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A MODEL FOR THE STRUCTURAL AND SEISMIC ASSESSMENT OF THE MOSQUE-CATHEDRAL OF CORDOBA

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Abstract

Monumental heritage buildings were typically built without considering the seismic action. Therefore, they are potentially susceptible to seismic damage. The structural assessment and the vulnerability analysis of historical buildings is a complicated task. It requires a complex study and specialised technical skills. In order to gain some insight and to provide refined results, a model for the structural and the seismic performance of the Abd ′al-Rahman I sector of the Mosque-Cathedral of Cordoba (Spain) has been presented. The Mosque-Cathedral of Cordoba, its monumental core and its surroundings were included in the World Heritage List in 1984. Its structural system is based on parallel arcades and naves. It was started to be built in the 8th century and it has been enlarged several times. This monumental building is located in a seismic area of the southern Iberian Peninsula, characterised by large/very large earthquakes of long-return periods. In this work, a novel 3D model has been developed based on the finite element (FE) method in the OpenSees framework. The model has been elaborated through the analysis of historical data and in situ verifications. An experimental campaign has been carried out to calibrate the numerical model. Gravitational and horizontal nonlinear static analyses have been performed to determine the structural and the seismic capacity of the structure. The results have shown that the building presents the worst seismic behaviour in the direction perpendicular to the arcades. This study represents a pioneer experiment concerning the structural and seismic analysis of this building.

Keywords: Seismic Performance, Cultural Heritage, Existing Buildings, Finite Element Method, Masonry, Nonlinear Static Analyses.

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1 INTRODUCTION

Monumental heritage buildings represent the cultural identity and the history of regions [1]. Moreover, they are valuable assets for the economy of cities and countries. From a structural point of view, they were typically built only considering the gravitational loads and omitting the horizontal loading, such as the seismic action. Furthermore, most heritage buildings were constructed with unreinforced masonry (URM) walls. This is one of the structural configurations most vulnerable to earthquakes [2] due to: i) their aseismic design, mainly given by their poor horizontal connections between walls and diaphragms; ii) their low tensile strength; and iii) the difficulty in predicting their behaviour due to their heterogeneity and constructive procedure [3]. Hence, heritage buildings are potentially susceptible to seismic damage, which might result in catastrophic consequences from a social, cultural and economic point of view. In fact, this type of buildings suffered from considerable seismic damage during the recent earthquakes of 2011 in Lorca (Spain) (M_w=4.6) [4], 2009 in L'Aquila (Italy) (M_w=5.8) [5] and 2012 in Emilia (Italy) (M_w=6.1) [6].

In recent years, a strong will to conserve and to protect such buildings against seismic events has increased in all European countries [7]. However, the structural assessment and the vulnerability analysis of historical buildings are complicated tasks. This is mainly due to the transformations that the buildings have undergone over the years and the difficulty in obtaining the *as built* information [8]. Furthermore, this type of analysis requires a complex study and specialised technical skills.

The diagnosis and the safety assessment of historical buildings enables characterising their current state for a better understanding of their structural features and safety [9]. For this purpose and to provide reliable safety assessment, it is necessary to obtain validated and calibrated numerical models [10]. This type of careful analyses of case studies or sectors can provide useful information [11], which can be used later in global analyses. For the seismic assessment of historical buildings, nonlinear static analysis (NLSA) is usually carried out [12]. Despite its simplifications, it has been proved to be a suitable approach for a preliminary assessment of historical buildings, as suggested in [8,13].

Owing to the impossibility of performing destructive campaigns, non-destructive tests (NDT) are usually carried out for the structural analysis of URM heritage buildings. Operational modal analysis (OMA) is one of the most used NDT to calibrate numerical models, as suggested in [8]. This enables obtaining the dynamic properties of the buildings. The numerical modelling of URM heritage buildings is usually carried out in 3D through the finite element (FE) method. In spite of the computational burden, FE modelling enables modelling the geometry and simulating the behaviour of the structure in a more realistic way compared to other modelling strategies. Such is the case of the equivalent frame method that leads to several assumptions and limitations as concluded in [14].

The Mosque-Cathedral of Cordoba (Spain) is one of the most visited monuments in Spain. It constitutes a unique artistic achievement due to its building techniques, the wideness of the space created and its relevance in later Muslim architecture. Owing to these features, its monumental core and its surroundings were included in the UNESCO World Heritage List in 1984 [15]. The complex is composed of parallel naves of arches and vaults built with stone columns and URM walls. The structural and the seismic performance of the complex has not been performed to date. Nevertheless, several works can be found on its acoustic and geometric features [16–18]. The building is located in a moderate seismic activity region, in southern Spain. This is due to the convergence between the Eurasian and the African tectonic plates [19]. The area is characterised by intraplate seismicity of moderate magnitude and offshore far away earthquakes of large/very large magnitude with long return periods [20,21].

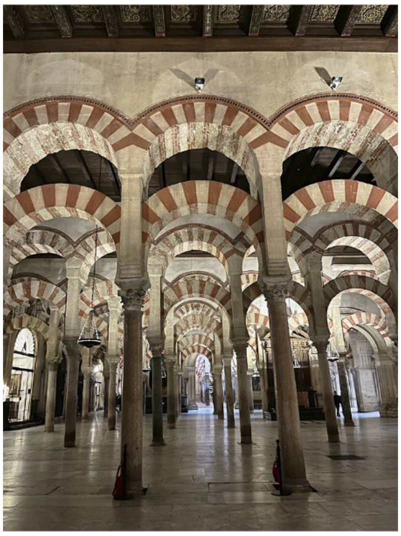
The area had to endure the well-known 1755 Lisbon earthquake (M_W=8.5), which resulted in several parts of the asset being damaged.

Given the cultural relevance and the potential seismic risk of the asset, this work presents the preliminary structural and seismic performance of one of the sectors of the Mosque-Cathedral of Cordoba, the Abd ′al-Rahman I sector. This dates from the 8th century, being the oldest part, which was mainly constructed with reused materials. In this work, a novel 3D model has been developed based on the FE method in OpenSees framework. The model has been elaborated analysing historic data and performing in situ verifications. Gravitational and horizontal NLSA have been performed to obtain the structural and the seismic capacity of the structure.

2 THE MOSQUE-CATHEDRAL OF CORDOBA

The Mosque-Cathedral of Cordoba is among the most visited monuments in Spain [22]. The asset is one of the most significant buildings in the history of the Mediterranean Muslim world. Its construction started in the 8th century and it was finished in the 15th, after being expanded in several occasions. The asset is composed of a courtyard with a minaret and a rectangular hypostyle hall, following the typical mosque configuration (Figure 1(a)). The relevance of the monument relies on the use of elements hitherto unheard-of in Islamic architecture, such as the double arches in the form of horseshoes to create the parallel naves or the 'honeycomb' capitals [15] (Figure 1(b)). This configuration allows creating a diaphanous structure based on parallel arcades and naves that went through various extensions, such as transitioning from Islamic to Christian worship. The arcades were constructed with masonry walls and arches, stone columns and timber roofing. The perimeter walls are composed of limestone masonry. Further information on the historical and architectural features of the asset can be found in [23].

(a)



(b)

Figure 1: (a) View of the Mosque-Cathedral of Cordoba highlighting the extensions. (b) Interior view of the Abd 'al-Rahman I sector, with the coffered ceiling and the double horseshoe arches (photograph taken by the authors).

2.1 Geometrical and structural features

Due to the complexity of the monument, this work has focused on the development of a 3D FE model for the structural and seismic assessment of the Abd 'al-Rahman I sector (Figure

1(a)). The dimensions of the sector are 90 m wide and 20 m long and it is composed of 11 longitudinal naves (Figure 2). The arches are made up of stone columns with bases, shafts, capitals and cymatiums that date from the Roman period. The arches follow an aqueduct-like composition to evacuate the rainwater and to support the timber roofing. They also sustain the coffered ceiling. In some naves, the timber trusses were changed for steel ones in the 20th century (Figure 3).

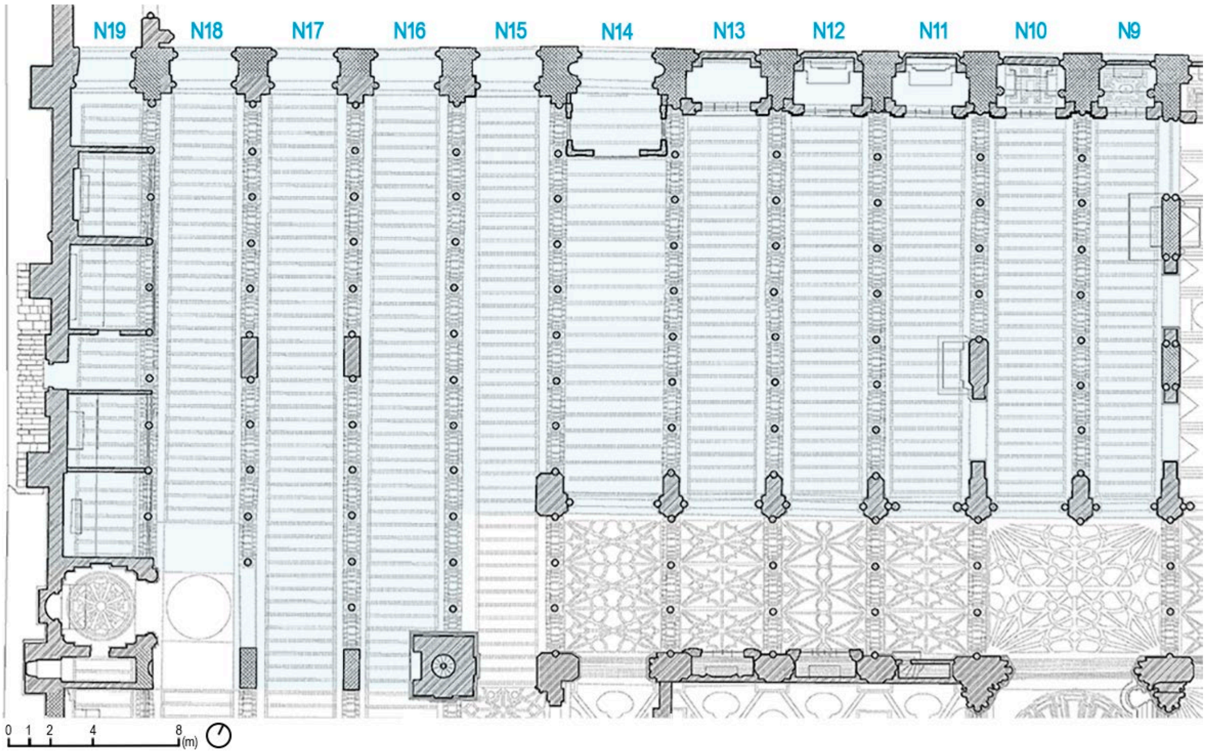


Figure 2: Plan view of the Abd 'al-Rahman I sector (naming the naves in light blue, N_i).

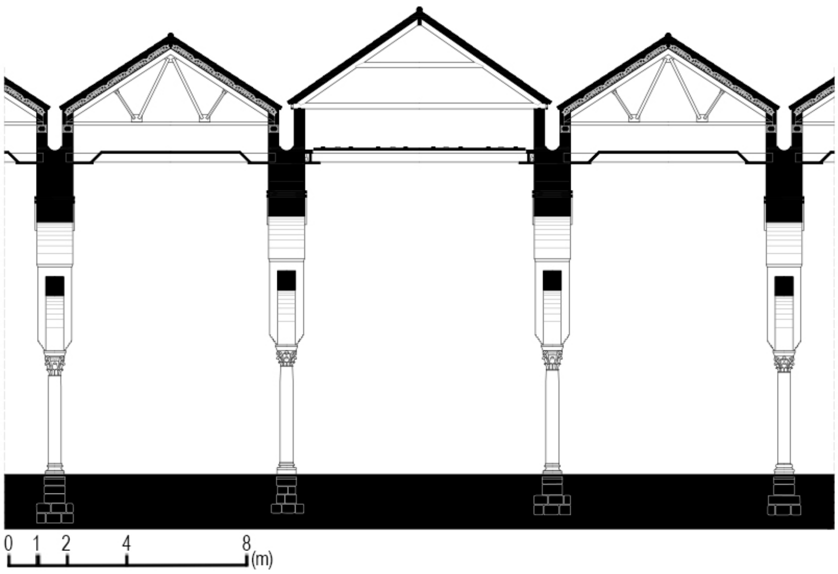


Figure 3: Type of roofing found in the Abd 'al-Rahman I sector. Drawings adapted by the authors from [24], including the foundation.

2.2 Mechanical parameters

The NDT campaign has been based on performing Schmidt Hammer (SH) tests to obtain the compressive strength of the structural materials and an OMA to identify the overall dynamic behaviour of the building. For each structural element tested, nine different SH tests were performed. It was observed that columns were mainly composed of marble (69%), granite (26%) and limestone (5%). The maximum, the minimum and the average compressive strength (f_c) obtained for each structural material is listed in (Table 1).

Material	$f_{c,max}$ (MPa)	$f_{c,min}$ (MPa)	$f_{c,aver}$ (MPa)	σ
Marble	84	35	67	± 7.90
Granite	89	35	63	± 8.16
Limestone	89	59	75.5	± 4.16
URM brick walls	28.5	25	27	± 1.10
URM limestone walls	46	22	30.20	± 4.70

Table 1: Maximum ($_{max}$), minimum ($_{min}$) and average ($_{aver}$) compressive strength (f_c) obtained from the SH tests, including the standard deviation (σ).

The OMA has been performed to obtain a more reliable modelling of the building. To do so, force-balance triaxial accelerometers were used and placed on the arcades (Figure 4). The ARTeMIS Modal software [25] has been employed to calculate the modes of vibration and the frequencies of the sector under study. The Enhanced Frequency Domain Decomposition (EFDD) method has been followed, similarly to [26,27]. According to the results obtained, the complexity of the modes has been lower than 10%. After performing the calibration, a relative error between the experimental and numerical frequencies lower than ±3% has been obtained. It has been found that the Mode 1 and 3 are translational in the Y direction (east-west). By contrast, Mode 2 is rotational.

Figure 4: Location of the accelerometers in the arcades for the OMA.

The mechanical parameters have been defined according to the NDT campaign and the calibration of the model: the compressive strength (f_c), the elastic modulus (E) and the density (ρ). The rest of the mechanical parameters needed for the NLSA have been defined according to the reference values proposed in [28], being, in this case, the shear modulus (G), the tensile strength (f_t) and the Poisson ratio (v). They have been selected according to the values obtained in the NDT campaign.

Material	ρ (T/m3)	E (GPa)	G (GPa)	v	f_c (MPa)	f_t (MPa)
Clay bricks and lime mortar masonry	1.8	2.0	0.55	0.2	3.57	0.21
Limestone and lime mortar masonry	1.8	1.8	0.42	0.3	2.78	0.10
Marble	2.8	16	6	0.25	67	3.50
Granite	2.8	42	14	0.25	63	3.45
Limestone	2.7	10	2	0.25	75.5	3.80
Timber (old roof)	0.45	11	4	0.4	40	10
Steel (new roof)	78	210	81	0.3	410	275

Table 2: Mechanical properties of the materials used for the NLSA.

3 NUMERICAL MODELLING

The 3D numerical model has been developed in the OpenSees framework [29], based on the macro-mechanical approach and using solid elements (Figure 5). These elements enable obtaining a comprehensive stress-strain behaviour of the elements and the materials needed for a thorough analysis of historical buildings [9]. The FE meshing has been developed in a pre- and post-processor for OpenSees, the STKO software [30]. The FE mesh has been developed to preserve, as much as possible, the real configuration of the building. As followed in [13,31,32], the mesh size has been defined to obtain a fair compromise between the computational burden, the accuracy and the robustness of the modelling. Hence, a mesh size of 0.3 m and 0.2 m has been defined for the external walls and the inner parts, respectively. Tetrahedron elements have been needed to fit the irregular parts. In total, the FE mesh is composed of 306,386 nodes and 1,124,353 solid elements. The model has been fixed at the base. The presence of the adjacent structure has been considered through zero-length 3D elements and the materials have been calibrated with the OMA.

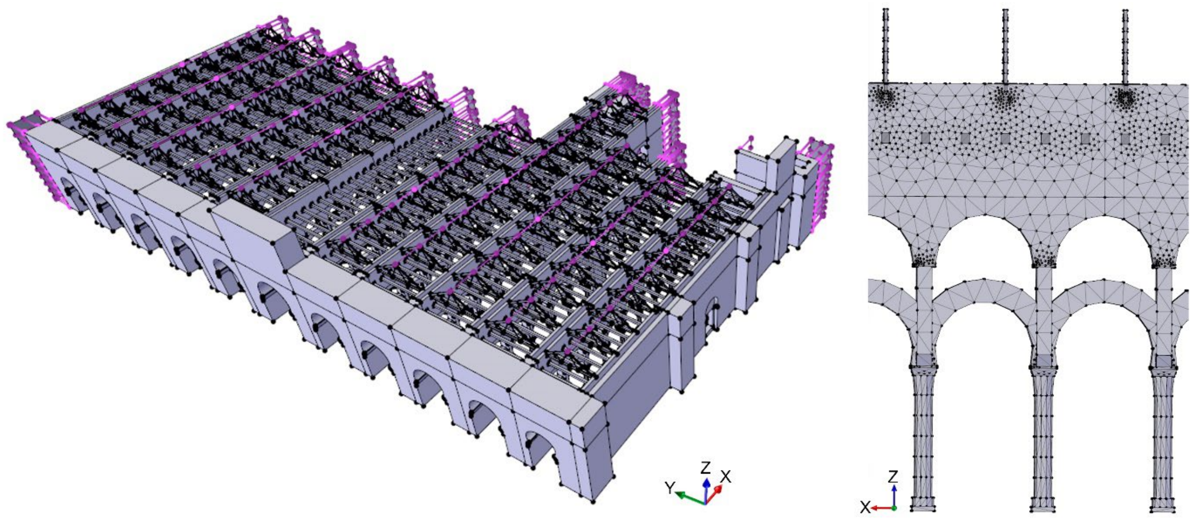


Figure 5: 3D model developed for the Abd 'al-Rahman I sector and detail of the FE meshing.

In order to simulate the nonlinear behaviour of the structural elements, a damage-plasticity material has been implemented [33]. This is typically used to simulate the nonlinear behaviour of quasi-brittle materials such as masonry. This material is based on two independent tensile and compressive constitutive laws (Figure 6). The parameters needed for the definition of the material have been defined according to [34]: the peak (f_{cp}), the elastic (f_{c0}) and the residual (f_{cr}) compressive strengths and the strain (ε_{cp}) at peak compressive strength. The compressive (G_c) and the tensile (G_t) fracture energies have been computed according to the equations proposed in [35]. In this case, the input fracture energies have been divided by the characteristic element length (l_{ch}) to obtain a response that is mesh-size independent.

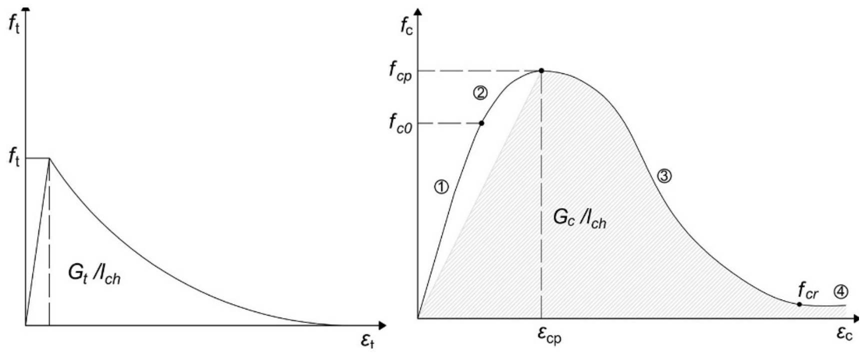
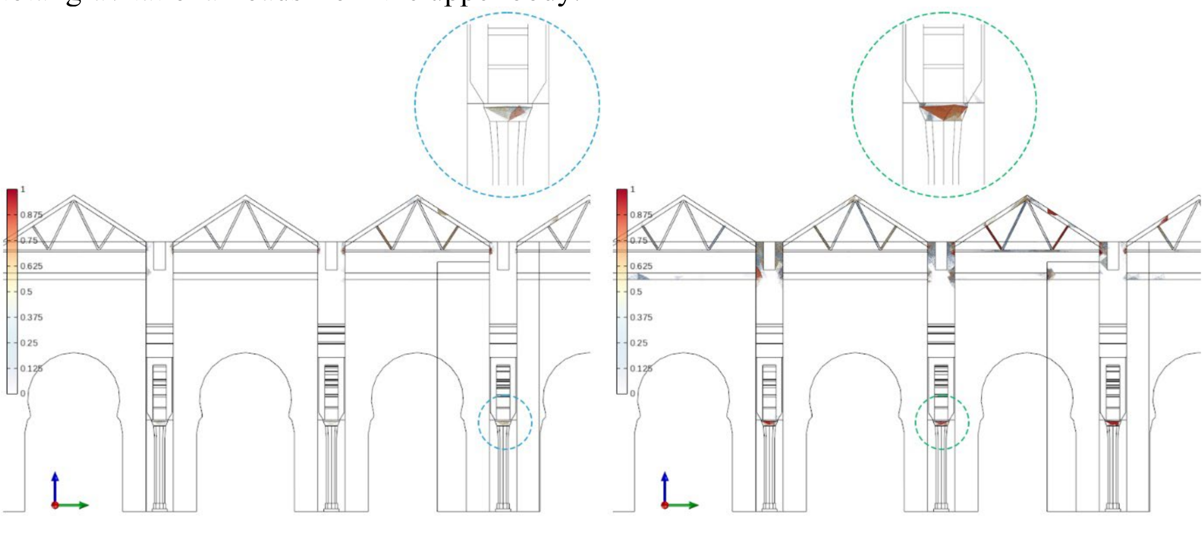


Figure 6: Tensile and compressive constitutive laws, adapted from [33].

4 RESULTS

First, a gravitational NLSA has been performed. To do so, the gravitational loads have been applied as volume forces according to the density of the materials. A load-control integrator has been used to perform this analysis. In Figure 7, the damage pattern in compression (d^-) and in tension (d^+) are presented after applying the gravitational loads. Damage is expressed as a range between 0, non-damaged, to 1, totally damaged. As can be observed, the structural elements do not present damage after the application of the gravitational loads. This is in good agreement with the real situation of the building. However, it has been obtained that the cymatiums present damage that reaches values close to 0.7 and 0.9 in d^- and d^+, respectively. This is mainly because they are the most demanded structural elements as they receive the total gravitational loads from the upper body.



(a) (b)

Figure 7: Damage in compression (d^-) (a) and in tension (d^+) (b) after the performance of the gravitational loads.

Later, horizontal NLSA has been carried out, considering a uniform load pattern, which is generally accepted for the seismic analysis of historical buildings [8,13]. The loads have been applied in the two main directions of the building, the X (north-south) and Y (east-west), for both the positive and negative orientations. The ultimate displacement has been obtained by considering a 20% decay of the pushover curve, as suggested in [31] and in the Eurocode-8. In Figure 8, the pushover curves obtained are plotted. They have been normalised by a load factor (total base shear (V_b) divided by the weight (W) of the structure) and considering the displacement of the control node at the top (d_{top}). The control node is located at the centre of the masses. The pushover curves are expressed for a single-degree-of-freedom (SDOF) system. As observed, the initial stiffness of the curves is similar. However, the system presents a higher resistance in the X (N-S) direction compared to the Y direction (E-W). This might be due to the bracing effect generated by the arcades in the X direction.

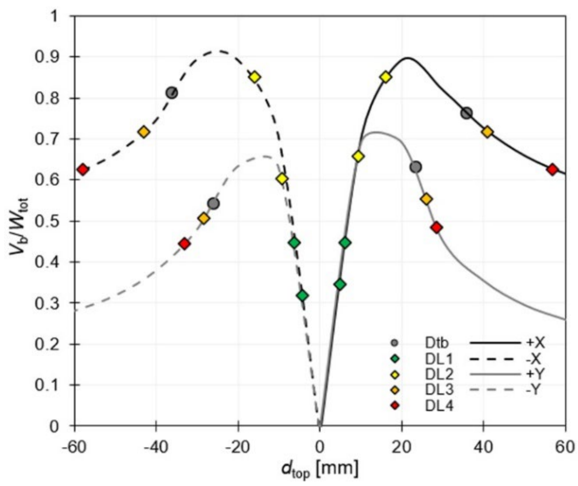
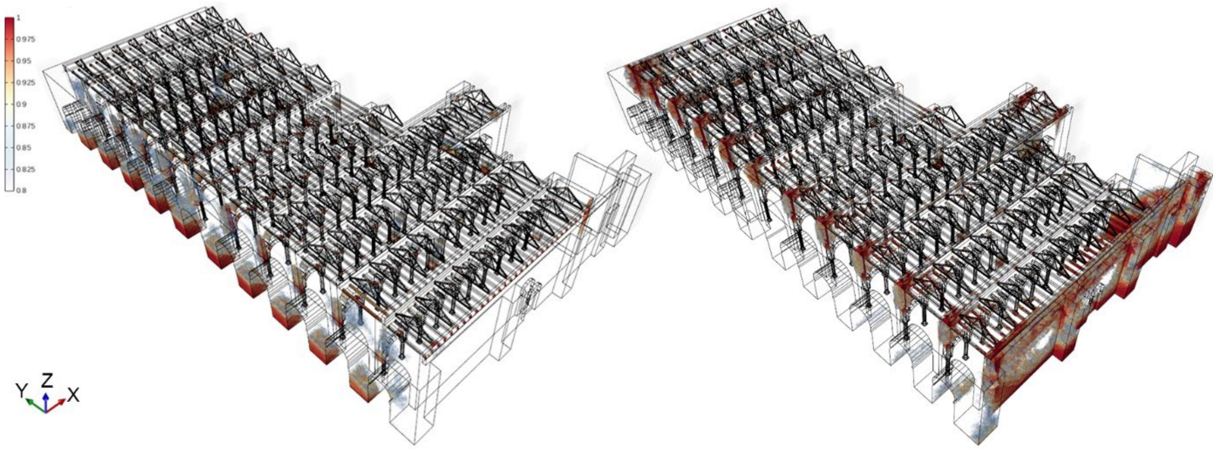


Figure 8: SDOF capacity curves obtained in the NLSA.

After the NLSA, the performance assessment has been carried out. In this work, the guidelines proposed in [8] for the global scale approach have been followed. In total, four performance levels have been defined, which are correlated to the different damage limits (DL): DL1, no damage; DL2, damage limitation; DL3, significant but repairable damage; and DL4, near collapse. Each DL has been computed as follows: DL1≥$0.5V_b$; DL2=$0.95V_b$; DL3=$0.8V_b$; and DL4=$0.7V_b$. The seismic demand has been computed according to the N2-method [36]. The response spectrum has been defined according to the Eurocode-8 part 1 provisions. A peak ground acceleration (PGA) of 0.09g has been defined for Cordoba considering the updated values of the Spanish seismic code proposed in [37]. This PGA is expressed for a probability of exceedance of 10%, which represents a return period of 475 years. This probability of exceedance corresponds to the severe damage limit state, DL3.

As can be seen in Figure 8, for the seismic demand displacement (d_{tb}) expected, in both directions, the damage ranges between DL2-DL3. As defined in [8], this represents the light damage state, which leads to some parts of the building being slightly damaged. It can also be seen that the damage in the Y direction (N-S) is expected to be higher than in the X direction (E-W).

Detailed 3D models with damage concentration have been obtained for each of the two principal directions of the building and for the attainment of the d_{tb}. Only the damage patter in tension is presented as no excessive damage in compression has been obtained. As observed in Figure 9(a), for the +X direction, the tension is mainly concentrated in: the low part of the north wall of the building, its contact with the arcades and in the first row of the arches. As for the +Y direction, in Figure 9(b), it can be observed that the western wall might be significantly damaged, mainly in the lower parts. Also, the north wall would be notably damaged in its contact with the transversal arcades.



(a) (b)

Figure 9: Damage pattern in tension for the +X (a) and +Y (b) directions.

5 CONCLUSIONS

This paper has presented the preliminary structural and seismic performance of one of the sectors of the Mosque-Cathedral of Cordoba, the Abd `al-Rahman I sector, which dates from the 8th century. The building is located in a seismic-prone region and it was declared as a UNESCO World Heritage Site. Despite the cultural value of the complex and the seismic hazard of the region, the analysis of its structural features and its seismic behaviour has not been performed to date. In this paper, an advanced FE-based model has been developed and calibrated through NDT and OMA. The compressive strength of the materials and the dynamic properties of the building have been obtained. It has been found that the columns are mainly composed of marble (69%) and, to a lesser extent, of granite (26%) and limestone (5%).

The results of the gravitational NLSA are in a good agreement with the real situation of the monument. It has been concluded that particular attention should be paid to the cymatiums, as the damage expected reaches values close to 0.7 in compression and 0.9 in tension. The horizontal NLSA has resulted in different capacity curves for each of the main directions of the building. In this case, the higher capacity has been obtained for the +X direction (N-S), which can be due to the bracing effects of the arcades. According to the damage assessment, for the seismic demand of Cordoba, the damage is expected to range between DL2 andDL3. This represents a light damage that could be easily restored. As previously indicated, in the Y direction (E-W), the building presents a lower capacity and, therefore, the damage expected is higher.

This work represents the first attempt carried out by the authors to study the structural and the seismic performance of the Mosque-Cathedral of Cordoba. The goal is to obtain some preliminary results and conclusions on the structural and seismic features of the monument. Despite its advantages, horizontal NLSA leads to certain simplifications and limitations. Therefore, for complex buildings such as the Mosque-Cathedral of Cordoba, it might not be enough to obtain a detailed seismic assessment. Hence, as a future development of this work, special interest is taken in the study of the dynamic behaviour of the building and in considering the effects of the soil-structure interaction [38,39].

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