RESTORATION AND STRENGTHENING OF TWO VAULTS OF PALAZZO DEGLI UFFIZI IN FLORENCE (ITALY)

S. Lagomarsino¹, S. Degli Abbati²

¹ DICCA, University of Genoa
Via Montallegro 1, 16145 Genova, Italy
sergio.lagomarsino@unige.it

² DICCA, University of Genoa
Via Montallegro 1, 16145 Genova, Italy
stefania.degliabbati@unige.it

Keywords: Historical Buildings, Static strengthening intervention, Vaults.

Abstract. The restoration and strengthening of ancient masonry buildings represents a very interesting and rather complex issue, because ready-made solution cannot be adopted and it is necessary to define ad hoc interventions, case-by-case developed on the basis of the specific features and the interpretation of damage. So, in order to properly address each strengthening choice, the use of accurate and detailed numerical models can be very helpful. The correct modeling strategies have to be defined through a preliminary deep knowledge phase: only in this way, the model will be able to assess the structural safety, by guaranteeing at the same time also the monument conservation through the adoption of the minimum intervention. In this context, the paper illustrates two examples of restoration and strengthening works carried out on two different vaults of Palazzo degli Uffizi in Florence (Italy). The two cases are very different for constructive features, dimensions and damage pattern and, as a consequence, specific technical solutions and methods for the structural analysis have been used for the design of interventions.
1 INTRODUCTION

In the restoration and strengthening of ancient historical buildings, the different technical features and the possible causes of damage exhibited by the structure make impossible to define a priori an unique technical solution: rather, on the contrary, a proper intervention must be able to get into the specific case. In this context, a proper intervention can not disregard the use of accurate and detailed numerical models, able to quantify the structural safety and to address, at the same time, the more suitable interventions and techniques, guaranteeing in this way also its conservation [1] by the adoption of the principle of “minimum intervention”. Nevertheless, the awareness that structural models now available for the assessment of masonry buildings are affected by uncertainties makes necessary to address the definition of the best modeling approach through a preliminary deep knowledge phase. In this context, the paper illustrates two examples of restoration and strengthening projects carried out on two different vaults of Palazzo degli Uffizi in Florence (Italy). The Palace, designed by Vasari (1511-1574) in 1560 from an idea of Cosimo I [2,3], in order to reform the Dukedom by concentrating in only one building the offices of the Magistrature or Arti Maggiori of the State of Florence, is now subjected to a wide restoration and strengthening campaign (named Grandi Uffizi), started in the 1965 and actually in full swing. In this framework, the strengthening interventions provided for the two above mentioned case studies are illustrated in details in the following, by presenting the results of the structural models adopted to select the best technical solution for each case. In particular, both cases interested two vaults: in the first case, the vaulted system was characterized by a non-serious damage pattern, while in second one, the vault was close to collapse.

2 THE VAULTED SYSTEM IN THE FLOOR OF THE NIØBE HALL

The first case study is the structural vaulted system which supports the Niobe Hall floor, a majestic room on the second floor of the Palazzo degli Uffizi, in the Western Wing of the building. The chamber (Figure 1a) was designed by the architect Gaspare Maria Paoletti, at the end of XVIII century, to hold a large number of roman statues (the sculptural group of the Niobidi and some others), brought at that time in Florence [4]. Recently, the chamber has been subjected to a considerable subsidence and deformation in a small area of the floor, connected to the collapse of a secondary small vault, built on the abutment of one of the main vaults (Figure 1b).

Figure 1: (a) General view of the Niobe Hall; (b) Partial view of the vault system, after removing the pavement and the filling material, with the secondary vault that collapsed.

Even if the trigger was surely connected to the presence of very massive statues and their recurring movement by means of heavy machineries, this failure was due to an original constructive deficiency: in fact, the small vault lacked of an effective abutment on one side, that
on the border with the adjacent cross vault. A vast georadar diagnostic campaign and a detailed survey of the floor levels through laser-scanner highlighted the possibility that similar conditions could be present in other part of the room; thus a strengthening intervention extended to the whole chamber has been provided.

2.1 Description of the vaulted system

The *Niobe Hall* is a wide elongated rectangular room, which is placed above a series of small rooms, of irregular shape and with vaults of different types (Figure 4). The preliminary removal activities of pavement and filling material over the vaults allowed to bare the peculiar constructive system of *Vasari*, characterized by the presence of two orders of vaults: a principal one, formed by structures directly supported by the perimeter vertical walls of the rooms at the lower level; a secondary system of small vaults, set up the main vaults and the vertical walls, with a lightening function. In particular, in the *Niobe Hall*, the main structure consists of four vaults very different for geometry and dimensions: a cross vault; a barrel vault on trapezoidal plan (Figure 2a); a barrel vault with *lunette* (on the archway named *Lambertesca*); a pavilion vault (Figure 2b). Above all of them, a complex secondary system characterized by small and almost flat vaults or arches is present; for example, in the barrel vault, some flatter structures were placed in parallel to two sides, while in the cross vault four small vaults were localized on each edge (Figure 2c). Furthermore, in the final part of the chamber, a complex system of arches, built upon the underlying main vault along the two orthogonal directions, was realized to support the weight of the final chamber transversal wall, which was built upon the vault itself without a continuity at lower levels (Figure 2d).

![Figure 2: Details of the primary (a-b) and secondary (c-d) vaulted systems.](image)

Sometimes, the original shape and continuity of the vaulted system was interrupted by the presence of a large number of rather invasive ancient interventions stratified along the time: three old aeration conduits, which interrupted the continuity of the vaults (Figure 3a), and the presence of an ancient flue, disused at present (Figure 3b). Moreover, close to the final part of the room, a huge area, subjected to past demolitions and later filled with incoherent materials and debris, has been discovered.
2.2 Damage survey at the vaults extrados

The damage pattern surveyed at the extrados shows phenomena of deformation and cracking of different severity in the principal and secondary vaulted systems; however, in general, no dangerous situations have been revealed on the main vaults (Figure 4).

In particular, the cracks are mainly localized in the cross vault (the one characterized by the bigger span) and in the barrel vault with lunette, where the cracks present a transversal direction lightly inclined. On the contrary, all the small vaults of the secondary system have been subjected to localized loss of shape, in some cases even significant, usually determined by concentrated loads due to the presence of heavy statues. The local collapse occurred in the secondary system of the trapezoidal barrel vault; regarding the lightening vaults of the Lambertesca Archway (characterized by dimensions and span more significant than the previous one), they appeared considerably flat and cracked (Figure 5a). Finally, all the intricate secondary systems of the remaining chamber were generally not interested by cracking phenomena, but in some cases appeared visibly slack and warped; in fact, in this area some arches or small vaults were interested by loss of interlocking with the adjacent main structure (Figure 5b).
5b) or sensible loss of shape, mainly due to the above mentioned invasive interventions, stratified in the past.

![Figure 5](image)  
Figure 5: Details of the observed crack pattern (a) and loss of interlocking (b) in the secondary vaulted systems.

2.3 Description of strengthening interventions

Starting from the results of the systematic damage survey, different types of intervention have been provided, which are function of the different observed damage level. In the following paragraphs the main realized interventions are described in details.

2.3.1 Intervention extended to all the vaulted system

First of all, micro-injections of mortar have been extended to all the areas characterized by very fine cracks, in order to fill completely all the voids and restore the vaults original continuity (Figure 6a); for these reasons, the composition of mortar have been selected in order to guarantee a sufficient fluidity and chemical compatibility. Once all the cracks have been filled up, the old filling (constituted by heavy incoherent materials and debris) has been replaced with a new light one (specific weight equal to 600 kg/m$^3$), made by expanded clay kneaded with cement grout (Lecacem Maxi – http://www.leca.it/prodotti/lecacem-maxi/). This new filling is characterized by not negligible mechanical properties, both in terms of stiffness and strength (average compressive strength after 28 days equal to 2.5 MPa). This intervention prevents the activation of possible damage mechanisms in the vaults, without realizing the usual reinforced concrete slab with welded steel mesh. Furthermore, this material seems to be reversible, since the expanded grey conglomerate is easily to crumble by means of a soft mechanical action. Upon the new filling and for the last 5 cm of thickness, an analogous material characterized by the same lightness, but smaller expanded clay balls and with slightly higher mechanical features (average compressive strength after 28 days equal to 5 MPa) has been then casted in place; this is used as a base for the repositioning of the original flooring in marble tiles, which has been directly placed on it through the use of a thin layer of hydraulic mortar.

2.3.2 Introduction of tie-rods

In order to guarantee the connection between the longitudinal walls of the chamber, five tie-rods have been placed at the extrados of the main vaults (Table 1). In the other direction, no connections is needed, due to the continuity of the wall hallway on one side (Figure 4) and to the presence of pre-existing ancient tie-rods on the other one (whose anchor bolt was visible on the façade). All the added tie-rods (pre-tensioned at 20 kN) were inserted inside a sheath and formed by two parts connected through a coupling nut located at about 1/3 of the whole length, in order not to interfere with the vaults below.
Table 1: Main characteristics of the tie-rods (from www.dywit.it).

<table>
<thead>
<tr>
<th>Tie-rod type</th>
<th>Nominal Diameter (mm)</th>
<th>Steel Type (N/mm²)</th>
<th>Yielding Load (kN)</th>
<th>Ultimate Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dywidag 26WR</td>
<td>26.5</td>
<td>950/1050WR</td>
<td>535</td>
<td>580</td>
</tr>
</tbody>
</table>

In most cases, the tie-rods have been anchored to the perimeter walls thorough a passive system (Figure 6b), formed by an injected anchoring with a sock (named Bossong system - http://www.bossong.com/en/strengthening/installing-anchors-controlled-injection.html); in fact, in the examined case it was not possible to use a mechanical anchoring system, since on the hallway side, it would have been necessary to dismantle the flooring and on the opposite side, the presence of some external buttresses made it very complex. The use of the sock guarantees the complete control of the injection, by avoiding leakage of material inside the masonry and by overcoming possible problems concerning the compatibility with the original mortar: the contact between injected mortar and masonry, in fact, is limited just to the cylindrical surface of the hole, and this guarantees a major safety against possible chemical reactions. The anchoring installation has been provided inside 70 mm diameter holes, realized into the masonry walls (calculated anchoring length equal to 50 cm), tried to avoid as far as possible vibrations dangerous for the structures. Then, specific protection pipes have been inserted inside the holes, in order to simplify the anchoring insertion and to avoid the pre-holes obstruction. Finally, the injections on the sock have been realized at a suitable pressure about equal to 3-4 bar.

![Figure 6](image-url)  
**Figure 6:** (a) Details of the mortar injections in all the vault cracks; (b) Details of the injected passive anchoring system.

### 2.3.3 Interventions provided on the secondary vaults

The main aim of all strengthening interventions on the secondary vaults was to preserve as much as possible the original Vasari constructive system. In this framework, each intervention has been properly modulated in function of the specific local damage state.

In particular, once all the thinnest cracks have been filled with mortar micro-injections, the structural continuity has been completely restored by providing some “masonry integrations”, graduated on two different levels: for the vaults interested by a localized damage or past partial demolition, a *cuci-scuci* intervention was sufficient; where the structures were subjected to a prominently loss of shape (so, the condition was irremediable compromised) a complete dismantling and re-construction of the system was realized.

Just in the case of one of the lightning vaults standing on the Lambertesca Archway (which appeared visibly flat and interested by a rather serious crack pattern), a more significant strengthening intervention has been designed (Figure 7). The intervention consisted in eight...
extrados steel arches (formed of welded steel plates 10 mm thick, 50 mm high and at a mutual distance of 500 mm), with the thrusts supported by a tie-rod placed on the arch base, that goes under the original flat vault; the intervention requires very few irreversible works, in particular the holes for the tie-rod. These steel arches have been located in the most deformed area of the vault and connected one each other through welded transversal steel plates (10 mm thick and 50 mm high); in this way, a sort of grid was realized, able to redistribute the applied loads and partially unload the vault, but preserving at the same time the Vasari constructive system. Furthermore, each extrados arch was placed in order not to touch the vault below: so, if the structure in the future will be subjected to some deformation due to the presence of an applied load, no effects on the vault below will occur.

The stress state in a single arch has been evaluated through a FEM analysis, by modelling the steel plates in Ansys [5] as PLANE43, by using a linear, elastic and isotropic constitutive law (Young Modulus E=206000 N/mm$^2$; Poisson coefficient $\nu=0.3$). Furthermore, an applied vertical load equal to 600 kg (calculated as the weight acting on each wheel of a standard transpallet usually used for the statues movement, plus the loads of the statue and its marble basement) have been considered in three different configurations, connected to its possible positions on the vault.

![Figure 7: (a) Details of the crack pattern on the vault; (b) Provided strengthening system.](image)

Table 2 illustrates, for each loading combination: the values of the maximum displacement ($D_{\text{MAX}}$); the values of the maximum tensional state ($\sigma_{T,\text{MAX}}$) and compressive state ($\sigma_{C,\text{MAX}}$); the values of the reaction forces, respectively in the horizontal and vertical direction ($V_x$, $V_y$); in Figure 8, instead, the results obtained in terms of deformed shape and stress state for one of the examined configuration load are shown. As it is possible to deduce, the designed arch has been subjected to a stress state compatible with the reference values of a standard steel.

<table>
<thead>
<tr>
<th>Load Configuration</th>
<th>$D_{\text{MAX}}$ (mm)</th>
<th>$\sigma_{T,\text{MAX}}$ (MPa)</th>
<th>$\sigma_{C,\text{MAX}}$ (MPa)</th>
<th>Edge 1 $V_x$ (N)</th>
<th>Edge 1 $V_y$ (N)</th>
<th>Edge 2 $V_x$ (N)</th>
<th>Edge 2 $V_y$ (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.9</td>
<td>157</td>
<td>232</td>
<td>16500</td>
<td>3000</td>
<td>-16500</td>
<td>3000</td>
</tr>
<tr>
<td>2</td>
<td>2.3</td>
<td>123</td>
<td>210</td>
<td>21108</td>
<td>6000</td>
<td>-21108</td>
<td>6000</td>
</tr>
<tr>
<td>3</td>
<td>1.9</td>
<td>137</td>
<td>189</td>
<td>10558</td>
<td>1056</td>
<td>-10558</td>
<td>4944</td>
</tr>
</tbody>
</table>
DEFORMED SHAPE

TENSIONAL STATE

COMPRESSIVE STATE

Figure 8: Results obtained by the performed FEM analyses in terms of deformation and stress state.

3 STRENGTHENING OF A VAULT AND OF THE WALL BUILT ON IT

The second case consists of a vault interested by a condition close to collapse, due to the presence of a concentrated load transmitted by a wall built on it. As a consequence, it was necessary to provide a double interventions, involving both the vault and the wall. It has been developed in two phases. Firstly, two lattice trusses realized with steel plates have been placed side by side the wall and then connected trough transversal steel bars; since the wall, after this intervention, was supported by this mixed masonry-steel truss, it was possible to disconnect the wall by the vault below. Secondly, the vault has been strengthened trough the use of extrados post-tensioned cables, in order to center the thrust line and partially recover the deformations that were present. In the areas where the vault had lost its curvature, the cables have been linked at the intrados with a plate and steel bars, passing through the wall thickness, in order to exert an active upward action. In the following paragraphs, the intervention has been illustrated in detail.

Figure 9: The condition of the vault before (a) and after the strengthening intervention (b) (Laser Scanner survey from Università degli studi di Pavia and Università degli Studi di Firenze).
3.1 Strengthening intervention on the wall built on the vault

The strengthening intervention for the wall was aimed not to realize an independent truss below the wall (invasive solution which would have required to partially demolishing the wall, with the need to support it by shoring element during the works), but to place side by side to the wall a steel lattice structure, made by steel plates connected to the masonry, in order to create a mixed-masonry-steel supporting structure. More in detail, the intervention consists of a steel truss forms of steel plates located on the two faces of the wall and connected through bars (Φ8 mm), subjected to a pre-tensile stress and welded to the plates, in order to: 1) connect the steel truss to masonry wall by means of friction; 2) avoid buckling effects on compressed steel plates; 3) allow the truss to benefit from the presence of masonry in the compressed parts. Once the truss has been realized, since the wall was supported, it was possible to disconnect it by the vault below in the central part, while in the edges the contact was kept, in order to guarantee the stabilizing effects on the vault itself. Figure 10 illustrates some general view of the truss and a constructive details of a joint.

3.1.1 Design of the steel truss

The truss (1.2 m high, 7.8 m long and characterized by 8 almost square meshes, each of about 0.98 m) has been studied with Ansys program [5], by realizing a FEM model of the structure (each beam was modeled through Link8 element) and considering a total load equal to 400 kN (which consists of the wall weight and of the load transmitted by the part of the roof applied on it). In particular, two different configurations have been analyzed, alternatively considering (“Configuration 1”) or neglecting (“Configuration 2”) the contribution offered by the masonry compressed struts. Figure 11 illustrates the numbering of the elements and the corresponding geometrical dimensions of the section (in mm), corresponding to one of the truss placed on one of each side of the masonry wall. It has to be noticed that, in the case of Configuration 2, the masonry struts have been modeled as further elements (with a transversal section of 240x200 mm², equal to their influence area). The assumed values are: Young modulus for the steel elements: $E_S = 206000$ MPa; Young modulus for the element of the masonry compressed struts: $E_M = 1500$ MPa. In Figure 12, instead, the results of the two configurations are illustrated in terms of: deformed shape; diagrams of the axial forces of the couple of trusses, distinguishing between steel elements and masonry struts in Configuration 1.
Figure 12: Results obtained by the performed FEM analyses in terms of deformed shape and axial force (in N).

Table 3 compares the main results obtained for the two considered configurations in terms of: value of the axial force for steel elements ($N_{S,max}$) and masonry struts ($N_{M,max}$) and the corresponding element number ($EN$); the value of the maximum displacement $D_{max}$.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>$N_{S,max}$ (kN)</th>
<th>EN (Steel)</th>
<th>$N_{M,max}$ (kN)</th>
<th>EN (Masonry struts)</th>
<th>$D_{max}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>139</td>
<td>24 - 31</td>
<td>48.5</td>
<td>34 - 37</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>165</td>
<td>24 - 31</td>
<td>-</td>
<td>-</td>
<td>12</td>
</tr>
</tbody>
</table>
3.2 Strengthening intervention on the vault

As above mentioned, the vault was considerably deformed in the part subjected to the presence of the wall built on it, with some cracks both in the extrados and in the intrados. So, an active intervention has been provided [6], consisting of the use of 4 extrados post-tensioned strands (Φ16 mm, pre-load equal to 15 kN), placed each couple in correspondence of the support between the lunettes, sheathed and anchored by means of traditional anchoring plates (Figure 13). As a consequence, by pre-stressing the cables, they applied a radial pressure on the vault with an entity directly proportional to the tensile action in the strand and inversely to the curvature radius; as a result, the original shape of the vault has been partially restored and the line of thrust has been almost centered again. Furthermore, in the areas where the vault had lost its curvature, the cables have been also linked at the intrados with a plate and steel bars in order to exert an active upward action (Figure 13c/d). It is important to notice that, during the realization of the intervention, the vault profile has been monitored by means of a Scanner survey (performed by Prof. Arch. S. Parrinello of Università degli Studi di Pavia and Arch. S. Porzilli and Arch. G. Pancani of Università degli Studi di Firenze), in order to take under control the strengthening intervention effects. Then, once part of the deformations have been recovered, all the voids have been filled with mortar and sometimes with a localized cuci-scuci intervention.

![Figure 13: General view of the strengthening intervention on the vault (a, b); some details on the extrados (c) and on the intrados (d).](image)

3.2.1 Verification and strengthening intervention on the vault

The static assessment of the vault has been developed graphically, by using a “Thrust Line Method”, verifying that the curve of pressure was contained into the middle third (in order to avoid cracking) and however far enough from the intrados or the extrados (in order to ensure the stability and to guarantee compressive stresses compatible with the strength of masonry) [7]. So, starting from the survey of the deformed extrados of the vault, a portion of it (1 m long and 15 cm thick) has been considered; then, the section has been divided into 35 blocks.
with homogeneous dimensions and geometry. The blocks 1, 2 and 34, 35 have been defined by supposing the presence of a lateral abutment, whose presence is reasonable because it is justified by the dimensions of the examined vault.

Once realized the drawing in Autocad, for each block of the vault and of the filling above the weight and the center of gravity position have been considered, in order to obtain the line of thrust.

In particular, the following configurations have been analyzed:
- **Configuration 1**: the vault is considered in the state before the wall was built, subject to its own weight and with the weight of the filling (14 kN/m³);
- **Configuration 2**: the presence of the load of the wall built on the vault without continuity at lower levels is also considered, evaluating the mass below the unloading arch naturally formed in the wall (8.6 kN/m);
- **Configuration 3**: it corresponds to the design state, where the vault is considered strengthened with the extrados strands; in this configuration the weight of the vault and of the new light filling (6 kN/m³) has been considered, in addition to the effect of the active strengthening of the extrados cable and of an accidental load equal to 5 kN/m² (corresponding to the museum function).

Figure 14: Results obtained in the three configurations in terms of thrust line.
The results obtained in the three different configurations in terms of thrust line (shown in Figure 14) highlighted that:

- In the original state (Configuration 1), the curve of pressure (identified in red) is not contained entirely in the middle third (the blue and magenta lines delimit the reacting part of the cross-section); a reduction of the effective section occurs in the part close to the deformed area. However, the original outline of the vault, before the damage, was surely more regular, so almost certainly the vault was verified;

- In Configuration 2 it is not possible to find a line of thrust contained into the middle third due to the presence of the applied concentrated load; this determines a raising of the curve near to the wall toward the extrados with a consequential cracking at the intrados (as the observed damage highlights). The stability is, as a result, significantly precarious and the collapse did not occur probably thanks to the contribution of the filling and of the lateral abutment;

- Finally, in Configuration 3, the strengthening intervention centers again the line of thrust, determining a raising of the deformed areas, too.

4 CONCLUSIONS

As highlighted by the two above examined case studies, it appears clear as the strengthening of an ancient historical building represents a very challenging issue. In fact, even in two cases apparently easy as those illustrated in the paper, the different specific features and starting damage conditions made necessary to modulate each intervention, in order to guarantee the most appropriate strengthening choice. In this context, a preliminary detailed knowledge phase combined with a proper use of numerical models help to address the best technical solutions, quantifying the monument structural safety and guaranteeing at the same time its conservation.

ACKNOWLEDGEMENTS

The authors would like to thanks the building enterprise CMSA (Cooperativa di Muratori, Sterratori ed Affini) for the material made available, in particular Eng. Sirio Orsi and Eng. Andrea Ulivelli, as well as all workers of Cantiere Nuovi Uffizi of Florence, in particular Eng. Alessio Brogi and the master builder Marcello Fragai.

REFERENCES


