Structural Analysis of Historical Constructions, New Delhi 2006
P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.)

Pathologies Caused by Man on the Foundations of Historic Buildings

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ABSTRACT: The church of Losa del Obispo presents a series of cracks and displacement, both brought about by the same cause. During the nineteen sixties, two buildings that stood beside it were demolished and up to 1.5 metres of soil was removed from the side of the church leaving the foundations of the bell tower and the right wall of the church laterally exposed. This action triggered the following pathologies: leaning of the lateral facade of the church into the square and subsidence of the lateral wall of the church. After analysing the situation and evaluating the different options for its repair (pre-tensing the wall against the ground, adding piers, inserting abutments…), the restoration project drawn up for the church involves the lateral underpinning of the foundations in the form of a bench for public use and the insertion of trusses in the vaults to prevent further movements of the wall.

1 GENERAL INSTRUCTIONS

The church of San Sebastián Mártir in the town of Losa del Obispo (Valencia, Spain) was built in 1720 on the site of a little church (Sanchis 1922) that had become too small to hold the growing number of inhabitants in the town. This church was built to supplement the parish church in the nearby town of Chulilla, because Losa del Obispo did not acquire the status of a separate municipality until 1795 (Torres Faus 1995).

It is a church with a single nave and chapels between the piers with a barrel vault divided by ribbed arches with lateral lunettes, comprising five sections plus the chancel (Fig. 1). The slim bell tower, which stands at the foot of the church, comprises two major parts, namely the base and the belfry proper, crowned with a little dome. From an urbanistic viewpoint, one side of the church is attached to the existing building and it is practically free-standing on the other three sides, namely the main facade giving on to the street, the lateral facade facing the square and the back facade looking on to a garden.

In 1913 the presbytery was expanded, endowing the church with an extremely longitudinal character. Traces of these works can be seen in the change of fabric of the last three piers of the church, the last two directly caused by the expansion works and the other for causes unknown.

Figure 1 : General view of the church.

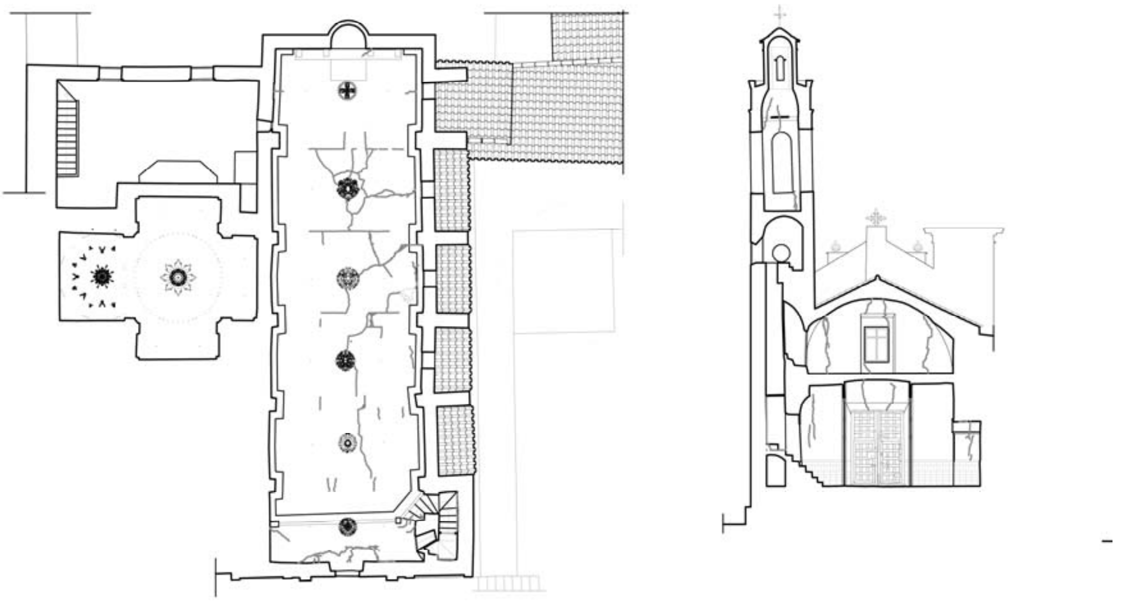

Figure 2 : Map of cracks in the church.

During the first half of the 20th century, the interior of the church underwent several transformations. A choir was built and later modified, the church was connected to the sacristy and the abbey in several places, the pulpit attached to a pier was demolished and the stairs leading up to it sealed, and an arch in poor state of repair was refurbished (Q.Libri 1939-1952).

In any case, after a detailed analysis, none of these changes of function or maintenance works seemed to be the cause of or even to have any connection with the structural pathologies the church presents today. As we shall explain below, these problems seem to have more to do with external actions by man on the church, completely destabilising its original equilibrium and giving rise to many structural pathologies.

2 DESCRIPTION OF THE MAJOR STRUCTURAL PATHOLOGIES

In the first place, a preliminary study was carried out with a thorough survey of the church, and a detailed map of the existing cracks was drawn up (Fig. 2). The combined systematic study of the direction, trajectory, opening typology, rotation and relative displacement of the cracks made it possible to elaborate a rigorous diagnosis of the phenomena caused and their extent. These damages can be summed up in the two points below:

- Leaning of the lateral facade of the church into the square, with the subsequent longitudinal cracking of the interior vault. The tilt of the lateral facade is evident and has been marked in the plans (Fig. 3). This overhang also exists in the church's bell tower.

- Subsidence of the lateral wall of the church, which is reflected in the cracking in the paving and the cracks at the corners of the windows at lunette level, typical of this sort of movement.

3 OTHER LESS SERIOUS PATHOLOGIES

Other minor pathologies have been observed, such as the subsidence of the arches in the south-east side of the nave, detected in the flat line of the arches, and cracks in the upper fenestration of the lunettes. Furthermore, the back facade of the church is slightly leaning outwards as well. There are minor cracks in other spots, due to isolated subsidence of arches and leaning of the exterior facade, presumably stable, and, in any case, the lack of activity will be verified by inserting movement monitors. Besides, the church presents problems of damp from leakage around the soffit and intrados of the vaults, resulting from the cracks produced by the movements of the church and lack of maintenance on the roof. The problems of damp are also caused by capillarity in the socles in the side chapels, the chancel and the main facade.

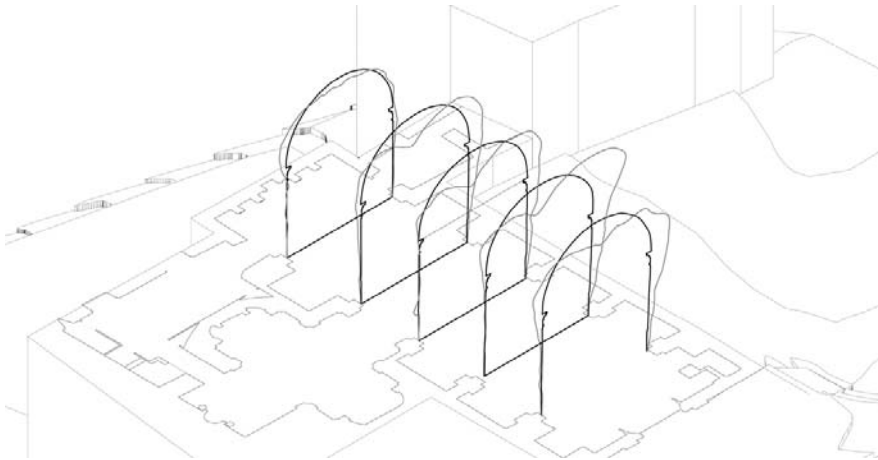

Figure 3 : Deformation of the arches.

4 HYPOTHESES AND PRELIMINARY WORKS

All the important phenomena of cracks and displacement of the church seem to stem from the same cause, which was our initial working hypothesis. After dreadful floods in the town in 1957, two damaged buildings that stood in the square, separated from the northwest facade of the church by a narrow lane, were demolished. At the same time, the square was levelled, and up to 1.5 metres of soil was removed from the old lane, leaving the foundations of the side wall of the church exposed (Fig. 4). This undermining of the lateral wall of the church no doubt gave rise to the most serious pathologies the church suffers today.

In order to verify this hypothesis, probes were performed on the lateral wall of the church, particularly on the thicker part of the socle, to find out whether there were foundations and, if so, how deep they went. In fact the three probes carried out revealed in the first place that there were no thicker foundations under the wall, and, more important still, the foundations of the lateral facade of the church and the bell tower had been completely undermined when the paving of the square was levelled. The soil under the foundation wall was covered laterally by a socle 1 metre high made of rubble and rough masonry to avoid the direct view of the danger. In the light of these data, it is surprising that the structural pathologies of the church are not a great deal worse than they actually are.

Furthermore, a geotechnical survey was carried out to dismiss the additional influence of possible irregularities in the ground on the behaviour of the lateral wall. This geotechnical study yielded the result of a section of ground with an initial 30 cm of concrete from the paving of the square, 7.20 m of clays with a very hard consistency, sandstone up to 9.5 m deep and again clay until 11 m deep (SEG 2001). All the materials extracted demonstrated great compactness in the Standard Penetration Tests, where rejection was found systematically. Besides, this high degree of compactness prevented taking unaltered samples. The conclusion of the geotechnical study dismissed the influence of the underlying ground on the pathologies found in the church. On the contrary, it demonstrated that the soil was very suitable as a basis for the foundations because of its hardness and compactness.

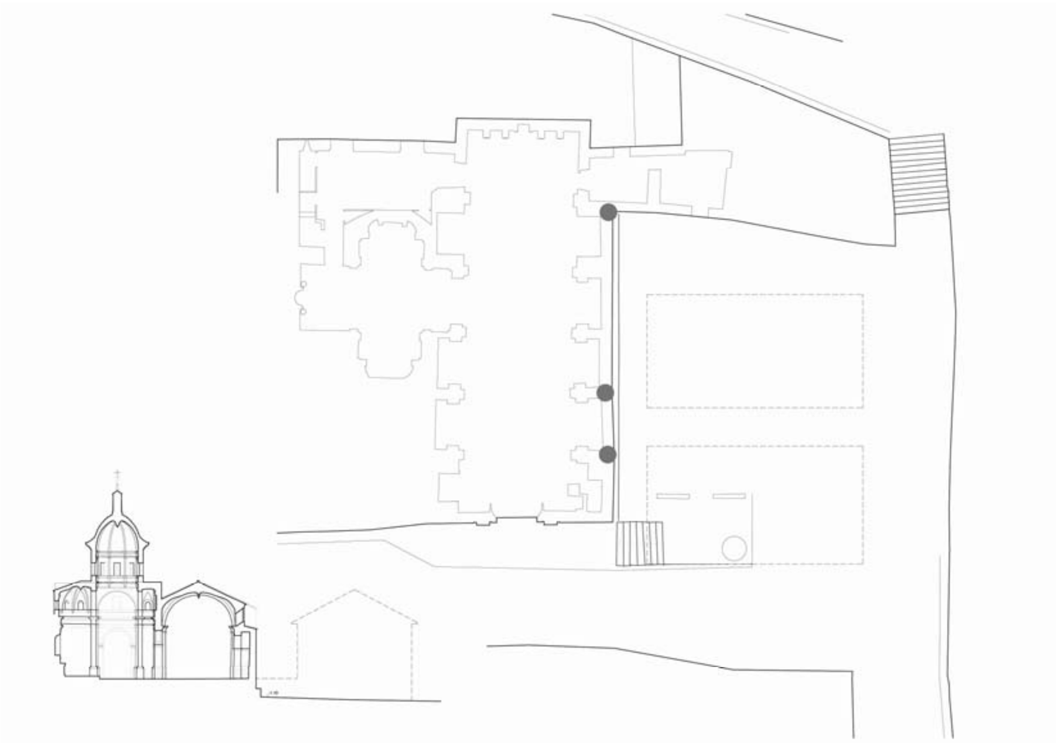

Figure 4 : Ground plan of the church and square, site of the missing buildings and position of probes made.

Other probes were performed on the roof to determine the real weight and thrust of the vault on the walls and lateral piers of the church. These probes yielded the result of a building section made up of a vault covered in bricks of different thickness (presumably two or three, but this needs to be verified), on which little transversal partition walls rest as a support for the ceramic panel that forms the hip of the roof.

Movement monitoring will be inserted to measure the evolution of the cracks in the church during the final stages of the study, the restoration works and some time afterwards. A dozen monitors will be placed in the major cracks and a follow-up measurement process will be carried out for a whole year after the structural restoration of the church is completed so as to make sure they are still inactive.

5 MODELLING AND ANALYSIS OF THE SITUATION

In view of the structural pathologies described above, it was considered appropriate to perform an analysis of its stability during the diagnosis stage as complementary research to the restoration process to provide reliable models about its structural behaviour and exact criteria for the restoration project of the monument (Fig. 5).

On the basis of a detailed map of the geometry of the vault, arches, walls and piers, their building characteristics and an estimation of the gravitational loads that act on them, the analysis methodology put into practice to evaluate the structural behaviour, which serves as a criterion on which to base the restoration, is the following:

a. An initial analysis of the tensional state of the church based on the models of the buildings by superficial and solid finite elements, under the hypothesis of gravitational loads.
b. An analysis by means of a non linear static model of damages for the hypothesis of gravitational loads with a rank of compression resistance in the brick fabric of 4 N/mm^2 and of traction resistance of 0.2 N/mm^2 (Lourenço 1998), and for the masonry 6 N/mm^2 compression resistance and 0.2 N/mm^2 traction resistance respectively.

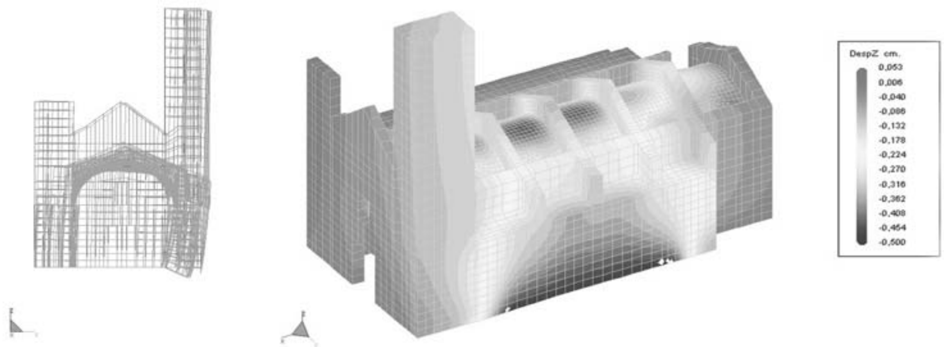

Figure 5 : Deformation model.

c. An analysis was carried out to determine the possible repercussion on the monument of the demolition of the buildings that had been attached to it and the rehabilitation and levelling of the adjacent square. For this purpose, on the basis of the structural models and the state of gravitational loads defined above, differential subsidences were applied on the supports of the piers looking on to the square, which cause perceptible leaning in these piers and, consequentially, possible structural damage in the vaults and walls of the nave of the church.

The numerical model was made with a calculation programme drawn up by the authors where the models are produced with a standard CAD programme (Alonso Durá 2003). The non-linear model is an isotropic damage model with a scalar variable (Hanganu et al. 1997) (Oller 2001). To calculate the damage variable at all times during the deformation process of the structure, norm r_n of the deformation tensor is evaluated. The expression used is:

$$r_n = \left(\theta + \frac{1-\theta}{n}\right)\sqrt{\bar{\sigma}^T D^{-1} \bar{\sigma}} \tag{1}$$

Where D^{-1} is the inverse elasticity matrix so that $\varepsilon = D^{-1}\bar{\sigma}$

$n = \frac{f_c}{f_t}$ is the relationship between compression and uniaxial traction resistance of the material; θ expresses the predominant behaviour under compression or traction

$$\theta = \frac{\sum_{i=1}^{3} \lceil \bar{\sigma}_i \rceil}{\sum_{i=1}^{3} \left| \bar{\sigma}_i \right|} \tag{2}$$

The unharmed tensions being $\bar{\sigma} = D\varepsilon$ (3)

where $\lceil \bar{\sigma}_i \rceil$ is the Macaulay function of the principal undamaged tensions.

Damage begins to occur when the r_n index exceeds r_0, taken as a threshold,

$$r_0 = \frac{f_t}{\sqrt{E}} \tag{4}$$

The current value of parameter **r** is taken as the historical maximum in the whole load process $r = \max\{r_0, r_n\}$. The damage index then is:

$$d = 1 - \frac{r_0}{r} \cdot \exp\left\{ A\left(1 - \frac{r}{r_0}\right)\right\} \tag{5}$$

Parameter *A* is determined according to Simo and Ju's model [Oller 2001], depending on the energy dissipated in the uniaxial traction trial

$$\frac{1}{A} = \frac{g_f E}{f_t^2} - 0{,}5 \tag{6}$$

where

$$g_f = \frac{G_f}{l_c} \tag{7}$$

G_f being the fracture energy per area unit, taken as a specific property of the material; and l_c the characteristic dominion longitude of the point analysed in the mesh of finite elements. This longitude is determined according to the *V* volume associated to the node of the mesh of solids under consideration.

$$l_c = \sqrt[3]{V} \tag{8}$$

The whole building was modelled and the result was:

- 5,115 hexahedral solids with 8 nodes to characterise the masonry of the walls, the pilasters, the tower and the diaphragm arches.

- 4,186 quadrilateral elements with membrane and plaque behaviour to model the brick fabric vaults and lunettes.

The model comprises 11,889 nodes and 43,900 degrees of freedom. The characteristics of the materials used are as follows:

- For the masonry: density 2000 Kg/m^3; Modulus of elasticity 10,000 N/mm^2; Poisson coefficient 0.18; compression resistance 6 N/mm^2; traction resistance 2 N/mm^2; fracture energy 0.05 N.mm/mm^2

- For the brick fabric: density 1800 Kg/m^3; Modulus of elasticity 5,000 N/mm^2; Poisson coefficient 0.15; compression resistance 4 N/mm^2; traction resistance 2 N/mm^2; fracture energy 0.06 N.mm/mm^2

Below is the conclusion of the results of this analysis:

1. For these dimensions, form and gravitational loads, the monument as a whole does not present significant damage, as we can gather from the assessment of the model of damages in the non-linear analysis.

2. In the hypothesis of subsidence and leaning of the pilasters facing the square, the maps of damages of both the fabric and the masonry of the building present a good correlation with the real figurative map described above (Fig. 6).

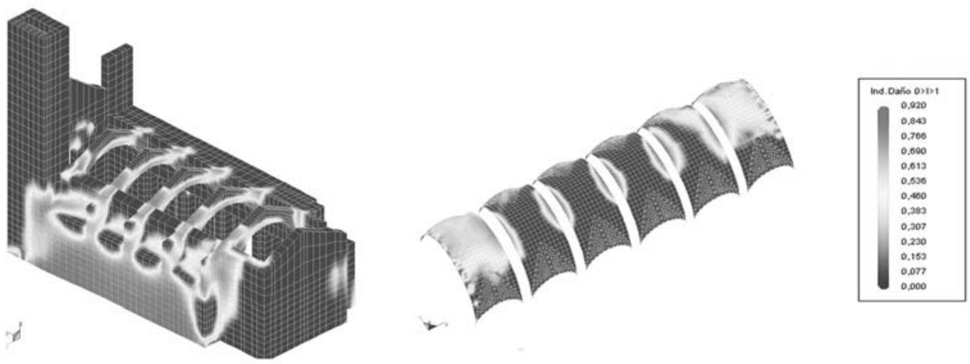

Figure 6 : Model of damages.

6 POSSIBLE SOLUTIONS

Given the situation and since the wall is leaning and there is no possibility of straightening it, several solutions were contemplated to counteract the leaning and subsidence of the lateral wall of the church. To avoid further leaning, there are fundamentally four types of possible solutions: the creation of new sloping piers for the buttresses, the location of vertical weight on top of them in the form of pinnacles, the pre-stressing of these buttresses or even of the whole wall against the ground or the insertion of braces in the ribbed arches of the church.

The first option of creating sloping piers and the second of placing pinnacles on the piers were dismissed immediately because of the serious visual impact they would have on the town square and the facade of the church. The third option consisted in pre-stressing the vertical elements against the ground to provide them with the rigidity necessary to support the thrust of the vault, as though it were a built-in vertical cantilever. This option, albeit more innocuous as it has no visual impact on the square, was discarded because the foundations were unreliable, undermined on one side and therefore unsuitable for added stress. The fourth option does have a certain visual impact inside the church but it is the safest, most viable and harmless for the existing fabric.

To avoid subsidence, the possibility of underpinning the lateral foundations was studied directly. On the basis of the geotechnical study, the ground was found to be resistant enough to withhold the weight of the church and, for that reason, it was decided not to practice injections in the subsoil. The slight subsidence of the lateral wall detected inside the church is attributed therefore to the incautious undermining of the foundations. As a result, lateral underpinning was designed for the foundations, as it would have the least noticeable impact on the ensemble of the town square.

7 TECHNICAL SOLUTION ADOPTED

The results of the structural analysis show that the maximum traction of the brace is 13,500 Kg, so, by using a 20 mm twisted cable, the maximum work tension is 1,679 Kg/cm^2, which can be quite definitely resisted.

Before locating the brace, holes will be drilled in the piers one-third way up the arch, according to the plans. Topographical methods were used to decide where to drill the holes with a view to affecting the facade and the inner decoration as little as possible.

Once the holes have been drilled, the twisted cable with 20 mm nominal diameter and 15 m long with two hexagonal tightening devices to be activated with a dynamometric key would be attached. After attaching the cable provisionally, the 40×40 and 10 cm anchors would be screwed in place and levelled, and the work would conclude by cutting off the ends of the cable and a double-threaded tightening device would be added to tauten the cable (Fig. 7).

Because of the difficulty involved in determining the stresses that affect the foundations, the constructive arrangement of a retaining wall has been adopted at the support level of the church walls. On the basis of the data yielded by the geotechnical study on the characteristics of the site, the solution of a bench resting on a site with an admissible tension of 0.2 N/mm^2 was adopted. This bench is made of reinforced concrete and steel, in observance of the legislation in force in Spain. The structural ensemble will form a continuous retention wall 60 cm thick.

The retaining wall shall take the shape of a bench running along the side of the church opposite the town square, so that this structural reinforcement, which we could describe as a crucial and necessary emergency measure, in the townspeople's eyes will be a new element of urban furniture that has been added to enjoy the socio-cultural activities of the town.

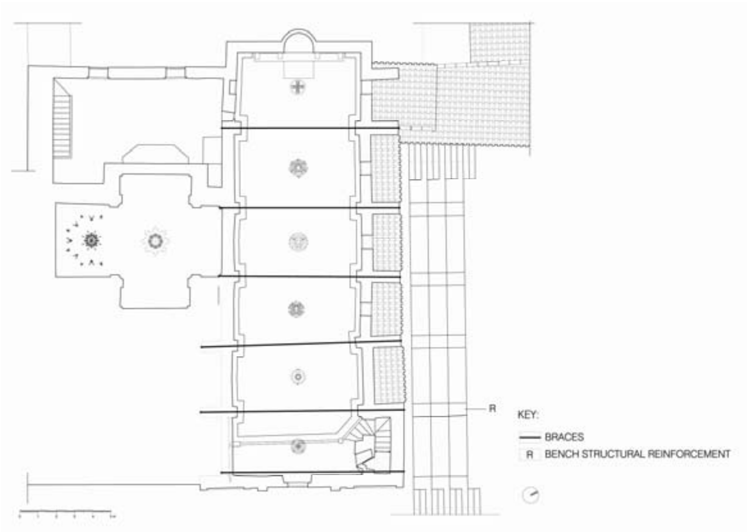

Figure 7 : Solution of braces and reinforcement.

8 CONCLUSIONS

This case demonstrates how it is sometimes the incautious acts of man himself on the foundations of historic buildings that endanger their integrity and survival. In the case of Losa del Obispo, the alteration of the conditions around the church and the levelling of the ground left a whole side of the foundations of a church and a bell tower virtually unprotected and it is surprising that the structural pathologies of the church are no worse than they actually are. In view of the action taken and after carrying out the surveys and structural analysis, the only reasonable solution consists in bracing the vaults to stop the leaning of the wall and a discreet reinforcement of the foundations that are quite exposed at the present time.

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