STRENGTHENING AN EARLY 20TH CENTURY RC BRIDGE WITH FRP (UNIDIRECTIONAL CARBON FIBRES).

G. Pistone

Department of Geotechnical and Structural Engineering, Architecture, Turin Politecnico, Italy.

ABSTRACT

The bridge was built in the 2nd decade of the twentieth century on a large canal (Naviglio) in the park around the Royal Castle of Racconigi (Piedmont, Italy). The bridge consists of a fine looking arch structure with a span of 14.65. It has 4 slender ribs, supported by masonry shoulders, and is topped by a reinforced concrete deck.

The strengthening of the structure was undertaken not so much to remedy the deterioration of the material (not very pronounced) as to adapt the bridge to carry heavier loads, such as fire engines, concrete mixer for maintenance works to the existing buildings, etc., since the bridge is the only means of access to the innermost part of the park.

In addition to strengthening the foundation by means of micro piles, it was deemed necessary to improve the mechanical performance of the bridge through measures that would not entail conspicuous alterations to the original structure. The bridge, in fact, is protected by the National Authority for the protection of the cultural heritage as an important r.c. bridge prototype.

Due to the slenderness of the ribs, with a predominantly flexural behaviour, it was necessary to increase the stiffness of the structure. This objective was attained by changing the static situation of the bridge from simply supported to restrained, and by increasing its cross-section through the addition of a layer of reinforced concrete (only 10 cm thick at the midspan) to the upper surface, and 8 layers of FRP, with a total thickness of ca 1 mm, to the 4 ribs.

As a result, stiffness was greatly improved, mostly on account of the tensile resultant being tangent to the lower surface. Moreover, shear forces are absorbed by 3 FRP bands glued to the sides of the ribs at a 45° angle.

KEYWORDS

Reinforced concrete, arches, reinforcement.

INTRODUCTION

The “Ponte del Naviglio” in the Racconigi Park is a bridge connecting an island, called “Isola dei Daini”, to the rest of the park; there are other bridges across the canal (Pistone 2004), but they all have an insufficient capacity for the loads envisaged by the applicable regulations on heavy vehicle traffic. The need to reach the island at least at one point with relatively heavy road vehicles is associated with service requirements such as ensuring the access of fire engines if a fire should break out, truck mixers for maintenance works on the structures and infrastructures of the island, etc.

The bridge in question was selected in view of its good conditions of repair and its having been constructed with good materials: the aim of the intervention was to adapt it to the requirements established for 2nd class roads.
The design load for a second class road, in the particular case of the bridge in question, with respect to the most adverse design assumption, consists of a series of 3 concentrated loads, 15 tons each, spaced 1.50 m apart. These loads must be multiplied by a dynamic coefficient which is a function of the span of the bridge (1.4 in this case). Obviously, the positions of the loads must be varied to obtain the highest possible bending moment and shear values.

**TYPE OF BRIDGE**

The time of construction is not documented, but it surely dates back to the period between the two world wars of the twentieth century, when it was probably built to replace an earlier bridge with a metal or wooden structure. The static scheme consists of a slender arch in r.c., with a net span of 14.60 m, restrained at the ends (Fig. 1). The deck consists of an arch slab (rise of ca 78 cm) with thickness ranging from 21 cm at the key to 31 cm at the impost, supported by 4 arch ribs, set at lower level relative to the slab (by 28 cm at the key and 100 cm at the impost) (Fig. 2). While the two central ribs do not project out from the extrados of the slab, the two side ribs have projecting portions on either side of the deck that are used as sidewalks. For the latter two ribs, the upper surface of the raised portion obviously remains level with the sidewalk (level over the entire span of the bridge). At either end of the bridge, integral concrete castings provide stabilising abutments, in their turn covered externally with a layer of solid bricks.

![Figure 2. View of the intrados of the bridge](image)

**SURVEYS PERFORMED ON THE EXISTING STRUCTURE**

The geometric survey was completed with a study of the extrados and the abutments, as these elements were laid bare only during the design stage. Minor dismantling works were also performed both at the intrados and at the sides of the ribs in order to identify the quality and the diameters of the steel used. The results were satisfactory as far as the concrete was concerned, since the goal of the designer was not to determine whether the existing concrete would be able to withstand the maximum stresses specified for the bridge in its strengthened version, but rather to assess the quality of the existing material to make sure that it would be able to collaborate effectively with the strengthening materials. To this end, two 90 mm diameter cores were drilled and tested in the laboratory, obtaining strength values > 30 MPa. However, it became immediately clear that steel quality and diameters would be a problem. It proved impossible to take samples of the steel bars to perform laboratory tests, as this would have destroyed exceedingly big portions of the structural members. An estimate was worked out on the basis of the materials used at the time of construction of the bridge and from bibliographical data (Mörsch 1909, Marrullier 1921, Sabatini 1923). The strength to be attributed to steel was estimated as 100 MPa. The arrangement and the cross-sectional areas of the reinforcing steel seemed very precarious in the light of present-day detailing criteria:

- absence of evenly distributed stirrups;
- but few large diameter bars distributed in an irregular fashion, at various heights: the diameters measured were 26 mm.

**CONCEPTION OF THE STRENGTHENING WORKS**

Once laid bare at the extrados by removing the roadbed and the filler material, the bridge displayed a modest section at the key, which made it difficult to work either in arch form or in the form of a simply supported beam.
The Protection Authority required a strengthening method that would ensure the performances typical of 2nd class roads without making drastic changes to the architectural features of the bridge.

The structural profile of the bridge clearly shows that it was designed from the very start as a reinforced concrete arch restrained at the ends: this can be seen from the section, deep at the ends and narrow at midspan, conceived to withstand appreciable bending moments, albeit within the limits entailed by its construction with rigid restraints at the ends.

In particular, the arch profile is deemed appropriate to carry distributed loads, while for concentrated loads the arch must be able to provide a response in bending and sufficient tensile strength.

In view of its low height at the centre (49 cm, between the slab and the rib) the section can only carry modest bending moments: in the case of a simply supported beam and a clear span of 14.60 m, the loads associated with 2nd class roads entail exceedingly high bending loads at the midspan.

The original type of restraint is not sufficient to prevent rotation: it is a concrete block, 5 m long on either side, with a height tapering gradually to ca 50 cm at the outer edges; moreover the concrete block is simply rested on a soil having poor mechanical properties, which perhaps were supplemented with wooden piles, of which, however, no trace was found during our surveys.

The preliminary calculations demonstrated that the arch, with its original shape and sections, was able to withstand the loads envisaged, albeit in limit conditions, as long as it worked as a perfectly restrained arch.

Under this assumption, the project focused on two primary goals:
- produce restraint conditions at the ends that would be able to reduce the bending moment at midspan down to about one-third compared to the simply supported configuration;
- use additional material to withstand the loads at midspan of non negligible magnitude.

**STRENGTHENING PROJECT**

The strengthening project was drawn up by taking into account the preliminary checks and, in particular, by considering that:
- the static scheme to be complied with was that of an arch restrained at the ends;
- under this assumption, the concrete could withstand the maximum bending moments both at the ends and at midspan, albeit in limit conditions, whereas it was indispensable to supplement the steel reinforcement;
- the structure had to be equipped with shear reinforcement, altogether lacking in the existing structure.

Besides the aforementioned conditions it was necessary meet the request not to alter the profile of the bridge and to retain its current shape, without any substantial additions as might introduce dramatic changes to its appearance.

In order to meet the first requirement, it was necessary to anchor the abutments of the bridge by means of micro piles, in order both to support the vertical loads (vertical micro piles) and to block the rotation (slanted micro piles). The low thickness of the current counterweight required the casting of a thick layer of concrete above it and made integral with it: the ends of the micro piles, fitted with ad hoc anchoring handles, were embedded in the mass of newly added and pre-existing concrete.

![Arrangement of the reinforcement of the bridge](image-url)
The freshly placed concrete was connected to the earlier casting by means of stakes calculated as a function of shear stresses. In this manner it proved possible to increase the section both at midspan and at the restraints (Fig.3).

The other two requirements were met with the aid of unidirectional carbon fibre bands, (CFRP - Betontex system by an Italian company, FTS) 200 and 300 mm wide, characterised by a 480 MPa strength (modulus of elasticity > 240 GPa) (Fig. 4,5).

![Figure 4. Detail of the longitudinal section](image)

![Figure 5. Gluing the FRP to the intrados of the bridge](image)

To withstand the tensile stresses it was necessary to fit 8 overlapping bands, adding up to a total section of 424.80 mm$^2$; gluing them to the intrados afforded the substantial advantage of making optimal use of the concrete section; disregarding the pre-existing reinforcement, we get the maximum arm obtainable for the internal resisting torque and hence the greatest possible resisting section. However, there is a high risk of debonding, precisely because the bands are glued to a concave surface and are subjected to high tensile stresses.

In order to prevent this phenomenon, the presence of the bands put in place to withstand shear was used: these bands, set at a 45° angle, arranged in 3 layers on each side and over the entire length of the ribs, were glued so that they would overlap and constitute a bandage with the first seven layers of longitudinal fibres; the eighth layer of longitudinal fibres was glued over all the bands of the shear bands, to serve as a surface finish at the intrados and to transfer its strength to the rib to which it was applied. The top end of the shear bands was wrapped around 8 mm diameter carbon fibre bars, then it was introduced into holes passing through the bridge slab and bent at its extrados, and at this point it was glued to the slab so as to ensure an extremely secure anchoring system for the bundle of slanted bands (Fig. 6).

![Figure 6. Details](image)
The arrangement obtained will prevent the debonding of the intrados layers, while the bandage, produced through the systematic overlapping of the fabrics on the sides and at the intrados of the ribs, ensures optimal bending and shear behaviour. Carbon fibre bands withstand a maximum bending stress of 160 MPa; shear is also absorbed by fitting carbon fibre designed to withstand tensile stress of a similar level. A sufficient reserve of capacity to failure is obtained by adopting a serviceability strength to failure strength ratio of 3 (safety coefficient). The maximum design strain of the fibre is 0.006, evaluated for a modulus of elasticity of 240 GPa.

EXECUTION AND RESULTS OF THE INTERVENTIONS

During the second half of 2006 and until March 2007, the strengthening intervention was performed in compliance with the design provisions and the requirements identified initially, i.e.:
- the increase in rib width was maintained below 10 mm;
- the concrete castings at the impost of the arch, all of them below the walking floor, did not alter in any respect the slender profile of the original structure;
- the structural response is excellent as borne out by a recent loading test performed on the deck (Fig. 7, 8-tables 1, 2).

![Figure 7. A stage of the recent loading test of the bridge](image)

![Figure 8. Arrangement of the deflectometers](image)

Proof testing was performed by loading the bridge, without roadbed, with trucks filled with soil, previously weighted in order to determine the load on each axle shaft. Moreover, this load not being sufficient, concrete blocks were placed at the midspan, as can be seen from the photo in figure 7. As for the bending moment, in consideration of the maximum design moments, of $M=1377.67\text{KN}\cdot\text{m}$ (at midspan) and $M=2435.20\text{KN}\cdot\text{m}$ (at the restraints), the trucks were arranged with the rear next to the concrete blocks, so as to have the heavier axle shafts in the central portion of the bridge. In this manner, the bending moment induced was $M=1403.32\text{KN}\cdot\text{m}$ at midspan and $M=2503.14\text{KN}\cdot\text{m}$ at the restraints. As for shear, it was not possible to reach the maximum design value (567 KN) during the test for lack of ballast: shear at the supports was 542 KN, which, at any rate, comes very close to the design value. According to Italian standards, the criterion for the acceptance or rejection of the proof test is that the test should...
be repeated if the first time it is conducted it is seen to give rise to appreciable residual deformations, and during
the second test the structure must display an elastic behaviour.
The data obtained with this test showed that the bridge is now able to carry the loads associated with 2nd class
roads in the conditions of a perfectly elastic regime.
Proof testing consisted of performing a first loading stage according to a scheme reproducing the highest
bending moments; after that, the bridge was loaded according to a scheme reproducing maximum shear, with
bending moments only slightly lower than those applied at the previous stage.
Deflections at full load were seen to be ca one-fifth of those determined for the structure with a FEM calculation,
but, following the first removal of the load, the bridge displayed residual deflections corresponding to ca 25%
the deflections measured at full load: the non negligible residual deformation observed after the first loading
stage should be substantially ascribed to plastic adaptations in the restraint zones. In actual practice, the latter
work as sections in ordinary r.c., on account of their being subject to a negative moment, without the aid of
carbon fibres (not used at the extrados); furthermore, the restraint sections were loaded for the first time in the
life of the structure precisely on account of the substantial additions of material produced with the concrete
castings, especially at the abutments. Accordingly, the result can also be interpreted as an indirect confirmation
of the validity of the restraint system produced, since it shows that a plastic adaptation has occurred in the only
zone where this was possible, the restrained ends: it is known, in fact, that the zones where the carbon fibre is the
primary factor at work have a predominantly elastic behaviour, owing to the low ductility of FRP (carbon fibre)
materials.

Table 1: First loading cycle
(The deflections are corrected for the influence of temperature)

<table>
<thead>
<tr>
<th>Time</th>
<th>Loading stage</th>
<th>Sens. 1 mm</th>
<th>Sens. 2 mm</th>
<th>Sens. 3 mm</th>
<th>Sens. 4 mm</th>
<th>Sens. 5 mm</th>
<th>Sens. 6 mm</th>
<th>Sens. 7 mm</th>
<th>Sens. 8 mm</th>
<th>Sens. 9 mm</th>
<th>Sens. 10 mm</th>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
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<td>-0.02</td>
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<td>0.49</td>
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<td>-0.02</td>
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<td>-0.06</td>
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<td>0.49</td>
<td>-0.07</td>
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<td>-0.02</td>
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<td>0.03</td>
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<td>0.62</td>
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Table 2: Second loading cycle
(The deflections are corrected for the influence of temperature)

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<th>Defl. 2 mm</th>
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<th>Defl. 6 mm</th>
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</table>
Conversely, the second loading stage displayed a perfectly elastic behaviour: maximum deflections were slightly smaller than those obtained with the first application of the load (also on account of the lower bending moment), but the residual deformations, after the removal of the load, were of the order a few hundredths of the deflections at full load. Having exhausted the plastic adaptations of the initial stage, the structure now has a behaviour dominated by the carbon fibres. Accordingly, no surface cracks whatsoever were observed either in any of the ribs or in the sides of the bridge, perfectly finished recently (Fig. 9).

Figure 9. The bridge at completed works

CONCLUSIONS

With the intervention, this fine r.c. structure from the twentieth century has been fully rehabilitated and meets the requirements set forth in the present-day regulations on bridges subject to high loads. Of special relevance, in our opinion, is the fact that it proved possible to make the necessary adaptations without marring the slender and elegant original shape of the bridge.

REFERENCES


