STRENGTHENING OF MASONRY ARCH BRIDGES: RESEARCH AND APPLICATIONS

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Abstract

Masonry arch bridges represent an invaluable architectural and cultural heritage. Often this heritage is insufficiently maintained and subjected to increasing traffic loads, which leads to damage. This paper provides a survey of a dozen bridges in Northern Portugal, recent results in FRP bond and arch strengthening, and selected case studies of repair works in bridges in Portugal.

Keywords: Masonry arch bridges, survey, strengthening, FRP, case studies, NDT.

1.0   Introduction

Existing masonry arch bridges in Portugal were built from the Roman period to modern times. It is still possible to find Roman bridges, characterized by their flat pavements and semicircular arches of equal dimensions, as well as medieval bridges, with larger central spans, semicircular or pointed arches, cutwaters and humpback pavements. The works that bridges were unavoidably submitted to across time resulted in a difficult dating, leading sometimes to erroneous classifications. With time, the change of loads for which bridges were built, the decay of the materials and the lack of maintenance have led to states of damage, in many circumstances not compatible with their use or even their safety. The presence of damage, namely cracking, is not inevitably a sign of danger, since it may produce only a redistribution of stresses, not affecting safety. Nevertheless, when damage threatens safety of historical bridges, it becomes necessary to ensure their structural stability, by carrying out repair and strengthening measures, motivated by both the importance they still assume in the actual road network and the architectural, historical or social value they represent. Here, different activities carried out at University of Minho regarding arch bridges are given, namely:

[1] A survey carried out in twelve bridges from the Northern area of Portugal, including geometrical characterization and damage survey.

[2] A research project on the strengthening of masonry arches using FRP, including bond characterization, according to the bond length and curvature of substrate, and tests with strengthening at the extrados and intrados.

[3] Recent case studies carried out in Portugal, including repair measures proposed to restore safety and taking into account the principles of intervention in structures with heritage value.

2.0   Survey of old bridges over rivers Ave and Vizela (Portugal)

Roman bridges were planned and built in a context of a large centralized empire with a system of imperial roads, while medieval bridges are essentially regional and result from local political and economical circumstances. Therefore, the role of the bridge as a construction to connect different cities or regions is partially lost. In fact, the medieval bridge assumes a border role related to administration (tax collection) and safety (defence against invaders). Due to the limited economical and technical resources, the medieval structure shows usually a lower number of foundations located in the river-bed. In opposition to the Roman bridge concept of arches with equal dimensions, the medieval bridge exhibits larger flexibility, with one or several large span arches in the centre of the bridge and progressively smaller span arches towards the ends. Often, the bridge deck was not rigorously horizontally as in the Roman constructions due to the larger dimensions of the central arch. The building techniques and materials were also different. The know-how related to Roman pozzolanic "concrete" was partially lost and the medieval bridges adopt stone as the fundamental structural material with a non-structural, non-cohesive soil infill [1]. Without the pozzolanic mortar, medieval structures resist only marginally to shear actions. In the case of bridges, the shear action becomes particularly relevant due to the water pressure, being this effect aggravated by the floods. For this reason, it was usual to adopt lateral flood arches and cutwaters. Amazingly, the construction of masonry arch bridges survived until the 20th century, when this structural solution was gradually substituted by the rapid growth of reinforced concrete and steel structural solutions.

The present study focuses on the bridges over rivers Ave and Vizela, in the Minho region, belonging to the North of Portugal, see Fig. 1. The old roads have been analyzed and the bridges over rivers Ave and Vizela were identified. All bridges were surveyed taking into consideration the following items: (a) localization, (b) construction period, (c) description, (d) geometry, (e) state of conservation, (f) earlier interventions and (g) special remarks. A total of twelve bridges were surveyed, photographed in detail and drawn at an adequate scale, see Fig. 2 and Table 1. Additional information can be found in [2]. The most remarkable results of the investigation are: 50% present a humpback shape, 40% present cutwaters and 30% present small flood arches. With respect to the arching action: one flat arch bridge, seven semi-circular arch bridges, three segment arch bridges and one pointed arch bridge. The most relevant actions needed are: clean vegetation; remove pipes attached; remount parapets in original position; strengthen 15% of the bridges.
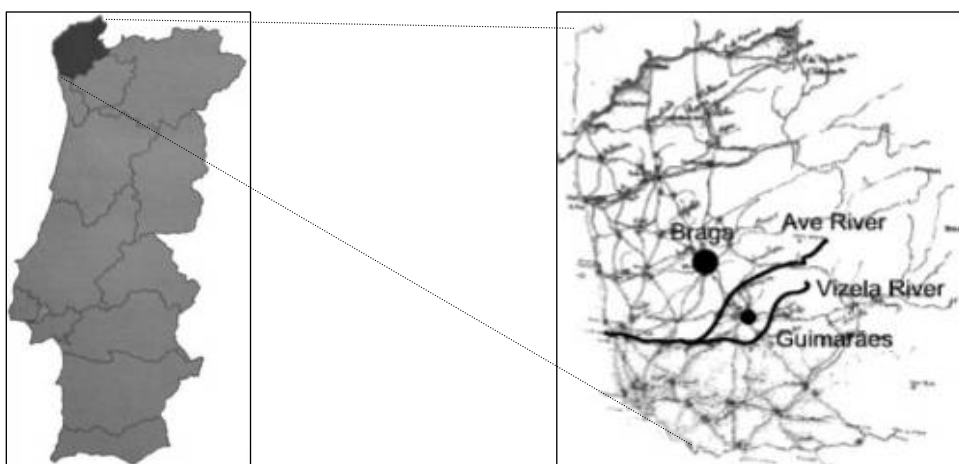

Fig. 1   Definition of the study area, with map of Portugal and Minho area, plus a zoom of old roads.

Table I.
Information collected about the bridges

Name	Arches	Span	Bridge Length	Road Width	Cut-water	Flood Arches	Deck	Conser-vation
S. João	4 semi-circular	4 × 7.5 m	43 m	3.5 m	1	–	Horizontal	Regular
S. Martinho do Campo	3 semi-circular	2 × 7.6 + 8.0 m	40 m	3.2 m	2	–	Horizontal	Bad
Taipas	34 flat	34 × 1.2 m	62 m	3.9 m	–	–	Horizontal	Regular
Donim	3 semi-circular	9.4 + 11.8 + 6.6 m	63 m	3.4 m	2	1	Horizontal	Bad
Caneiros	2 semi-circular	6.4 + 5.8 m	13 m	2.3 m	–	–	Humpback	Good
Selho	2 semi-circular	2.4 + 4.4 m	17 m	2.4 m	–	–	Humpback	Regular
Velha	3 semi-circular	3.3 + 10.8 + 8.6 m	38 m	3.2 m	2	1	Humpback	Good
Pombeiro	2 seg-mental	4.3 + 10.5 m	29 m	3.4 m	1	–	Humpback	Regular
Domingoterres	1 pointed	16.7 m	44 m	3.0 m	–	–	Humpback	Regular
Serves	3 semi-circular	4.7 + 10.6 + 9.7 m	54 m	5.1 m	–	1	Horizontal	Good
EN205	1 seg-mental	18 m	35 m	4.5 m	–	1	Horizontal	Good
Silvares	3 seg-mental	6.4 m + 5.8 m + 6.2 m	40 m	2.3 m	–	–	Horizontal	Good

(a) (b) (c)

(d) (e) (f) (g)
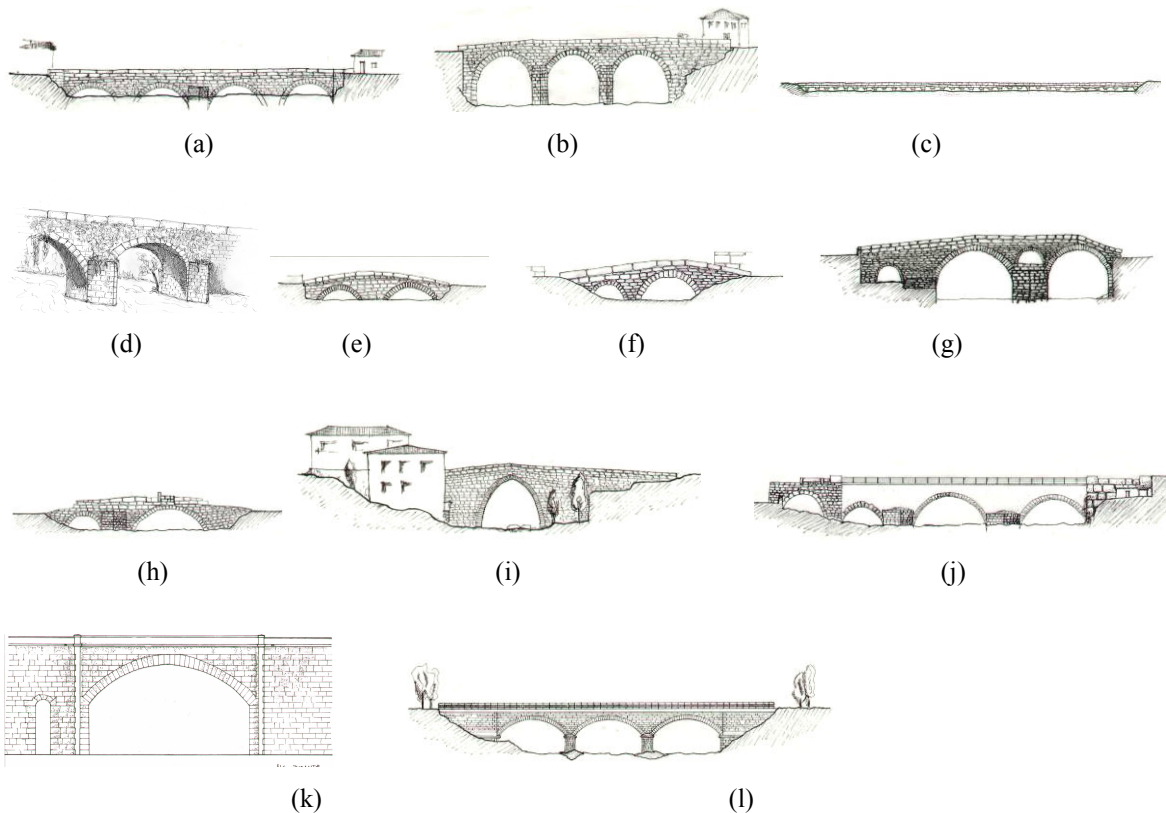
(h) (i) (j)

(k) (l)

Fig. 2.   View of the bridges: (a) S. João, (b) S.Martinho do Campo, (c) Taipas, (d) Donim, (e) Caneiros, (f) Selho, (g) Velha, (h) Pombeiro, (i) Domingoterres, (j) Serves, (l) EN205 and (l) Silvares.

3.0   Strengthening of arches

Among the materials used for strengthening, there has been an increasing interest devoted to the use of FRP (fibre-reinforced polymer) composites in the form of bonded surface reinforcements, which are being more and more used. FRP exhibits several advantages, as low specific weight, corrosion immunity, high tensile strength, adaptability to curved surfaces and ease of application, which makes it highly attractive and cost effective to be used in strengthening/repair works. Old and new researches in masonry, e.g. [3, 4], do not generally adopt low strength masonry materials. In the present testing program, handmade bricks and weak mortar were used in all specimens in order to replicate old masonry constructions. Initially, bond tests have been carried out using four clay bricks, see [5] for details. Fig. 3 illustrates the branch tree divisions of the experimental possibilities tested (5 specimens of each type). The following variations with respect to the standard specimen were considered: Masonry-FRP interface length; Anchor scheme used; FRP type of material; Masonry shape. The typical brittle stress-displacement diagram and the stress profile are shown in Fig. 4. For a lower stress level, shear stresses are mobilized in a relatively small interface length, close to its beginning. For higher stresses, shear stresses developed along the entire interface, but keeping a parabolic shape stress distribution. When strip detachment was accompanied by masonry material, optimal strength was taken from the substrate. Failure mode mechanism is strongly dependent upon the anchorage type. An interface length of 150 mm was enough to obtain maximum anchoring force. For the concave geometry variation, the results presented must be taken as indicative only.
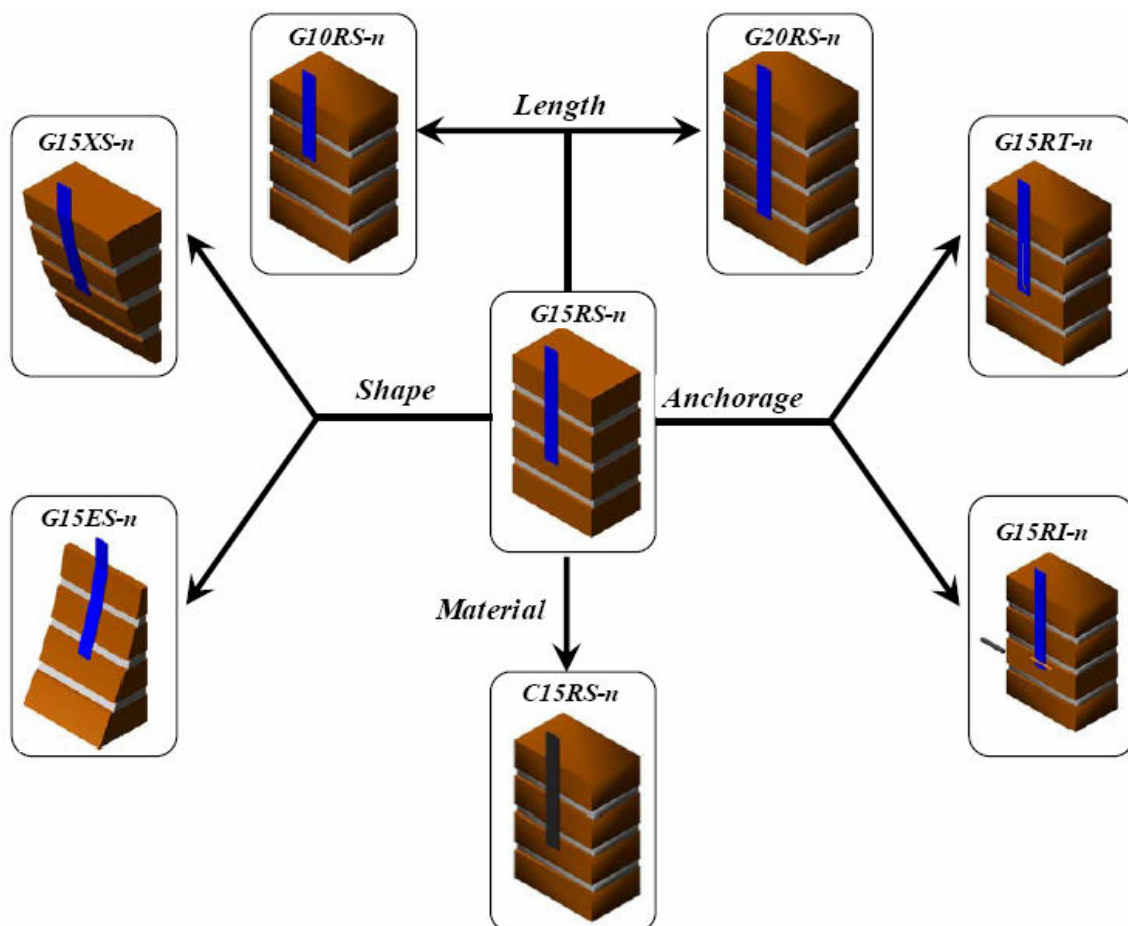

Fig. 3   Different prototypes considered for the bond test.

The absence of the force component perpendicular to the masonry surface, responsible for the strip detachment, might reduce the figures obtained. Note that, in practice, the shape of the masonry surface can be hardly considered as plain.
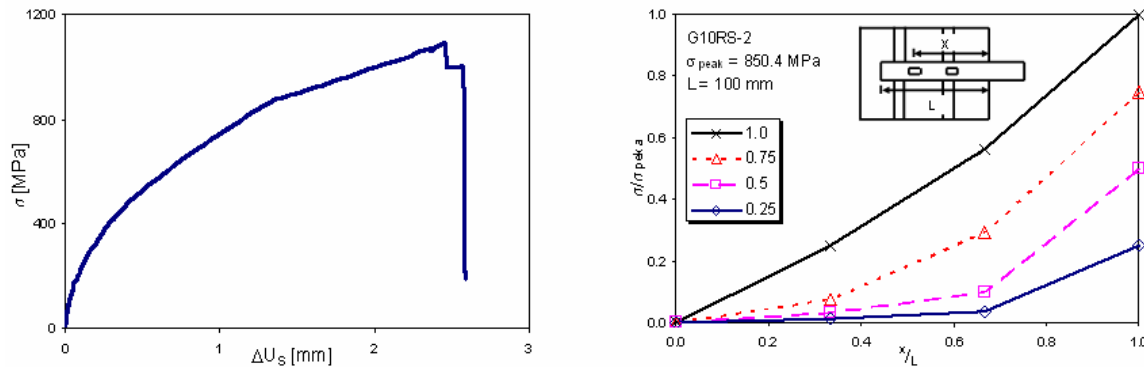

Fig. 4   Typical responses in terms of shear stress-relative displacement diagram and stress distribution along interface length upon increasing loading.

The experimental program includes also the testing of eight scaled semi-circular brick masonry arches, plain and strengthened with GFRP strips, aiming at: Characterization of the structural behaviour of both unstrengthened and strengthened masonry arches loaded monotonically until failure; Assessment of the influence of the reinforcement on the mechanical behaviour and failure mechanism; Creation of a reliable database on the experimental behaviour of masonry arches, able to be used in the calibration of both analytical and numerical tools. Each semi-circular single-ring arch was composed of 59 brick courses and had a 750 mm radius, 500 mm width and 50 mm thickness (thickness/span $\approx$ 1/30). The mortar joints were kept with a constant intrados thickness of approximately 10 mm. The first set of specimens was composed by two unstrengthened arches. Since both arches did not fall down at the end of the test, due to stability provided by the arch self-weight, it was decided to use a localized strengthening arrangement composed of two GFRP strips of 80 mm width each, placed over the hinges at either the intrados or the extrados, and test them again, see Fig. 5a. In addition, four undamaged arches were strengthened with two continuous GFRP strips of 50 mm width each. Two arches were strengthened at the intrados, and the other two were strengthened at the extrados, see Fig. 5a. All specimens were tested for a monotonic load applied at the quarter span, until the formation of the correspondent failure mechanism was achieved.

The unstrengthened specimens exhibited a structural behaviour characterized by the formation of the typical four-hinge mechanism, which occurred for small displacements and just after reaching the maximum load. On the other hand, all the adopted strengthening arrangements caused an increase in terms of load capacity. However, new dominant failure modes were observed, namely detachment of the fibres from the arch surface and sliding along a mortar joint, see Fig. 5b. The debonding phenomenon only affected the arches where the GFRP strips were placed at intrados, whereas for specimens strengthened at extrados failure occurred due to slipping of one part with respect to the other along a mortar joint. Another important feature of the continuously strengthened specimens is the large deformation capacity exhibited prior to failure, which provides the arches with important ductility behaviour. The experimental results show that the adopted continuous GFRP strengthening arrangements provided an enhanced arch behaviour, with respect to the unstrengthened specimens, both in terms of load capacity and in terms of ductility, see Fig. 5c. While specimens strengthened in the intrados got the maximum load increase, specimens strengthened in the extrados exhibited the larger ductility.
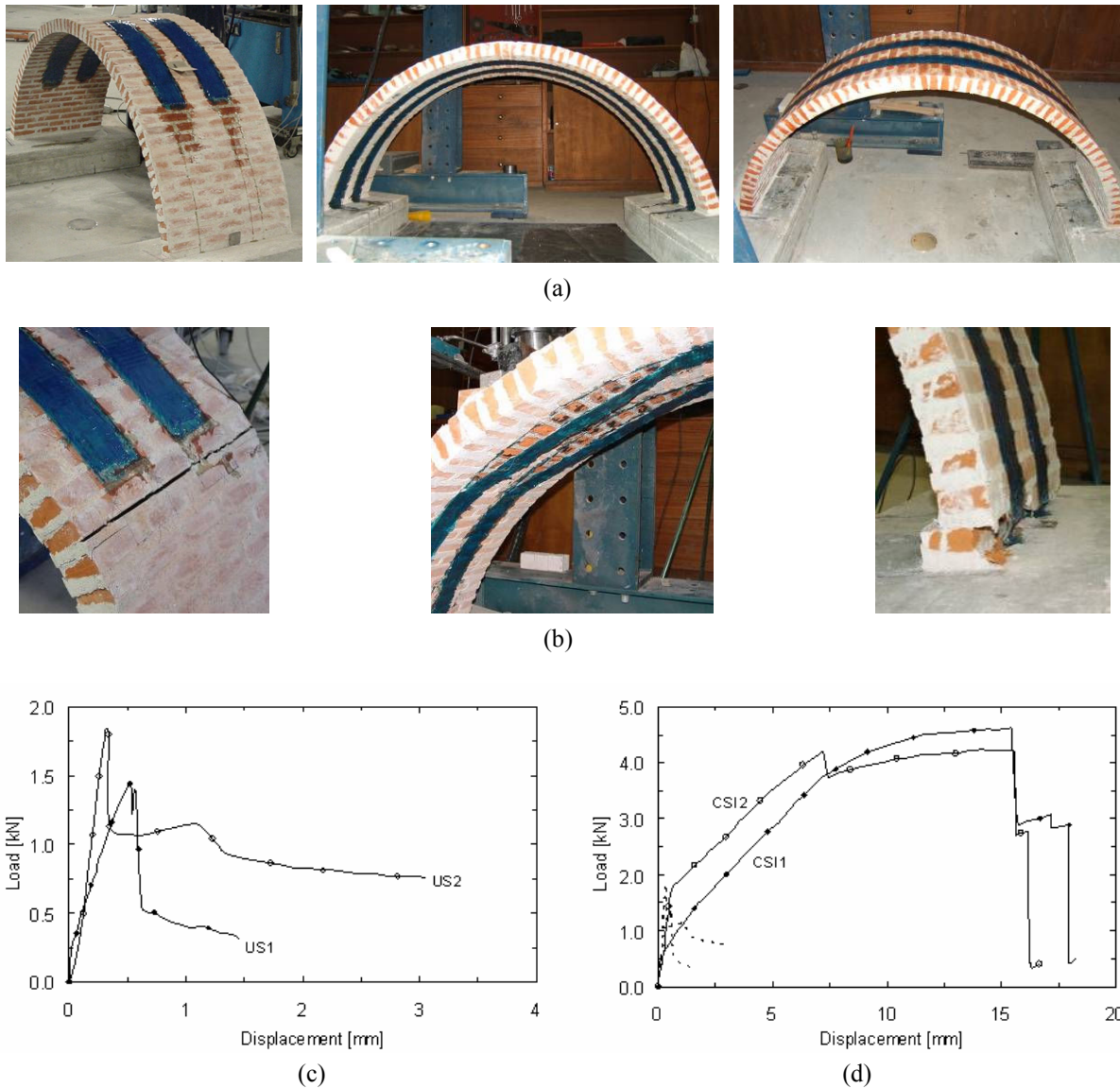
(a)

(b)

(c)

(d)

Fig. 5   FRP strengthened arches: (a) examples of location reinforcement; (b) examples of failure modes; load-displacement diagrams for (c) unreinforced arches and (d) arches strengthened in the

4.0   Case Study

4.1   Negrelos Bridge

The multi-span Negrelos Bridge is located close to Guimarães over the Vizela river. With time, the bridge has lost its regional importance, though still assuming a great significance to the local network. The bridge has a flat roadway, supported by three semi-circular granite stone masonry arches, with different free spans (8.0 m + 6.4 m + 8.0 m), as schematically represented in Fig. 6. The bridge reaches a total length of approximately 30 m and has a roadway width of about 3.0 m. The central arch is supported by two massive piers, endowed with two triangular cutwaters at upstream and two rectangular cutwaters at downstream. Within a governmental program to clear the river from pollution, a drainage pipe was placed on the left shore, on top of an embankment made beneath arch A1 and close to the left abutment, see Fig. 6 (the pipe is not visible). Both the spandrel walls and

the parapets were built with stone masonry, but successive repair works carried out over the years have changed some original characteristics as it can be noticed by the parapet wall partially rebuilt with concrete blocks.
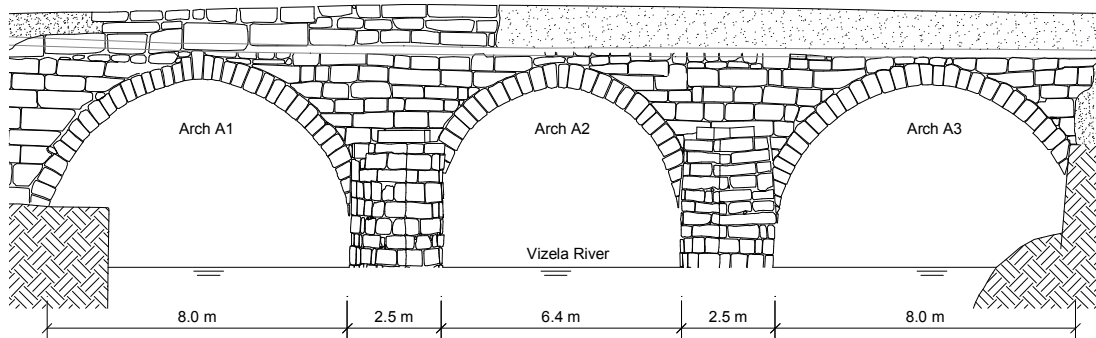

Fig. 6   Negrelos Bridge (upstream view).

The survey carried out has showed that the bridge presented a pronounced damage state, where damage was mostly characterized by, see Fig. 7a: extensive longitudinal cracking exhibited by the central arch (A2), close to the downstream spandrel wall, clearly visible at the intrados. This is mainly caused by earth pressure in the spandrel walls; lateral movements of the spandrel walls near the left abutment, which became out of plumb, most likely originated by lack of maintenance in conjunction with increasingly heavy loads; generalized damage caused by vegetation, spread all over the bridge; extensive cracking in the left downstream cutwater and minor cracking in the other three cutwaters, mainly due to existing vegetation and the lack of adequate stone interlocking. Also, some stone blocks were cracked. Most probably, some of the cutwaters were built or extended after the construction of the bridge.

Fig. 7   Examples of damage: longitudinal cracking in the central arch and vegetation and cracks in the downstream cutwaters (downstream view).

The bridge was modelled as an in-plane three-span semicircular arch bridge. The necessary geometrical data was obtained from topographic surveying and visual inspection. In the absence of in-situ test results, the material properties were considered to assume typical values found in similar structures, see [6] for details. Besides the self-weight of the materials (masonry and fill), a moving load was also considered, composed by three axles equally spaced by 1.5 m and with a 200 kN load per axle. Using a computer program developed within the rigid block limit analysis method [7], the minimum failure load factor is equal to 1.67. Fig. 8 illustrates the associated four hinges failure mechanism found, where both the dead and live load pressures applied to the arch, the hinges and the thrust-line are showed. Assuming that the vehicle crosses the bridge from left to right, the minimum

failure load factor was found for the vehicle central axle positioned at 31.9 % of the left arch free span (arch A1). Since symmetrical geometry and vehicle are used, the same result is obtained considering that the vehicle crosses the bridge from right to left instead.
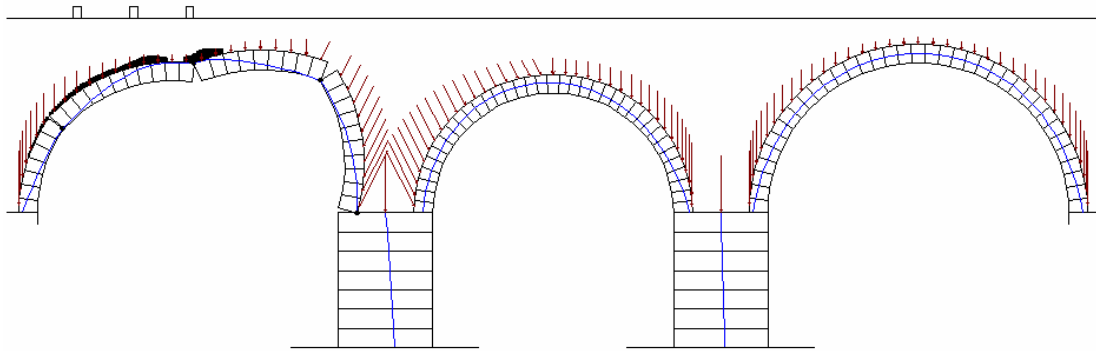

Fig. 8   Minimum load failure mechanism of Negrelos Bridge.

The visual inspection carried out showed that a set of repair actions were necessary, namely to stop the progression of the longitudinal cracking along the central arch, to counteract the outward movement of the spandrel walls, to prevent the failure of the cutwaters and to clean all vegetation from masonry. Therefore, a set of four horizontal stainless steel anchors across the full bridge width, endowed with cylindrical steel anchorage plates at each side of the arch, were proposed, see Fig. 9. Two short stainless steel anchors were used close to each springing. In addition, it was recommended a light injection of the arch, at the intrados.
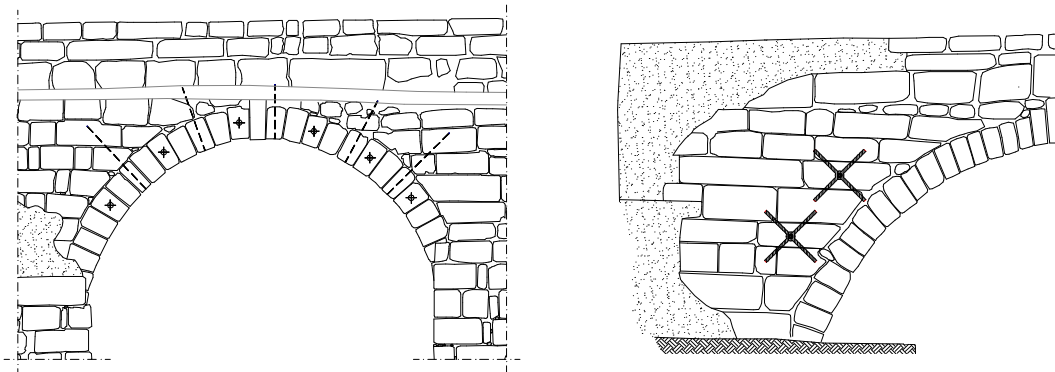

Fig. 9   Examples of adopted strengthening for central arch and spandrel wall.

4.2   Donim Bridge

This structure presents a pronounced damage state, with extensive longitudinal cracking in the left arch and flood arch, clearly visible at the intrados, see Fig. 10. The right pier is very damage, where some stone blocks are cracked and a foundation stone is missing. The vegetation, spread all over the bridge, caused severe damage to the right cutwater. The spandrel walls were subjected to lateral movement and are clearly out of plumb. Moreover, boroscopic inspection indicated that the infill is full with voids. In order to discuss the behaviour of the bridge and adopt an adequate model for the analysis, two different structural analysis models are shown, assuming that the truck is placed over the central, larger span arch. The models include the following features:

Structural fill and lateral soil pressure in the spandrel walls, and non-structural fill and lateral soil pressure in the spandrel walls. It is obvious that the lateral soil pressure could be obtained implicitly by considering inelastic soil behaviour but this would result in a significant number of additional parameters for the analyses. For this reason, the simplified lateral soil pressure coefficient was adopted. The results of a non-linear analysis for a unitary load factor are illustrated in Fig 10, see also [8]. Here, the maximum principal strains are a measure of cracking and can be assumed to indicate the expected damage in the bridge. The importance of the infill can be immediately observed from the deformed shapes and the predicted damage in the structure. If the lateral pressure from the infill is not considered as an external load, the spandrel walls move partly inwards, but, if the lateral pressure is considered, the spandrel walls move outwards. Damage in the structure is also localized in different areas. For the first model, damage occurs in the spandrel walls and, longitudinally, in the arches. For the second model, extensive damage occurs in the spandrel walls due to out-of-plane bending.
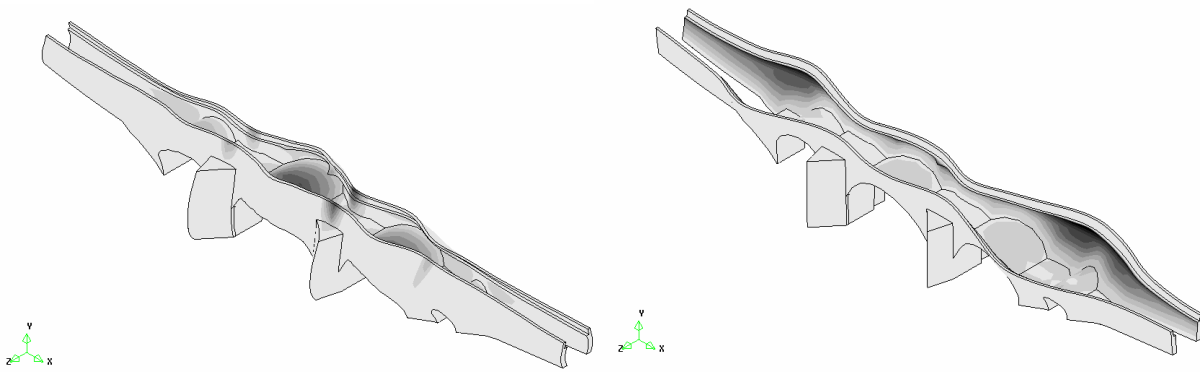

Fig.10   Examples of damage in Donim bridge, using a non-linear finite element analysis (arch splitting or spandrel collapse).

4.3   Boutaca Bridge

The Boutaca bridge has a flat roadway, supported by six pointed masonry arches equally spanned, as illustrated in Fig. 11. Lateral masonry walls, supported by a buttressing system, make the connection between the pointed arches and a semicircular masonry tunnel, belonging to the old railway network. The bridge reaches a total length of 60.0 m and has a roadway width of 7.0 m. At the ends, it is noticeable the existence of four box-houses used by road-menders. Diagnosis was carried out with the assistance of GPR, see Fig. 11. To prevent the outward movement of the masonry walls and to lighten the load in the buttresses, it was proposed the execution of a set of horizontal anchors across the full width of the road, endowed with plates, which have been designed to be aesthetically attractive. Four vertically aligned tie rods between each two buttresses were adopted. In order to reduce the lateral pressure on the walls and the appearance of humidity at the intrados of arches it was proposed to waterproof the pavement and the reconstruction of all the drainage system. Also, the cleaning and removal of infesting vegetation in the bridge should be periodically carried out. The missing plaster and painting should be replaced, using materials similar to those used in the construction.
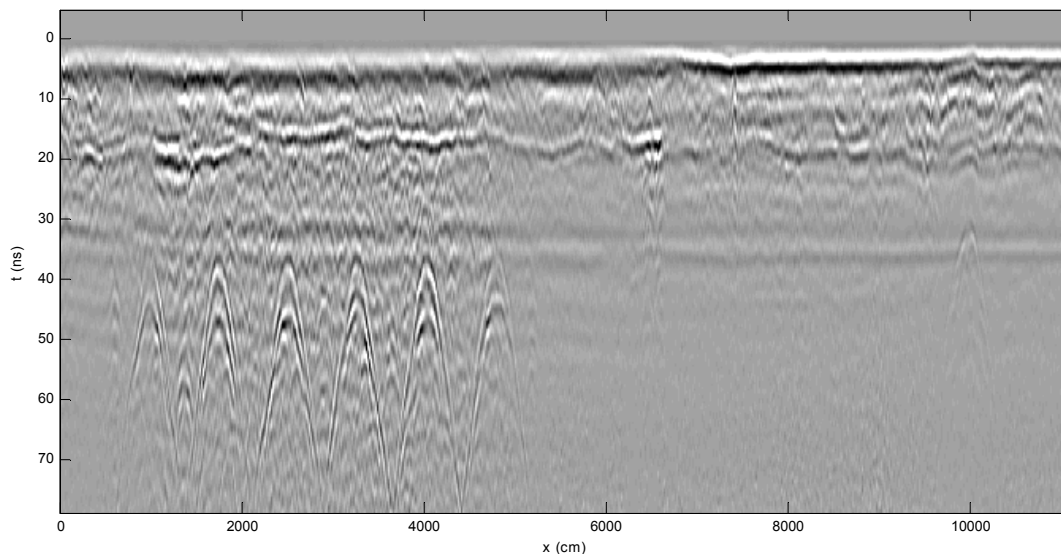
Fig. 11   Boutaca Bridge: upstream view and detection of arch thickness and infill characteristics using GPR inspection.

Acknowledgement

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