# Reinforcement work carried out on the Todolella Parish Church after the collapse of a pilaster supporting the classical style dome; Castellon, Spain.

A. Martinez Boquera

Dr.Architect, PhD Professor of the Polytechnic University of Valencia, Spain

A. Alonso Durá Dr.Architect, PhD Professor of the Polytechnic University of Valencia, Spain

# R. Soler Verdu

Dr.Architect, PhD Professor of the Polytechnic University of Valencia, Spain

A. Soler Estrela *Dr.Architect* 

ABSTRACT: This paper is particularly interesting since it provides unpublished data on the reinforcement carried out in 2006. Cross-sectioned plan and elevation drawings give a detailed account of the geometry, illustrating cracks and strains with precision. The building specifications of walls and masonry shells with poor mortar, the depth and bond of brick arches and the walled vaults and domes are also depicted. Non-linear structural damage models of the fractured structure were made to reproduce the settling of the foundation and its effect on the fracture of structural elements such as the dome, drum and the outer buttress. The paper also describes the stability analysis performed on the damaged construction elements, drum and dome. The criteria and intervention techniques applied and described are supported by the results of these analyses.

# 1 INTRODUCTION

The village of Todolella in the El Maestrazgo highlands of Castellón, Spain, has a rich architectural and environmental heritage. The parish church of St. Bartholomew is in the Gothic style with a star-shaped vault within the side-chapel and diaphragm arches in the central nave. Later additions increased its complexity, particularly a dome over the circular apse.

Its latest extension (18th century) consisted of a classical style chapel with a Greek-cross plan on the western side of the nave. The drum arises from four main arches with a hemispherical dome covered with tiles.

The chapel was in a particularly ruinous state. Due to the collapse of a pilaster, the two main arches resting on it were cracked through, since the pilaster was located close to the keystone. The other two arches only showed fissures, since their supports had not been affected. Two outer walls were also cracked through in parallel with the arches. The main cylindrical drum under the main dome was fractured in the area between the crack in the pilaster and the fracture of the keystone.



Figure 1. Greek-cross plan of the chapel D.



Figure 2. Cracks on the two main arches.



Figure 3. Main dome fractured.

#### 2 STRUCTURAL DAMAGE

The condition of the structure needed an urgent inspection to evaluate its safety. The most serious problem of the construction was its structural-mechanical nature, which meant that the stability, structural pressures and the reactions on the foundations had to be determined. The first step was to study the structure for damage and to discover its composition and geometry.

The break at Point A of arch, between Sections 1-2 is clean and has a width, that varies according to height, of between 23mm and 50 mm. In addition, it can be seen that the interior surface of the arch is smaller on the right than the left by 75 mm (central observation point). There is also a radial displacement of 5 mm.

At Point B of arch, there was an 8 mm wide crack, but with no reduction and both sides of the crack are on the same level. The depth is difficult to specify. At Point C of arch there is a similar crack to the one in arch T in the key, but its width is 2mm and no reduction can be seen.

At Point D of arch, here again the break was clean with width of around 200mm, varying according to height. The interior surface of the arch on the left hand side is smaller than the right by 75 mm (central observation point). Radial displacement is 20mm.

At Points A and D the arches are cracked in the proximity of the key in a very similar way. The drum is completely cracked between the keys of the R and S arches, following the directrix of a load-bearing arch in response to the base of arches R and S.

In the outer walls between abutments, there is a break approximately at the center of the wall throughout its width, being wider at the bottom than the top. The third side does not show any cracks. The fourth side does not exist, since it connects the nave with the presbytery.



Figure 4. arch A cracked in the key.

#### 3 DESCRIPTION OF THE BUILDING CONSTRUCTION

The walls are of great thickness, following the Greek cross plan. They are made of limestone and joined together with earthen mortar containing almost no lime. In the corners they are stiffened by large and well fitted stone blocks. The inside walls are covered with plaster and paint, while the outer walls do not have any coating.

On the top of the walls there is a cornice of alternate layers of tiles and brick.

The arches are well-proportioned and made of brick masonry with lime mortar. The pendentives are made of masonry in the form of successive stringcourses.with a double-walled arch, separated by an air chamber, on the outer surface.

The circular drum, supported by the pendentives, is also made of masonry. Its outer covering prevents the determination of the dimensions, ties and mortar used in its construction. The zones open to view are of similar characteristics to the walls.

The dome itself was constructed with a double layer of bricks held together with plaster, the outside having a covering of curved ceramic tiles over the simple stringcourse cornice of brick and tiles.

To determine the mechanical behaviour of the structure, the characteristics of the materials that had

to be defined were: compression strength, traction strength and the deformation and density modulii.

The analysis of the building structure was carried out to facilitate the creation of theoretical calculation models to compare with the models of the structure. The form of the construction, in which shape plays a decisive role, provides structures of extraordinary, and sometimes surprising, rigidity.

#### 4 THE STRUCTURAL STUDY

From the data collected, it is deduced that pilaster, on which arches A and D rest, had descended approximately 75 mm and caused the break in the arches. No settling was detected in the other three pilasters. Analyzing the geometry of the deformation, the agreed interpretation is that the abutment pilaster AD, decreases to precisely the same value as the difference of level on both sides of the crack of the arch. The cause of the fracture, therefore, was attributed to the settling of the foundations due to ground movements.

The structural-mechanical study is based on a three-dimensional model of the building by the Finite Element Method. The mesh is composed of solid, hexahedral elements for the walls and the drum and tetrahedral for the arches. The brick dome is modeled with quadrilateral surface elements.

The model is made up of 15088 nodes, 480 surface elements and 22379 solid elements with 45545 degrees of freedom.



Figure 5. Complete mesh of the model.



Figure 6. Meshes of the arches and the drum.

Three types of analysis were performed.

1. Linear static analysis to evaluate the reactions in the foundations and tension in the undamaged masonry and without considering the settling of the foundations.

2. Nonlinear static analysis, applying a damage model to characterize the behavior of the masonry against cracking and material breakage. The process followed in this analysis consisted of applying the load of the structure's own weight in stages until reaching the total load. Next, the load due to the settling of the foundations was applied. With this analysis it was possible to determine approximately the state of cracking and fractures in the masonry. By comparing the model with the real structure, it is possible to value the validity of the ground movement hypothesis assumed and the final state of tension of the masonry in extreme conditions.

3. Nonlinear static analysis with damage model, applied to the resulting structure of cracking and breaks in the masonry to evaluate the load-bearing capacity and degree of safety of the structure under extreme conditions. This was done by including breaks in the meshes at the points where the fractures appeared.

#### Linear static analysis for weight loads.

The elastic-linear analysis was carried out to obtain the pressures on the foundations due to the weight of the structure.

The characteristics of the materials of the different elements used are the following:

Stone rubblework for walls and drum: Deformation Module  $E = 9000 \text{ N/mm}^2$ Poisson Coefficient v = 0,20Density  $\rho = 2100 \text{ kg /m}^3$ Brick masonry for the arcs: Deformation Module  $E = 6,000 \text{ N/mm}^2$ Poisson Coefficient v = 0,20Density  $\rho = 1,900 \text{ kg /m}^3$ Brick masonry of the dome:

The reactions on the nodes at the base of the model that form the foundations are distributed in the following way:



Figure 7. Load distribution on foundations.

The loads on the foundations are distributed as shown in the previous figure. In order to analyze the effect on the foundations, these were divided into seven parts, the loads on each of which were calculated.



Figure 8. Results of loads on the foundations

The loads were not vertical. In the following table, the values of the loads are presented, together with their components with respect to three axes, XYZ, and the pressure on the ground obtained by the distributed vertical component on the corresponding area.

Table 1.	Values	of the	loads	on the	foundations
----------	--------	--------	-------	--------	-------------

ZO	Push	E.vertical.	E.horizo	E.horizontal	Pressure
NE	kN	Z kN	ntal X kN	and kN	kN/m2
1	656,53	649,46	-44,68	85,12	278,97
2	655,92	648,83	-44,66	-85,10	278,70
3	776,71	776,38	-9,41	20,62	213,29
4	776,98	776,65	-9,34	-20,64	213,36
5	928,64	924,28	45,90	77,25	230,14
6	928,64	924,29	45,79	-77,19	230,15
7	704,89	704,70	16,38	0,00	193,60

From the linear analysis carried it can be concluded that the structure is very rigid and the deformation level is very low. Maximum deformation is less than 1 mm in the dome.



Figure 9. Vertical deformations for the weight of the structure.

#### Nonlinear static analysis for structural loads.

A nonlinear analysis of the structure was performed to approximate the numerical calculation to the real structure, considering cracking by traction of the masonry or crushing by compression.

The calculation carried out corresponds to an isotropic damage model in which the scalar damage index for each of the Gaussian points of each element is evaluated. The value of d varies between 0 for no damage and 1 for structural failure.

In order to carry out the analysis of nonlinear behavior regarding geometry and material with a damage model, the iterative *incremental method* process was performed divided into load steps.

For the type of structures used in this method, being frictional material, rubblework or brick masonries, and geometries corresponding to historical buildings, geometric non-linearity is small, since the level of deformation is low. Mechanical nonlinearity is somewhat higher, the rigidity variations corresponding to the state of a much smaller degree of cracking and crushing by compression. In general, a large number of iterations are not needed to obtain results of acceptable precision.

The incremental process, in this case, was divided into 12 steps. The first 5 steps correspond to the weight of the building itself. From step 6 to step 12, the fraction of settling of the foundations at the corners of the building is introduced as incremental load. The value of the maximum settling entered was 6 cm. There is no analysis available of the properties of the materials that compose the structure. For this reason, the parameters chosen are similar to cases taken from other publications that have been previously referenced.

The values of the parameters of the materials considered in the calculation are the following:

Stone rubblework for the walls and the drum:

Brick masonry of the arches:

Brick masonry of the dome:

The levels of deformation, tension and damage resulting from the nonlinear analysis for the total gravitational load are shown in the following figures.



Figure 10. Rank of nonlinear vertical tension for total gravitational load.



Figure 11. Damage indices in all the structure for total gravitational load.



Figure 12. Damage indices in arches and pendentives for total gravitational load.

The calculated deformations are slightly higher than those produced by the linear analysis, confirming the great rigidity of the masonry.

The low state of tension values are within the normal range for this type of construction.

The damage index graphs show very few zones with traction fissures. These are typical fissures in vaulted structures, such as the keys of the arches and where these are joined to the walls.

# Nonlinear static analysis for the settling of the foundations.

In order to introduce the load hypothesis, whose action is the settling of the foundations at the corners, several ground movement models were constructed and the results were compared with the fissures observed.

The actual settling of the foundations forms a set of complex movements impossible to measure and therefore impossible to calculate numerically. A simplification was introduced, which consisted of rendering the settling zones by planes which turn on a line that intersect the walls at their mid points. The following figure shows the foundation settling planes:



Figure 13. foundation settling planes.

The maximum settling entered was 6 cm. The numerical calculation produced a mechanism in the structure when the corner sinks 3 cm.

In the actual structure, the 7 cm. sinking was measured at the corner. This is because the settling of the foundations follows the failure of the masonry. The model measures up to the moment of failure, that is when the structural mechanism appears.

The model reproduces the cracked surfaces with a high degree of similarity to the real surfaces.

# Crack modeling. Nonlinear static analysis with discontinuity of mesh.

A model of the building was constructed that reproduced in the mesh the fissures of the structure, with breaks in the zone of arches and drum.



Figure 14. Damage to arches, drum and walls.



Figure 15. Cracking of the walls due to gravitational load and settling of foundations.

The numerical analysis of this sample show that the damaged structure is stable and new mechanisms do not take place. This is consistent with the real structure, which has remained stable since the intervention was carried out.

In the part of the drum over the horizontal fissure, a type of load-bearing mechanism is formed, as the horizontal tensions in the previous figure shows. This causes a sort of projection to appear in a curve that remains stable.



Figure 16. Mesh reproducing the fissures with breaks in the archs.



Figure 17. Model of damage applied to the mesh modified by the fissures.

0.057

-0,041 -0,085 -0,089 -0,114 -0,138 -0,162



Figure 18. Horizontal tensions in the drum.

We ruled out any rectification of the arch, as we preferred to leave visible the joints, the response of the broken structure and its search for balance. But we had to cure the wounds, restore the lost continuity, close the cracks, avoid the entrance of water while leaving testimony of the episode in the building's history.

Luckily, the correct strengthening of the foundations with a reinforced concrete wall eliminated the cause. The slope became stabilized and the foundations of the heavy rubblework walls now properly transmitted their loads onto the lower supporting layers.

With the reproduction of a broken arch on a 1:1 scale, load tests were performed and the cracks were opened and closed. The tests provided valuable information on the behavior of the adopted solution.

In the case of the arches with fissures, several coats of dilute lime washes were applied.

The vaults. The vaults were reconstructed due to hygrothermal requirement that affects people's comfort and the durability of the construction. A weak ventilation was created, which improved the elimination of damp and lengthening the life of the wooden elements prone to rotting.

Thanks to this action, the large windows have recovered their mission of renovating the air by incorporating grids.

The vaults were partially reconstructed, the fallen sections reinforced and the cracks were cleaned.

The drum and dome. The techniques described for the consolidation of the rubblework were used. Fissures and cracks were stopped with flexible mortar in such a way as to leave a testimony of the event.

# CONCLUSIONS

From the analysis carried out in the evaluation of the loads of the structure on the foundations, it is possible to conclude that:

The distribution of pressures on the ground is carried out homogenously and is centered on the base. The average value of tension in the ground is 233 kN/m2 [2.33 kp/cm2], a value that can be considered as low.

The settling of the foundations that took place was not due to pressure on the ground, but to slippage of the terrain. The underpinning carried out in the foundations will ensure their stability.

The nonlinear analysis of the damage model provided a hypothesis on the cause of the cracking in the masonry. There is a good agreement between the cracking provided by the numerical model and the actual cracking of the structure. The calculation of the model that includes the cracks in the arches and the drum allows us to conclude that the post-breakage structure is stable and this is corroborated by the behavior of the building since the intervention.

### REFERENCES

- ACI 530-99. American Concrete Institute. 1999. Building Code Requirements for Masonry Structure.
- Code UIC 778-3. Union International des Chemins de fer. 1995. Recomendations pour l'evaluation de la capacité portante des ponts-voûtes existants en maçonnerie et beton.
- Eurocódigo 6.UNE-ENV 1996-1-1.Marzo 1997. Proyecto de Estructuras de Fábrica.
- Hanganu, A.D.; Barbat, A.H.; Oñate, E. Metodología de evaluación del deterioro en estructuras de hormigón armado. Monografía CIMNE nº 39, Julio 1997. Pag. 183-203. Aplicación del Indice Global de Daño al estudio de Construcciones Históricas

Hendry, A. Macmillan Press Ldt. 1998. Structural Masonry .

- Heyman, Jacques. Cambridge University Press, 1998. Structural Análisis. A Historical Aproach
- León, J.; Martín-Caro, J.A.; Martínez, J.L. "Comportamiento mecánico de la obra de Fábrica". Monografía sobre el análisis estructural de construcciones históricas de fábrica. Departamento de Mecánica de los Medios Continuos y Teoría de Estructuras. E.T.S. Ingenieros de Caminos, Canales y Puertos. Madrid. 2001. Comportamiento mecánico de la obra de Fábrica.
- López, J.; Oller, S.; Oñate, E.; Monografía CIMNE nº 46. Diciembre de 1998. Cálculo del Comportamiento de la Mampostería mediante Elementos Finitos.
- Lourenço, P.B. Tesis Doctoral. Universidad Tecnológica de Delf. Delf University Press. 1996. Computational Strategies for Masonry Structures.
- Luccioni, B.; Martín, P.; Danesi, R. Mecánica Computacional, Vol.17, 373-382. Tucumán. Argentina. 1996. Modelo elastoplástico general para materiales ortótropos.
- Martinez Boquera, A.; Alonso Durá, A.; Llopis Pulido, V. Structural analysis for the diagnosis of crackings in the Gothic masonry structure of the vaults of Trinidad Bridge in Valencia, Spain. 1639-1644. Proceedings of the 6th international conference on fracture mechanics of concrete and concrete structures.
- P.I.E.T. 70. Consejo Superior de Investigaciones Científicas. Madrid. 1971. Obras de fábrica. Prescripciones del Instituto Eduardo Torroja.
- Tosca, Thomas Vicente "Tratados de arquitectura civil, montea y canteria, y reloxes". Valencia: Oficina de los Hermanos Orga, 1794. . "Compendio matemático en que se contienen todas las materias principales de las ciencias que tratan de la cantidad..." Valencia: Antonio Bordazar, 1707-1715. 9 vols.