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SEISMIC ASSESSMENT OF HISTORIC COMPLEX TOWERS. THE CASE STUDY OF THE GIRALDA TOWER

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Abstract

The Giralda tower of Seville is an ancient complex tower. It is composed of an inner and an outer wall connected by an ascending ramp that varies with its height. The ascending ramp makes every façade be different in height and composition. Also, the unreinforced masonry varies and it has some parts made of stone. Moreover, the western façade partly connects with the cathedral and the southern area. It reaches a total height of 95 m and it is one of the most ancient buildings in Seville. It was listed as a World Heritage Site by the UNESCO in 1987 with the maximum level of protection: Outstanding Universal Value. The tower is located in a seismically-prone area and it is known to have been severely damaged during past historic earthquakes. Therefore, a complete and adequate seismic analysis of its seismic behaviour is mandatory. This work has focused on a refined analysis of its seismic performance to provide reliable results. A detailed 3D architectural model of the monument has been created in the Rhinoceros software. The seismic assessment has been carried out following the Finite Element Method (FEM) with the OpenSees software. This approach is based on a numerical procedure grounded on the use of nonlinear static analysis. Equivalent boundary conditions have been introduced to assess the interaction among the different parts of the tower with the cathedral. To minimise the uncertainties in complex masonry buildings and to calibrate the numerical model in situ experimental tests have been carried out. Finally, different horizontal load patterns have been considered in the analyses to assess its seismic behaviour.

Keywords: seismic performance, cultural heritage, existing buildings, finite element method, masonry, nonlinear static analyses.

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1 INTRODUCTION

Built cultural heritage has a big impact on the European Union (EU). Its conservation is a current and difficult issue. In this regard, the EU countries have committed themselves, through policies and programmes, to preserving and to improving their cultural heritage.

Seville is a Spanish city located in the southwestern Iberian Peninsula (IP). The IP seismic activity is moderate. However, the southern area is characterized by the occurrence of large earthquakes (M_w>7) with long-return periods [1]. This is due to the contact area between the Eurasian and African tectonic plates. According to the earthquake record, the south of the IP is the most seismic area. Seville has a moderate seismic risk, but its soft alluvial strata are known to increase the effects of earthquakes [2–4].

The city has a great cultural heritage which depicts its historic and social identity. Furthermore, it is one of the bases for the social and economic development of the city. In this context, the Giralda tower is the most representative building of the cultural identity and economic base of Seville. Due to its cultural relevance, it was declared as a Word Heritage Site of Outstanding Universal Value by the UNESCO. Originally, it was built as a minaret for the major Islamic mosque of the city in 1198. However, the tower has undergone different construction phases and significant modifications over time. Furthermore, it has been damaged by several historic earthquakes [5].

The seismic assessment of heritage buildings is an important issue, which has been studied in depth due to their complexity [6–9]. This is a complicated task mainly due to the difficulty in characterising the constructive materials (they are heterogeneous), the different construction phases and the rehabilitation interventions. In these buildings, their original appearance cannot be altered with *in situ* characterisation surveys. In that sense, non-destructive testing (NDT) has been discussed and illustrated in the literature [10,11] as a procedure to typify and calibrate the numerical models for the analytical analysis [12].

The main objective of this work is to numerically model the Giralda tower using the 3D Finite Element Model (FEM) method to assess its seismic behaviour. For this aim, the OpenSees [13] and the STKO softwares [14] have been employed. A modal and a nonlinear static analysis (Pushover) have been carried out to obtain a first approximation of the seismic behaviour of the tower. This analysis is important because of the great amount of historic architectural heritage that was constructed with masonry structures (buildings, towers, castles, churches, etc.) [15] .

The rest of the manuscript is organised as follows. In Section 2, the case study is described. In Section 3, the methodology followed in this work is shown and the numerical modelling of the tower is described. In Section 4, the results are presented and discussed. Finally, in Section 5, the conclusions of the work are drawn.

2 CASE STUDY: THE GIRALDA TOWER

The Giralda is the most representative building of Seville. It is an excellent example of the different cultures that have lived in the city. The tower has undergone several construction phases and modifications over time. Furthermore, it had to endure several historic earthquakes, such as the 1755 Lisbon earthquake (M_w=8.9), that led to the tower being severely damaged [16].

Originally, the tower was built between 1184 and 1198, as a minaret for the main Islamic Mosque of the city. It was crowned by four golden spheres, which fell during the 1356 earthquake. The most relevant intervention was the addition of the Renaissance bell tower designed by the architect Hernán Ruiz in 1568 [17]. The building acquired a similar appearance to its current one (Figure 1).

The tower was built with ceramic brick and stone masonry. The structural system is made up of two parallel brick masonry walls. The floor has a quadratic dimension of 13.60x13.60 m and the height reaches around 95 m (Figure 1). The thickness of the outer masonry wall ranges between 2.00 and 2.30 m. The inner body is of 6x6 m composed of brick masonry walls of around 1.30 m thickness. It is important to note that both walls are connected by the ascent ramp that reaches the belfry level. These ramps are composed of brick masonry vaults and compact limestone with different thicknesses to build the slope. Furthermore, several brick masonry vaults limit seven inner rooms in the core of the tower.

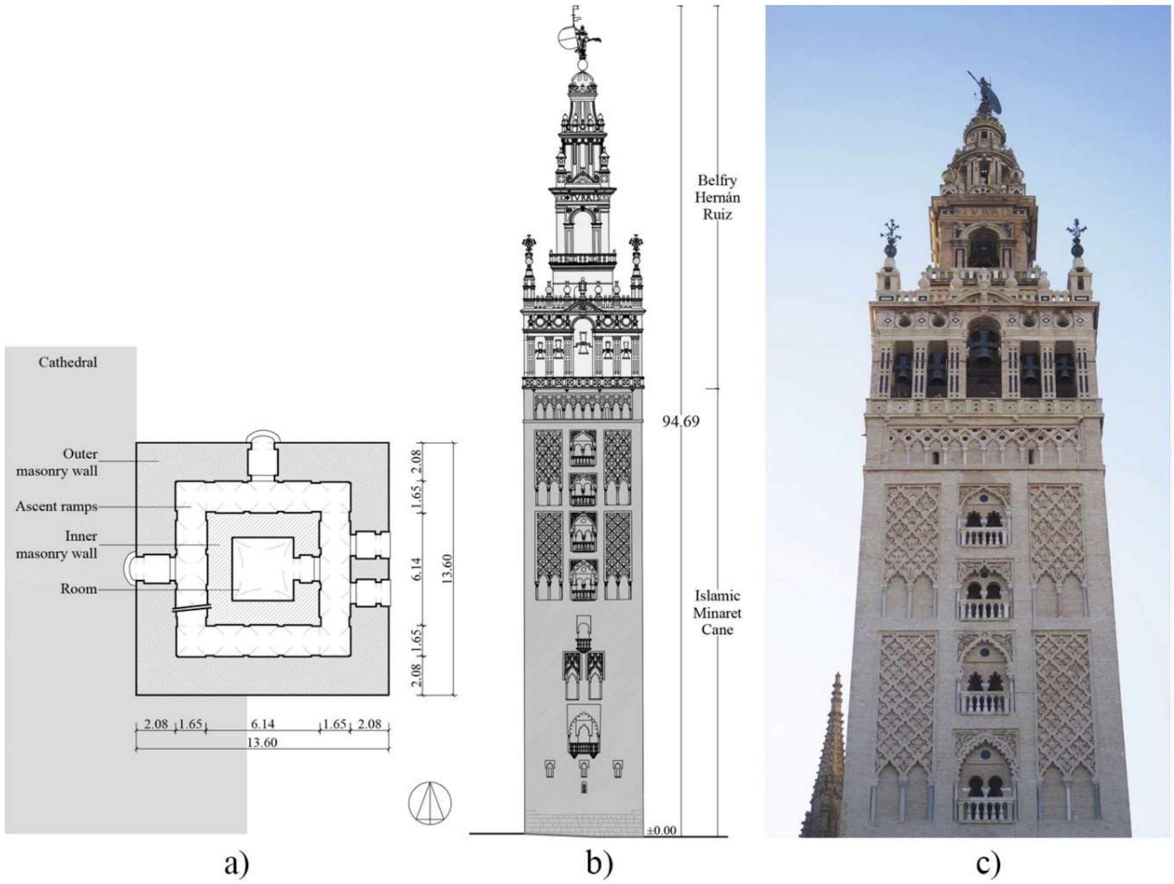

Figure 1: Schematic configuration of the Giralda tower. a) plan; b) west façade and c) current appearance.

3 METHOD

In this section, the methodology used to develop the work is shown. The steps followed have been plotted in Figure 2.

First, a historic analysis of the tower and its surroundings has been performed. For this purpose, the available information of the building has been collected. During this phase, different research works and previous projects [17–19] have been consulted and evaluated. This is a significant phase to know the historic evolution of the building, identifying its historic construction phases and its rehabilitation projects.

Second, an accurate geometry of the building has been developed. By doing this, the available tower blueprints [20] have been compared and completed with several *in situ* surveys. This *in situ* campaign has consisted of a visual inspection, digital images analysis and laser measurements. An *in situ* ambient vibration test has been carried out to obtain the dynamic characteristic of the tower.

Next, a simplified 3D CAD model has been developed based on the precise geometry obtained in the previous phase. For this, the Rhinoceros software v7 [21] has been employed. Later, this 3D CAD model has been exported to the STKO software to develop the FEM. The different parameters of the numerical model (solid mesh, boundary condition and materials properties) have been defined in this software.

Finally, a modal analysis and a nonlinear static analysis (NLSA) have been performed using the OpenSees open-source software [13]. The modal analysis has been done to calibrate the numerical model, comparing the values of the periods and the frequencies with the *in situ* measurements obtained with the Operational Modal Analysis (OMA). This is important in order to obtain a reliable and robust FEM [7]. Then, a gravitational and horizontal NLSA have been carried out in both directions to obtain the capacity curves of the building, applying several load patterns.

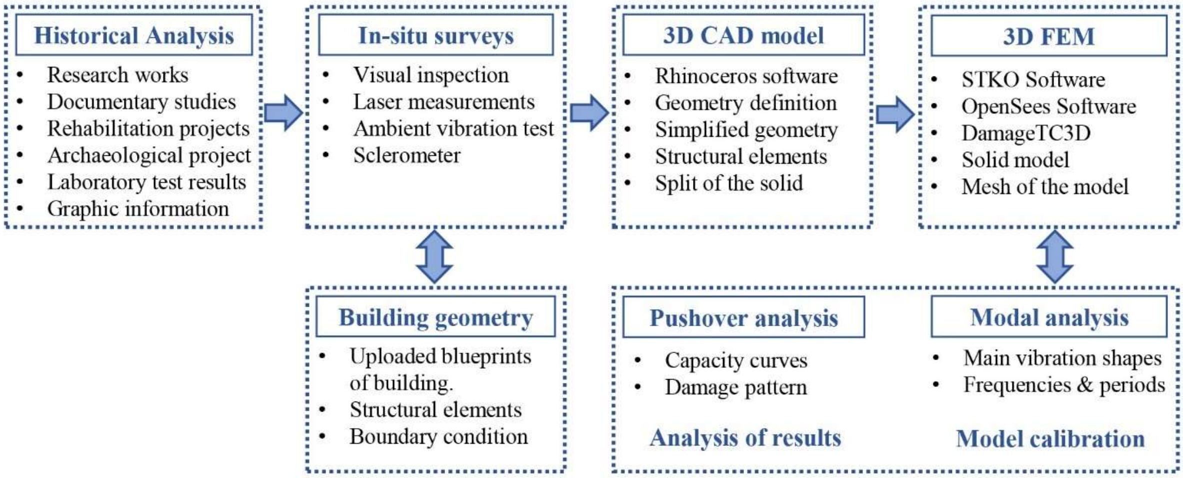

Figure 2: Schematic diagram of the method.

4 NUMERICAL MODEL

In this section the process to obtain the numerical model of the tower has been presented.

4.1 3D CAD MODEL

The 3D CAD model of the building geometry has been developed according to available information and to the *in situ* surveys (Figure 3). This model has been carried out in the Rhinoceros v7 software [21] which is a NURB-based 3D modelling tool.

Several factors have been considered in order to develop the geometry model. In that respect, the inner body has been modelled considering the different levels and openings up to the upper level (94.69 m height). The ramps have been modelled as different horizontal planes, reaching the level of the belfry. The outer wall has been modelled considering the different openings and the bell tower on the top. Different wall thicknesses have been borne in mind for the numerical model. In addition, the boundary condition of the nearby Cathedral has been considered. Accordingly, two equivalent masonry walls, that belong to the cathedral, have been modelled on the western and southern façades (Fig. 3).

Finally, it is important to highlight that the model has been split into different parts according to the different inner levels. This division has been made with the aim of improving the exportation to the STKO software.

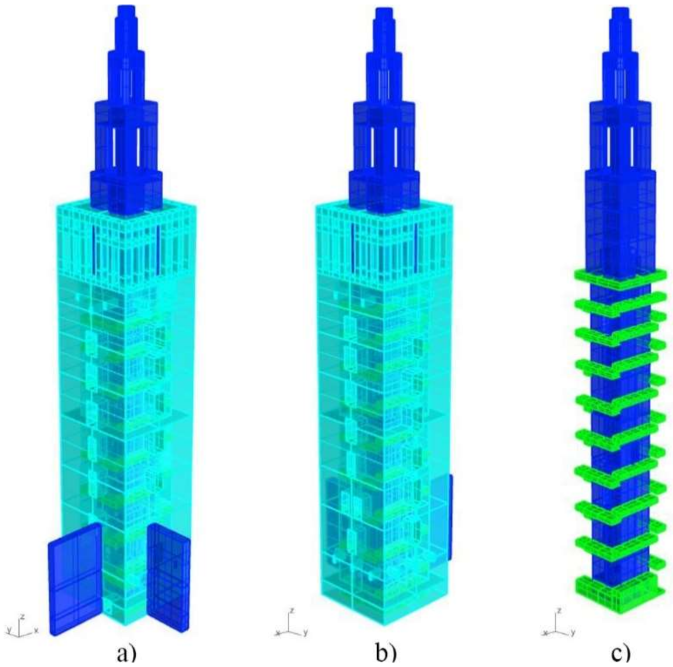

Figure 3: 3D CAD model. a) and b) outer wall and upper body; c) inner body and ramps.

4.2 3D FINITE ELEMENT MODEL

The definition of the 3D FEM model (Figure 4) has been carried out using the pre- and the post- STKO processor for OpenSees. The 3D CAD has been exported to obtain the base of the building geometry. Due to the complexity and to the size of the tower, the 3D FEM model has been developed as a solid model, following the macro-mechanical approach [22]. It has been meshed as non-structured tetra mesh, applying a four-node tetrahedron element (Figure 4). Furthermore, a 40 cm mesh size has been adopted due to its balance between accuracy and computational effort.

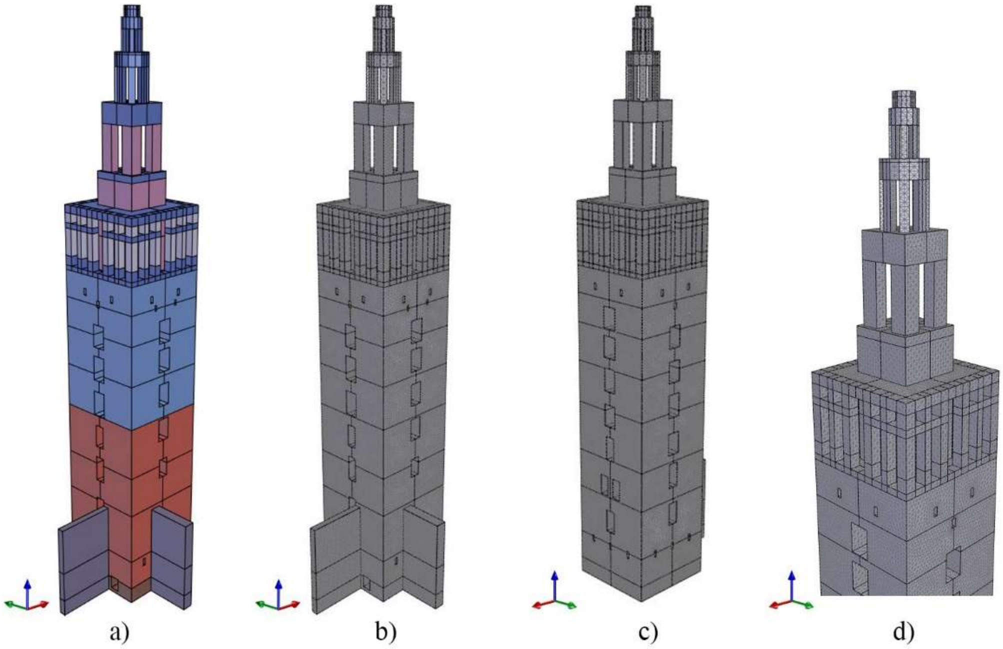

Figure 4: 3D FEM model. a) material properties; b) and c) mesh; d) belfry level.

A 3D Tension-Compression damage material has been applied to define the nonlinearity behaviour of the masonry material. This material model is implemented in the OpenSees software. It is expressed as two independent failure criteria for the tensile and for the compressive range, respectively. Different materials have been defined: brick and stone masonry for the walls and another one for the ramp. The mechanical properties of the materials have been listed in Table 1. These have been obtained from previous laboratory tests [23], building codes [24–26] and research works [27]. Furthermore, these parameters have been calibrated according to the OMA results.

Mechanical parameters	Brick masonry			Stone Masonry	Stone masonry base	Lime concrete ramp	Units
	Bottom	Middle	Upper				
Mass							
Density *(ρ)*	1,800	1,800	1,800	2,200	2,500	2,500	kg/m^3
Elasticity							
Young's modulus (E)	2.90	2.27	1.81	2.50	2.50	3.00	GPa
Poissons's ratio (ν)	0.2	0.2	0.2	0.25	0.25	0.2	
Tension							
Tensile strength (f_t)	218	170.4	134	143.8	130.2	312	kN/m^2
Compression							
Comp. elastic limit (f_{c0})	2,441	1,988	1,582	1,677	1,519	2,600	kN/m^2
Comp. peak strength (f_{cp})	3,630	2,840	2,260	2,396	2,170	5,200	kN/m^2
Comp. residual strength (f_{cr})	363	284	226	239	217	520	kN/m^2

Table 1: Mechanical properties used in the 3D FEM.

Finally, two horizontal load patterns have been applied: proportional to the mass and to the height of the interior levels (pseudo-triangular) and proportional to the mass (uniform). The control node has been placed in the centre of the masses at the top level of the building. The model has been calculated with the parallel option in OpenSees, using multiprocessors. Due to the characteristics of the computer, the model has been divided into 24 parts.

5 RESULT AND DISCUSSION

In this section, the results obtained from the modal analysis and the NLSA have been analysed and discussed. Furthermore, the damage analysis of the tower has been shown by means of the damage patterns.

5.1 Modal analysis

The modal analysis has been carried out in order to obtain de main vibration shapes and the periods of the tower. In this regard, the results have been compared with the *in situ* measurements used to calibrate the numerical model. Mode of vibration 1 and 2 are translational in the X and the Y direction, respectively. Mode 3 is torsional. It is important to note that the translational modal shapes deform diagonally. The experimental dynamic parameters of the tower have been extracted from the OMA. For this, the ARTeMis Modal Pro software [28] has been used. The values of the periods and the frequencies obtained are very close to the *in situ* measurements results (OMA). The period values obtained with the FEM are 1.51 s and 1.46 s for Mode 1 and Mode 2, respectively, and 0.43 s in the torsional mode (Table 2).

Modes	FEM Frequencies (Hz)	OMA Frequencies (Hz)	FEM Periods (s)	OMA Periods (s)	Difference (%)
1	0.66	0.66	1.51	1.51	0.0
2	0.68	0.70	1.46	1.42	-2.9
3	2.28	2.01	0.43	0.49	11.9

Table 2: Results of modal analysis. Experimental and numerical results of three vibration modes.

5.2 Non-linear static analysis

The global seismic behaviour of the tower has been assessed through NLSA. This method has been widely used for this purpose [29] and suggested in the Eurocode 8 [30]. Due to the unsymmetric configurations caused by the irregular distribution of the ramps and the openings, the NLSA has been conducted in both directions (X and Y). It has also been applied in the negative and the positive orientations. Furthermore, two load patterns (triangular and uniform) have been employed to compare the results (Figure 5). The analysis has been performed until the base shear dropped by 20%, usually indicating the collapse state of the structure.

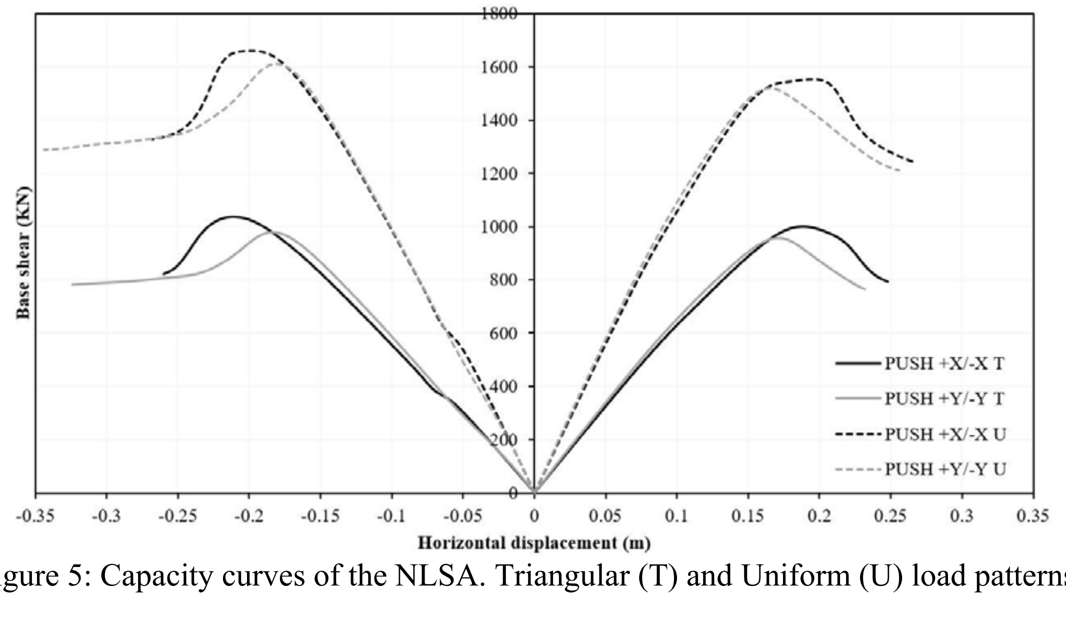

Figure 5: Capacity curves of the NLSA. Triangular (T) and Uniform (U) load patterns.

As Figure 5 shows, the models calculated with the uniform load pattern have a higher capacity than those calculated with the triangular one. This may be explained by the application of the horizontal load, which is added throughout the cane. This is more massive than the upper bodies. In general, the capacity in the X direction is higher than in the Y direction.

5.3 Damage pattern

The global damage distribution has been analysed using the different damage patterns from the NLSA in the X and Y direction (Figure 6 and 7). In this regard, the tensile damage models have been plotted because they are more severe than the compressive damage. Furthermore, these models have been plotted for the ultimate displacement. The main objective is to identify the areas of damage concentration in the tower.

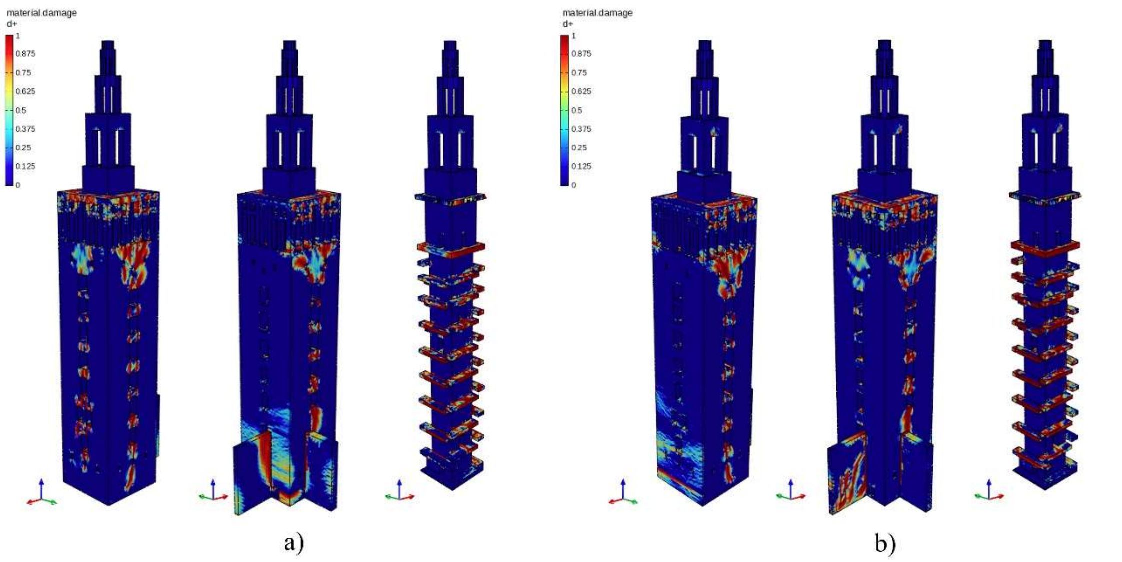

Figure 6: NLSA damage patterns X direction. a) +X direction and b) -X direction.

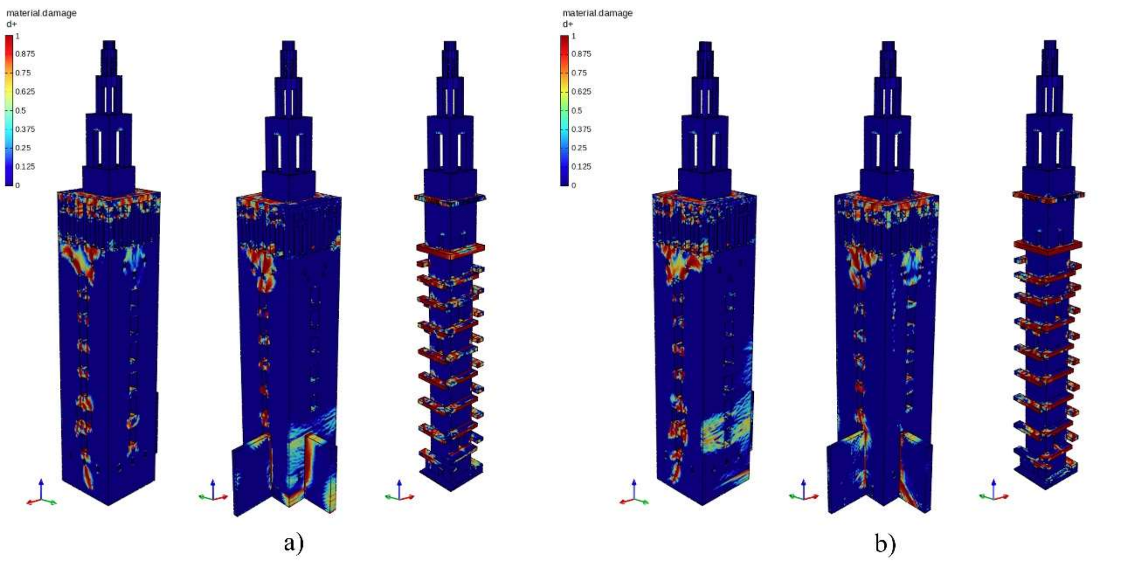

Figure 7: NLSA damage patterns Y direction: a) +Y direction and b) -Y direction.

In the cane of the tower, the greatest damage is concentrated near the openings since it is a very weak area: the hollows are irregularly located in the middle of each façade. Overall, the highest damage is concentrated near the belfry level. Figures 6 and 7 show that the ramp elements would be seriously damaged. Damage is also concentrated in the contact between the tower and the equivalent wall of the Cathedral. Furthermore, these walls produce a significant irregularity in the deformation of the numerical model. This is due to the fact that these walls are not symmetrical, being only in two façades. Finally, the upper body is not expected to suffer from significant damage.

6 CONCLUSIONS

This work has presented a preliminary seismic assessment of the historical Giralda tower. To do so, an advanced 3D FEM model has been developed, considering the available information of the building and *in situ* surveys. The results of the numerical modal analysis and the ambient vibration test (OMA) have been used to calibrate the 3D FEM. Finally, the global seismic behaviour of the tower has been assessed through NLSA. The following conclusions are derived from this work:

- These types of work provide useful information and knowledge about the seismic safety of complex heritage buildings. The results obtained can be used in future restoration projects of heritage buildings. Furthermore, the method developed in this work can be also applied for the assessment of other masonry towers.
- The field surveys conducted in this study have been important to identify the mechanical characteristics of the materials and the geometrical issues for the development of the numerical model. In this regard, the ambient vibration test is fundamental in the calibration of the FEM of complex ancient buildings.
- This work has revealed that it is important to apply at least two different load patterns in the NLSA, as recommended by Eurocode 8. Furthermore, it is highly important to consider the boundary conditions since they might introduce some irregularities in the behaviour of heritage buildings.
- The highest damage is concentrated near the belfry level because it is a weaker part compared to the cane of the tower. Furthermore, the interior ramps might be completely damaged. These are very weak structural element between two rigid bodies. Note that these structural elements were the most damaged ones in past earthquakes, according to the testimonies.
- In this study, NLSA has been applied to assess the global seismic behaviour of the tower. In future studies, nonlinear dynamic analyses are expected to be carried out.

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