Effect of FRCM properties on masonry out of plane strengthening

Giancarlo Ramaglia¹², Gian Piero Lignola¹, Francesco Fabbrocino² and Andrea Prota¹

¹ Department of Structures for Engineering and Architecture, University of Naples Federico II, Via Claudio 21, 80125 Naples, Italy
² Department of Engineering, Telematic University Pegaso, Piazza Trieste e Trento, 48, 80132 Naples, Italy

ABSTRACT: Significant part of the world cultural heritage is represented by masonry buildings. Many of them are in need of strengthening, especially in high seismic risk areas. The recent earthquakes showed that the Historical and Monumental buildings are often characterized by high seismic vulnerability especially if there are thrusting elements as arches and vaults. The present work focuses on a particular typology of innovative strengthening system, compatible with masonry, the Fiber Reinforced Cementitious Matrix (FRCM). In particular this material presents an high compatibility with masonry, but has a peculiar constitutive relation characterized by a linear behavior up to cracking of the mortar matrix, then a tension stiffening branch up to dry fiber behavior. The scope of this work is the evaluation of the impact of cracking and nonlinear behavior of this material on the global response of masonry. A proper design of the strengthening intervention should involve also the correct use of these peculiar properties of FRCM. The strengthening material provides tensile capacity to masonry, but this has not only an impact on strength, but also on ductility. To this aim both PM domains and moment curvature relationships are discussed. The main application considered herein is the out of plane improvement of masonry performance, as it can be the dominant failure mode in masonry walls, but also masonry curved elements like as vaults and domes.

1 INTRODUCTION

In the scientific literature the benefits of innovative strengthening systems are shown. They are a useful solution to improve the ultimate capacity of masonry elements. However, when strengthening strategies regard historical buildings, some innovative solutions can show some drawbacks. In particular, carbon fibers and epoxy matrices have high mechanical properties and they can improve the seismic capacity of masonry elements. However, these materials are often incompatible with historical masonry. The drawbacks are due to an incompatibility of the strengthening components with the masonry substrate, Papanicolaou et al. (2008). Conversely, typical historical masonries can be strengthened using systems based on inorganic matrices. The inorganic matrix can be made of several materials. The nature of the materials depends on the characteristics of the substrate, Lignola et al. (2009) and Lignola et al. (2012). In some cases the lime-mortar results preferable if compared with the cementitious-mortar. However, this aspect is inessential for the present work where the attention focuses on both inorganic matrices like as cementitious-mortar or lime-mortar. Similarly the fibers can be made of different materials. The
characteristics of the fiber depend on the mechanical properties of masonry. For weak masonry, the natural fibers can be an adequate solution. Indeed, for modern masonries, fibers like as basalt, steel and glass can be the better solution, De Santis et al. (2017), Lignola et al. (2017), and Fabbrocino et al. (2015).

In order to plan the strengthening strategies on historical structures some of the previous fibers cannot be used. In particular, for historical masonry the basalt fibers guarantee a more suitable solution for compatibility in terms of mechanical and physical properties. The strengthening systems provide effects both on the load capacity and on the ductility of the structural elements, Ramaglia et al. (2015). Capacity in ductility represents a key aspect to assess the ultimate behavior of strengthened masonry elements. In particular, in order to assess the actual capacity of strengthened masonry elements, the capacity in ductility represents fundamental information. The impact of composites based on inorganic matrix on the ultimate behavior of masonry elements is discussed in the present paper.

2  ULTIMATE BHEAVIOUR OF STRENGHTENED MASONRY ELEMENTS

2.1  P-M interaction domain assessment

First goal involves finding the P-M interaction domain of a generic strengthened masonry element. The envelope of the maximum bending moment value \( M_{\text{max}}(P) \), given an axial load value \( P \), provides the P-M interaction domain. The maximum bending moment \( M_{\text{max}}(P) \) can be obtained according to several approaches. In this work the \( M_{\text{max}}(P) \) value has been calculated starting from the bending moment-curvature diagram. This solution is due to the need to estimate capacity in ductility, too. In fact, the capacity in ductility represents a key information to assess the actual ultimate behavior of masonry structures. Finally, the strength bending moment \( M_{\text{max}}(P) \) can be obtained as maximum value of the bending moment-curvature diagram (Figure 1).

![Bending moment-curvature diagram](image1)

![P-M interaction diagram](image2)

Figure 1. P-M interaction domain assessment.

2.2  Bending moment-strain diagram assessment

The bending moment-curvature diagram was evaluated by means of a discrete model. The work focuses on the strengthened masonry elements. For this work several strengthening systems have been considered. The masonry material was modelled in compression according to the EuroCode 6, EN Eurocode6 (2005). The tensile strength of the masonry is generally negligible given its low impact on the ultimate behavior. However, recent studies showed that particular
Masonry structures like as slender curved elements are particularly sensitive to tensile strength, Ramaglia et al. (2016). Therefore, in order to assess the actual capacity of these structures a non-zero tensile strength must be considered. Therefore, the masonry was modeled as brittle in tension.

The strengthening systems were modeled by using experimental results, De Santis et al. (2017) and Lignola et al. (2017). In particular, capacity tests have been performed on several strengthening systems based on inorganic matrix. The experimental tests were performed by varying the fiber elements. The experimental tests provide the stress-strain curve of each composite. This information allows to model the strengthening systems. In particular, numerical stress-strain curve was obtained by an interpolation of the experimental response. This approach allows to assess the impact of different numerical modelings of the stress-strain curve.

The bending moment-curvature diagram was assessed by using a numerical approach. A solver algorithm was implemented in order to calculate the bending moment-curvature. It is based on the progressive increase of the curvature values. In particular, starting from a pure axial load, the curvature is increased up to ultimate condition. The ultimate condition is related to the failures of materials. The ultimate behavior occurs when the masonry achieves the compressive strength $\sigma_m$. This is the typical failure mode when the strengthening system shows mechanical properties higher than masonry. Strengthening systems performed on ordinary masonries show this failure mode. In particular, for ordinary strengthening systems both the matrices (high-strength mortar) and fiber elements (steel, glass or basalt) are characterized by high mechanical properties. Conversely, when the goal is to improve the ultimate capacity of poor masonries, the previous strengthening systems can be replaced by more compatible systems. In these cases, the matrix and fiber can be replaced by ordinary mortar and natural fiber like as hemp, respectively.

The solver algorithm allows to assess bending moment varying the curvature value. The numerical model is based on a discrete approach. In particular, the cross section was discretized by a finite number of parts. The discretization depends on the precision in the calculation. In this work the convergence criterion is based on an axial equilibrium of the cross section. In particular, once fixed the axial load value $P_i$ and a generic curvature step $\chi$, the neutral axis is changed to satisfy the equilibrium equation. When the convergence criterion is satisfied, the bending moment $M(P, \chi)$ can be calculated with classical equilibrium equations. The numerical process continues up to the ultimate condition. The ultimate behavior can be achieved according to the failure modes of materials. Figure 2 shows the ultimate behavior according to failure modes of materials. In particular, the ultimate behavior can be due to the local crushing of the masonry (Figure 2a). In this way ultimate strain equal to 3.5% occurs at the compression side, EN Eurocode (1996). Where the tensile stress is higher than the tensile strength the cross section cracks. Finally, the strengthening system shows a stress level lower than its tensile strength. The failure mode shown in Figure 2a is typically achieved in strengthening systems characterized by mechanical properties higher than masonry. When the strengthening strategy focuses on poor masonry the strengthening components can be replaced by compatible materials. In this background the ultimate behavior can be due to the failure of the fiber element as shown in Figure 2b.

Figure 2 shows a typical strengthening solution where the composite is applied on one side. This solution provides different impact depending on whether the bending moment is positive or negative. In fact, the strengthening system in compression is generally negligible given the instability phenomena. In this case the bending moment-curvature diagram depends on the masonry only.
Figure 2. Failure modes when the strengthen system reacts: a) achievement of the ultimate strain of the masonry, b) failure of the strengthening system.

3 GEOMETRICAL AND MECHANICAL PROPERTIES OF THE SECTIONS

3.1 Geometrical characteristics

Figure 3 shows the geometrical characteristics of the cross section used for the numerical analyses. These geometrical characteristics refer to a single wythe clay masonry cross section with unitary depth. The single masonry block has dimensions equal to $25 \times 12 \times 5.5$ cm$^3$. Two strengthening systems have been considered. The first strengthening system is made of inorganic matrix and basalt grid. While the second strengthening system is made of inorganic matrix and steel mesh. Table 1 shows the geometrical properties of strengthening systems; the area of the fibers $A_f$ and thickness of the matrix $t_m$. 
Table 1. Geometrical characteristics of the strengthening systems

<table>
<thead>
<tr>
<th>Strengthening system</th>
<th>Grid area $A_f$ [mm$^2$/m]</th>
<th>Thickness of matrix [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basalt fibers</td>
<td>39</td>
<td>15</td>
</tr>
<tr>
<td>Steel fibers</td>
<td>138</td>
<td>15</td>
</tr>
</tbody>
</table>

Figure 3. Geometrical characteristics of the strengthened cross section and of the single clay brick element.

3.2 Mechanical properties

The mechanical properties have been chosen for masonry and strengthening system according to different approaches. The compressive and tensile strengths $\sigma_m$, $\sigma_{mt}$ of masonry have been estimated as average values typically found in the scientific literature. In particular, the masonry aims to simulate an existing masonry. Therefore, the compressive strength $\sigma_m$ has been assumed as 3 MPa (table 2). Conversely, three values for tensile strength have been chosen (table 2). These values have been calculated as function of the compressive strength through the coefficient $\alpha = \sigma_{mt}/\sigma_m$ (table 2). The behavior of the masonry has been modelled according to the Eurocode 6, EN Eurocode6 (2005). In particular, the masonry can be modelled by means of two curves. Starting from zero strain up to a value of $\varepsilon_0=2\%$ the stress-strain curve can be defined by a parabolic curve. While for strain values ranging from $\varepsilon_0$ to $\varepsilon_{mu}=3.5\%$ the stress value is constantly equal to $\sigma_m$. For the strengthening systems a different approach has been adopted. In particular, starting from the experimental tests conducted on several strengthened specimens, De Santis et al. (2017) and Lignola et al. (2017), the stress-strain curve has been satisfactorily interpolated by a linear function. The mechanical properties of the strengthening systems have been reported in table 2. The experimental curves used for the numerical analysis have been shown in figure 4. The following diagrams refer to representative average curves where the direct tensile test results in terms of stress are limited by the bond capacity to the substrate.

Table 2. Mechanical properties of materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Compressive strength [MPa]</th>
<th>Tensile strength $\sigma$ [MPa]</th>
<th>Young’s modulus [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry</td>
<td>3</td>
<td>$0$ ($\alpha=0%$); 0.15 $\alpha=5%$; 0.30 $\alpha=10%$)</td>
<td>3</td>
</tr>
<tr>
<td>Basalt grid</td>
<td>-</td>
<td>1538</td>
<td>45.2</td>
</tr>
<tr>
<td>Mortar homogenized to basalt fiber</td>
<td>-</td>
<td>485</td>
<td>1243.0</td>
</tr>
<tr>
<td>Steel grid</td>
<td>-</td>
<td>900</td>
<td>112.5</td>
</tr>
<tr>
<td>Mortar homogenized to basalt fiber</td>
<td>-</td>
<td>256</td>
<td>611.8</td>
</tr>
</tbody>
</table>
4 NUMERICAL RESULTS

4.1 Bending moment-curvature diagram

A solver algorithm based on a discrete approach has been used. The bending moment vs. curvature diagram has been evaluated for two strengthening systems actually tested. Figure 5 shows the results for the strengthening systems changing the axial load value \( P \) (0\% and 10\% of ultimate axial load \( P_0 \) respectively). Figure 5 allows to compare the behavior between basalt fiber (green solid line) and steel mesh (red solid line). The axial load values have been chosen according to the typically axial load values achieved at collapse by masonry elements, Ramaglia et al. (2016a) and Ramaglia et al. (2016b). In fact the compressive stress level at collapse is much lower than the compressive strength. For this reason, the axial load was changed from about zero up to 20\% of the axial strength value \( P_0 = \sigma_m \cdot b \cdot s \). Furthermore, it is interesting to monitor the stress achieved in the strengthening systems during the entire load history. For this reason figure 5 shows the comparison of stress levels, in dotted line, referring to secondary vertical axis) achieved for the basalt grid (dotted thin line) and steel (dotted thick line).

Figure 5. Bending moment-strain diagram: masonry section strengthened with basalt and steel grid comparison: \( P = 10\% P_0 \).
It is interesting to note that the stress level achieved in the strengthening system is lower than the tensile strength when the global collapse occurs. In the case of pure flexure the strengthening system reaches about 80% of its tensile capacity, while in the case of higher axial load, less than 70% of FRCM capacity, $\sigma_r$, is used. In particular, the numerical results confirm the failure of the masonry to be the preferential failure mode of strengthened masonries with steel or basalt grids. This result is due the high mechanical properties of the strengthened systems. Furthermore, the tensile strength is significant only when unreinforced masonry is considered (and it is well known that under pure bending the flexural capacity would be zero, if no tension is assumed). In fact, for the strengthened sections, the impact of the masonry tensile strength on the ultimate behavior can be neglected.

Finally, matrix cracking provides the transition from the first elastic branch (in moment curvature diagram of strengthened masonry) to the second “hardening” phase where a pseudo ductility can be found. It is worth noting that in the case of pure bending (i.e. $P=0%P_0$) the strengthening system provides some pseudo ductility to the masonry, while in the case of higher axial load, the FRCM system reduces the ultimate curvature compared to unreinforced masonry.

4.2  P-M Failure domain assessment

Starting from the bending moment-curvature diagrams, the maximum bending moment can be evaluated. In particular, for each specimen the bending moment value has been calculated varying the axial load value. The axial load value $P$ was changed from the zero up to the pure compressive strength $P_0$. The increase of the axial load has been pointwise. The envelope of the P-M points provides the failure surface of the specimen. Figure 6 shows a comparison of the failure surface calculated for several specimens. It is interesting to note that the tensile strength of the masonry provides a not negligible effect when the strengthening system does not react (grey lines). In fact, only if the strengthening system reacts (i.e. negative bending moments), the low value of tensile strength of masonry provides a negligible impact on the ultimate behavior. However, when the strengthening system is compressed (i.e. positive bending moments) the tensile strength provides a not negligible effect on the ultimate capacity of masonry, Ramaglia et al. (2016a).

![P-M Failure surfaces of the two strengthened masonry specimens varying the tensile strength value.](image)

Figure 6. P-M Failure surfaces of the two strengthened masonry specimens varying the tensile strength value.
5 CONCLUSIONS

The ultimate capacity of masonry strengthened with several FRCM strengthening systems was assessed. The strengthening systems are made of inorganic matrix and basalt or steel fibers. The ultimate behavior was assessed both in terms of maximum bending moment and curvature capacity. In particular the maximum bending moment has been evaluated starting from the bending moment-curvature diagram. The solver algorithm considers a cross section discretized into a finite number of strips. The masonry in compression was modelled according to the Eurocode 6. Instead, in tension a brittle behavior was modelled for the masonry. The strengthening system was modelled according to the experimental results by means of a linear interpolation. The numerical results provide key information on the ultimate behavior of strengthened masonry. A first important result confirms that the failure of masonry is the preferential failure mode when the strengthening system is based on steel or basalt fibers, due to the high mechanical properties of the strengthening systems. In particular, the stresses achieved at collapse by the strengthening system are lower than the tensile strength. Therefore, small amounts of strengthening systems provide a strong improvement of the ultimate behavior of the masonry elements. Finally, the role of matrix is very important, it is clearly seen that the limit of the first elastic branch of strengthened masonry is strictly related to the cracking of the FRCM matrix. So the effect of FRCM strengthening is to improve the flexural capacity, but this provides a small amount of ductility in the case of pure bending, while it limits the ultimate capacity in the case of higher axial loads. The matrix cracking provides the transition from the first elastic branch in moment curvature diagram of strengthened masonry to the second “hardening” phase where a pseudo ductility can be found.

6 REFERENCES