IMPORTANCE OF BUILDING KNOWLEDGE FOR A CORRECT STRUCTURAL ASSESSMENT. THE CASE OF SANTA MARIA DELLA CARITÀ CHURCH IN ASCOLI PICENO (ITALY)

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ABSTRACT

The aim of this work is to demonstrate how the wide knowledge of the building, especially when it is precious for its historical and cultural value, is at the basis for a correct structural assessment and so, to a good intervention. The case of Santa Maria della Carità church, in Ascoli Piceno (Italy), hit by L’Aquila’s earthquake of 2009 is an example of this. After the seismic event, the church was closed because several cracks appeared, in particular the triumphal arch was the element that appeared mainly damaged and presented the most serious and spread damaging. Two different approach are developed to assess the safety of this element. First a common approach, based on FEM seismic analysis of the arch panel, performed following procedures from literature, was carried out. Results were not comparable with actual cracking pattern. Then a deep historic research was performed in order to better understand the building evolution and how this could have affected its structural behaviour. From this, a ground sinking in bell-tower foundation area seems to be responsible for the activation of a mechanism that might account for the present damage. This hypothesis was validated comparing results obtained by FEM analysis first, kinematics analysis and the actual state of building. This confirms the importance of knowing building history and evolution for a correct structural assessment, but also demonstrate how fundamental may it be for restoration design purpose in order to avoid erroneous interventions and to save money.

Keywords: Building knowledge, Triumphal arch, FEM analysis, Kinematic analysis

1. INTRODUCTION

A large portion of the Italian cultural heritage is represented by monumental masonry buildings. One of the prevalent kind of monumental buildings, the church, has a particular seismic vulnerability due both to the mechanical properties of the masonry material and to the particular configuration (often characterized by open space, slender walls, lack of effective connections among the structural elements). This features could be recognized also in the Santa Maria della Carità church in Ascoli Piceno (Italy). The church has a large historical and architectural value: it is one of the most important example of the Barocco age in the region, and contains a lot of precious paintings of local artists. Moreover it has also a social values for the city of Ascoli Piceno, because it is the only one that is opened to the devotees for every moment of day and night. Build in the form that we can see today after several steps during the XVI century, the church has an only nave of 12.93 meters wide, 23.15 meters long and has a maximum height of 16.5 meters, it ends with a rectangular apse 6.55 meters

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wide, 8.30 meters long and an height of 11.95 meters (Fig. 1). The bearing structure are made by stone masonry, the main facade, characterized by three front doors, is ornamented with classical pattern (columns, tymanum, cornice) and the masonry external layer is constituted by white travertine. The nave is covered by a brick elliptic vault (like the apse zone) with lunettes that create small ornamented niches jointing with the pilasters included in side walls. Also in S.M. della Carità like many other Italian churches, the triumphal arch separates the zone of nave from the apse. It is inserted in a masonry panel, with an elliptic shaped opening has a thickness of about 1,20 m. After the 2009 seismic event, the church was closed for several damaging, in particular the triumphal arch was the element that appeared mainly damaged and presented the most serious and spread damaging and looked as in Fig. 2.

![Fig. 1 Church ground floor layout](image1)

![Fig. 2 Triumphal arch cracking pattern](image2)

The following damages can be found on the arch macro-element: (i) a very deep crack (4-5 cm width) in the right top of the masonry panel, with a quite vertical trend; (ii) several deep cracks (1 cm width) with vertical trend and clearly visible fallings (2-3 cm) of stone block composing windows square and ornamental display over the arched opening; (iii) several detachments between arch stone blocks. All these problems forced the authorities to close the church to preserve human safety, but they have the firm intention to reopen it as soon as possible, because of cited great artistic, cultural and social value of the building. So a correct structural evaluation was required, focusing on triumphal arch status diagnosis, in order to provide guidelines for a restoration intervention based on “minimal intervention” principle, that is sign also of economic convenience.
2. **FIRST APPROACH: SEISMIC ASSESSMENT**

It is common knowledge that monumental historical buildings are difficult to reduce to standard structural schemes because of the uncertainties that affect the structural behaviour and mechanical properties of the material (highly nonlinear behaviour and very small tensile strength), unless apposite solution strategy are provided. However, to better understand the behaviour of monumental buildings (and in particular of historical churches) subjected to seismic actions, it has been shown in several studies, according to the experience of past earthquakes, that monumental buildings can be seen as an assemblage of several masonry portions (called “macro-elements”) with self-contained behaviour, that respond as single units to seismic action, and for which the main features of the collapse mechanisms are at least approximately known [1-5]. So each historical church can be considered composed by few components (facade, lateral walls, bell tower, etc.), and one of these is the triumphal arch. From the structural point of view, the triumphal arch is a masonry wall panel, with an arched opening in the middle zone that discharge the upper loads on the lateral piers. Its name derives not only from the shape (usually semi-circular but in some cases elliptic) but also from the overshadowing dimension that bring back to the triumphal arch of the Roman age. To have the possibility of evaluate macro-element seismic capacity the equilibrium limit analysis is usually applied; it represents a user-friendly tool for estimating the collapse load of structural systems, since it provides an extent of the horizontal bearing capacity. Three hypotheses are used to describe masonry behaviour: no tensile strength, infinite compression strength and absence of sliding at failure. Under these hypotheses, the masonry structure can be considered like an assemblage of rigid bodies, and the collapse of the structural elements is characterised by the development of non-dissipative hinges transforming the structure into a mechanism [6]. The number and the hinges position determine the shape of the rigid block in which the macro-element could be divided. For instance, the typical failure mechanism for an arch can be represented as in Fig. 3a, 3b, 3c.

![Fig. 3a, 3b, 3c Typical failure mechanism for an arch, global (a, b) e semi-global mechanism (c)](image)

For every mechanism pattern, displacement components, using kinematics chain or analytical expressions, could be derived. Next, the displacement components of the loads application points are used in the equation of the principle of the virtual works (PLV) to calculate the horizontal collapse multiplier. As shown in [7], for the PLV we have:

\[
L_{Fi} = L_{Fe}
\]

(1)

Where: \(L_{Fi}\) is internal forces work and \(L_{Fe}\) is the external forces one. The (1) became, under the hypothesis of rigid bodies system:

\[
L_{Fe} = 0
\]

(2)

In particular, pointing with \(P_i\) be the generic applied load, and \(W_j\) the weight of the single part forming the mechanism and with \(\delta_h\) and \(\delta_v\) the vertical and horizontal virtual displacement component of the load application points, respectively, and \(\delta_{hj}\) and \(\delta_{vj}\) those of the weights \(W_j\) the principle of the virtual work equation is (3):

\[
\alpha \left( \sum_i P_i \cdot \delta_{hi} + \sum_j W_j \cdot \delta_{hj} \right) + \sum_i P_i \cdot \delta_{vji} + \sum_j W_j \cdot \delta_{vji} = 0
\]

(3)

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Since all displacement components depend on a single arbitrary Lagrangian parameter, the equation (3) is a homogeneous equation in the only unknown $\alpha$. This coefficient, non-dimensional, represent the vertical load multiplier after then we can have the activation of the mechanism and so the collapse of the structure. To define the collapse multiplier of the element, that is the minimum among all the possible ones, it is necessary to do an iterative procedure changing the position of hinges and, clearly, also the areas of the rigid parts, the position of their centre of mass and the forces involved. Instead, in this work, following a similar procedure proposed also in [7], it will be started from the FEM analysis of the structural element considering the masonry like a continuous material, to locate the most probably hinges position and then, on this conformation, it will applied equilibrium limit analysis. Therefore, following other experience about masonry modelling [8-11] the arch has been modelled with brick elements, eight nodes isoparametric elements with crushing and cracking features. For these elements the hypothesis of non linear elastic behavior has been adopted. Non linear properties of masonry have been modelled by using the yield Drucker-Prager criterion with associated flow rule. The yield surface has been assumed not to change with progressive yielding, hence there is no hardening rule and the material is elastic-perfectly plastic. The assumed failure surface is the one proposed by Willam and Warnke [12]. Table 1 reports the selected values (material mechanical features, yield Drucker-Prager criterion and Willam and Warnke failure surface) for the model parameters, derived by Italian Technical Laws [13, 14] and literature works [8-11]. In Fig. 4a, b, triumphal arch geometrical dimensions and FEM model, with the load distributions considered in the analyses are provided. The arch model has been supposed constrained at the basis of the piers for all possible degrees of freedom, subjected to self weight (roof loads can be ignored) and to a horizontal load of increasing intensity, constantly distributed along the height of the element. In Fig. 5a, 5b, respectively, the deformed configuration under load distribution, with the vertical stress distribution, and collapse mechanism configuration (shape and dimension of rigid blocks and hinge position) selected after FEM analysis are shown. Notably that, as underlined in [7], the vertical load multiplier is affected not by large variability within the same mechanism “family” and so, once the mechanism pattern has been chosen on the basis of FEM analysis, the results of equilibrium limit analysis can be considered acceptable with a small, certain approximation. In this case, for example, the vertical load multiplier obtained applying the PLV to the rigid bodies system is 0.44. Fig. 6 presents the comparison between the non-linear FEM analysis and the equilibrium limit analysis.

![Table 1](image)

**Table 1 Model reference values**

<table>
<thead>
<tr>
<th>Material mechanical properties</th>
<th>Drucker-Prager yield criterion</th>
<th>William and Warnke failure surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>$c$</td>
<td>$\xi$</td>
</tr>
<tr>
<td>2100 kg/m$^3$</td>
<td>0.1 MPa</td>
<td>0.1 MPa</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>$\eta$</td>
<td>$\eta_l$</td>
</tr>
<tr>
<td>1.5 GPa</td>
<td>15°</td>
<td>2.6 MPa</td>
</tr>
<tr>
<td>Poisson’s modulus</td>
<td>$\varphi$</td>
<td>$\beta$, $\beta_l$</td>
</tr>
<tr>
<td>0.25</td>
<td>38°</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.15</td>
</tr>
</tbody>
</table>

Fig. 4a, 4b Triumphal arch geometrical dimensions (left, values in meters) and FEM mesh, with the load distributions considered in the analyses (right)

From the results obtained by this procedure, the arch seem to be safe under the seismic load that attacked the building during the 2009 L’Aquila earthquake and also respect to reference values of peak ground
acceleration (PGA) proposed by Italian Technical Law [13, 14]. Therefore, the FEM model allows to know the position of cracks and crushed zone due to compressive and tensile stresses (Fig. 7), that are mainly located on the piers and on arch boundary and windows corners. Comparing this representation with crack pattern, it looks to be not so compatible so a different approach has been proposed.

![Fig. 5a, 5b Deformed configuration under load distribution, with the vertical stress distribution (left), and collapse mechanism: shape and dimension of rigid block, hinge position applied loads (right)](image)

**Fig. 5a, 5b** Deformed configuration under load distribution, with the vertical stress distribution (left), and collapse mechanism: shape and dimension of rigid block, hinge position applied loads (right)

![Fig. 6 Comparison between the non-linear FEM analysis and the equilibrium limit analysis.](image)

**Fig. 6** Comparison between the non-linear FEM analysis and the equilibrium limit analysis.

![Fig. 7 Cracks an crushed zone distribution concentrated on the piers and on arch boundary and windows corners](image)

**Fig. 7** Cracks an crushed zone distribution concentrated on the piers and on arch boundary and windows corners

### 3. SUBSEQUENT ELABORATION

Since results obtained by first approach seem to be not so compatible with the actual state of building, the problem was studied with a different approach. In particular, looking a picture of the triumphal arch taken around 1970, found in church archives, it is possible to see that the main deep crack located
on the right top was visible at that time, and was repaired only externally (Fig. 8). So, the idea that the seismic event could be not the only, and certainly not the first, cause of the building actual state seems to be plausible. First of all, a deeper historical research about the building was tackled and, after that, a different model of the arch possible damaging was developed.

Fig. 8 S. Maria della Carità church picture of 70’s

First available information on construction of Santa Maria della Carità church, at the beginning built in the service of an oldest attached hospital, date back to years 1387. At that time, it has to be very different from the one that we can see today. Later, since the early years of 16th century to 1583, the
The building has been partially demolished and re-built, in several steps, in the same place, modifying basically the facade with the tympanum raising and front doors opening. Further to this intervention the nave has been covered with masonry vault and has been connected by triumphal arch with the apse, built at the same time. Therefore, iconographical documents attest the presence of a pre-existing bell-tower on the opposite side respect the one we can see today. So, the triumphal arch was built during 16th century but bell-tower was built only at the end of 17th century, after almost one hundred years. So, it is reasonable to expect that bell-tower construction, could determine a falling of foundation ground around arch left pier (a first layer about 5 meters deep is composed by compressible sandy silt and silty sand (Fig. 9). This hypothesis could also catch the reason of deep crack developing in the right top of the masonry panel: in fact it is probable that during the raising phase, the old bell-tower has been included in masonry panel without connection with new portion and, after ground falling, the lacking of clamping has determined the crack opening. Fig. 10a, b, c resume, synthetically, the church building history and evolution.

Following this assumption, the same 3D FEM model of arch panel used for the seismic non linear analysis, has been supposed constrained at the base of the right piers for all possible degrees of freedom and has been subjected only to self weight (roof loads can be ignored), let it free of falling down on the left pier (Fig. 11).
Fig. 12 a shows the vertical displacements distribution and it is interesting to notice that vertical displacements assume different values close to windows contour, that is in line with the actual state of this part (Fig. 12b) that presents clearly visible lowering between stone blocks composing window square. To validate this hypothesis, first, ground falling entity was approximately evaluated, using geotechnical data obtained by standard penetration test executed on ground of church area (presented in Fig. 9). Referring to dimension directly measured and assuming construction materials density derived by Italian Technical Laws [13, 14], bell-tower load was calculated, and using literature methods about sinking calculation, the falling was assessed in about 200 mm (20 cm). Then, assuming that arch panel can be portioned in a system of rigid bodies composing a mechanism, kinematics analysis was developed in order to obtain a comparison between theoretical and in situ measured values.

![Fig. 12a, 12b Arch vertical displacements distribution (left), and clearly visible lowering between stone blocks composing window square (right)](image)

Rigid blocks composing mechanism shape and dimension (and so hinges position, too) are determined observing masonry arch panel crack pattern (Fig. 2). Then, applying a vertical displacement in falling direction of the same value that the calculated one – “$\delta$” –, kinematics analysis was developed in order to obtain in particular, three selected points displacement (“PC1”, “PC2”, “PC3”, Fig. 13), so called “control points”, obviously depending by “$\delta$”. Notably, that kinematics vector entities (translations and rotations) sign, are considered according to convention showed in Fig. 13. From direct measurement, it is possible to determine with no great error of “PC3” horizontal displacement, in 0.07 m (7 cm). Starting on this assumption, $\theta$ angle could be determined and so, it was possible determine every system point displacement.

Then, in relation to lowering and detachments between stone blocks composing window square, values from in situ measurements relatively at “PC1” and “PC2” position, and analytical procedure are compared. In particular was observed a correspondence between relative vertical displacement measured and the one calculated, around the value of 0.025 m (2.5 cm). As regards the horizontal one, the calculated (0.27 m) is overestimate respect to the one measured (4 cm), but the model summarizes the component displacement all in the horizontal distance between control points PC1 and PC2, while in the real structure detachment is shared in five locations, of the same dimension (4-5 cm), along the decorative pattern placed above arched opening. This second approach seems to better catch the damaging cause of the masonry element and permit to better evaluate what it is necessary to do to restore the safety church, obviously respecting monumental value of the building. In fact, the ground falling of foundation area, can be considered a spent process, developed in the past and with no implication for the future. It is reasonable to put most of the cracks visible on this element, down to this past event and not to the recent earthquake, that could be considered only the cause of the detachment and dropping of superficial esthetical plaster retouch, applied during years. Assuming this concept, no consolidation of foundation area is needed. The retrofit intervention can be developed using together the following techniques: (i) injections of mortar compatible with masonry materials, to repair small size cracks; (ii) local reconstruction of the wall around the deep crack placed on the right top of masonry arch panel; (iii) fixing the arch stone blocks with steel or FRP rods anchored in masonry above arched opening, in order to discard the blocks falling possibility.
4. CONCLUSION

In this work, the results of studies aiming at interpreting the damaging causes of S. Maria della Carità church in Ascoli Piceno (Italy) triumphal arch have been presented. After the 2009 L’Aquila’s earthquake the church was closed due to the several damaging suffered, in particular placed on masonry arch panel. Two different approach are developed to assess the element safety. After a first approach to the problem, based on seismic analysis of the arch panel, performed following procedures from literature (non linear FEM analysis and equilibrium limit analysis), that brings out to be not so compatible with the condition of things, another approach was developed. A deep historic research was performed in order to better understand the building evolution and how this could have affected its structural behaviour. From this, a ground falling in bell-tower foundation area seems to be responsible for the activation of a mechanism that might account for the present damage. This hypothesis was validated by comparison between results obtained using non linear FEM analysis first, and kinematics analysis then, and the actual state of building and respect to a series of “in situ” measures. The findings of old bell-tower included in masonry panel during the recent restoration works confirms that what has been supposed was right. This results confirms the importance of knowing building history and evolution for a correct structural assessment, but also demonstrate how fundamental may it be for restoration design purpose in order to avoid erroneous interventions and economic waste so.

REFERENCES


