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Seismic Vulnerability Mitigation of a Masonry Church by means of CFRP Retrofitting

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Abstract

The paper presents some numerical results on a Romanesque masonry church located in Emilia-Romagna (Italy), a region recently stricken by a devastating seismic sequence on 20th - 29th May 2012. A full investigation of the damages and their comparison with advanced FE analyses, in both linear and nonlinear range are carried out. FE limit analyses are performed through non-commercial software proposed by one of the authors. A remarkable consistency is found among limit analysis results, real performance of the structure under seismic excitation and advanced nonlinear dynamic analyses. In particular, both damage patterns and active failure mechanisms found numerically are consistent with that observed on the church after the seismic event. The results put in evidence the insufficient strength of the apse for combined shear/bending actions, the columns of the central nave for bending, as well as the façade for overturning of the upper part. A seismic upgrading by means of CFRPs composite materials is proposed, designed and analysed quantitatively using FEs, finding an optimal fit between the required performance and the invasivity reduction. The interaction between CFRP strips and masonry substrate is accounted for assuming the behaviour of the reinforcement in agreement with Italian Guidelines for r.c./masonry strengthening with composite materials (CNR DT200). It is found that, with a targeted design, it is possible to prevent premature collapses of the macro-elements, strongly increasing the load carrying capacity of the structure.

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Keywords: Masonry Buildings; Strengthening Techniques; Composites; Seismic Vulnerability; CFRP Reinforcement

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1. Introduction

Masonry churches are traditionally quite vulnerable to seismic actions, exhibiting rather usually partial failure mechanisms active or prone to be active [1–14], even at very low levels of horizontal accelerations. This is a consequence of the much reduced interlocking between perpendicular walls, the almost absent box behavior and the inability of masonry to withstand tensile stresses.

Unfortunately, churches cannot be reduced to any standard static scheme [9, 15, 16]. Italian Guidelines for the Cultural Heritage [9] help practitioners in the safety assessment. To evaluate the acceleration at collapse, they suggest a quite rough and conventional approach based on pre-assigned partial failure mechanisms and the utilization of the kinematic theorem of limit analysis within the assumption of a no-tension material model for masonry. An abacus of twenty-eight possible collapse mechanisms, based on experience of failures observed during past earthquakes, is provided. On the other hand, a FE upper bound limit analysis procedure has been recently applied successfully by one of the authors of this paper on a variety of different examples, including many churches in seismic regions [17–22]. The material is modelled either homogeneous or heterogeneous, referring respectively the un-retrofitted and retrofitted masonry walls. In order to model the unreinforced masonry wall, a homogenization technique [19, 20], i.e. by replacing the heterogeneous assemblage of bricks and mortar with a continuous homogeneous material, is considered. For the retrofitted masonry walls a heterogeneous approach, non-homogenization between the masonry homogeneous material and the CFRP composite material is considered. Material properties are taken from Italian Code recommendations, presuming a limited knowledge (i.e. LC1 which recommends considering the median values for the elastic moduli and minimum ones for the strength).

One masonry Romanesque church located in the province of Ferrara is investigated. The case under study has been severely damaged by the recent earthquake occurred in 2012 in Emilia Romagna, Italy. A long seismic sequence occurred in the second half of May, with two peak events of *5.8* and *5.9* magnitude respectively on 20th and 29th May 2012. The seismic vulnerability is numerically assessed by mean of different analyses conducted on the church in the undamaged state. The analyses were able to mimic the damage patterns and active failure mechanisms observed after the seismic sequence. A strengthening intervention conducted by means of CFRP strips is numerically analyzed in detail, assuming the behavior of the strips –especially for what concerns delamination- in agreement with Italian Guidelines for r.c./masonry strengthening with composite materials (CNR DT200) [23]. Application of CFRP composites is analyzed in previous studies and it results an optimal technique for a consistent seismic upgrading [18, 24–27]. In order to have a better and more realistic insight into the behavior under horizontal loads of the church before and after strengthening, the commercial codes STRAUS7 and SAP2000 are utilized to perform both non-linear static and non-linear dynamic simulations. At this aim, respectively an elastic perfectly plastic approach obeying a Mohr-Coulomb failure criterion (STRAUS7) and an isotropic smeared crack "concrete" model are adopted to simulate masonry behavior under pushover and non-linear dynamic analyses. After proper scaling of the mechanical properties, indeed, it has been found that such approaches may fairly reproduce masonry behaviour in the inelastic range, even if orthotropy due to the texture is disregarded.

2. Brief description of the church

The church of Natività della Beata Maria Vergine, is located in Vigarano Mainarda, a small city near Ferrara, Italy. This church is a structure constituted by three naves, with approximate dimensions equal to 31 m × 20 m × 13 m (length × width × maximum height). The frontal view and the section along the aisle are shown in Figure 1. The façade results poorly interconnected with perpendicular walls, thus making the hypothesis usually done in global finite element analyses (perfect interconnection among transversal walls) rather questionable. The lateral walls of the central nave result highly perforated, both for the presence of large openings surmounted by arches interconnecting central and lateral naves and for the presence of relatively large windows in the upper part. The church is connected in the rear to a secondary sacristy on the left and to a large oratory on the right; only a part of this latter structure is considered in the FE model, since the focus remains the analysis of the church. Finally, it is worth emphasizing that a light wooden roof is present, which is in agreement with the building traditional technology of such typology of structures in the region considered. Its membrane stiffness is safely assumed negligible for horizontal loads, as well as the box behavior induced by its presence in the FE models. For this reason, only vertical loads transferred by the roof to the head of perimeter walls are introduced in the models.

a) b)

Figure 1. Natività della Beata Maria Vergine Church. a) Frontal view; b) Longitudinal Section.

a) c) e) g)

b) d) f) h)

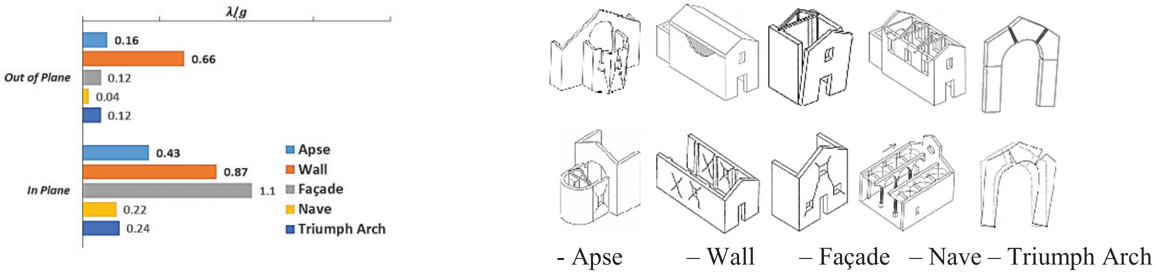
Figure 2. Observed damages after the Emilia Romagna Earthquake. a, b) Overturning of the structural elements. c) Overturning of the tympanum. d, e, f) Detachment of the vaults from the walls. g) Diagonal cracks in the chorus walls and h) longitudinal tie rods.

3. Observed damages

The church exhibits a fully active first mode failure mechanism on the façade, with visible overturning along a straight horizontal yield line. In correspondence of the upper part, the façade shows a large detachment from lateral walls, with displacements exceeding 10 cm, also demonstrating the rather poor interconnection between perpendicular walls, see Figure 2. This mechanism is evident in the upper part of the façade, the tympanum, because the lower part is connected with the rest of the construction with tied rods, which seem to have worked properly during the seismic event, see Figure 2 - h. The apse exhibits an almost active failure mechanism, with the formation of a 45° inclined crack departing from the upper part and developing for a couple of meters, see Figure 2 - b. The failure mechanism, albeit not fully active, suggests the out-of-plane rotation of a portion of the apse around a horizontal hinge placed in correspondence of the middle height. A notable detachment between the ceilings and the walls in almost all the vaults is present. The arches and ribs of the vaults are conceived with a wood frame, with a lathing surface and finished with plaster. Its low stiffness compared to the stiffness of the masonry walls made to concentrate the deformations in the joints, see Figure 2 – d, e, f. In absence of experimental data available, mechanical properties adopted are generally those available in the Italian code for masonry made by clay bricks and lime mortar (the common one in that region). They are the following: 1500 MPa for the Young modulus; 2.4 MPa for the compressive strength; 0.1 MPa for the tensile strength [15]. A smeared crack total strain material model for masonry, which allows for an investigation of the non-linear behavior, is adopted for the FE code.

4. Limit analyses results

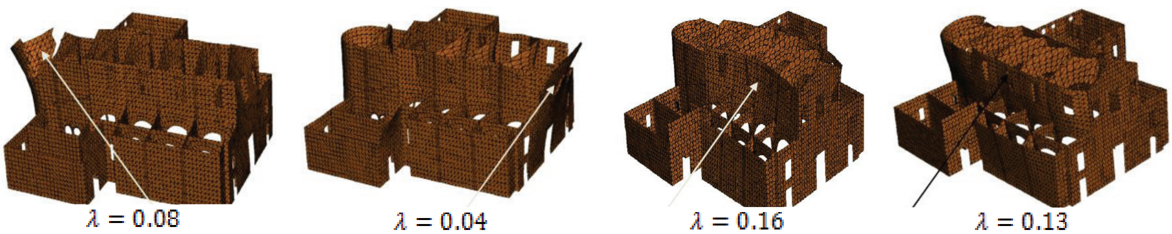
The seismic vulnerability of the church is estimated by means of both limit analysis and a nonlinear static approach (pushover). In agreement with Italian code, as far as limit analysis is concerned, the numerical evaluation of the collapse load (λ) is done investigating each possible pre-assigned failure mechanism. Ten failure mechanisms on a total of twenty-eight are identified as possible for the church, see Figure 3-b. Assessing the seismic vulnerability by mean of macro-elements is recommended by Italian Guidelines for Cultural Heritage [9], and already successfully applied recently [2, 28]. The results are depicted in Figure 3-a). They highlight a particular vulnerability for rocking, especially on the façade, the central nave arcades and the apse. However it is worth mentioning that the hypothesis of null interlocking between perpendicular walls is adopted, meaning that probably safe estimations of the collapse accelerations are provided. The second strategy is based on a FE limit analysis, proposed by Milani et.al. in e.g. [5, 29, 30]. FE limit analysis has the great advantage of automatically providing the active failure mechanism, utilizing the same discretization used for linear FE computations, see Figure 4, [8].



a) b)

Figure 3. a) Normalized multiplication factor by the gravitational acceleration and the considered failure mechanism for limit analyses; b) Considered possible failure mechanisms of the church macro-elements.

The four failure mechanisms in Figure 4 are identified using FEs and applying horizontal loads along the longitudinal and transversal directions. As can be noted, the most vulnerable elements are the tympanum and the upper part of the apse, both of them exhibiting very low collapse multipliers. The corresponding deformed shapes fit well the actual damage state of the church, compare Figure 2 with Figure 4. The other considered direction (transversal) exhibits relatively higher collapse multipliers, but still too low if compared with the peak ground acceleration (PGA) by the Italian Code. Summarizing, all limit analyses results address a high seismic vulnerability of the church.

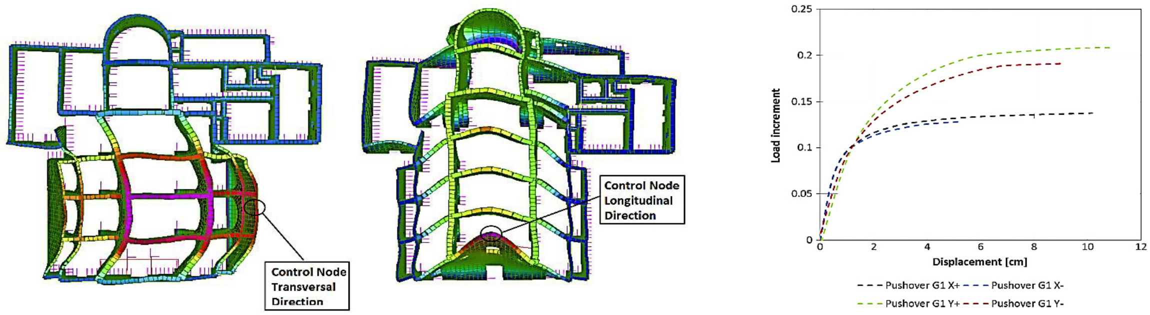


a) b) c) d)

Figure 4. Deformed shapes from FE limit analyses. a) Apse top portion overturning (*Y*+); b) Façade out-of-plane failure/tympanum rocking (*Y*-); c) Central nave longitudinal walls overturning(*X*+); d) Central nave longitudinal walls overturning (*X*-)

5. Pushover analyses results

The church has been subjected to eight pushover simulations, one for each direction (four directions are investigated, namely the longitudinal direction X+/- with positive and negative verse and the transversal direction Y+/-, again with positive and negative verse, as well as two load cases called by the Italian Code as G1-linear and G2-constant), iteratively choosing a control point belonging to an active failure mechanism, refer to Figure 5-a). The results of the pushover analyses in terms of node displacement and load increment are depicted in Figure 5-b). It was found, as it almost always occurs for such kind of structures that G2 load distribution is associated with a greater total base shear when compared with that provided by the corresponding G1 distribution. For this reason, hereafter, only results obtained with G1 load distributions are shown for the sake of conciseness. It is worth noting that the limit multiplier indicated in the y-axis of each curve represents, as by explicit choice of nodal forces applied to the models, the ag/g ratio between the horizontal acceleration associated with the activation of a failure mechanism and gravity acceleration. As can be noted, the acceleration at collapse ag is sometimes surprisingly low. As a rule, due to the general symmetry of the structures along their longitudinal axes, no meaningful differences are experienced between X+ and X- or Y+ and Y- directions. For this reason, here only one mechanism for X and Y directions is depicted for the sake of conciseness. As can be noted, failure mechanisms are local and usually involve either the façade or the apse for the longitudinal direction and lateral walls for the transversal direction.



a) b)

Figure 5. a) Deformed shapes at collapse from pushover analyses along transversal and longitudinal directions. b) Pushover capacity curves.

Pushover results estimate an overall collapse multiplier ranging between 0.12 g and 0.2 g, again consistent with previous outcomes. It is also worth noting, along with the high vulnerability, the good agreement of the failure mechanisms found with both those provided by limit analysis and what observed in reality. Considering the particularly high vulnerability of the church, a seismic upgrading is needed and requested to the authors by the owner of the church (Curia di Ferrara).

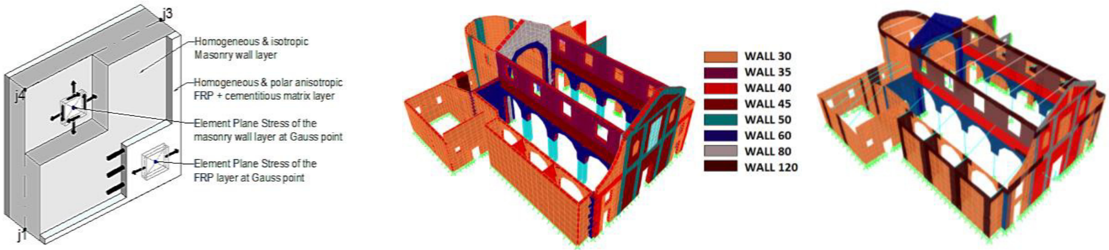
6. CFRPs retrofitted model

A comprehensive retrofitting strategy is conceived for the church under investigation. In particular, all the vulnerable macro-elements are upgraded in terms of strength capacity suitably using CFRP strips. The material properties adopted in the FE simulations are in agreement with indications by the literature and take into account both the almost null compressive strength of the strips (for buckling) and the reduced strength in tension, that takes into account possible delamination phenomena. The mechanical properties and the geometry of fibers implemented in the FE model are given in Table 1.

Table 1. Mechanical Properties of the CCFRP and the implemented fibbers for each wall.

Mechanical/physic char.	W [kN/m3]	fm [MPa]	τ_o[MPa]	E[MPa]	G[MPa]	ν	ε_u [%]
Mechanical/physic value	17.5	-	3700	230000	-	-	1.5
Wall Type	Wall 30	Wall 35	Wall 40	Wall 45	Wall 50	Wall 60	Wall 80
CFRP (mm/cm/cm)	0.4/20/40	0.4/20/40	0.4/20/40	0.4/20/40	0.4/20/40	0.4/20/40	0.4/20/40

The finite element implementation of retrofitted walls is done by mean of multilayer shell elements, available in SAP2000 FE commercial code, see Figure 6 – a). The CFRP strips plus the cementitious and adhesives layer are assumed applied on one or both surfaces of the shell element, ensuring a high strength along the preferential direction of the strip. In the present study, vertical walls of the chapels are upgraded in vertical bending by mean of vertical strips, whereas the façade is wrapped with horizontal strips anchored on perpendicular walls. In order to achieve interlocking between orthogonal walls, suitable anchoring systems must be designed.



a) b)

Figure 6. Detail of CFRP retrofitting for masonry walls. a) FE model of the church, wall thicknesses legend (in mm) and position of CFRP strips depicted in black.

The final configuration and the position of CFRP reinforced elements is depicted in Figure 6 - b) where a comparison between the actual situation (unreinforced case) and the retrofitted model is available. Obviously, the elements exhibiting a critical horizontal bending condition are retrofitted with horizontal strips, as for instance the upper part of the façade. The interlocking of the façade with nave walls is secured along two levels through the height. Some local interventions are also conceived on chapel walls, disposing vertical strips.

7. Nonlinear dynamic analyses results comparison

In non-linear dynamic analyses an acceleration time-history at the base of the structure is applied, defined by natural or artificial records. Such method may be time-consuming, but it is much more accurate and reliable than other approaches for its capability to identify in- and out-of-plane, as well as local and global failure mechanisms. An accelerogram registered on 20th May is utilized in the unreinforced case and scaled considering higher peak ground acceleration, equal to 0.156 g instead of 0.132g, for the reinforced structure. Indeed, it is considered reasonable to carry out numerical analyses with higher PGA in the reinforced case, just to have an insight into additional resources present in the strengthened case. To summarize, four mono-directional nonlinear dynamic analyses are performed considering the PGA mentioned here, two for the unreinforced and two for the reinforced case respectively. Results depicted in Figure 7 represent time history displacements of the macro-elements top node in the un-retrofitted case. When analyzed together with deformed shapes, it is reasonable to conclude that they are in fair agreement with damage patterns and failure mechanisms observed in reality. Damages are concentrated in all those macro-elements such as the façade, the apse, the nave and partially the triumph arch that result active also in reality. The results of the retrofitted model are reported in Figure 8. It is worth mentioning that the accelerograms used are the same of the unreinforced case, but with higher PGA. The simulated analyses on the retrofitted model exhibit an elastic response of the system, showing a strong seismic upgrading obtained through the introduction of CFRPs. In the models, it is also required that stresses in the retrofitting layer for each element are lower than a threshold level corresponding to the conventional delamination stress, evaluated in agreement with Italian Code and considered here equal to about 120 MPa. In addition, the stress is imposed to be non-negative, to avoid spurious effects due to strip buckling. Authors observed a maximum stresses on CFRPs not exceeding 12 MPa, quite far from delamination.

8. Conclusions

A numerical strategy in agreement with Italian Code recommendations has been applied to a valuable case study, namely a small/medium size Romanesque church damaged by the recent Emilia Romagna 2012 seismic sequence. A variety of non-linear analyses has been performed, including limit analysis (with both pre-assigned failure mechanisms and using FEs), pushover and non-linear dynamic analyses. A possible seismic upgrading intervention made with CFRPs has been also investigated, finding that the efficiency of retrofitting is quite high, also considering the application easiness. A global performance improvement, which ensures almost an elastic behaviour for the whole structure was observed. The CFRP stress always resulted within the imposed elastic limits, moreover far from the allowable stress limits corresponding to delamination and local instability in compression.

9. Acknowledgements

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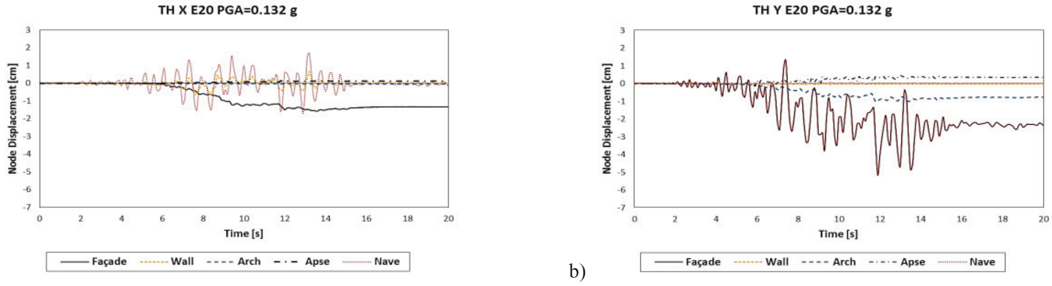
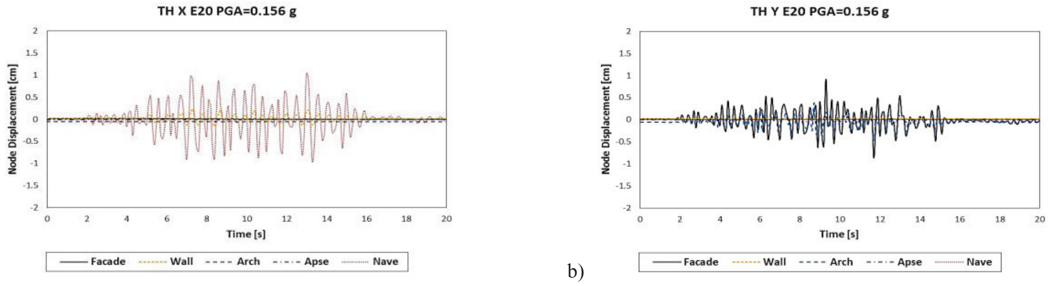


a) b)

Figure 7. Dynamic response of the unreinforced church, macro-elements top nodes time histories displacements. a) Time History response, acceleration applied along “X” longitudinal direction; b) Time History response, acceleration applied along “Y” transversal direction.



a) b)

Figure 8. Dynamic response of the CFRP reinforced church, macro-elements top nodes time histories displacements. a) Time History response, acceleration applied along “X” longitudinal direction; b) Time History response, acceleration applied along “Y” transversal direction.

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