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Numerical Analysis by FEM for the assessment and the strengthening of the Masonry Vaults of the "Pieve" in Cavalese

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ABSTRACT: A fire on March 29th 2003 destroyed the timber roof of the "Pieve" in Cavalese (Italy), which ruined on the masonry vaults underneath. The stability and the strength of the masonry structures gave causes for concern, because of the roof collapse and the high temperature developed during the fire. Therefore, a comprehensive survey of the condition of the whole building had been exigently carried out, in order to highlight any contingent vulnerability. The collected data provided the source material for the structural analysis, which has been performed in order to go back to the effective cause of the observed damage and to formulate the consequent strengthening proposal

1 INTRODUCTION

The "Pieve" in Cavalese is one of the most important and ancient churches of the Fiemme Valley, in Trentino (Italy). The original fabric dates before 1111 although its only remaining parts are the two bays of the nave at the east end and the corresponding bays at the south aisle. The Romanesque bays are vaulted with intersecting ribbed vaults (Fig. 1). The long rebuilding programme began in the fifteenth century with the four bays, built in the gothic style. The nave is covered by a barrel climbing vault with lunettes (Fig. 2). The vault surface is decorated by juxtaposed foliated bosses and moulding that simulates the structural elements of a quadripartite rib vault. The aisles are covered by unusual sexpartite vaults (Fig. 3). Indeed the diagonal ribs don't spring from the main springer, but rather form the haunches of the transversal arch. The voussoirs of the transversal arch are interrupted at the intersection with the diagonal ribs and the two parts of the arch are connected through a straight stone element. The triangular spaces between the diagonal ribs are then bisected by horizontal groin lines, which are continuous between two bays, producing uninterrupted webs between the bays in the form of elongated triangles. In 1610 a four-bay aisle was erected, on the north side. The stark cross vaults are not separated by transverse arches. In 1640 a polygonal Baroque chapel, covered by a dome, was added at the west end of the aisle. The dome bears on a cylindrical drum. The transition between the circular base of the drum and the piers of the nave is achieved by the remaining portions of the original groined vaults, instead of proper squinches (Fig. 4). The porch in front of the main entrance, as well as the Firmian Chapel on the south side, date from the second half of the seventeenth century. A domical vault covers the square space of the chapel and rises over an octagonal drum on pendentives. In 1780 the choir was modified in the classical style and partitioned into two bays, with groined vaults. At the beginning of the nineteenth century the main sacristy covered by intersecting vaults was built.

Figure 1 : The nave vaults (XII century)

Figure 2 : The nave vaults (XIV century)

Figure 3 : Northern aisle (XIV century)

Figure 4 : Detail of the Rosario's Chapel

A fire on March 29th 2003 destroyed the timber roof of the "Pieve" in Cavalese, which ruined on the masonry vaults underneath. The stability and the strength of the masonry structures gave causes for concern, because of the roof collapse and the high temperature developed during the fire. Therefore, a comprehensive survey of the condition of the whole building had been exigently carried out, in order to highlight any contingent vulnerability. The collected data provided the source material for the structural analysis, which has been performed in order to go back to the effective cause of the observed damage and to formulate the consequent strengthening proposal.

2 THE SURVEY PHASE

2.1 Introduction

In a restoration process preliminary evaluation comes before any eventual decision relative to the artefact. In view of understanding the behaviour of the structure and the causes of contingent damage, the acquaintance phase leads up to the structural analysis, that is the evaluation of the actual static condition of the building and the state of stress.

During the acquaintance phase the condition of the whole building has been analysed as regards its geometry, the typological and technological features, the state of damage and the mechanical properties of the materials. The 'structural history' has been investigated as well, being the 'Pieve' the result of different building phases.

A preliminary in-situ survey permitted to identify the points where more accurate observations had to be concentrated. The investigation phase has been performed in detail at different levels. Single substructures cannot be studied ignoring the building as a whole and the interactions between its parts. Hence, the masonry walls and piers have been properly analyzed and modeled together with the vaults surfaces. The boundary conditions have been evaluated as well: the geotechnical characterization has been performed and the influence of the timber roof on the underlying structures has been assessed.

2.2 Geometrical and crack pattern survey

Advanced survey methodologies, such as laser scanning, integrated with direct surveying and topography, have been adopted to provide the three-dimensional models of the vaults. Because the point clouds, resulting from laser scanning, contained redundant information, they have been post- processed in order to obtain the geometrical model for the FEA.

The provisional metallic frames, built into the nave to shore up the vaults, as well as the external one, built to bear the temporary roof, permitted to closely inspect the masonry. Hence, a detailed crack pattern survey both of the extrados and the intrados of the vaults has been carried out (Fig. 5). Cracks have been evaluated as regards their position, direction, length, width and thickness. Extensive internal cracks were present in the extrados of the vaults at the haunches and in the intrados at the crown, according to the typical 'three-hinged arch' scheme (Heyman 1996). The analysis of the splits observed along the ribs as well as around the 'keystones' of the nave vaults, has led to the conclusion that neither the ribs nor the rose at the keystones are structurally connected to the masonry vaults; ribs rather behave as independent arches. Disconnections between the two systems certainly occurred already in the past, as it is confirmed by the presence of metallic stirrups securing the bosses.

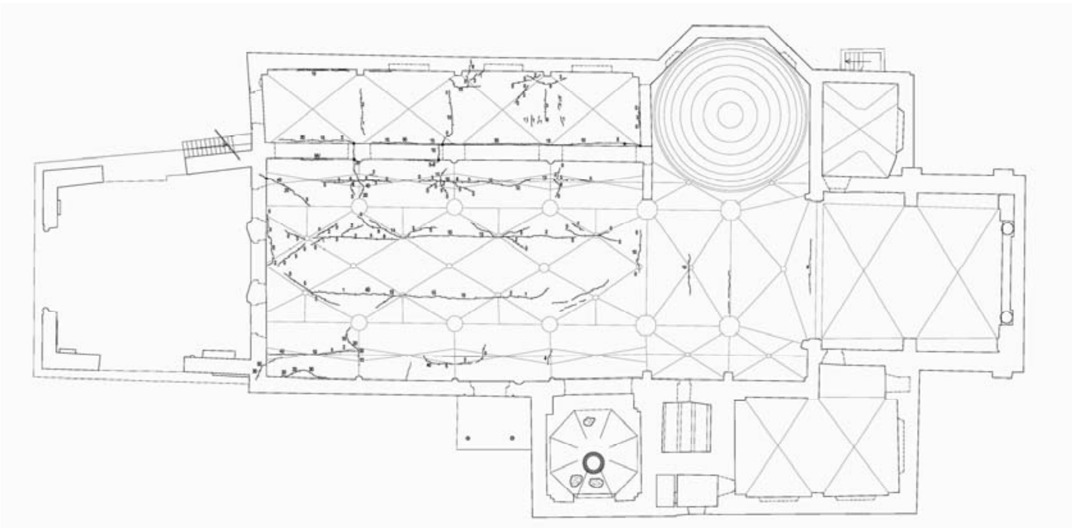

Figure 5 : Crack pattern survey (Plan view)

2.3 Minor destructive tests

In order to understand the morphology of the masonry wall section boreholes have been cored in the most representative points of the structure. In particular, drilled cores have been done in some piers of the nave, in order to reconstruct the thickness of the masonry structural layers, as well as of the binders. The vaults thickness ranges from 20 to 30 cm. Inside the boreholes additional investigations have been made by the use of borescopy. They allowed a detailed study of the borehole surface and the detection of inner voids.

2.4 Laboratory tests

Laboratory tests have been carried out on materials sampled from the construction. The aims of these tests were to characterize the material from a chemical, physical and mechanical point of view, to know its composition and content in order to use compatible materials for the repair, and to assess the effects of fire on the exposed masonry

Basement walls were heavily splotched with efflorescence, so chemical analysis has been carried out both on samples of masonry cores and efflorescence.

The texture homogeneity of the piers' masonry, resulting from the analysis of both the boreholes and the drilled cores, enabled the derivation of the mechanical properties of the material by means of laboratory mechanical testing. In particular a mean Young modulus of 40000 MPa for the piers has been obtained from compressive tests.

The masonry consists of travertine stone bound with a mortar composed by sand and a magnesic calcium-based binder. Because of the presence of Magnesium salts in the masonry composition, a chemical incompatibility with lime-based grouts, normally used for strengthening interventions, has been highlighted. Thermal analysis has been carried out on masonry vaults, in order to evaluate the effects of high temperatures developed during the fire. A differential scanning calorimeter (DSC) has been used for determining the heat changes occurred during the fire and consequently the range of thermal and chemical transitions in the material. Both the superficial layers (Fig. 6a) and the deeper layers of the samples (at 3 cm from the surface) (Fig. 6b) have been analyzed. The most heated area was the Rosario Chapel dome, where the timber roof frames were very close to the vaults extrados.

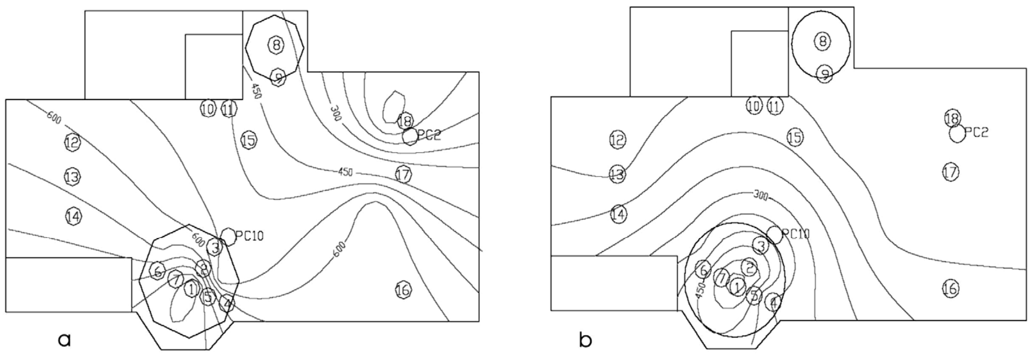

Figure 6 : Analysis of the fire effect. Isothermal lines maps (a) superficial samples (b) 3 cm from the surface (RTeknos s.r.l., Zeila lab. s.r.l.)

2.5 *Flat jack test*

Two types of flat-jack tests have been performed on the masonry structures: the in-situ stress or single-jack test and the in-situ deformability or two flat jack tests (ASTM 1991). The evaluation of the in-situ compressive stress was carried out by introducing a thin flat-jack into a horizontal cut in the masonry. The stress relief caused by the removal of the portion of material determined a partial closing of the slot, the original state of stress was then restored by pressurizing the flat jack. Consequent displacements have been measured. Equation 1 permits to calculate the local compressive stress:

$$\sigma = P \cdot K_m \cdot A_m / A_t \qquad (1)$$

with K_m =calibration factor of the flat jack; P = 'canceling pressure' of the hydraulic system; A_m= area of the jack; A_t= area of the slot.

To determine the deformability characteristics of the masonry, flat-jacks were inserted into two parallel cuts, which delimited a masonry sample of appreciable size (ca. 500-600 mm high and 325 mm wide). Then a uni-axial compression stress has been applied. The displacements refer to different loading-unloading cycles. The deformability modulus of the masonry nave vault, ranges from 180 to 270 MPa and from 89 to 177 MPa, for stress rates respectively from 0.16 to 0.32 MPa and from 0.32 to 0.64 MPa. A linear response of the material has been observed up to a stress value equal to 0.40 MPa. Two-flatjack tests gave a mean deformability modulus of the masonry walls equal to 3000 MPa.

2.6 *Sonic pulse velocity test*

The velocity of the sonic pulse has been correlated, to various degrees of accuracy, with the masonry mechanical properties. In addition, sonic techniques have been used to detect material flaws and voids as well as to find crack and damage patterns. The sonic tests were also been used in order to determine the improvement in the mechanical characteristics of masonry of the gable, in front of the church, repeating the same tests before and after the lime injection.

2.7 Analysis of the boundary condition: geotechnical characterization

The geotechnical characterization of the site was obviously done, by means of some direct inspections inside the soil. It must be observed, synthetically, that the foundations of the walls and piers are on bed-rock. The ground-penetrating-radar testing technique was also been used for identifying and mapping man made underground cavities and manufactures, in order to help the subsequent archaeological exploration of the site.

3 STRUCTURAL ANALYSIS

3.1 Introduction

Structural analysis has been performed as culminating step of the acquaintance phase, in order to highlight the causes of the observed damage and understanding the global and local behavior of the structure. Moreover, a further structural analysis permitted, in the design phase, to verify the effectiveness of the proposed interventions.

In this paper only the structural analysis of the lunette barrel vault of the nave is described. The proposed approach resorts to the finite element method and includes linear and non-linear behaviour of the material. Different scales of modeling have been adopted for the analysis of the vaulting: a single bay has been modeled in order to analyze its behavior and its interaction with the adjacent structures; on the other hand, a quarter of the vault, has been studied in detail, in order to analyze local phenomena such as cracking. These numerical results have been compared with the crack pattern survey data, in order to assess the validity of the model. A Macro-modeling approach have been chosen, where the different features of structural masonry are represented by an equivalent, homogenized continuum (Lourenço 1998). A multi-step analysis has been performed. At a first step, linear finite element analysis identified local areas of significant tension and, therefore of possible resultant structural distress and cracking under normal service load condition. At a further step, nonlinear finite element analysis highlighted nonlinear phenomena, particularly cracking, causing a significant redistribution of stresses. An incremental analysis has been performed in order to evaluate the influence of intrinsic characteristic, such as dead loads, on the behavior of the structure. The structural analysis referred to the actual condition of the vaults permitted to formulate the consequent remedial measures. Subsequently, on the bases of the same model formulation, a finite element analysis of the strengthened structure has been performed.

3.2 Global models

A global Finite Element model has been analysed considering a single bay-transversal module that includes the nave and the three side aisles. The model has been implemented by means of about 8000 nodes, ~500 beam elements for timber structure and for piers and pilasters (for which all the preliminary analyses have clearly excluded structural damage or material decay problems), ~7000 “thin” 4-node shell elements for the vaults and the walls.

A linear elastic analysis has been performed and the mechanical characteristics of masonry have been deduced by the results of the preliminary investigation phase, as reported in Table 1.

Table 1: Mechanical properties of masonry (linear elastic analysis).

	E (MPa)	υ	Specific gravity (kg/m^3)
vaults	250	0.15	1350
walls	3000	0.15	1800
piers	40000	0.15	2800

In particular, the Young moduli of the masonry have been deduced by the values obtained from the two-flatjack testing for both the vaults and the walls and from lab testing for the piers.

Two different boundary conditions have been modelled at the pier base: a) hinged supports (fixed translation); b) fixed supports.

The loading condition associated with the state of the structure after the fire has been modelled. Hence, the only masonry dead loads have been considered, bearing no longer the roof on the underlying structures. The results of the analysis are given in Fig. 7 in terms of maximum axial stresses. Maximum values of the tensile stresses occur at the apex of the intersections of the barrel vault with the lunettes in the nave, at the keystone of the sexpartite vaults in the side aisles and at the crowns, in the areas where the vaults of the aisles meet the walls. The model results are validated by the evidences of damage observed in the masonry structures. Indeed cracks are present at the intersections between lunettes and barrel vault, as well as along the longitudinal ribs of the sexpartite vaults. A further analysis has been carried out adding the dead loads transmitted by the timber roof (Fig. 8). Maximum horizontal displacements referred to the two loading conditions have been compared, in order to highlight contingent dangerous effects of the roof on the stability of the masonry structures. As resulting from the comparative analysis, the presence of the roof doesn't influence significantly the behaviour of the underlying structures. The observed damage is rather to be ascribed to the material quality and the structural shape of the vaults themselves.

Figure 7 : Global model without roof: Axial stresses (hypothesis of hinged supports at the bases)

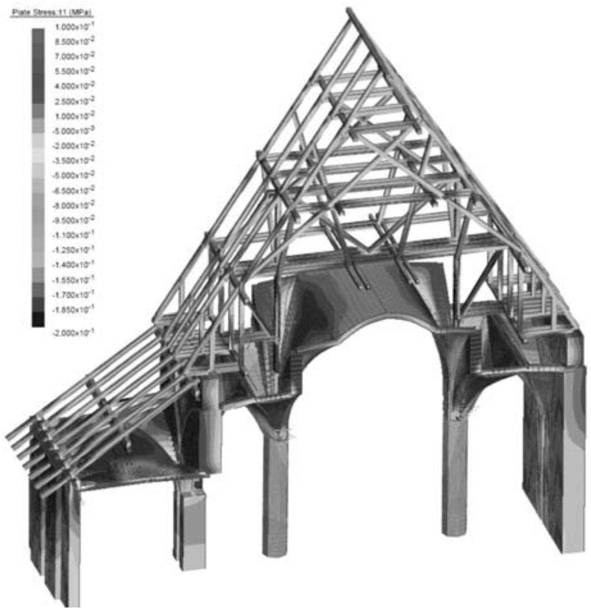

Figure 8 : Global model with the roof: axial stresses (hypothesis of hinged supports at the bases)

3.3 Detail models of the nave vault: unrepaired condition

Detailed models of the nave vault have been studied considering them both before and after repairs. Because of its double-symmetry just one quarter of the vaulted bay has been modelled. About 4200 nodes and approximately 33700 brick 8-node hexahedron elements were adopted for the analysis. Different models of increasing complexity have been used to analyse the vault in the unrepaired condition. In particular a first model assumes linear elastic behaviour of the material, whose properties are reported in Table 1. The global modelling demonstrated that the presence of the roof can be neglected, hence, only the masonry dead loads have been considered. The tensile zones, resulting from the first level modelling, highlight the areas of actual and potential cracks (Fig.9a). At a further level, the cracks actually present in the vaults have been introduced in the linear elastic model, disconnecting the elements on either side of the crack. Two different damaged models have been implemented: 1) with longitudinal cracks along the crown at the intrados and tangent to the lunette apex at the extrados (Fig. 9b), 2) with longitudinal cracks and transversal cracks at the intersection between barrel vault and lunette (Fig. 9c). In both cases, the analysis results highlight the presence of residual tensile zones, thus confirming that further local failures could occur. Subsequently, the non linear behaviour of the material has been considered. A no-tension model has been assumed, as represented in the σ - ε curve of Figure 10. Loading steps have been applied in order to evaluate the influence of load increments on the behaviour of the structure. Also in this case an undamaged model has been studied at first. The analysis results relative to the 30% of the total load, exhibit critical tensile zones at the crown in the intrados and at the haunches in the extrados (Fig. 11a). A damaged model has been then considered, introducing the longitudinal cracks really existent. In this case maximum tension values occur at the intersection with the lunette (Fig. 11b). A further model that considers also the transversal cracks, exhibits marginal tensile zones (Fig. 11c). The incremental analysis permitted to ascribe the most significant damage to the intrinsic characteristic of the structure, such as dead loads. Indeed at a load step relative to the 40% of the total load, convergence problems occurred, thus confirming the severe static condition of the vaults.

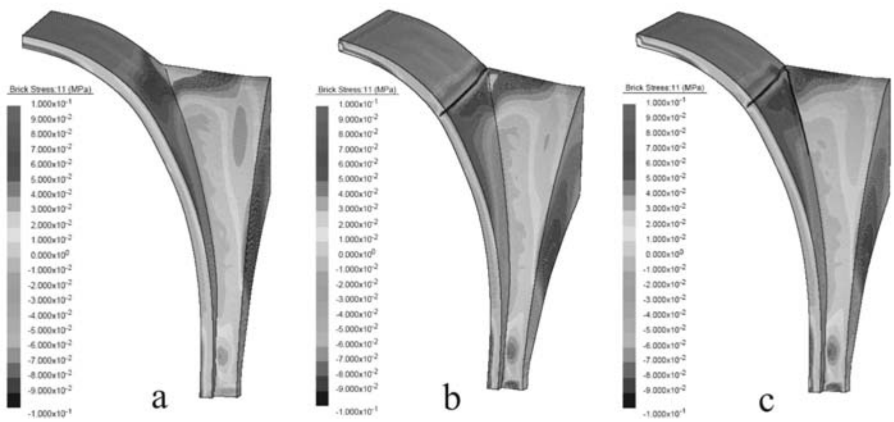

Figure 9 : Detail model of the (unrepaired) nave vault. Linear analysis

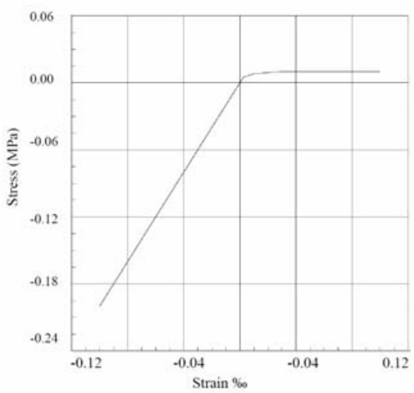

Figure 10 : Non linear behaviour of the material

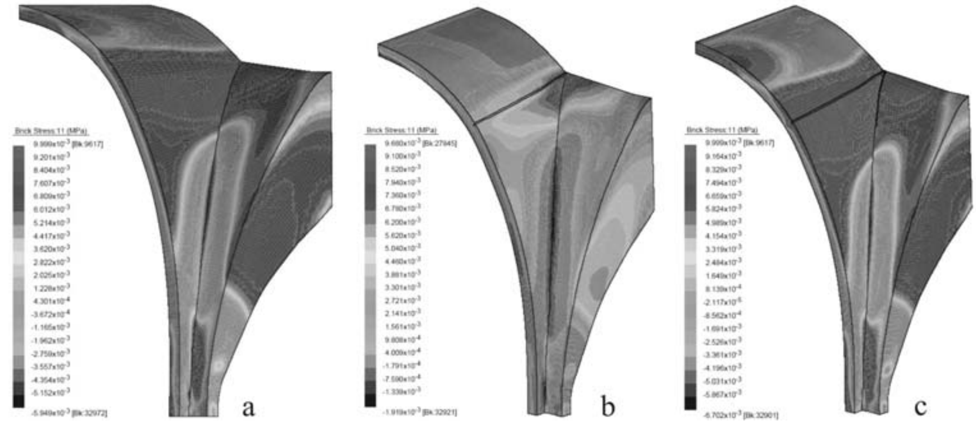

Figure 11 : Detail model of the (unrepaired) nave vault. Non linear analysis; (a) undamaged model (30% dead loads), (b) longitudinal cracks (10% dead loads), (c) initial transversal crack (35% dead loads)

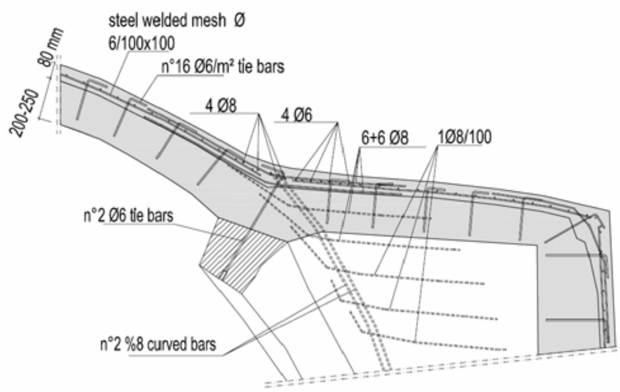

Figure 12 : Strengthening of the nave vault

3.4 *Detail models of the nave vault: repaired condition*

The structural problems, arising from the visual inspection and confirmed by the structural analysis, required adequate solutions. Fig.12 shows the local interventions on the nave vault.

The design phase has been supported by a detailed structural analysis, modelling the same portion of vault studied in the unrepaired condition. The masonry vault is considered in a “damaged state”, taking into account a fictitious low value of the Young modulus of elasticity (different tests have been done with values in the range 1 - 10 MPa). The strengthening skin has been modelled with ‘shell’ elements, and a modulus E=12500 MPa has been considered, as resulting from the lab testing. Figs. 13-14 show the plots of bending and axial stresses in the direction of local axes X and Y.

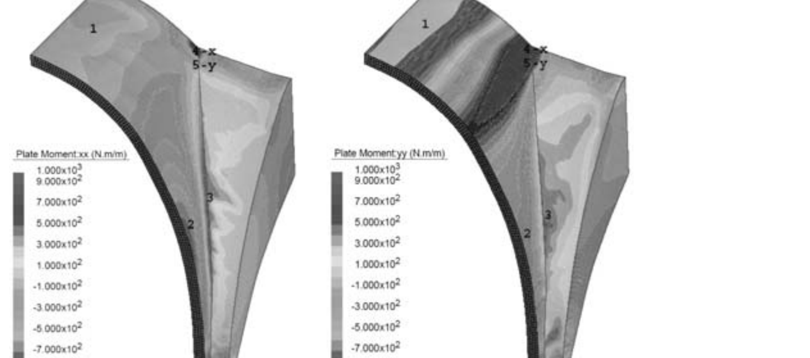

Figure 13 : Bending stresses; (a) X and (b) Y direction. Detail model. Repaired state.

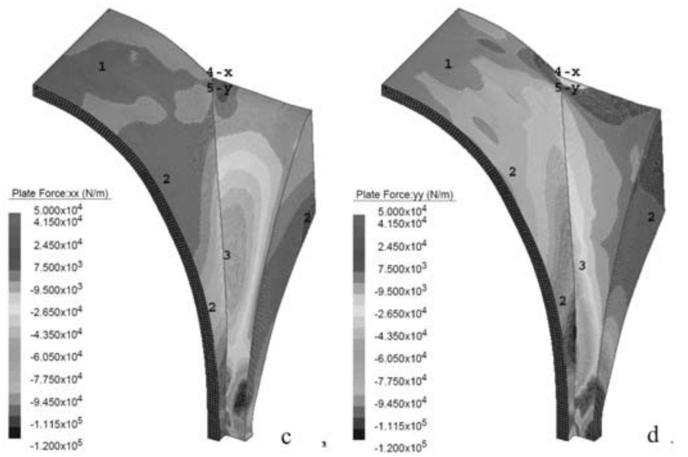

Figure 14 : Axial stresses; (c) X and (d) Y direction. Detail model. Repaired state.

4 CONCLUSIONS

The multi-level modelling approach proved to be effective to avoid highly time-consuming and costly analysis. Indeed more complex analysis has been focused on the critical areas highlighted in simplified global models. However a detailed preliminary throughout investigation is the key of a valid modelling, both as regard the characterization of the material and the understanding of the behaviour of the structure and the nature and causes of the damage.
The proposed modeling approach demonstrated predictive capacity and has permitted to adequately link investigation and restoration.

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