Structural Assessment of vaults of the Igreja Matriz de Bucelas in Portugal

A. Gago
CERIS, Department of Civil Engineering, Architecture and Georesources, I.S.T, Av. Rovisco Pais nº1, 1049-001 .Lisboa, Portugal

J. Alves
CERIS, Department of Civil Engineering, Architecture and Georesources, I.S.T, Av. Rovisco Pais nº1, 1049-001 .Lisboa, Portugal

R.Ferreira
CERIS, Department of Civil Engineering, Architecture and Georesources, I.S.T, Av. Rovisco Pais nº1, 1049-001 .Lisboa, Portugal

ABSTRACT: Since its construction, the Bucelas church has suffered significant damage due to several seismic events. After of those events, some structural repairs were performed, mainly in the vault of the nave. However, there are no accurate records about the nature of those repairs. In 2009, during the refurbishment of the paintings of the vaults, concerns were raised regarding the structural safety of the nave and aisles vaults given the high deformation exhibited by transverse arches.

The scope of this article is the description the structural assessment carried out by I.S.T regarding the structural safety of those structural elements.

Preliminary information regarding the geometrical survey is presented has well as a description of the existing damages both in the intrados and the extrados of the church vaults. Numerical studies carried out using the UDEC and 3DEC software of the vaults are also presented.

1 INTRODUCTION

The Igreja Matriz de Bucelas (Bucelas Church) is the most important church of the Bucelas parish and is an important religious monument of the Lisbon area.

Built in sixteenth century, the Bucelas church particular relevance is appointed by the artistic quality of its interior, mainly its sculptural works and interior paintings. Beside its historic value, this church is a remarkable example of the Portuguese baroque.

The Bucelas church has a rectangular plan of 43.5 m x 10.9 m. The church has a 5.5 m wide nave and two 2.7m wide aisles. Peripheral walls of the church consist in 1.0 m thick masonry walls with some stone brick inlays. The Narthex is located at the South end of the Church, while chancel and Main altar are located at the opposite end of the Narthex.

The nave and the aisles have an upper masonry barrel shaped vaults, reinforced by stone brick arches. These transverse arches are spaced 5.0 m in the longitudinal direction (N-S). Vaults and transverse arches springing consist in two longitudinal masonry arcades supported by stone brick columns spaced 5.0 m from each other. The church has also five buttresses located in the East wall

Both vaults of the nave and the aisles are decorated with XVIII paintings. The inner sides of the lateral walls of the church are decorated with XVIII hand painted tiles.

In 1969, as result of a 7.3 magnitude earthquake, the vault of the nave and the roof of the church suffered severe damages.

Subsequently, repair works of the roof and the transverse arches were carried out. During these, the fill existing in the extrados of the vault was partially removed.

The new roof (which is the existing roof over the vaults) consists in two symmetrically inclined hollow slabs, using ceramic hollow blocks, prestressed concrete joists and thin upper layer of cast in place concrete filling.

Each roof slab has three supports: two extreme supports and one intermediate support. One of the extreme supports is at the ridge of the roof and was made by considering a continuous RC beam all across of the roofs ridge. Several small brick piers standing over the extrados of the vault support this RC beam. Supposedly, each one of these small pies is align with the keystone of each transverse arch of the nave. The other extreme support of the roof slab, consist in small continuous brick wall over the external wall of the church. The intermediate support consists also in a small, continuous brick wall, but this time constructed over the longitudinal arcade, which separates the lateral aisle and the nave.

Also during the refurbishment works carried out after the 1969 earthquake, several voussoirs of the transverse arches were clamped together and joints were filled with mortar.

In 2007, started some refurbishment works of the murals and paintings of the chancel and the main altar. In 2009, the refurbishment works extended to the vault of the main nave. During this latter refurbishment works, excessive deformation and the existence of slipping of some of the voussoirs of the transverse arches raised some concerns about the structural safety of the church vault. Hence, before repairing the vault paintings, some interventions were made attempting to correct those damages. The slits detected in the intrados of the vault were filled with mortar and additional clamping was executed.

Given the high architectural interest of the church and the level of uncertainty regarding the structural safety of the existing structure, ICIST took interest in this matter and started a structural safety assessment. The main purpose of this article is to present the preliminary proceedings carried out regarding this process. (1) (2)

Figure 1 - Bucelas Church. South Wall of the Church. Main entrance)

Figure 2 - View of the church interior .Nave and lateral arcades.

2 STRUCTURAL ASSESSMENT

In order to determine the cause of the existing deformation, several inspections were made, namely inspections of the intrados and extrados of the nave and aisles vaults. The information gathered was used for the development of several preliminary bidimensional and tridimensional analytical models. The purpose of these models was to simulate, as realistically as possible, the actual structural behaviour of the church vaults. (3)

As aforementioned, the new roof construction led to the removal of a considerable portion of the spandrel filling of the nave vault. Hence, the church vaults are now subjected to an unforeseen loading configuration and the influence of this load on the existing deformation pattern should be assessed. (3) (5)

The main structural pathologies of the Bucelas church are located in the transverse arches of the nave vault, cracks in the masonry of the vault and slipping of voussoirs of the transverse arches.

Figure 3, illustrates the distribution of the transverse arches of the vault of the nave (inverted plan of the intrados of the vault).

A preliminary geometrical survey was carried out to determine the dimensions of the vaults and the transverse arches. The preliminary results of this geometric survey are presented herein:

Arco 1 (Transverse Arch 1): Arch embedded in the main entrance wall. This is less deformed arch. However, near arch 1, the vault shows significant deformation and cracking. Transverse Arch 1 has a 5.50 m span and 2.76 m rise.

Arco 2 (Transverse Arch 2): This is the most deformed arch. The span of transverse arch 2 is 0.35 m longer (6.4% longer than the span of Transverse arch 1).Three hinges were detected over the length of the arch (one hinge beside the keystone and two hinges in both sides of the arch, positioned approximately 2.8 m away from the arch springing. Steel clamps and mortar fillings of joints of the transverse arches voussoirs were found, mainly near the keystones and the adjacent voussoirs. Nevertheless, significant slippages of the keystone were observed.

Arcos 3 e 4 (Transverse arches 3 and 4): Spans of transverse arches 3 and 4 are 0.22 longer than the span of transverse arch 1, i.e. about 4% longer transverse arch 1. Despite some particularities, the deformation pattern of these arches is similar to the transverse arch 2 and hinges located approximately in the same location along the length of the arch.

Arco 5 (Transverse arch 5): Span of transverse arch 5 is 0.19 m longer than the span of transverse arch1, (3.5% longer than transverse arch 1). Although this arch is less deformed, it shows similar deformation pattern to the previous transverse arches. The hinges were also detected along the length of the arch, in similar locations the hinges detected in the previous arches.

Aisles vaults and transverse arches: Aisles vaults and transverse arches also showed some damages, namely hinges and crushing of some voussoirs adjacent to the keystones of the transverse arches. However, in these cases, hinge rotation occurred in the opposite direction when compared to the rotation of the hinges observed in the nave transverse arches.

According with the geometrical survey carried out, both vaults of the nave and the aisles are 0.18 m thick and the voussoirs of the transverse arches have a rectangular cross-section of 0.17 m x 0.30 m. The columns are circular, having a total height of 5.20 m (excluding the column base and capitel) and diameter varying in the upward direction from0.56 m to 0.67 m. The arches of the longitudinal arcade have a rise of 2.15 m, free span of 4.29 m and thickness of 0.56 m.

Inspections of the extrados of the nave vault allowed obtaining further information about the existing structural elements in the roof of the church, namely: 1). In the extrados of vault longitudinal, a longitudinal crack was observed. This crack might result from hinge formation in the vault; 2) One of supporting piers of the roof showed cracking of its cross-section evidencing that the pier was subjected to tension or imposed displacement. This phemomena might contribute to the increase compression forces of the adjacent piers over the vault; 2) Fill in the vaults was only partialy removed. Some of this initial filling was preseved.

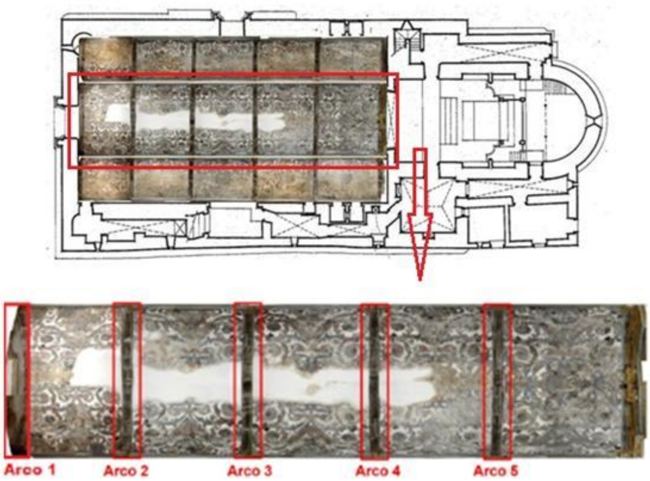

Figure 3 - Plan view of the intrados of the nave, with indication of the transverse arches position

2.1 *Bidimensional models*

Numerical studies were performed to assess the structural behaviour of the original undeformed configuration of the nave vault. These numerical studies involved the analysis of in plane and tridimensional behaviour of the vault (3). Numerical studies were also carried out to determine the in plane and tridimensional behaviour of the global structure of the nave and the aisles, including the stone brick columns and the lateral walls of the church.

The numerical studies were performed using finite elements software SAPV16 (CSI 200) and the distinct elements software UDEC and 3DEC (6). Numerical studies comprised five numerical models: two in-plane models using UDEC software and two tridimensional models using the 3DEC software. The first step of this analysis was to obtain estimate for the self-weight load imposed by the roof slab on the vault of the nave. To determine the value of the load imposed on the extrados of the vault, a three-dimensional model of the roof was constructed using SAP V16 software (7). The value obtained for this load was P_v=32 kN.

Afterwards, two in-plane numerical models were conceived to analyze the local and global vault structural behaviour, assuming it's undeformed configuration and subjected to self-weight load and also a single load applied on the keystone (P) or imposed displacements (Δ) at supports.

Firstly, an in-plane model was constructed to assess the local structural behaviour of the vault without the transverse arches. For the masonry vault, 0.18 cm thick blocks were used to simulate the thickness. Null tension and limitless compression was considered for block joints. Joints shear resistance was defined considering a Mohr-Coulomb model. The mechanical properties considered in all distinct element numerical models are shown in the following tables 1 and 2.

Table 1 – Mechanical properties of structural elements.

Elements	γ_s (kg/m3)	E_s (GPa)	v_s
vaults	2100	-	-
arches	630	-	-
fillings	2100	1	0.2

Where: γs represents the density of the material, Es represents the Young modulus and νs represents the Poisson coefficient.

Table 2 – Mechanical properties of joints between blocks.

Element	kn (GPa/m)	ks (Gpa/m)	c (MPa)	Φ (º)
joints	1000	1000	0.5	30

Where: *kn* represents the compressive stiffness of the block joints, k*s* represents the shear stiffness of the block joints, c represents the cohesion in the block joints e Φ represents the internal friction angle in joints between blocks.

Restrains for the in plane model M1 are shown in Figure 4. In this model, four rigid blocks were used to establish the vault springing and all lateral displacements of these rigid blocks were restrained. Partial filling of the vault spandrel was considered according with the information gathered during the roof inspections.

Structural collapse was attained for the following scenarios: a maximum vertical load P_{max}=6kN/m or a maximum span increase of Δ_{max}=0.32 m.

This last result is similar to the result obtained when the Ochsendorf (4) model is used. Using Ochsendorf analytical model considering α=60º e t/R=0,062, maximum admissible span increase obtained was $\Delta_{máx}\approx$28 cm.

The in-plane model (model M1) considered the combined behaviour of the masonry vault with the transverse stone brick arch (cross section 0.17x0.30m). Figure 4 presents the mechanical properties considered for the vault, arch and filling. In this numerical model, an elasto-plastic behaviour model was considered for the joints between the vault and the arch blocks. It was assumed that tension resistance was null, a compression stiffness of 1000 GPa/m and shear resistance of 1000 GPa/m. Shear resistance of block for arch behaviour simulation was obtained using a Mohr- Coulomb model, considering cohesion c=0.5 and internal friction angle of ϕ=30º. Mechanical properties of the vault were maintained equal to the properties used in the preliminary model.

In M1 model, collapse was attained for a load P of 108 kN or a maximum span increase of Δ_{max}=0.43 m.

Results from distinct element model, considering each structural set and respective applied loads, allowed also to observe the formation of an hinge positioned approximately 1.25 m above the infilling of the vault. This observation matched with the cracking pattern observed the extrados of the vault. These results gave some confidence about the quality of approximation of the numerical relatively to the real behaviour of the structural and the influence on stability of the partial infilling existing in the extrados of the vault.

Another in-plane model (model M2) was conceived in order to assess the global behaviour of the structural set vault+arch, considering the structural effect of the stone brick columns, the aisles vaults and of the church's lateral walls. Consequently, model M2 would also allow a more accurate estimate of the load (P_{max}) applicable to the keystone of the nave vault, assuming the undeformed configuration of the entire structural set.

In M2 model, the mechanical properties of all structural elements were equal to the ones used in the previous models. Both the columns and the lateral wall were modeled using rigid distinct blocks and end supports (foundations) of these elements were restrained for all lateral displacements. Figure 5 illustrates the global in-plane model M2.

Results from M2 model determined that the maximum applicable load to the keystone of the nave vault was P_{max}= 91 kN. This result is slightly lower than the result obtained from previous in-plane model M1. On the other hand, the deformation pattern of vaults of the nave and aisles in this M2 model allowed to observing that, due to stiffness of the lateral walls, the outward thrust of the nave vault induces a deformation of the lateral vaults. According to the deformed shape results for the applied load P_{max}, the nave vault has a downward deformation pattern while the aisles vaults have a contrary deformation pattern.

Additionally, the formation of hinges in the lateral vaults was also observed. These hinges were observed near the joints of the keystone.

2.2 *Tridimensional models*

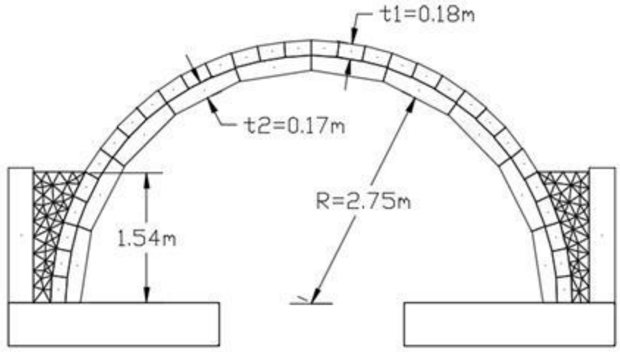

Figure 4 – Bidimensional model M1

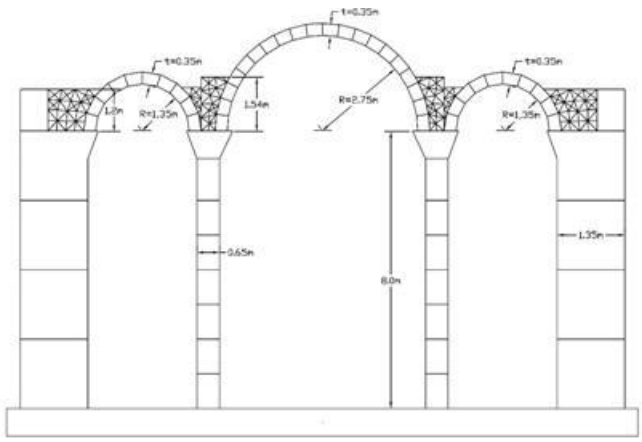

Figure 5 – Bidimensional model M2

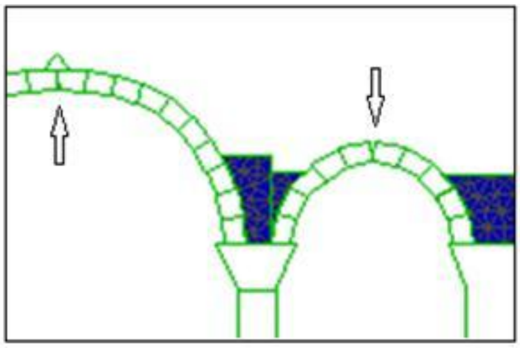

Figure 6 –Deformation pattern and hing formation

Two tridimensional models (model 3 and Model 4) were constructed using 3DEC software. Mechanical properties of materials and block joints behaviour models were similar to the ones used in the previous in-plane models.

Model 3 was constructed to analyze the local behaviour of the structural set composed by a continu-

ous barreled shaped masonry vault and reinforcing stone brick arches with a longitudinal spacing of 5 m. Figure 7. All distinct block elements were considered rigid, except for the blocks simulating the infilling existing in the extrados of the vault (blue). In addition, all base supports of the continuous vault and arches restrict all degrees of freedom.
Beside the self-weight load, a single load P_{max} was appointed on the vault, aligned vertically with the keystone of the transverse arch 2. The value of this load was sequentially increased until structural collapse of the vault. Figure 8 illustrates the tridimensional model considered for the analysis of global behaviour of the nave vault subjected to a single load P in transverse arch 2.

Using model M3, value of collapse load was P_{max}=190 kN. This value is higher than P_{max} obtained in the in plane model M1 (P_{max}=108 kN). This difference may be due to fact that in the three-dimensional model accounts the bearing capacity of higher area of the vault as well as the contribution of the adjacent transverse arches.

Model M4 was constructed in order to achieve ultimate realistic approximation of the structural behaviour of the church structure, hence, to achieve a more accurate a more estimate of P_{max} of the nave vault. This model included the nave and aisles vaults, the transverse arches, the stone brick columns and the Church's lateral walls. Materials and mechanical properties of distinct blocks were the same as used in the previous models. However, in this model, as in the real structure of the church, the nave vault is supported, in both sides, by the longitudinal arcades. The internal columns of the church then support these longitudinal arches.

Similarly has in the previous tridimensional model (model M3), the transverse arches longitudinal spacing was 5 m. The vault distinct elements were considered to have 0.18 m depth and the transverse arches were modeled used distinct blocks with 0.17 m depth by 0.30 m wide. Columns were considered to have equal spacing as the transverse arches. Columns diameter (D=0.65 m) was considered constant along its height (5.20 m). (Figure 8)

Once again, a numerical analysis was performed to attain an estimate P_{max}. A single incremental load was applied on to the extrados of the vault, align with the keystone of the transverse arch 2.

A value of P_{max}=117 kN was achieve using model M4. The value of P_{max} is lower than the value obtained in the previous model (P_{max}=190kN), however is higher than P_{max} obtained in the in-plane global model (model M2). Once again, values of P_{max} obtained in tridimensional models have shown to be higher than those obtained using two-dimensional models.

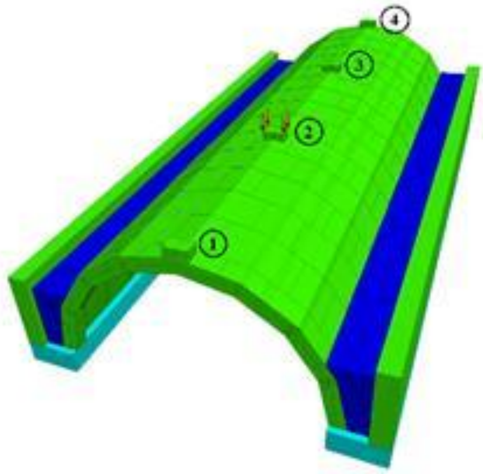

Figure 7 – Tridimensional model M3

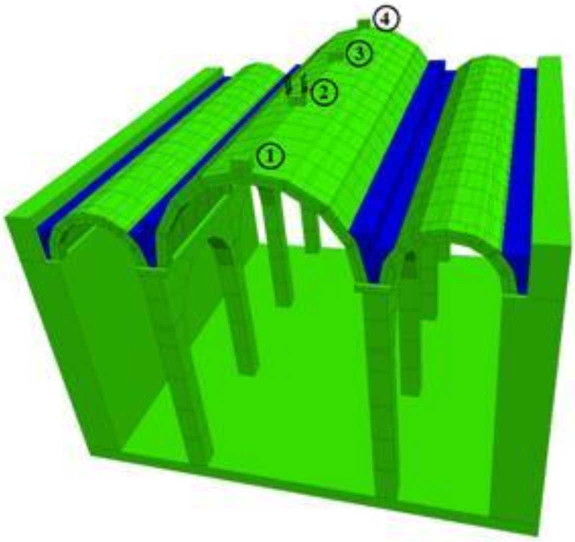

Figure 8 –Tridimensional model M4

2.3 *Bearing capacity of the vaults*

The M1 model was also used to analyze collapse scenario considering the combine effect of a single load applied on the keystone of the set vault+arch and the application a sequential imposed displacement Δ on the springing. In this model, clamping of the voussoirs of transverse arches was disregarded. In addition, it was considered that loads P and Δ, acted upon the undeformed configuration of the structural set.

.Equilibrium of the structural set was assessed using the analytical model, considering the combined effect of self-weight, imposed displacement Δ and P_{max}.
Table 3 presents results obtained for the maximum load P (P_{max}), for different values ofΔ.

These results show that all P_{max} obtained considering different values of Δ are lower than load P_v. Considering these results, one could observe that the span increase observed in the transverse arches is not compatible with the acting load P_v and, thus, structural collapse should have occurred. Since the structure is still standing, three hypotheses can be considered: a)Value of load P_v is lower than the value estimated; b)Real deformations values of the transverse arches are lower than the values obtained from the geometrical survey; c) Clamps in the voussoirs play a crucial rule in the internal balance of the existing structure;

From these three hypotheses, we suppose that hypotheses b) and c) are the most probable. Therefore, a detailed geometric survey is being carried out to determine more accurately the deformed shape of every transverse arch. In addition, further analytical studies are being performed in order to assess the influence of the clamps on the bearing capacity of the structural set, assuming, once again, different values of Δ.

Table 3 - Results for P_{max} considering different Δ

Structural set	Δ (m)	P_{max} (kN)
Vault+Transverse arch 2	0.35	14
Vault+Transverse arches 3 and 4	0.22	18
Vault+Transverse arch 5	0.19	21

3 CONCLUSIONS

Analytical models M1 to M4 using distinct elements have shown very useful in the analysis of the structural behaviour of the Bucelas church vaults and traverse arches. Considering the vertical loading of the vaults, overall deformation patterns obtained have shown to be very similar to the deformation pattern observed in the church vaults. Hinge formation was also attained in the analytical in similar location to the hinges observed during the structural inspections. Influence of the extrados infilling on the stability of the vaults was also observed in the analytical model analysis.

It was observed that, in a scenario were the existence of voussoir clamping is neglected and the deformed shape of the vaults assumes different values of Δ, the maximum bearing capacity of the existing vaults P_{max} is always lower than the initial estimate $P_{v.}$

Analysis of models M1 to M4 also allowed also to obtain other conclusions. Firstly, estimates for P_{max} obtained from bidimensional models (M1 and M2) are more conservative than values of P_{max} obtained from tridimensional models (M3 and M4). On the other hand, local analysis (M1 and M3) model overestimate P_{max} when compared values obtained from global analysis models (M2 and M4). In the first case, it seems clear that lateral restrains imposed in the structural set tend to overestimate transversal stiffness, whilst in global model; transversal stiffness of the supports is more accurate since all adjacent structural elements where considered. In the second case, tridimensional models tend to give higher values of P_{max} because of the contribution of adjacent transverse arches to vertical stiffness of the structural set.

The use of global models also allowed the observation of the structural effect of the aisles vaults on the deformation pattern of the nave vault. It is quite clear that the existing equilibrium of the nave vault also depends on the structural stiffness of both aisles vaults. Figure 6 illustrates the deformation pattern of the aisles transverse as well as an example of hinge formation near the keystone of one of the transverse arches.

Another important conclusion is that the existing infilling in the extrados of the vaults plays an important role in the existing structural equilibrium. This infilling has a stabilizing on the arches by restraining the horizontal displacement of the voussoirs, hence decreasing the free span of the vault. Nevertheless, existing longitudinal cracks in the extrados of the vault indicate the formation of hinges and, hence, the formation of potential collapse mechanism.

However, questions were raised about the actual load capacity of the vaults since all values obtained for the P_{max} of the vaults considering several cases of imposed deformation Δ (model M1 - Table 3). In these studies was disregarded the effect of the clamps of the voussoirs of transverse arches. Therefore, further numerical will be performed in order to assess the influence of such elements in the internal equilibrium of the structure. Additionally, since that Bucelas Church is located in a high seismicity area, numerical studies will performed to assess the seismic performance of the Church and, if necessary, seismic retrofitting will be proposed.

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