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PERFORMANCE ASSESSMENT FOR UNREINFORCED MASONRY BUILDINGS IN LOW SEISMIC HAZARD AREAS

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SUMMARY

The Eixample district of Barcelona has an important historical, architectural and cultural value, covering about 750 hectares of the city. Most of its housings are unreinforced masonry buildings having an average age of about 100 years. These buildings are tall and they have been designed and built without any earthquake resistant consideration. Furthermore, they show some particular features, typical of the constructive techniques at that time, which have been identified as additional potential damage sources. In order to evaluate the expected seismic performance of these buildings, a typical six-story unreinforced masonry building was modeled. The building was designed and constructed in 1882 and contains details which are typical of that constructive period of the Eixample district. The dynamic behavior was studied by means of a structural analysis procedure, which uses macro elements to model the masonry panels. This model describes the nonlinear in-plane mechanical behavior of the panels and assesses the expected damage in masonry buildings due to earthquakes. Monte Carlo simulation has been used to take into account the uncertainties in the mechanical properties of the materials. In this way, the mean seismic capacity curves of the building and their corresponding standard deviations have been obtained. The seismic demand has been considered by using response spectra proposed by the Cartographic Institute of Catalonia (ICC, Irizarry [1]). The results here obtained for the seismic performance of this type of buildings, make clear their high vulnerability and, therefore, it is advisable retrofitting them in order to improve their seismic behavior.

INTRODUCTION

The emblematic zone of the central district of Barcelona, Spain, denominated “Eixample”, was designed in the middle of the nineteenth century. This urban area has an important historical, architectural and cultural value and covers approximately 750 hectares of the city. The most representative typology of this district corresponds to unreinforced masonry buildings (URM), which are incorporated into numerous

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almost square blocks, denominated “islands”. The construction of these buildings took place between 1860 and 1940, with 25 buildings in average for each block. They were designed only to vertical static loads, without any consideration of seismic design criteria, because they were built prior to the first Spanish seismic code. All of the existing URM buildings in this area already have exceeded their life period and only a small part of them are new reinforced concrete buildings.

The slabs of these buildings are wooden, or are made of reinforced concrete or steel (according to the building period) with ceramic ceiling vaults. Due to the great height of the first floor of these buildings, almost all of them have soft first floors. Moreover, due to the need of bigger commercial areas in the first floors, cast iron columns were used instead of masonry walls, reducing even more the stiffness of the buildings. Therefore, we expected high vulnerability for this building typology.

Recently, researchers at the ICC, have re-evaluated the seismic hazard of Barcelona, approaching the problem from two points of view: deterministic and probabilistic (Irizarry [1]). So, two types of response spectra are available: the first one corresponds to the biggest historical earthquake in the city (determinist case) and the second is the 475 year return period earthquake, namely, the earthquake whose intensity has a 10% probability to be exceeded in a 50 years period (probabilistic case). Obtaining these two elastic response spectra has been an important contribution to the definition of the seismic hazard, because we are now able to apply capacity-demand based analyses to evaluate the seismic performance of the buildings in the city. The N2 method proposed by Fajfar [2] is used with this aim. Starting from the obtained demand of spectral displacement and using the damage states proposed by Calvi [3] for URM structures, the expected damage grade and thus the performance of the building can be easily determined.

For many years in the past, the mechanical properties of the materials used to build the structures of the Eixample, were determined empirically. Therefore these properties may show a wide range of variability and a high uncertainty. In order to keep these uncertainties within a reasonable range in the case of our building class, we have requested expert opinions and we have used Monte Carlo simulations to evaluate probabilistic capacity spectra. As a result, the main parameters defining the mechanical characteristics of the model are defined by random variables which, starting from simple assumptions about their probability density function, can be characterized by a mean value and its covariance. A number of building samples are then generated in such a way that the numerical values for the parameters and properties involved in the model fit well the corresponding probability density function. This simulation process allows describing the behavior of a wide group of buildings showing similar geometrical and constructive features. Furthermore, the Monte Carlo techniques allow studying the influence of the uncertainties in the structural parameters on the evaluation of the seismic performance level.

STRUCTURAL TYPOLOGY

The typical Eixample URM building, which has been studied, has six stories, brick walls of 30 cm for the façade walls and 15 cm in the other walls. The two first floors have metallic beams and ceramic ceiling vaults simply supported on metallic main beams and cast iron columns. Rubble is placed on the upper part of the vaults and above it there is a lime mortar layer and the pavement (see Figure 1). For the other stories, the slab is made of wooden beams, supporting the ceramic ceiling vaults, as it can be seen in Figure 2. At the basement and ground floors, the masonry bearing walls of the upper part of the structure are supported on metallic main beams, which in turn, are supported on cast iron columns. The columns are supported on a block, which is supported on the masonry foundation, being this type of connection very deformable.

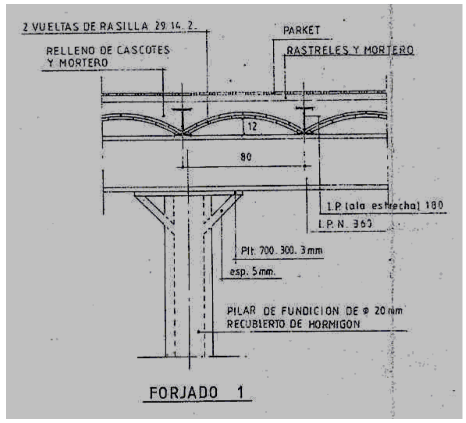

Figure 1. Detail of the slab with steel beam and ceramics ceiling vaults (taken from the original architectonic plans of the building).

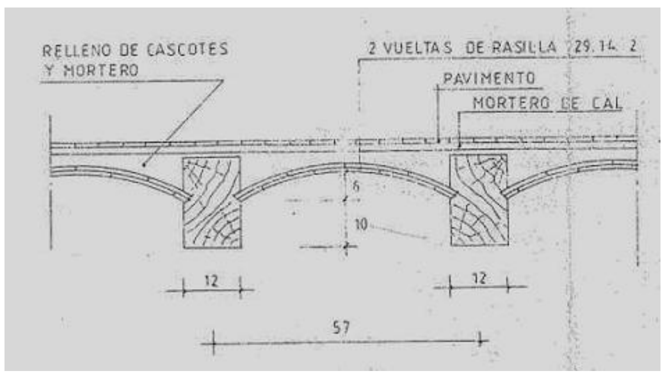

Figure 2. Detail of the slab with wood beam and ceramics ceiling vaults (taken from the original architectonic plans of the building).

The Eixample district has about 9000 housings and about 70% of them correspond to the URM typology here described. Therefore, this six-story URM building constructed in 1882, with details typical of that constructive period in the Eixample district, has been chosen for a detailed study of this type of buildings. The main purpose of this work has been evaluating the dynamic behavior and seismic performance of the buildings of Barcelona which are well represented by this typology.

The distribution in plant of the building is almost rectangular (18.9 m × 24.5 m) and the building has a central and two lateral squared patios. In elevation, the building shows certain irregularities, such as: cast iron columns at the ground floor, masonry bearing walls directly supported on metallic main beams, which, in turn, are supported on the mentioned columns. Therefore, there is a considerable variation of the stiffness with the height of the structure, reducing its seismic capacity in such a way that we may expect the typical collapse mechanism produced by the presence of a soft floor.

SEISMIC DEMAND

Barcelona, city located in the northeast of Spain, has a moderate seismic hazard and low tectonic activity. Starting from 1998 a detailed analysis of the microzonation of the city has been undertaken. This analysis allowed classifying the soil of the city in four types corresponding to 4 homogeneous areas (Cid [4]). The seismic hazard, considering the size of the action in terms of the intensity and spectral accelerations for periods of 0, 0.3, 0.6, 1.0 and 2.0 s, has been recently reevaluated. The problem has been analyzed starting both from the deterministic and probabilistic points of view. Finally, the seismic demand was defined by means of the elastic response spectra for the four zones of the city (Irizarry [1]). The Acceleration Displacement Response Spectra (ADRS), corresponding to the deterministic and probabilistic hazard scenarios, can be seen in Figure 3.

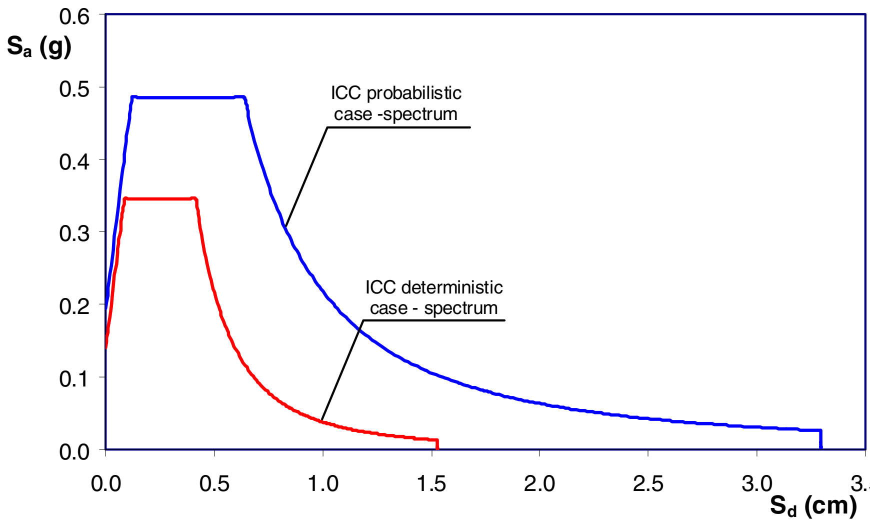

Figure 3. Response spectra for deterministic and probabilistic scenarios.

STRUCTURAL CAPACITY

Building Structural model

TREMURI program has been used for modeling the considered building type. This program was developed by Galasco [5]. A non-linear macro-element model, able to reproduce earthquake damage to masonry buildings and failure modes observed in experimental testing, is implemented in the program: it allows 3-dimensional modelling and several seismic analysis procedures.

The 3-dimensional modelling of whole URM buildings starts from some hypotheses on their structural and seismic behaviour: the bearing structure, both referring to vertical and horizontal loads, is identified, inside the construction, with walls and floors (or vaults); the walls are the bearing elements, while the floors, in addition to share vertical loads to the walls, are considered as planar stiffening elements (orthotropic 3-4 nodes membrane elements), on which the horizontal actions distribution between the walls depend; the local flexural behaviour of the floors and the walls out-of-plane response are not computed because they are considered negligible with respect to the global building response, which is governed by their in-plane behaviour (a global seismic response is possible only if vertical and horizontal elements are properly connected). A frame-type representation of the in-plane behaviour of masonry walls is adopted: each wall of the building is subdivided into piers and lintels (2 nodes macro-elements) connected by rigid areas (nodes). Earthquake damage observation shows, in fact, that only rarely (very irregular geometry or very small openings) cracks appear in these areas of the wall: for this reason these regions deformation is assumed to be negligible, relatively to the macro-elements non-linear deformations governing the seismic response. The presence of stringcourses (beam elements), tie-rods (non-compressive spar elements), previous damage, heterogeneous masonry portions, gaps and irregularities can be easily included in the structural model.

The non-linear macro-element model, representative of a whole masonry panel, proposed by Gambarotta [7], permits, with a limited number of degrees of freedom (8), to represent the two main masonry failure modes, bending-rocking and shear-sliding (with friction) mechanisms, on the basis of mechanical assumptions. This model considers, by means of internal variables, the shear-sliding damage evolution, which controls the strength deterioration (softening) and the stiffness degradation.

Figure 4 shows the three substructures, which a macro element is divided: Two layers, inferior① and superior③, in which is concentrated the bending and axial effects. Finally a central part②, this one suffers shear deformations and presents no evidence of axial or bending deformations. A complete cinematic model should take into account the three degrees of freedom for each node "i" and "j" on the extremities: axial displacement w, horizontal displacement u and rotation φ. There are two degrees of freedom for the central zone: axial displacement δ and rotation ϕ (Figure 4).

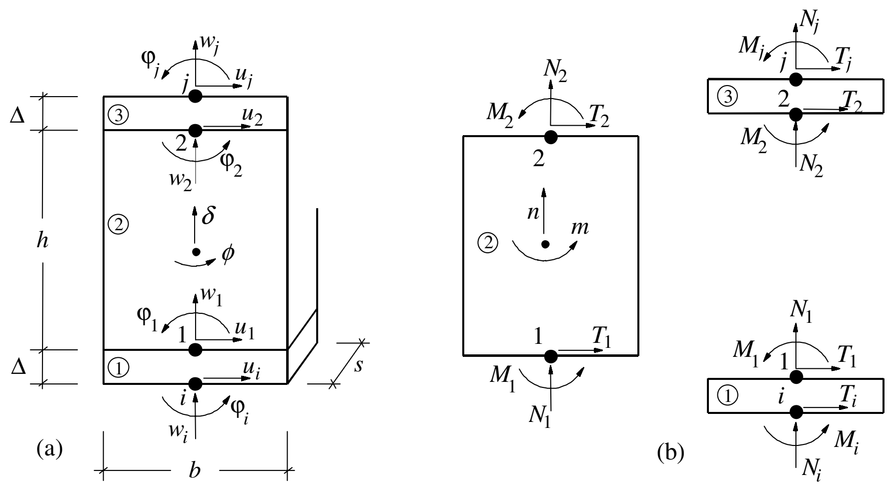

Figure 4. Cinematic model for the macro element [6].

Thus, cinematic is described by an eight degree freedom vector, $a^T = \{u_i\ w_i\ \varphi_i\ u_j\ w_j\ \varphi_j\ \delta\ \phi\}$, which is obtained for each macro element. It is assumed, for this hypothesis, that the extremities have an infinitesimal width ($\Delta \to 0$).

The overturning mechanism, which happens because the material does not resist traction stress is modeled by a mono lateral elastic contact between ① and ③ interfaces. The constitutive equations between the cinematic variables w, φ and the correspondent static quantities "n" and "m" are uncoupled to the limit condition $\left|\frac{m}{n}\right| \le \frac{b}{6}$, when the section is smaller than the entire compression zone.

For substructure ① the following equations are obtained:

$$N_i = kA(\delta - w_i) + N_i^* \tag{1}$$

$$M_i = \frac{1}{12} kAb^2(\varphi_i - \phi) + M_i^* \tag{2}$$

Where $A = s \cdot b$, corresponds to the transversal section of the panel. The inelastic contribution N^*_i and M_i^* are obtained from the unilateral condition of perfect elastic contact:

$$N_i^* = \frac{-k \cdot A}{8|\varphi_i - \phi|}\left[|\varphi_i - \phi|b + 2(\delta - w_i)\right]^2 H\left(|e_i| - \frac{1}{6}b\right), \tag{3}$$

$$M_i^* = \frac{k \cdot A}{24(\varphi_i - \phi)|\varphi_i - \phi|}\left[(\varphi_i - \phi)b - (\delta - w_i)\right]\left[|\varphi_i - \phi|b + 2(\delta - w_i)\right]^2 H\left(|e_i| - \frac{1}{6}b\right) \tag{4}$$

Where $H(\bullet)$ is the Heaviside's function.

The panel's shear response is expressed considering a uniform shear deformation distribution $\gamma = \frac{u_i - u_j}{h} + \phi$ in the central part ② and imposing a relationship between the cinematic quantities u_i, u_j and ϕ, and the shear stress $T_i = -T_j$. The cracking damage is usually located on the diagonals, where the displacement take place along the joints and is represented by an inelastic deformation component, which is activated when the Coulomb's limit friction condition is reached. From the effective shear deformation corresponding to module ② and indicating the elastic shear module as "G", the constitutive equations can be expressed as:

$$T_i = \frac{GA}{h}\left(u_i - u_j + \phi h\right) + T_i^* \tag{5}$$

$$T_i^* = -\frac{GA}{h}\frac{c\alpha}{1 + c\alpha}\left(u_i - u_j + \phi h + \frac{h}{GA}f\right) \tag{6}$$

Where the inelastic component T_i^* includes the friction stress f effect, opposed to the sliding mechanism, and involves a damage parameter α and an un-dimensional coefficient which controls the inelastic deformation c. In this model, the friction plays the role of an intern variable defined by the following limit condition [6]:

$$\Phi_S = |f| - \mu \cdot N_i \leq 0 \tag{7}$$

Where μ corresponds to the friction coefficient. These constitutive equations can represent the panel's resistance variation due to changes on axial stresses $N_j = -N_i$. The damage and its effects upon panel's mechanical characteristics are described by the damage variable α which grows according to failure criteria [5]:

$$\Phi_d = Y(S) - R(\alpha) \leq 0, \tag{8}$$

Where $Y = \frac{1}{2}cq^2$ is the rate of energy liberation by damage; R is the resistance function and $S = \{t\ n\ m\}^T$ is the internal stress vector. Assuming R as a growing function of α to the critical value $\alpha_C = 1$ and decreasing for higher values; the model can represent the stiffness degradation, the resistance degradation and pinching effect.

The complete constitutive model, for the macro element, can be expressed in the following finite form:

$$Q = Ka + Q^* \tag{9}$$

$Q^* = \left\{ T_i^* \; N_i^* \; M_i^* \; T_j^* \; N_j^* \; M_j^* \; N^* \; M^* \right\}$ contains the nonlinear terms evaluated by the evolution equations for the damage variable α and the friction, f . Finally K is the elastic stiffness matrix:

$$K = \begin{bmatrix} GA/h & 0 & 0 & -GA/h & 0 & 0 & 0 & -GA \\ 0 & kA & 0 & 0 & 0 & 0 & -kA & 0 \\ 0 & 0 & kAb^2/12 & 0 & 0 & 0 & 0 & -kAb^2/12 \\ -GA/h & 0 & 0 & GA/h & 0 & 0 & 0 & GA \\ 0 & 0 & 0 & 0 & kA & 0 & -kA & 0 \\ 0 & 0 & 0 & 0 & 0 & kAb^2/12 & 0 & -kAb^2/12 \\ 0 & -kA & 0 & 0 & -kA & 0 & 2kA & 0 \\ -GA & 0 & -kAb^2/12 & GA & 0 & -kAb^2/12 & 0 & GAh + kAb^2/6 \end{bmatrix} \tag{10}$$

The nonlinear terms N^* and M^* are defined though the following equation:

$$N^* = N_j^* - N_i^* ; M^* = -M_j^* - M_i^* + T_i^* h \tag{11}$$

The macro element shear model is a simplification of a more complex continuous model (see Gambarotta [7]) whose parameters are directly correlated with the masonry elements' mechanicals properties. The macro model parameters should be considered as a representative of an average behavior. In addition to its geometrical characteristics, the macro element is defined from six parameters: The shear module G, the axial stiffness K, the shear resistance of the masonry f_{vq_0} , the un-dimensional coefficient that controls the inelastic deformation c , the global friction coefficient f and β factor which controls the softening. The last factor is defined by pillar, as well as lintels.

The macro-element used in the program to assemble the wall model keeps also into account the effect (especially in bending-rocking mechanisms) of the limited compressive strength of masonry (Penna [13]). Toe crushing effect is modelled by means of phenomenological non-linear constitutive law with stiffness degrade in compression: the effect of this modellization on the cyclic vertical displacement-rotation interaction is represented in Figure 5.

In order to perform non-linear seismic analyses of URM buildings a set of analysis procedures has been implemented: incremental static (Newton-Raphson) with force or displacement control, 3D pushover analysis with fixed load pattern and 3D time-history dynamic analysis (Newmark integration method; Rayleigh viscous damping). The pushover procedure, with an effective algorithm, transforms the problem of pushing a structure maintaining constant ratios between the applied forces into an equivalent incremental static analysis with one d.o.f. displacement control. Additional information and further descriptions of the non-linear macro-element modelling and analysis of URM buildings can be found in Galasco [14]

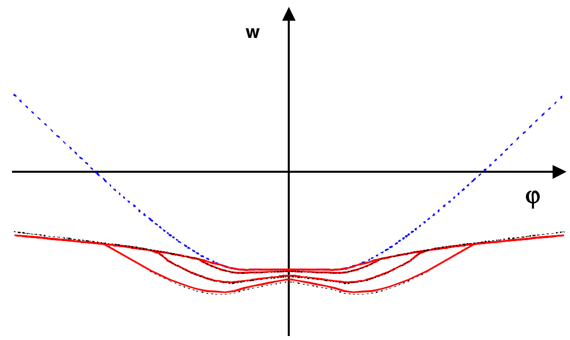

Figure 5. Cyclic vertical displacement-rotation interaction with (red line) and w/o toe crushing (blue dots) [13]

Macro element model for the studied building of the Eixample

Figure 6 shows a three-dimensional view and in plant of the model used for the representative building of the Eixample. The model is defined by 8 walls in the *x* direction (walls M1 to M8) and 6 walls in the *y* direction (walls M9 to M14). Each wall has been modeled as an assemblage of piers, lintels and frame elements (in some cases) connected to the nodes of the model by means of rigid joints. All the nodes have 5 degrees of freedom (3 displacement components and 2 rotation components corresponding to the axes *x* and *y*) except the base nodes of the model. The slabs have been modeled as an orthotropic finite element diaphragm, defined by 3 or 4 nodes connected to the three-dimensional nodes of each level. A main analysis direction is identified, which is characterized by a Young´s modulus E_1 and the direction perpendicular to this one is characterized by a Young´s modulus E_2. Figure 7 shows the macro element model corresponding to walls 1 and 2.

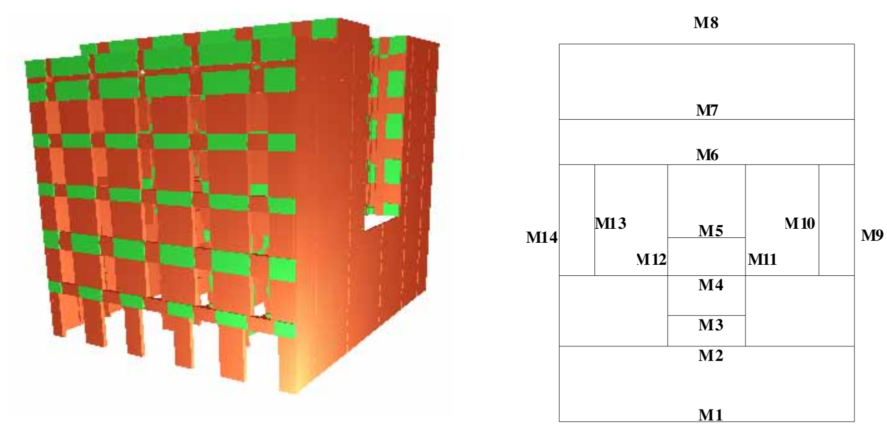

Figure 6. Three-dimensional model of the analyzed typical building of the Eixample.

In order to analyze the constructive system of the URM buildings of the Eixample, it is necessary to have a good knowledge on the materials used for their main elements. Bricks are the basic material of these buildings, being used widely in walls, stairs and slabs. The typical dimensions of the used bricks are 30×15 cm and their thickness varies between 3 and 11 cm. This kind of man-made bricks were used until the beginning of the XXth century. Later, mechanical systems were used, considerably improving their quality and compactness. Lime mortar was used in the constructive process of the buildings of the

Eixample. The wide use of this material is associated to constructive tradition, to consumption habits and, apparently, to its strength which was considered to be adequate at that period.

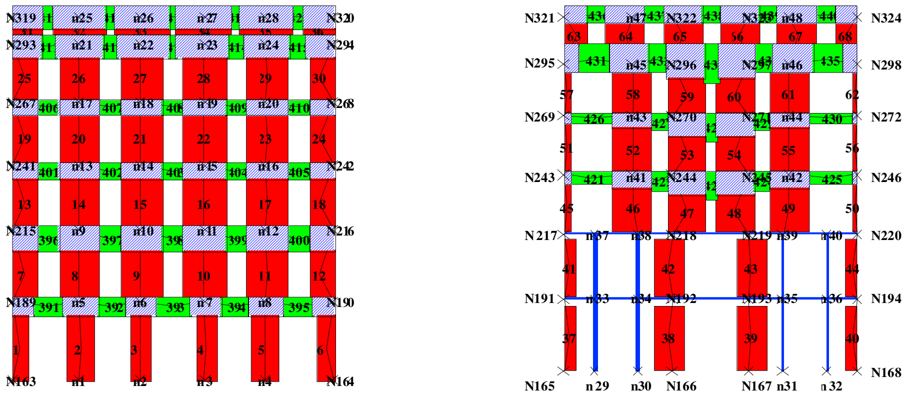

Figure 7. Macro-element model. Walls 1 and 2.

As said before, in this work, probability density functions, *pdf*, are used to define the most important parameters of the model. These functions are characterized by a mean value and a covariance. The definition of the mean value of each parameter has been defined using the opinion of experts, who provided sufficient information for defining a model. Nevertheless, due to the subjective character of this information, the main parameters have been considered as random variables with their uncertainties. The most important mechanical properties of the materials used in the analysis of the building of the Eixample are described below.

Masonry
Young´s modulus of the wall $E = 2.10 * 10^9$ N/m^2
Shear modulus $G = 0.7 * 10^9$ N/m^2
Shear strength $\tau = 1.0 * 10^5$ N/m^2
Softening factor for the piers $\beta_p = 0.5$
Softening factor for the lintels $\beta_d = 0.05$

Cast iron columns
Young´s modulus $E_s = 2.10 * 10^{11}$ N/m^2
Specific weight $\gamma_s = 7850$ kg/m^3

Concrete columns
Young´s modulus $E_h = 2.8 * 10^9$ N/m^2
Specific weight $\gamma_h = 2500$ kg/m^3

Slabs
Young´s modulus in the main direction $E_1 = 4.20* 10^9$ N/m^2
Young´s modulus in the orthogonal direction $E_2 = 4.20* 10^7$ N/m^2
Shear modulus $G = 0.4 * 10^9$ N/m^2

Among all these characteristics, those shown in Table 1 have been defined as random variables because they have an important influence on the structural response of this type of buildings. The normal probability distribution function has been used for the three variables, where the mean value of each parameter corresponds to the values proposed by experts. The covariance has been defined in such a way to cover a reasonable variation range for each parameter.

Table 1. Probability distribution functions for random variables. Mean value and covariance.

Parameter	fdp	Mean	Covariance
Young´s modulus E	Normal	$2.1*10^9$ N/m^2	0.3
Shear strength τ	Normal	$1.0*10^5$ N/m^2	0.3
Softening factor β_p	Normal	0.5	0.3

Capacity curve

The capacity curve generally corresponds to the first mode of vibration of the structure, based on the assumption that the fundamental mode of vibration contains the predominant response of the structure. In the case of the analyzed building, a force distribution was established corresponding to the bending modal shape oriented along the *y* axis. Therefore, the walls 9, 10, 11, 12, 13 and 14 are involved in the analysis (see Figure 7). However, for the sake of simplicity, the loads only will be applied to the walls 9 and 14, which really provide the greater stiffness in that direction.

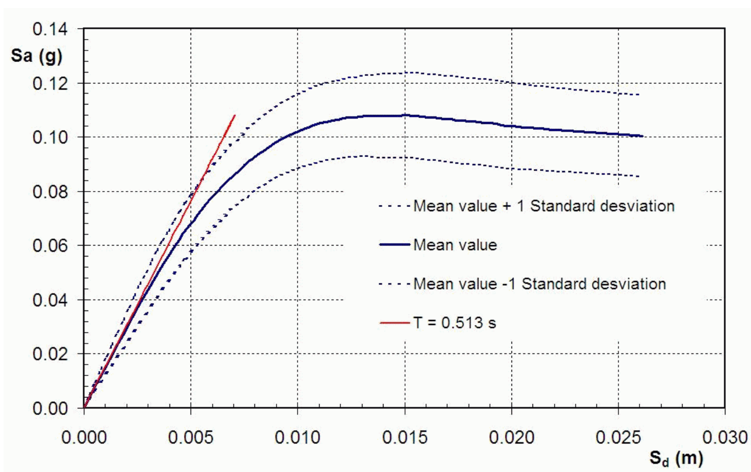

Figure 8. Mean, mean + 1σ and mean – 1σ capacity spectra.

The capacity curve is obtained by performing a pushover analysis with this load pattern. This curve describes the relationship between base shear and the roof displacement of an equivalent single degree of freedom model, characterized by the period and the modal mass of the third mode of vibration. The response of the model of the typical URM building is defined by means of the capacity curves obtained by means of the Monte Carlo simulation technique. Thus, 100 samples for each variable were generated and a structural model was defined for each sample group. One hundred capacity curves were thus obtained. The advanced computational tool STAC [8] has been used in the simulation process. Figure 8 shows the mean capacity spectra together with their standard deviations. This type of the representation shows the sensitivity of these methods to the uncertainties in the structural parameters.

The bilinear representation is obtained for these three spectra using the values of the spectral displacement and acceleration for the yielding point $\left(D_y^*, S_{ay}^*\right)$ and for the point of the ultimate capacity $\left(D_u^*, S_{au}^*\right)$. Table 2 shows these values for the mean capacity spectrum.

Table 2. Bilinear representation parameters of the capacity spectrum.

Capacity Spectrum	$D_y^*(cm)$	$S_{ay}^*(g)$	$D_u^*(cm)$	$S_{au}^*(g)$
$\bar{x}$	0.69	0.105	2.61	0.100

DAMAGE STATE LIMITS

In order to obtain the damage state limits or the performance levels of the URM building of the Eixample, there are neither laboratory tests nor available values calibrated from observed damage during earthquakes. Additionally, the values of the mechanical properties of the materials used in this structural typology are not completely known. Taking into account all these aspects, the thresholds of the spectral displacement for the discrete damage states are defined based on the bilinear representation of the capacity spectrum. Table 3 shows the expressions proposed by Lagomarsino [9] to define the variation intervals of the spectral displacement for the five damage states here considered: no damage, slight, moderate, severe and complete.

Table 3. Spectral displacement for the damage states [9, 12].

Damage state	**Spectral displacement, S_d**		
No damage		$S_d <$	$0.7D_y^*$
Slight	$0.7D_y^*$	$< S_d \leq$	D_y^*
Moderate	D_y^*	$< S_d \leq$	$D_y^* + 0.25(D_u^* - D_y^*)$
Extensive	$D_y^* + 0.25(D_u^* - D_y^*)$	$< S_d \leq$	D_u^*
Complete		$S_d >$	D_u^*

Starting from the expressions of Table 3 and using the values of D_y and D_u obtained for the six-story building (see Table 4), the thresholds of the spectral displacement are obtained for the five damage states.

Table 4. Thresholds for the spectral displacement.

Damage state	**Threshold, S_d (cm)**		
No damage		$S_d <$	0.48
Slight	0.48	$< S_d \leq$	0.69
Moderate	0.69	$< S_d \leq$	1.17
Extensive	1.17	$< S_d \leq$	2.61
Complete		$S_d >$	2.61

SEISMIC PERFORMANCE

In order to evaluate the seismic performance of the typical URM building of the Eixample, the N2 method proposed by Fajfar [2] was used. Starting from a first version, published in 1987, the method has been revised and updated to the present version, in which the Acceleration-Displacement format is used. Nowadays, the method combines the visual representation advantages of the capacity spectrum method [10] with the physical basis of the inelastic demand spectrum [11]. The basic characteristics of the method are: use of two different mathematical models, application of the response spectrum, nonlinear static analysis (pushover analysis) and the selection of a model, which takes into account the cumulative damage. This last aspect is very important for existing buildings, which frequently have not been designed to resist many hysteretic cycles within inelastic ranges [2].

In this case, two response spectra (one deterministic and one probabilistic) have been used to describe the seismic demand. For each of them, the spectral displacement demand is obtained and the performance point is evaluated (see Table 5). Figures 9 and 10 show the graphical representations of the performance point corresponding to the deterministic and probabilistic cases of the seismic demand, respectively.

Table 5. Damage state and performance levels.

Demand spectrum	S_d (cm)	Damage state	Performance levels
Deterministic	0.67	Slight	Operational
Probabilistic	1.13	Moderate	Life-safe

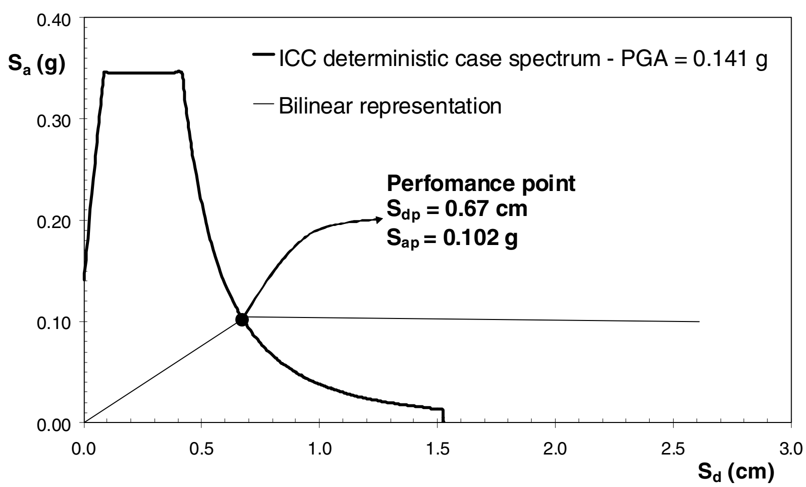

Figure 9. Seismic performance point (deterministic case).

FRAGILITY CURVES

Fragility curves have been generated starting from the assumption that the cumulative probability of reaching or exceeding a particular damage state follows a lognormal distribution. Therefore, for a given spectral displacement and damage state, this probability can be obtained by means of the following equation:

$$P\left[DS \geq DS_i / S_d\right] = \Phi\left[\frac{1}{\beta_{DS_i}} \ln\left(\frac{S_d}{\overline{S_{d}}_{,DS_i}}\right)\right] \tag{12}$$

$\overline{S_d}_{,DS_i}$ is the mean value of the spectral displacement at which the building reaches the damage state threshold DS_i, β_{ED_i} is the standard deviation of the natural logarithm of this spectral displacement and Φ is the cumulative standard normal distribution function. The subscript i stays for the damage state: slight $(i=1)$, moderate $(i=2)$, extensive $(i=3)$ and complete $(i=4)$. In order to calculate the probabilities starting from the distribution function $\Phi[\cdot]$ (equation 12), it is necessary to define $\overline{S_d}_{,ED_i}$ and β_{ED_i} for each damage state. Table 6 and Figure 11 show the parameters and the fragility curves obtained for the studied URM building.

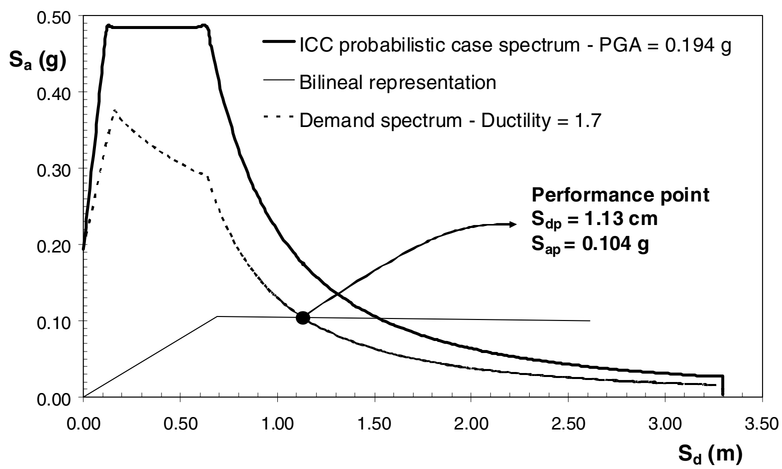

Figure 10. Seismic performance point (probabilistic case).

In order to estimate the expected damage for each seismic hazard scenario (deterministic and probabilistic), we use the spectral displacements in Table 5 and the fragility curves in Figure 11 to obtain the probabilities of each damage state. We can see in Figure 12 how the *Slight* damage state is the most probable damage state in the deterministic case, while it is the *Severe* damage state in the probabilistic case.

Table 6. Parameters of the lognormal distribution function.

Damage state	$\overline{S_d}_{,DS_i}$	β_{DS_i}
Slight	0.481	0.30
Moderate	0.688	0.45
Extensive	1.168	0.65
Complete	2.610	0.65

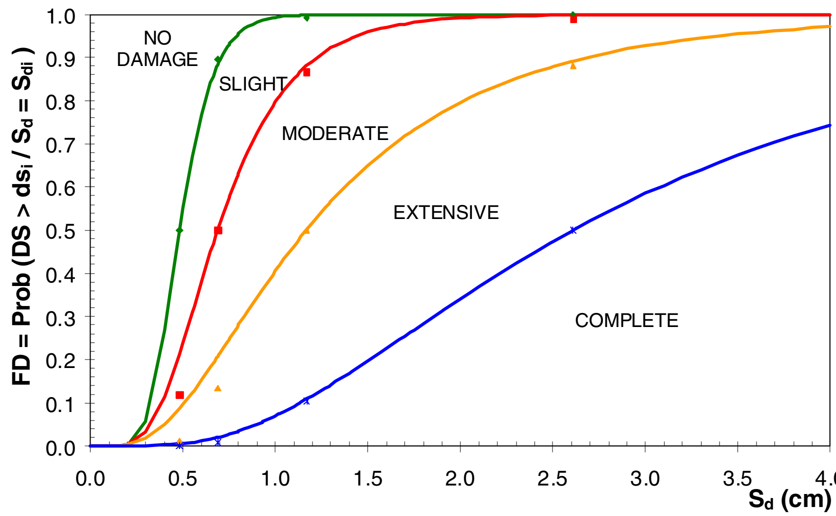

Figure 11. Fragility curves for six-story URM building of the Eixample .

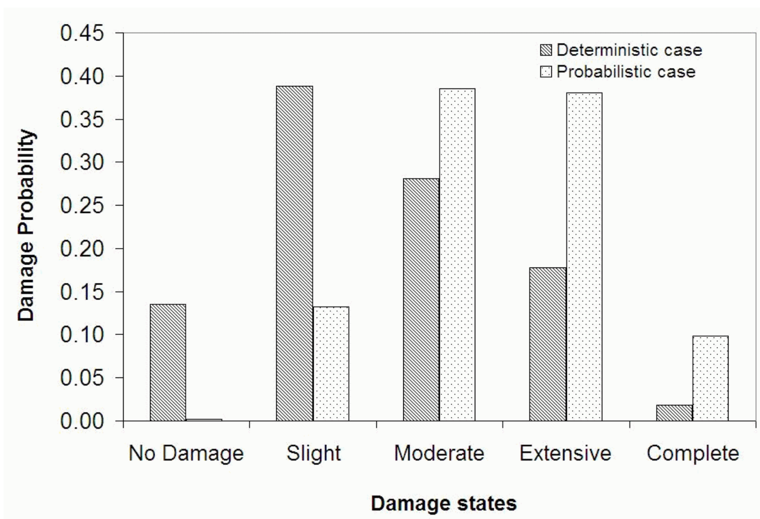

Figure 12. Damage probabilities for the deterministic and probabilistic seismic scenarios.

DISCUSSION AND CONCLUSIONS

The seismic performance of a typical unreinforced masonry building of the Eixample district in Barcelona, Spain, has been analyzed. The capacity of the building was studied by using a structural model, which uses macro elements for the masonry panels. The expected demand has been defined by two response spectra proposed by the Cartographic Institute of Catalonia. The first one corresponds to the biggest historical earthquake in the city (deterministic case) while the second corresponds to a 475 years return period earthquake (probabilistic case). The mechanical properties of the materials used for the construction of the URM buildings in Barcelona, show a high variability and Monte Carlo simulation has been performed to take into account the uncertainties. In this way we have obtained mean seismic capacity curves together with their corresponding standard deviations. The results show an important dispersion, which can also be observed in the expected damage. The performance point of the URM building of the Eixample for the deterministic case remains within the elastic range and the most probable damage state is the slight. Nevertheless, when the probabilistic case is analyzed, the most probable damage state is the severe or pre-collapse state. This situation is typical of areas with low to moderate seismic hazard. Any way, in both cases, probabilistic and deterministic, we found significant probabilities for the severe and collapse damage states, indicating the high vulnerability of most of the buildings in the Eixample district. This high vulnerability and expected damage is due to the neglect of any seismic consideration in the city. This fact is increased because the low seismic requirements planned in the Spanish seismic codes for the city and would result in considerable damage in the case of a relatively low earthquake. Therefore, an important conclusion of this work is that it is very convenient to seriously consider retrofitting and upgrading the seismic performance of the buildings of the city, particularly those whose function is important in the post-earthquake emergency, as for example, hospitals.

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