

Vulnerability and seismic improvement starting from experimental investigation

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ABSTRACT: The paper presents an investigation carried out to characterize the seismic response and structural vulnerability of St. Giovanni Battista church, typical of Italian structures built around the XVII century in Italy. The applied methodology combines a set of integrated activities, including the dynamic numerical analysis of the church by means of detailed FEM modelling, with experimental tests aimed to evaluate mechanical properties of materials. Minor destructive techniques was carried out. In particular, this paper describes in detail the technique of sliding shear tests, useful for shear strength characterization of historical un-reinforced masonry. Starting from surveys addressed to the description of geometrical and morphological features, the investigation process allowed vulnerability identification. Results are presented for both, before and after strengthening interventions.

1 INTRODUCTION

This paper reports the structural analysis of St. Giovanni Battista church (Figs 1 and 2) located in Carpenedolo (Brescia – Italy). After the earthquake which struck Brescia district in November 2004, this church was analyzed in its entirety, in order to identify appropriate strengthening interventions. Religious buildings generally present a pronounced seismic vulnerability related to their significant dimensions and masses, reduced horizontal connections, tall masonry elements lacking orthogonal stabilizing walls, and presence of vaults and arches that can increase their thrust following the seismic event. Damage in such churches have often been detected after low/moderate intensity earthquakes (Lagomarsino 1999) indicating for this structural typology an actual safety problem. The analysis of the Church was conducted with respect to (Casarin & Modena 2006) its constructive history and structural evolution, post earthquake damage manifested and current condition. Subsequently an on-site investigation campaign was erected. Finally, the seismic assessment of relevant parts of the structure, involving different modelling strategies, was carried out. Global linear elastic numerical models, calibrated based on the results of the experimental phase, were used.

Only reversible and not invasive interventions were considered, to maximise conservation of the existing structure and reach only the necessary safety level.



Figure 1. Panoramic view of St. Giovanni Battista church.



Figure 2. Wooden trusses and extrados nave barrel vault.



Figure 3. “Palladiano” scheme truss.

2 CHURCH DESCRIPTION AND ELEMENTS OF VULNERABILITY

The church presents a Latin cross plan, with only a central high nave. The maximum length of the church is 64.20 m, the maximum width, where there are the altars, is 24.70 m. The vault rises for 23.50 m from the ground, and the dome reaches inside the maximum height of 32.50 m.

The western part was built between 1693 and 1713, whereas the eastern part was built after 1761. The facing east apse, following the tradition, has a simple round arch vault morphology without any opening. An elliptic dome is sustained above the extrados of a big octagonal structure, at the cross between the nave and the transept. A top lantern crowns the edge of the dome. The wooden roof is a double pitched one characterized by a truss structure. In particular, the pre-existing truss structure is a simple truss, whereas the following intervention presents a “Palladiano” design (Fig. 3).

The nave has a 180 mm thick barrel vault structure. In add there are wall ties on the arches impost. The main structures are made of bricks with dimensions $290 \times 140 \times 60$ mm (l x h x t). The masonry pattern is regular, with horizontal bed joints.

The lime mortar is present only in the horizontal bed joints, whereas the vertical joints are, at least superficially, without mortar. The vertical head joints, interrupted by the bricks, are generally of poor quality and weaker than the continuous horizontal bed joints. The average thickness of the nave’s walls is 950 mm at the base. At the impost level it becomes thinner, with a 750–800 mm wall thickness, and it reaches a thickness of 500 mm on top, where the trusses stand. The wall behind the apse is characterized by a greater thickness, and it is without openings. The thickness of the façade is about 1200 mm at the base, and it decreases towards the top, where the thickness of the tympanum is about 900 mm. On the other side of the façade, there

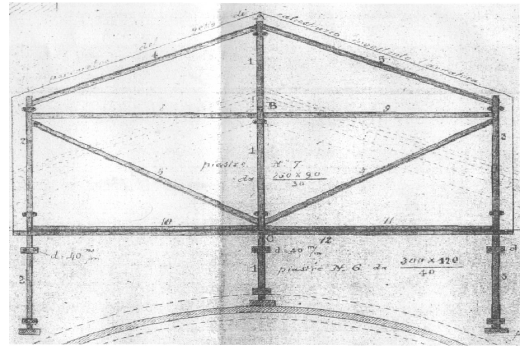


Figure 4. Drawing concrete tympanum Ing. Mazzocchi (1908).

is another tympanum made of concrete and added later, during the strengthening interventions of 1908. This element, which has a thickness of 300 mm, functions as a counter-weight.

2.1 Façade intervention of 1908 a.c.

In 1902, the restoration of the façade caused static problems especially on the decorated pediment and on the pitched roof. Both the pediment and the roof had slanted towards the below square. In 1908 the engineer Luigi Mazzocchi created a steel truss structure covered by concrete, firmly fixed to existent tympanum and supported by three steel beam to lower vault (Fig. 4).

The new pediment was 8 m high, excluding cross and pinnacles, positioned in the façade’s ledge at the height of 24 m from the below square. Its total weight was estimated in 185 Mg. The back surface not covered by the pitched roof was estimated at 115 m². The thickness of the concrete was 300 mm. A geometrical survey has shown the façade tilting approximately 20 mm/m, with a consequent “out of plumb” of 160 mm in the highest part of the façade.

3 DAMAGE SURVEY

From the analyses of the damage survey and of the crack pattern due to the earthquake in November 2004, it is possible to observe that:

- the church, on the whole, does not show significant static damage relating to the collapse under static service loads;
- in contrast, there are many locally critical situations in the highest part of the construction. The inappropriate supports of the trusses caused a worsening of the seismic behaviour under horizontal force. Moreover, even only under static condition (i.e. dead load only) they induced cracks;

- wall enlargements used as support of the wooden trusses are in general untied from the rear masonry; this causes significant instability and bending;
- under seismic loading asynchronous motions have damaged the highest part of the lateral wall of the church, in correspondence to wooden truss supports, thereby creating a different morphology and crack thickness;
- the absence of an element able to redistribute the horizontal forces has allowed the formation of a “Rondelet” mechanism of collapse of the historical un-reinforced masonry (URM) wall between the two wall enlargements used as support of the trusses;
- the façade, which can be seen as a macro-element able to start the kinematic process during an earthquake, shows significant vulnerability for two reasons. First, the concrete tympanum is very stiff. Second, the presence of diagonal cracks in the lateral walls of the nave indicates that the façade may overturn;
- the global behaviour of the structure under horizontal forces can not be considered satisfied.

4 IN SITU INVESTIGATIONS

To understand behaviour during an earthquake, some minor destructive testing was necessary:

- sliding shear tests (SST);
- single and double flatjacks.

A total of 2 SST and 4 tests using flatjacks – 2 tests using a single flatjack in order to investigate the masonry stress distribution under service load and 2 tests using a double flatjack in order to evaluate the failure strength of the masonry pattern – were carried out.

4.1 Sliding Shear Test (SST)

The aim of this paragraph is to describe a particular investigation technique, a sliding shear test, on a URM wall to define a shear resistance useful for numerical models, assuming that wall shear strength is limited by the shear of the mortar joints rather than shear through the units. This technique belongs to push/shove tests. The maximum horizontal force recorded during the test divided by the gross area of the upper and lower bed joints is the mortar joint shear strength index.

Shear failure is characterized by joint failure, as a function of the bond strength linked to the frictional resistance at the brick-mortar interface and the compressive stress normal to the bed joints. The criterion is known as Coulomb type failure criterion:

$$\tau_u = \tau_0 + \mu \cdot \sigma_n$$

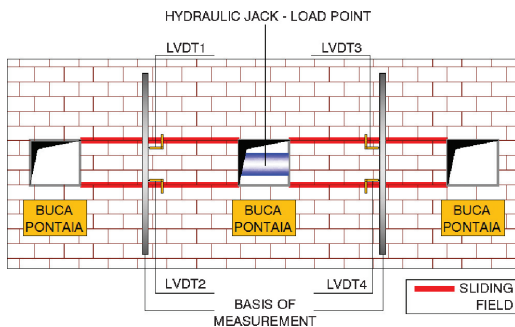


Figure 5. Sliding Shear Test (SST) scheme.

where τ_u is the average shear sliding stress at failure in the wall; τ_0 is the shear stress without pre-compression; σ_n is the average normal stress.

The SST is made using a hydraulic jack positioned in a pre-existing hole (200x200 mm) of the investigated masonry able to create a force parallel to mortar bed joints. After having chosen the test area, the positions of the measurement points are identified as shown in figure 5, using an appropriate contrast base linked with a piece of undisturbed masonry out of the sliding phenomena.

The test was carried out using a hydraulic jack positioned, in this specific case, into a “buca pontaiia” (i.e. a pre-existing hole). In this way, a pseudo-isolated piece of masonry was tested, as this part was between other two “buche pontaiie” and so it was free of moving. When masonry is subjected to tensile loads, along the parallel direction of bed joints, the brick and the head joints are subjected to compressive stresses, while the bed joint is subjected to shear stresses. The displacements – associated with the force applied by the jack on the piece of masonry – allow the calculation of the shear strength of that piece of masonry and of the mortar stiffness, knowing the flow surface of the mortar. Figures 6a-6c show some moments of the test.

The mortar joint shear strength index is directly affected by the magnitude of vertical load at the point of measurement. In this case the component of orthogonal stress acting locally on the piece of masonry can be neglected because the test were carried out on the top of the church, in order to obtain the masonry shear strength in the absence of vertical load (τ_0 or $f_{v,k0}$ for Italian Code D.M. 20/11/1987). Sliding displacements are determined by a couple of measurement point jack in parallel to the force direction and symmetrically placed between the hydraulic jack (Fig. 6a). In this way it is possible to draw graphs load-displacements able to show in detail how the displacement changes with the increase of the load (Fig. 7). The stiffness and the shear strength of masonry bed joints can be determined increasing the jack pressure until the appearance of micro-cracks in mortar and bricks identifying the yield

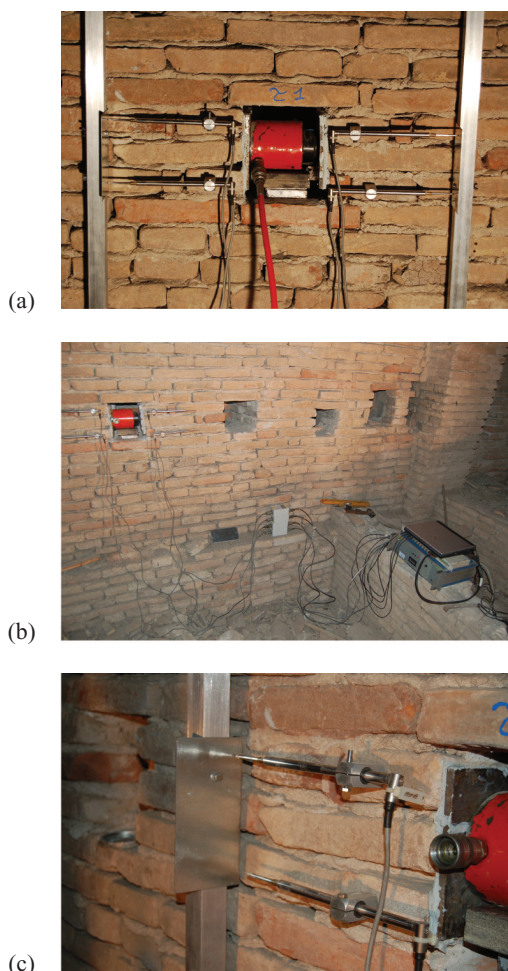


Figure 6. Sliding Shear Test (SST) set-up. Hole dimension 200×200 mm, distance between holes 600 mm, distance between upper and lower instruments 150 mm.

strength point. More in detail, this graph allow also the identification of the piece of masonry (left or right side) that has to be used in order to quantify, in the best way, the sliding shear strength of the wall. At this point, knowing the force applied (F) and the sliding surfaces (A), it is possible to obtain the average shear sliding stress of the pieces of masonry located on the left or on the right of the jack, with the simple equation $\tau_u = F/A$.

This experimental test is particularly indicated for masonry like that of this study (bricks and horizontal bed joints) and allows a double control on the observed measurements (Rossi & Riccioni 1995), as shown in figure 7. First, the values obtained for the left part are directly comparable with those of the right part,

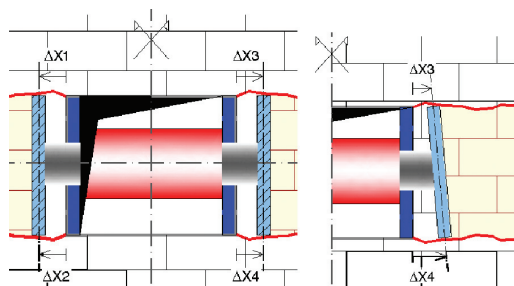


Figure 7. Homogenous and non-homogenous sliding.

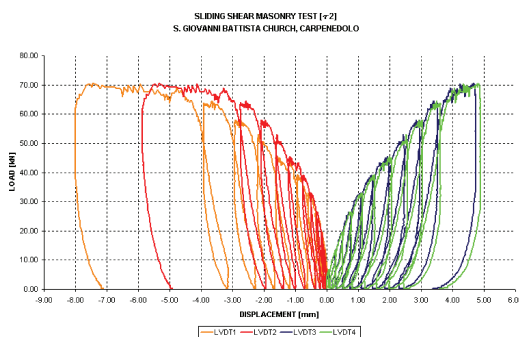


Figure 8. Load-displacement graph of Sliding Shear Test.

symmetrically located between the jack. Second, upper and lower measurement points allow the identification of potential relative displacement of the existing piece of masonry.

The mechanical behaviour of URM, for each basis of measurement, is thus characterized by (Fig. 8):

- a first elastic phase, until the 50% of the maximum load, identified by an high gradient (stiffness) calculated as the ratio between load and displacement;
- a second phase, with a smaller stiffness and characterized by the presence of no longer negligible inelastic phenomena. These indicate a cracking in action;
- a third and last phase characterized by a plastic behaviour with significant deformations till the break for sliding shear of the mortar joints.

Masonry investigated in the highest part of the church – up the barrel vault and under the wooden roof – shows sliding shear strength of about 0.168–0.172 MPa.

One of the advantages of this technique is the possibility to obtain in situ sliding shear tests using an undisturbed sample representative of a broader piece of URM. When the test is completed, the jack is

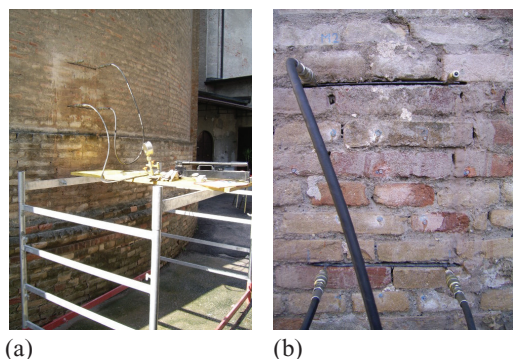


Figure 9. Flat jack tests.

removed and, if necessary, the masonry can easily be re-established plugging up the holes with bricks.

4.2 Double and single flat jack

This kind of test methods concern the measurement of in-situ compressive stress and the deformation properties in existing masonry by use of thin, flat jack devices that are installed in saw cut mortar joints in the masonry wall (Figs. 9a–9b). These test methods provide an in situ minor destructive technique (MDT) determining masonry stress distribution under service load by using one flatjack and the deformability characteristics of the masonry by using two parallel flatjacks.

The masonry investigated in the lower part of the church – in the apse external side – shows a compressive stress of about 0.75–1.15 MPa and an average deformability modulus of 4800 MPa.

5 STRUCTURAL BEHAVIOUR

The response of an URM building to horizontal seismic actions can not be correctly defined, in the majority of cases, by considering the global behaviour of the structure (Modena et al. 2006). Anyway in order to study the church in its entirety, trying to understand the main vulnerability, a numerical simulation was carried out using a finite element model (FEM). The FE model has been calibrated with the experimental results, in particular using the deformability characteristics of the masonry (elastic modulus). In order to realize a seismic improvement on the roof structure making it stiffer, some reversible and not invasive interventions have been planned. The roof pitches are transformed into folded plates, which gather and transfer the seismic action to the shear resisting walls. Diaphragms are created on top of the existing structures without significantly modifying the roof overall layout (Giuriani &

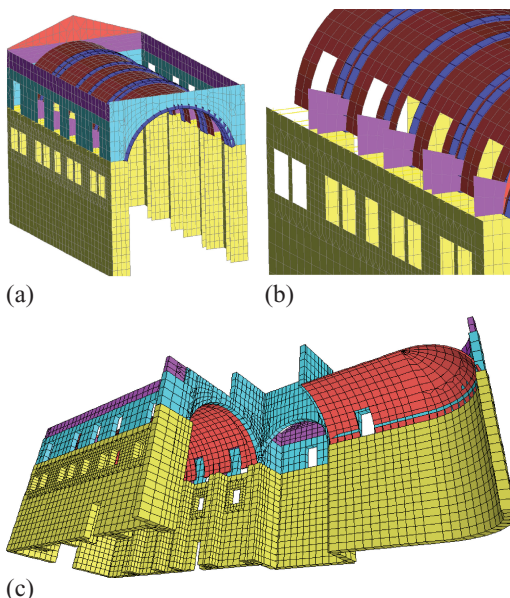


Figure 10. FE model views.

Marini 2006). Two different FE models (as explained below) were studied in order to compare results.

5.1 Global analysis: FE model

The perimeter walls were modelled using “plate” elements, split in different groups depending on the different identified thicknesses. Then buttresses were introduced – always using “plate” elements – in the nave, orthogonally to its walls. The tympanum on the top of the façade was modelled, considering that at the beginning of 1900 a new tympanum made of concrete was added against the original one, like an element stiffer than the lower masonry (see section 2.1). Vault ribs, identified during the geometrical survey in correspondence of the buttress, were modelled as “plate” elements with appropriate thickness (Figs 10a–10c). The wooden trusses were introduced in the model as “beam” elements. Finally, the structure was restrained at the base by, in first approximation, rigid connections (no degrees of freedom). Consequently a dynamic analysis with spectral response has been conducted in agreement with the Italian Code.

The main frequency of the structures related to the first transversal flexional mode is 1.43 Hz before intervention and 2.05 Hz after intervention. At the same time the main frequency of the first longitudinal flexional mode is 2.03 Hz before intervention and 2.87 Hz after intervention. The stress distribution under vertical load showed compatible values for this kind of structure. In particular, the linear elastic analysis

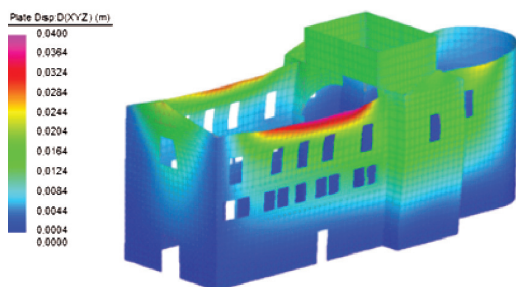


Figure 11. Displacement envelope under transversal seismic force and dead load – configuration BEFORE strengthening.

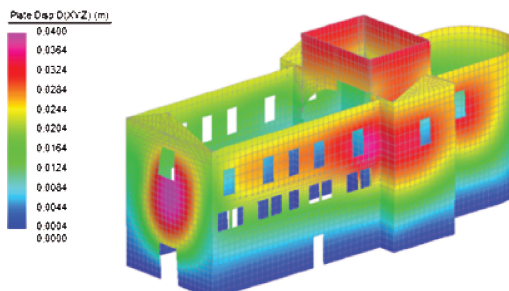


Figure 13. Displacement envelope under longitudinal seismic force and dead load – configuration BEFORE strengthening.

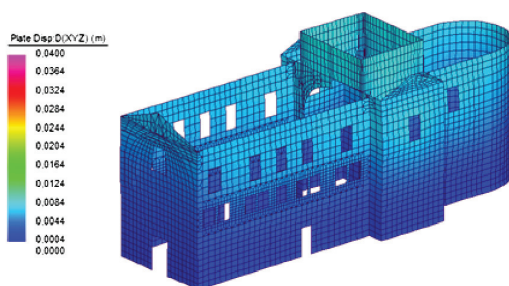


Figure 12. Displacement envelope under transversal seismic force and dead load – configuration AFTER strengthening.

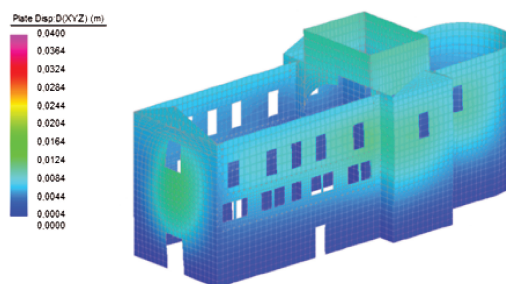


Figure 14. Displacement envelope under longitudinal seismic force and dead load – configuration AFTER strengthening.

showed that the stress value at the base of the church 0.40–0.90 MPa is the same of that measured during the tests with single flatjack (see section 4.2).

5.1.1 Dynamic modal analysis with spectral response

Figures 11–14 show the displacements obtained by numerical analysis under horizontal loads. Results are presented using the same chromatic scale (displacement range 0.00–40.00 mm). From the results of the global modal analysis, it is possible to conclude that an intervention aimed to link all the walls including also the wooden truss can be efficient against earthquake.

The increase of the stress acting on the main structural elements due to this kind of intervention appears compatible with the URM wall strength.

The displacements limitation obtained thanks to this hypothesis allows the church to have a more homogeneous response to earthquake in terms of deformation. The displacements under longitudinal seismic force decrease of the 60% in comparison with the actual situation. The projected strengthening intervention, in fact, was able to countervail locally out-of-plane displacement and redistribute horizontal forces to the lateral walls in a “global” way.

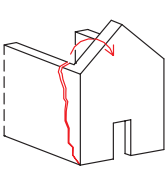
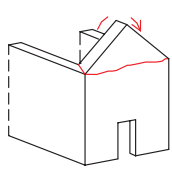
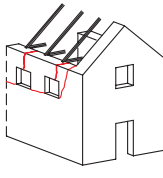
5.2 Local analysis: Mechanism of collapse

A vulnerability analysis was conducted, based on the calculation of collapse load factors associated with mechanisms of collapse, which best represented the surveyed damage patterns. The observation of local characteristics after seismic damages indicated the necessity to evaluate also the seismic response of individual structural elements, thus implementing suitable structural models. Previous research works carried out (D’Ayala et al. 1997) show the validity of a limit-state approach in the assessment of the seismic behaviour of URM buildings. The observation of the seismic damage on existing buildings allows the classification of the collapse mechanisms in two groups:

- I type mechanisms (OP): the collapse is due to seismic actions orthogonally directed to the wall;
- II type mechanisms (IP): the collapse is due to seismic actions parallel to the wall.

During earthquakes, OP mechanisms are more likely than IP mechanisms in buildings without an appropriate global and whole behaviour. OP mechanisms can be evaluated through the URM limit state analysis, where the masonry walls are simulated with

Table 1. Main collapse mechanism.

| | | |
|---|---|---|
|  |  |  |
| Case I Façade/transept | Case II Façade/transept | Case III Nave |
| Global Overturning | Overturning of upper part | "Rondelet" mechanism |

a system of rigid bodies with no tensile strength, articulated by hinges, whose geometry and distribution are defined by the failure mechanism. IP mechanisms, which happen more frequently than OP ones, cause cracks in the shear resistance walls. This means that IP mechanisms happen when these walls reach their shear strength.

Gruppo Nazionale Difesa Terremoti (GNDT) has a database for classification of damage by structural macro elements and associated collapse mechanisms. From the crack pattern and the damage survey combined with the global numerical analysis, only the OP mechanisms shown in Table 1 have been considered. The methodology is described into the Italian Code OPCM 3431/05.

Table 2 summarizes spectral acceleration values before and after strengthening intervention of some main elements in comparison with the values required by the Italian Code OPCM 3431/05.

Analyzing the results, some important conclusions about the OP mechanisms can be drawn. The improvement of the roof due to the introduction of the metallic beam-tie at the top of the perimeter walls, better described in the paragraph 6.2, and of the tympanum, allows a significant reduction of the collapse multiplier. This factor is calculated as the ratio between the minimum horizontal acceleration (responsible for collapse mechanisms) and gravity acceleration (a/g coefficient in Table 2).

6 STRENGTHENING INTERVENTION

Below, the strengthening interventions chosen in order to increase the seismic safety of the church are described. These interventions were projected as uniform interventions and using compatible materials. In fact, in case of particular structures, such as churches or at-sight wooden roofs, only roof diaphragms were arranged. The in-plane shear resistant floor and roof

Table 2. Kinematic analysis results.

| Structural ID case | a/g | Sa Capacity | DLS | Sa* CLS | demand DLS | Check CLS |
|-----------------------|-------|----------------|--------|------------|---------------|--------------|
| I Façade | 0.05 | 0.51 g | 0.24 g | 0.29 g | Yes | Yes |
| II Façade | 0.21 | 0.27 g | 0.33 g | 0.41 g | No | No |
| I Transept south | 0.04 | 0.36 g | 0.23 g | 0.29 g | Yes | Yes |
| II Transept south | 0.16 | 0.21 g | 0.33 g | 0.42 g | No | No |
| I Nave | 0.02 | 0.02 | 0.28 g | 0.36g | No | No |

DLS: damage limit state; CLS: collapse limit state.

diaphragms, transforming the building into a box structure, has been a viable solution to restrain the wall overturning (Giuriani & Marini 2006). The following significant aspects were considered:

- inappropriate connections between roof and perimeter walls have to be made effective especially in transversal response (e.g tying beam-ties realized with steel elements and masonry with injected anchors);
- elements characterized by significant vulnerability had to be modified or eliminated (e.g. adjusting non-homogeneity in terms of strength and stiffness);
- to verify that the introduction of local reinforcement would not cause a decrease in global ductility;
- roof interventions considered substitution of highly deteriorated elements;
- highly vulnerable elements like the tympanum and façade had to be tied to the roof structure to avoid global or partial overturning;
- local damage starts with cracks opening at the corner between the church façade and the orthogonal walls: these cracks indicate a possible stiff movement of overturning. Consequently, even if kinematic results are not worrisome, metallic ties, at different levels (13 m and 24 m from the ground), were designed. They limit potential façade overturning and re-establish a continuity between pieces of crakes masonry. This was adopted for the transept;
- finally, the truss improvement, due to cross-bracing in the pitched roof, implied a reduction of the overturning action of the façade and especially of the lateral walls of the nave increasing the collapse multiplier.

6.1 Wooden roof stiffening

The proposed strengthening technique could be easily applied for the construction of a seismic wooden roofs. First, post-tensioned cross bracing in the pitched roof made of steel cables would be placed between the wooden trusses. Then, near the tympanum, a steel beam truss, characterized by thin profiles where possible were projected. Also asynchronous motions

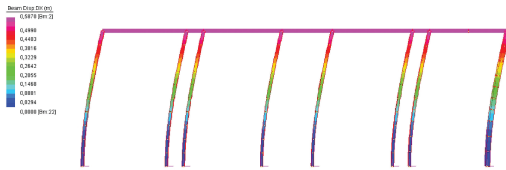


Figure 15. Roof displacement under longitudinal seismic force and dead load – configuration BEFORE strengthening.

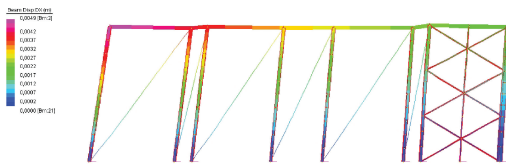


Figure 16. Roof displacement under longitudinal seismic force and dead load – configuration AFTER strengthening.

were considered. A specific local numerical analysis was conducted to understand the roof's behaviour before and after intervention (Figs. 15, 16).

6.2 Injected steel ties

Steel ties are elements typically engaged to improve static structural behaviour. They also can be applied for absorption of horizontal forces and to give connection between different parts. Technical improvements allow to avoidance of anaesthetic plates using anchors injected with mortar lime. All the steel ties have a post tension level useful for realizing “an active anchor”, except the ties placed in the south transept, characterized by significant cracks.

6.3 Edge curb

The seismic vulnerability of historical constructions is heavily influenced by the typology and quality of the connections between the different elements. In order to limit vulnerability, an entire level of edge curb have been projected. This technique consists in the realization on the church top, along perimeter walls, of a structural element. This element:

- creates an uninterrupted connection between the wooden roof and masonry walls;
- creates a retaining action of URM walls;
- connects the orthogonal masonry;
- improves the whole behaviour;
- contrasts the overturning of walls.

The edge curb can be built without dismantling the cover but scheduling single phases working inside the church under the wooden roof. The edgecurb becomes also a steel ties able to absorb the horizontal forces.

7 CONCLUSIONS

Seismic improvement strengthening interventions have been conducted considering historical, cultural and architectural requirements. This methodology should always be considered for historical URM. Consequently, simple and reversible techniques were chosen allowing to reach a satisfy structural safety level. Local injected anchors, even if not completely reversible, can be useful adopted using appropriate products (e.g. lime mortar contained into a specific sock) and, when necessary, removed by a bit bigger drilled hole. Experimental tests identifying mechanical properties of St. Giovanni Battista church in Carpenedolo (BS) specified sliding shear strength of URM. This was important to realize an efficient roof pitch diaphragm able to reduce the vulnerability of the structure, by transforming the building into a box structure, and avoiding walls overturning.

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