses.

The buildings were modelled with a new clay block "cBloco" developed in order to fulfil the new thermal requirements [www.cbloco.com.pt] and bedded on two strips of mortar. Given the unit and bonding types, the compression and pure shear strengths were reduced by 50% according to applicable clauses of EC6 [18]. The properties of the materials of the masonry are given in Table 2.

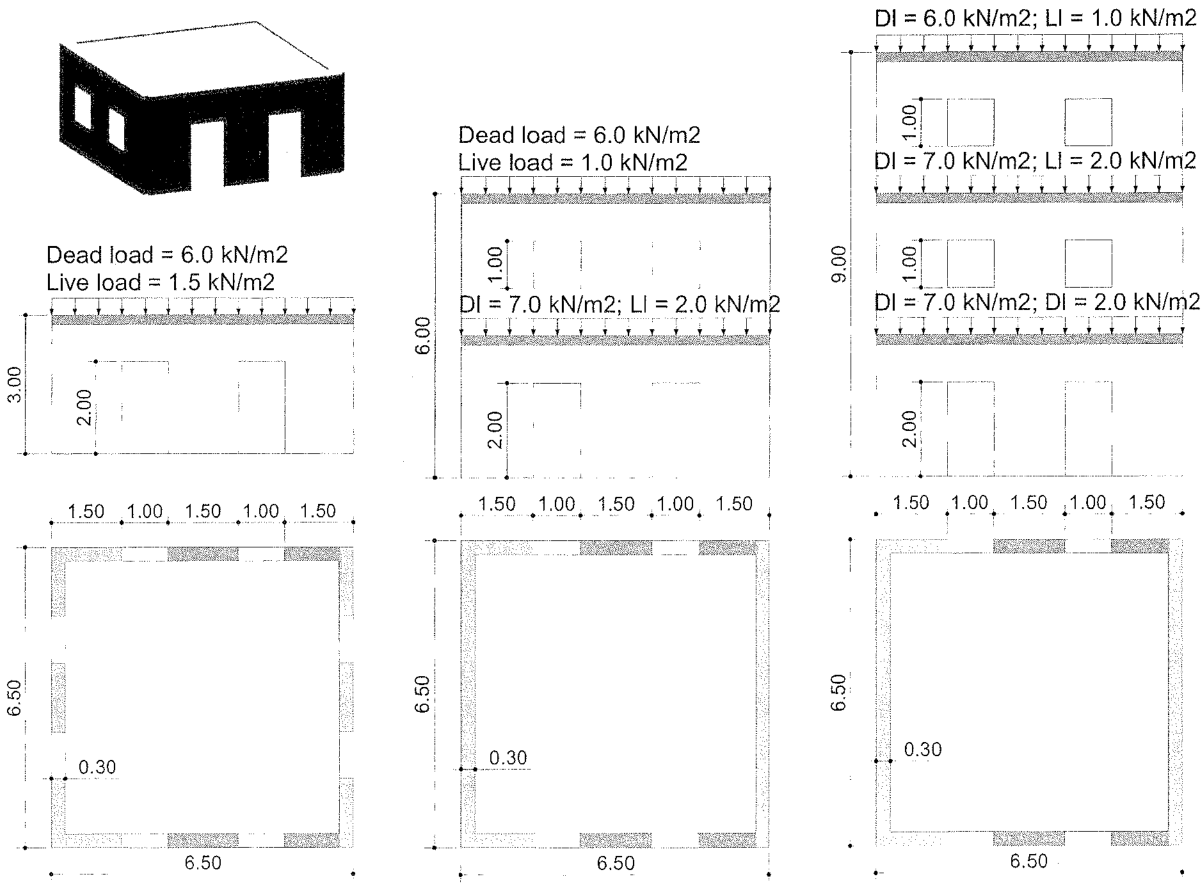


Figure 6: Building configurations studied

UNITS	Type according to EC6	Clay units of Group 2
	Compressive strength, f_b	12.0 MPa
MORTAR	Type according to EC6	M10
MASONRY	Specific weight, γ	17.0 kN/m^3
	Compressive characteristic strength, f_k	2.56 MPa
	Pure shear characteristic strength, f_{vk0}	0.15 MPa
	Normal elasticity module, E	2560 MPa
	Tangential elasticity module, G	1024 MPa

Table 2: Masonry properties

Figure 7 illustrates the ultimate response in terms of deformed configuration and damage of the three buildings, where it can be observed that the collapse mechanisms are essentially induced by flexure, while plastic mechanisms by shear are only found for the three-storey building in spandrels adjacent to the first slab.

Based on the requirements for earthquake resistance imposed by the Italian code OPCM 3431/2005, and assuming the seismic parameters defined in the Portuguese Annex to EC8, the possibility to construct the studied buildings in Portugal was evaluated using 3Muri, which is given in Figure 8. The results indicate that the Portuguese Annex is over conservative.

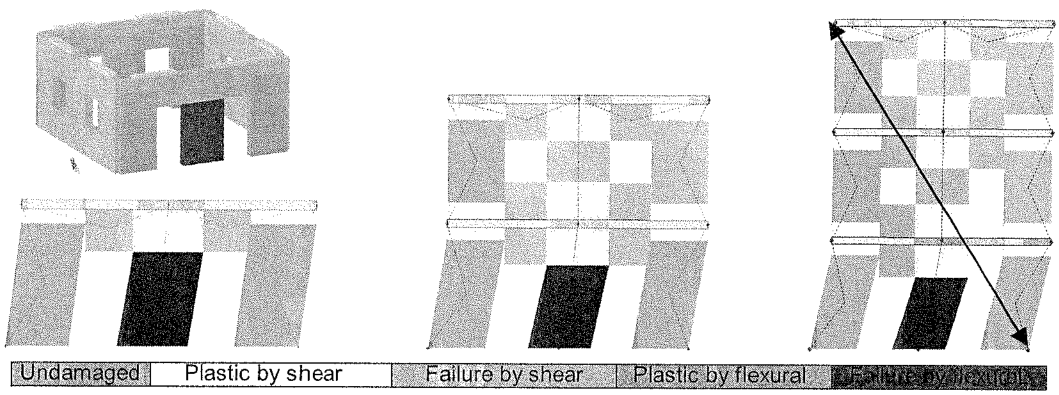
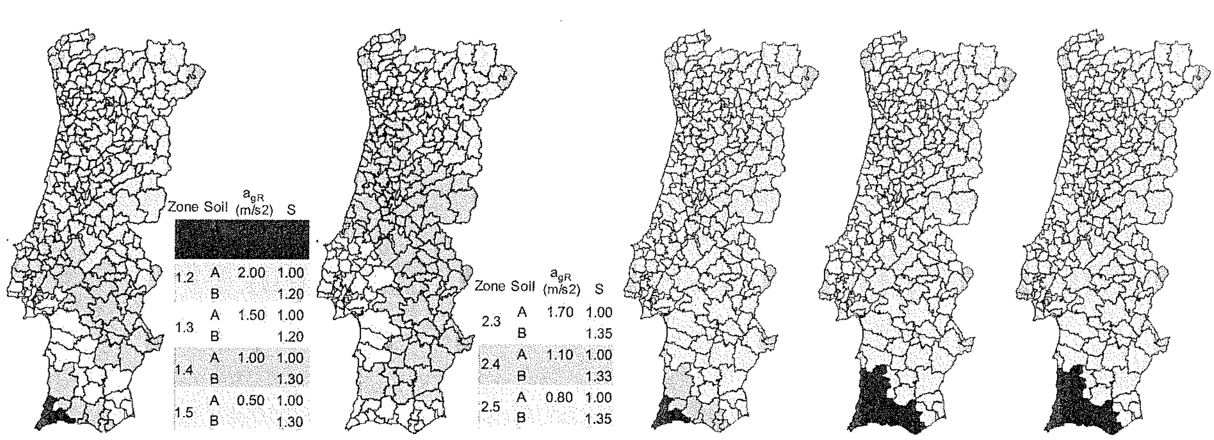


Figure 7: Assessment of ultimate damage and deformed configurations

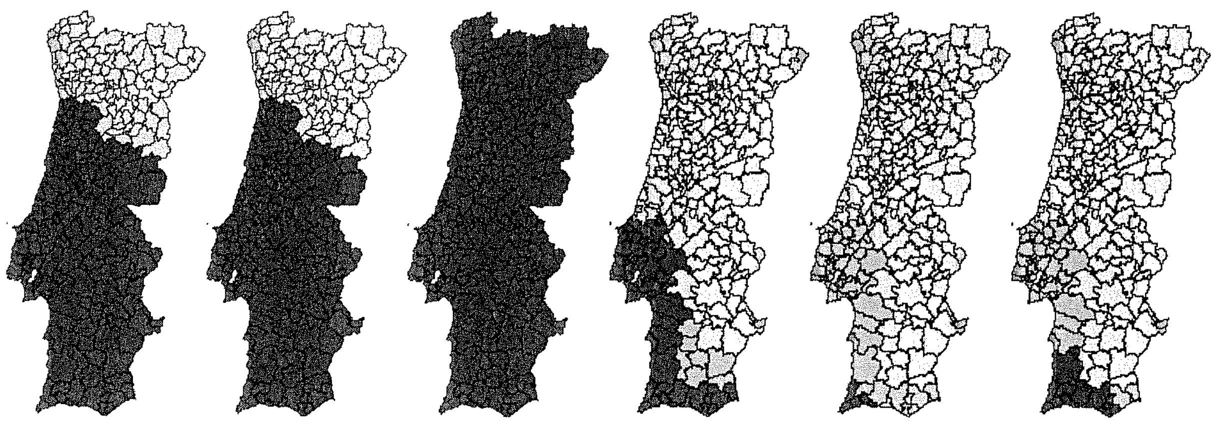


(a) Seismic zones (b) 1-to-3 storeys

Unsafe Safe in soil type A Safe in soil types A and B

The proposed buildings can be constructed in most of Portugal, with only absolute restrictions in Seismic Zones 1.1 in general, and 1.2 for buildings of two- and three-storeys. Performing an elastic analysis adopting a behaviour factor of 1.5, as recommended by EC8, the safety verification is over conservative, as shown in Figure 9a. A better correspondence between the pushover and linear analysis is achieved by assuming the behaviour factor values proposed by OPCM 3431, as shown in Figure 9b. In the case of the regular building configuration adopted, behaviour factors of 4.0, 3.0 and 3.5, respectively for the one-to-three storey buildings allow a perfect match between the linear and nonlinear analysis.

From Figure 7 it is possible to observe that the displacement at the top of the 1-to-3 storey buildings is determined by the displacement of the first storey. Thus, for higher buildings, the increase of maximum deformation d_u is not proportional to the elastic displacement d_y (see Figure 5c). This explains why the ductility decreases with elevation, which provides a decrease of the basic value of the behaviour factor q_0. On the other hand, the overstrength ratio (F_y/F_{el}) increases with the number of storeys. Further studies are needed to explain this dual-tendency. In the numerical analysis, it was necessary to create a building of five storeys to observe a deformation mechanism involving also the second storey, and activating the shear failure of spandrels adjacent to the first slab, with the corresponding loss of strength (Figure 10).



(a) Portuguese Annex to EC8 (b) OPCM 3431

Unsafe Safe in soil type A Safe in soil types A and B

Figure 9: Possibility of construction in Portugal using a linear elastic analysis

3 Masonry Structures without Box Behaviour

Differently from the structures considered in the previous section, unreinforced masonry structures without box behaviour have shown poor performance in many past earthquakes (*e.g.* Figure 11). The reasons for the poor performance are the inherent brittleness, lack of tensile strength, lack of ductility, the flexible floor diaphragms and the lack of connection between the structural elements.

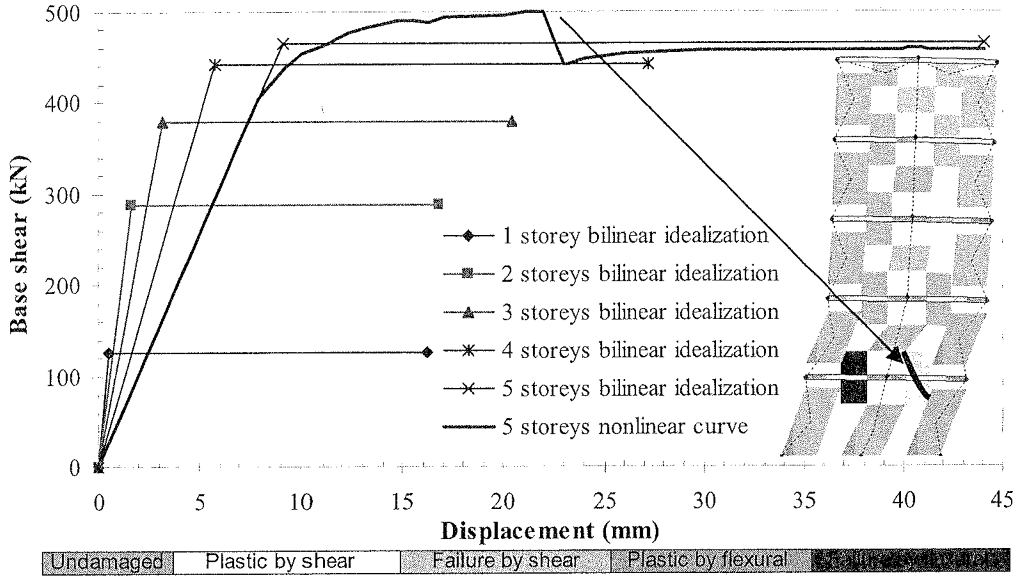


Figure 10: Base shear-displacement relations for buildings with increasing height and ultimate state of a 5-storey building

Research conducted on flexible diaphragms, *e.g.* [20-23], showed that flexible diaphragms lead to the following: (a) supports at floors behaving as a spring support (Figure 12); (b) large deformation capacity and high strength of the floor with respect to its mass. Failure mechanisms of flexible diaphragms are related to the lack or weak connections between the masonry walls and diaphragms. The diaphragm may slip off its supports and collapse if the diaphragm is not appropriately connected to the masonry walls; (c) a highly non-linear hysteretic behaviour when the peak ground acceleration is high; (d) strengthening of the horizontal diaphragms as a natural solution even if an increase of the in-plane stiffness *per se* is not enough to improve the global response of the building.

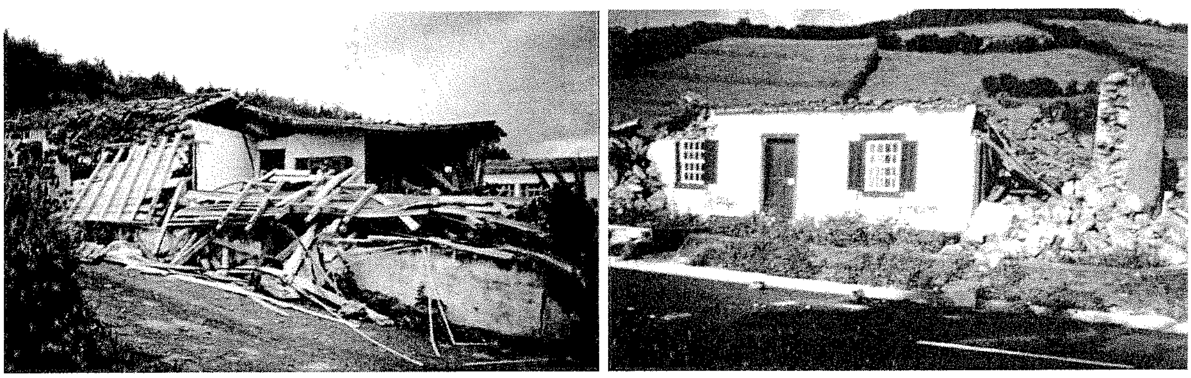


Figure 11: Examples of the URM buildings damage, Azores 1998 [19]

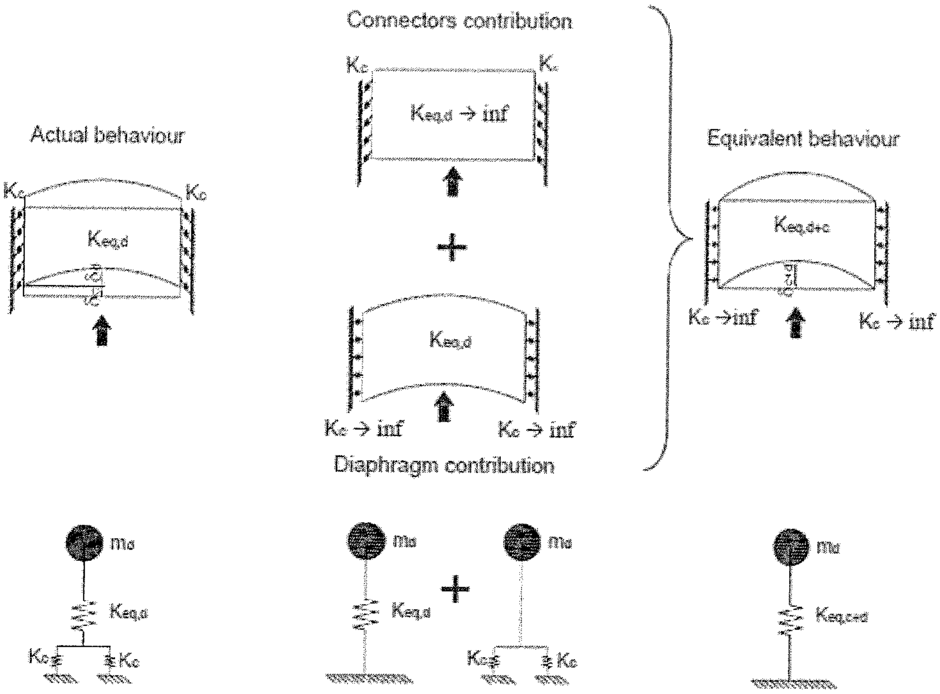


Figure 12: Schematic contributions of connectors and diaphragm stiffness to the overall floor system stiffness [20]

University of Minho and the National Laboratory of Civil Engineering (LNEC, Lisbon) are carrying out experimental tests on a URM building with flexible floors. The experimental model has 4 storeys, masonry walls and timber floors. The walls are made using stone masonry (limestone and lime mortar). In the construction of the timber floors, medium-density-fibreboard (MDF) panels connected to a set of timber joists were used. It is noted that the experimental model was prepared on 1:3 reduced scale (Figure 13). The first results of the dynamic identification tests (output only) showed that the flexibility of the floors have significant contribution on the dynamic properties of the structure (Figure 14). In the mode shapes, it is clearly observed that the floors do not completely prevent the out-of-plane displacement of the masonry walls, causing local modes of the façades.

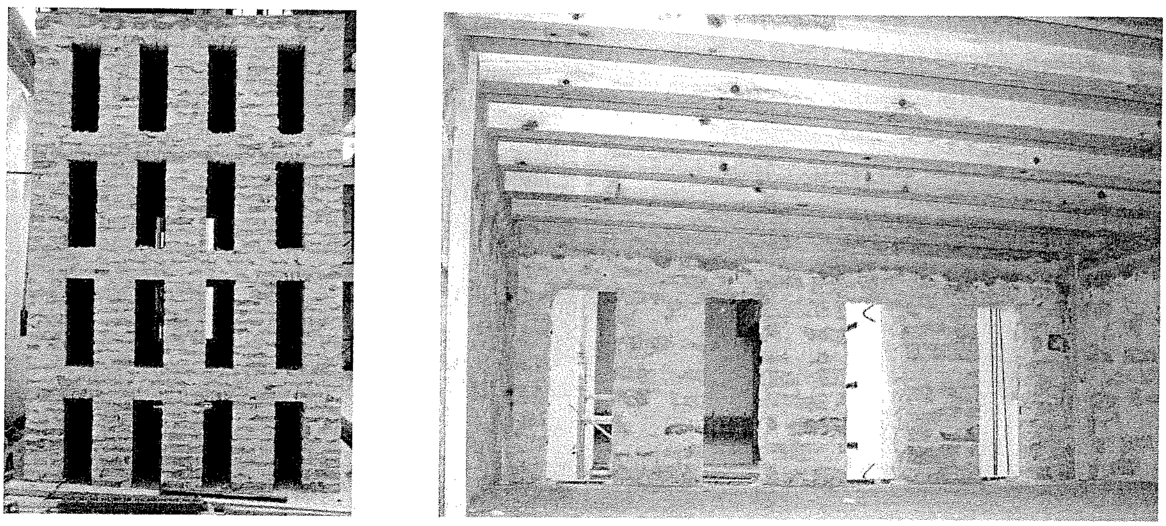


Figure 13: Experimental model of an URM building with flexible floor diaphragm

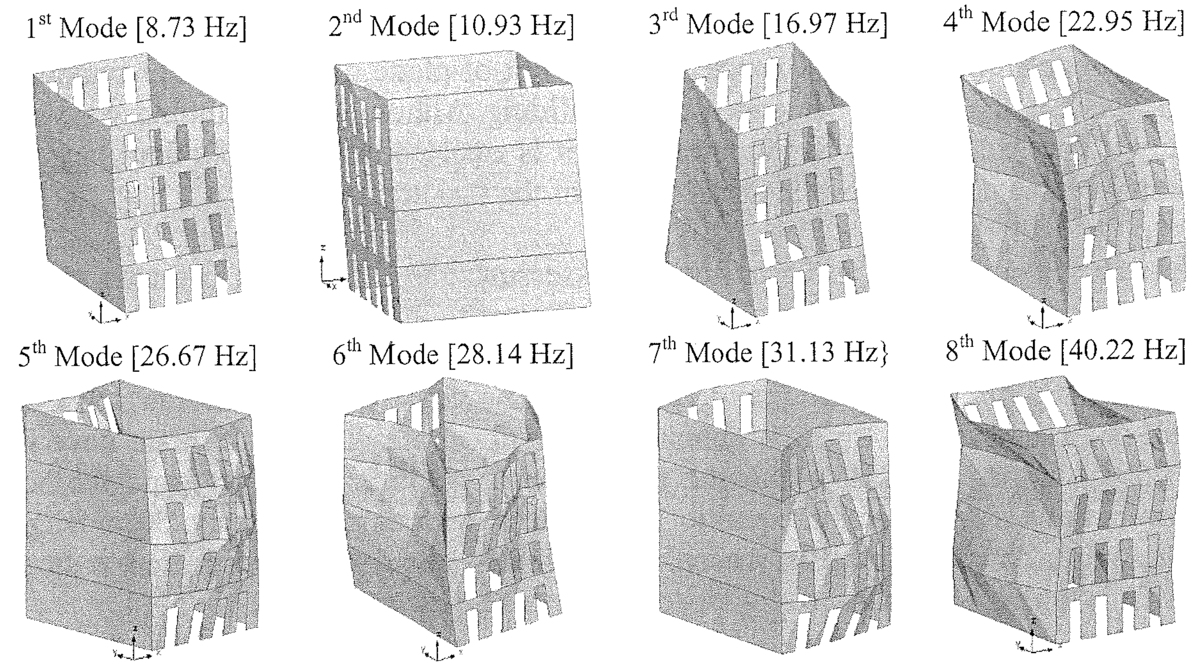


Figure 14: Mode shapes estimated trough dynamic identification

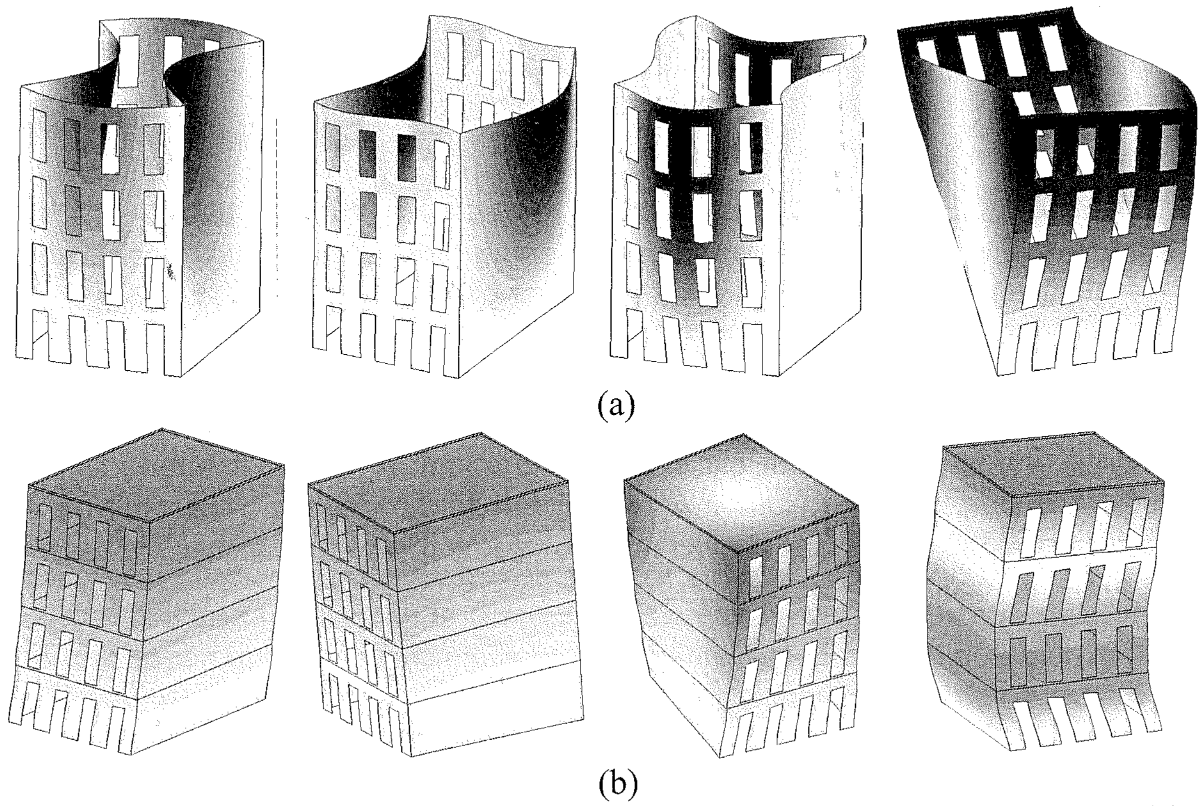
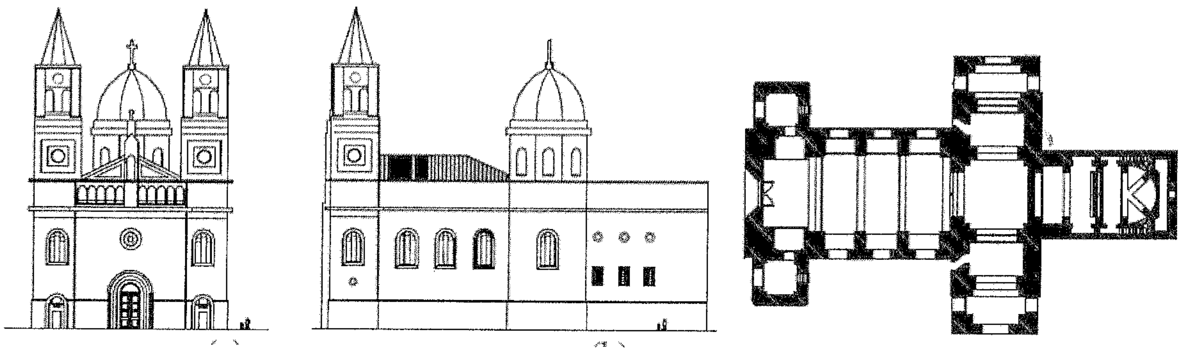


Figure 15: Numerical mode shapes of an URM building: a) without floors, b) with "rigid" floors diaphragm

A simplified numerical study was also made. Here, the eigenmodes were calculated, considering two hypotheses: (a) structure without floors; (b) structure with "rigid" floors. The results are clearly different. In the first case, the structure presents the typical out-of-plane modes of the masonry walls, as is shown in Figure

15a. The numerical model with "rigid" floors (Figure 15b) presents global eigenmodes, in which it is expected that the out-of-plane displacements have minor relevance in the seismic behaviour of the building. Although the preliminary study is only addressed to the dynamic properties, it is easily understandable that these are directly related to the seismic performance of the structure.

Another interesting example of seismic assessment is given by S. Torcato church (Figure 16) located in the village of S. Torcato, 7 km north from the city of Guimarães (Portugal). The church combined several architectonic styles, like Classic, Gothic, Renaissance and Romantic. The construction started in 1871 and still continues to this day. The dimensions involved are significant: main nave has 57.5×17.5 m^2 and 26.5 m height; the transept has 37.1×11.4 m^2; and the bell-towers have a cross section equal to 7.5×6.3 m^2 with, approximately, 50 m height. The entire church is built in masonry with locally available natural granite stones and dry joints. The walls are composed by three leaves.



(a) (b) (c)

Figure 16: S. Torcato church: (a) main façade; (b) lateral view; (c) plan

In seismic assessment of the S. Torcato church limit analyses using macro-blocks were carried out. In existing masonry buildings partial collapses often occur due to seismic action, generally, with the loss of equilibrium of parts that behave as rigid bodies. The simplified verification with q factor (linear kinematic analysis) for the ULS is verified if the spectral acceleration a_0^* that activates the mechanism satisfies the following inequality:

$$a_0^* \geq \frac{a_g S}{q}\left(1 + 1.5\frac{Z}{H}\right) \tag{2}$$

where: a_g is the ground acceleration; S is the soil factor; Z is the height from the building foundation to the centre of gravity of the weight forces, whose masses generate horizontal forces on the elements of the kinematic chain and which are not efficiently transmitted to the other parts of the building; H is the total height of the building from the foundation; q is the behaviour factor.

In this case study, four mechanisms were defined, based on the inspection and analysis of the structure [24,25]. Figure 17 shows the mechanisms considered in the linear limit kinematic analysis. According to the limit analysis the S. Torcato church is safe and the lowest safety factor is equal to 1.69 (with overturning of the tympanum).

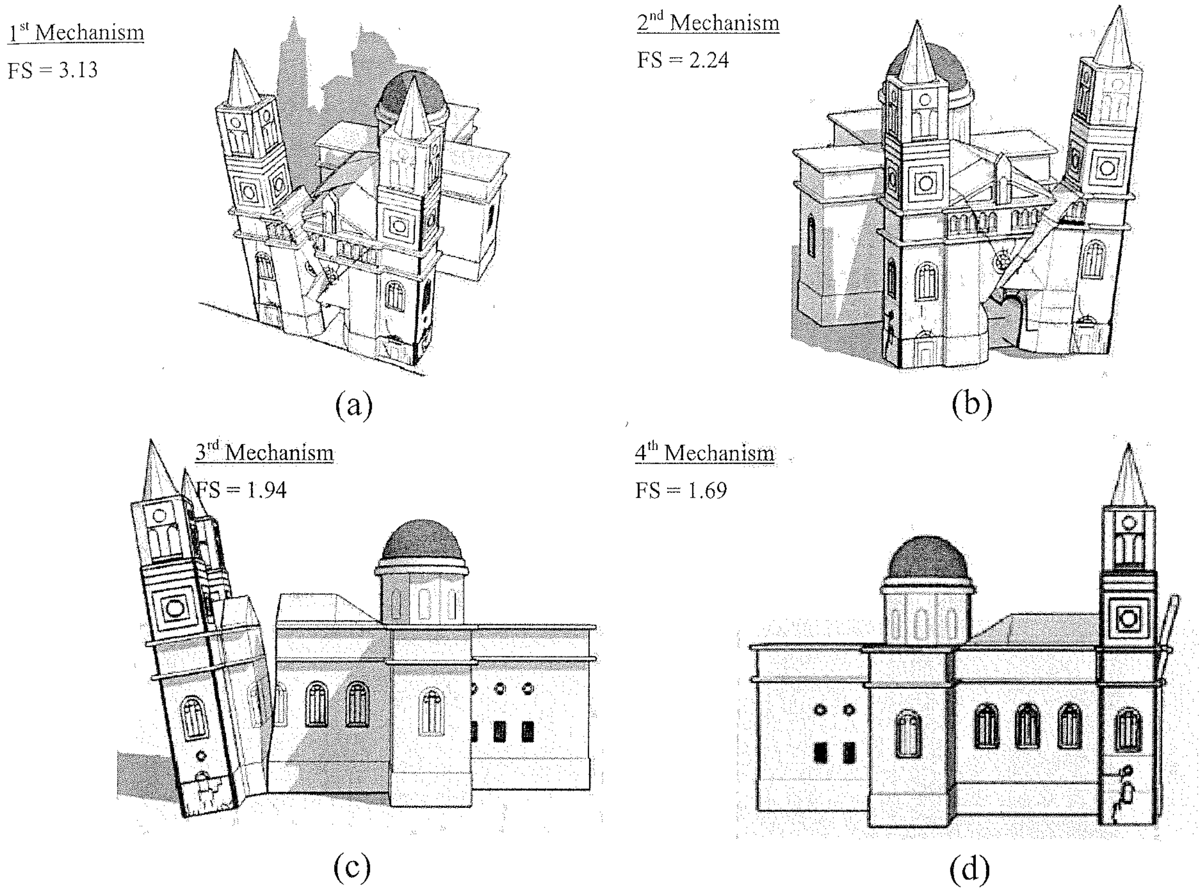


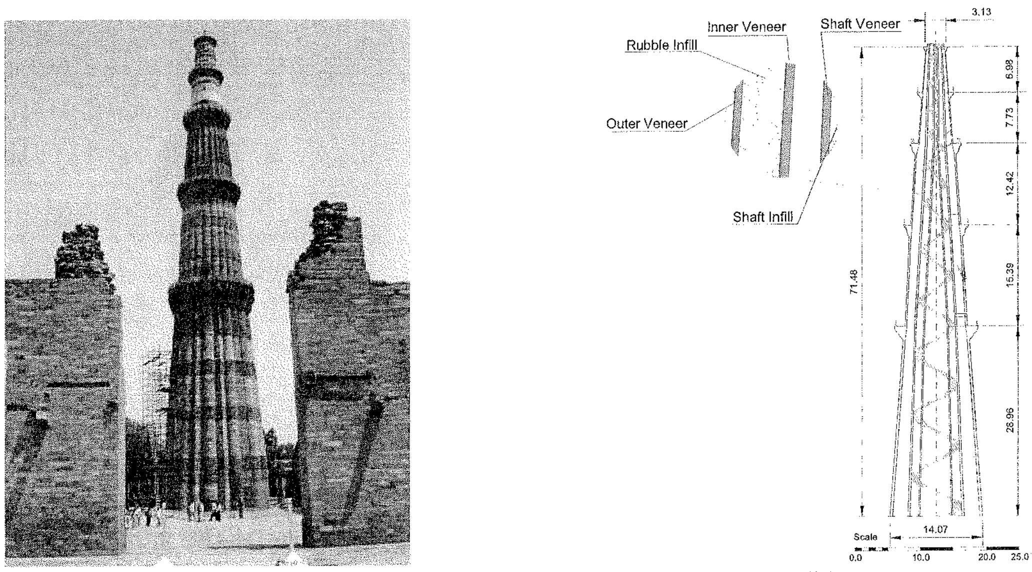
Figure 17: Mechanisms: (a) overturning of the left tower; (b) overturning of the right tower; (c) overturning of the façade; (d) overturning of the tympanum (FS is the safety factor)

In Table 3 the parameters considered in the analysis are presented (α_0 is the load multiplier that activates the local damage mechanism; M^* is the participating mass; e^* is the fraction of the participating mass; a_0^* is the spectral acceleration; FS is the safety factor). Despite the strong capabilities of such a limit analysis and the existence of abacus of possible mechanisms, it is believed that the selection of an adequate collapse mechanism is complex and requires a careful *in situ* inspection. Still, the experience and structural capacity of the practitioner are subjected to a significant demand, as the process is difficult to control and the selection of erroneous mechanisms might result in totally incorrect structural assessment and remedial measures. For this reason, standard push-over methods are next confronted with nonlinear dynamic time integration results for two selected case studies.

	α_0	M^* [kg]	e^* [m/s^2]	Capacity a_0^* [g]	Demand a_0^* [g]	FS
1st Mechanism	0.186	434.37	0.947	0.197	0.063	3.13
2nd Mechanism	0.184	425.45	0.953	0.193	0.086	2.24
3rd Mechanism	0.164	883.01	0.968	0.169	0.087	1.94
4th Mechanism	0.205	33.91	0.982	0.208	0.123	1.69

3.1 The Qutb Minar in New Delhi, India

The Qutb Minar (Figure 18a) is the highest monument in India and one of the tallest stone masonry towers in the world, dating from the 13th century. The cross-section is circular/polilobed, having a base diameter equal to 14.07 m and tapering off to 3.13 m at the top, over a height of 72.45 m (Figure 18b). The tower is composed of an external shell corresponding to a three leaf masonry wall and a cylindrical central core [26]. The core and the external shell of the tower are connected by a helicoidal staircase and by 27 "bracings" composed of stone lintels. The staircase is spiral, disposed around the central masonry shaft, and it is made of Delhi quartzite stone. Each storey has a balcony and the uppermost storey finishes with a platform.



(a) (b)

Figure 18: Qutb Minar: (a) general view; (b) dimensions (in meters)

To evaluate the seismic performance of the Qutb Minar different techniques of structural analyses were performed: (a) non-linear dynamic analysis; b) (non-linear static analyses (pushover analyses). In the analyses different numerical models were considered. Two models were prepared using the Finite Element Method (FEM); both are three-dimensional models but one uses 3D solid elements (Solid Model) while the other one was performed with 3D composite beams (Beam Model). The numerical models were updated with dynamic identification tests [27].

In the FEM models, the physical non-linear behaviour of the masonry was simulated using the Total Strain Crack Model detailed in [28]. Complete details on the analysis can be found in [29]. The dynamic analyses were carried out using five artificial accelerograms compatible with the elastic response spectrum of the Indian Seismic code for Delhi (PGA = 0.20g) [30].

Figure 19 shows the maximum seismic coefficient and displacements for each level with the Beam Model. The average seismic coefficient at the base is 0.16 and increases to 0.18 for the first level. The second balcony has an average seismic coefficient of 0.28, while the third and fourth balconies have an average seismic coefficient of 0.47 and 0.9 respectively. The maximum displacements at the top vary from 0.35 to 0.65 m. This means that the amplification of the seismic loads is concentrated at the last two levels. Displacements of levels 1 to 3 increase practically in a linear way, while displacements of level 5 are almost the double the displacements of level 4. Levels 4 and 5 are the most vulnerable, where the behaviour of level 5 (maximum drift equal to 3.0%) is highlighted.

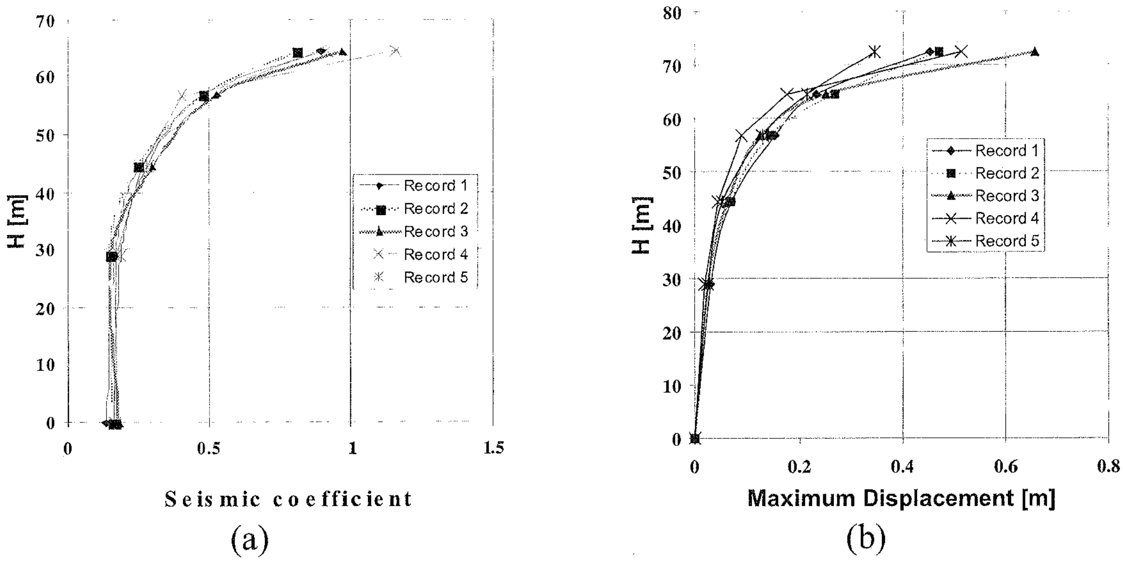


Figure 19: Maximum absolute results along the height of the minaret for dynamic analyses with Beam Model: (a) seismic coefficient; (b) lateral displacement

Pushover analyses were carried out using the numerical models considering a uniform acceleration distribution. The load was applied with increasing acceleration in the horizontal direction and a control point at the top of the tower was considered. Figure 20 shows the capacity curves (lateral displacement – seismic coefficient). Rather similar behaviour was found for the models. It can be observed that the average load factor is 0.20. It is worth noting that the tower collapses by overturning at the base.

In order to study the influence of the distribution of the lateral load into the pushover analyses, new non-linear static analyses were performed. Three different configurations of lateral loads were considered: (a) loads proportional to the mass (uniform acceleration); (b) linear distribution of the displacement along the height; (c) loads proportional to the first modal shape; (d) adaptative push-over analysis, changing the load distribution according to the changes in the first modal shape during the analysis; (e) modal push-over analysis [31]. The results of the pushover analyses do not change qualitatively from what is shown in Figure 20 and the failure mode and displacements' distribution along the height are not in agreement with the non-linear dynamic analysis.

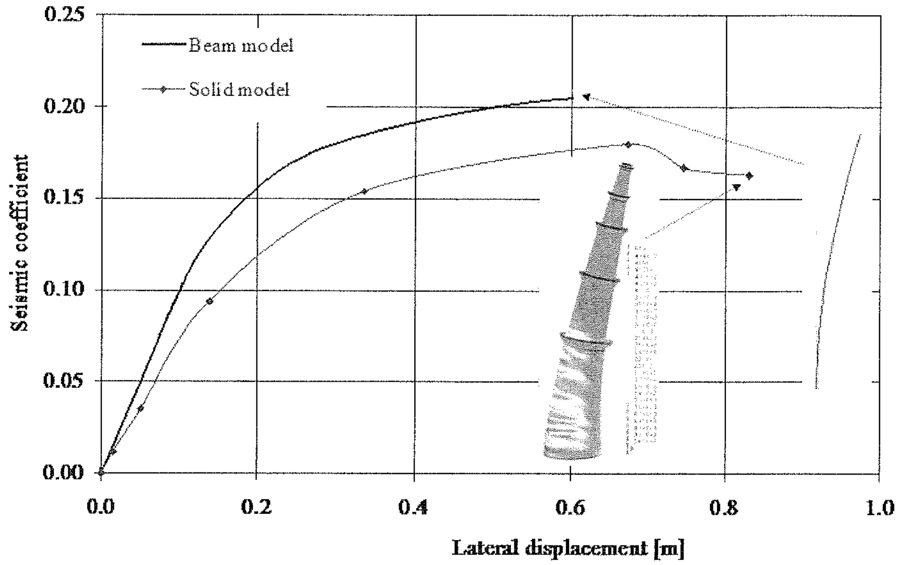


Figure 20: Capacity curve of the pushover analyses proportional to the mass

3.2 "Gaioleiro" Buildings in Lisbon

The "gaioleiro" buildings (Figure 21) were developed between the mid 19th century and beginning of the 20th century, mainly in the city of Lisbon (Portugal), and remain still much in use nowadays. These buildings characterize a transition period from the anti-seismic practices used in the "pombalino" buildings originated after the earthquake of 1755 [32], and the modern reinforced concrete frame buildings. These buildings are four or five storeys high, with masonry walls and timber floors and roof. The external walls are, usually, in rubble masonry with lime mortar.

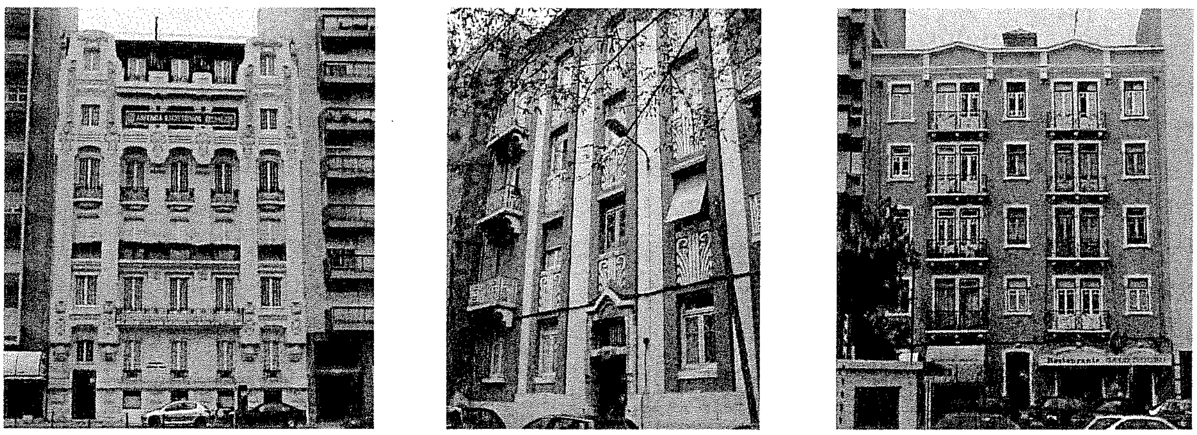


Figure 21: Examples of "Gaioleiro" buildings, Lisbon, Portugal

In the seismic assessment of the "gaioleiro" buildings, shaking table tests and numerical analyses were performed. The tests were carried out in the 3D shaking table of the National Laboratory of Civil Engineering (LNEC), Lisbon, with the purpose of evaluating the seismic performance of the "gaioleiro" buildings, before and after strengthening [33]. A prototype of an isolated building was defined, constituted by four storeys with an interstorey height of 3.60 m, two opposite facades with a percentage of openings equal to 28.6% of the facade area, two opposite gable walls (with no openings), timber floors. Due to the size and payload of the shaking table, the experimental models were built using a 1:3 reduced scale [33], taking in account Cauchy's law of similitude [34]. The external walls, originally built in poor quality rubble masonry with lime mortar, were replaced by a self compacting bentonite-lime concrete. In the construction of the timber floors, medium-density fiberboard (MDF) panels connected to a set of timber joists oriented in the direction of the shortest span were used. The panels were cut in rectangles of 0.57 m x 0.105 m and stapled to the joists, keeping a joint of about 1 mm for separating the panels. The purpose was to simulate flexible floors with very limited diaphragmatic action.

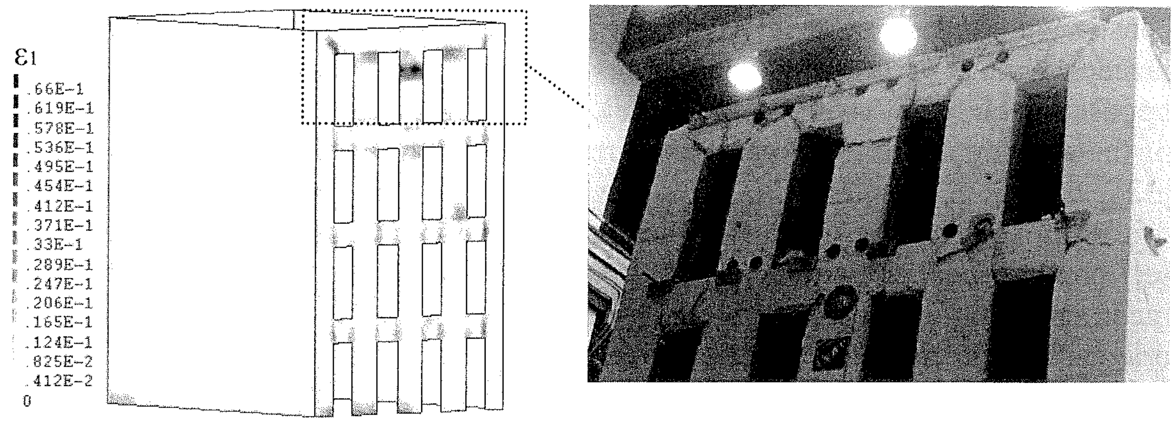
In the numerical modelling non-linear dynamic and pushover analyses were performed. The numerical model was prepared, on the 1:3 reduced scale, using the Finite Element (FE) software DIANA [28], by using shell elements for the simulation of the walls and three dimensional beam elements for the timber joists, all based on the theory of Mindlin-Reissner. In the modelling of the floors, shell elements were also used with the purpose of simulating the in plane deformability. After calibration (Figure 22) the material properties of the numerical model were obtained [35]. For detailed information about the results of the dynamic tests, see [36]. Here, only brief results of the numerical analyses are presented, see [37] for full details.

3.2.1 Non-Linear Dynamic Analysis

In the non-linear dynamic analysis the horizontal seismic action was described by two orthogonal and independent components, represented by the same response spectrum. Three earthquakes were used, composed of two uncorrelated artificial accelerograms. The artificial accelerograms are compatible with the elastic response spectrum (type 1) defined by the National Annex of EC8, for the zone of Lisbon.

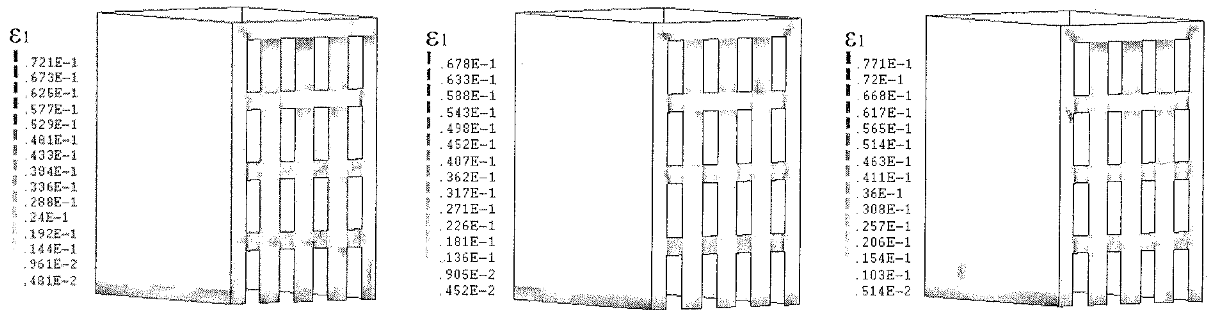
Due to the fact that non-linear dynamic analyses are very time consuming and the response spectrum of type 1 (interplate earthquake) is usually more stringent for Lisbon and the type of structures being considered, only one type of earthquake was considered. Using the 1:3 reduced scale, the accelerograms have a total duration of 6 s, from which 3.33 s correspond to the intense phase, and a *PGA* equal to 4.51 m/s^2.

Figure 23 presents the maximum values of the tensile principal strains ε_I for the three earthquake records. The results indicate that the façades at the 4th floor and the base of the structure are the zones of larger damage concentration, highlighting the high level of damage in the 4th floor's piers. Figure 24 presents the maximum displacement in the middle of the walls, in which the out-of-plane mechanism of the piers is observed.



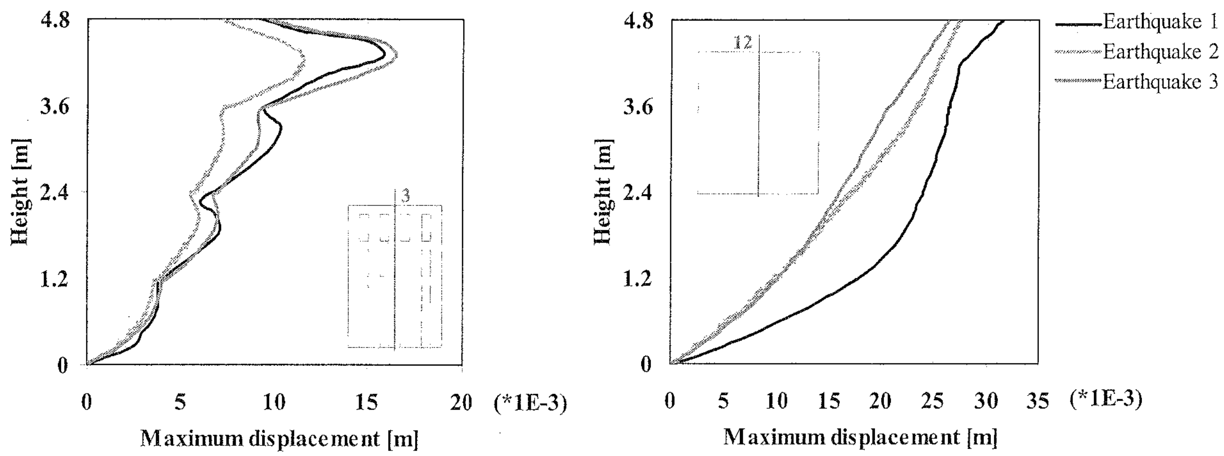
(a) (b)

Figure 22: Damage the model: (a) numerical; (b) experimental (Model 1)
(ε_1 is the principal tensile strain, which is an indicator of crack width)



(a) (b) (c)

Figure 23: Tensile principal stains (outside surface): (a) earthquake 1; (b) earthquake 2; (c) earthquake 3



(a) (b)

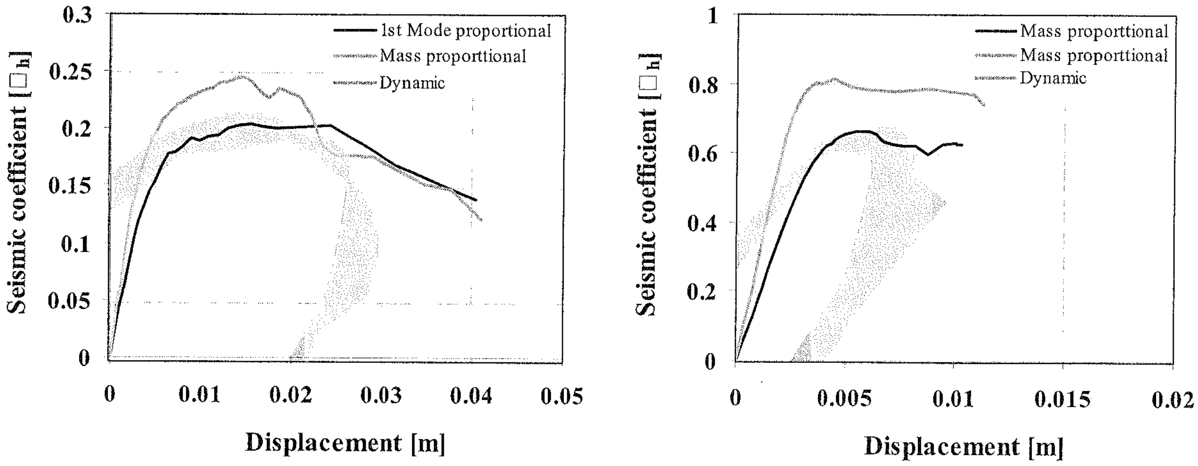
Figure 24: Maximum out-of-plane displacement in the middle of the: (a) façades; (b) gable walls

3.2.2 Pushover Analyses

Two vertical distributions of lateral loads were used: (a) uniform pattern, based on lateral forces proportional to mass regardless of elevation – uniform response acceleration; (b) modal pattern, proportional to forces consistent with the 1^{st} mode shape in the applied direction.

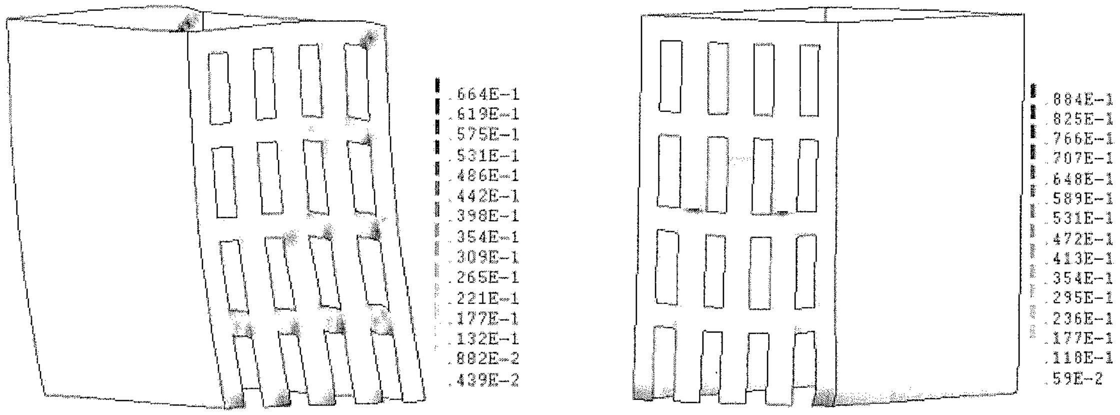
In the capacity curves of the pushover analyses proportional to the mass (Figure 25), the maximum seismic coefficients are higher than the dynamic analysis (about 24%) and the damage concentration only appears at the lower zone of the structure. It is noted that in the dynamic analysis the damage concentrates at the 4^{th} floor (façades) and at the base (Figure 21). Thus, this pushover analysis does not simulate correctly the performance of structure under seismic load.

The capacity curves of the pushover analysis proportional to the 1^{st} mode (in the applied direction) show that the maximum seismic coefficients approach the dynamic analysis. The crack patterns only provide in plane damage (Figure 26), which is not in agreement with the out-of-plane mechanism found in the time integration analysis and shaking table test (Figure 22 and 23). In an attempt to explore the pushover analyses, an adaptive pushover was performed. Here, the lateral loads, proportional to the 1^{st} mode shape in the applied direction, were updated as a function of the existing damage. The aim was to understand how the update of the external load vector can influence the structure response. However, this analysis did not provide any improvement in terms of load-displacement diagrams or failure mechanisms [37]. The typical pushover analyses did not simulate correctly behaviour of the "gaioleiro" buildings under seismic load, namely the out-of-plane behaviour.



(a) (b)

Figure 25: Tensile principal strains of the pushover analysis 1^{st} mode or mass



(a) (b)

Figure 26: Tensile principal strains of the pushover analysis proportional to the 1st mode in the: (a) transversal direction; (b) longitudinal direction

4 Conclusions

The present chapter addresses the possibilities of assessment and design of unreinforced masonry structures subjected to seismic loading. It is advocated that linear elastic analysis can hardly be used, as masonry features low tensile strength, and different models must be used in the presence or absence of adequately connected floors, the so-called box behaviour.

In the case of box behaviour the available methods have been briefly reviewed. Their performance is good and the knowledge is sound, with some corrections needed in the recent European regulations (Eurocode 8).

When box behaviour cannot be guaranteed, the analysis of masonry structures becomes rather complex. The use of macro-models and limit analysis seems the current trend but difficulties arise in the practical use, namely with respect to validation of the hypothesis of the user and the risk of selecting inadequate failure mechanisms. The non-linear static analysis could be a good and easily understood approach, because it is based on the simple evaluation of the requested deformation with respect to the displacement capacity of the building. This approach is in agreement with the modern provisions for structural assessment. Still, two case studies have been discussed, and the results obtained from the non-linear static and dynamic analyses indicate quite different responses of the structures to earthquakes. It is therefore concluded that non-linear pushover analysis does not simulate correctly the failure mode of masonry structures without box behaviour, even if higher modes are considered via modal pushover analysis.

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