Structural Analysis of Historical Constructions, New Delhi 2006
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Analysis of the Seismic Behaviour of a Masonry Bell Tower

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ABSTRACT: The structural behaviour of a masonry bell tower located in a seismic area is studied when an earthquake is introduced at the ground level. Masonry material macromodelization, artificial accelerograms synthetically generated compatible with a response spectrum according to standards, crack pattern and non linear behaviour, time history analysis and the maximum seismic action in terms of peak ground acceleration that the bell tower can withstand are the main points treated in the present paper.

1 INTRODUCTION

Nowadays there are many masonry constructions which structural stability to dynamic actions (such as small earthquakes, strong winds, etc) is so precarious that could lead to a sudden failure as shown in Binda et al. (1992).

The present work explains the structural response of the tower of the San Pedro Apóstol church in Agost (Alicante, Spain) (Fig. 1) when a seismic action proposed in the current Spanish standard is involving the tower.

(a) (b)

Figure 1 : Tower of San Pedro Apóstol; (a) North-West façade. (b) South-East façade

2 MATERIALS

The tower of Agost is built in masonry with bricks and mortar. The mechanical properties of this material clearly complicate the structural analysis of the construction (Eurocode 6 1996) since masonry is an heterogeneous material with a strong anisotropic character and a non-linear behaviour (Page 1981).

The material has been modelled using a macromodelling approach because of the great number of elements forming the structure, and because of the geometrical complexity. In this way, masonry is treated as a homogeneous continuous medium with general mechanical properties that show the global behaviour of the structure (Martínez et al. 2001). These average properties assumed for masonry are:

Uniaxial compression strength: $fc = 6.5\ N/mm^2$.
Uniaxial tensile strength: $ft = 0.2\ N/mm^2$.
Young's modulus: $E = 6000\ N/mm^2$.
Poisson's coefficient: $\nu = 0.2$.
Density: $\rho = 1900\ kg/m^3$.

3 MODELLING

The structure is built with solid brick of the type which was common in XVIII century and mortar, as shown in Cantera (1983). The height of the tower's crown is 35.6 m and presents a square plant (6 m per side) with the three typical bodies in bell towers at that time: basement, bell tower and crown. The thickness of the walls varies in the base from 1.60 m to 0.6 m, getting thinner as the height increases (Fig. 2).

When performing the calculations, the aesthetic ornaments of the façades and the roof are not taken into account since they have not a significant influence in the final results, as commented by Ivorra (2002).

Many models can be used to reproduce the behaviour of a particular masonry structure (Lourenço 1996) but, in this work, a 3D model has been used to reproduce the actual geometry of the tower accurately (Fig. 3a). This geometry and the subsequents numerical calculations have been made employing the commercial software ANSYS (Ansys manual). Boundary conditions are introduced in the nodes at the basement level of the tower, restraining vertical displacement and allowing free horizontal movements to apply the earthquake loading properly.

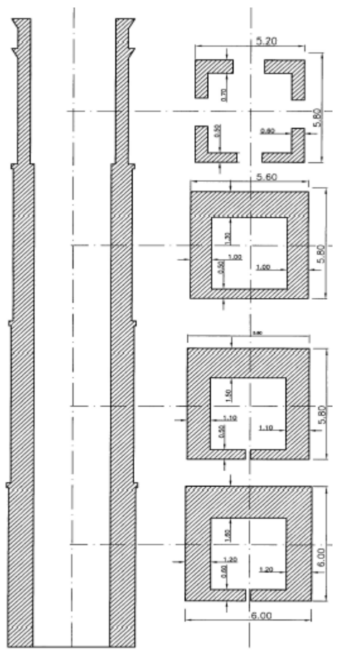

Figure 2 : Section of the tower in the West façade.

The finite element method is applied to this masonry structure (Page 1978) thus, a spatial discretization of the structure is performed. The continuum structure with infinite degrees of freedom is simplified into a discrete one. (Fig. 3b).

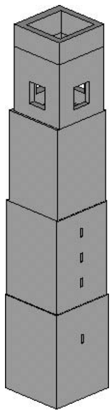

Figure 3(a) : Tower model.

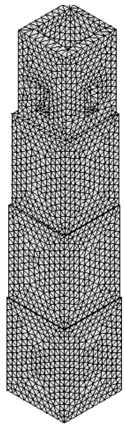

Figure 3(b) : Discretization by means of elements of 0.6 m, resulting 23741 elements and 6192 nodes.

The element SOLID65 (3-D Structural Solid) from the ANSYS element library is used in the 3-D model of the tower. The element is defined by eight nodes having three degrees of freedom per node: translations in the x, y, and z directions. The initial behaviour of this element is isotropic but it possesses cracking and crushing capabilities. Once the element has cracked the shear transfer can be reduced through the use of some coefficients depending on the cases "smooth crack" or "rough crack". This fact could cause sliding in the crack plane. If the crack is closed by means of compressive stresses, all these compressive stresses can be transmitted and only part of the shear forces can be transferred, if desired.

A linear elastic behaviour has been considered until cracking or crushing appear. When cracking arises in a integration point, the stress-strain matrix is adjusted introducing a plane of weakness in the direction normal to the crack face under the theory of smeared cracking. The well known Willam-Warnke's criterion has been used in order to collect this behaviour (Fig. 4).

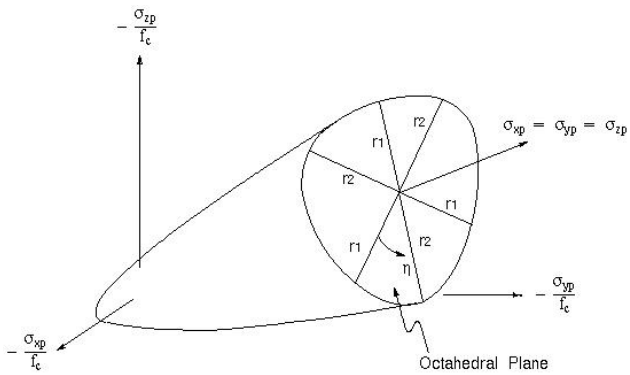

Figure 4 : Willam-Warnke failure surface.

4 DEFINITION OF THE SEISMIC LOADING

The seismic action has been artificially generated compatible with a response spectrum in accordance with the present regulations about seismic-proof constructions in the city of Agost, and scaled until the failure of the tower was achieved.

The Spanish standard NCSE-02 (2002) states an elastic response spectrum of horizontal accelerations at ground level with 5% of structural damping.

In order to obtain the synthetically generated accelerogram compatible with the previous response spectrum, the method proposed by Gasparini and Vanmarcke (1976) is used. The artificial accelerations are obtained through an stochastic process involving trigonometric functions, as shown in equations (1) and (2):

$$a(t) = I(t)\sum_{j=1}^{n} A_j \cos(\omega_j \cdot t + \phi_j) \tag{1}$$

$$A_j \approx \sqrt{2 \cdot G_z(\omega_j) \cdot \Delta\omega_j} \tag{2}$$

where I(t) is the intensity function proposed by Jennings et al. (1968), A_j are the amplitudes and G_z is the power spectral density function calculated from the response spectrum. The variables ϕ_j are n- independent random values uniformly distributed in the $[0\ ,\ 2\pi]$ interval.

Once adjustments in the resulting values are performed, the artificial accelerogram shown in Fig. 5 is achieved.

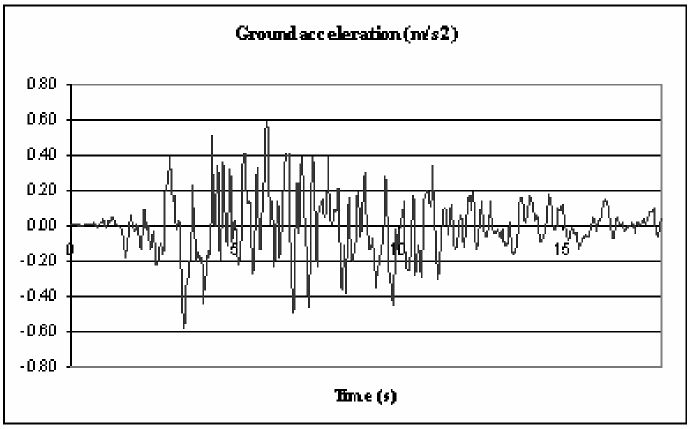

Figure 5 : Accelerogram number 1.

Five synthetic accelerograms are generated because it is the minimum number required, according to the Spanish standard, to perform a transient analysis like the one carried out here.

5 CALCULATION ASPECTS

The tower is considered with 3% of structural damping according to Paulay and Priestley (1992).

The Newmark integration scheme is applied for the integration in the time domain. The time step used tries to guarantee that the main response of the structure is collected in the calculations. A minimum time step of $\Delta t = 1/(20f)$ is used in this sense, where f is the fundamental frequency of the structure.

The non linear system of equations is solved by means of the incremental-iterative Newton-Raphson method, updating the matrix in every single iteration. The use of the secant matrix is alternated to improve the convergence, when possible.

5.1 Earthquake proposed by NCSE-02

When the earthquake proposed by the standard is applied (peak ground acceleration 0.11g m/s^2), the Tower of San Pedro Apóstol is not able to resist since excessive cracking leads to the collapse of the tower.

The results obtained in Table 1 show that the worst seismic motion is the earthquake that affects the structure according to the N-S direction because it leads to the earliest collapse.

Three directions have been separately tested in order to obtain the one most affecting the structure (E-W, N-S, NE-SW).

Table 1: Results summary

	E-W Direction	N-S Direction	NE-SW Direction
First crack (basement)	t = 4.66 s.	t = 5,42 s.	t = 4.66 s.
New crack pattern (shaft)	t = 6.80 s.	t = 6.78 s.	t = 5.88 s.
Bell tower cracks	YES	YES	NO
Collapse	t = 8.66 s.	t = 8.08 s.	t = 8.92 s.
Cause of the collapse	cracking at the base	cracking at the base	cracking at the base
Material crushing	NO	NO	NO

The cracking begins later for the N-S direction because the most slender wall is not so stressed as in the other cases. It can be seen, though, that the collapse is reached earlier, being the tower more resistant to a seismic motion in the E-W direction compared to the N-S direction.

Cracking is mainly located at the first part of the tower, where three crack patterns can be clearly distinguished: at the basement, at the central area and at the section change in the main body. North and South walls undergo this kind of crack pattern when they are orthogonal to the earthquake acting direction (Fig. 6a to Fig. 6d) leading to the failure of the structure at 8.08 s from the beginning of the seismic motion.

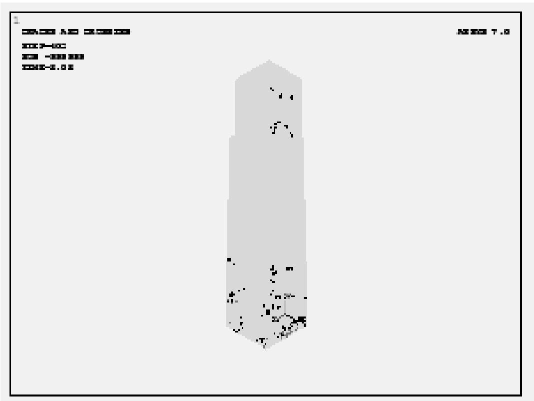

Figure 6(a) : Cracks at t=8.08 s. Isometric view.

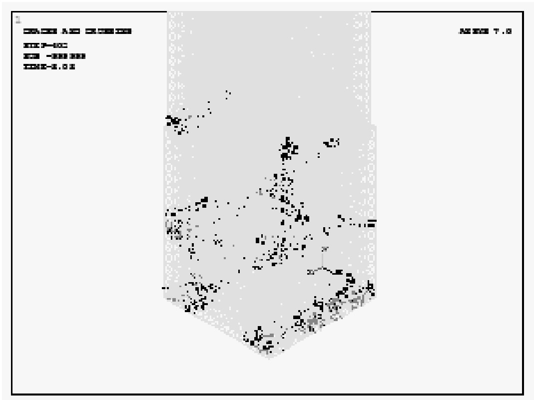

Figure 6(b) : Cracks at t=8.08 s. Detail of the lower part, isometric view.

At the bell tower area, some cracks appear around one of the voids in the West wall, the thinnest one. Due to the small thickness of this wall, these cracks are spread all around this area.

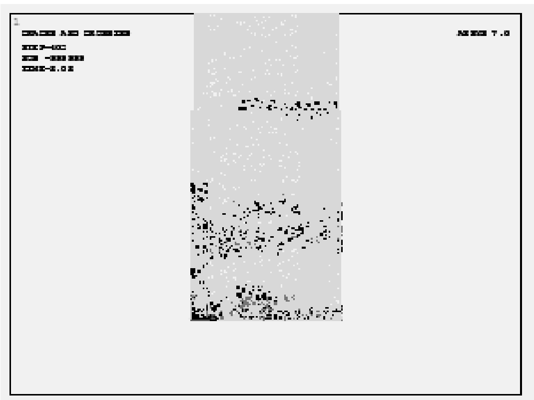

Figure 6(c) : Cracks at t=8.08 s. Detail of the basement, lateral view.

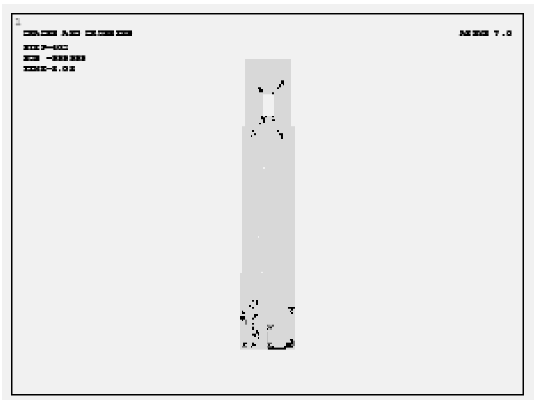

Figure 6(d) : Cracks at t=8.08 s. Rear view.

5.2 Collapse earthquake

The initial earthquake is scaled modifying the peak ground acceleration using an increment of -0.01g and the dynamic analysis of the structure is re-done again in an iterative process. That is how the maximum earthquake that the Tower of San Pedro Apóstol can withstand is obtained.
As a conclusion, the largest earthquake the tower can resist has a peak ground acceleration of 0.09g. For this earthquake, the crack pattern at the base of the tower is shown in Fig. 7, but there are not cracks along the shaft. There are also cracks in the thinnest wall of the bell tower, although they do not have much interest.

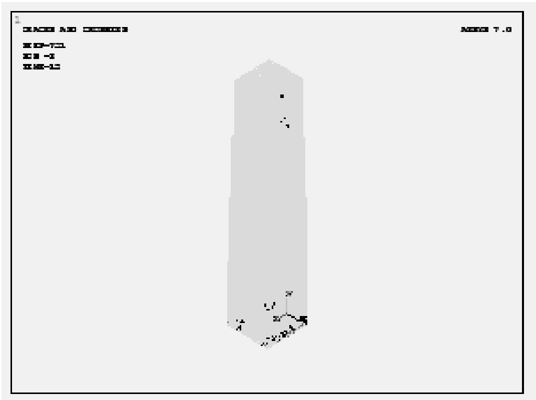

Figure 7(a) : Crack pattern after earthquake a_b=0.09g, N-S direction. Isometric view.

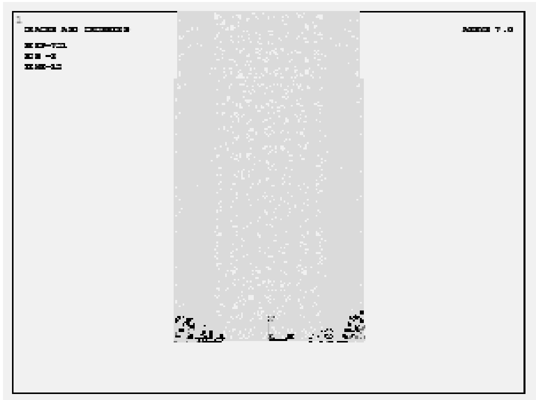

Figure 7(b) : Crack pattern after earthquake a_b=0.09g, N-S direction. Base. Frontal view.

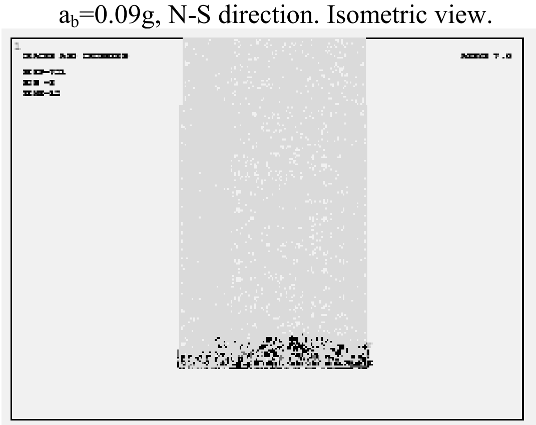

Figure 7(c) : Crack pattern after earthquake a_b=0.09g, N-S direction. Lateral view.

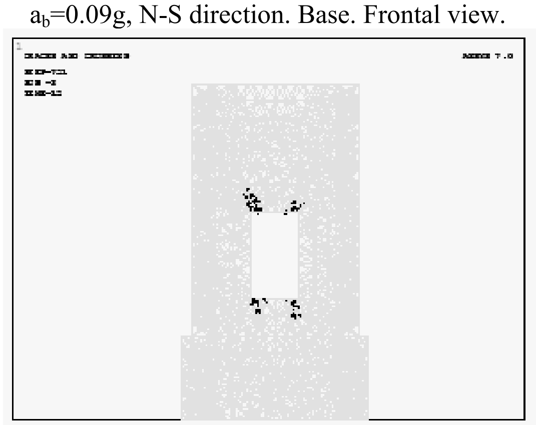

Figure 7(d) : Crack pattern after earthquake a_b=0.09g, N-S direction. Bell tower.

6 CONCLUSIONS

Results show that the tower is not able to withstand the seismic requirements of the current Spanish standard. For any of the tested earthquake directions, failure of the structure occurred, being the worst seismic loading that in the N-S direction.

It is worth noting that this church is located in one of the most seismic active areas in the Spanish Mediterranean coast with high seismic activity (NCSE 2002). If this construction were located in any other part of the Comunidad Valenciana, it would be considered safe by the current standard. As an example: the peak ground acceleration given by the standard in the city of Valencia is 0.06g, half the value that the Tower of San Pedro Apóstol must resist in the town of Agost. It can be concluded that the tower has a high to moderate seismic risk with the limitations surrounding the analysis (material data, geometry, seismic characteristics of the area).

Regarding tensile stresses, they reach their maximum values at the base of the structure, but also in the shaft. Mainly, they reach the maximum value in the bottom part of the tower.

Regarding displacements, as it is logical, amplifications at the crown level of the tower are produced relative to those introduced at the base, due to the flexibility of the structure. Referring to the time history analysis of the displacements at basement and crown, the required amplification factors could be determined if desired. As an example, displacements obtained for the first earthquake (number 1) for two nodes located at the basement and crown respectively are shown in Fig. 8.

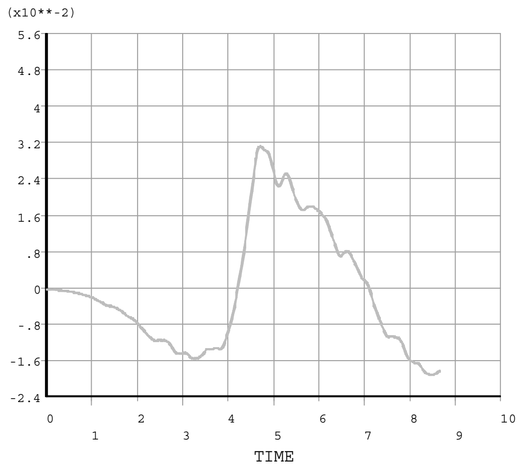

Figure 8(a) : Time history results for horizontal displacements in the earthquake direction. Node 2545 (basement of the tower).

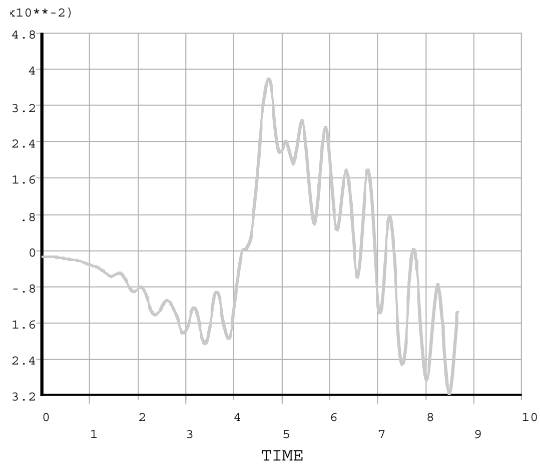

Figure 8(b) : Time history results for horizontal displacements in the earthquake direction. Node 1929 (crown). Amplifications can be observed with respect to the node displacement at the basement.

None of the calculations carried out produced crushing in masonry. This is due to the fact that the material has a low tensile strength so, before crushing by excessive compressions, cracks appear and collapse is produced due to excessive cracking.

It can be concluded that compressive strength is not a decisive parameter with great influence in the results obtained, but tensile strength is.

First cracks appear in the base of the structure spreading afterwards to the rest of the lower part of the tower body. Failure occurs in the central core of the base due to excessive cracking.

It has been determined that the greatest earthquake that the structure can resist in terms of peak ground acceleration is a_b=0.09g, inducing cracks in the basement of the walls perpendicular to the direction of the earthquake.

In case the structural damping of the tower is neglected, early collapse of the structure would occur thus, it is recommended to take into account this factor to obtain satisfactory results.

The Willam-Warncke criterion allows obtaining detailed information about the seismic behaviour of the tower, first crack appearance, crack patterns, time history analysis plotting the evolution of the crack pattern, weak parts of the tower to reinforce, etc. In this model, as stated before, tensile strength adopted for the material is the most decisive factor in the results obtained.

ACKNOWLEDGEMENTS

Authors want to thank the Generalitat Valenciana for funding this research through the granted projects GV06/135 and GVA 05/085.

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