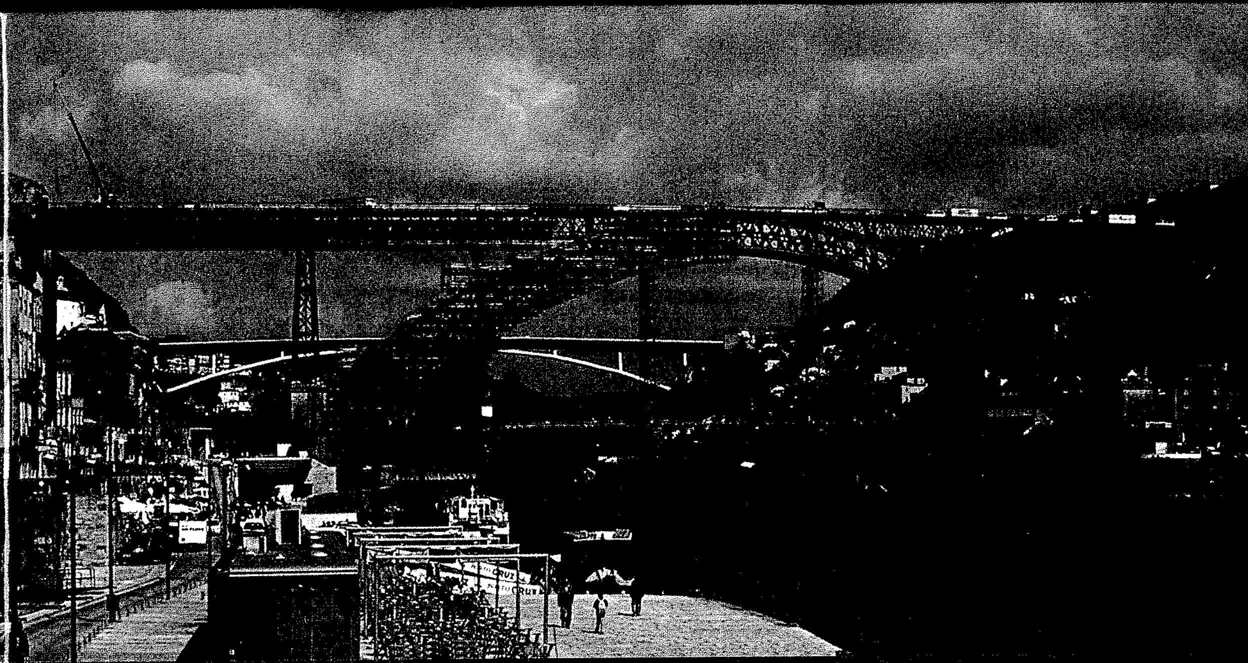

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost

Editors

Paulo J.S. Cruz, Dan M. Frangopol & Luis C. Neves

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FRP strengthening of masonry arches towards an enhanced behaviour

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ABSTRACT: This paper deals with the experimental behaviour of brick masonry arches strengthened with glass composite materials (GFRP). Eight 1:2 scale models of 1.5 m span arches were tested under a monotonic vertical load applied at the quarter span. The FRP strengthening was applied either at the extrados or at the intrados of the specimens. The experimental results presented in this paper show that the adopted GFRP strengthening provides an enhanced arch behaviour, with respect to the unstrengthened specimens. The ultimate strength was considerably increased and also a noticeable improvement in ductility was possible. The collapse mechanism of the strengthened arches was no longer related to the formation of a classical four-hinge mechanism, but it was characterized by the occurrence of new failure modes at some critical sections instead.

1 INTRODUCTION

As part of the widespread European cultural heritage, historical masonry constructions deserve particular attention. In particular, masonry arches are often subject to rise of loads, movements in the abutments or ageing effects, which can originate important structural damage. Therefore, an efficient strengthening/repair measure would be able to re-establish the performance of these structures, preventing its brittle collapse or even increasing its load capacity. Being most of the historical masonry constructions of considerable architectural and cultural historical significance, their study and preservation constitute current issues in scientific research.

Among the materials used to repair or upgrade civil engineering structures, there has been an increasing interest devoted to the use of FRP (fiber-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. However, FRP is a brittle material and its behaviour has to be further investigated, particularly some aspects related to its long term durability.

Following the initial researches concerning the use of FRP in masonry structures (Schwegler, 1994; Triantafillou, 1998; Kolsh, 1998), numerous experimental works were carried out showing that this technique is effectively valid as an option to strengthen or repair masonry structures, in particular arched ones, see Valluzzi et al. (2001), Lissel & Gayevoy (2003) and Foraboshi (2004) for further details. On the other hand, available experimental results show that the strengthening of masonry arches with glass fibers, which exhibit lower mechanical properties than carbon ones, allow a better control of the collapse mechanisms and provide higher strength and better global ductility characteristics (Valluzzi et al., 2001).

2 BEHAVIOUR AND FAILURE MECHANISMS OF MASONRY ARCHES

Assuming that masonry has zero tensile strength, which can be justified by its relatively low or even zero tensile strength, an arched masonry structure is kept in compression as long as the thrust line (or pressure line), which represents the eccentricity of the compressive force at every cross-section, is kept inside the central core. When the thrust line moves outside the central core, at a given cross-section, the formation and consequent opening of a crack takes place. In this way, safety is maintained as long as the thrust line is kept inside the thickness of the arch. Naturally, the crack development leads to the formation of a plastic hinge at the compressed edge of the arch. However, in most cases masonry crushing is not likely to occur. Then, the formation of successive hinges leads to the formation of a mechanism that causes the arch failure. This means that unstrengthened masonry arches fail essentially by the occurrence of plastic hinges enough to form a mechanism (Heyman, 1982). Figure 1 represents the classical four-hinge mechanism of a masonry arch submitted to an asymmetrical loading.

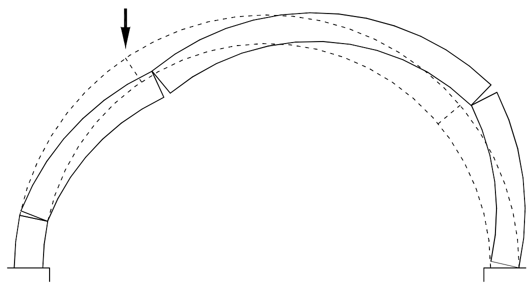


Figure 1. Four-hinge failure mechanism of a semi-circular masonry arch submitted to an asymmetrical loading.

As expected, the presence of a bonded FRP strengthening changes completely the structural behaviour of a masonry arch. The fibers, which possess a high tensile strength, prevent the afore-mentioned hinge formation and may change significantly the failure mechanism. Since the use of FRP strips provides bending moment resistance, the thrust line may now safely move outside the thickness of the arch.

For the arch illustrated in Figure 1 and considering the reinforcement located either at the extrados or at the intrados of the arch, the formation of a fourth hinge mechanism is prevented. Therefore, only three hinges are able to rise, transforming the arch into an isostatic structure. This means that new failure mechanisms different from the one afore-mentioned have to be considered. Due to the FRP high tensile strength, the compressive stress in masonry may now assume higher values so failure of the arch caused by masonry crushing has to be taken into account. The presence of the reinforcement also allows the development of higher shear stresses in masonry and, therefore, shear failure due to sliding along a mortar joint may occur. Moreover, in addition to the usual stresses parallel to the fibers, the curved shape of arches originates stresses with a component normal to the fibers, which may lead to the detachment of the reinforcement from masonry, namely in arches strengthened at the intrados. Consequently, the following failure mechanisms are usually added to the afore-mentioned one:

- Failure due to masonry crushing;
- Failure due to detachment of the fibers;
- Failure due to sliding along a masonry joint.

Sliding between the fibers and its support is usually neglected since shear stresses at the FRP-masonry interface are of minor magnitude (Valluzzi et al., 2001). Also FRP tensile failure is not likely to occur due to its high tensile strength.

It is known that, for a given arch shape, the type of failure to be obtained depends both on the mechanical properties of the materials (brick, mortar and FRP) and on the quantity and location of the reinforcement. In order to evaluate the behaviour of brick masonry arches strengthened with FRP, a combined experimental-numerical research project was started at Universidade do Minho, see also Lourenço & Martins (2001). This paper presents the first experimental results concerning the behaviour of brick masonry arches strengthened with glass composite materials (GFRP) and tested under a monotonic loading scheme.

3 EXPERIMENTAL STUDY

The experimental program carried out consisted partially in the testing of twelve scaled semicircular brick masonry arches, plain and strengthened with GFRP strips. This experimental program was designed to attain the following main objectives:

- 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.

All arch specimens were constructed at scale 1:2 in order to optimize expenses related to raw materials and workmanship as well as to achieve a quicker construction process and a feasible testing setup. In order to replicate old masonry constructions, handmade bricks and a suitable mortar were selected. For that purpose, 100×50×25 mm^3 clay bricks were especially made, reaching an average compressive strength of about 6.3 N/mm^2, whereas a pre-mixed hydraulic lime based mortar was adopted for the joints. 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 ≈ 1/30), see Figure 2. The mortar joints were kept with a constant intrados thickness of approximately 10 mm.

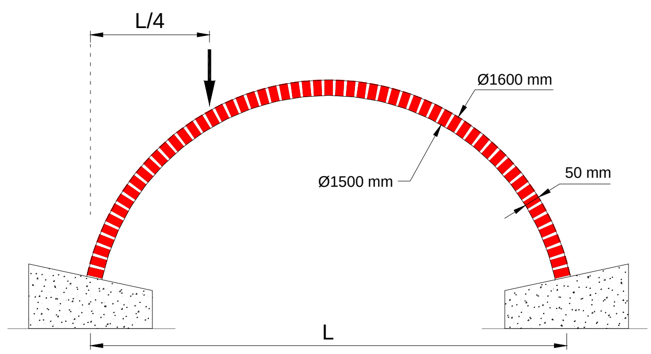


Figure 2. Adopted arch geometry and load scheme.

Two concrete blocks fixed to the laboratory rigid floor were used as supports, whereas the arches were constructed over a rigid wooden mould, as represented in Figure 3a, b. One week after the construction, the mould was removed, see Figure 3c, and the GFRP strips (with an average tensile strength of approximately 1470 N/mm^2), if any, were applied on the arch surface. The application of the strips was carried out using the typical multi-layer system (formed by epoxy primer, epoxy resin and GFRP strips) either at the extrados or at the intrados of the arches. The definition of the GFRP strip width to be used was based on a previous numerical analysis (Basílio et al., 2004). All tests took place two weeks after the construction of the arches.

(a) (b) (c)

Figure 3. Different phases involved in the construction of the masonry arches.

The first set of specimens was composed by two unstrengthened arches (US1 and US2). However, since both arches did not fall down at the end of the respective 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 (specimens LS1 and LS2), see Figure 4a. In addition, four undamaged arches were strengthened with two continuous GFRP strips of 50 mm width each. Two arches were strengthened at the intrados (CSI1 and CSI2), see Figure 4b, and the other two were strengthened at the extrados (CSE1 and CSE2), see Figure 4c. In total eight tests are described in the paper.

All specimens were tested for a monotonic load applied at the quarter span, as illustrated in Figure 2, until the formation of the correspondent failure mechanism was achieved. The experiments were performed under displacement control using the vertical displacement underneath the load line as the test control parameter. Negligible horizontal displacements were recorded at the springers.

(a)

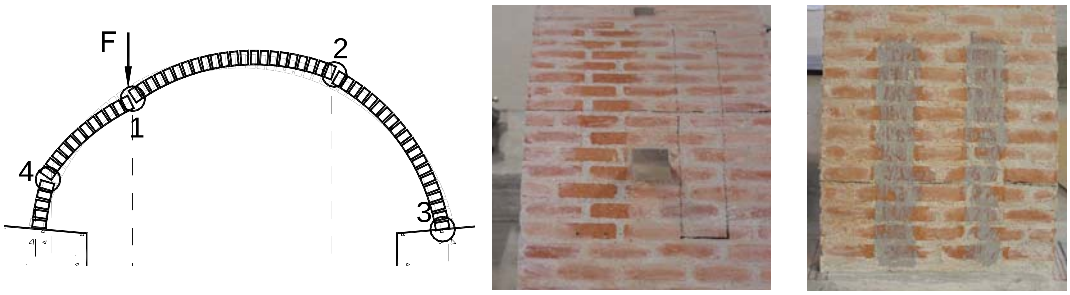
(b)

(c)

Figure 4. Strengthening arrangements adopted: (a) localized strengthening; (b) continuous intrados strengthening; (c) continuous extrados strengthening.

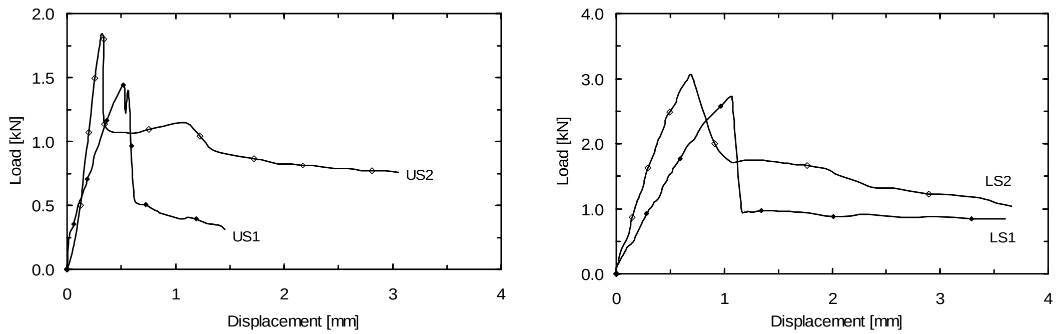
4 TEST RESULTS

Both unstrengthened arches US1 and US2 presented a similar structural behaviour, essentially characterized by the formation of the classical four-hinge mechanism, see Figure 5 where two of the hinges are clearly visible. Despite the observed resemblance, slight differences were observed, namely in terms of pre-peak stiffness and peak load achieved, see Figure 6a. An important feature is the low ductility exhibited by both specimens. Failure occurred suddenly, for small displacements and just after the maximum load has been reached.



(a) (b) (c)

Figure 5. Unstrengthened arches: (a) typical four-hinge failure mechanism developed; (b) extrados view of hinge 2; (c) extrados view of hinge 4.

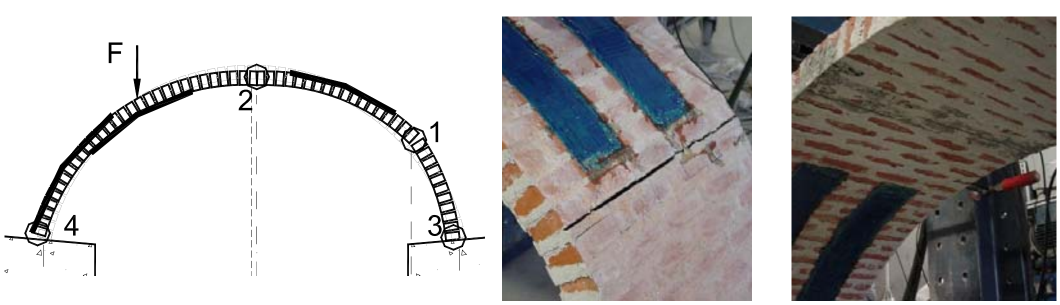


(a) (b)

Figure 6. Vertical load-displacement diagrams measured at the load point for the plain arches: (a) before strengthening (arches US1 and US2); after localized strengthening (arches LS1 and LS2).

As afore-mentioned, since arches US1 and US2 did not fall down at failure, GFRP strips were applied locally over hinges 1, 2 and 4 (see Figure 5a). These new specimens are here denoted as arches LS1 and LS2, respectively.

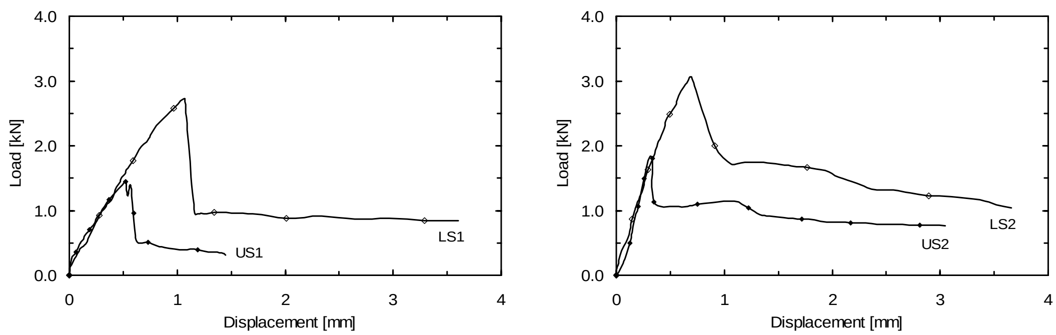
The use of a strengthening strategy aiming at repair locally the damaged hinges did not avoid the formation of a four-hinge mechanism, see Figure 7a. In fact, the GFRP strips used were able to prevent the re-opening of the existent cracks but new hinges appeared beyond the strip length instead, as shown in Figure 7b, c. The formation of new hinges far from their typical locations, forced by the bonded GFRP strips, allowed an increase of the peak load in both arches. The new load-displacement diagrams obtained after strengthening are shown in Figure 6b. The average increase is in the order of 76%.



(a) (b) (c)

Figure 7. Localized strengthening: (a) failure mechanism developed; (b) extrados view of hinge 1; (c) intrados view of hinge 2.

For a better comparison, the results exhibited in Figure 6 were rearranged in order to gather the structural response by specimen, see Figure 8. As it can be observed, the reinforcement did change neither the pre-peak stiffness nor the previous fragile behaviour of the unstrengthened specimens. However, besides the load capacity increase also a slightly larger post-peak branch was possible to attain.

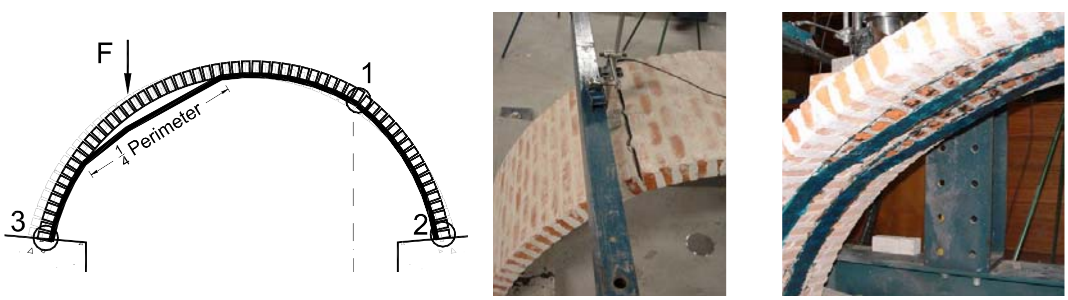


(a) (b)

Figure 8. Vertical load-displacement diagrams measured at the load point: (a) arch US1 before and after strengthening (LS1); (b) arch US2 before and after strengthening (LS2).

For the arches strengthened with continuous strips, a different collapse mechanism was expected since the presence of the fibers along the extrados or intrados prevents the fourth plastic hinge from occur.

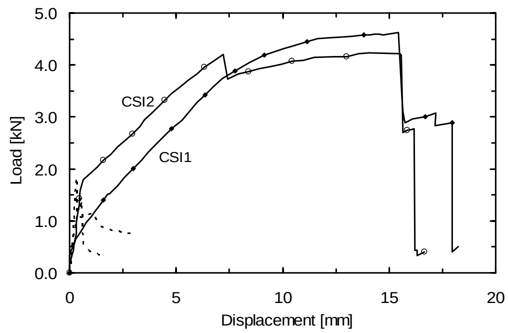
For the continuous strengthened specimens at intrados (CSI1 and CSI2), the mechanism observed is illustrated in Figure 9a. Two of the hinges were formed at the supports and the third one appeared on the less-load half of the arch, see Figure 9b. The fibers were able to maintain equilibrium until total collapse of the specimens.



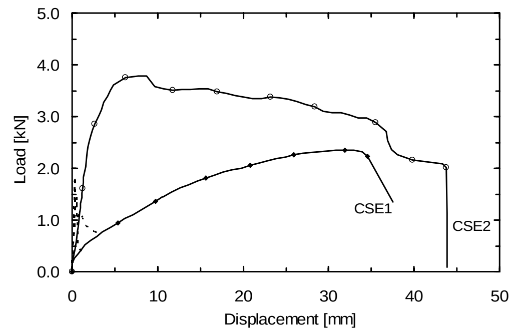
(a) (b) (c)

Figure 9. Continuous intrados strengthening: (a) failure mechanism developed; (b) extrados view of hinge 1; (c) detachment of both GFRP strips at the end of the test.

In terms of global load-displacement response, noticeable increases both in terms of load capacity were possible as shown by the responses depicted in Figure 10a. On average terms, the load capacity was increased in 170% and the maximum load was achieved for a displacement of about 35 times greater than the one corresponding to the unstrengthened specimens. On the other hand, the GFRP strengthening did not increase the initial stiffness of the arches. Despite the occurrence of the first hinge for different load values, both specimens presented a quite similar behaviour. The abrupt drops in load observed in Figure 10a are due to the detachment of the GFRP strips. This means that failure, which occurred for high deformations, was dictated by the successive detachment of the two reinforcement strips, caused by the ripping of a thin layer of brick, see Figure 9c. This phenomenon is due to the higher tensile strength of the epoxy resin when compared to the brick one, as corroborated by pull-off tests carried out on masonry prisms strengthened with GFRP (Basílio et al., 2005a).



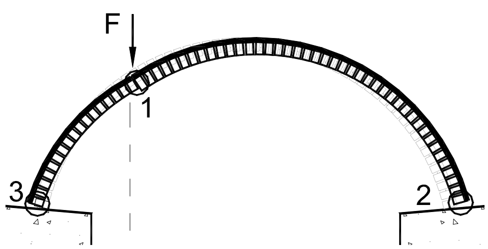
(a)



(b)

Figure 10. Vertical load-displacement diagrams measured at the load point: (a) arches CSI1 and CSI2 (intrados strengthening); (b) arches CSE1 and CSE2 (extrados strengthening). In both diagrams the responses of the unstrengthened specimens are represented by dashed lines for a better comparison.

For the continuous strengthened specimens at extrados (CSE1 and CSE2), the mechanism developed during testing is illustrated in Figure 11a. In these experiments, the first hinge was formed underneath the load point, see Figure 11b, whereas the other two hinges appeared at the supports.



(a)

(b)

(c)

Figure 11. Continuous extrados strengthening: (a) failure mechanism developed; (b) intrados view of hinge 1; (c) sliding along a mortar joint close to the right support.

The global load-displacement curves are included in Figure 10b. The different pre-peak stiffness is likely to be related with previous damage caused to specimen CSE1 during its curing or mould removal, as it becomes perceptible since a very low load. This feature causes also an important decrease in the maximum load achieved. Therefore, it seems reasonable not to consider the CSE1 response as typical of this kind of structures. This assumption is further validated by another set of experimental tests performed on arches strengthened at extrados and reported elsewhere (Basilio et al, 2005b).

Figure 10b shows that an important load increase was achieved (about 130% on average terms if the specimen CSE1 is not considered) comparatively with the unstrengthened specimens, however not as high as the increase enabled by specimens CSI. Also in this set, the reinforcement did not increase the pre-peak stiffness, whereas the maximum load capacity was achieved for a displacement approximately 20 times greater than the one corresponding to specimens US. A very important feature is the long post-peak branch recorded, which provides the structure with important ductility behaviour. In fact, the displacement measured at collapse doubles the corresponding one measured in specimens CSI. For specimens CSE failure was characterized by the slipping of one part of the arch with respect to the other along a mortar joint located close to the right springing, see Figure 11c, and was due to insufficient shear resistance.

In order to provide a general overview about the various strengthening arrangements, Table 1 summarizes the quantitative data regarding the load capacity of each specimen and the strength increase achieved by the application of the GFRP reinforcement.

As mention before, all strengthening arrangements adopted in this study were able to make available a load capacity increase. However, while the intrados strengthening allows for the maximum load increase, the extrados strengthening provides the most interesting solution in terms of ductility.

Table 1. Experimental results concerning the maximum load achieved and load increase provided by the GFRP strengthening.

Strengthening arrangement	Specimen	Maximum load [kN]	Average value [kN]	Load increase
Unstrengthened	US1	1.44	1.64	----
	US2	1.84		----
Localized strengthening	LS1	2.72	2.89	+89%
	LS2	3.06		+66%
Continuous strengthening (intrados)	CSI1	4.62	4.43	+170%
	CSI2	4.24		
Continuous strengthening (extrados)	CSE1	2.35	3.78(*)	+130%(*)
	CSE2	3.78		

(*) The result concerning specimen CSE1 was not considered.

5 MAIN CONCLUSIONS

The experimental behaviour of brick masonry arches, plain and strengthened with GFRP strips under different arrangements, has been presented and discussed in the paper. 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 fibers from the arch surface and sliding along a mortar joint. 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. While specimens CSI got the maximum load increase, specimens CSE presented the most interesting solution in terms of ductility.

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