Static and seismic retrofit of masonry arch bridges: case studies

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ABSTRACT: Thousands of road and railway masonry arch bridges are still in operation in the Italian transportation network: most of them need being improved in their carrying capacity and to be upgraded to the standards of the current seismic code. In this paper three case-studies of the static and seismic retrofit of historical masonry arch bridges are presented, outlining some methodological approaches to the renewal intervention according to the different typological characteristics of the bridges and their state of maintenance. The main phases of work, combining both traditional and innovative strengthening techniques, are described. In the S.Gallo Bridge the load bearing capacity of the existing structure has been preserved and increased through a thickening of the old arch with a new layer of brick masonry and the application of CFRP laminates. Many refurbishment techniques, derived from the historical heritage restoration field, have been used for the Rio Moline Bridge, where new longitudinal internal brick spandrel walls connected to the extrados of the vaults have been built to share some of the load and enhance the seismic resistance. In the case of the Gresal Bridge the seismic vulnerability has been reduced by creating a new structural arrangement through a new rc slab anchored to the piers with vertical ties and restrained at the abutments, collaborating with the existing structure in carrying horizontal loads. Appropriate analytical models comparing the load bearing capacity before and after the repair intervention have been implemented to prove the effectiveness of the strengthening techniques.

1 INTRODUCTION

Old masonry and stone arch bridges currently represent a large proportion of the Italian road and railway bridge stock: many of them are part of the historical heritage of the XIXth century and the public network authorities are charged not only with their routine maintenance but also with refurbishing and retrofitting interventions. Due to the intrinsic weakness of some structural components, to deterioration phenomena and to the updating of structural codes, many old arch bridges show inadequate structural performance considering the static and seismic requirements of the current codes.

In the framework of a Bridge Management System it could be useful to define some methodological approaches and repair techniques for the rehabilitation of these structures, and their upgrading to the current functional standards.

In this paper three case-studies are presented, evidencing the typological characteristics of the bridges, the inspections and tests carried out during the structural survey, the retrofit project and the work phases on site. Some synthetic results of static and seismic analyses comparing the structural capacity before and after the repair intervention are also discussed to prove the effectiveness of the strengthening intervention.

2 THE SANDRO GALLO BRIDGE

2.1 Description of the structure and structural survey

The upgrading of the “Sandro Gallo” brick masonry arch bridge was carried out in the framework of some maintenance works for the improvement of about 1 km bank sides of the Excelsior Canal at the Lido (Venice). The bridge was built in the XIX and in the first decades of the XX century, with a substantially homogeneous structural arrangement, and is currently used as vehicular bridge.

The Venice administration decided to upgrade the load bearing capacity of the bridge to the rank of 1st category, as defined by the Italian standards. The bridge consists in a brick arch, with a thickness of about 0.36 m (three brick layers) in the central part, and of 0.55 m (four brick layers) in the lateral part.
close by the springing. The abutments are made up of brick/trachyte masonry fixed with poor quality mortar, from -1.00 m to -5.90 m below the road surface in the older part, and mainly by a massive concrete structure (thickness 2.70 m) in the more recent part corresponding to the widening of the bridge.

The structural investigation consisted in three core samples, three single and one double flat jack tests, performed on the structure of the masonry arch. Two more core samples were taken vertically in correspondence of the abutments of the bridge, on both sides.

The results obtained from the preliminary tests defined the morphology of the arch: the core samples allowed to determine the thickness of the masonry arch at the crown (0.37 m) and at a distance of 1.27 and 0.53 m from the abutment (thickness of 0.47 and 0.55 m, respectively). The flat jack tests were performed on different points of the masonry arch (at 1.00m, 1.70m and 1.90 m from each abutment). The results revealed a moderate state of stress at all the points tested (0.25, 0.24 and 0.25 MPa respectively) and the masonry showed a fairly good response in terms of compressive strength (2.00 MPa).

The core samples on the two abutments of the bridge were taken to determine the morphology of the underlying structures and the soil stratigraphy. The samples showed the presence of a 1.00-1.60 m layer of a gravel, sand and cobblestones filling under the road surface, then a difference in terms of foundations characteristics and materials was found, as previously described. The soil was finally examined as deep as 15.00 m below the starting level of the foundations: the core sample disclosed a sequence of silty sand and clayey silt. Finally, between the depths of -6.00 and -7.50 m, a timber pile was found.

2.2 The repair intervention

The aim of the intervention is the substantial conservation of the bridge structure and the enhancement of the actual load bearing capacity; the upgrading foresees the utilization of the existing structure, strengthened by the use of innovative and traditional materials and by the implementation of removable or substitutable intervention techniques.

The central part of the arch span has thickened by the insertion of one more layer of bricks. The thrust inside the vault is transferred from the new elements to the older structure in correspondence of the thickening of the arch (from three to four bricks, at the springers); locally, the connection between the new and the old masonry has been improved by means of brick units and metallic dowels, used as shear connectors (see Figs. 3, 4), glued to the older part with epoxy resin. Moreover, the intrados of the masonry arch have been restored in a “traditional” way (cleaning of the surface, removal of the plaster, substitution of the most damaged bricks with new ones, excavation of the deteriorated part of the mortar joints and repointing with proper hydraulic-lime based mortar ($f_{cm} = 18$ MPa, $f_{hm} = 7.8$ MPa), and final repositioning of the plaster).

At the level of the abutments, a new foundation structure, positioned on micro-piles has been constructed: the new foundation is made of reinforced concrete and is positioned aside the existing abutments, externally in respect of the canal. It is designed to bear the increase of the horizontal and vertical loads transmitted by lateral friction from the existing foundations. It is on purpose molded with a saw-tooth shape to better transmit the thrusts to the new structure.

The works to be carried out on the lower structure of the bridge consist in the insertion, at the level of the abutments, of 2.00 m long timber piles, connected at their upper end with a reinforced concrete beam, to avoid possible damage to the submerged structures. The repair interventions to be performed at the extrados are subdivided into subsequent phases:
- removal of the internal filling of the arch;
- preparation of the horizontal level for the positioning of the concrete foundation beam;
- execution of the sub foundation micro-piles (diameter 200 mm) with an internal reinforcement made up by a steel hollow bar (external diameter 101.6 mm, thickness 10 mm);
- casting of the horizontal reinforced concrete beam, inclusion of the micro-piles upper ends inside the beam;
- construction, close by the springing, of a new masonry arch layer, regularizing the extrados structure and being connected to the old masonry;
- thickening of the existing masonry structure in the central part of the span, positioning of brick units orthogonal to the axial line of the arch used as connectors between the old and the new masonry, positioning of steel rods, diameter 20 mm, with the same function, glued to the old structure with epoxy resins;
- preparation of the upper surface of the arch and placing of the CFRP: removal of the damaged bricks and substitution with new ones, excavation of the deteriorated parts of the mortar joints and repointing with the same hydraulic-lime based mortar used at the intrados, application of a hydraulic-lime based mortar layer and smoothing of the external surface, positioning of the Carbon Fibers with previous application of primer and epoxy adhesive, final protecting cover;
- re-filling of the upper part of the arch with the same material previously removed, to reach the road level;
- closing of the 1st phase and moving of the work site to the 2nd symmetric part of the bridge.

2.1 Structural assessment

The verification concerns the structure of the bridge subjected to increased traffic loads. In the calculations a three point load of 200 kN (a conventional truck of 600 kN on three axles) distributed on the structure, is taken into account; due to the load diffusion through the filling, a reductive factor of 0.5 is considered. The dynamic amplifying factor is equal to 1.2.

A first step consists in analyzing only the masonry structures, completed with the insertion of the new brick units, in the central part of the span and at the springers. As a calculation method, a limit plastic analysis is carried out: two load cases are considered, the first presenting the three point load at
quarter span, the second presenting the three point load in central position. The Safety Factor is 2.24, being the minimum thickness required by the calculations and the one of the restored arch respectively 0.214 and 0.48 m.

The second level of analysis considers the connection of the CFRP fibers to the bridge structure. For the evaluation of the safety condition, the load combination presenting the three point load at one forth of the span is considered; a safety domain is determined and the force/moments combinations on the structures of the bridge are inserted in the same diagram. Assuming a rectangular stress-block diagram for the ultimate stresses on the masonry and an elastic behavior for the CFRP, an ultimate strain of 0.006 and a compressive strength of 2.50 MPa, evaluated on the basis of the flat jack test results, are taken into account for the masonry. The live and dead loads are amplified with the coefficient 1.50, both for the masonry and the CFRP.

\[
\frac{M_{\omega}}{h^2f_{u,s}} = \frac{1}{2} \omega \left[ \frac{1-x}{t} + 0.4 \left( \frac{x}{t} \right) \left( 1 - 0.8 \frac{x}{t} \right) \right]
\]

(1)

\[
\frac{x}{t} = \frac{\gamma_{u}}{1.6 \rho} \left[ N_{\omega} - \omega + \left( \frac{\omega - N_{\omega}}{f_{u,s}} \right)^{\frac{3}{2}} + 3.2 \omega \right]
\]

(2)

where \( t \) and \( l \) are the height and width of the section, respectively; \( f_{u,s} \) is the compressive strength of the masonry; \( x \) is the neutral axis depth); \( M_{\omega} \) and \( N_{\omega} \) are the bending moment and axial load defining the safety domain; \( \gamma_{u} \) is the partial safety factor for the masonry. The parameters \( \omega, \rho \) and the ratio between the CFRP and the masonry ultimate strains are defined as follows:

\[
\rho = \frac{A_{cfrp}}{A_{lt}} \quad \text{(CFRP area fraction)}
\]

(3)

\[
\omega = \frac{E_{cfrp}}{E_{lt}} \frac{f_{u,s}}{A_{cfrp}} \quad \text{(CFRP normalized area fraction)}
\]

(4)

\[
\frac{E_{cfrp}}{E_{lt}} = \frac{1-x}{t} \quad \text{(ultimate strains ratio)}
\]

(5)

where \( A_{cfrp} \) is CFRP cross-sectional area; \( E_{cfrp} \) is the Young’s Modulus of the fibers; \( E_{lt, u} \) and \( E_{lt, d} \) are the ultimate tensile and compressive strains for the fibers and the masonry, respectively.

The structural assessment is completed by the verification of the inclusion in the safety domain of the points corresponding to the couples of design axial forces/bending moment.

Figure 5. Moment capacity versus axial load

### 3 THE RIO MOLINE BRIDGE

#### 3.1 Description of the structure and structural survey

The bridge over the Rio Moline river is located in the Trento Province, in the North of Italy. It was built approximately in the XVIII century and consists of two arches of different length, (about 6.80m for the span on the orographic right side and 7.50m for the span on the left), with one central pier standing in the middle of the river bed. The arches are slightly stilted, with stone voussoirs of variable thickness (40-45cm); the central pier is 1.70m wide, and the abutments stretch to each side with wing walls 4.50m long, making the total length of the bridge about 24m. The bridge is one-way; the platform sloping with an incline grade of 5% is only 3m wide and has a total width of 3.90m. Because of these geometric constraints the bridge is to be used, also after the repair intervention, only for light traffic and is classified as 3rd category (live load=5.0 kN/m²).

The masonry characteristics and the repair state were investigated during the structural survey of September 2007; the bridge appeared in a very poor condition, and a provisional wooden shoring with steel ties had already been put in place. Exploratory wells were made for the determination of the stratigraphic layers of the flooring and the filling, and continuous inclined and horizontal cores and subsequent endoscopic probes were performed at the springing and in the pier and abutment elevation walls. The masonry was significantly affected by loss of mortar joint; the erosion in the joints had progressively led to loss of effective connection between the stone blocks and some of them had become loose. The surfaces at the intrados were also partially damaged by the presence of vegetation which deepened its roots between the joints.

In the stone vault structure, the surveys showed the presence of discontinuities and cavities and a filling
characterized by loose material with traces of earth; the abutments and the central pier have walls made up of an external curtain of larger blocks, 45-55cm thick, and an internal nucleus constituted of dry stone masonry, with decimeter-sized stones. The pier and abutment foundation level was not reached with the drilling bore, that was stopped at a depth of -1.50m.

3.2 The repair intervention

Essentially, the repair intervention involves the rehabilitation of the existing structure by providing specific strategies to recover the structural integrity of the arches so that they can bear vehicular traffic loads and improve element connections to upgrade seismic resistance. Many of the refurbishment techniques used are derived from the historical heritage restoration field, and the structural capacity of the existing structure is preserved and enhanced with strengthening interventions that are chemically and mechanically compatible with the ancient construction.

A general refurbishment of the masonry walls (spandrel walls, pier, abutments, wing walls) has been executed applying traditional techniques such as:
- injection of grout based on hydraulic lime (particularly suitable for consolidation of masonry with cavities);
- repointing of the stone joints with proper hydraulic lime mortar;
- local masonry reconstruction by manual methods.

As to the repair and strengthening of the stone vaults, the procedure consisted in the following steps:
- temporary removal of pavement and the existing fill (with the installation of temporary shoring);
- cleaning the lesions with compressed air and removal of the degraded mortar with scrapers;
- positioning of cannulas with a pitch of about 40 cm (at the intrados and extrados of the vault) and subsequent grout injection;
- applying tensioning wood or plastic wedges;
- repointing of the mortar joints;
- construction of the internal brick spandrel walls, (25cm thick) connected to the extrados of the vaults and application of the CFRP Fibers to the spandrel walls lateral surfaces (with previous application of primer and epoxy resin);
- insertion of 16mm stainless steel ties, applied at the vault extrados on a proper hydraulic-lime based mortar layer, and anchored to the lateral spandrel walls.

As for foundations, no specific consolidation technique has been adopted, considering that no evidence of foundation failure was found, and no sensitive increase of loads is expected; cyclopean stones have been used to protect the middle pier from foundation undermining.

Figure 6. Panoramic view of the Rio Moline Bridge.

Figure 7. Longitudinal section with new structural elements (rc slab, micropiles and vertical ties)
3.1 Structural assessment

In the initial state the analytical model tries to reproduce the effect due to the cracks in the masonry: in particular the loss of effective connection between the arches and the lateral spandrel walls has been considered, so that the vaults work independently. A uniform distributed crowd live load of $4 \text{kN/m}^2$ has been used in the model, and an additional load case with a conventional truck of $120 \text{kN}$ on two axles (a 35 KN front axle and a rear, just 3.00 m distant, of 85 kN) for the passing of an exceptional load has been introduced. The dynamic amplifying factor is equal to 1.4.

After the repair, the insertion of the new internal brick spandrel walls connected to the extrados of the vaults contribute to bear some of the load acting on the vaults and enhance the seismic resistance. The same effect have the lateral spandrel walls, which work as rigid load-bearing walls after the grout injection and the restoration of the effective connection with the arches. The brick normal stresses due to the point load of 120 kN positioned in the centre of the longer arch are reduced from 0.59 MPa to 0.32 MPa, as can be seen in Fig.8.

4 THE GRESAL BRIDGE

4.1 Description of the structure and structural survey

The Gresal bridge is located in the North-East of Ita-
ly, in the Belluno province; it was built in the XIX century and is currently used as a vehicular bridge, representing an important overcrossing of the Gresal river for the region road network. The structure is a three span stone masonry arch bridge, with a total length of 67.40 m: the three spans are almost equal, the single arch clear length being about 15m; their shape is almost semicircular with a radius of 7.39m, slightly increasing at the springers.

The average thickness at the crown is 0.60 m. The maximum height of the two piers, which are tapered between the bottom and the top, is 12.75m and their section is rectangular; at the bottom the pier section is 3.50x6.99m, with the bigger dimension orthogonal to the bridge axis. The roadway is 6.09m wide, and laterally the spandrel stone walls emerge beyond the deck level forming two 45cm thick parapets.

The structural investigation has consisted in three core samples taken in the stone arch and a geometrical survey. Two vertical core samples have been extracted; the first from the central part of the arch and the second from the pier; the third one has been drilled in the abutment, on an inclined plane. These investigations have allowed to determine the thickness of the brick stone, the layering of the material between the pavement and the masonry vault and to characterize the mechanical property of the infill material. In the center of the arch the layer of the infill is 0.75 m and the masonry thickness is 0.60 m. The infill has good mechanical characteristics and is made by loose material, mostly stones and pebbles.

4.2 The repair intervention

The project interventions have been decided on the basis of a preliminary seismic analysis and in the light of the results obtained from structural investigations. The bridge was rated highly vulnerable to seismic action, mostly due to the slenderness of its high piers.

The proposed intervention is conceptually very simple and it fully relies on the intrinsic load-bearing capacity and design characteristics of the existing structure, which have been preserved, and even enhanced, in their original configuration.

The repair intervention has been carried out in different phases:

− a thin portion of the internal infill layer (and of course the pavement) has been removed, with the aim of saving as much as possible the fill material with the best mechanical properties, which acts with a stabilizing function, maintaining the vault voussoirs under compression;
− a new 25 cm thick r.c. slab has been cast over the whole bridge length. The r.c. slab is anchored to the abutments, where new reinforced concrete plinths on micro-piles have been positioned outside the existing masonry abutments. In the transversal section the micro-piles have been disposed in two inclined rows in order to transfer the action to the ground, and at same time oppose the abutment overturning;
− new high strength bars (26.5mm diameter) have been placed inside the two central slender piers, in vertical holes drilled from above for the entire height of the pier and reaching the foundations; the bars are anchored at the top to the r.c. slab as well, and the combined action of the r.c. slab, vertical bars and “confined” infill allows the creation of a new resisting strut and tie scheme in the longitudinal direction, opposing the seismic load. In the transverse direction the vertical reinforcement enhances the pier resistance to combined bending-compressive stress states;
− the spandrel walls of the arches have been restored in a “traditional” way: the surface has been cleaned, damaged bricks have been substituted.

Figure 11. Pier’s section and abutment’s transverse section
with new ones, the deteriorated part of the mortar joints has been excavated and re-pointed with proper hydraulic-lime based mortar while stainless steel bars 6mm in diameter have been inserted in the mortar joints. This phase has been concluded by the placement at the new rc slab level, of transversal 24mm diameter stainless steel ties which restrain the walls at the top and avoid out-of-plane overturning. It has to be noticed that the repair intervention has increased the structure dead loads only by about 1%.

4.3 Structural assessment

The effectiveness of the strengthening technique has been tested with appropriate analytical models, comparing the seismic capacity of the bridge before and after the repair intervention. A seismic assessment of the masonry arch bridge has been carried out through non linear static analyses with a 3d F.E. model using Drucker-Prager failure criterion, characterized by the values of friction angle $\phi$ and cohesion $c$, which can be expressed in terms of uniaxial tensile strength $f_t$ and uniaxial compressive strength $f_c$. In the model the following values have been adopted:

- for the masonry
  
  $f_c = 1.50$ MPa, $f_t = 1/10 f_c$,  
  $\sin \phi = (f_c-f_t)/(f_c+f_t)=0.82$  
  $c = (f_c f_t)/(f_c+f_t) \tan \phi = 0.47$  

- for the fill material

  $f_c = 0.10$ MPa, $f_t = 1/2 f_c$, $\sin \phi = 0.33$ $c = 0.04$

The capacity of the bridge has been increased by the retrofit both in the longitudinal and transverse direction; the latter was the most vulnerable in the former structure, mostly due to the slenderness of the high piers. After the repair, as reported in Fig.12, even if the elastic stiffness of the bridge seems to vary less, the ultimate displacement capacity in the inelastic field increases considerably so that the displacement seismic demand can be satisfied.

5 CONCLUSIONS

The retrofit design of old masonry and stone arch bridges requires a very complex approach: it starts with the assessment of the actual structural behaviour by means of a series of experimental and theoretical analyses and it ends with the choice of the proper intervention both in terms of materials and application techniques.

In this paper three case studies of the static and seismic retrofit of masonry arch bridges have been presented: the interventions consisted in the combination of traditional and innovative strengthening techniques, defined according to the different typological characteristics and the bridge maintenance state. In these examples some methodological approaches to the structural restoration and strengthening of such constructions are outlined and might suggest standard repair methods to be adopted in a Bridge Management System.

REFERENCES


J. Heyman, The Masonry Arch. 1982, Ellis Horwood Limited