Systems of arches and columns strengthened with FRP at the extrados

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ABSTRACT: The topical problem of the maintenance and upgrading of the masonry building heritage has led, in these last years, to an increasing interest in new and suitable material and technologies. As a consequence the application of composite material, also on masonry structures, is nowadays considered an assessed and quite usual practice. Nevertheless the adoption of new techniques requires a deep knowledge of the behavior of the reinforced structure and a definition of adequate models able to simulate it. Aim of this paper is the evaluation of the structural behavior of a system of arch and columns, strengthened with FRP at the extrados and loaded by a vertical force at the crown and by two horizontal forces at the abutments of the arch. The scope is pursued by means of a proposed methodology, which, with some simplifying assumptions, allows defining the failure interaction locus of the strengthened structure. The effectiveness of the strengthening intervention is evaluated on the basis of these strength domains defined in the plane of loads.

1 INTRODUCTION

The preservation of the existing masonry structures, particularly in seismic areas is nowadays considered a topic of great interest. In this framework, in the last years, the use of innovative systems to retrofit and/or reinforce structures has been widely developed in the field of civil engineering. Specifically the use of composite sheets (FRP) applied at masonry structures surfaces represents a very attractive solution in the field of research as well as to practical and professional applications. The advantages provided by this material are well known (light material, ease of installation, no corrosion problems, a high specific strength) and the reversibility of the intervention is also a very positive aspect.

Anyway even if applications on real structures are spreading, many uncertainties still remain on the effective global response of the reinforced structure, and particularly on the models to be adopted for the design of the reinforcement, and then for the behavior simulation.

In this work a reference masonry structure made by two piers and an arch, subjected to vertical and horizontal forces, is analyzed. The choice of this peculiar scheme is motivated by its frequent presence in many masonry structures that characterize European historical constructions.

In the following the case of the complete reinforcement of the surface at the extrados of the masonry frame will be considered. This arrangement of the reinforcement prevents the opening of "classical" hinges at the un-reinforced side of the structure, similarly to the case of the reinforcement of the intrados (Ianniruberto & Rinaldi, 2004) and in the case of arches loaded by vertical forces (Como et al., 2001). Then, no kinematically admissible mechanisms can occur in the structure and the classical limit analysis theorems (Kooharian, A. 1953, Heyman, 1966 and 1982), cannot be applied for the evaluation of the collapse behavior of the reinforced construction. The ultimate strength of the structure is governed by local failures due to masonry crushing, masonry slip at a mortar bed or FRP debonding.

In a previous paper the authors analyzed the collapse response of a system of arch and column reinforced at the intrados only. It was highlighted the enhancement of the structural capacity due to the application of FRP sheets, particularly for horizontal actions, and the influence of the debonding phenomenon of the composite material.

Aim of this paper is to show an analytical model for the evaluation of the bearing capacity of the reference masonry frame reinforced at the extrados taking into account failure criteria at both local and global level. Moreover the ultimate domains of the reinforced
structure subjected to combined vertical and horizontal loading conditions, are defined and discussed.

2  COLLABORATION ANALYSIS OF THE UNSTRENGTHENED MASONRY FRAME

The influence of the FRP reinforcement applied at the extrados of a system of piers and arch will be evaluated with reference to the masonry frame, shown in Figure 1, loaded by combined vertical and horizontal concentrated forces. The frame has a square cross section whose side is equal to 0.42 m.

In order to judge the effectiveness of the intervention, the collapse multiplier of the masonry frame without any reinforcement is evaluated within a limit analysis approach based on the classical constitutive hypothesis of rigid-in-compression no-tension material (Heyman, 1982). In this framework for each assigned kinematically admissible collapse mechanism it is possible to define the relationship between the horizontal and vertical loads corresponding to the onset of that failure mode. Then the collapse domain of the structure is obtained by the inmost envelop of the curves related to each possible mechanism.

In this particular case the interaction loci is characterized by a linear pattern and it is related to the asymmetric mechanism reported in Figure 2.

3  BEHAVIOR OF THE MASONRY FRAME STRENGTHENED WITH FRP AT THE EXTRADOS

The strengthening intervention on the reference scheme of Figure 1 is carried out by reinforcing with a composite sheet the entire surface at the extrados of the frame.

It is assumed that the fibers are not bonded below both base sections of the piers. This arrangement of the strengthening sheet is the most frequent because the foundation wall below the column is usually thicker than the pier, which lies on it.

The lack of anchorage length at the bottom implies that the thrust line must lie in the thickness of the pier at the base section.

When the FRP sheets are applied to the entire extrados of the structure, the classical mechanism of the un-reinforced structure can never form and therefore the failure criteria used in the un-strengthened case, which refers to a global collapse, is no more appropriate to evaluate the failure load of the structure. Indeed, as shown in many experimental results, completely reinforced structures exhibit brittle failures due to the crushing of the masonry (Briccoli Bati & Rovero, 1999), the shear failure at the brick-mortar interface (Valluzzi et al., 2001), the debonding of the sheets.

Furthermore in the examined case also the possibility of a global overturning of the entire structure must be taken into account because of the assumption of sheets not bonded below the springing sections.

In order to apply the mentioned failure criteria the evaluation of the internal stresses is required. This paper shows a model that, in the framework of some simplifying assumptions, provides a method to calculate the internal forces in each cross section of the structure, given the horizontal and vertical loads. Then the application of a failure criterion provides the failure locus related to the selected collapse mode. The inmost envelope of the obtained curves gives the failure locus of the reinforced structure.

The behavior of the FRP reinforced scheme is analyzed simulating a load process where the vertical load is kept constant and the horizontal forces increase.

If the structure is loaded only by a concentrated vertical force V at the crown, as soon as V is greater than the collapse load of the un-reinforced frame, the thrust line moves outwards at the haunches and the collapse occurs when three symmetrical hinges are
already formed. Then the vertical failure load can be easily evaluated on the statically determinate structure shown in Figure 3.

In the definition of the scheme in Figure 3 a simplifying assumption is introduced: at the hinges location the masonry is assumed to be rigid in compression with infinite strength. This hypothesis is on the unsafe side because of two reasons. First, the strain levels at the hinges cannot be evaluated and therefore the model cannot predict the collapse load of the structure if the failure occurs in these zones. On the other hand it must be remarked that usually, the flexural crisis does not occur at the hinges because of the requirement of admissibility of the stress fields. Indeed, at the hinges, the line of thrust must lie within the thickness of the masonry and the eccentricity is much lower than the maximum values we get in the structure.

The latter reason is related to the influence of the position of the hinges on the thrust line. Indeed, because of the finite stiffness and strength of the material the hinges must lie inside the thickness of the arch. This means that the eccentricity of the normal force in many cross sections is larger than that provided by the line of thrust passing through the hinges placed at the borders of the cross sections. However because the distance of the hinges from the external side of the cross section is low this approximation has not a great influence on the values of the collapse load of the structure.

Let's now start increasing the horizontal force H and consider the configuration of the thrust line close to the failure of the structure. For low values of H the pressure line still crosses the extrados at the position of the hinges reported in Figure 3.

Increasing H the intersection of the funicular polygon with the top section of the left pier moves leftwards and reaches the extrados. Let $H_1$ the value of $H$ at this stage.

A new hinge forms (Fig. 4) but the arrangement of the hinges cannot convert the structure into a mechanism. Indeed, the displacement field related to the possible mechanism is not kinematically admissible (Fig. 4).

Indeed, if the increment $\Delta H$ of the horizontal force must do a positive work the top sections of the piers must move rightwards and these displacements can occur only if the hinges at the bottom and at the crown close.

We assume that the closure of the hinge at the bottom left pier occurs before the closure of the hinge at the crown. Then for $H > H_1$ the frame behaves as shown in Figure 5.

Increasing $H$ the line of thrusts at the bottom of the left pier goes inwards until it touches the intrados where a new hinge forms. Let $H_2$ the value of $H$ which causes the new hinge opening (Fig. 6).

Once again the hinge pattern of Figure 6 cannot convert the structure into a mechanism because the related displacement field is not kinematically admissible. Therefore, as in the previous case, one or more hinges must close when the horizontal forces increase. We assume that the hinge at the keystone closes as soon as $H$ increases.
Figure 7 shows the three pin arch which forms for $H > H_2$. This hinge arrangement remains constant at further increase of $H$, but as soon as the vertical component of the reaction at the bottom of the left column is equal to zero the structure overturns around the right corner at the bottom of the right pier (Fig. 8).

In each of the schemes of Figures 3, 4, and 5 the structure behaves as a statically determined three-pin arch and the position of the hinges is not dependent on the value of the loads.

As shown in Ianniruberto & Rinaldi (2004), the case of the complete reinforcement of the intrados is quite different because the hinge patterns are not the same and in each phase the position of some hinges is not known as it depends on the values of the loads. When the FRP is bonded at the intrados, admissibility conditions for the line of thrusts must be added to the system of equilibrium equation in order to evaluate both hinge reactions and positions. On the contrary the structure reinforced at the extrados shows a set of positions of the hinges, which is load, independent in each of the intervals $0 \leq H \leq H_1, H_1 \leq H \leq H_2, H \geq H_2$. Therefore the evaluation of the collapse locus of the structure is easier than in the case of the reinforcement of the intrados.

4 THE CONSTRUCTION OF THE FAILURE LOCUS

As discussed in the previous paragraph, while the un-reinforced frame collapses with the same hinge pattern for every value of horizontal force, as shown in Figure 2, the evaluation of the failure load of the FRP reinforced masonry frame requires the solution of the schemes of Figures 3, 4 and 5. The response of the retrofitted frame is quite different from the unreinforced one as the system always behaves as a three-pin arch but the position of the hinges varies as shown in the previous figures.

4.1 The influence of the debonding

The phenomenon of the debonding of the FRP sheet is considered following the simplified approach proposed for concrete in the FIB Bulletin N.14, (2001) and already applied by the author in a previous paper (Ianniruberto & Rinaldi, 2004). In particular, the debonding effect is accounted for by reducing the ultimate strain in the composite material. This limitation should be related to the mechanical characteristics of the FRP and masonry, by the geometrical properties of the sheets and by the curvature of the surface of the extrados.

Actually due to the lack of indication on the ultimate fictitious values to be assigned at the masonry-FRP interfaces, a parametric analysis is carried out on the reference frame by varying the limit strain in the composite materials.

It must be remarked that in the case of FRP sheets bonded at the extrados the debonded sheet remains close to the masonry surface because of the curvature of the element. However the behavior of the structures is strongly modified and should be analyzed as shown in Focacci (2003) with suitable unbonded length.
4.2 Results

In the following we refer to the material properties reported in Table 1. The sheets are assumed linear elastic in tension and unable to sustain any compressive stresses. The masonry is assumed to have the constitutive law reported in the EC6, (1996) with an ultimate strain equal to 0.0035.

Figure 9 shows the response of the frame of Figure 1 reinforced at the extrados with CFRP sheet whose properties are reported in Table 1. The failure domain is obtained by means of a flexural failure criterion of the cross sections based on the attainment of the ultimate strains in the masonry or in the fibers.

The influence of the shear stresses on the maximum compressive stress in the masonry is negligible and therefore has been neglected.

The segment AB of Figure 9 is obtained by the solution of the system of equations relative to the scheme of Figure 3 and by the application of the mentioned failure criterion. In this case the most critical cross section is placed at the right haunch of the arch. The maximum vertical bearing capacity is equal to 191.8 kN, much greater than the value of the maximum vertical load that the un-reinforced structure can sustain.

The curve BC is related to the Figure 4 and the crisis in that structure occurs once again close to the right haunch of the arch.

In the last branch CD the frame behaves as the three-pin arch reported in Figure 7 and the point D corresponds to the simultaneous attainment of the failure of the right haunch and of the global overturning of the frame.

The arbitrary choice of the hinge that closes in Figure 4 gives some uncertainties in the shape of the locus close to the corner C. More refined analyses are required to define the loci in the connection zone between the segments AB and CD. The line DE is the locus of the couples (H,V), which produce the overturning of the frame around the right corner of the section at the bottom of the right column. This failure mode, which is never activated in the unstrengthened structure, must be always considered because of the great increase of the horizontal bearing capacity of the FRP reinforced structure.

The influence of the value of the limit strain in the fibers, accounting for the FRP debonding, is shown in Figure 10. The failure locus plotted with the solid line is the same curve of Figure 9, whilst the dashed, dotted and gray curves are related to $\varepsilon_{f,\lim} = 0.6\varepsilon_{f,u}$, $\varepsilon_{f,\lim} = 0.4\varepsilon_{f,u}$ and $\varepsilon_{f,\lim} = 0.2\varepsilon_{f,u}$ respectively.

Each collapse couple (H,V) that belongs to these last three curves correspond to a debonding failure. As a consequence, starting from the locus related to $\varepsilon_{f,\lim} = 0.6\varepsilon_{f,u}$ the failure domains reduce quasi-proportionally to the reduction of the limit strain.

On the contrary for $\varepsilon_{f,\lim} = \varepsilon_{f,u}$ the failure is always due to the crushing of the masonry. The maximum strain in the fibers at failure is everywhere lower than 0.11%.

Table 1. Mechanical and geometrical properties of the FRP sheets and masonry

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<th>FRP properties</th>
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Figure 9. The failure domain of the FRP reinforced frame.

Figure 10. The effect of the reduction of the limit strain in the fibers.
The comparison with the un-reinforced structure highlights a great increase of the bearing capacity of the reinforced structure even when the ultimate strain is set equal to \(0.2 \varepsilon_{f,u}\).

In this last case the limit strain is so low that, under pure horizontal loads, the debonding occurs before the overturning of the entire structure.

5 CONCLUSIONS

The paper shows an analytical model to evaluate the strength of a masonry frame made by two piers and an arch under a vertical force at the crown and two horizontal concentrated loads at the abutments of the arch. The model can be easily used to design the strengthening intervention, but it requires the knowledge of both the material properties of the masonry and the limit strain of the fibers, accounting for the FRP debonding.

The parametric analysis performed to evaluate the influence of this phenomenon shows the great sensitivity of the failure loads at the reduction of the maximum strain available in the sheets before the occurrence of the debonding.

The comparison with the un-reinforced case proves the effectiveness of the FRP reinforcement of the extrados that allows the development of a great increase of strength even when the limit strain in the composite material is very low.

REFERENCES


Eurocode 6 – Design of masonry structures, ENV 1996.


