Structural scheme of the Cathedral of Burgos

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ABSTRACT: Structural analysis of the Burgos Cathedral was carried out in order to establish the structural scheme of the main church building. The main tool in this study was a 3D Finite Element model that simulated in sufficient detail one representative bay of the Cathedral. The model provided the opportunity to examine the behaviour of the main load-bearing elements in their real loading and boundary conditions and assess the contribution of each in the structural scheme of the building. The low level of stresses demonstrated the high reserves in strength of the original design. The analytical model can further gauge the applicability of more simplified modelling strategies for the building or parts of it, while it can also be used to examine more complex loading conditions or the origin of some more specific defects.

1 INTRODUCTION

The conservation of large-scale monuments in Europe has become increasingly complex as heritage is viewed nowadays as a source of development and the conservation of the built environment has penetrated a wider variety of areas of everyday life. The issues have moved beyond the strictly technical challenges, which in turn must incorporate into a flexible framework of actions and uses. In particular, there is a need to establish a reliable global structural scheme into which the behaviour of important load-bearing or not elements can be referred or interventions can be gauged.

The cathedral of Burgos in Spain is one of the classic Spanish cathedrals and an important work of art within the world heritage. The cathedral was built in the 13th century during a relatively rapid and consistent campaign that conveyed a stylistic and structural unity. The structural scheme of the main church building follows a typical yet well defined configuration. No significant problems have been recorded during the life of the cathedral, so a structural analysis can provide some useful recommendations on the performance of critical load-bearing elements.

The original configuration of the church is examined as was determined through surveys and analysis of the system of proportions. The main analytical tool was a 3D Finite Element (FE) model of one of the bays of the nave, generated in sufficient detail to permit visualisation of stresses along the thickness of some critical elements and the assessment of the transfer of the loads between the vaults and the lateral walls. The action of the self-weight of the structure was mainly considered in this stage as the application of imposed loads due to wind would be quite complicated due to the geometry of the buttressing system.

Furthermore, the behaviour of the vaults will be examined in detail, assessing the effect of the boundary conditions and loading of the rest of the structure upon them. In addition, the effect of the construction sequence on the structural behaviour of the building will be studied by removing critical structural elements like the buttresses from the main FE model.

2 THE CATHEDRAL OF BURGOS

2.1 *Description*

The Cathedral of Burgos is situated at the centre of historic centre of Burgos. Extensive research on the function of the main structural elements and their construction sequence can be found in (Rico 1985, Andrés 1993, Karge 1989, Martinez 1866, Torres Balbás 1952). The entire complex is supported on the slope of the citadel hill and a series of underground chapels. The main church is a Latin cross in plan, arranged in a nave and two aisles (Fig. 1). Longitudinally, the church is enclosed and fixed by a façade characterised by a rich decoration that incorporates deep buttresses and spires that crown

the two towers (Fig. 2). On the east end, the structure is retained by the ambulatory, to which the large chapel of the Constables of Castile is added, while the two-level cloister braces the SE quarter.

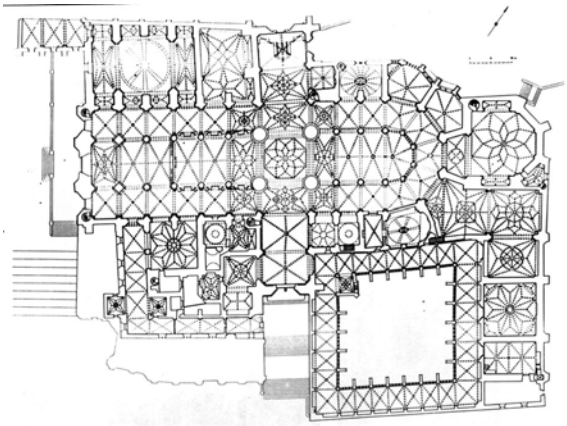

Figure 1. Plan of the Cathedral (Karge 1989)

Figure 2. External aspect of the Cathedral (Andres 1993)

Highly decorated façade structures that integrate stiffening buttresses and turrets at their corners perform a similar retaining function for the transepts as well. A freestanding octagonal tower marks the crossing and is supported on four heavy piers.

The zone of the apse is where most of the construction phases can be identified. A first ring of radial chapels was added after the Renaissance and traces of other lesser chapels can still be seen at the external perimeter of the apse. The entire Cathedral is flanked by chapels, most of which have been accommodated in various configurations and styles between the outer pier buttresses and therefore affect only locally the structural scheme of the Cathedral.

The transept constitutes actually the limit between the two main construction phases of the apse and the nave, as stylistic differences can testify (Karge 1989). Additionally, a study of the plan (Fig. 1) shows a misalignment between the axes of these two parts, most probably due to the need to re-use the foundations of the pre-existing Romanesque Cathedral.

Figure 3. The interior of the Cathedral

The crossing tower (Fig. 3) is a focal point of the scheme not only because of its position, volume and decoration but also due to the major reconstruction of the entire crossing area its collapse caused in 1539. The heavy pilasters that were then incorporated provide further stiffness to the ends of the choir and the nave and to some extent compartmentalise structurally the building.

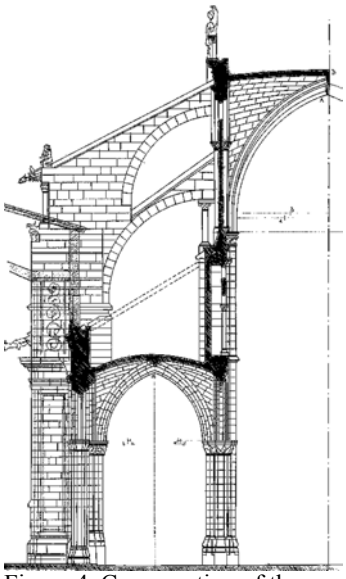

Figure 4. Cross section of the nave – third bay from the facade

The elevation of the nave follows the typical division into nave arcade, triforium and clerestory (Fig. 4) and the differences with that of the choir are mainly stylistic. The compound piers are formed by a cluster of 8 shafts that join the ribs of the vaults above, providing an aesthetic unity that will be discussed later. The nave arcade is a pointed arch with a not very deep section, which means the *tas-de-charge* ending of the wall above has a large offset (Fig. 4). As a consequence, the blind triforium passage formed behind a single arch span is quite shallow. The top windows occupy the most part of the clerestory but apparently they do not compromise the lateral stability of the wall as will be seen below.

The dimensions of the elevation however depend significantly on the vaulting system. Quadripartite ribbed cross vaults of good quality ashlar masonry were used, following the French type bond of laying the units in courses parallel to the edges. The vaults in the nave (Fig. 5a) span over a 5.3 x 10.4 m compartment. The longitudinal vertex is emphasised by a rib that runs along the main axis of the nave, while the vaults next to the transept and the façade are formed by a more complex net of tierceron ribs (Fig. 3). An interesting feature of the transverse vault is that it is stilted, so the corresponding thrusts are applied with an offset from the springing.

(a) Nave vault

(b) Aisle vault

Figure 5. The main vaulting types of the Cathedral

The vaults at the aisles span over a 6.3 x 5.3 m compartment (Fig. 5b). As the transverse vertices however are longer a smooth curve was applied to reduce the deflections.

The transverse thrusts of the vaults are contained using the classic system of double flying buttresses upon large pier buttresses, which are capped by a sculpture functioning as the necessary pinnacle (Bork 1997). At the interior, the springings of the nave vaults meet over a wider area and thrust is spread more efficiently around the head of the rampant arch with the help of rubble filling the conoid pockets behind the spandrel walls.

All the stone units of the Cathedral are made of accurately dressed limestone (caliza de Hontoria) and the joints are very thin and uniform. The load-bearing capacity of the stone masonry appears to be in a very good condition in general. No major faults have been reported in the cathedral itself (Alonso et al 1994) except at the steeples due to an inappropriate use of cast iron reinforcement in 1900 and subsidence at the cloister.

The study of the structural behaviour of the Cathedral can be simplified to that of a typical bay of the nave as there are sufficient transverse structures that function as diaphragms (plates with high in-plane stiffness), isolating parts of the building into areas with an almost uniform stress regime. The third bay from the façade appeared to be quite representative in terms of structural layout and support or even decoration (Fig. 1). The section and plan in Fig. 4 were produced from an in-situ survey, critical analysis of existing measured surveys (Karge 1989, MEC 1983, Velazquez Bosco 1885) and the establishment of a system of proportions that could have been used in the original design (Merino 1993, Karge 1989).

The configuration of the bay as originally designed, while ignoring time-dependent deformations, had to be determined in order to understand the inherent stability of the original project. The analysis of the initial state can therefore indicate areas of weakness or high stress concentration and allow certain present signs of distress to be interpreted.

2.2 *Construction history*

The most important phase in the development of the Cathedral is the foundation and construction of the original design that succeeded a previous Romanesque cathedral. On 20 July 1221 the foundation stone was laid and with some interruptions and minor changes (Karge 1989) the main fabric can be considered as completed by 1277 when Master Enrique, the first known architect of the church, dies.

The next major phase (1446-1568) is characterised by the important work of a well-known family of German master masons (Juán, Simón and Francisco de Colonia). They are responsible for most of the style of the Cathedral as they designed the spires in fine tracery at the west façade, built the crossing tower and redesigned some of the major chapels. The original crossing tower collapsed after a tempest in 1539 and was reconstructed by 1568.

The establishment of the major chapels was concluded in 1736 with that of S. Thecla (behind the north side of the façade). Stylistic changes and conservation works started with the reformation of the façade in 1753 and major works took place in 1885-1900 by R. Velazquez-Bosco and V. Lamperez.

3 BEHAVIOUR OF THE NAVE

A detailed 3D FE model was the main tool for the analysis of the bay. Compared with a 2D model, this analysis permits the visualisation of stresses along the thickness of some critical elements and the assessment of the transfer of loads between the vaults and the lateral walls. At this sage also, only the action of the self-weight of the structure was considered as the application of the other imposed

loads like the wind would be complicated due to the geometry of the buttressing system.

The model was generated with a CAD pre-processor and then solved with the FE program Abaqus. Due to symmetry only a quarter of the bay was examined and 3D "brick" type elements were used. The material was initially considered as linear elastic and orthotropic properties were assigned to the webs of the vaults. Due to lack of specific data, reasonable assumptions were made for the values of the mechanical properties based on rules specified for modern masonry (Hendry 1997) and data on the *caliza de Hontoria* constituent limestone (PINACAL 2004): modulus of elasticity E = 7645 N/mm^2 and compressive strength f_c = 23 N/mm^2.

These values, in combination with a low mortar strength, yield a compressive strength for the masonry normal to bed joint f_{cy} = 6 N/mm^2 while along the bed joint the strength can be considered to be 1/3 of this value, i.e. f_{cx} = 2 N/mm^2 (Hendry 1997). Using an empirical formula for the initial modulus of elasticity normal to the bed joint, $E_{y,0} = 700 \cdot f_{cy}$ = 4.2 kN/mm^2, which should be further reduced to 1/3 due to long term deformations, therefore E_y = 1.4 kN/mm^2. The characteristic flexural strength of the masonry parallel to the bed joints F_{ux} was considered as 0.7 N/mm^2, while normal to the bed joints F_{uy} = 0.2 N/mm^2. Keeping these proportions, $E_x = (0.7/0.2) \cdot 1400 = 4900$ N/mm^2. The unit weight of masonry is close to that of the stone and here it is 24 kN/m^3. These values are summarised in Table 1.

TABLE 1. Mechanical Properties Considered for the Stone Masonry of the Cathedral

Material properties	Parallel to bed joint (x-axis)	Normal to bed joint (y-axis)
Elasticity modulus, E (N/mm^2)	4900	1400
Compressive strength f_c (N/mm^2)	2	6
Flexural strength, F_u (N/mm^2)	0.7	0.2

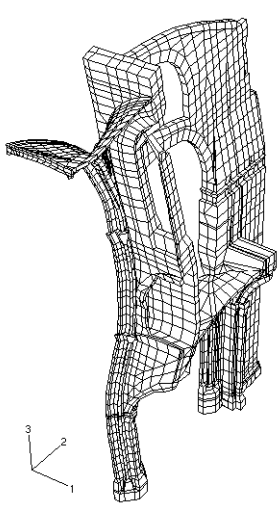

Figure 6. Deformation under self weight of a bay at the nave (magnified 300 times)

A linear elastic analysis was performed under self weight of the materials. The resulting deformation of the FE model (Fig. 6) and the stresses, as will be discussed below, are generally low and this indicates the efficiency of the design of the building, in response to the high compressive strength of the constitutive materials and elements.

The behaviour of the building as results from the FE model will be examined in more detail, by correlating the response to the loads of the two main areas, the nave vaulting system and the aisle. Other aspects that will be discussed are the function of the flying buttresses, the degree the windows or triforium can weakness the fabric, the distribution of thrusts to the lateral walls etc.

3.1 *Upper structure*

The nave vault can be simulated as a parabolic barrel vault running along the main axis of the nave, stiffened at the edges with the deeper transverse vault. As in these proportions the parabolic profile is close to the catenary, the hoop stresses S22 are low, despite the presence of low tension (Fig. 7). The associated high thrusts are balanced by the flying buttresses and the vault is further stiffened by the transverse vault due to its circular profile and stilted arrangement.

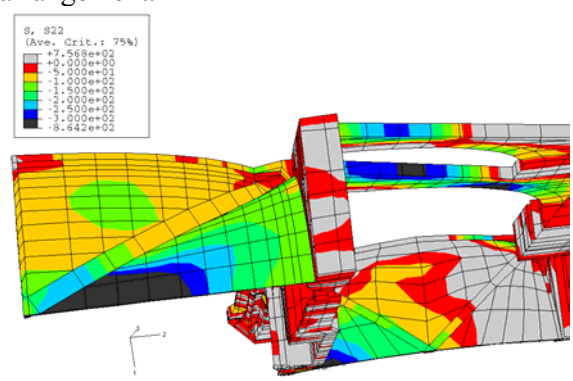

Figure 7. Stresses S22 at the upper surface of the nave (in kN/m^2)

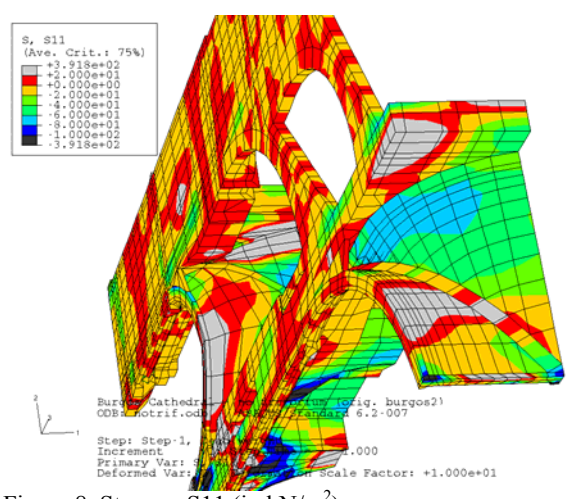

Figure 8. Stresses S11 (in kN/m^2)

However, the central portion of the vault still tends to spread outwards. The stiffer transverse web transmits most of this horizontal deformation onto the wall and tension develops there mainly at the area between the vault and the crown of the clerestory window (Fig. 8). As a consequence, compression results along the straight vertex of the transverse vault, which diminishes closer to the wall.

The relative stiffness of the transverse web is further demonstrated by the mainly compressive low hoop stresses (Fig. 8). The increasing values at the haunches indicate a part of the weight of the vault is indeed supported by the rib and subsequently the springing. The associated thrust twists the longitudinal web, causing longitudinal tension at most part of this area (Fig. 8).

The rib is a quite crucial element in this scheme as it guarantees the good execution of the critical area of the intersection as well as the continuity between the webs (Fitchen 1965). As a consequence, the sharp change in the curvature stiffens each of the adjoining webs but also functions as a barrier against the spread of high stresses between the webs.

To summarise, the symmetrically supported nave cross vault functions as an assembly of four barrel vaults that form a common boundary along the groins (Theodossopoulos et al 2003). The supports in each of these vaults vary in their stiffness between the fixed transverse edge on the wall and the flexible symmetric boundary of the longitudinal webs. The strength of the groin-rib system regulates the degree this assembly functions as a two-way structure. If this system is weak, each web behaves as a cantilever, in accordance with tests in another Gothic vaulting system that have shown that the deflections were increasing towards the keystone (Theodossopoulos et al 2003).

Figure 9. Aspect of the flying buttresses of the nave

So far as the flying buttresses are regarded (Fig. 9), the relatively high hoop compressive stresses at their top edge and tensile stresses at the bottom edge, at the area where they abut the wall, indicate that both the arches perform their function to provide a thrust against that of the nave vault. In this sense, the spandrel fill is essential for the function of the lower arch as it provides a wider area for the application of the thrust. However, the counteraction seems to exceed the thrust of the vaults, especially at the upper tier. It has been demonstrated that the role of this arch is to contain thrusts developing due to wind (Bork 1997), therefore these stresses are expected to reduce reasonably.

3.2 *The lateral wall*

Due to the dynamic equilibrium at the upper zone the horizontal component of the thrust is therefore cancelled and this allows the forces from the upper zone to be transmitted on the lower, triforium area through mainly vertical resultants. Fig. 10 supports the observations at the deformed bay in Fig. 6, as the axial forces are not constant through a section, showing that they are transmitted with an offset through this intermediate zone.

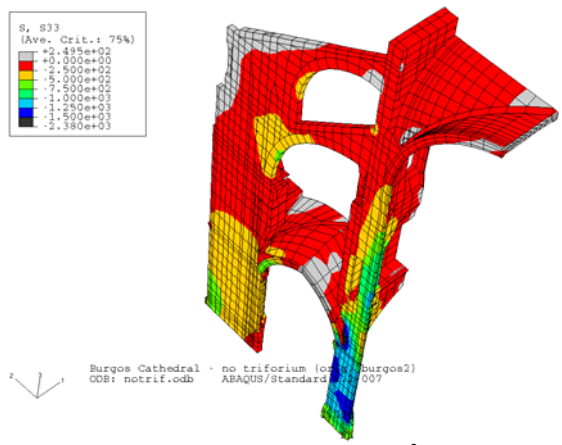

Figure 10. Vertical stresses S33 (in kN/m^2)

The triforium enclosure in itself has not been modelled, except from the arcade, as it is more of a decorative screen with no real load-bearing capacity. The decay observed today in some of the mullions is mainly due to differences in temperature and humidity and every damage should rather be classified as a serviceability defect. Fig. 10 shows also the flow of forces around the triforium arch and the concentration of compressive forces at the extension of the pier. The maximum stresses in this area (1 N/mm^2) are still below the compressive strength.

The transition from the section of the wall at the triforium to the narrower section above the arcade pier is the critical area of the *tas-de-charge* (Fitchen 1961). Apart from the loads of the nave being transmitted through, further difficulties arise from the need to fix the roof that protects the aisle vault (Fig. 9). The configuration of this area was determined by a critical study of similar solutions in other cathedrals (Fitchen 1961) and Velazquez-

Bosco's 1885 survey of other areas of the Cathedral. The FE model shows the sharp increase in the compressive forces around this neck, but the stresses reach a maximum of 2.4 N/mm^2, still below the strength normal to the bed joint.

The behaviour of the lateral wall can also be illustrated by the deformation of the pier extension (Fig. 11), in particular the shaft from the compound pier that runs continuously until the crown (Fig. 5). The diagram ("Dead load") shows clearly the hinge that can be formed at the *tas-de charge*, at 9m height. However, further above, the inward displacement at the middle of the nave arch highlights the effect of the thrust of the flying buttress on the nave, although the values are relatively small. In order to show the proportions of this problem, a comparison is made with the heavily deformed bays of Santa Maria la Vieja, the Old Cathedral of Vitoria (Azkarate 2001). These deformations were recorded through a monitoring programme and are attributed to the lack of flying buttresses from the original project and their belated addition once the nave of that Cathedral had started spreading.

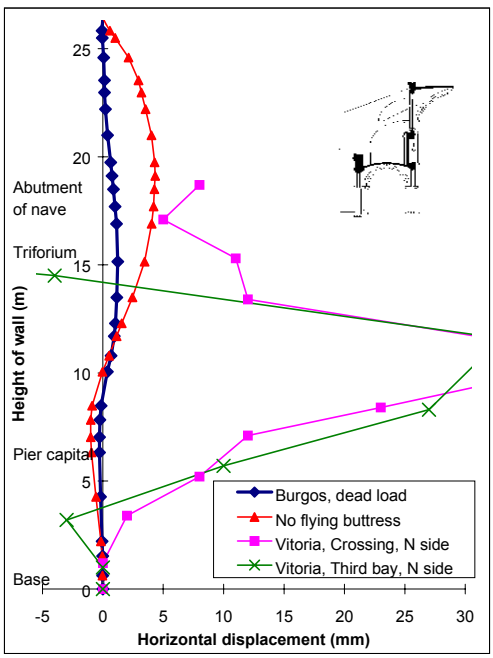

Figure 11. Lateral deformation of the pier extension

3.3 *Lower structure*

The main actions on the aisle vault are the self weight of the masonry and the loads from the upper structure applied asymmetrically over the arcade only. The vault is contained between the pier arcade and the pier buttresses and therefore the lateral thrusts are not contained in the same symmetrical manner as in the nave vaults. Both vertices are curved and this domical pattern was probably chosen intuitively to reduce bending as will be discussed later.

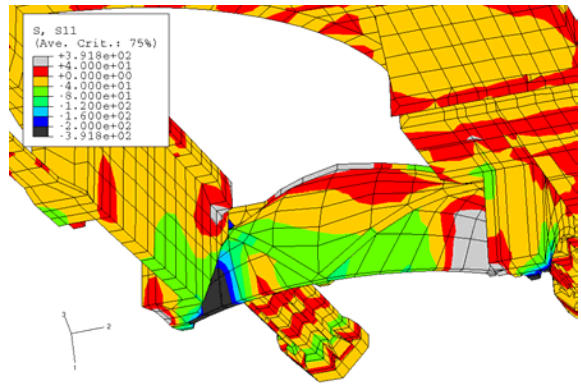

Figure 12. Stresses S11 at the extrados of the aisle vault (in kN/m^2)

The nave arch and its immediate area receive directly the load from the upper structure through the tas-de-charge. As a consequence, the arch is fully stabilised and develops mainly high compressive stresses in the hoop direction (max. 0.39 N/mm^2), which are anyway below f_{cy} (Fig. 12). This load twists the vault and pulls it from the adjacent vault along the transverse edge, making the increase the transverse rib causes in the stiffness of the edge quite important.

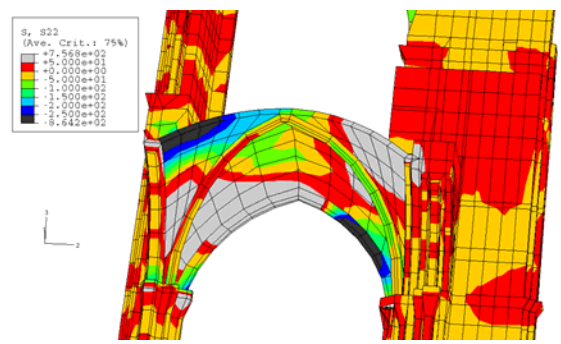

Figure 13. Stresses S22 at the intrados of the aisle vault (in kN/m^2)

This torsion affects the entire vault and a similar pattern develops in the other direction as well (Figs. 7 and 13). If the two patterns are examined together, an in-plane, shear deformation appears to develop due to the torsion of the vault. This rotation can be associated with the deformation of the upper structure due to the apparently excessive thrust from the flying buttresses observed earlier. The lateral wall seems to have sufficient stiffness to transmit most of this deformation directly on the aisle.

The domical layout of the aisle vault has not only reduced the deformations but also enhanced the two-way behaviour of the vault, making the loads to flow into the entire area of the supports. The diagonal ribs are now more organically bonded into the webs and, since the groins are smoother, their function is now mainly to increase the depth of the section and therefore reduce the deformations (Fig. 13). The tensile forces however are close to the modulus of rupture F_u and cracks are anticipated now on the apex of the edge of the vault on the wall and at the

front haunches. Another benefit of this pattern is therefore that the cracks along the longitudinal vertex that develop in vaults with straight vertices (Theodossopoulos et al 2003) are not expected to affect this vault. A closer inspection of the aisle vaults (Fig. 5b) shows that there have been repairs at the past mainly at these parts at the haunches. No serious problems have been reported however so it seems that the action of the wind has activated the flying buttresses and consequently reduced the torsion of the bay.

The combination of the in-plane rotation of the vault with the eccentrically transmitted vertical loads of the upper structure is somehow beneficial for the arcade pier that ultimately supports these loads. Despite the relatively higher compressive stresses, the variation of the vertical loads in the section is not strong and they appear to be transmitted axially (Fig. 10). The attached shafts and the base of the pier reduce the stresses, which stay in any case below f_c. So far as the pier buttress is regarded, the weight of the pier has managed to reduce significantly the overturning effect of the thrusts from the flying buttresses and the aisle. The bending stresses due to the offset with which these forces are transmitted to the ground are contained within the limits of strength (Fig. 10).

4 FUNCTION OF THE MAIN LOAD-BEARING ELEMENTS

The behaviour of some of the major elements of the structure like the vaults and the buttresses will be modelled in detail and compared with the major FE model. In this manner, the effect of the boundary conditions and loading of the entire structure upon these elements will be assessed, allowing better-validated and less computationally demanding models to be generated for specific problems.

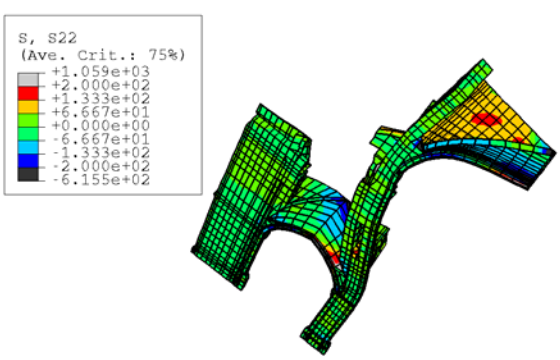

Figure 14. Stresses S22 at the structure after the flying buttresses have been removed (kN/m^2)

Using the same FE model, the role of the flying buttresses and the successive construction phases of the building have been studied by removing both tiers (cf. Croci et al 1995). The resulting stress pattern (Fig. 14) is characterised by the high compressive stresses developing in a wider area of the nave vault, while the transverse web in this area is now in tension. The aisle vault still follows the deformation of the nave with an in-plane twist, but now the shear forces effect is more pronounced in one direction as wider section of the transverse vertices are now involved.

The lateral displacement of the bay has increased as the higher inward deformation of the nave vault shows in Fig. 11. The values however do not compare with the magnitude of the deformations observed in Vitoria Cathedral, which must have been affected by time-dependent decay of the joints and the incorrect location of buttressing systems once the deformations became appreciable.

The range of horizontal displacements that is expected to develop at the springings of a vault can be assessed from available survey data on the deformation of the lateral walls of some cathedrals.

TABLE 2. Movement of abutments observed in historic masonry cross vaults

Case	Spread *u* (mm)	Span *s* (m)	u / s
Vitoria Cathedral	174	9.24	1/53
id	120	5.5	1/46
Amiens Cathedral (Bilson 1906)	50	7.52	1/150
Beverley Minster (Bilson 1906)	-		1/19
Holyrood Abbey	75	3.77	1/50
León cathedral (Martínez 2000)	150	7.5	1/50
FE model (Jagfeld 2000)	404	10	1/25
St. Martin, Landshut (*id*)	116	11.6	1/100

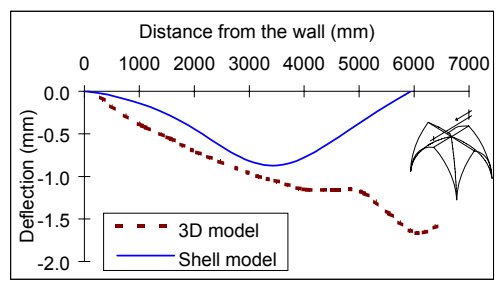

Figure 15. Deflection of the vertex of the aisle

Another interesting area to be investigated is the behaviour of the cross vaults. The aisle cross vault was modelled separately in more detail and both the webs and the ribs and arches were generated by shell elements, following a method outlined in (Theodossopoulos 2003). At this model, the nave arch is sufficiently stiff to provide an almost symmetrical support to the vault. The original conditions of the building, i.e. the weight from the upper structure on the arcade, as was also mentioned there, change completely the deformation of the front area of the vault (Fig. 15), with the nave arcade at the front deflecting the most.

Visually, the Cathedral is unified by the system of the ribs, among others. Eventually, the whole system of arches would serve to emphasise and

define the roles and hierarchy of the main vaulting components (Stalley 1999), to improve and stiffen complicated construction details and to further concentrate the loads into more reliable paths to the ground, permitting large openings instead of massive lateral walls (Acland 1972).

5 CONCLUSIONS

The 3D FE model of a bay from the nave of Burgos Cathedral demonstrated the success of the original design of the building as the stresses were overall below the strength of the constituent materials and the deflections were quite low. Continuity of the fabric is a fundamental assumption in this model, therefore an update of the support conditions of the main load-bearing elements using survey data might be required at a next stage in order to improve the knowledge of the behaviour and safety of the building.

The quality of the original design is also evident in the load-bearing elements of the Cathedral as they proved to make an efficient use of the strength of the stone masonry in compression while reducing tensile stresses to a minimum possible.

The model highlighted the interaction between the upper and lower structure as well as the factors that influence the flow of stresses towards the supports. The rib plays a key role in the execution of the difficult intersection between the webs and the efficient distribution of the loads within the entire vault according to a two-way pattern.

Further on, this analytical model can gauge the application of more simplified modelling strategies for the building or parts of it, while more complex loading conditions or the origin of some more specific defects can be examined.

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