Seismic behaviour of barrel vault systems

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ABSTRACT: In this paper, the experimental study on the rocking behaviour of a full scale barrel vaulted structure undergo cyclic horizontal loading is discussed. The study is the first part of an ongoing experimental and theoretical research program, developed by the University of Brescia, concerning the seismic behaviour of masonry buildings. The scope of the paper is to provide some evidence of the rocking mechanism experienced by barrel vaulted structures undergo horizontal loading. Understanding of the behaviour of such structural systems is fundamental for their seismic vulnerability assessment, as well as for the correct design of possible strengthening techniques. The structural behaviour is also investigated by means of non linear finite element analyses. Numerical results are validated through comparison with experimental results. After validation, the FE model can be applied to different case studies.

1 INTRODUCTION

Earthquake hazard reduction is progressively becoming one of the most important issues in the structural rehabilitation discipline. The assessment of the seismic vulnerability of historical constructions and the design of successful strengthening techniques are difficult tasks, mainly because the structural behaviour depends on too many parameters of uncertain definition, but also because of the little knowledge about traditional construction techniques, historic material properties and constraints (D’Ayala & Speranza 2002, Binda 1999).

However, important lessons have been learned from the analysis of the damage induced to historic constructions by recent earthquakes, lessons on the real structural behaviour of engineered and non engineered constructions (Lagomarsino et al. 2004), as well as lessons on how to improve structural performance (Borri et al. 2002, Modena et al. 2000). Recent earthquakes also provided an opportunity to evaluate previous structural interventions, based on actual performance during a real ground motion (Guerrieri 1999).

The earthquake behaviour of masonry structures has been widely studied experimentally, analytically and numerically. Although the knowledge of the structural behaviour of masonry structures has greatly enhanced in the last decades, little is still known about the structural behaviour of vaulted structures undergoing seismic action.

The survey of damaged vaulted structures shows that seismic action induces bending of the vault ring, as well as rocking of the abutments at the base. The rocking of vaulted structures and transverse diaphragm arches has been studied mainly analytically and numerically by different authors, whereas fewer experimental results are available.

Utilizing the principle of virtual work, focus has been paid to the onset of failure mechanisms and to their evolution in the large displacement field, to the definition of structure capacity curves (Abruzzese & Lanni 1999, Housner 1963, Lagomarsino et al. 2004), and to the out-of-phase rocking of the abutments (Como et al. 1991).

Furthermore, a simplified analytical method, based on limit analysis, was proposed by Giuriani et al. (2007) to assess the seismic vulnerability of transverse arches undergoing rocking of the abutments. The discussion focused on the different behaviour of transverse arch systems subjected either to rest or rocking conditions and highlighted the physics and
the phenomena involved in the mechanisms. It was shown that in the at-rest condition, the arch lateral thrust can be largely or entirely resisted by the buttress action of the abutments, depending on their geometry and massiveness, thus possible ties must confine only the part of the arch thrust exceeding the buttress action (Giuriani & Gubana 1993, 1995). Conversely, as rocking modifies the resisting mechanism of the abutments, increasing the arch thrust and considerably diminishing the confining buttress action, the arch thrust must be entirely confined by the existing ties (Giuriani et al. 2007). A significant increase in the tie tensile force is expected during rocking. The same conclusion was drawn by Lagomarsino et al. (2004) by analysing the kinematics of the problem using the principle of virtual work.

In this work, focus is paid to the experimental rocking behaviour of a barrel vaulted structure undergoing external lateral loading. The effect of seismic action on the vault crown is beyond the scope of this paper and will be studied in the future research.

The scope of the paper is to provide evidence of the rocking mechanism experienced by barrel vaulted structures undergoing horizontal loading, which is critical for their seismic vulnerability assessment, as well as for the correct design of possible strengthening techniques.

Focus is made on the modelling of the experimental test, on the choice of the geometry of the vault, the technical and technological aspects concerning its construction, as well as on the bench set up, the load application system, and the instrument set up.

Furthermore, the experimental results are used to validate a FE model. Validation of the FE model allows widening the field of its application to different case studies.

The study is the first part of an ongoing research program developed by the University of Brescia, which is focused on the role of vaults in the seismic behaviour of masonry vaulted structures, the assessment of the effectiveness of some retrofitting techniques, and on the proposal of simplified analytical methods.

2 EXPERIMENTAL TEST

2.1 Modeling of the experimental specimen

In this paper the behaviour of a barrel vaulted structure undergoing rocking is analysed. As a reference, a real two storey building having a barrel vault at the ground floor is considered (Fig. 1a).

Barrel vaults are 3D structures having a complex three-dimensional stress distribution (O’Dwyer 1999). However, in the frequent case of long barrel vaults lacking the connections to the head walls, an estimate of the structural behaviour can be obtained by modelling the barrel vault as a series of adjoining arches of unit width (Giuriani E. et al. 1999). In this scenario, according to this simplified analytical model, a unit stripe \( A \) was selected and analysed (Fig. 1a, b).

As a second step in the definition of the experimental model, the upper masonry wall were removed and their weight was introduced in the model by adding point loads at top edges of the abutments. (Fig. 1c).

The geometry of the experimental model is detailed and shown in Figures 2–3, respectively.

As for the construction technique, the wall layout and the layer by layer brick arrangement were derived by historic construction treatises (Carbonara 1996, 2004, Giardina et al. 2007a; Fig. 4a). The resulting wall cross section is a single leave three head wall.

The structure is built on two reinforced concrete support; each abutment is fixed to the support with vertical rebars which allow rotations at the base,
reproducing the real structural constraints, but inhibiting any shearing sliding (Fig. 4c).

An intrados tie rod confines the arch horizontal thrust at the springing. The 12 mm tie rod, having an initial pretension of 7 kN (equal to 75% of the vault lateral thrust) is anchored to the abutments using steel plates.

The backfill consists of fine sand, laterally contained by means of Plexiglas plates bolted to the masonry structure (Fig. 4d). Bolt-hole clearance avoids any participation of the Plexiglas to the resisting mechanism (Fig. 4e). Thin layers of chalk are inserted in the backfill to emphasise the vault crown deformation during loading and unloading cycles (Fig. 4d).

Four 7 kN point loads are applied at the top edges of the abutment to simulate the first floor masonry weight. Transverse steel profiles spread the load along the crown (fig. 5a,b). The load is applied through vertical rebars tightened to the concrete supports. Four packs of cup-springs are interposed at the rebar bases, as shown in Figures 5a–c, to reduce the stiffness of the rebars. This was done to prevent an increase in vertical load due to rebar elongation induced by increasing the applied lateral displacement.

The total dead load of the specimen, including the additional load of the vertical rebars is \( W = 90.7 \text{ kN} \).

### 2.2 Testing bench and lateral load application system

The experimental test aims at investigating the behaviour of vaulted structures undergoing earthquake-induced rocking of the abutments. Further studies will focus on the bending moment induced in the vault crown by seismic action.

The seismic action is simulated by introducing horizontal loads at the vault springing. Horizontal point loads are applied by means of an electro-mechanic jack fixed to a steel frame, as shown in Figure 6. The horizontal loading system was designed in order to impose the same force (instead of the same displacement) to each abutment (Fig. 6). The load is equally divided between the abutments through the pulley and rope system shown in Figure 6. The point load is uniformly spread at the vault springings by means of transverse loading beams.
2.3 Instrument set up

During the loading and unloading steps, vertical and horizontal displacement were recorded at some key points of the structure by means of linear variable displacement transducers (LVDT).

The drift of the abutments at different heights was monitored by LVDT A_x, B_x, F_x, G_x, whereas LVDT C xy, D_y, E xy were located to detect any flexural displacement along the vault ring (see Fig. 7a).

The vertical component of the displacements at the vault springing and keystone were measured using LVDT C_y, D_y and E_y. These instruments were fixed to the vault intrados trough spherical hinges sliding on horizontal supports (Fig. 7b), in order to reduce interference with the global lateral displacement.

The applied load is measured by means of a load cell placed between the jack and the transverse beam.

2.4 Loading modalities

Load-controlled cyclic tests were carried out with a loading rate of 600 N/min. The specimen was initially subjected to loading and unloading cycles, in the same direction. The horizontal point load was increased by 2 kN (corresponding to 2.2% of the total vertical load) from cycle to cycle, until the vault developed a four hinge mechanism. Later, two fully reversed cycle were applied by repositioning the steel frame. The maximum applied load was equal to 11 kN (equal to 12% of the total vertical load W).

It is worth noting that as a result of the increase in the horizontal displacement, a small increase of the axial force in the rebars occurred, changing the vertical confinement of the specimen, despite the use of the cup-spring systems.

Details on the experimental study can be found in Giardina et al. (2007b).

3 EXPERIMENTAL RESULTS AND DISCUSSION

Figure 8 shows the structural response in terms of applied horizontal force versus abutment top horizontal displacement (LVDT B_x, F_x). Figure 9a and 9b show the detail of the curve for positive and negative applied horizontal loads. The structural response remains basically linear elastic until 6–7 kN (equal to 6.6% of the total vertical load) for both directions of applied displacement.

As for the cyclic behaviour, damage is accumulated after a few load cycles, and the global stiffness slightly decreases.

Figure 8 shows that the structure is relatively ductile with little dissipation capacity and a pronounced self-centering behaviour (see the flag-type curve). Inelastic deformations are almost negligible compared to the maximum experienced displacements.
Figure 9. Details of the (a) positive and (b) negative applied horizontal force versus abutment top horizontal displacement curve (LVDT $B_x$, $F_x$).

The test was interrupted during the negative load application ($10 \text{kN} = 10\% W$, top displacement of 47 mm corresponding to a 1.5% drift) after a further vertical crack opened from excessive compressive strength on the four hinge mechanism.

For increasing applied lateral forces, cracks subsequently formed at the vault springing, where the vault crown thickness abruptly halves, and at the abutment bases. As a result, the global stiffness of the structure progressively decreases (Fig. 8) and the envelope curve shows a piecewise linear behaviour. Figure 10 shows the evolution of the structure toward the 4 hinge mechanism; numbers refer to the crack onset chronology, and to the value of the applied load triggering the crack.

It is worth noting that the mechanisms change for different directions of the applied loads (Fig. 10a,b). This is evidenced by the higher position of the crack in the left abutment base (Fig. 10b), which results in a different rocking behaviour of the two abutments.

All cracks perfectly closed upon the first load reversals, whereas in the final stages, for increasing drifts, the non-cohesive backfill material poured into the larger crack at the vault crown extrados (n.2 in Fig.10b), thus preventing its closing.

Figure 11 shows the structural response in terms of applied horizontal force versus horizontal displacement at the vault springings (LVDT $A_x$, $G_x$). When positive horizontal forces are applied, a maximum positive differential displacement of 0.04 mm was recorded, which corresponds to the lengthening of the vault span (Fig. 11a). Conversely, when negative horizontal displacement was surveyed, 4 mm of relative displacement was recorded, resulting in the shortening of the vault span (Fig. 11b).
Figure 12. Vertical displacement at the vault intrados springing and key sections (LVDT $C_y$, $E_y$, and $D_y$) in case of positive applied lateral loads (see Fig. 9a).

Figure 13. Sand backfill cracks due to internal tension force.

The horizontal displacements of the abutments induced the bending of the vault crown. The vertical displacements at the intrados of the vault springing and key, recorded by transducers $C_y$, $E_y$, and $D_y$ are shown in Figure 12 for positive lateral load cycles. The maximum uplift displacement of the vault intrados is equal to 5.6 mm at the right springing, corresponding to 1/3 of the abutment top lateral displacement; whereas a 2.5 mm maximum downward displacement, equal to 1/8 of the abutment top displacement, was recorded at the left springing. The vault key also rose 1.9 mm. Vault flexural deformations were emphasized by the crack extending into backfill (Fig. 13).

The tie tensile force significantly changed for increasing applied lateral loads (Fig. 14).

When positive maximum displacements were applied and the mechanism was that corresponding to the crack pattern of Figure 10a, the tie tensile force increased to 12% of its initial value ($\Delta F = 0.82 \text{kN}$, Fig. 14a). Note that the test was stopped in this direction well before the cracks had completely penetrated the element cross sections. Thus, for increasing lateral displacement, a further significant increment in the tie tension should be expected. Furthermore, the abutments underwent a positive relative displacement of $\Delta L = 0.04 \text{mm}$ (Fig. 11a) at the vault springings, which only partially explains the increment in the tie force. As a matter of fact, given the tie length $L = 3260 \text{mm}$, the axial strain is equal to $\varepsilon = \Delta L/L = 1.23 \times 10^{-5}$, thus the increment in the axial force is $\Delta F = \varepsilon E_S A_T = 0.29 \text{kN}$ (where $E_S = 210000 \text{MPa}$, $A_T = 113 \text{mm}^2$). The further tie increment indicates that following the development of the crack pattern, the ideal arch increases its span and the buttress action significantly reduces (Giuriani E. et al. 2007).

When negative maximum displacements were applied, the mechanism changed, as shown in Figure 10b. The crack pattern showed two wide cracks at the abutment bases located at different heights, thus resulting in a negative relative displacement of the abutments at the vault impost. In this case, the tie tensile force reduced to approximately 50% of its initial value (Fig. 14b).

Finally, the vertical tension in the rebars increased during the test, as the lateral load and displacement increased, despite the positioning of the cup-spring packs at the bar bases. However, the maximum increment was at most equal to 15% of the initial value.

4 NUMERIC ANALYSES

Numeric analyses were carried out prior and after the experimental tests. Pilot analyses provided some useful information to guide the modelling and design of the experimental test. On the other hand, upon completion of the experimental test, the experimental results were used to validate the numeric model.
Figure 15. Mesh B. Contact elements are located aside each white element along the vault crown and the abutments, and along the vault-to-backfill and backfill-to-abutment interfaces.

The model validation is necessary, in order to extend the application of the numeric model to different case studies, geometries and load layouts. This way, the FE analyses can help practicing engineers in defining retrofitting strategies, and in testing their effectiveness.

Different numeric models were implemented. First, a pilot linear elastic analysis was performed for the general understanding of the structural behaviour (Mesh A). Once the governing mechanisms were identified, the model was further refined both in terms of modelling assumptions and material behaviour. A second model was developed by adopting contact elements to simulate and allow the onset and development of cracks in the structure (Mesh B). Finally, in a third model, non linear constitutive laws for masonry were adopted (Mesh C). Mesh B showed the best fitting of the numeric results and is discussed in the following. The complete discussion concerning the numeric study can be found in Giardina et al. (2007c).

Geometry as well as loading conditions are derived from the experimental tests. Therefore, the loads acting on the model are:

- dead load
- vertical point loads at the abutment top edges
- horizontal point loads applied at the springing.

The analyses is performed by incrementing the horizontal points loads.

Mesh A was used to identify the zones of weakness where tensile stresses concentrate, i.e. where cracks might develop. The results highlighted that higher tensile stresses occur along the vault ring, at the abutment base and, when horizontal loads are applied, at the vault springing.

In Mesh B masonry and backfill were modelled by means of 3–4 node plain strain elements (Fig. 15). The non linear behaviour of the structure was modelled through contact elements, which were located between adjoining vault ring voussoirs, along the vault-to-backfill and backfill-to-abutments interfaces, and at the abutment bases, according to the result of Mesh A. Contact elements allow transferring the axial load only when the surfaces are in contact, whereas shear transmission is governed by friction. Mechanical properties are summarized in Table 1.

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<th>Masonry</th>
<th>Backfill</th>
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<td>1900 kg/m³</td>
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<tr>
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<td>Contact elements Friction coefficient</td>
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![Figure 16](image.png)

Figure 16. Numeric and experimental capacity curves: (a) applied lateral load vs top displacement of the abutments; (b) applied lateral load vs vault intrados vertical displacement.

Table 1. Mechanical properties in Mesh B.
opening is observed at the same locations where cracks opened in the experimental test. It is worth noting that the chronological order of crack development is also the same.

Further numeric analyses showed that the crack opening location is a function of the geometry and of the applied vertical loads. When large vertical load is applied to the structure (i.e. in the case of two or more storey buildings), besides the crack at the abutment bases, two further cracks form in the vault ring close to the springing where the thickness of the section abruptly decreases. On the other hand, for decreasing vertical loads, these two cracks form in the abutments, close to the vault springings, rather than in the vault crown.

Numeric and experimental results also compare well in terms of vertical displacements along the vault crown (Fig.16b), and tensile tie force.

The deformed shape at failure is shown in Figure 17. The crack locations correspond to the experimental mechanism surveyed during the experimental test.

5 CONCLUSIONS

In this paper, the transverse rocking mechanism of a barrel vaulted structure was investigated by means of a full scale experimental tests and a non-linear FE analyses.

The most significant results of the experimental study can be summarized as follows:

- The structure is relatively ductile in terms of displacements. However, little energy dissipation capacity and a pronounced self centering behaviour are observed. Failure is reached with a four hinge mechanism. Any possible retrofitting technique should be respectful of this structural behaviour, by strengthening the structure without reducing its capacity to meet to displacement demand;
- The tie tension significantly changes during rocking. The small abutment relative displacements induced by rocking is not sufficient to explain this increase in tie tension. The tension is likely to increase following the loss of the abutment buttress action and the increase of the ideal arch span. This result should be carefully taken into account in seismic vulnerability assessment, as tie rods might experience excess in tension during an earthquake, thus requiring additional strengthening. On the other hand, depending on the triggered mechanism, tie tension might also decrease if rocking induced displacements shorten the vault span.

The numerical study also confirmed that the crack opening locations are a function of the geometry and of the applied vertical loads. When large vertical loads are applied, two cracks form in the vault ring close to the springing where the thickness of the section abruptly halves. On the other hand, for decreasing vertical loads, two cracks form in the abutments, close to the vault impost, rather than in the vault crown.

Further experimental tests will be performed on unreinforced and reinforced masonry vaults, to evaluate the efficacy of some strengthening techniques, such as: extrados spandrel walls, thin slab of clay mortar, and FRP. Numerical and analytical models will be improved to simulate cyclic behaviour of the structures.

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