CONSTRUCTION AND STRUCTURAL ANALYSIS OF THE DOME OF THE CATHEDRAL OF VALENCIA

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ABSTRACT

The Iglesia Catedral Basílica Metropolitana de Santa María (Saint Mary Holy Metropolitan Cathedral Basilica) is one of the most representative buildings among the architectural heritage of Valencia. The museum houses an octagonal dome, called The Cimborio, is considered to be one of the most interesting works on the cathedral complex, which is even more valuable considering the few examples of such a dome built throughout history.

The dome was built by the masters Nicolás de Autún and Martí LLobet and was completed around 1430, it has suffered many repairs and restorations of the problems caused by its risky slenderness, which have led the dome into a new equilibrium states that appear to have controlled the constant damage suffered by this skylight tower, but this cannot justify its current safety level.

This study focuses on the use of new technical tools that have not been employed until now, to allow the performance of a strict scientific research in order to provide conclusions that clarify the doubts regarding to the structural behaviour of the dome. The graphic surveying is performed using the latest tools such as 3D Laser Scanner. This is a non-destructive technique that enables to exact plans to obtain reflecting the current deformation of the dome. These plans are then compared with plans made using traditional data collection techniques (plumb-lines, measuring tape, etc.). Based on this data, a structural study is performed enabling the general state of equilibrium to be assessed by means of limit analysis. Numerical methods have been used for nonlinear damage models and for mechanical fracture.

INTRODUCTION

The Cimborio, is impressive both for its beauty and the absence of supports, it rises over the transept of the Cathedral of Valencia, with a floor plan in Latin cross (*figure 1*), the nave is divided into three perfect squares, with a series of rectangular side-chapels on both sides. The ceiling consists of simple cross vaults with four stone ribs and several solid brick lay endwise. The polygonal apse is surrounded by the ambulatory that leads off to the chapels [1].

The almost identical nave and transepts height do not wholly conform to the French Gothic proportion tenets, but it is an evidence of the difficulty on load transfer from the nave to the transepts (f*igure 2*).

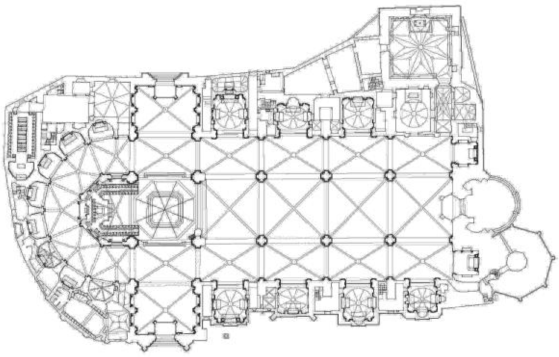

Figure 1: Plan of the Metropolitan Cathedral Basilica of Valencia

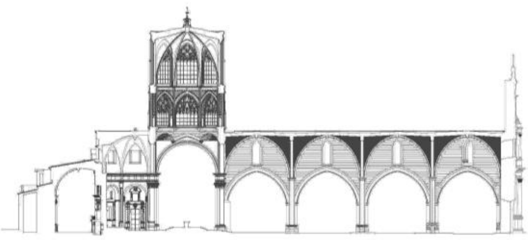

Figure 2:Longitudinal Section of the Metropolitan Cathedral Basilica of Valencia

The octagonal dome rising over the centre is supported by four pillars, (*figure 3*) 19.20m high and 6.18m wide, the dome is composed of two very similar tiers, the upper ones are taller than the ones below. This ties difference of size suggests that they were built in two different stages. There are large windows in the walls with elaborate tracery work in between, which gives the Cimborio its light and delicate appearance.

The Cimborio roof is a vault supported on ogival arch segments that spring from the first tier at the cornice level with the shape as defined by the octagon that converges on the central keystone. The bricks in the vaults are laid endwise in ascending horizontal courses. These severies are similarly arranged as those in the other vaults of the cathedral, in which concrete is used as filling up to the point where the roof starts to slope. The accessible part of the roof is divided into eight sections that starts from the centre of each side of the octagon and join up with another eight that start at the apexes. There are two gargoyles at each side of the apexes to carry rainwater away.

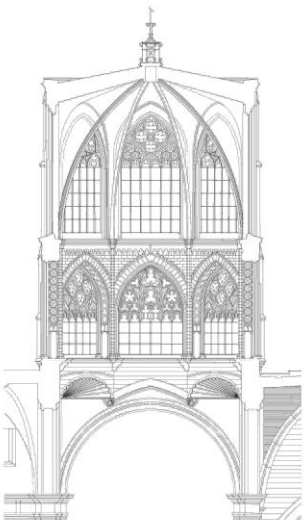

Figure 3: Section of the Cimborio

If these characteristics of beauty and lightness, together with the absence of supports have made the Cimborio a work of "great mastery", they have also made it necessary to carry out frequent repairs. The oldest repair on record was carried out after the earthquake in 1396 and was mostly done on the arches and the glass windows. Further work was carried out on the panes of the second tier in 1432 due to wind damage, and again in 1698, when they were replaced by alabaster panels. In 1731 general repairs were done to the arches and vaults, as a result of that new tracery and mullions were given. In 1863, under the direction of Joaquín Mª Calvo, new alabaster panels were put in place with iron frames. In 1919 Francisco Mora installed a reinforcing iron band over the windows in the upper tier. In 1978 Ramiro Moya replaced the iron frames of the alabaster panels with new ones made from stainless steel, the roof was strengthened with a reinforced concrete slab, a metal band was placed around the oculus and a stainless steel brace was added to the arches outside of the second tier (*figure 4).*

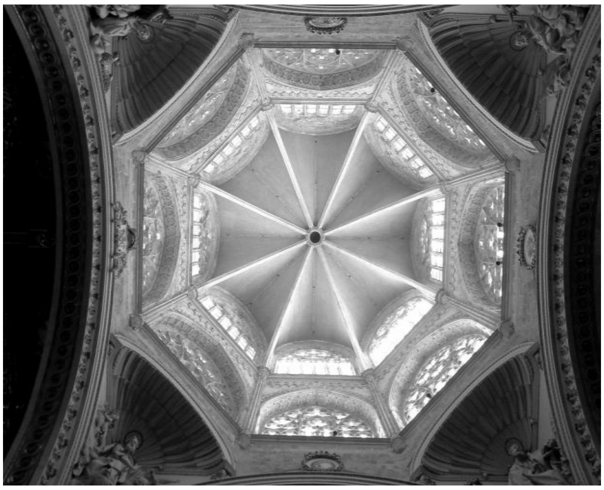

Figure 4:Image inside of the Cimborio

A second intervention took place in 1777 under the architect Antonio Gilabert. In this case the work was focused on the pillar between the Chapel of the Epistle and the nave, in which cracks had appeared. Although initially only the damaged section was going to be repaired, in the end it was decided to renovate it from top to bottom. The third most important operation took place in 1978 and was focused on various aspects again related to the tracery, mullions, the alabaster panels (f*igure 5)*, and the pillars foundations of the central nave, which had been observed to have sunk slightly. These were underpinned and braced with reinforced concrete beams. The work on the dome consisted of fitting three strengthening elements at three different levels: the first one on the first balcony at the squinches height, by means of an iron beam fixed to the wall with bolts, which involved cutting off the access to the balcony. The second one consisted of fitting girders to the second-tier balcony and the third was a strengthening of reinforced concrete added to the crown of the dome.

Figure 5:Image outside of the Cimborio

NUMERICAL ANALYSIS

Linear analysis

In order to make the best possible survey of the present state of the dome, a study was carried out by non-invasive laser scanner. The method involved in this technique is based on a capture of the points, their referencing, extraction and analysis, besides obtaining 2 and 3D drawings, which provide us precise information about the real situation of the structure. A high-density points cloud is generated that enables us to identify the current settling and deformations present in the dome (*Figure 5)*.

Based on the graphic geometric survey obtained from the laser scanner, a 3D model was generated by CAD applications that allowed us to create a calculation model based on a solid finite elements mesh to reproduce the 3D geometry of the Cimborio, with a total of 249252 d.o.f, 47615 solids, distinguishing between 2191 tetrahedral and 45424 hexahedral elements (f*igure 6, 7 & 8).*

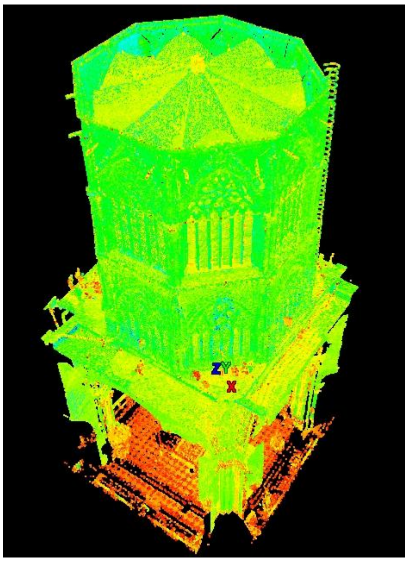

Figure 6:Image obtained with Laser scanner technique

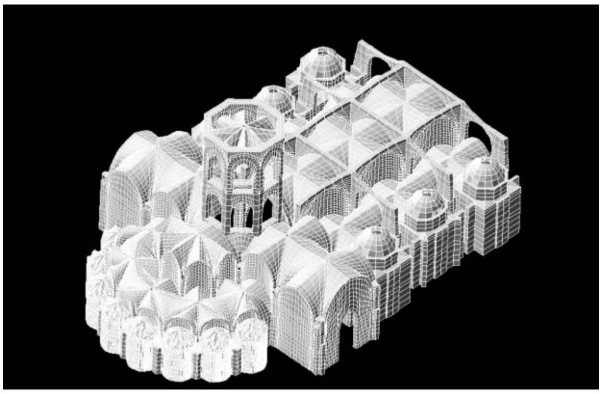

Figure 7:3D Cathedral and the solid finite elements mesh models.

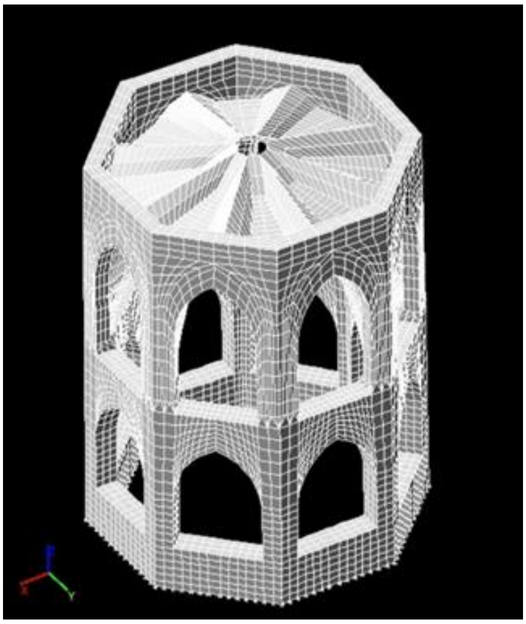

Figure 8:3D model of the Cimborio and the solid finite elements mesh model

A linear static analysis was first carried out to evaluate the masonry stresses, leaving the existing cracks out of consideration, and concentrating only on the self-weight hypothesis. We assumed the material, which was mostly masonry, to be homogeneous. As it was not possible to carry out tests on the dome itself, we had to resort to the data obtained from previous tests carried out on masonry from the Trinity Bridge in Valencia, which had similar characteristics and had been obtained from the same quarry. The test results are given in Table 1.

Test cylinders	Density T/m3	C. Breakage N/mm2	$\Delta\sigma$ N/mm2	$\Delta\epsilon$	Module E N/mm2
1	1.949	7.05	2.673	0.0017	1527.88
2	1.914	7.13	3.068	0.0020	1534.00
3	1.891	9.57	5.485	0.0042	1213.67
Average	1.918	7.916			1425.18

Table 1: Tests Results

Table 2 gives the most important mechanical properties used in the numerical model:

Ashlar masonry	
Density	1,918 T/m3
Module of deformation	1,425 N/mm2
Poisson's ratio	0.2
Compressive force	7.90 N/mm2
Tensile strength	0.15 N/mm2

Table 2: Mechanical properties

Non-linear analysis, damage model

In order to get the non-linear response of these structures, the fissures appearance and their development have to be calculated as well as the maximum structural load [2] [3]. The so-called isotropic damage model was used in this study [4] within the ANGLE finite elements software [5].

Damage mechanics is a branch of Continuum Mechanics, which, by means of internal variables introduces micro-structural changes into material behaviour.

The fissures emergence and their evolution over time in materials such as concrete and masonry can be described as the course followed by several points of damage.

If a damage function is defined representing correctly the response of the material, both in compression and traction, then the non-linear masonry behaviour can be represented in a model.

Cracking is represented in this case as an effect of local damage that can be defined according to the material known parameters and to functions that control the damage evolution with the successive tension loads at each point.

The model used takes into account the three assumptions necessary to correctly model the non-linear behaviour of masonry; the different behaviour in compression and traction; the rigidity deterioration through mechanical causes (tension loads) and the effect on the response of the finite elements mesh size.

Concept of isotropic damage

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 thus:

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

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 tension and the actual tension acting on the part of the effective resistant section is derived from the condition of equilibrium:

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

This scalar index is sufficient 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 9).*

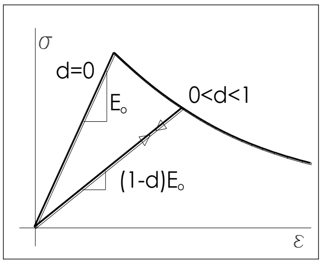

Figure 9: The effect on the mechanical behaviour of the material is a reduction of rigidity proportional to (1-d)

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

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

Where: *D* is the elastic constitutive matrix.

The scalar variable of damage is:

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

Where: The *r, r^o, and A,* values are obtained as in reference [3].

An important advantage of the damage formulation is its simplicity of calculation compared to other cracking models, since a special algorithm is not required to integrate the constitutive equations of the elasto-plastic models.

Model calibration

Described model has been implemented in ANGLE software by the authors. Attempting to calibrate this model with ashlar tests it results extremely difficult due to calibration is made with experimental concrete models, those are concrete elements with a similar behaviour to masonry, particularly Walraven [6] tests on a reinforced concrete beam have been employed, and the tests made at University Polytechnic of Valencia by Valcuende and other authors [7] about concrete non-reinforced beams. (Figure 10&11)

The results of the developed numerical model have a precise adjustment to the experimental results:

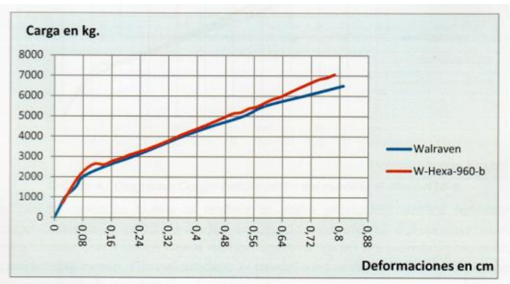

Figure 10: Load-Deflection, comparative between the experimental and numerical model

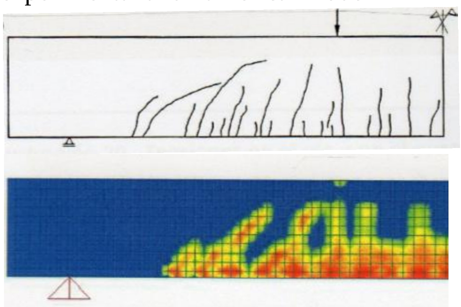

Figure 11: Strength distribution and cracking map

ANALYSIS OF THE RESULTS

It has been developed a non-linear static analysis of the Cimborio model. The most important parameter of the stability of the structure is the masonry tensile strength. As we do not have very reliable tests, it is considered the average strength values, which have been obtained from different authors. The model has been processed under two conditions. Firstly, we have used a 0,05 N/mm2 tensile strength, which is a much lower value compared to the reference one. This is a hardly null value compared to 0,1N/mm2 used in a second analysis (*figure 12).*

In order to study the results, the damage formulation is considered as the most important parameter in the structural behaviour. The damaged parts of the structure are compared under the two hypothesis described and it is found out that the safety margin of the structure is very low in the hypothesis with 0,05N/mm2 tensile strength. For this reason the structure degrades and starts to break down at a load of 1,05 its own weight. Whereas in the hypothesis with 0,1 Mpa, the safety margin of the structure is bigger, getting a safety coefficient of 1,60 its own weight *(figure 13).*

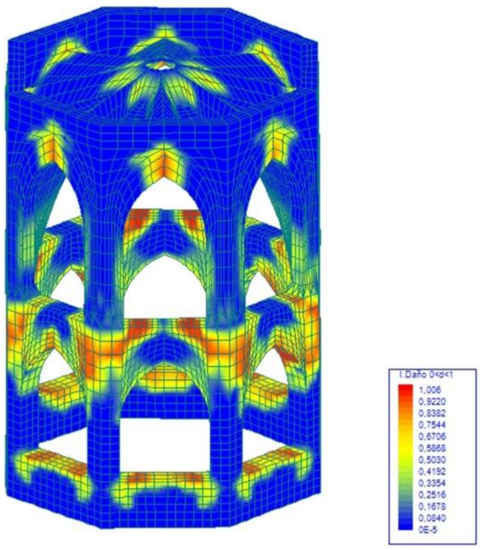

Figure 12: Model of damage dead load

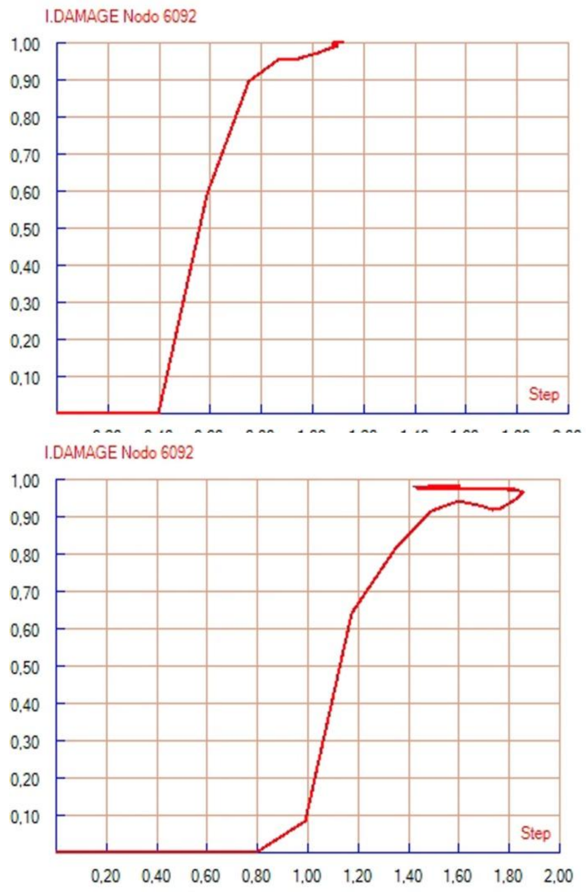

Figure 13: Two hypothesis of analysis with 0,05N/mm2 and 0,1N/mm2 tensile strength respectively

CONCLUSIONS

In this article, it has been described the process of data collection using the Scanner Laser 3D that allows us to draw the architectural and structural model with a very high precision. The Scanner Laser gives us three dimensional information using millions of points in the space that require a much elaborated work to develop the models. It is in a research phase the development of new methods to automate the transfer process from the points given by the scanner laser to solid models.

The structural analysis of the dome aims to determine the safety state of its stability. This historical aspect has been discussed a lot among society in Valencia.

The analysis shows that the safety state that we get under very restrictive hypothesis based on very low tensile strength of its masonry has a coefficient of 1,5G. G is its own weight. This result may seem lower than the one found in other historical buildings but high enough to explain its permanency for nearly 600 years.

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