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

Reinforcement and In Situ Testing of the Upper-Choir of Pópulo Church in Braga, Portugal

João Guedes
Faculty of Engineering of Porto University, Department of Civil Engineering, Porto, Portugal

Aníbal Costa
University of Aveiro, Department of Civil Engineering, Aveiro, Portugal

Esmeralda Paupério
Faculty of Engineering of Porto University, Construction Institute, Porto, Portugal

ABSTRACT: This paper presents the structural intervention on the upper-choir of the Pópulo Church in Braga carried out by DREMN – Regional Directorate for Northern Buildings and Monuments of Portugal, under the technical consultancy of NCREP – Nucleus for the Conservation and Rehabilitation of Buildings and Built Heritage of the Faculty of Engineering of Porto University (FEUP). Besides describing the intervention, it also shows that renovation works to be carried out in old structures, although previously planned and projected according to the available data, may suffer relevant modifications during the preparation and implementation process, which must be regarded as part of the intervention process. In particular, this work involved the in situ testing of the upper-choir structure to evaluate its vertical loading capacity.

1 INTRODUCTION

The Pópulo Church, dating from late 16th century, Fig. 1, located in Braga, Portugal, is made of masonry walls and vaults. Above the main entrance and over part of the church nave, there is an intermediate level, the upper-choir, which is also sustained by a vault.

The square in front of the main façade of the church went through major works in order to build an underground parking structure, which changed the ground conditions around the foundations of the church. This and possibly other reasons that fall outside the scope of this study led to important deformations on the upper-choir vault. In this context, NCREP was requested by DREMN to analyse these deformations, to decide on the need for repair and to present the best structural renovation strategy. Conclusions from the inspection and diagnosis that was carried out may be found elsewhere (Costa 2004).

Figure 1: Pópulo Church façade.

2 PROBLEM PRESENTATION AND STRUCTURE DESCRIPTION

During the restoration works that were carried out at the Pópulo Church, important deformations on the upper-choir vault characterized by joints opening that were visible from the vault's inner face, Fig. 2, were observed. These deformations were located between the arch above the main entrance and the ending arch below the upper-choir balustrade.

To better understand the structure under analysis, the upper-choir vault is described with more detail, particularly in what concerns its shape and materials. The two arches delimiting the damaged area will be referred to A and B; arch A represents the element bearing the balustrade, Fig. 3, and arch B the element over the main entrance of the church. Both arches are made of regular stone masonry and span approx. 9.70m. Near the columns, the arches follow a narrow curved section, which opens towards the keystone; more than 2/3 of the arch span follows an almost horizontal line, Fig. 3. The church plans illustrated in Fig. 4 show the location of the arches and the structure main dimensions.

Figure 2 : Upper-choir vault.

Figure 3 : Façade of arch A.

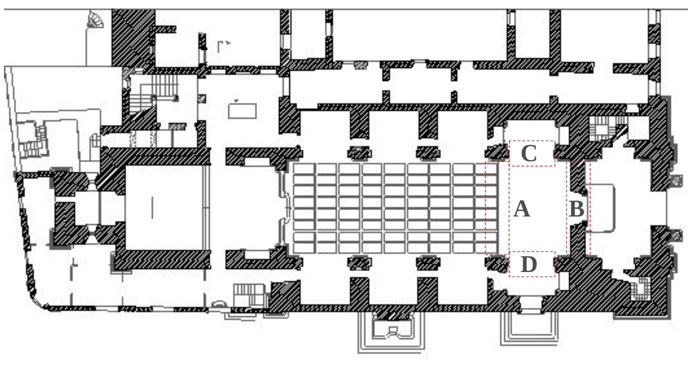

Figure 4 : Church plan with the location of arches A, B, C and D.

The two arches are 5.60m away from each other. The vault is made of regular stone masonry frames, with approx. $(1.70x1.70)m^2$ filled in with redbrick masonry. The vault follows the line of the arches. On the sides, the vault lies upon regular stone masonry arches, with free span of approx. 4.60m, and referred to as arches C and D in Fig. 4.

In terms of damage, arch A exhibit vertical deformations with openings of joints at the keystone. The upper-choir vault exhibits also vertical deformations, surface warping with double inverted curvature, with openings of joints reaching almost 10mm width. The arches B, C and D did not exhibit any deformation. Conclusions from the inspection and diagnosis that were carried out concerning arches C and D can be found elsewhere (Costa, 2000).

3 SEQUENCE OF THE INTERVENTION

3.1 Monitoring

During the first stage of the restoration works, both the vault and arch A were monitored using crack meters, Fig. 5, in order to detect possible displacements of the upper-choir structure, checking, if deformations were still occurring or had stabilised. Simultaneously, the distances between arches A and B (5.62m) and C and D (9.98m), as well as the vertical distance of several pre-defined points along the medium transverse line of the vault, were measured using a laser distance-meter, with respect to a fixed reference structure.

This monitoring campaign was carried out during a year and a half period without any relevant changes on the monitored values. This data is always crucial to decide on any structural intervention, since damage corrections, strengthening or consolidation procedures should not be applied to while structural deformations are still occurring.

Figure 5 : Crack meter on a stone frame of the vault.

3.2 1st action on the structure – global reinforcement

Although the monitoring phase did not bring to attention any active displacements, considering the current deformation of both the arch A and the upper-choir vault, as well as the almost horizontal shape of these elements at the central area, it was decided to strengthen the upper-choir structure.

An inspection of the vault inside the stone masonry frames allowed estimating a 30cm thickness shell and confirming that it was made of redbrick elements, Fig. 6. These and other preliminary actions allowed identifying the geometrical and material characteristics of the structure. After analysing this data, it was decided to design a steel structure to be set above the vault and under the wooden floor, to suspend the vault through vertical ties anchored with plumb on the crossing points of the stone masonry frames, Fig. 7. The structure would be tied up to the upper-choir side walls and would sustain vertical displacements that might occur, putting at risk the structural stability of the vault (Costa, 2004).

Figure 6 : Inspection to the inside of the vault.

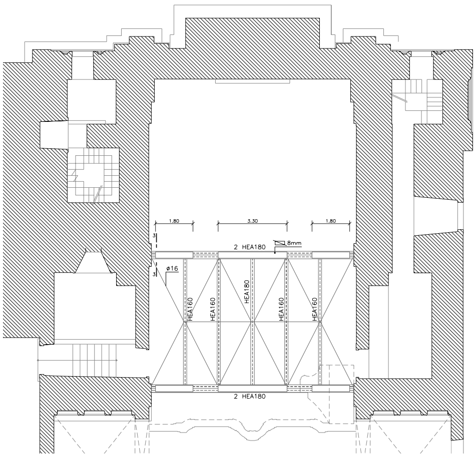

Figure 7 : Upper-choir plan; reinforcement steel structure.

Although this procedure seemed reversible and little intrusive, some physical limitations had to be overcome. In particular the steel structure introduced major forces at the anchorage points on the walls, demanding a good mechanical and structural characterisation of these areas in order to ensure the necessary capacity to sustain the anchorage forces. Although, apparently these areas corresponded to good quality regular masonry zones, in other parts of the church with similar appearance, the regular stone masonry was only used for covering masonry of inferior quality, Fig. 8. Hence, a visual inspection would not be sufficient to assess the strengthening capacity of these areas, and in situ tests had to be carried out (Roque, 2003).

Figure 8 : Faked regular stone masonry column at the upper-choir level.

At this stage two points were highlighted. First, from the monitoring of the vault and arch A it was observed that no active displacements on these two structures were detected i.e., the deformations had stabilised. Second, practice shows that, and despite of its flat shape, this kind of structures possesses a high strength capacity, as far as no longitudinal movements are allowed, in particular at the set-up points. However, and because of the deformations that the structure already experienced in the past, a load test was performed to confirm this point.

3.3 2nd action on the structure – load test

Arches and vaults are designed to sustain compression forces only. In most cases, these elements are oversized, showing a load capacity much higher than the one they were actually designed for. Based on these considerations and on the authors belief that the vault was able to resist the design live load, around $1.5x2.0kN/m^2 = 3.0kN/m^2$, a load test was performed to confirm this assumption. Moreover, such test would allow registering the deformation evolution according to the applied load and assessing the vault structural performance.

To prepare the test, a scaffold was placed to create a working platform at the level of the inner surface of the vault. It was positioned so that its vertical elements were aligned with the knots of the granite frames, leaving the top of the vertical elements at approx. 1cm away. With this procedure, the scaffolding structure acted also as an emergency prop in case any «plastic» deformation of the vault occurred.

During the tests, the load was applied by filling up with water 92 metal tanks with an average inner diameter of 58cm and an average inner height of 86cm, aligned on the upper-choir floor, Fig. 9.

Figure 9 : Hydraulic network, tanks positioning and measurement equipment.

To make the process easier and guarantee a homogeneous load distribution over the floor, the tanks were linked at the bottom using a hydraulic network, connected to the municipal water supply system. A valve and a counter were set at the entrance of the piping system to control and record the input water i.e., the actual load on the vault. To empty the tanks efficiently, the network was connected to 2 pipes with valves, discharging directly into a rainwater collector at the square in front of the church main entrance. During the tests the vault was monitored using digital and analogical comparing equipment measuring the vertical displacements, Fig. 10, and the crack meters previously installed to control the openings of the joints. The control of the vertical displacements was performed at the knots of the vault masonry frames. To proceed with this, steel threads were tied up to these points and kept straight through weights that were set against the cursors of the comparing devices fixed on the nave floor using heavy bases. This procedure allowed transmitting the vertical movements of the vault to the comparing devices that recorded the displacements.

The test was performed for 5 load steps that corresponded to load values of 6231kg ($0.91kN/m^2$), 10328kg ($1.50kN/m^2$), 13768kg ($2.00kN/m^2$), 17268kg ($2.51kN/m^2$) and 20081kg ($2.92kN/m^2$). To check the apparatus, the first load step was divided into lower load steps. Some water leaks detected in the network connection terminals were repaired and the tanks' water level was checked to compare it with the input water volume recorded by the counter. After concluding this procedure, the tanks were emptied and the test was reinitialised and the 5 load steps programme was followed.

After the last one, the water level in every tank was registered, in order to assess the total load actually installed at the upper-choir floor, around 3.0kN/m^2. Between each load step, the values registered by the comparing devices were recorded. The highest value, almost 3mm was registered at the vault central area. The crack meters were also continuously checked but no visible displacements were detected.

The outcome analysis and particularly the installed force versus measured vertical displacement curves at two central points - the most significant registered points - show an almost linear performance, with an insignificant loss of stiffness without evidence of imminent failure trend, Fig. 10. In this analysis, the possible occurrence of an immediate failure without any “plastic deformation” is safeguarded. On the other hand, the small vertical displacements registered, associated with the non visible openings at the monitored joints, confirmed that the structural capacity is within the acceptable safety limits. Considering this new outcome, the strengthening solution proposed on section 3.2. was abandoned, and the intervention was reformulated.

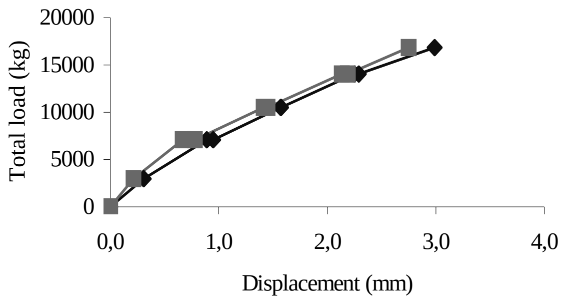

Figure 10 : Load versus vertical displacement curve.

3.4 3rd Action on the structure – final solution

After the results of the load test, two possible actions were considered: a) to restore the initial conditions regarding the material on the vault (gravel and stones), the levelling of the upper surface to support the beams for the new wooden floor, without carrying out any structural repair or strengthening action, or b) to work locally, just by strengthening the connection between the stones of the masonry frames before proceeding with procedure a). After a careful analysis, and taking into account that most of the vault upper face was already uncovered and exposed, the b) solution was adopted.

The stones of the masonry frames were linked together with inverted U or E-shaped 50mm width and 5mm thickness stainless steel plates anchored to the stones with a delivery of around 50mm, in order to strengthen the upper face of the stone masonry frames, as shown on Fig. 11. The U-shaped plates were used to connect two stones and the E-shaped to connect three stones, linking one or two joints, respectively. E-shaped plates were used in small-sized stones, where joints were too close. Holes were made on the stones and the plates were anchored by the traditional process of melted lead. Whenever the stones surface was irregular, after the above procedure, a SikaGrout mortar was placed between the plate and the stones, enabling a uniform contact between both elements.

It should be stressed that the reinforcement performed on the upper face will prevent flexural deformation of the vault on the sides, and it will contribute to increase the vaults stiffness and, consequently, to decrease the ability of the structure to deform. On the other hand, by including the strengthening of the vault central area, this procedure provides a more uniform behaviour and, therefore, a better global performance of the structure, fulfilling the purpose of the intervention (ICOMOS, 2001).

Finally, the joints that were opened were closed using non retractile mortar compatible to the vault masonry (Luxan, 2002) in order to seal them. In the future, this procedure would also allow controlling further displacements that would occur at the vaults.

Figure 11 : Stainless steel plates anchored to the stones.

4 FINAL CONCLUSIONS

This paper describes the renovation works performed at the upper-choir of the Pópulo Church in Braga. This was a technical consultancy work requested by DREMN to NCREP, to help to decide which kind of intervention would be suitable to repair the damage found at the structure: deformation of the upper-choir vault with cracking and opening of the joints visible at the inner face, as described on section 3.

As such, after monitoring the vault and confirming the stability of the observed deformations - crack metres positioned at key points for a year and a half have not detected any displacement - a strengthening structure was designed, consisting of a steel structure to be installed above the vault keystone and linked to the vault on the stone masonry frames knots through vertical ties in order to sustain any future vertical movement of the vault. The implementation of this solution, however, presented some obstacles, referred to on section 3.2. Considering this analysis, it was then decided to perform a load test on the upper-choir vault, in order to verify the capacity of the structure to resist the design live load that would be compatible to the accepted use of this area. In fact, the test showed the good performance of the upper-choir vault for a live load of 2.90kN/m^2, with a maximum 3mm displacement, largely compatible to the use of this space conceived for restrained access.

The proposed strengthening solution was then abandoned and a minimum action was suggested and implemented. It consisted of anchoring stainless steel plates on the upper face of the masonry frames in order to link the stones together. This procedure decreased the ability of the structure to deform and guaranteed a more uniform performance of the vault, due to a better connection between the frame stone elements.

Finally and as a general conclusion, this paper shows the peculiarities of the intervention works on old buildings. Despite previously planned and projected according to the possible knowledge of the existing structure, during the preparation and implementation of the intervention procedures they may suffer relevant modifications that must be seen as being part of the whole intervention process.

ACKNOWLEDGMENTS

The authors wish to express their gratitude to DREMN and to Eng.Filipe Ferreira from the company Augusto de Oliveira Ferreira L[da].

REFERENCES

ICOMOS 2001. Recommendations for the analysis, conservation and structural restoration of architectural heritage.

Costa, A. Guedes, J. e Paupério, E. 2004. Relatório de Intervenção Estrutural no Coro Alto da Igreja do Pópulo; Abóbada e Arco Principal, *Technical Report*, *Construction Institute*, Faculty of Engineering of Porto University, Porto, Portugal.

Costa, A. Arêde, A. e Costa, C. 2000. Relatório de Inspecção ao Coro Alto da Igreja do Pópulo *Technical Report*, Construction Institute, Faculty of Engineering of Porto University, Porto, Portugal.

Luxán, P., Dorrego, F. 2002. Morteros Antíguos y la Intervención el Patrimonio – *A Intervenção no Património Práticas de Conservação e Reabilitação* p.183-191– FEUP.

Roque, J.C.A, Lourenço, P.B. 2003. Caracterização Mecânica de Paredes Antigas de Alvenaria. Um Caso de Estudo no Centro Histórico de Bragança, *Engenharia Civil*, 17, p. 31-42.