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Vulnerability of Historic Religious Buildings in Nera's Valley, Italy

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***Abstract* – This article presents a study carried out in the Nera's Valley of Umbria, Italy, funded by the ReLUIS Programme. The aim is to identify recurring collapse mechanisms for historic religious buildings, which, while providing important information for the improvement of their seismic performance, will be useful for a wide range of masonry constructions with similar geometrical parameters, traditional construction techniques and constituent materials. Three earthquake-damaged churches were studied and the collection and analysis of the data obtained from the survey have been discussed in order to gain information regarding their structural behavior. This has been conducted according to some commonly recognized vulnerability indicators.**

I. INTRODUCTION

Religious buildings, for their age and geometrical parameters, represent a peculiar class of construction, whose local and global structural behaviors have to be studied to define their vulnerability in earthquake-prone areas[1][2][3][4]. In 2016, the Nera's valley, in Umbria, Italy was struck by a destructive earthquake. The most part of structural collapses was produced by out-of-plane mechanisms of wall panels, mainly the churches' façades. These mechanisms were often facilitated by no connection with the orthogonal bearing walls or the roof structural scheme[5][6]. The façade's behavior corresponded to the structural scheme of a vertical cantilever. The existence of vaulted structures within the buildings also contributed to activate this collapse mechanism. In some cases, damages were produced by adjacent or interconnected structural elements, like the collapse of the St. Eutizio Church inPreci or the Basilica of St. Benedict in Norcia, both caused by the ruin of the adjacent bell tower. In many cases, the large width of the lateral wall panels and the limited thickness of these walls were the main cause of their collapse, with the subsequent ruin of the roof[7][8]. The dismissed church of St. Francis and the church of St. Augustine in Norcia are interesting case-studies of this mechanism. The collapses of these churches would have been difficult to prevent using standard retrofitting methods[9]. Only the application of buttresses could solve this problem. Hundreds of religious buildings collapsed during the 2016 earthquake in Umbria.

Two churches are studied in this paper: St. Andrew and St. Mary. All of them are located in Campi, near Norcia and they collapsed due to the seismic events dated 26 and 30 October 2016.Little attention was paid to their structural behaviour during previous restoration interventions. The authors believe the implementation of simple, cost-effective and non-intrusive retrofitting works could have been decisive. Nobody can say with confidence that their collapse would have been prevented by applying these interventions, but, for sure, their vulnerability would be reduced.

II. ST. MARY

A. Geometry

The Church of St. Mary (*Santa Maria in Piazza* in Italian) was funded 1331 and has experienced numerous earthquakes through the ages, before being completely destroyed by the 2016 earthquake. It has been has been repeatedly restored. The Church was located at the ground floor in the town centre of Campi, with a high level of interconnection between adjacent buildings, making it difficult to identify and isolate structural elements.

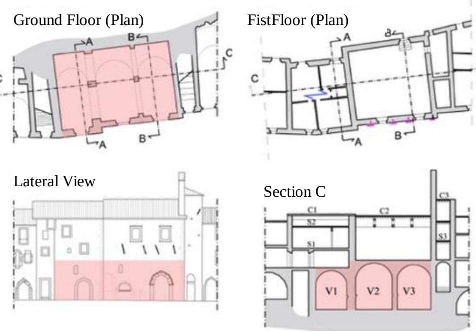


Fig. 1. Plan, section and lateral view of the Church of St. Mary[10].

The church consists of two masonry box-like structures (Fig. 1). Structure US-01 (right in Fig. 1) was located near a tower and it was a 2-storey construction. The wall panels of the ground floor have never been reinforced, while the walls at the first floor have been injected with new grout after the 1979 earthquake. The roof consisted of timber trusses, timber beams and a reinforced-concrete (RC) slab. A masonry vault was used to provide the space with a ceiling at the ground floor.

The second structure (US-02, left in Fig. 1) was only partially used for worship or related uses. The façade was not in-line with adjacent box-like structure. It was a 3-storey building with RC suspended beam and block flooring. The roof structure had a similar arrangement. The beams were supported by the lateral walls, perpendicular to the façade.

B. *Analysis of the damage*

At the ground floor, the vaulted structure was partially supported by two small stone pillars. These pillars were made of a soft type of granite, characterized by weak mechanical properties. Previous analysis [10] using ultrasonic testing demonstrated the existence of serious vertical cracks and material expulsion on one of the two. Results of further investigation [11] show that years of deterioration, limited maintenance, damage caused by past earthquakes, dead load increments (RC beam and block flooring) have caused a critical structural state, with diffuse cracking, especially in the vaults.

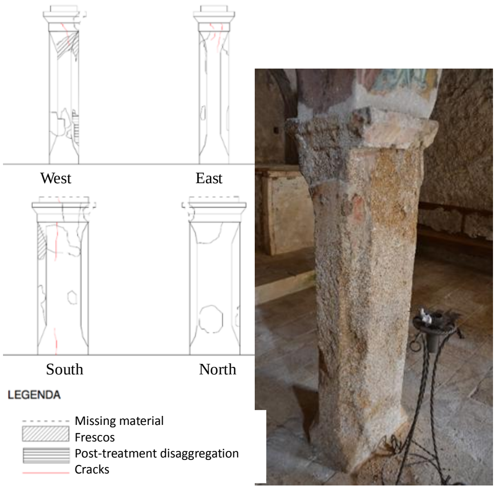


Fig. 2. Crack pattern surveyed on the granite pillar.

A high level of vulnerability was evident for the building's façade. The use of grout injection did not improve the structural behavior against out-of-plane collapse mechanism.The church survived the 24 Aug. 2016 earthquake with apparently little damage. Technicians have conducted a survey in September 2016 reporting damage to the belfry and the detachment of the façadefrom the vaults. No further damage was noted for the soft stone pillars.A video filmed after the 26 October 2016 earthquake show the existence of a long vertical crack on the façade, demonstrating an initial out-of-plane mechanism of the façade, likely prevented by steel ties. The 30 October 2016 earthquake caused the total collapse of the church and adjacent buildings (Fig. 3).

Fig. 3. The collapsed church in October 2016.

C. *Collapse mechanism*

It is difficult to state with any certainty which was the most important cause of the collapse. Surely, the cracked pillar is the main suspect. In fact, as the thin pillar was subjected to compressive stress, buckling occurred and cracks opened before the seismic event. Stresses in this pillarwerevery high as it supported the vaults, the thick bearing wall separating the two structures at the first level and, throughthis, the beam floors.

The calculation of the pillar's compressive stresses, only generated by the dead loads and the lateral thrust of the vaults, produced maximum values of approx. 20 MPa, not far from the compressive strength of the material (28 MPa), according to[10] and [12]. The 30 October 2016 earthquake had likely produced that additional compressive stress needed to provoke its failure. Figure 2 shows the vertical cracks caused by the static loads on the pillar's south face, as surveyed by [10]. These cracks are consistent with the crack pattern of the vaults and both are likely the consequence of an out-of-plane rotation of the façade of the second structure.

However, it cannot be ruled out that the collapse of the building initiated not with the pillar crushing, but with the out-of-plane rotation of this thin façade. Moreover there were no steel ties on this façade and the beam bearing ends werewell supported on the perpendicular walls and not on the façade.The façade's behavior corresponded to the structural scheme of a vertical cantilever with no constraint at the floor and roof levels able to prevent its out-of-plane rotation.Given the static vertical loads acting on the first level floor (filling material of the vault, pavements, etc.), the magnitude of the thrust of the vault was also very high, and this could be the origin of the façade rocking.

Another possible cause of the collapse of the church could be identified in the ruin of the adjacent bell tower. The bell tower was very slender and it is possible that this could have collapsedunder the action of the earthquake over the church by out-of-plane rocking. The beams of the church's roof were in part fixed to bell tower's walls and this connection would have been critical for the collapse of the church.

D. Numerical non-linear analysis

For this study a pushover analysis was used for capturing the seismic demands of the masonry structure[14][15][16]. The commercial software PCM was employed for the analysis by considering a 3D (three-dimensional) framed structure, as imposed by the current Italian code[13] (Figure 4). The roof top displacement was taken as the displacement of a point on the roof itself.

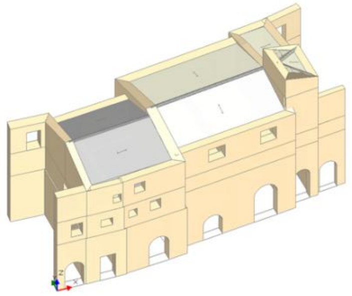


Fig. 4. Model used for pushover analysis.

It is noteworthy to point out that a conventional pushover was assumed in the study, i.e. loads applied on the building did not change with the progressive degradation of the building that occurs during loading. Considering that the building was interconnected with adjacent buildings and the beam and block flooring (rigid diaphragm), torsional effects were not studied and discussed. On opposite, wooden beam floors and vaults were modeled as non-rigid diagrams.

The following assumptions were considered for the analysis, as defined by the Italian Building Code: ground type B, occupancy category III, site class T2, ground acceleration a_g = 0.281g (g = gravitational acceleration). The numerical model also considered the contribution of several interconnected walls belonging to adjacent buildings. This was necessary in order to compute for their contribution to the overall stiffness of the masonry building.

For the mechanical properties of the construction materials of the masonry historic structure, the data provided by the Italian Building code for rubble stone masonry, applying a Knowledge Level LC1 and a confidence factor of 1.35, were used[13].

The horizontal lateral load curves in the sense of the rocky mountain rear of the building (sense +Y in Figure 0) were excluded from the analysis. The pushover analysis wasdone in both transverse and longitudinal directions and six base shear vs. roof top displacement plots were studied. The seismic vulnerability was conveniently detected by computing the risk index I_r, which is a number lesser than 1 that figures out how vulnerable the building is (the closer to 0 is the index, the more vulnerable is the building). For direction and sense (-Y) this was I_r = 0.691using a lateral load of a uniform vertical distribution pattern on the structure, corresponding to the direction of collapse really caused by the earthquake (Fig. 5). The return period T_r was 209 years for a limit state of Collapse Prevention (CP) and an actual residual life V_n of 14.6 anni.

On opposite, by considering direction X (Fig. 5b), the risk index I_r was 0.901 with limited seismic vulnerability. This was mainly because of the existence of lateral adjacent buildings ("terrace building effect") able to absorb and transfer to the foundation the seismic forces.

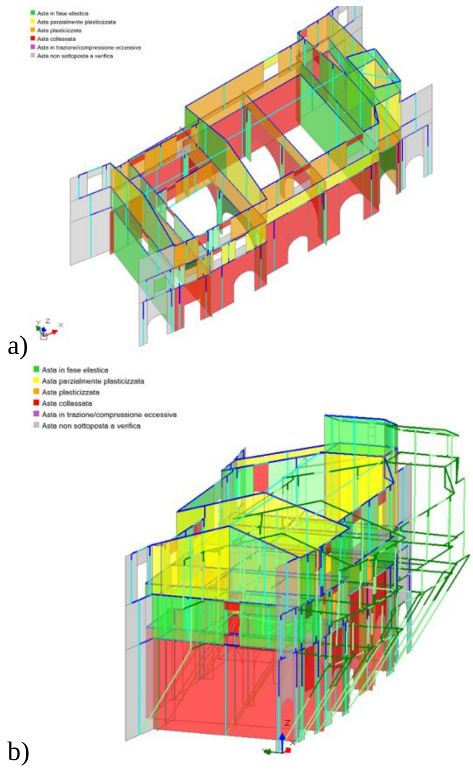


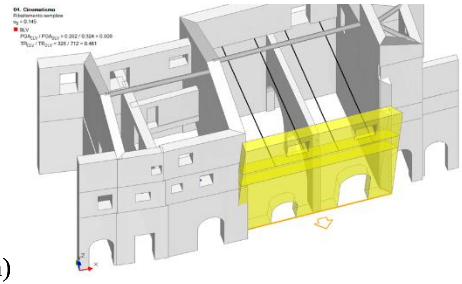
Fig. 5. Results of the pushover analysis: a) direction/sens –Y, b) direction/sense +X.

E. Linear Kinematic Analysis

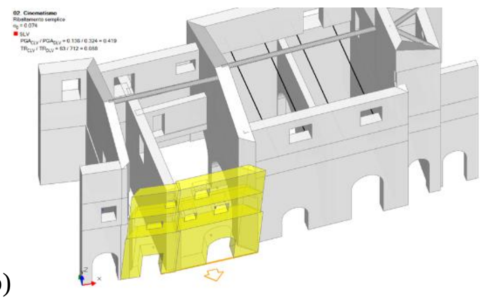
Local collapse mechanisms were computed using a linear kinematic analysis with a behavior factor q =2, as defined by the Italian Standard[13]. Mechanisms were defined based on the inspection and structuralanalysis of the masonry structure. The masonry structure was subdivided in rigid macro-blocks, able to translate and rotate, according to the compatibility equations derived from the external constraints[17][18].

The existence of the steel ties was considered in the kinematic analysis. On opposite, the effect of the ring beams was ignored given the fact that evidence seems to suggest their limited effectiveness in preventing out-of-plane mechanism when applied over rubble unreinforced wall panels[19].

Figure 6 shows three different out-of-plane collapse mechanisms. The corresponding risk index I_r varies between 0.311 and 0.808.



a)



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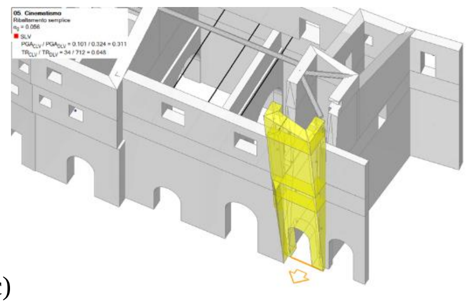


c)

Fig. 6. Collapse mechanisms: a) Facade of US-01, b) Facade US-02, c) Bell Tower

III. ST. ANDREW

A. Geometry

St. Andrews was a church located near the castle gate of Campi Alto. The church, erected in the 14^{th} century, was a single-nave masonry construction. A second nave was added in the 16^{th} century (Fig. 7).

During the 16th century a triangular-based portico in attachment of the main church facadewas also added. This was composed of 5 arches and was restored in 2009 with a new timber beam floor.

Both naves were covered over with cross vaults. The original apse was now used as sacristy: this was separated from the church with a wooden decorated wall, added in 1596. A massive bell tower was located over the apse: this was 15 m higher than the church (Fig. 8).

Fig. 7. St. Andrews

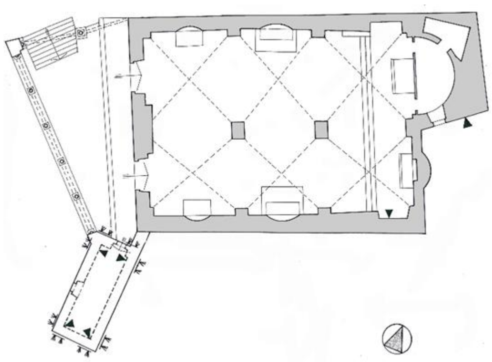


Fig. 8. St. Andrew's: plan (Cardani, 2013)

Indoor, an antique stone flooring and numerous frescoes decorated the church.A new roof was constructed in 1960s: this was made of Reinforced Concrete joists and metal ties. The lateral walls of the church (but not the façade) were reinforced with a RC beam at the eaves level. The roof ridge was made using a RC beam and the RC joists were placed parallel to the slope of the roof. A single wooden tie was placed parallel to the façade, during old restorations in the $18\text{-}19^{th}$ century.

Inside the church, the thrust of the vaults was not neutralized by any tie: this represented a significant vulnerability for the façade of the church against anout-of-plane collapse mechanism.

B. Analysis of the damage

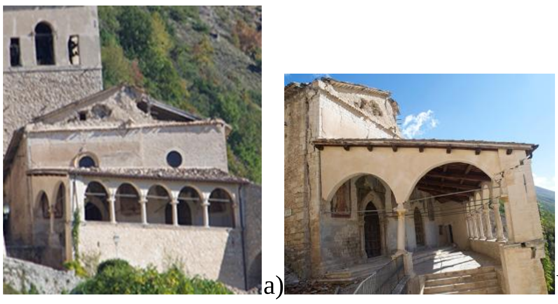
Following the 24 August 2016 earthquake, a structural survey, aimed to evaluate the damaged suffered by the church, showed the existence of a serious crack pattern on the cross vaults (Fig. 9a) and the detachment of the façade from the church's lateral walls (Fig. 9b).A second survey demonstrated that the cross vaults were unreinforced and without any in-fill material. On this occasion, a serious crack was noted on a masonry pillar adjacent to the façade, near the wooden tie (Fig. 10). This may have been generated by the action of the adjacent ring beam. It is likely that the overturning of the façade was prevented, during the first seismic event, by the action of portico, structurally efficient and with steel ties able to prevent out-of-plane collapse mechanisms of the walls and pillars.

a) b)

Fig. 9. a) Cracks in thr cross vaults, b) detachment of the façade from the lateral bearing wall

Fig. 10. Serious crack at the abutment of the arch in the area near the old wooden tie.

The subsequent 26 October 2016 earthquake caused a partial collapse of the church. Figure 11a shows the disaggregation of the upper part of the façade. In Figure 11b the overturning of the façade is clearly evident, but the action of the portico could again prevent it.



a) b)

Fig. 11. State of St. Andrew's church after the 26 October earthquake: a) front view, b) detail of the overturning mechanism of the façade.

It is worth noting that the façade was made of two different rubble stone masonry walls, constructed in different periods. During the seismic event, the two walls did not separate and the stone masonry only crumbled in the upper area. This was probably caused by the structural pounding of the timber beams during earthquake.

The 30 October 2016 earthquake struck an already seriously damaged structure and provided the final blow. The façade and the adjacent portico completely collapsed (Fig. 12). During earthquake, the wooden tie failed and the steel connections between the wood parts of the tie pulled out.

Fig. 12. St. Andrew's church: state following the 30 October earthquake

C. *Collapse mechanism*

The main seismic vulnerability of the building was the action of the stone masonry cross vaults (horizontal thrust), not effectively absorbed by the tie and lateral masonry walls. With the exceptions of the old wooden tie, there were no ties at all on this important listed structure. The façade behaved like a vertical cantilever and the joint action of the thrust and the seismic motion were the main cause of the overturning mechanism.

The portico contributed to prevent the out-of-plane mechanism of the façade. A positive action was also guaranteed by the three-hundred year old wooden tie, but the point of application of the thrust of the cross vaults was different from the location of the tie. Unfortunately, this thrust acted in conjunction with the seismic action and these were the main causes of the collapse of the roof and of the church's façade.

IV. CONCLUSIONS

Technical skills, needed for the evaluation of the seismic vulnerability of historic masonry constructions, are sometimes missing in existing employs of conservation statutory bodies. Several collapses of listed masonry buildings were discussed in this paper and most of them could have been avoided if preventive reinforcement interventions have had done. Unfortunately, a strict application of the principle of minimum intervention govern the decision-making process of conservation bodies.

We cannot conclude that conservation bodies are liable or to be blamed for these serious collapses of the historic buildingsof the Nera's valley, but it should be probably stated that this missing strategy in ways to protect these buildings and to prevent such serious damage during earthquakes, is the consequence of an inadequate knowledge of the structural behaviour of historic masonry constructions and their constitutive materials.

Failures of Campi's churches demonstrated this. All of them could be probably saved or the extension of the damage reduced, by applying preventive upgrading intervention, such as steel ties.The most evident example is St. Andrew's church: the old wood tie resisted three consecutiveearthquake motions (24 August 2016, 26 October 2016 and 30 October 2016). It is clear the application of some more ties in critical positions would have been sufficient to prevent the collapse of this church.

St. Mary's church is another striking example of the lack of attention and consideration to structural issues, even if a high seismic vulnerability was evident. Structural deficiencies were clearly denounced by previous scientific researches, but no actions were taken to protect this church with its frescoes on the walls. All that remained was a heap of rubble. It is likely that similar situations will occur again if no structural interventions will be decided to protect this important architectural heritage.

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