REPAIR AND SEISMIC RETROFIT OF A XV CENTURY BUILDING COMPLEX HEAVILY DAMAGED IN THE L'AQUILA EARTHQUAKE

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ABSTRACT
Micheletti Palace is one of the oldest building complexes in the Spanish castle area in L'Aquila. After the severe 2009 earthquake, the structure, composed of heterogeneous stone textured walls and timber floors, exhibited serious damage, with cracks, deformations, and local expulsion of the external stone leaf.
The main drawbacks of the building are connected to the typical geometry of the medieval buildings with a “stillicidio” wall pair cutting the building plan, and the presence of a great number of fireplace flues. Even the very bad mechanical properties of the typical L’Aquila mortar containing rounded sandy particles create an overall mechanical deficiency of the building.
The restoration design used non-destructive techniques in order to assess the status of the structure, and a number of hidden geometrical and mechanical features were clarified. Afterwards, a careful seismic analysis was performed on the original and restored configuration in order to assess the safety enhancement introduced by the new design.
The main interventions involved filling of the voids in the masonry material with injections and masonry reconstruction, including a framework of FRP and steel ties being placed down in order to guarantee the skeleton stiffness against geometry changes during the seismic motion.

KEYWORDS: stone masonry, historic building, seismic damage, retrofitting

INTRODUCTION
Micheletti Palace (figure 1) has arisen from a long path of modifications and additions to the original medieval building. It suffered diffused strong damage in the April 9th 2009 L’Aquila earthquake. In the reconstruction phase begun after the shock, a careful survey of the building has been completed concerning the historical, geometrical and mechanical data of the building.

The kernel of the building was certainly built after the 1461 earthquake, with the L’Aquila characteristic set up made of a ground floor with shops and stores, dwelling on the first floor, and open warehouses and deposits on the second floor (figure 2). There is evidence of damage and reconstruction after the 1703 earthquake; at that time stone buttresses were added to the corners, and an internal masonry wall was erected as intermediate support for the timber floors. With the addition of the new XVIII century rear volume, the building shape changed to a rectangular plan with a nearly central small court, one masonry vaulted floor, one timber floor and a timber roof over an in-folio vault system built for insulation purposes (figure 3). During the 2009 earthquake the building suffered very severe damage encompassing extensive wall cracking, leaning of exterior walls due to overturning forces, loss of vault - wall connection and local fall down, timber floor disaggregation, and collapse of architraves (figures 4 to 6).

Figure 1: View of Micheletti Palace and open internal court

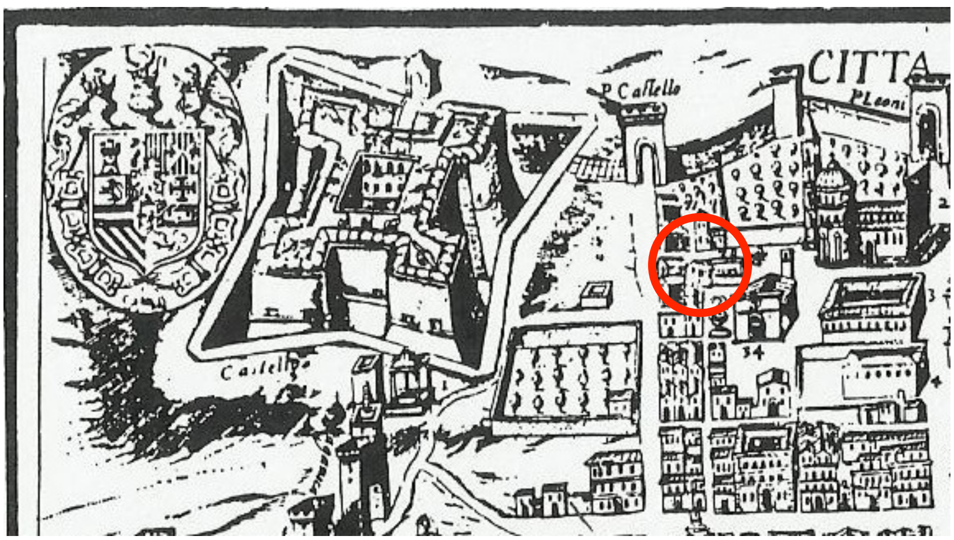

Figure 2: Map of L'Aquila city, year 1600, with Micheletti original manor. The map was created by Giacomo Lauro and dedicated to Don Fernando de Castro, Naples viceroy

East elevation along Via Ed Arco Delle Terziarie | North elevation along Via Castello

Figure 3: Schematic evolution in time of the building shape

The initial intervention design was created only to produce a rough estimate of the costs and plan the investigation activity. After the first draft, some preliminary interventions were executed and finalised with plaster removal from the walls, and pavement demolition of the floors. With this "naked" structure exposed, a detailed geometrical and mechanical investigation was carried out; the result of this phase was a clear and extensive identification of all the vulnerability issues of the building skeleton, as far as the mechanical properties of the heterogeneous masonry material was concerned.

Figure 4: View of damaged vaults and architraves after the earthquake

Figure 5: View of chaotic wall organization with flues and heterogeneity of texture

Figure 6: View of timber beams and frames with damaged sections

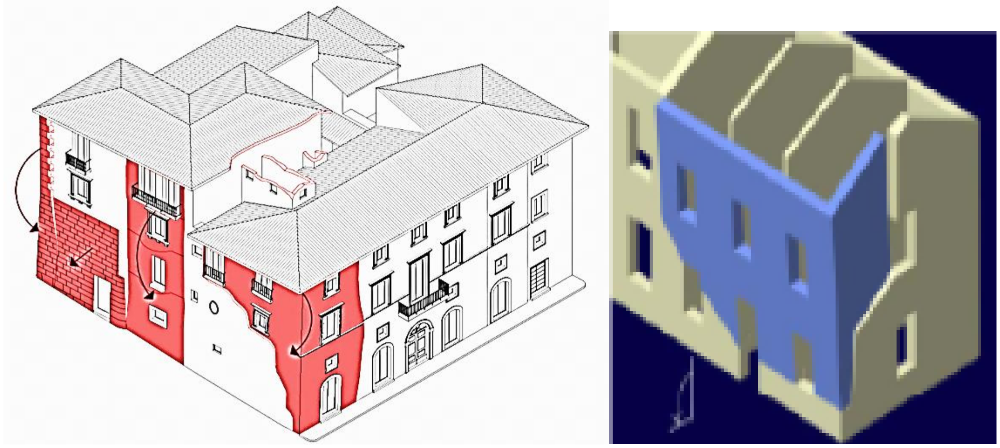

Figure 7: Indication of the external wall leaning due to incipient overturning

Figure 8: Barrel vault with stone stiffeners and cross vault made with one head bricks

MECHANICAL PROPERTIES OF THE MATERIALS

The properties of the masonry have been defined on the basis of several non-destructive and laboratory tests. In particular, penetrometric tests were carried out on the mortar and several brick and masonry cores were cut with the aim of obtaining compressive and tensile strengths by means of crushing and splitting tests (see [1] and [2]).

Unfortunately, the largest part of the external walls (figure 7) is composed of stone masonry, so that coring suitable for laboratory tests is not viable. In this case laboratory tests were restricted to tests on stone specimens and punching tests on mortar layers. By this way very different strengths were obtained for sandstone and mortar. Typically compressive strength of stones was around 90 MPa while mortars rarely exceeded 3 MPa. The situation was different for the first floor vaults constructed with thin clay fired bricks (figure 8).

In this situation the interaction of stone and mortar does not lead to cracking of the stones due to the horizontal self-stress, but the mortar collapses in confined compression instead [3]. As a rough approximation, the confinement stress in mortar is up to 30% of the vertical one; in this situation the use of a Bresler-Pister type constitutive criterion [4] leads to the evaluation of the multi-axial compressive strength of mortar, which can be considered the limit strength of the stone masonry too. In figure 9 the vertical stress amplification due to confinement in horizontal direction is calculated by means of the cited criterion.

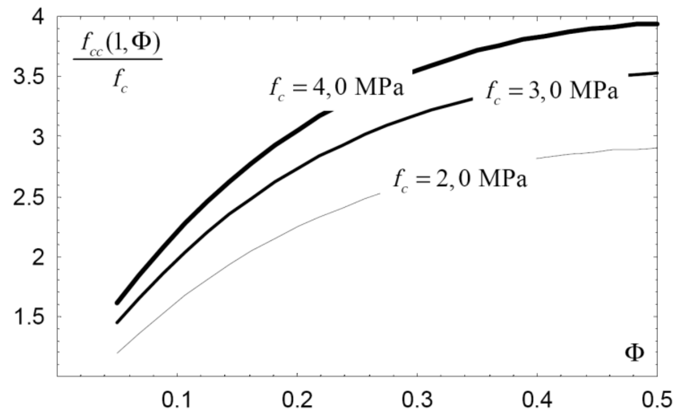

Figure 9: Tri-axial amplification factor for compressive strength of mortar

VERIFICATION OF SEISMIC VULNERABILITY

The verification of the seismic vulnerability of the Micheletti Palace has been carried out by means of a comprehensive model including shell, beam and truss elements (figure 10). The model is used in a response spectrum linear dynamic analysis in order to evaluate the seismic actions. The design acceleration is defined as a_g= 0.31 g on the basis of a structural behaviour factor of 2.25. The stress distributions were extracted from a subset of relevant sections for each wall, as depicted in figure 10.

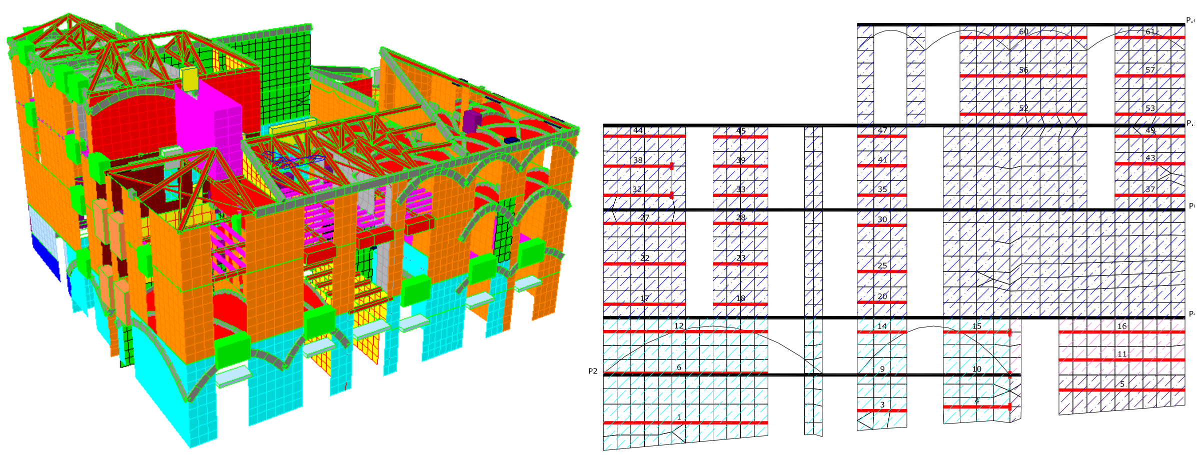

Figure 10: 3D FEM Model and schematic view of the verification sections for a wall (in red)

The most important consideration arises from the fact that, due to the non-linear nature of the masonry constitutive relationship, linear stress distribution is largely incorrect. Therefore a suitable verification procedure encompasses the integration of the linear stress distribution in order to obtain the local stress resultants of the given sections, and then the verification of the safety condition by comparison with the resisting stress resultants compliant with the no tension hypothesis [3, 5]. In particular, in the calculation of the resisting stress resultants the compressive strength of the masonry was set to 4.0 MPa, and the cohesive shear resistance to be used in the Coulomb formula was assumed as 0.5 MPa.

Even for the masonry vaults the procedure is based on the comparison of the stress resultants (figure 11). In this case the zones with tensile stresses are identified and the tensile resultants are computed in both X and Y directions. Owing to the need to fulfil the internal shell equilibrium with a no-tension material, these forces shall be carried on by a reinforcement network connected to the vault surfaces.

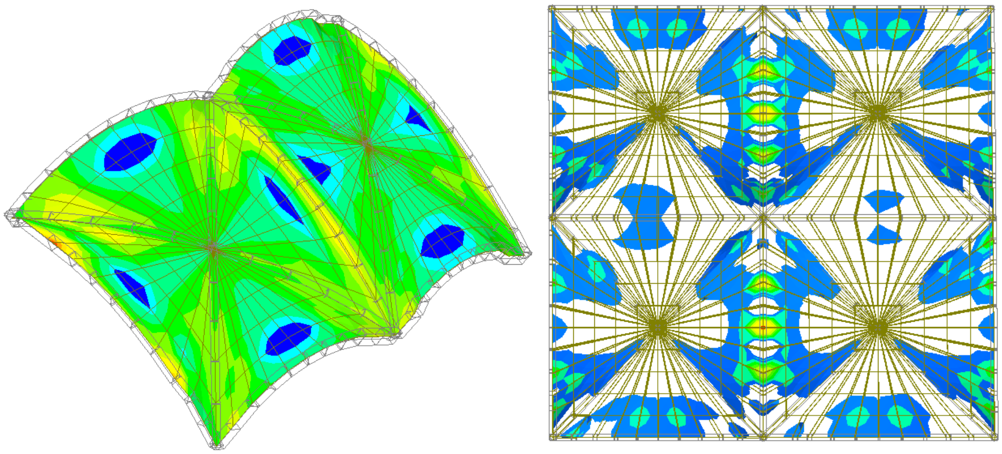

Figure 11: Tensile stresses in X-X direction for the 1st and 2nd floor cross vaults

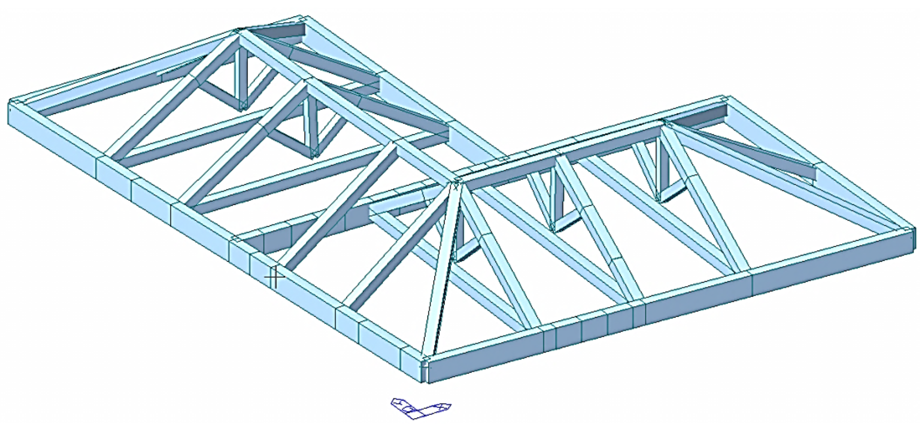

Figure 12: Model for the analysis of the timber roof in design situation

Finally, it should be mentioned that the timber roof (figure 12) was reconstructed in recent times but the lumber deck has a relatively small stiffness in its plane due to the soldier organization. In spite of this, the ceiling of the second storey, originally realized with in-folio vaults, was substituted with a timber deck composed with a lightweight concrete slab.

DESIGN OF THE STRENGTHENING INTERVENTIONS

After the verification phase, the strengthening interventions were planned and designed. As a concern, the provided works included general upgrades of the masonry skeleton and local reinforcement lay outs, in order to improve systematically the capacity of the building to resist seismic forces. It should, however, be mentioned that the replaced second storey vaults, the fill substitution in remaining vaults, and plaster thickness reduction, obtained a sensible mass reduction, with a considerable increase of the safety margin.

The diffused strengthening works encompass masonry injection with lime mortar, formation of steel connectors between masonry leaves by using the injection holes, replacement of bad or cracked bricks with reconstruction of the masonry texture, and filling of flues or cavities (figure 13). Perimeter stiffening of window and door openings was achieved by means of steel frames.

Local interventions were aimed at the equilibrium fulfilment where the internal actions exceeded the resisting ones computed with the no-tension material hypothesis [6]. In this case, tensile reinforcements were designed such that the increase in resisting resultants will satisfy the safety requirement. In particular, fibre reinforced strips were used in the formation of local reinforcement grids; both carbon fibres and micro-wire steel bundles were employed. In the first case epoxy resin was used as a bonding agent, while in the second high strength superplastic mortar was employed.

All the bonded reinforcements were designed according to the CNR DT 200/2004 document, which is now worldwide known as a reliable source of design rules for externally bonded fibre reinforcements [7]. Local anchorage enhancement was introduced in vault boundaries and at the wall toes and tips, by means of dry rope stitching or plate clamping of the strip ends.

Figure 13: Injections, brick texturing and frame stiffening around windows

One of the most interesting stiffening networks is based on the connection of the adjacent window steel frames with screw bars (figure 13). By this way the shear performance of the wall is enhanced and the stiffness and the compressive strength are increased by the confinement effect of the linked frame borders. It should also be mentioned that, because the window parapet was initially thinner than the neighbouring panels, the thickness was preliminarily reset by texturing and sewing a new leaf.

A specialized retrofit technique was used for vaults (figure 14); first of all, all the cracks were filled with fluid mortar and sewed with fibre strips. Afterwards, the masonry shells were strengthened with a grid of fibre strips and by formation of masonry reinforced ribs (figure 15). Finally, the vault volume was filled with expanded clay aggregates, a nonwoven film, and a thin finishing concrete slab. Where necessary, inside the vault volumes were placed the tie bars connecting the facing walls supporting the vault.

As said before, in order to maintain the transpiration of the masonry shells, steel wire bundles were bonded with high strength lime mortar. This made easier the fastening of the strips into the neighbouring walls simply by inserting the twisted ends in drilled holes and filling with mortar.

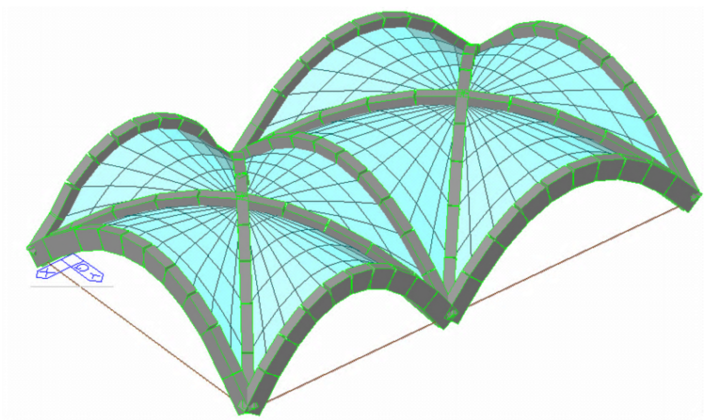

Figure 14: Model for the restoration design of the rib-stiffened cross vaults

Figure 15: Hardwire and C-FRP repair and strengthening of vaults

The second floor ancient timber beams with lacunar plank presented a number of local defects including an anomalous deflection subsequent to creep; moreover, the diaphragm action was insufficient due to the rough organization of the upper deck planks (figure 16).

The retrofit was based on the addition of a new solid timber beam above the original one; this additional section was inserted in between the joists and tightened with wedges and fastenings. Even the rafters were connected with angles and screws for obtaining a stiff grid. The in-plane shear stiffness was enhanced with screws fixing the planks and connecting a new lightweight concrete slab to the timber structure.

Figure 16: Increasing depth of timber beams with timber blocks and fastening of lintels

The new floor slab is reinforced with a single wire mesh but all the sides are organized with chevron bars able to transfer shear from the slab to the wall (figure 17). Furthermore, the slab contains tie bars running inside the diaphragm and restraining the facing walls around the slab.

Concerning the timber roof trusses, several missing local portions were reconstructed by using prosthesis elements and fibre reinforcements [8]. Few timber tendons were also made safer by adding new steel bars as supplemental resisting sections.

Figure 17: Steel ties placed around rooms and edge connectors to be grouted into the slab

CONCLUSIONS

The paper presents an organized system of investigations and retrofit measures finalized for the seismic safety of an historic building damaged by the strong 2009 earthquake in L’Aquila.

As shown in the included photos, local and diffused strengthening interventions were employed in order to obtain a homogeneous behaviour of the resisting elements with a significant cooperation toward the seismic resistance. The system ductility was largely enhanced by a diffused network of ties connecting the walls in their plane and face-to-face; by this way not only was the shear resistance increased but so was the compressive strength as a consequence of the high confinement effect of these tensile elements.

In spite of the condensed presentation, it is worth noting that all interventions were designed by using numerical rules and safety verification formats, noting it is very complicated to insert the retrofit measures in a numerical model able to evaluate the increase of the seismic acceleration leading to collapse of the building.

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