

STRUCTURAL BEHAVIOUR ANALYSIS OF THE BELL TOWER OF SANTA CATALINA'S CHURCH OF VALENCIA

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Abstract. *The Bell Tower of the parish church of Santa Catalina Martir of Valencia is definitely a key piece of the Valencian Baroque. In the thirteenth century the Church rose over an ancient mosque, and the new tower was built between 1688 and 1705. It is unknown who the master builder of this “campanar salomonich” is although everything seems to point to Pérez Castiel. The Bell Tower is a hexagonal plant, completely built of masonry and has significant baroque decoration of the last quarter of the seventeenth century, transferred effectively to the stonework.*

The Church, originally conceived of a single nave, has major structural problems resulting from the emptying of the buttresses to generate the side chapels in further enlargement. This structural pathology became patent in 1529 and lasts until today. Improper geometry of its constructive elements and wrong construction result in serious structural problems. For all these reasons, in addition to its high slenderness, the Bell Tower of Santa Catalina can be considered a unique example of constructive and structural analysis.

This study focuses on the use of numerical models in order to develop an accurate scientific research. It may provide conclusions which clarify doubts regarding the structural behaviour of the belltower. Graphic surveying is performed using the latest tools such as advanced photography techniques to obtain exact plans reflecting the current deformation of the monument. Based on the results achieved, several analytical methods have been carried out: a linear static analysis for gravitational loads, a modal analysis, a nonlinear static push-over procedure and a non linear time-history analysis (dynamic). The seismic response of the tower has been evaluated using a damage-model testing. Comparing the results obtained permits evaluating the structural response of the Bell Tower.

1 ANALYSIS OF ITS STRUCTURAL HISTORY

The Bell Tower of the Church of Santa Catalina is such an extremely important architectural element that it is considered to be a key example of the Valencian Baroque. Such is its value that it became an object of patrimonial valuation in 1981. This architectural jewel was able to prevent an urban plan of the early 20th century. This plan aimed to demolish both tower and church in order to prolong the current Calle de la Paz. Nowadays the Bell Tower presents its role as a landmark and a focal point of this emblematic road.



Figure 1: The Bell Tower of Santa Catalina's Church

Santa Catalina's Church construction began after the Conquest on an ancient mosque. Towards the year 1300, the Tapineros Guild decided to extend it due to its limited capacity, forming a parish church of a single nave with side chapels between buttresses, probably with polygonal head and cubic Bell Tower to the right of the access through its feet, as it can be seen in the image of the city by Anthonie van den Wijngaerde in 1563. The nave is 28.56 meters long by almost 13 meters wide, with a height up to the cornice of 11 meters. The Church is located between houses attached to it, thus only its main façade can be seen from the street, facing Lope de Vega square. Another gate of Gothic style overlooks the square of Santa Catalina, located between the current Bell Tower and another house. (Figure 2)

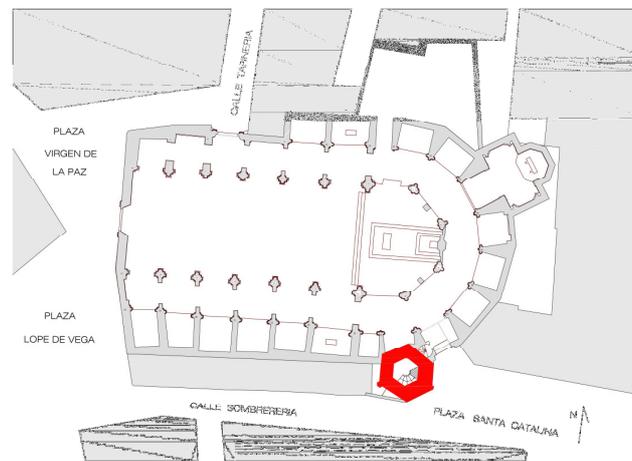


Figure 2: Santa Catalina's Church plan

There can be found documented works from Pere Balaguer between 1406 and 1411. About 1472 the Church renewed its head in Gothic style with ambulatory, imitating the Cathedral. The Church was consecrated in 1520. The emptying of the buttresses to form the side chapels caused structural problems that became patent in 1529. On the 29th of March 1584 the Church suffered a fire that forced to repair it a few years later in order to recover the masonry. The first meeting of parishioners is documented on the 1st of January 1688, when it is spoken of the desire to build a new Bell Tower. On the 5th of October of this year it began to be built up from the hand of Juan Bautista Vinyes and was finished in 1705. The documentary sources do not clarify if both belfries once coexisted.

The process of selecting the new location, tracking of the works, funding, construction and assembly of the coronation and the acquisition and assembly of the bells is very well documented, according to Orellana, but it does not refer to the author of the trace. However, based on compositional similarities with other churches nearby, everything seems to indicate that it is the same way to work of Juan Perez Castiel [1]. The initially estimated duration of the works was of 8 years, but the economic crisis forced it to be extended a few years more than expected. These economic difficulties can justify a poor construction, far away from the agreed. Another misfortune was that during its construction they realized that the execution of a staircase had not been planned. The problem was taken to the Courts and eventually the parish paid a staircase which was built partially out of the tower [2]. On the 10th of January 1730 new bells were placed, being cast in London by Richard Phelps the previous year, and the bell-ringer Antoni Balaguer replaced the ones of the old belfry in two months' time.

The floor plan of the new belfry is hexagonal and is built entirely in stone. The bell tower has three main bodies: base, bells room and coronation. The base is 29.48 m high with 1.57 m wall thickness; in one of its sides there is a stairway that gives access to the bells room. The bells room reaches the height of 38.00 m. The total height (coronation included) is 50.00 m. The perimeter in its base is 21.60 m.

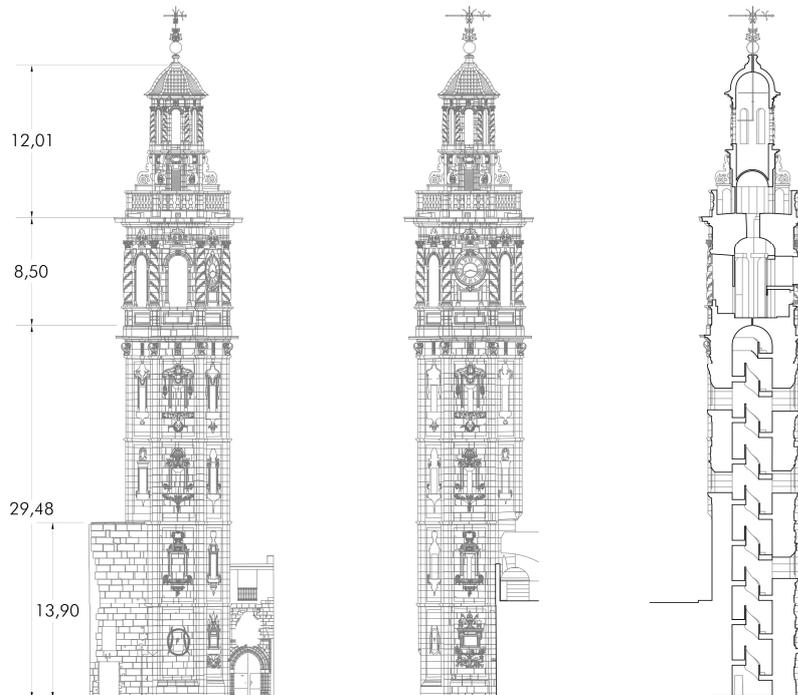


Figure 3: The South-East façade, the East façade and a vertical section of the Bell Tower

It combines the constructive Gothic tradition of the beginning of the 17th century, buttresses in corner and fascias separating the levels, with the baroque decorativeness of the end of the same century transferring it to the stonework, fact that turns it into one of the most important baroque Valencian Bell Towers. The windows are decorated with baroque motives. It possesses a powerful entablature above which the bells room is located, in this level the buttresses are replaced by spiral columns. The lower part of the tower ends with a balustrade and the coronation is composed by another little tower also hexagonal in two tiers. The low tier has trabeated holes and diagonal supports. The upper tier has arches and spiral columns that support an entablature and a small dome of tiles. The tenet of straight cornice is broken since all the cornices reproduce in their flight the perimeter of the buttresses, which constitutes a compositional novelty. The fact of considering in the original tracing of the Bell Tower the top body makes the designed vaults capable of supporting them and allows higher and more slender solutions, being necessary to introduce the diagonal supports to stabilize the upper tower.

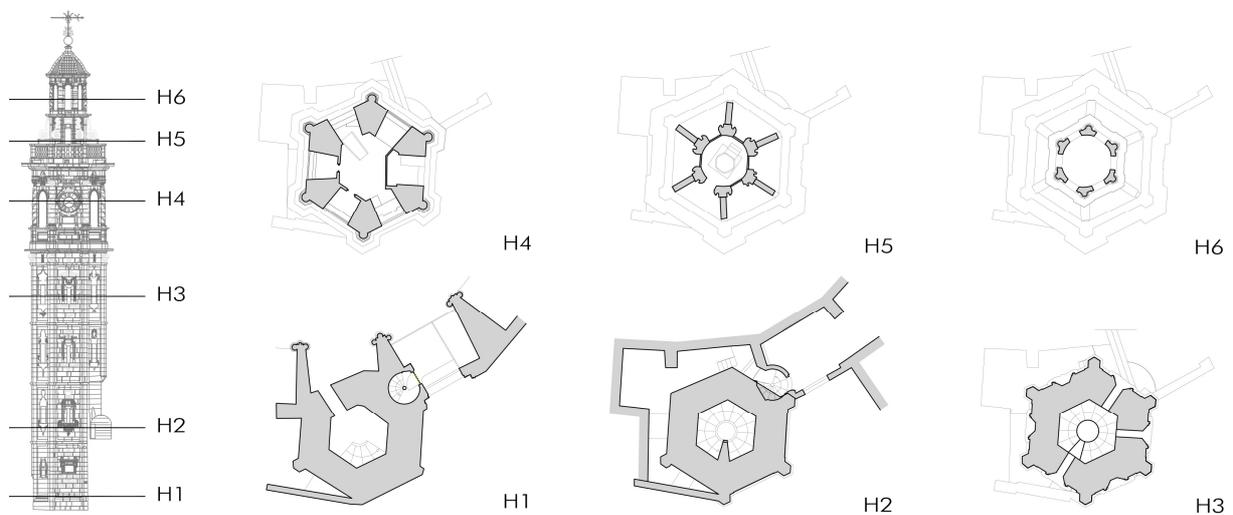


Figure 4: Several horizontal sections of the Bell Tower

Between 1740 and 1755 there were interventions carried out inside the Church whereas in the belfry none took place until 1867. The scope of the works is unknown, although all seems to indicate that the tile covering was renovated. It is not until 1914 that the clockmaker José Gómez installs the clock of the Tower, blinding one of the windows of the bells room. It is on that date when the Town Hall has already dismissed the demolition of the tower to get the extension of the Calle de la Paz. In 1936 the Church suffered a great fire, forcing to intervene in its structure once the war was finished. A global restoration of the whole temple to its original state was planned from 1950. The Valencian architect Luis Gay Ramos directed the works, which were carried out in three phases. There is no doubt that these actions did not affect the tower, since it had not been damaged by the bombing of 1938 and because the masonry was apparently very solid, so it did not need any renovation. The last intervention, and the only one in its stonework since it was built, was in 2004 as emergency works to consolidate and to preserve the Baroque Tower. The works were directed by the architect José Ignacio Casar Pinazo and consisted of solving problems of sternotomy and re-joint of the stone, works that were pending since its construction, therefore symptoms of an inadequate execution. In addition, the clock was dismantled and the Bell Tower was returned to its original architectural form, restoring the original cover of the top formed by a stone dome of a single leaf with exterior finishing carved in fish-scale pattern.

2 NUMERICAL ANALYSIS

The seismic response of the Bell Tower of the Santa Catalina's Church has been evaluated through several analytical methods such as a nonlinear static analysis for gravitational loads, a modal analysis, a nonlinear static push-over procedure and a non linear time-history analysis (dynamic).

2.1 Structural model.

To do the numerical calculation the three bodies of the Bell Tower have been modelled. The 3D model is based on a solid finite elements mesh that reproduces the geometry of the belfry. The model consists of 101,094 degrees of freedom, a total of 33,962 nodes, with 50,352 solid elements of which 32,148 tetrahedral and 18,204 hexahedral elements. The tower is supposed to be clamped at the ground level. Horizontal displacements are restrained with spring-link elements in the North and North-West corner up to a height of 13.90 meters because of the contact with the wall of the aisle nave [3]. In order to evaluate the stiffness of those springs, the total horizontal force applied in the bell tower has been divided into the number of its nodes in contact with the church. The stiffness of springs has been calculated connecting that force to a restricted horizontal displacement. These constraints can be observed in figure 5.

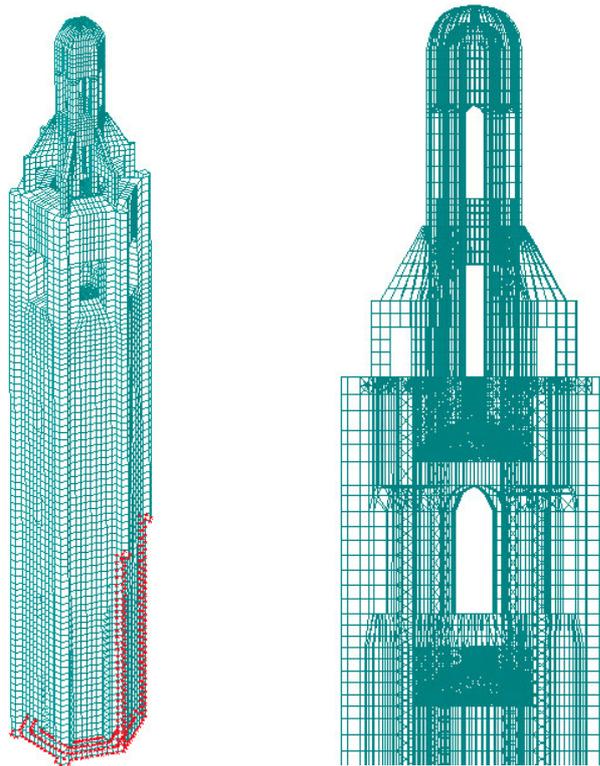


Figure 5: Structural model

2.2 Nonlinear static analysis.

The first analysis that was carried out allows us to know the stresses in the masonry. This study has been done concentrating only on the self-weight hypothesis and leaving the current cracks out of consideration. The self weight has been defined through the parameters given in

Table 1, which also contains the most important mechanical properties considered in the numerical model [4].

Table 1: Mechanical properties.

Stone masonry	
Density	2,200 t/m ³
Module of deformation	10000 N/mm ²
Poisson's ratio	0,2
Compressive strength	12,00 N/mm ²
Tensile strength	0,40 N/mm ²
Fracture energy	0,1 Nmm/mm ²

2.3 Nonlinear analysis damage model.

It is necessary to calculate the maximum structural load as well as the position and value of the cracks to obtain the non linear response of this structure. This study has been realized by the ANGLE finite elements software [5] based on the doctoral thesis of professor A. Alonso Durá [6], in which both the so-called isotropic damage model and Newmark method for dynamic time-history analysis were used. [7, 8]

Damage mechanics, within Continuum Mechanics, consists of introducing micro-structural changes into material behaviour through several internal variables. These variables reproduce in the model the influence of the historical behaviour of the material in the stresses evolution.

The mechanical response of a material, as the appearance of fissures and his evolution over time, can be described by a function. Therefore the non-linear masonry behaviour can be represented in a model. The fissures are showed as a local damage effect; it is defined according to the material known parameters and to functions that control the damage evolution with the successive tension gravitational and seismic loads at each point.

The concept of isotropic damage is a point in the material with a certain degree of damage is considered, deterioration is represented as hollows in the fabric.

The damage variable “d” is defined as equation (1) shows, where: S= is the total surface under consideration; \bar{S} = the effective resistant area; and S- \bar{S} = the hollowed surface. This index expresses the material deterioration degree. The zero value represents the undamaged state, while 1 is the total damage of the resistant area.

The relationship between Cauchy’s standard stress and the actual stress acting on the part of the effective resistant section is derived from the equilibrium condition.

This scalar index is enough to adequately represent the materials behaviour such as concrete, brick and stone. The effect on the mechanical behaviour of the material is a reduction of rigidity proportional to (1-d) (Figure 6).

$$d = \frac{S - \bar{S}}{S} \quad (1)$$

$$\sigma = (1 - d)\bar{\sigma} = (1 - d)E\varepsilon \quad (2)$$

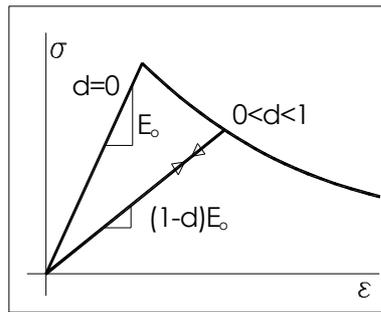


Figure 6: The effect on the mechanical behaviour of the material is a reduction of rigidity proportional to $(1-d)$

In the repeated FEM process the constitutive matrix $\bar{\mathbf{D}}$, being \mathbf{D} the elastic constitutive matrix, is calculated as:

$$\bar{\mathbf{D}} = (1 - d)\mathbf{D} \quad (3)$$

The scalar variable of damage is d , being the r , r^0 , and A the values obtained as in reference [9, 10].

$$d = 1 - \frac{r^0}{r} \exp \left\{ A \left(1 - \frac{r}{r^0} \right) \right\} \quad (4)$$

2.4 Model calibration.

The model before described has been implemented in ANGLE software by the author. It is extremely difficult to calibrate the model before described with ashlar test due to calibration is made with experimental concrete models. Those models have concrete elements with a similar behaviour to masonry, particularly Walraven tests on a reinforced concrete beam have been employed, and the tests made at Polytechnic University of Valencia by Valcuende and other authors about concrete non-reinforced beams. [11, 12]

2.5 Seismic action

In order to study the pushover analysis an acceleration spectrum is used for the area of Valencia and for a mean return period of 950 years, as it is specified in EC8 [13].

So as to the non linear time-history analysis (dynamic) there are not enough real records about accelerograms of this area. Some artificial accelerograms have been generated and they are compatible with the spectrum defined by EC8, which had been obtained with the program SIMQKE_GR [14].

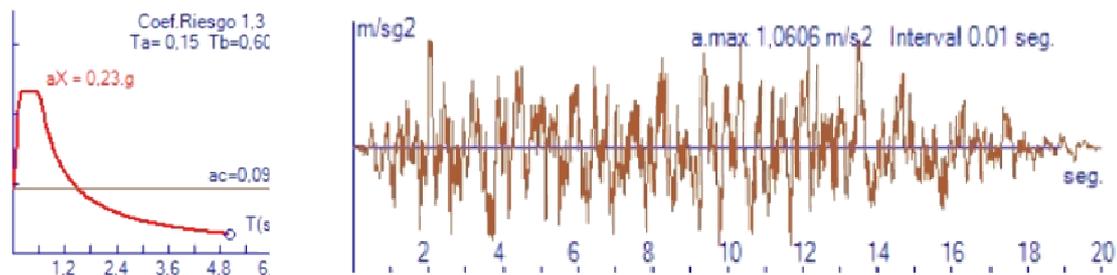


Figure 7: Elastic response spectrum. Artificial accelerogram

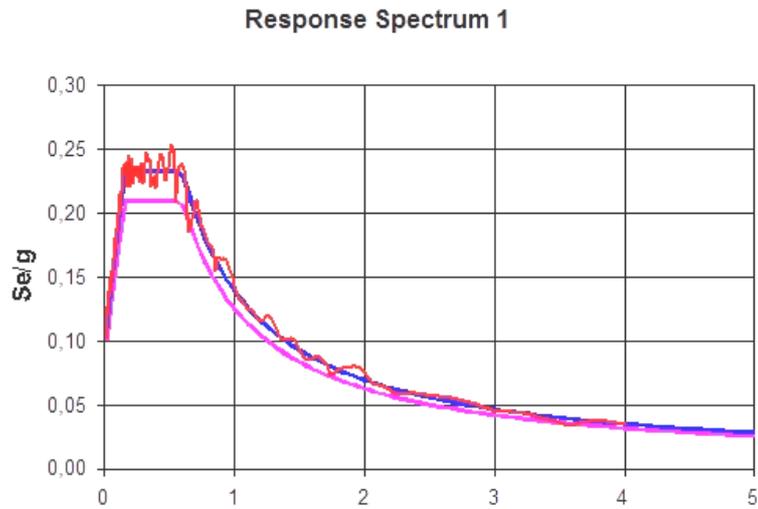


Figure 8: Comparison of design spectrum and the spectrum generated by the artificial accelerogram

3 ANALISIS OF RESULTS.

It has been considered a total of 15 vibrational modes. The table 2 shows the dynamic characteristics and the figure 10 several images of the most representative modes 2, 4, 6, 10 and 11.

Table 2: Dynamic properties.

Mode	Frequency (Hz)	Period (seg)
2	1.568482	0.6375593
4	6.039143	0.1655864
6	8.844769	0.1130612
10	16.38383	0.06103578
11	16.76815	0.05963688

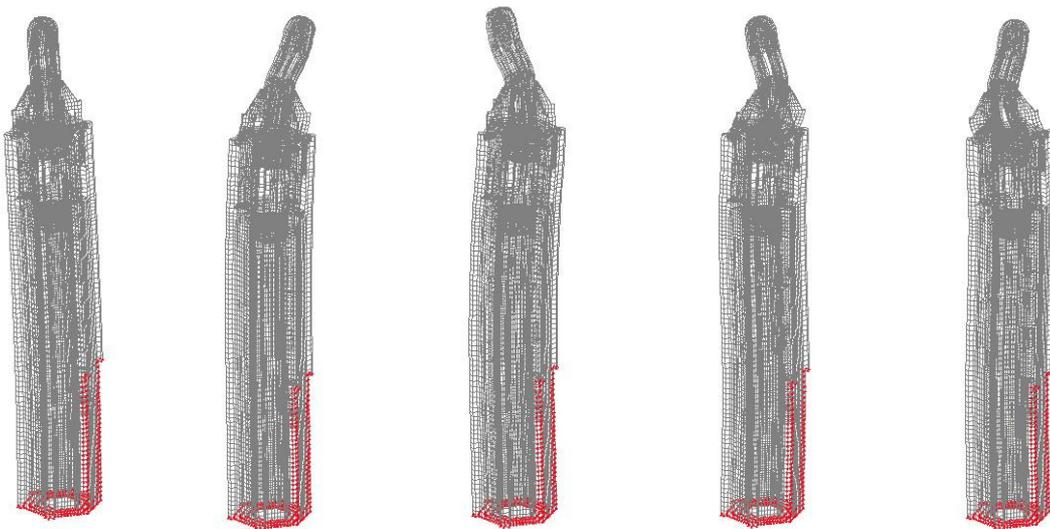


Figure 9: Vibrational modes (2, 4, 6, 10 y 11)

As shown in the following graphic, the dynamic properties of the model are located in the areas where the spectrum is higher and therefore the effective accelerations have higher values.

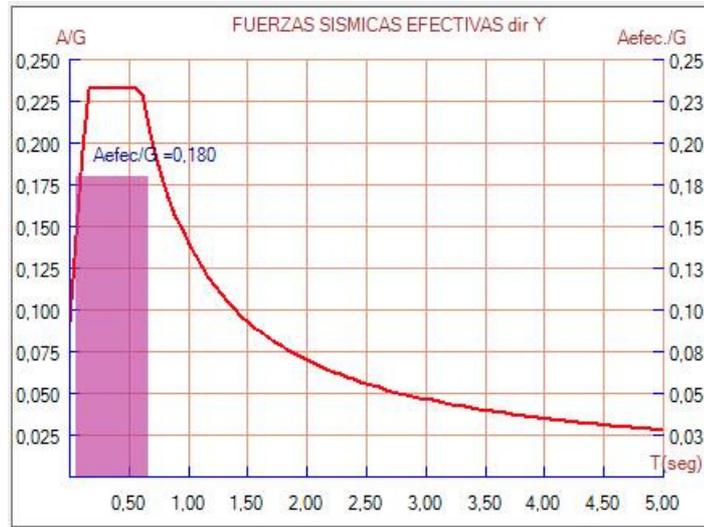


Figure 10: Seismic effective forces

Below are represented the isovalues for stresses to gravitational loads of compression in vertical direction. The images show as the maximum stresses are concentrated at the base of the Bell Tower reaching values of 1.948 N/mm^2 . It is a very lower stress than 12 N/mm^2 , which is the resistance to compression of the masonry considered in the calculation. The index of damage is insignificant to gravitational loads as the graphic of values indicates.

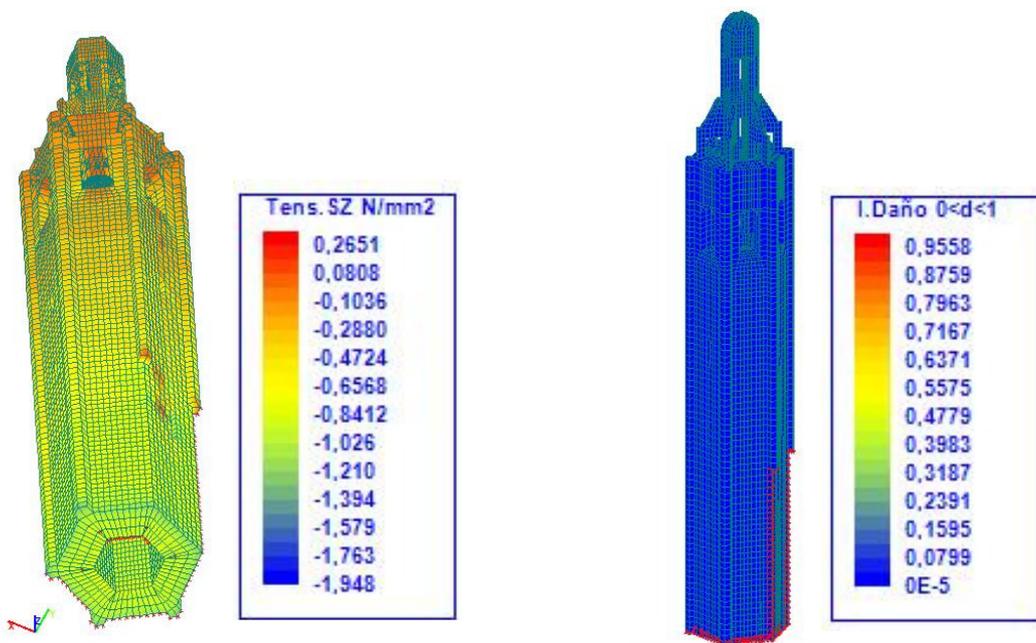


Figure 11: Stresses (Sz) and index of damage for gravitational loads

The control point (highest point) in the structural model has been selected for which the graphs listed below have been obtained. Through pushover method there has been obtained the capacity curve of the structure.

According to the Annex B of Eurocode 8: Design of structures for earthquake resistance, the initial stiffness of the idealized system is determined in such a way that the areas under the actual and the idealized force - deformation curves are equal, as shown in Figure 12.

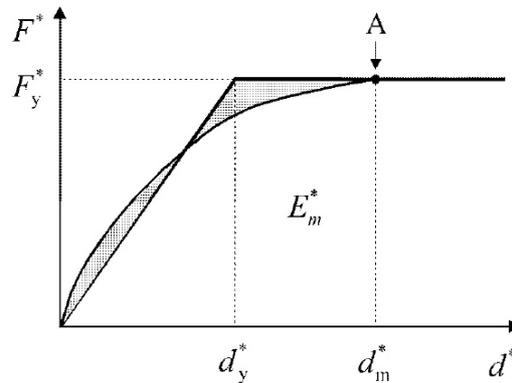


Figure 12: Determination of the idealized elasto- perfectly plastic force-displacement relationship

Where A is the plastic mechanism and E_m^* is the actual deformation energy up to the formation of the plastic mechanism.

In the image on the right the capacity of the structure is compared with the demand caused by the earthquake with a return time of 950 years. The maximum deformation in X direction is 2.62 centimeters (SDOF) and 4.54 centimeters (MDOF), correspondent with the performance point in an equivalent system of multiple degrees of freedom. This point is the intersection of the capacity curve and the demand spectrum that takes place after the point of break. Therefore, the point of performance indicates a level of complete damage, the collapse takes place.

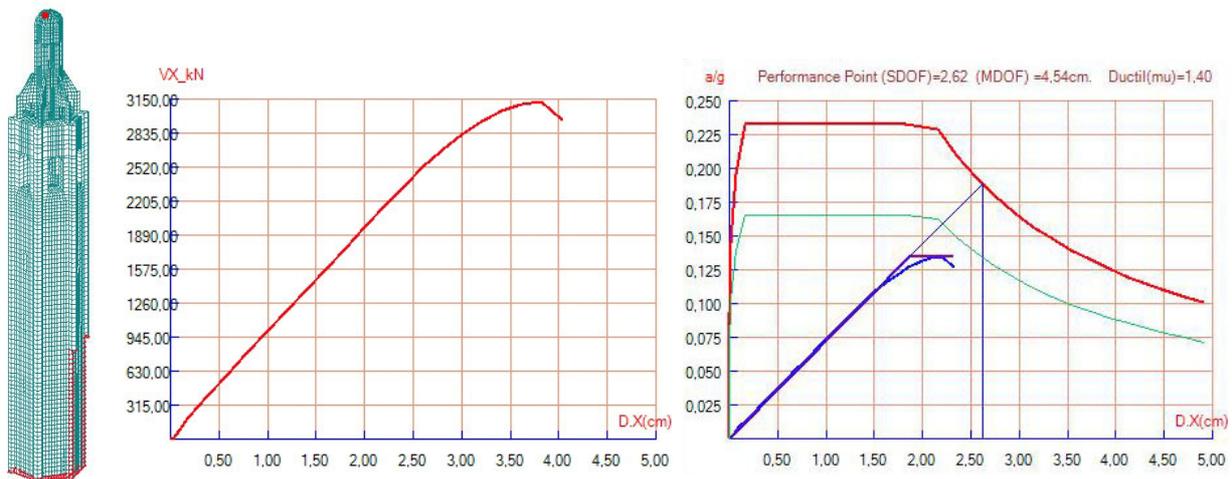


Figure 13: Capacity curve and performance point for demand spectrum

By the dynamical analysis method, the maximum deformation obtained for the control point is a displacement of 6 centimeters for 11.6 seconds.

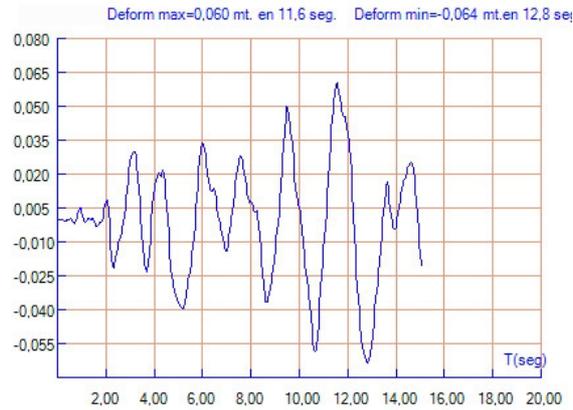


Figure 14: Movements registered by the control point

The lower image shows the damage index corresponding to the second 10.7 for the maximum displacement. It proves that the damage is concentrated at the upper body (coronation), which is the most vulnerable point of the Bell Tower. The calculations revealed the collapse of this part of the belfry.

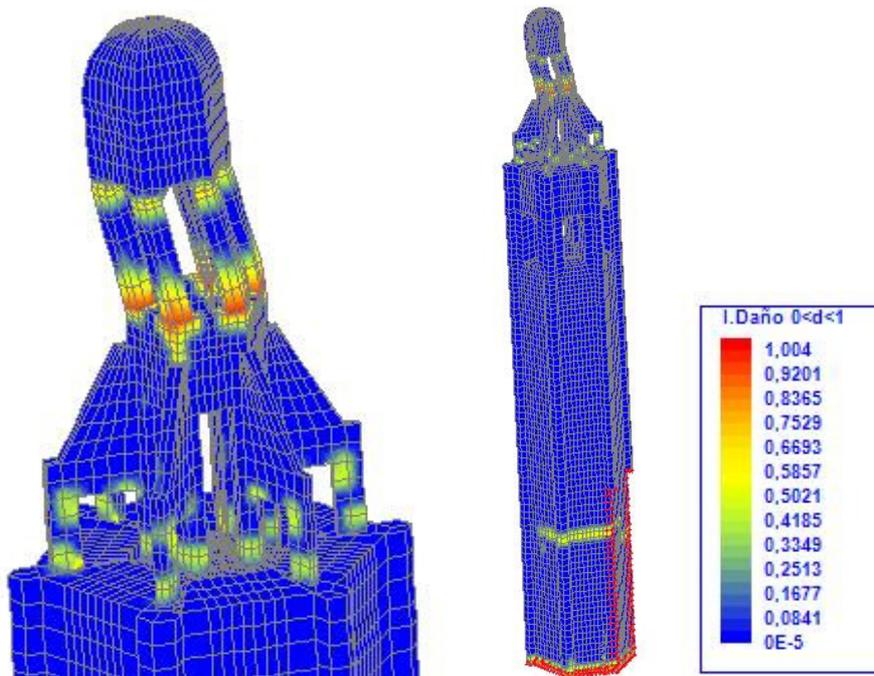


Figure 15: Index of damage for horizontal loads of seismic

4 CONCLUSIONS

This research verifies the concordance between the results obtained in the methods used for pushover and dynamic in the time analysis. It is important to highlight the contrast between the computational time of 7 hours and 40 hours respectively, which in spite of the complexity of the model is an attainable time.

From the results obtained by both methods we can distinguish that the first two bodies (base and the bells room) have an acceptable behavior to earthquake and on the other hand the upper volume presents a high degree of vulnerability with the possibility of the collapse taking place.

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