ASSESSMENT OF STRUCTURAL DAMAGE IN HISTORICAL CONSTRUCTIONS USING NUMERICAL MODELS: THE CASE OF THE CHURCH OF THE POBLET MONASTERY

Savvas Saloustros, Luca Pelà, Pere Roca, and Jorge Portal

[1] Universitat Politècnica de Catalunya
Jordi Girona 1-3 08034, Barcelona, Spain
e-mail: savvas.saloustros@upc.edu
luca.pela@upc.edu , pere.roca.fabregat@upc.edu , jportal@coac.net

Keywords: Masonry, Diagnosis, Finite Element Analysis, Continuum Damage Model, Sequential Analysis, Graphic Statics.

Abstract. *This paper presents the assessment of the structural damage and stability of the church of the Royal Monastery of Santa Maria de Poblet, one of the UNESCO World Heritage sites in Spain. The structure presents damage affecting mostly the main nave and lateral aisles, including cracking in the vaults and significant deformation in the clerestory walls and the barrel vault of the main nave. Following the historical survey, a sophisticated finite element model was used for the structural analysis of a damaged bay of the church. The 3D model was developed on the basis of the results of a terrestrial laser-scanning survey. In this way, the current deformed state of the structure was taken into consideration in the performed analyses. A continuum damage model allowed a realistic representation of the masonry behavior under tension and compression, which is a key factor for the assessment of masonry structures. The simulation of past reported or possible actions i.e. earthquakes, structural alterations and settlements provided valuable information on the causes of the present deformation and damage of the church. The influence of the material parameters in the structural response was evaluated with a detailed parametrical analysis. The paper ends with the graphic statics solution for the structure and a comparison with the equivalent finite element analysis outcomes.*

1 INTRODUCTION

Historical events or alterations having affected ancient structures may significantly alter their original equilibrium and stress condition. Therefore, analysis on such structures should consider and integrate the most meaningful events occurred in the history of the structure. This is particularly important in the case of structural alterations having caused significant changes on the geometry or on the structural arrangement of the building.

The study of historical constructions can benefit from the combination and integration of different approaches such as (a) historical survey, (b) inspection, including non-destructive testing (NDT) and minor-destructive testing (MDT), (c) monitoring and (d) structural analysis [1], [2]. However, before carrying out comprehensive inspection and monitoring, an initial structural analysis, even with limited information on the materials and morphology of the structure, may be useful as a way to obtain a deeper understanding of the structure and allow a more accurate design of the inspection and monitoring programs. Even in these cases, the analysts should take advantage of available historical information in order to take into account the influence of the meaningful alterations experienced by the structure.

This study presents the structural analysis of the church of the Poblet Monastery, located in the municipality of Vimbodí and Poblet in Spain, using numerical and graphic statics analysis. The developed finite element model includes the present deformation of the structure derived from the use of a terrestrial laser-scanning survey. The numerical modelling of the various events that have affected the structure, as derived from a historical survey, has been important to identify the possible causes of the existing damage. Parametric analyses have shed light on the influencing material parameters and validated the high importance of the geometry for the equilibrium of masonry structures. This study is the prelude for the organization of the necessary inspection and monitoring activities, which, along with detailed structural analysis, should contribute to an accurate diagnosis and definition of the necessary intervention.

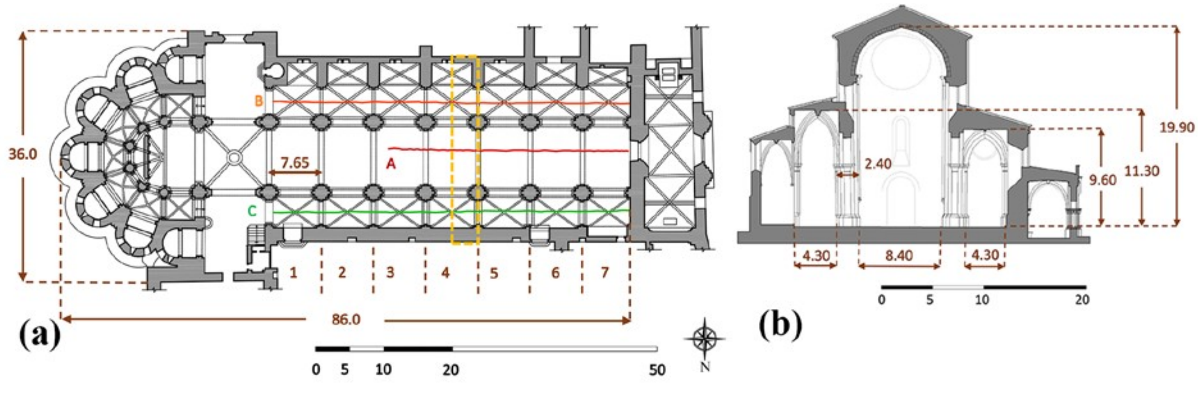

Figure 1: The church of the Poblet Monastery: (a) plan at the roof level, with the numeration of the bays and indication of the main cracks, and (b) cross-section of a typical bay.

2 THE CHURCH OF THE POBLET MONASTERY

2.1 Description of the structure

The church of the Poblet monastery follows a Latin cross shape with overall length of 86 m and width of 36 m (see Figure 1). The structure resembles a typical example of the Cistercian architecture, which is based on the Romanesque style but introduces also some Gothic features. An example of this synthesis are the pointed barrel vaults used in the main nave and southern and northern part of the transept, as well as the pointed groin vaults, which roof the northern lateral aisle, the ambulatory and the crossing. The southern lateral aisle, as well as the adjacent chapels, was constructed in a Gothic style due to their later addition in the 14th century, as described in section 2.2.

The masonry is composed of limestone, largely available at the local quarries. Regarding the morphology and internal composition of the structural elements of the church, little information is up to date available. An in-situ inspection on the roof in 2012 [3] revealed that the deep infill of the barrel vault is composed by a layer of soil material (0.90 m) and an overlying layer of lime mortar mixed with stone fragments (0.65 m), both showing low mechanical characteristics. The type of the filling over the vaults of the lateral aisles is still unknown. No information is available either on the morphology of the walls of the church; however, a comparison with similar structures, including other buildings within the monastery, suggests that they are three-leaf walls with an interior rubble filling.

Today, the church presents damage mainly in the western part including the main nave and the lateral aisles. The bays affected by past fires present stone deterioration with significant loss of material. Additionally, the corrosion of the large amount of metal elements inserted in joints and open cracks during interventions after year 1940 accelerates the stone degradation. Apart from this material deterioration, the structure presents significant deformation and structural damage. The clerestory walls exhibit an outwards rotation, which reaches values of 14 cm in the southern one and 10 cm in the northern one. This deformation is possibly due to the inadequate equilibrium condition caused by the low height of the lateral aisles, as can be seen in Figure 1b. The barrel vault has a significant deflection progressing from the 1st and 7th to the 4th bay (see Figure 1a for the numeration of bays) and reaching values of 50 cm (Figure 2a). In addition to this deformation, the barrel vault presents a wide crack near its key (crack A in Figure 1a), which continues longitudinally across almost all the bays of the church (Figure 2b). Cracking of the stone is also evident near the key of the vaults of the lateral aisles (cracks B and C in Figure 1a), which is accompanied by relative vertical displacements of 5 ÷ 7 cm (Figure 2d-e). Finally, local stone crushing is visible at the middle height of the arches in the main nave (Figure 2c).

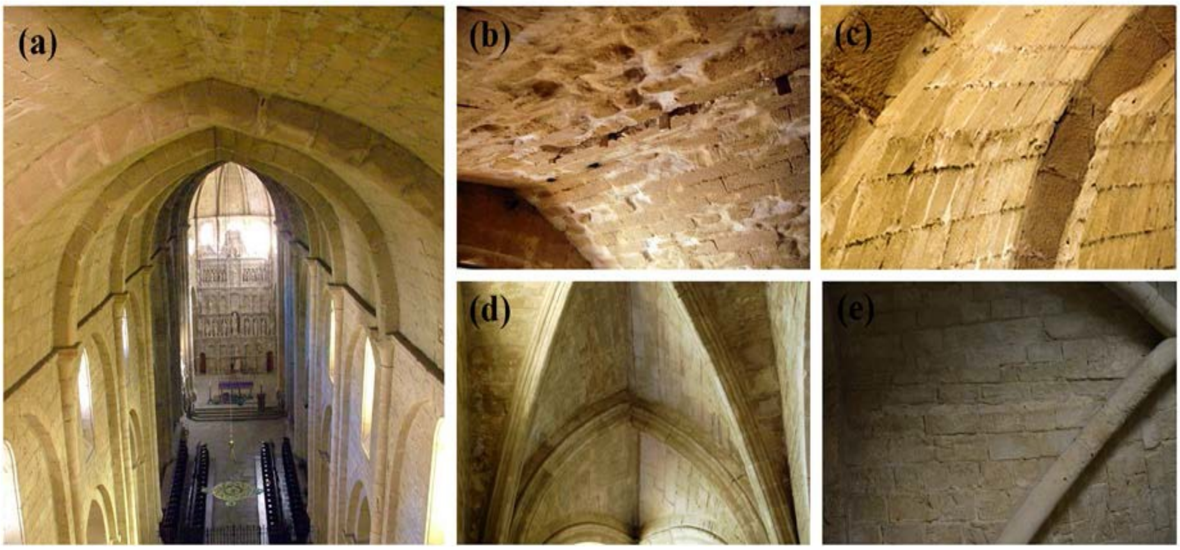

Figure 2: Deformation and damage in the main and lateral aisles of the church: (a) deformation in the arches and barrel vault of the main nave, (b) cracking in the barrel vault, (c) crushing in the arches of the main nave, (d) cracking in the vault of the southern aisle, and (e) repaired cracking in the vault of the northern aisle.

2.2 Historical survey

The construction of the church of the Royal monastery of Santa Maria de Poblet started in 1170, soon after the establishment of the monastery in 1153. The erection of the east end, including the apse, the ambulatory and its radiating chapels, was completed in 1196. The transept followed and the main and lateral naves were constructed afterwards, concluding the main body of the church in 1276. The Galilee, located at the entrance to the church from the west, was finished by the end of the 13th century. Finally, the "cimborio" over the crossing in the transept is an addition of the 15th century.

An important structural alteration in the church was the demolition and reconstruction of the southern aisle between 1316 and 1348. Unfortunately, no information is available on the causes that motivated this action. It is interesting, however, that before Poblet, a similar action occurred in another Cistercian abbey, that of Fontfroide of France. In both churches, chapels were added adjacent to the new lateral aisle. The use of chapels by noble families of the era might have benefitted the monastery with additional income, suggesting that the reasons for these alterations might not necessarily have been related with structural problems.

In addition to the latter architectural alteration, physical actions have also affected the church of Poblet monastery. In particular, various earthquake events have occurred in the region of Catalonia during the life of the structure [4]. More specifically, an earthquake event in year 1792 is reported to have affected buildings of the monastery such as the church and the adjacent dormitory [5]. The addition, between 1823 and 1830, of three flying buttresses in the southern part of the church aligned with the 4th, 6th and 7th arches of the main nave and a buttress bracing the southern part of the façade, was probably carried out to restrain the propagation of existing damage and deformation.

Finally, a certainly disastrous era for the structure came after the confiscation of the ecclesiastical property by the Spanish government in 1835. The monastery suffered various fire events and significant deterioration due to abandonment until its restoration in 1940 [5], [6].

3 NUMERICAL ANALYSES

3.1 Analysis lay-out

The analysis faces some uncertainties due to insufficient knowledge on the morphology of the structure (for instance, the nature of the fillings over the lateral vaults) and the mechanical properties of the materials. These uncertainties have been addressed by carrying out analyses according to different hypotheses.

The masonry of the walls, as well as that of the arches and the vaults, has been modelled as a single homogeneous material in all the performed analyses. Its mechanical properties, (material A, Table 1) have been estimated based on experience on similar masonries and by considering the recommended values of [7] and [8]. Regarding the infill of the barrel vault, a different material has been used (material B in Table 1) due to the substantial different properties comparing to the masonry, as described in section 2.1. Taking into account the nature of this infill, the values adopted for the corresponding mechanical parameters are significant lower than the ones for the masonry. For both materials the compressive fracture energy has been considered equal to Gf_c = 50000 J/m^2, whereas the adopted tensile fracture energy has been Gf_t = 200 J/m^2.

Table 1 Material parameters adopted in the numerical analysis

Material	γ (kg/m^3)	E (MPa)	ν (−)	f_c (MPa)	f_t (MPa)
A	2200	2000	0.2	4.00	0.20
B	1800	25	0.2	0.5	0.025

Due to the aforementioned uncertainties in the material properties, resulting from the lack of experimental testing, a parametric analysis with the variation of the most influential properties has been carried out and is presented in section 3.8. In order to investigate the influence of the unknown type of infill over the vaults of the lateral aisles, two alternative cases have been considered and compared. In the first case, the filling of the lateral vaults is assumed to consist of good quality masonry with the same mechanical properties defined for the limestone of the vaults (material A). This model, labelled as "reference model", has been used to study the performance under self-weight (section 3.3), the effect of the past structural alterations (section 3.5), earthquakes (section 3.6) and soil settlements (section 3.7). In the second case, the infill over the lateral vaults has been modelled with the same properties of the infill of the barrel vault (material B). This second model has been used to analyze the performance under self-weight (section 3.4).

3.2 Finite element model

The FE model utilized for the present analyses has been defined with the real deformed geometry of the structure in order to take into account the influence of the existing deformation on the stability and structural performance.

The acquisition of the present geometrical state of the church has been possible with the use of the results of a terrestrial laser-scanning survey. The FE model has been developed having as a reference the cloud of points obtained by the laser-scanning survey and includes bay number 4 (see Figure 1). This bay presents the highest deflection in the barrel vault and lacks the support of a flying arch at its southern part, increasing in this way its potential vulnerability. The effect of the adjacent bays has been taken into account with appropriate boundary conditions representing the symmetry in the longitudinal direction of the nave. The FE model includes a bay of the cloister, adjacent to the northern aisle, to take into account its possible interaction with the church.

The continuum damage mechanics constitutive law presented in [9] has been adopted for the description of the nonlinear material behavior. This model benefits from the use of two scalar damage variables, one for tension (d^{-}) and another for compression (d^{+}), with their values varying from 0 for intact material and 1 for completely damaged material. In this way, cracking or crushing of the stone, due to tension or compression respectively, is easily visualized in the numerical analysis results. GiD software [10] has been used for the pre- and pro-processing and COMET solver [11] for the analysis, both of them developed at CIMNE in Barcelona.

3.3 Analysis under self-weight (reference model)

The deformation levels obtained for the sole action of the self-weight are low in comparison with the overall deformation that the structure has experienced during its life. In particular, the numerical model predicts a maximum deflection of 10 mm near the key of the barrel vault. In addition to the instantaneous application of the self-weight, the real deformation is also the result of other causes such as centering deformation and mortar settlement during construction, structural alterations and long-term creep and damage, among other possible ones.

Concerning the stress-state of the bay, tensile damage, and thus cracking of the material, is clearly visible in the intrados of the arch in the main nave and the intrados of the arches and vaults in the lateral aisles (Figure 3). This result is in very good agreement with the damage observed in the real building, which suggests that the cause of this damage is actually related to the design of the structure and particularly to the low position of the lateral vaults with respect to the height of the central nave.

Finally, the numerical analysis does not predict any crushing of the stone in compression for the assumed value of the compressive strength. Among the highest compressed areas—with decreasing magnitude—are the northern springing of the vault in the southern aisle (left aisle in Figure 3), the intrados of the arch in the northern aisle (right aisle in Figure 3), the base of the piers, and finally the intrados of the arches in the barrel vault. Consequently, self-weight alone does not seem to produce the crushing of stone in the arches of the main nave and therefore other possible causes for this damage are investigated in the following sections.

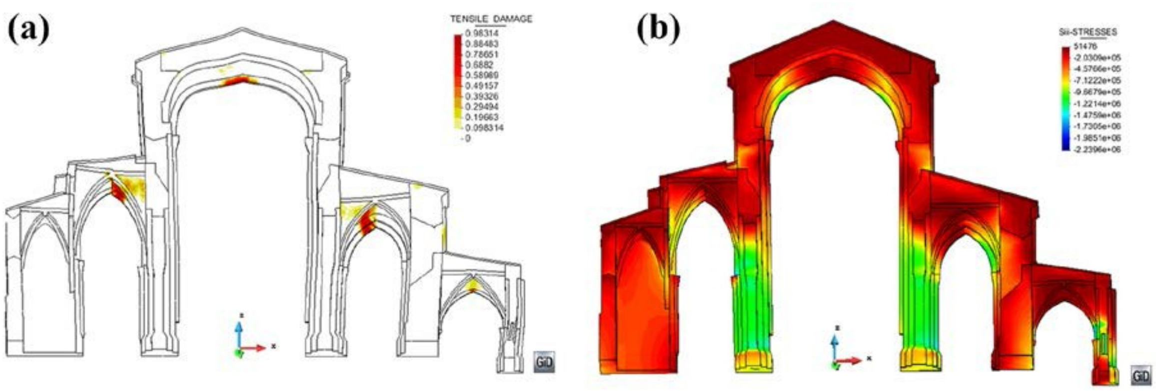

Figure 3: Numerical analysis results for the action of the self-weight: (a) tensile damage and (b) compressive stresses.

3.4 Analysis under self-weight with poor material in the infill of the lateral vaults

Due to the uncertainty regarding the type of material above the infill of the lateral vaults, an alternative analysis has been carried-out by considering that it consists of the same poor material as for the infill of the central barrel vault, as described in section 3.1.

This analysis shows that an infill with low structural capacity would result to significant tensile damage in the barrel vault of the church (Figure 4). The predicted level of damage is inconsistent with the damage actually existing in the structure, suggesting that the infill above the lateral vaults is probably composed of a structurally more competent material. However, future in-situ inspection is necessary to identify and characterize the type of existing infill material and allow new analyses with a consistently updated model.

The reason for the higher damage, in case of a poor infill over the lateral vaults, is found in the lowering of the counteracting thrust of the lateral aisles. When a structural infill (material A) is assumed above the lateral vaults (reference model, Figure 5a) the thrust of the lateral aisles counteracts the one coming from the main nave at a higher position compared to the case with non-structural infill (material B) (Figure 5b). This difference leads to higher eccentricity of the thrust line within the width of the clerestory walls and thus to higher wall rotation and damage in the main nave.

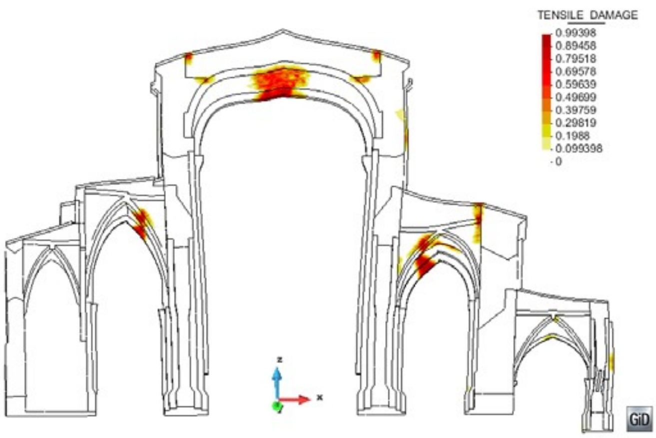

Figure 4: Tensile damage for the model with non-structural infill over the lateral vaults.

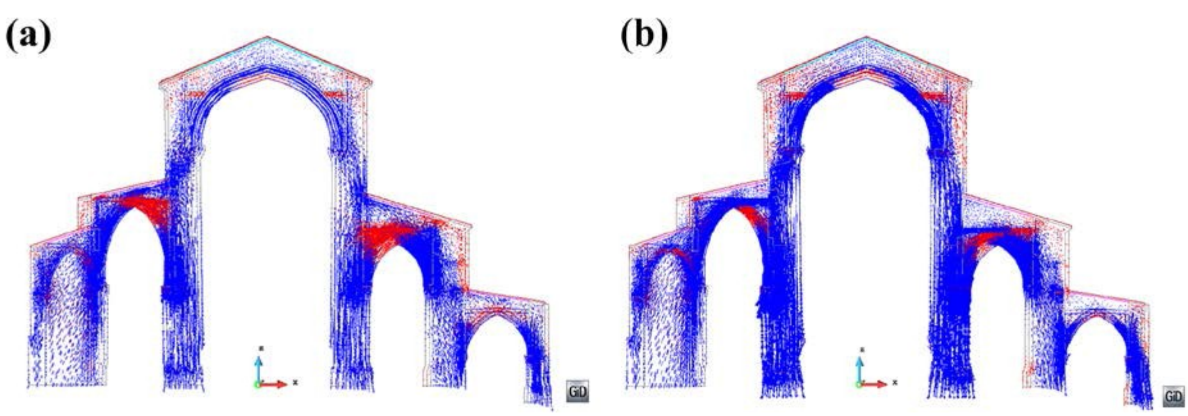

Figure 5: Vectors of principal stresses (tensile in red, compressive in blue) for: (a) reference model with structural infill (material A) over the lateral vaults, and (b) model with non-structural infill (material B) over the lateral vaults.

3.5 Effect of the demolition of the southern aisle

Looking to a better understanding of the possible effects of the alteration of the 14th century on the structure's stability, a specific numerical analysis has been performed. This analysis aimed to investigate the possible scenario of the demolition of one lateral aisle without the use of any auxiliary devices. For this reason, a different finite element model has been prepared, which includes one bay of the southern half of the church and two adjacent bay quarters (Figure 6a). Appropriate boundary conditions have been defined to consider the effect of the neighboring bays. The analysis is performed in two steps: firstly, self-weight is applied to the whole model and secondly, one lateral bay (green part in Figure 6a) is deactivated to simulate in this way the demolition. This sequential analysis has been possible with the use of the numerical strategy presented in [9].

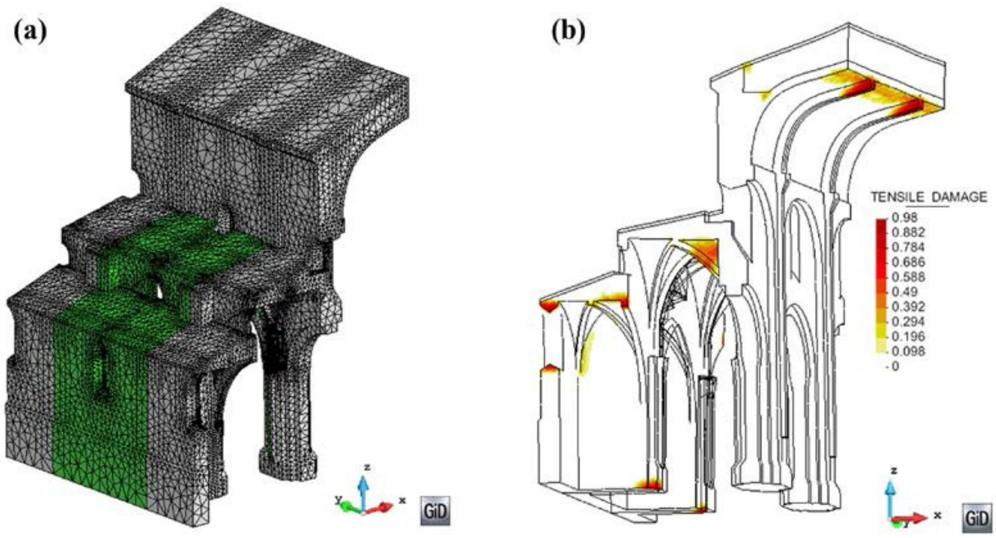

Figure 6: Simulation of the demolition of the southern aisle: (a) FE model (in green the demolished part), and (b) tensile damage contour.

The analysis provides valuable information on the possible influence of such demolition on the present damage and deformation of the church. In particular, the demolition of the lateral aisle causes a higher rotation of the southern clerestory wall and a higher deflection in the barrel vault with an increase of 430% and 30% respectively comparing to the initial state. This deformation is an immediate consequence of the deprival of the counteracting thrust from the lateral aisle. In fact, in the present condition the structure shows a larger rotation in the southern clerestory wall than in the northern one. The deflection of the central barrel vault may have also increased due to this demolition. This deformation entails greater tensile damage near the key of the barrel vault as Figure 6b illustrates. In this case, tensile damage is clearly visible in the barrel vault, which reveals that the present cracking in this area was probably initiated or extended during this demolition.

3.6 Effect of past earthquakes

The possible effect of past earthquakes in the structure has been assessed with the use of nonlinear static analysis (pushover). Due to the different geometry of the lateral aisles, two analyses have been performed with the seismic forces oriented towards the south and towards the north. The obtained tension and compression damage distribution is mostly symmetrical in the two cases.

The distribution of the damage suggests two potential collapse mechanisms: (1) the local collapse of the barrel vault in the main nave, and (2) a global collapse including the main nave and one lateral aisle. The final state of the tensile and compressive damage distribution, according to performed the pushover analysis (Figure 7), shows that the second collapse mechanism is the more likely one.

Interestingly, the cracking and stone crushing in the intrados of the main and lateral naves predicted by the analysis is in good agreement with actually existing damage in the bays of the church. In particular, the numerical analysis indicates that an earthquake event could have initiated or extended the wide cracking near the key of the barrel vault (Figures 7a,c), producing at the same time crushing of the stone in the arches of the main and lateral aisles (Figures 7b,d).

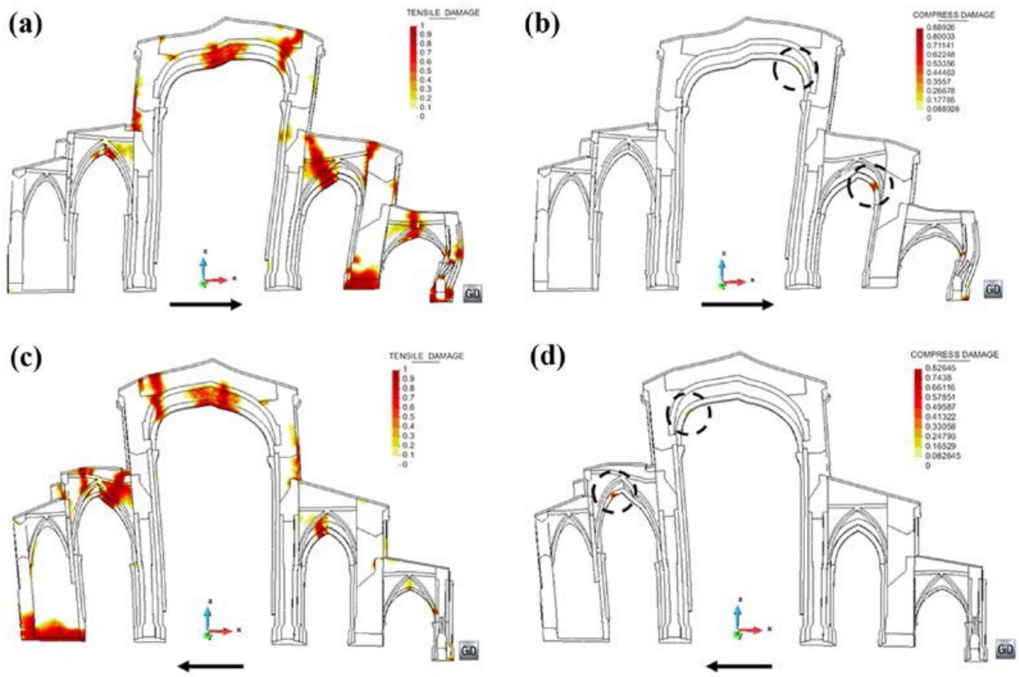

Figure 7: (a) Tensile damage and (b) compressive damage contours for seismic forces towards north; (c) tensile damage and (d) compressive damage contours for seismic forces towards south.

3.7 Effect of soil settlements

The analysis for self-weight shows that the sections at the base of the piers and at the base of the external walls are subjected to very different compressive stress levels (Figure 3b). As should be expected the compressive stress values are significantly higher at the base of the piers. For this reason, and due to the fact that no experimental information is available on the type of subsoil or the foundations of the church, three scenarios have been investigated regarding the simulations of possible soil settlements. These scenarios include the settlement of (a) the southern pier, (b) the northern pier, and (c) both piers of the main nave. The analysis has involved two steps corresponding to firstly, the application of self-weight and secondly, the simulation of the settlement by imposing small vertical displacements at the base of the piers.

The results regarding tensile damage (Figure 8a) show that a moderate settlement (20mm) of one or both piers of the main nave will affect mostly the lower parts of the church. Specifically, the cracking near the key of the vaults in the lateral aisles is the first to propagate, accompanied with cracking in the nave walls. The latter damage, or possible related repair interventions, is not visible today, suggesting that soil settlements should not have occurred in significant values.

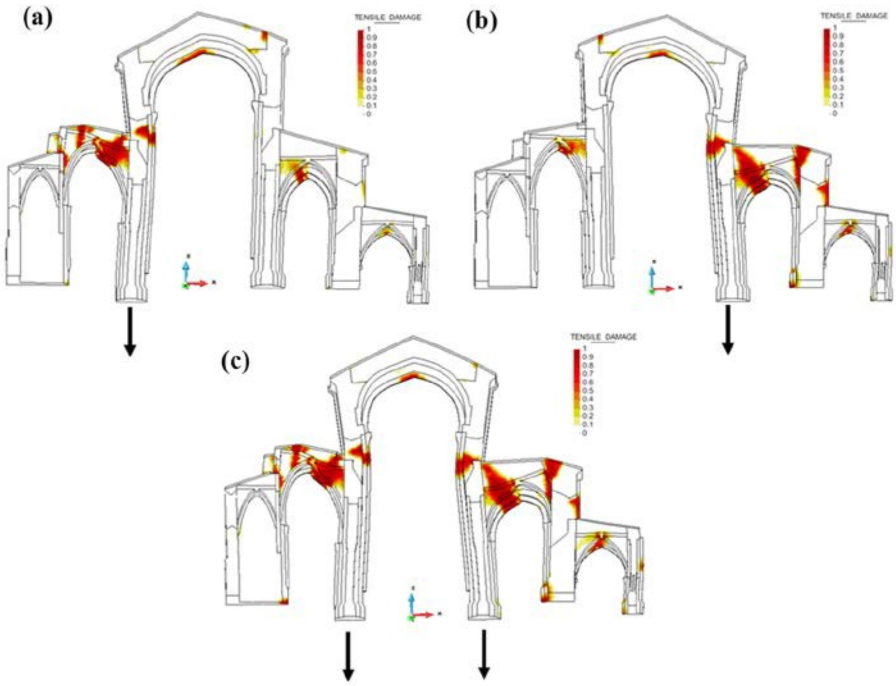

Figure 8: Tensile damage for: (a) settlement of the southern pier (35 mm), (b) settlement of the northern pier (24 mm) , and (c) settlement of both piers of the main nave (25 mm).

3.8 Parametric analyses on the material paràmetres

Parametric analyses have been performed to assess the influence of the adopted material parameters in the structural stability of the church. Analyses have been carried-out for self-weight for a variation of the Young modulus ($E = 250 \div 1000 \cdot f_c$) and the tensile strength ($f_t / f_c = 1/80 \div 1/20$). The tensile fracture energy (Gf_t) is defined in proportion to the tensile strength, with values varying between 50 J/m^2 and 200 J/m^2. The same parameters and variation has been used for the parametric analysis against an earthquake action. Note that in the performed analyses, the non-investigated parameters keep their values as defined in the reference model.

In the analysis for self-weight, the variation of the material parameters influences mostly on the damage intensity. The damage location and distribution are not significantly modified. The decrease in the tensile strength and fracture energy, produces, as expected, greater damage. The influence of the Young modulus is limited and its influence on the damage intensity stabilizes for $E \geq 650 \cdot f_c$ In none of the studied cases concerning the application of only self-weight the stability of the structure was compromised.

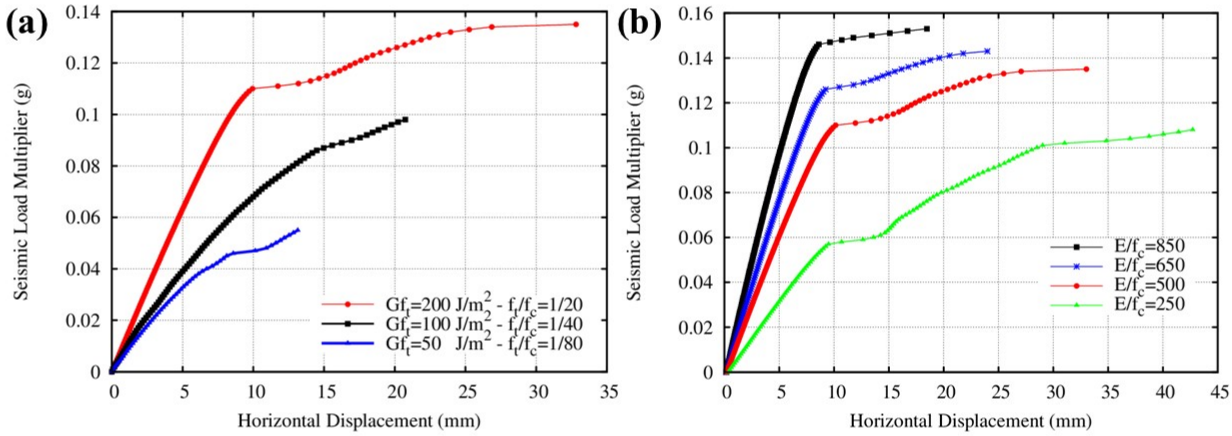

Figure 9: Capacity curves obtained for the variation of the: (a) tensile strength (ft) and fracture energy (Gft), (b) Young modulus (E). Control point at the key of the barrel vault.

The variation of the material parameters affects significantly the capacity of the structure against earthquake actions. In particular, the parametric analyses show a variation of the capacity between 0.058 g and 0.16 g (Figure 9). The lowest capacity is obtained for an almost null tensile strength (f_t / f_c = 1/80 = 0.05 MPa) and low fracture energy (Gf_t = 50 J/m^2). The low capacity obtained in this case, which suggests that excessive damage is possible even for low intensity earthquake events, advices a further investigation on the material properties of the existing masonry. For the rest combinations of the analyzed values, the structure shows a sufficient capacity for the earthquake demand of the area [12].

The influence of a variation of the compressive strength (f_c) has been also investigated for values between 1 and 4 MPa. The specific results show that very low values (below 2.0 MPa), which in principle are not expected for the type of existing masonry, would significantly alter the response and the capacity of the structure. The reader is referred to [13] for more detailed results on the uncertainty of the material and structural parameters and their effect in the structural capacity against seismic actions for the church of the Poblet monastery.

4 COMPARISON OF GRAPHIC STATICS AND FEM

In arched masonry structures, and when the aim of the study is the assessment of the structural stability, graphic statics constitutes a powerful tool normally requiring less effort than the preparation and use of a numerical model. In the current study, a comparison has been carried out between the results obtained for dead load with graphic statics and with the numerical model.

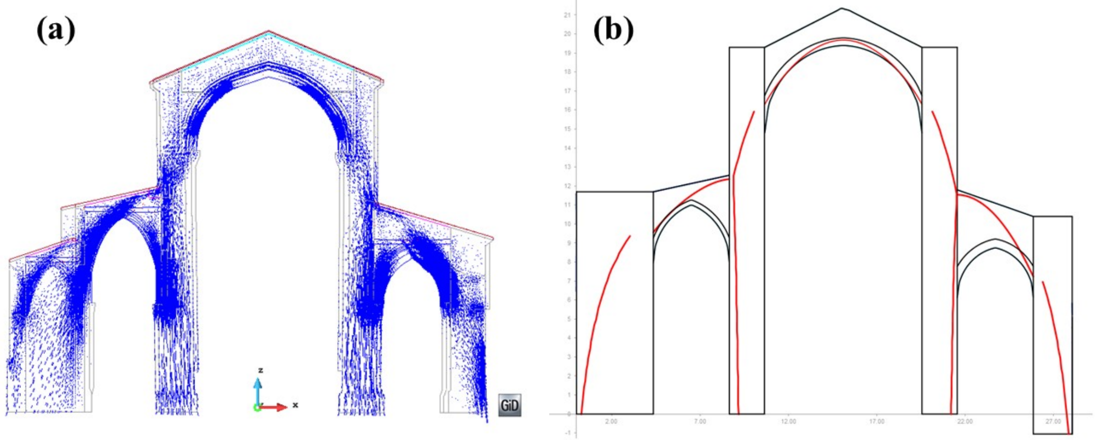

Figure 10: Comparison of the FE with the graphic statics: (a) vectors of minimum principal stresses from FEA and (b) equivalent thrust lines solution.

In order to allow a direct comparison, a numerical analysis has been performed following assumptions similar to those of the graphic statics, i.e. infinite compressive strength and null tensile strength.

Figure 10a presents the results of such an analysis in terms of minimum principal stresses. These results can provide guidance for selecting the position and value of the thrust in various locations within the analyzed bay and consequently investigating if an equivalent graphic statics solution is possible. The graphic statics solution, derived with the use of an excel spread-sheet (Figure 10b) is in very good agreement with the numerical results and clearly shows the appearance of very eccentric thrust lines within the depth of the clerestory walls due to the limited height of the lateral vaults. As mentioned, this eccentricity can explain the outwards rotation of the clerestory walls and the cracking in the barrel vault. In turn, the high position and shape of the thrust lines in the lateral vaults explains the cracking experienced by them.

5 CONCLUSIONS

This paper presents the assessment of the structural stability and damage of the church of the Poblet monastery by means of advanced numerical and classical analysis tools. Important events occurred during the life of the structure and identified thanks to historical research were investigated with the use of the finite element method. The developed finite element model includes the current deformation of the structure derived from the use of a terrestrial laser-scanning survey. The performed analyses have contributed to a better understanding of the causes of the damage and deformation shown by the structure.

The structural analyses point out the necessity for improving the knowledge on the morphology of structural elements and on the mechanical properties of the materials used in the church. Without this information, the analysis has required a number of assumptions regarding the type of infill above the lateral vaults, the interior morphology of the masonry walls and the adopted material properties. Information available on these aspects will permit in the future new analysis based on an updated model. Monitoring of cracks and displacements in the structure would be also advisable as a way to analyze their possible stabilization or progress.

ACKNOWLEDGEMENTS

The authors wish to thank the Comunitat Cistercenca del Monestir de Santa Maria de Poblet for the information and support provided.

This research has received the financial support from the Ministerio de Educación y Ciencia of the Spanish Government and the ERDF (European Regional Development Fund) through the research project MICROPAR (Identification of mechanical and strength parameters of structural masonry by experimental methods and numerical micro-modelling, ref num. BIA2012-32234).

REFERENCES

[1] ISCARSAH Committee, ICOMOS: Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage, 2003.

[2] ISO/TC98. ISO/FDIS 13822: Bases for design of structures – assessment of existing structures, Genève, 2010.

[3] ARCS Patrimoni Cultural SLL. Memòria de la intervenció arqueològica al Reial Monestir de Santa Maria de Poblet. Coberta Església Major (Vimbodí i Poblet, Conca de Barberà), 2012, (in Catalan).

[4] E. Fontserè, Recopilació de Dades Sismiques de les Terres Catalanes entre 1100 i 1906, 1971 (in Spanish).

[5] J. Bossegoda i Nonell, Historia de la Restauracio de Poblet : destrucció i reconstrucció de Poblet, *Abadia de Poblet*, 1983 (in Spanish).

[6] L. Domènech i Montaner, Poblet, *Montaner y Simon*, Barcelona, 1916

[7] P.I.E.T 70, Prescripciones del instituto Eduardo Torroja, Madrid, 1971 (in Spanish).

[8] Italian Ministry of Infrastructure and Transport, Circolare 2 Febbraio 2009, n. 617. Istruzioni per l'applicazione delle nuove norme tecniche per le costruzioni, 2009 (in Italian).

[9] P. Roca, M. Cervera, L. Pelà, R. Clemente, and M. Chiumenti, Continuum FE models for the analysis of Mallorca Cathedral, *Engineering Structures*, **46**, 653–670, 2013.

[10] GiD: the personal pre and post-processor, Barcelona: CIMNE, 2002.

[11] M. Cervera, C. Agelet de Saracibar, M. Chiumenti, COMET – data input manual version 5.0. Technical report IT-308, *CIMNE*, Barcelona, 2002.

[12] S. Saloustros, Structural analysis of the church of the Poblet monastery, MSc SAHC thesis, Universitat Politècnica de Catalunya, July, 2013

[13] Y. Petromichelakis, S. Saloustros, L. Pelà, Seismic assessment of historical masonry construction including uncertainty. M. Papadrakakis, C. Sousa Oliveira, A. Campos-Costa, K. Pitilakis, R. Paolucci eds. *MS01 - Aseismic design of structures and lifelines (EuroDyn 2014)*, Porto, Portugal, June 30-2, 2014.