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DAMAGE SURVEY AND STRUCTURAL ASSESSMENT OF THE ROSARIO CHURCH IN FINALE EMILIA AFTER THE MAY 2012 EARTHQUAKE IN EMILIA-ROMAGNA, ITALY

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Abstract. *The paper is aimed at presenting some results regarding the causes of damage of a masonry church (Chiesa del Rosario) located in Finale Emilia, a town stricken by the May 2012 Emilia Romagna seismic event. The church exhibits a diffused state of damage on the façade (rocking along a horizontal hinge), lateral nave walls (shear), transect (shear and out-of-plane bending) and bell tower (detachment at the connection of the tower to the church and subsequent rotation along a vertical axis). The approach adopted is multidisciplinary and it is conducted within a research project that involves the Technical Universities of Milan, the Technical University of Bari and the Italian Cultural and Architectural Heritage Ministry (MiBAC). The survey analyses conducted comprise a detailed geometric recover of the volumes and materials used, basing on post-seismic photogrammetric studies, historical research on existing documentation and repeated in situ investigation. Preliminary structural analyses are conducted in agreement with the Italian Code. In particular, full pushover analyses on a detailed FE discretization are performed, providing an estimate of failure seismic acceleration and automatically identifying the most critical parts of the church as well as the failure mechanism active, to compare with the 28 possible local collapse mechanisms provided by the Italian Guidelines on the built heritage.*

1 INTRODUCTION

The evaluation of the seismic vulnerability of masonry churches [1]-[12] still remains a fundamental task in highly civilized countries. This is particularly true for the Italian Emilia-Romagna region, where the devastating 5.9 and 5.8 magnitude shakes, occurred respectively on the 20^{th} and 29^{th} of May 2012, caused both the partial or total collapse of many historical structures and damages of monuments located in the wide area stricken by the seismic event. It has been recently calculated that, only in the provinces of Modena, Ferrara and Bologna, the churches considered unsafe after the seismic sequence are more than 500.

Generally, all existing masonry structures are rather vulnerable to earthquakes [13]-[16], but churches in particular are not conceived to properly withstand horizontal loads. Their peculiar geometry, usually constituted by very long and slender naves carried by slender columns, façades scarcely interconnected with the perpendicular walls and built with materials exhibiting very low tensile strength, presence of flexible wooden roofs, do not allow them to properly withstand horizontal actions.

The post-earthquake problems still open are (1) to evaluate the most suitable strategy of rehabilitation for such kind of structures, simultaneously limiting times and costs of interventions and (2) to have a precise insight into the residual resistance of un-collapsed structures [17].

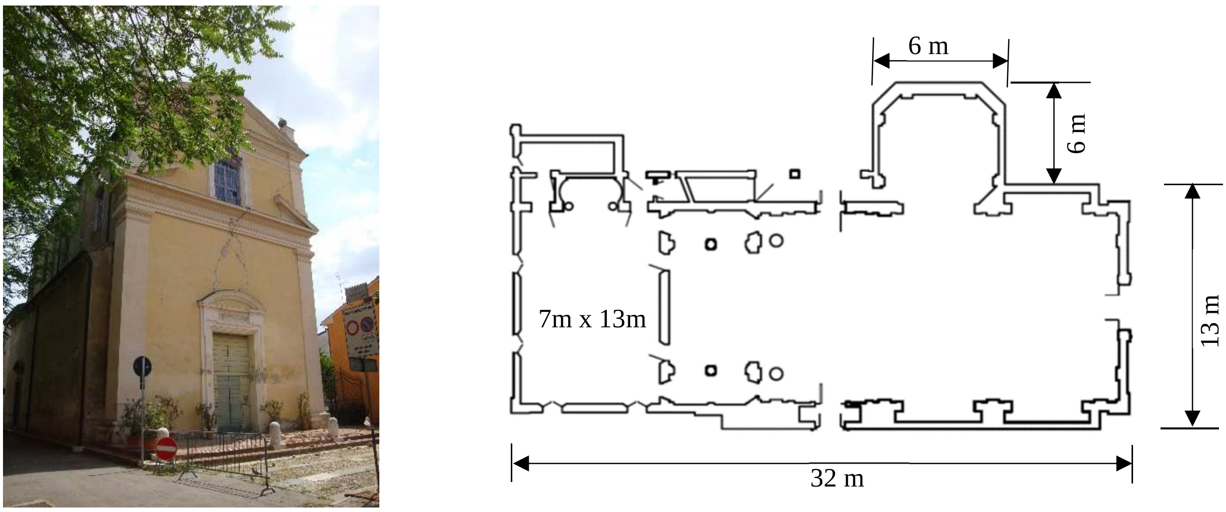
All the aforementioned tasks are quite specific and hard to be tackled for churches that, as already pointed out, cannot be reduced to any standard static scheme. At present, the most diffused approach adopted in practice, as recommended by Italian Guidelines for the Cultural Heritage [18], is the utilization of the kinematic theorem of limit analysis for no tension materials, with pre-assigned failure mechanisms. A collection of 28 possible partial failure mechanisms (according to their statistical occurrence observed in previous earthquake surveys) is assumed as the most probable. Seismic vulnerability considerations are therefore associated with the identification of the failure mechanism linked to the lower value of the collapse multiplier that, in the model adopted, coincides with the non-dimensional horizontal acceleration at collapse. Possible failure mechanisms comprise façade and tympanum overturning, apse shear and rocking failure, triumphal arch four-hinges mechanisms, etc.

Whilst the approach proposed by Italian Guidelines is very straightforward, easily applicable by everyone, even not familiar with limit analysis concepts and provides upper bounds of the actual collapse multiplier with pre-assigned failure mechanisms, it has two rather relevant drawbacks. The first is linked to the risk of an overestimation of the horizontal acceleration at failure; as a matter of fact, indeed, the upper bound theorem of limit analysis states that the true collapse multiplier is the lower bound of the set of all multipliers kinematically admissible. Therefore, if the actual failure mechanisms activating a lability in the structure are different from those contained in the collection of possible mechanisms provided by Italian Guidelines [18], an overestimation of the multipliers occurs. Secondly, the utilization of a no-tension material model, whilst for sure representing a safe approach from a material standpoint (masonry is scarcely resistant to tensile stresses), does not take into account several peculiar characters exhibited by brickwork, which may play a relatively important role in the formation of the failure patterns. Among the others, the most important may be orthotropy at failure and the actual texture, especially along the thickness of the wall. This latter feature considerably influences the monolithic behavior against out-of-plane loads.

In the general framework of the study of masonry churches up to failure under increasing static horizontal loads, a case study is analyzed here, consisting into the structural analysis of a church (Chiesa del Rosario) located in Finale Emilia, a town very near the epicenter of the 20 May 2012 seismic event, which exhibits a diffused state of damage on the façade (rocking along a horizontal hinge), lateral nave walls (shear), transect (shear and out-of-plane bending) and bell tower (detachment at the connection of the tower to the church and subsequent rotation along a vertical axis). The approach adopted is multidisciplinary and it is conducted within a research project that involves the Technical Universities of Milan, the Technical University of Bari and the Italian Cultural and Architectural Heritage Ministry (MiBAC). The survey analyses conducted comprise a detailed geometric recover of the volumes and materials used, basing on post-seismic photogrammetric studies, historical research on existing documentation and repeated in situ investigation.

The structural approach adopted is a global pushover analysis conducted with a commercial code (namely Strand7 [19]), assuming that masonry behaves as an isotropic elastic-perfectly plastic material, obeying a Mohr-Coulomb failure criterion. Despite the fact that the material assumption is rather simplistic for masonry, it is allowed by Italian Code [20][21] and has been utilized many times in the recent past for the analysis of similar problems [11][14][16]. In addition, it allows performing non-linear analyses by means of different computer codes available in the market and now widespread in the professional environment too. As a matter of fact, indeed, the elastic-plastic material model is probably the simplest one that can be made beyond linear elasticity. Despite the fact that the global pushover curves so obtained do not exhibit a drop of the load carrying capacity, a feature required by Italian Code of practice NTC 2008 [20][21] for a reduction of the multi-DOF (degree-of-freedom) to a single-DOF system and subsequent global assessment, Italian Guidelines on the built heritage allow the reduction of the capacity curves by means of an alternative simplified procedure. Such a procedure is particularly suited when it is not possible to reduce the structure to an equivalent frame, where the softening branch may be managed by the software more easily, since the non-linearity is concentrated exclusively in plastic hinges. When dealing with 3D FE models of masonry churches and towers, where gravity loads play a crucial role in the load carrying capacity under horizontal actions, a drop of the load carrying capacity is usually much smaller, even in presence of damaging materials. While the FE pushover model is global, the reproduction of partial failure mechanisms is obviously still possible [5][6], especially when the in-plane stiffness of the floors and the roof is totally neglected.

From simulations results, it is found a good agreement between numerical and real failure mechanisms active and a totally insufficient capacity of the structure to withstand properly the horizontal accelerations induced there by the seismic sequence.



(a) (b)

Figure 1: (a) Chiesa del Rosario (b) Plan view of the Church

2 GEOMETRIC SURVEY AND STATE OF DAMAGE

The Rosario Church, see Figure 1, is a historical unreinforced brick masonry structure located in Finale Emilia, Italy. Geometrically, it is constituted by a large single nave 20 meters long and 13 meters wide and a presbytery with rectangular shape with approximate dimensions equal to 7 x 13 m. Roughly, the structure has rectangular dimensions equal to 32 × 13 m, with an average thickness of the walls equal to 50 cm. The height of the façade is around 17 m and a two story building 13 m high is directly attached to the back of the apse. The interior walls are supported by semicircular arches and columns. The bell tower has a height of around 20 meters and is partially connected to the church at one corner of the structure.

An accurate survey is the main guarantee of a correct knowledge about the structure, especially in cases such as that analyzed here, in which the building, seriously damaged, must be studied in detail in its material, dimensional and physical form after the seismic event, highlighting the state of damage in a proper manner.

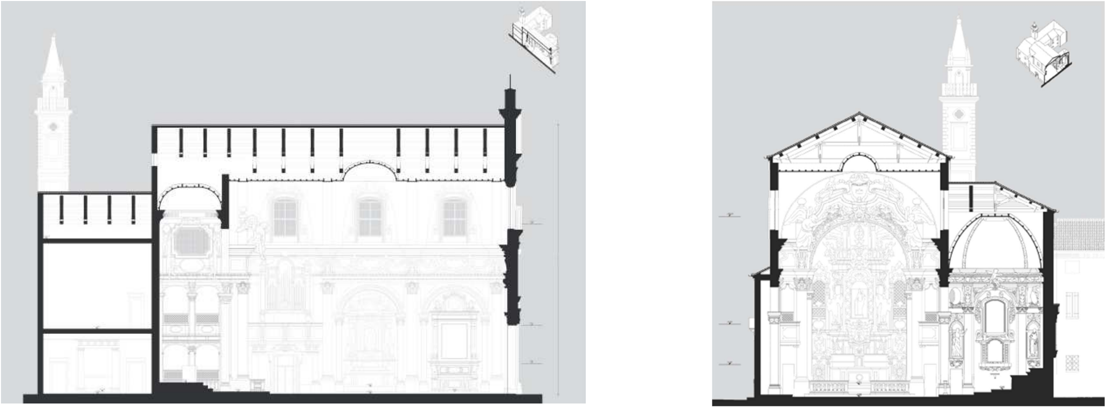
The geometric survey is the most suitable way that has allowed to understand the architecture in its articulation, identifying relationships between parts of the building that would escape to a concise observation. At the same time, it has been strongly useful to highlight immediately features and peculiarities as the thicknesses of the walls, misalignments, irregularities, and therefore also providing a partial insight into the materials that constitute the structure.

To carry out the survey of the church, the first approach used was a celerimetric survey of the external volume and the internal envelope in order to derive an indirect estimation of external walls thickness and the relationship between the interior and the exterior part of the structure. The fact that the church is inserted into a built aggregate, however, allowed the placement of topographic stations for celerimetric mapping for the survey of only two of the four exterior elevations and a partial characterization of the interior parts, because of the presence of scaffolds to safeguard the building after the seismic event. The celerimetric survey was then deepened with direct methods, using the same scaffolds that previously precluded the indirect survey. Such temporary structures enabled, in opposite, to approach -even in elevation- the structure.

The cornices, the upper ends, the openings and some decorative elements were restituted through mapped points by means of the total station used in celerimetric surveying.

The interior elevation, in the basement, was found through a series of triangulations based on a fixed uniform quote-plan that allowed resolving the presence of an altitude difference in the floor of the church. The upper part was surveyed by composition of the different methods described above.

The decorative details such as capitals, moldings, corbels and statues, and structural elements subject to dislocations and instabilities were cataloged with photographs and sketches that would indicate the main measures to perform for a photogrammetric restitution through interpolation of celerimetric data on a photo-modeling program that uses photos kept with a metric-camera. In this way, a cloud of points similar to that provided by laser scanner was created, ensuring the faithful reconstruction of the 3D model. Some transversal sections of the church so derived (after post processing) are depicted in Figure 3.



(a) (b)

Figure 2: (a) longitudinal and (b) transversal section obtained from standard topographical survey and photo-grammetry.

From the conducted survey, it was found that the church exhibits a diffused state of damage on the façade (rocking along a horizontal hinge, with detachment from perpendicular walls), lateral nave walls cracks due to shear (especially in correspondence of openings), transect partial collapse for shear and out-of-plane bending and a quite relevant detachment at the connection between the bell tower and the church, with subsequent rotation along a vertical axis of the tower.

3 BUILDING GEOMETRY AND MODELLING

For the computational analysis, see Figure 3, the structure is modelled using a combination of 6 noded wedge elements and 8 noded solid elements. The non-structural elements like floors, ceilings, roofs, stairs that are made of wood are not considered in the FE computations and therefore their limited stiffness is totally neglected. In this way, it is expected to obtain conservative results. Vertical dead and live loads and equivalent horizontal seismic static action are directly applied to masonry walls. All the degrees of freedom of the nodes located at the base of the structure are fully restrained. The material is assumed to be isotropic and the typical mechanical properties used are kept in agreement with Italian Code provisions. The modulus of elasticity is considered to be 2000 MPa, Poisson's ratio is taken equal to 0.2 and the density is chosen as 2000 Kg/m^3.

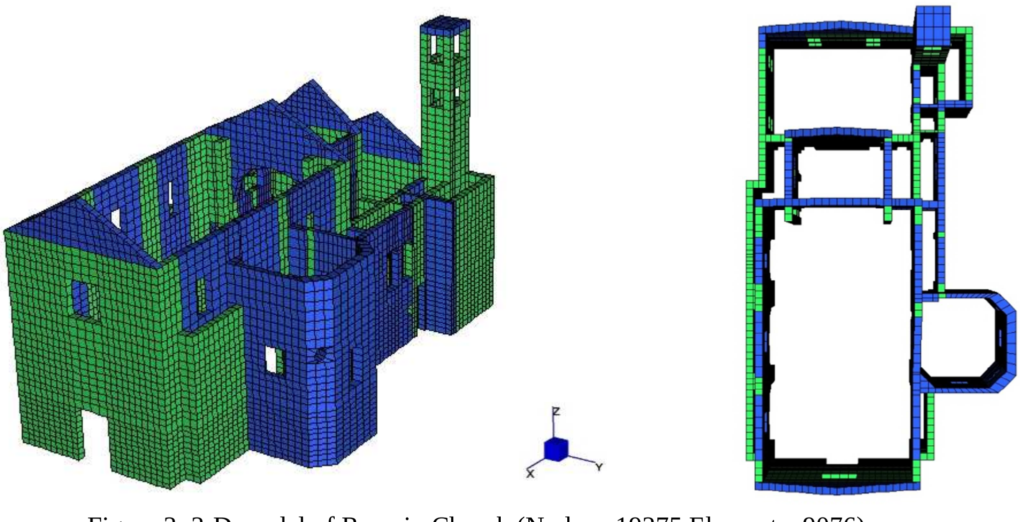
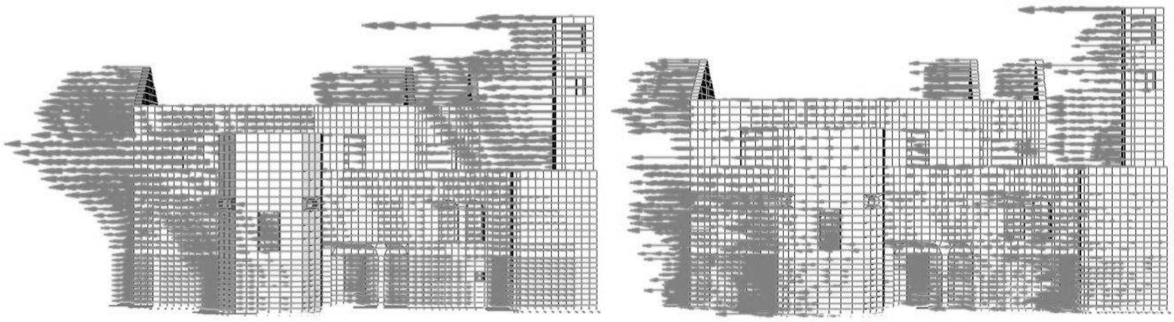


Figure 3: 3-D model of Rosario Church (Nodes= 19275 Elements=9076).

The non-linear static pushover analyses are carried out using the commercial code Strand7 [19]. The hypothesis of elastic perfectly-plastic behavior has been made for masonry, within the general scheme of isotropic media. A Mohr-Coulomb failure criterion with a friction angle of 30^o and cohesion of 0.15 MPa is initially considered considered. Such a high cohesion is unlike for masonry material, but it is the higher value that can be adopted according to the Italian code making the most favorable hypotheses on the thickness of joints, texture quality along the thickness, etc. Horizontal loads are applied in agreement with G1 and G2 distributions, in agreement with Italian Code specifics [20], see Figure 4. G1 and G2 distribution are labeled as Load Case 1 and Load Case 2 in the numerical computations reported hereafter. As expected, G2 distribution is always less conservative than G1. This occurs almost always for masonry churches. Therefore, vulnerability considerations are usually made with reference to G1 pushover response.



(a) (b)

Figure 4: Load in positive X direcxtion (a) Nodal load distributin for load case G1 (b) Nodal load distribution for load case G2.

4 NUMERICAL ANALYSIS AND RESULTS

The non-linear static pushover analyses are carried out for two load cases (G1 and G2) in both positive and negative directions along the longitudinal (X) and transversal (Y) church directions.

As can be noticed from Figure 5, where the deformed shape at the last iteration for G1-X+ case is shown, the non-linear pushover analyses along the longitudinal direction (with loads from the apse to the façade) results into the formation of a failure mechanism that involves façade overturning (upper part), bell tower failure and damage of lateral walls of the central nave. The general state of inelastic deformation and the active failure mechanisms appear in satisfactory agreement with the actual state of damage occurred in reality.

In Figure 6, the pushover curves obtained with the numerical model discussed are represented for all the load cases investigated. Subfigure (a) refers to horizontal loads acting along the longitudinal direction, whereas subfigure (b) to the transversal direction. Along y-axis, the non-dimensional quantity a_g/g is represented, where a_g is the horizontal acceleration applied, i.e. the resultant static seismic load applied dived by the total mass of the structure.

Such a representation allows determining in a very straightforward manner the collapse acceleration that can be carried by the structure under the numerical and theoretical hypotheses done, to be directly compared with spectral acceleration provided by the Italian Code.

As can be seen, collapse a_g/g is rather high, even for G1 distribution. Such result does not completely justify the state of damage induced by the earthquake occurred and is mainly linked to the high cohesion assumed for masonry.

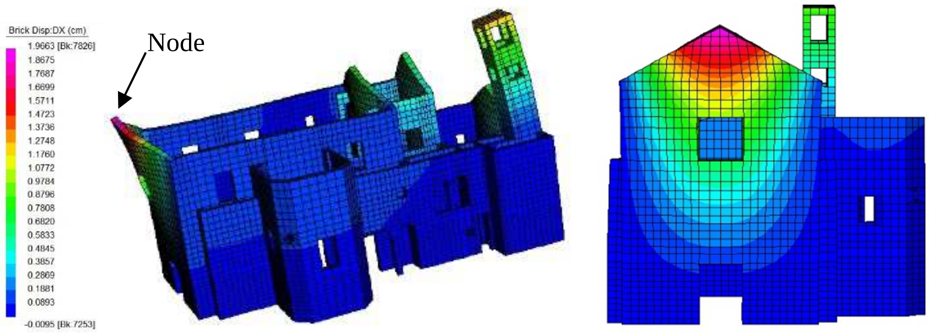


Figure 5: Deformed shape at collapse with the contour plot of maximum displacement due to Load case I in Positive X Direction.

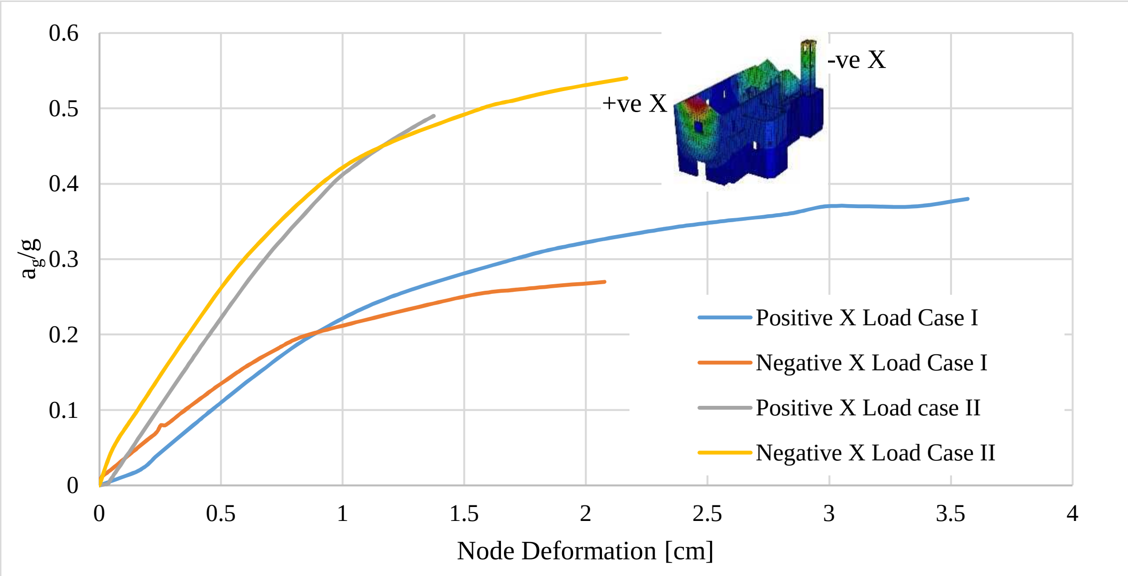
In order to have an insight in such issue, a sensitivity analysis is conducted, reducing the cohesion value and, in particular, investigating the response of the church for c= 0.10 and 0.05 MPa. It is expected that the analysis with the lower cohesion, which well approximates the situation of a no-tension material, is the nearest to the reality.

Results obtained in terms of non-dimensional pushover curves are depicted in Figure 7 and Figure 8 for c = 0.10 MPa and c = 0.05 MPa respectively.

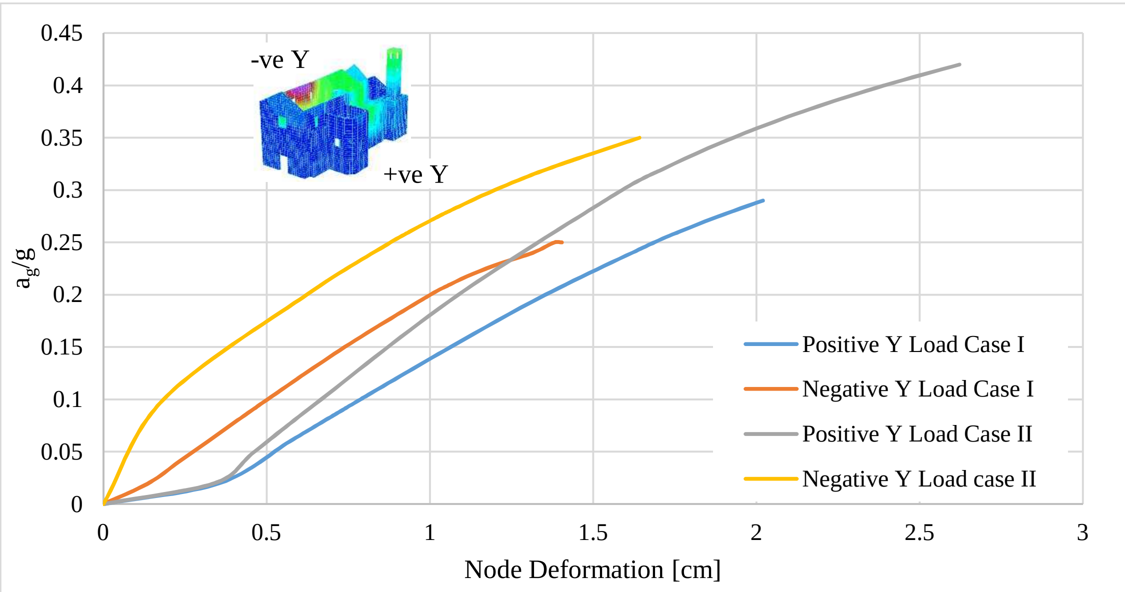
As can be seen, active failure mechanisms do not change considerably when compared with those found for a higher cohesion value. However, the maximum acceleration found, corresponding to the collapse of the structure, sensibly reduces. In particular, a gradual reduction in capacity of the pushover capacity curves generated with different cohesions is observed.

Especially for the case with very low cohesion, it is found that an acceleration lower than 0.1g is responsible for the collapse of the structure, which appears reasonable in light of the experience collected with damages induced by the seismic sequence and the knowledge about the characteristics of the accelerograms.

It is interesting to notice that, along the transversal direction, the activation of a partial failure mechanism involving one of the long lateral walls of the central nave and the bell tower, occurs for an acceleration slightly lower than that found for the longitudinal direction. Visible inelastic shear deformation may be also noticed on the façade, a result which appears consistent with that observed in practice.

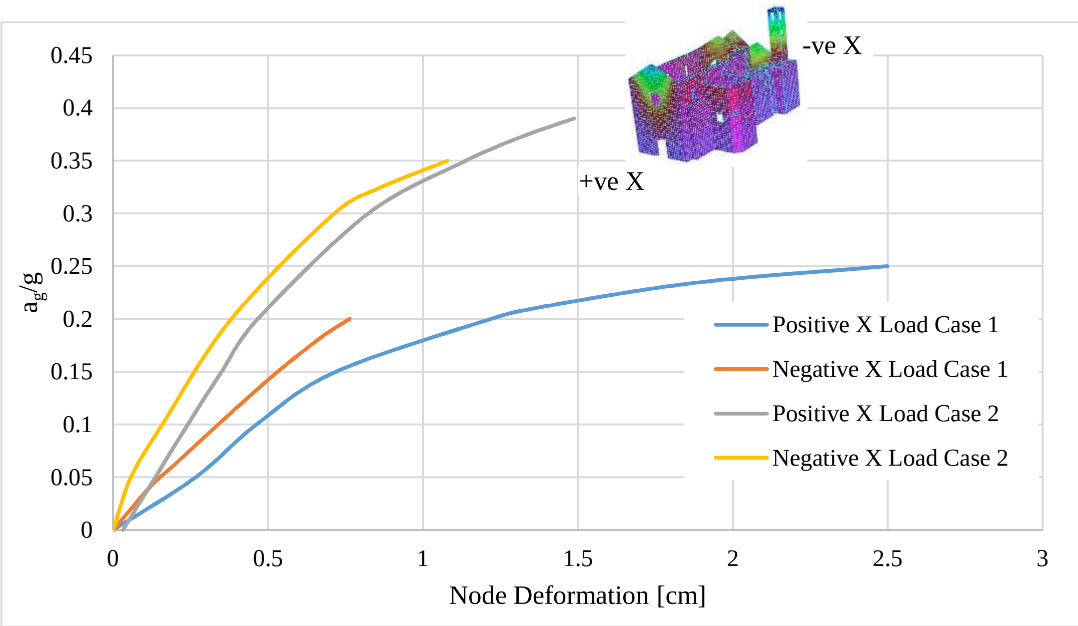


(a)

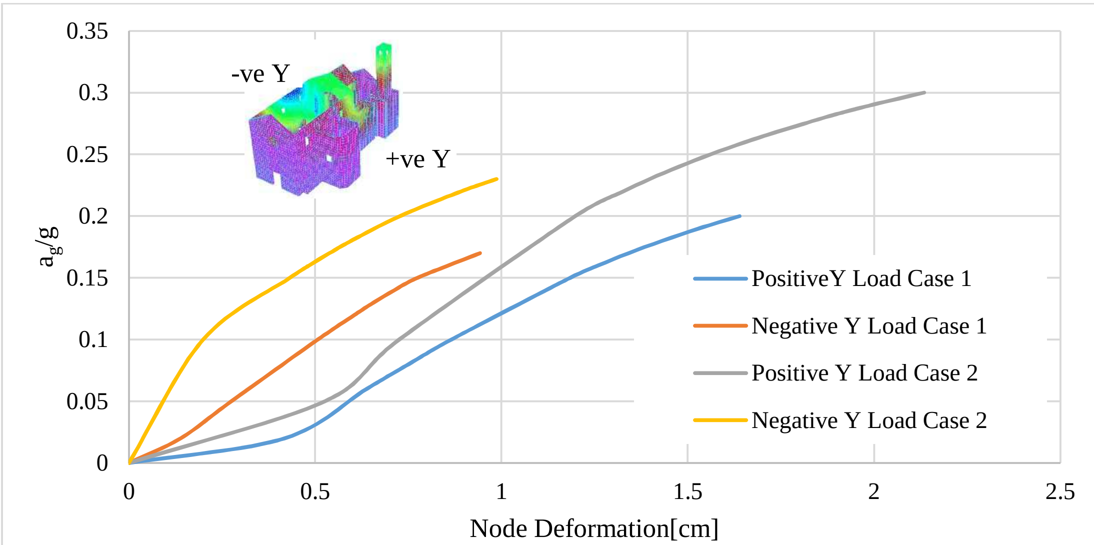


(b)

Figure 6: Pushover curves with cohesion 0.15 MPa (a) Load applied in X Direction (b) Load Applied in Y direction

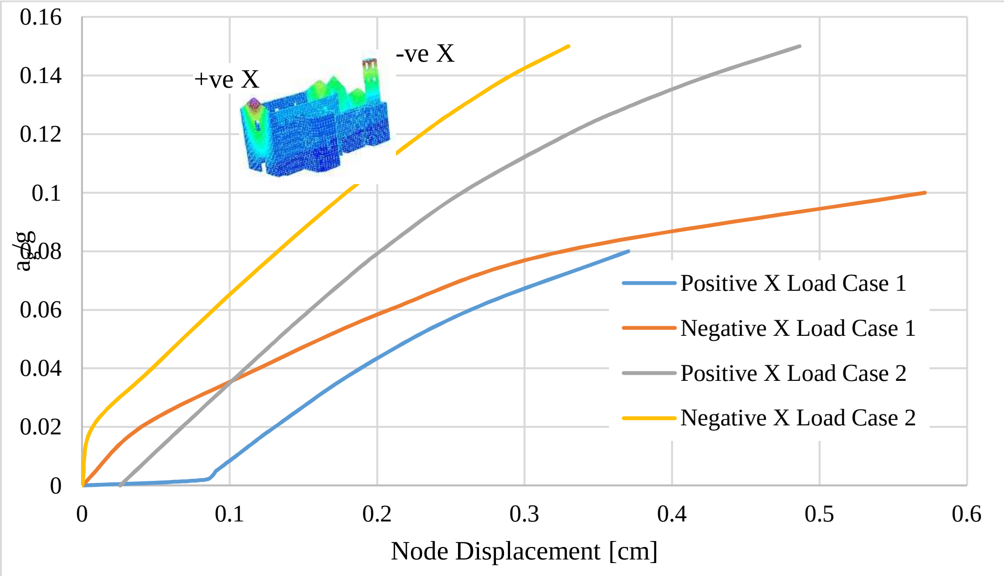


(a)

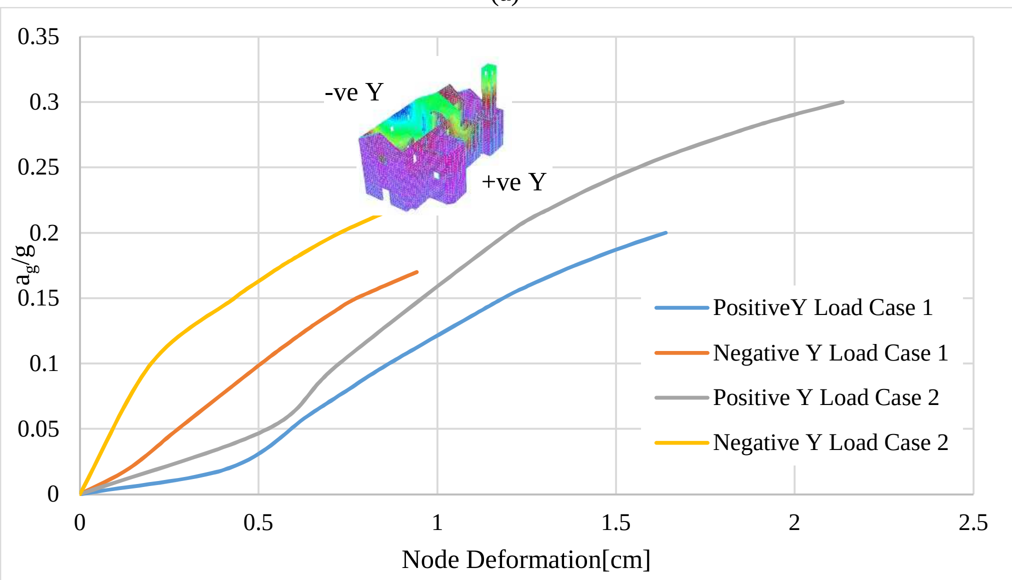


(b)

Figure 7: Pushover curves with cohesion 0.1 MPa (a) Load applied in X Direction (b) Load Applied in Y direction



(a)



(b)

Figure 8: Pushover curves with cohesion 0.05 MPa (a) Load applied in X Direction (b) Load Applied in Y direction

5 CONCLUSIONS

In the paper, some detailed numerical results regarding the pushover static analysis of a masonry church (Chiesa del Rosario) located in Finale Emilia, a town stricken by the May 2012 Emilia Romagna seismic event, have been reported. The aim was to have a quantitative insight into the main factors that caused damage and to identify the most vulnerable structural elements, in light of an effective restoration intervention. The church exhibits a diffused state of damage on the façade (rocking along a horizontal hinge), lateral nave walls (shear), transect (shear and out-of-plane bending) and bell tower (detachment at the connection of the tower to the church and subsequent rotation along a vertical axis). The approach adopted is a multidisciplinary one and hase been conducted within a research project that involves the Technical Universities of Milan, the Technical University of Bari and the Italian Cultural and Architectural Heritage Ministry (MiBAC). Results obtained with the pushover analyses highlight that the façade, the bell-tower and lateral walls are critical and their collapse is associated to a quite low horizontal acceleration, especially in presence of masonry with poor mechanical properties. Such results appear consistent with the damage surveys conducted by the authors and provide useful hints for effective and optimized restoration.

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