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Numerical and Experimental Study of the Dynamic Behaviour of San Nicolás Belltower (Valencia, Spain)

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ABSTRACT: This study shows the works carried out to characterize the dynamic behaviour of the San Nicolás belltower in Valencia (Spain). This is a masonry belltower finished in 1755. A complete geometric description of the tower has been carried. Different numerical models were developed and calibrated previously by means of dynamic tests carried out directly on the real structure. The seismic response of the tower has been evaluated using the Spanish Seismic Standards. On the numerical model the seismic forces are introduced using five different accelerograms in the base of the tower. This analysis has been carried out considering a non lineal analysis and a failure criterion to evaluate cracks on the tower. In spite of presenting a high grade of deterioration, the numerical results have been satisfactory for the basic seismic acceleration proposed by the Spanish Standard in the area.

1 DESCRIPCIÓN GENERAL

1.1 Introduction

The historical city centre of Valencia (Spain) is characterized by the large number of existing masonry belltowers. The historical reason for this belltowers is due to the Reconquista, which led to the end of the Moorish occupation driving the Arabs out of Valencia in 1238. A church was erected over every mosque and each parish constituted a clear separated sector in the new Christian city. San Nicolás church was one of the first ten parish churches built in the city after the Reconquista. First references are from 1245; since then, many restoration and consolidation works have been developed adapting the church to the architectural styles. The predominant architectural style is Gothic, covered in many places of the church by baroque paintings very common during XVII and XVIII centuries in Valencia.

The construction of the current belltower is later than the church itself, starting in 1658 and reaching the final shape in 1755. This is the reason why the style of the belltower is baroque exclusively, very similar to other next belltowers, excepting the Gothic Cathedral belltower. The belltower is located at the south-west corner of the church nave, as can be observed in the ground plan by De la Campa (2003) shown in Fig. 1.

The construction and remodelling of churches in Valencia was so intense that regulations were introduced by Aliaga archbishop in 1631 (Belloch, 1995) to standardize and rationalize the construction of these towers: "*Recommendations for church buildings and factories and for the things for the divine service*". In spite of these regulations, this belltower has a spiral staircase contradicting Aliaga archbishop. This makes more credible the hypothesis about San Nicolás belltower is the extension of a previous tower.

Looking at Fig. 1 it can be concluded that the base of the tower is resting on two buttresses from the nave. This construction system is very common in surrounding churches.

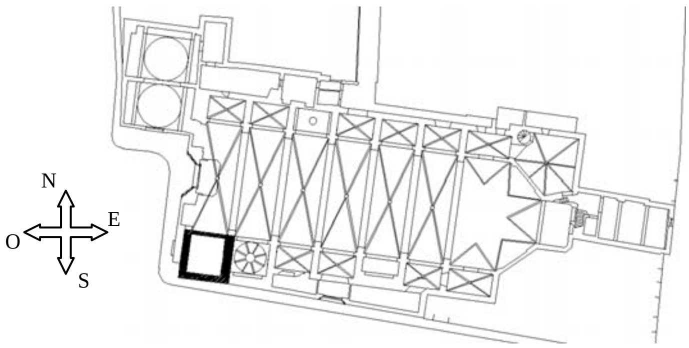


Figure 1 : Belltower location.

1.2 Geometric description.

The belltower has three main bodies typical from the baroque belltowers built in XVIII century: base, bell room and coronation, see Fig. 2. The main body possesses a square section with 6.62 m of side; inside there is a stairway that gives access to the body of bells. In the low part of the structure the masonry wall arranges brick masonry with ashlars (h=14.5 m) , maintaining a constant thickness of 1.0 m along the whole first body of the tower. From 14.5 m high, the tower is composed of brick masonry with walls thickness similar to those found in the first part of this body. The total height is 24.5 m. The bell room reaches the height of 32.5 m with 0.85 m walls thickness. Four windows are inside this body (in each wall) to accommodate the bells with 6.0 m^2 approximately each window.

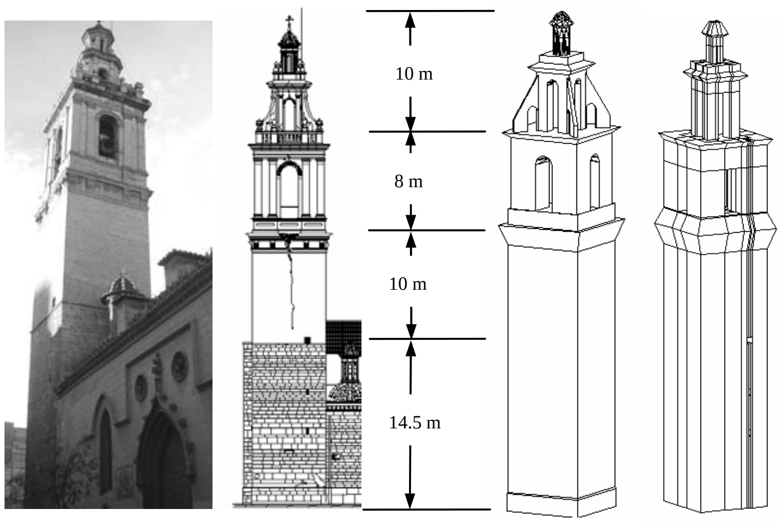


Figure 2 : South façade. Simplified models

The tower has a rigid joint with the lateral walls of the main body of the church. This joint can be observed in the north and east sides of the tower. The joint with north wall of the nave reaches the height of 14.5 m, although the joint of the east wall reaches 10 m high. Several vertical cracks of special relevance in the south wall of the tower can be observed.

2 DYNAMIC TEST

Several dynamic tests have been performed on the belltower to know the mechanical parameters, vibration modes (bending and torsion) and structural damping. All of them are based on the registration of ambient vibrations at different heights and directions. Only bending and torsional vibrations are registered due to the high longitudinal stiffness.

The works of Bachmann (1997) and Casolo (1998) fix the main torsional and bending frequencies between 0.9 and 2 Hz for slender towers. The work made by Ivorra (2006) in a similar belltower allows to evaluate frequencies and the experimental procedure.

From equation (1) proposed in NCSE 2002, belltower frequencies can be estimated:

$$\omega_1 = \frac{\sqrt{L}}{0.06 \cdot H \cdot \sqrt{\frac{H}{2 \cdot L + H}}}$$
$$\omega_2 = 3 \cdot \omega_1 \tag{1}$$
$$\omega_3 = 5 \cdot \omega_1$$

where: L is the plan dimension along the vibration direction and H is the height.

So, it is expected to measure a first frequency round 1.2 Hz or higher, since the stiffness is higher due to the contact with the church.

In order to make the dynamic experimental measurements, two piezoelectrical accelerometers have been placed at the height of the bell room as shown in Fig. 3.The work range of these accelerometers varies between 0.5 and 2000 Hz, with a conversion factor equal to 1000 mV/g. According to the arrangement commented, belltower vibrations in the E-W and N-S directions could be determined. The dynamic data obtained from the ambient vibration have been registered by a Kyowa PCD-320 equipment with a sample rate of 200 Hz. Temporal acceration movements have been analysed with DAS-100a Kyowa software to obtain frequency results.

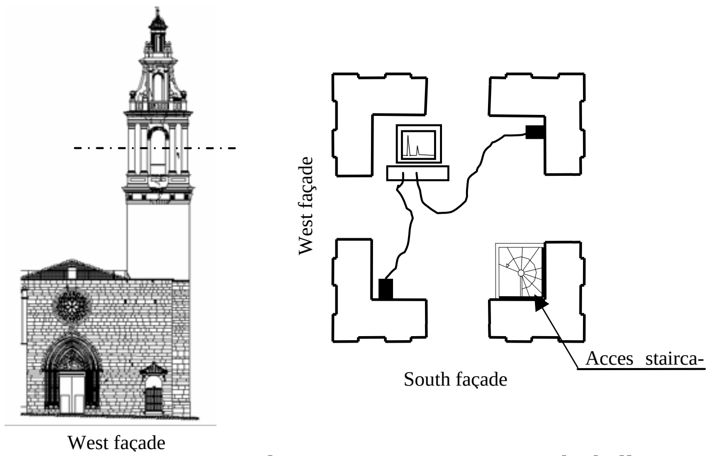


Figure 3 : West view. Accelerometers arrangement in the belltower.

Power spectra response in E-W and N-S directions have been obtained from ambient vibrations, and the modal parameters can be concluded from them as shown in Table 1:

Table 1 : Belltower natural frequencies.

Direction	Frequency (Hz)	Mode classification
E-W	1.6	Bending
N-S	1.759	Bending
	6.62	Torsion

The structural damping ratio is obtained from the results in Ivorra (2006), since the structures involved have very similar characteristics and similar period of construction. The average damping ratio used in the present work for the masonry belltower is 0.0159.

3 DYNAMIC STUDY

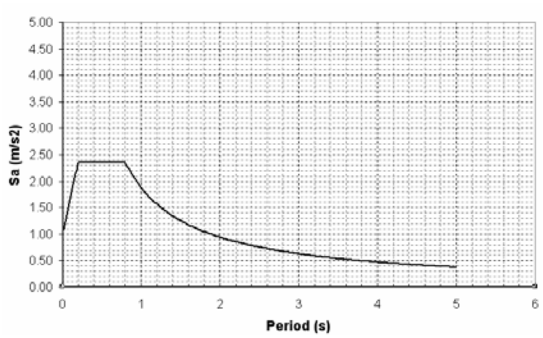
3.1 Loading

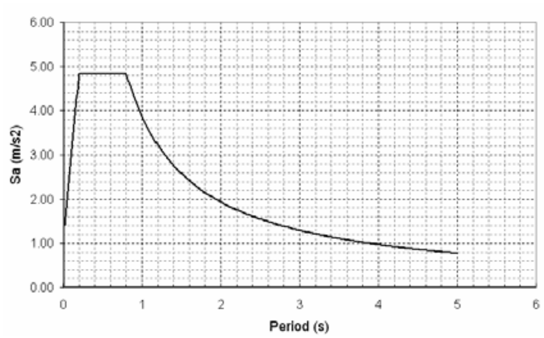
The loads used to assess the structural behaviour of the belltower have been the self weight and the seismic action. The self weight has been defined through the density of the material, with an average value of 1800 kg/m^3.

The seismic elastic spectrum value has been obtained according to the Spanish standard NCSE-02. The basic seismic acceleration at ground level for the city of Valencia is 0.589 m/s^2. (0.06g). The ground type is a soft clay. Using this data, the seismic spectrum curve is shown in equation (2). Fig. 4 displays this spectrum.

$$\left.\begin{array}{ll} T < 0.2 & S_a(T) = 0.942\cdot\left(1+1.5\cdot\dfrac{T}{0.2}\right)\ m/s^2 \\ 0.2 \le T \le 0.8 & S_a(T) = 2.5\ m/s^2 \\ T > 0.8 & S_a(T) = \dfrac{2}{T}\ m/s^2 \end{array}\right\} \tag{2}$$

Where T is the period and $S_a(T)$ is the ordinate of the elastic seismic response spectrum,





(a) (b)

Figure 4 : Pseudo-acceleration spectrum. (a) Damping ratio 5%. (b) Damping ratio 1.59%

A non linear analysis is performed so, synthetic accelerograms have been generated to introduced the seismic load at ground level as a transient load. The method proposed by Gasparini and Vanmarcke (1976) has been used to generate these accelerograms. The standard NCSE-02 states the use of, at least, five synthetic accelerograms when a transient calculation like this is going to be carried out. Results for one of the generated accelerograms are presented in Fig. 5.

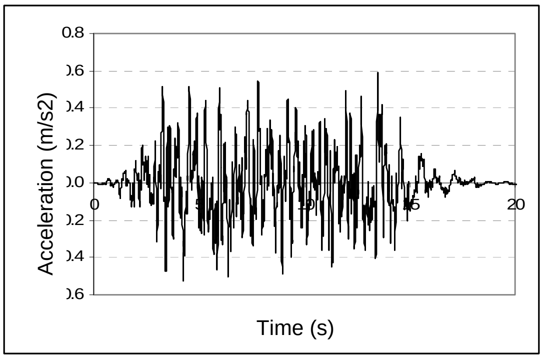


Figure 5 : Artificial accelerogram generated.

The time integration of these accelerograms offers the velocities and displacements produced at ground level, as shown in Fig. 5. Since the aim of the paper is the study of the behaviour of the structure and the failure mode in the event of a seismic motion referred to the city of Valencia, the accelerograms, velocities and displacements are reproduced only for one of them.

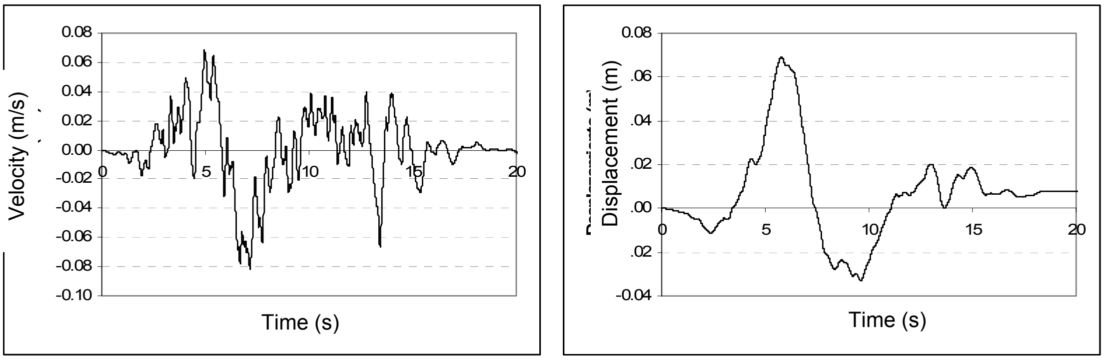


Figure 6 : Velocities plot. Displacements plot.

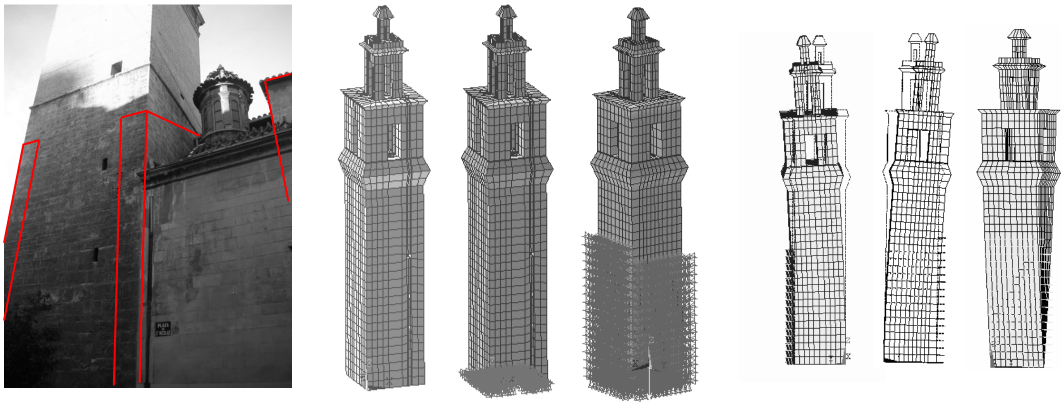
3.2 3D finite element model

As a first approximation, a simplified numerical model as shown in Fig. 1 have been used to know the belltower response to seismic forces. The numerical model has been done using the commercial software ANSYS. SOLID65 finite elements have been used to mesh the model, 8 node-hexaedral elements with cracking and crushing capabilities and three degrees of freedom per node. An iterative process has been performed to fit, through a modal analysis, the fundamental frequencies of the initial model and those registered from the real model. The results of this first stage are shown in Fig. 6 and Table 2.

The main assumptions for the numerical model are:

a. Average material density 18 kN/m^3 constant, as stated in the Spanish standard NBE AE –88 applied to masonry structures with solid brick. In the beam models it is supposed to be uniformly distributed.
b. The Poisson´s ratio of the masonry was held constant and equal to 0.15.
c. Linear and elastic mechanical behaviour during the calibration stage and modal analysis.
d. Non linear mechanical behaviour during the seismic study.
e. The tower is supposed to be clamped at the ground level.
f. Displacements are restrained in the W-E direction on the East wall up to a height of 14.5 m because of the contact with the lateral wall of the nave. Equally, displacements in the N-S direction are restrained in the North wall for the same reason. These constraints can be observed in Figs. 2, 3, y 6.

Fig. 7 shows the model, where 2262 solid elements, 390 spring-link elements, 99 mass elements and 26.964 degrees of freedom.



(a) (b) (c) (d) (e) (f) (g)

Figure 7 : Buttress in the tower. (a) Finite elements model of the tower. 3D view (b) West-South; (c) South-East; (d) North-East. Dynamic analysis results

After the iterative process, the Young's modulus is fitted for the masonry material (1500 N/mm^2. Table 2 and Fig. 6 show the agreement in the values for each frequency.

Table 2 : Finite element model. Natural Frequencies

Natural frequencies	3D Model (Hz)	Dynamic test (Hz)	Mode classification
1	1.65	1.6	Bending (N-S)
2	1.72	1.759	Bending (E-W)
3	6.5	6.62	Torsion

3.3 Failure criterion

Several authors have studied failure criteria in masonry, e.g. (Pallarés et al. -2004-). The failure criterion used in this work to account for the cracking phenomenon is that stated by Willam-Warnke (1975), which failure surface is shown in Fig. 8 in the principal stress space. The analytical expression can be found in Chen & Saleeb (1982). Plasticity can be accounted for, so the yield surface associated with this other nonlinear material behaviour must lie inside of the Willam and Warnke failure surface. Otherwise, the material will completely fail and never yield. The required masonry material parameters of the model are the ultimate tensile strength, f_t , the ultimate compressive strength, f_c , the ultimate biaxial compressive strength, f_{cb}, the ambient hydrostatic stress state, σ_{ah} , the ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state, f_1 , and the ultimate compressive strength for a state of uniaxial compression superimposed on hydrostatic stress state, f_2. The first two constants f_t and f_c are required. The others can be used by default to $f_{cb}=1.2f_c$, $f_1=1.45 \cdot f_c$, $f_2=1.725 \cdot f_c$. only valid for situations with a low hydrostatic stress component, or $\sigma_h \leq \sqrt{3} \cdot f_c$.

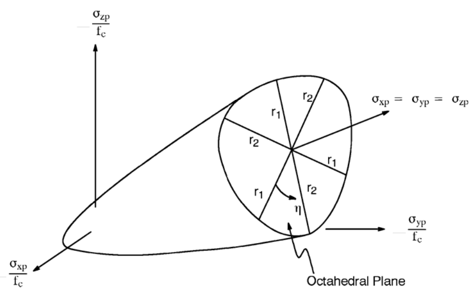


Figure 8 : Willam and Warnke failure surface

The masonry material of the tower has been considered as brick masonry stone with ashlars, having similar strength properties to brick masonry: f_c = 6376 kN/m^2 y f_t= 392 kN/m^2.

Although large deflections or large strains are not expected due to structure stiffness and masonry, a non-linear geometric analysis has been performed in order to consider possible P-Δ effects.

3.4 Results

Following the seismic calculations and testing several directions for the seismic load, the ground acceleration proposed by the standard to the city of Valencia (0.06g peak ground acceleration, (probability of occurrence in a period of 500 years) is resisted by the construction without cracks in the model. To assess the ultimate seismic action that the tower can withstand, the synthetic accelerograms have been scaled up to 0.08 g – which leads to a period of 1000 years-. Results for the latter case are shown in Fig. 8. Cracks appear firstly in the E-W façades when the accelerogram is introduced in the same direction. For the N-S direction, the crack pattern is smaller than the one presented in Fig. 9. The tower is more sensitive in the E-W direction relating to seismic loads. In the Spanish standard NCSE-02 "*the catalogued constructions as historical or artistic monuments*" will be classified as "Of special importance" reason why the reference return period for the earthquake used in its verification will be of 1000 years. This is not directly indiqued in the standrad but it appears reflected in the coefficient by which there is necesary multiply the basic seismic acceleration for constructions of “special importance”: 1.3 that corresponds to the expression: $a_n = a_b \cdot (P_{Rn}/500)^{0.4}$. Where a_b is the basic seismic acceleration referred to a period of 500 years and P_{Rn} is the new period.

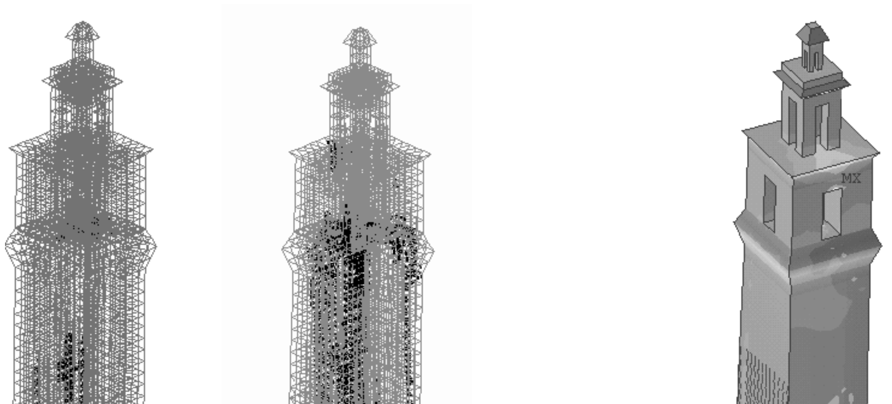


Figure 9 : Crack pattern. 1.4 s, 1.6 s, 2.0 s.

Studying the crack pattern it can be concluded that vertical cracks along the East wall of the tower would appear in the whole wall as a consequence of the seismic load.

4 CONCLUSIONS

From the numerical study of the San Nicolás belltower it can be concluded:

a. A numerical model including cracking criteria can estimate the non linear behaviour of the structure to seismic motions.

b. A first approach has been conducted fitting the model through a modal analysis. Stiffness changes in the contact with the church lead to consider this approach as an approximation.
c. The seismic risk (probability in a period of 500 years) is low with an admissible response of the structure.
d. Higher periods (1000 years) could lead to severe damage in the structure or collapse.
e. The worst direction for the action of the seismic load is E-W which leads to vertical cracks in the whole East wall spreading later to the West wall.

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