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FRP-Strengthening of Curved Masonry Structures: Local Bond Behavior and Global Response

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Abstract. The aim of the paper is to propose and assess the reliability of a modeling strategy which combines the homogenization of the masonry material and the use of zero-thickness interface elements. This strategy is specifically proposed for numerically investigating the structural response of FRP-reinforced curved masonry structures. Indeed, in order to consider the influence of the geometry curvature of the masonry substrate on the local bond behavior of the FRP-strengthening system, bond-slip laws which specifically account for the geometric curvature of the substrate are introduced at the FRP/substrate interface layer. Numerical analyses concerning masonry arches selected from the current literature are presented in the paper in order to assess the reliability of the proposed modelling approach.

Introduction

Arches, vaults and domes represent important structural components of existing masonry constructions. The curved configuration of these structures influences considerably the characteristics of their collapse mechanism and, at the same time, the type of strengthening intervention to provide for improving their behavior.
Innovative materials together with new strengthening techniques are object of numerous researches focused on both their applicability and the development of numerical models [1]-[8]. Recent and widespread strengthening techniques are based on the use of fiber reinforced composite materials FRP externally applied at the intrados and/or the extrados in the form of sheets or strips. Experimental evidences available in literature show the beneficial effect of this type of strengthening on the global response of curved masonry structures [9]-[11]. At the same time, these studies also emphasize the influence of the geometry on the local bond transferring mechanism between the FRP-strengthening system and the curved masonry substrate. In a recent study carried out by the authors [12], simple formulas for deriving bond-slip laws by taking into account the radius curvature of the masonry substrate have been proposed.
The aim of the present paper is to numerically investigate the influence of the local bond behavior of FRP-strengthening systems on the global response of reinforced curved masonry structures by using a simplified approach which merges two strategies: the homogenization of the masonry material (performed with two different approaches), and the use of shear deformable interface

elements where the effect of the geometric curvature is directly introduced in the evaluation of the shear strength.
Taking into account this approach, finite element numerical analyses are developed considering un-strengthened and FRP-strengthened masonry arches derived from the current literature [10]. The obtained results are presented and critically commented, showing good match with experimental evidences.

Proposed modeling strategy

The proposed modeling strategy for the nonlinear static analysis of curved masonry structures strengthened by FRP-systems applied on the external surfaces, is based on a classic two-step approach:

- In the first step, the homogenization of the masonry material is performed;
- In the second step, the derivation of an equivalent shear stress-slip law for simulating the behavior of the interface layer interposed between the masonry support and the reinforcement [11] is proposed.

This approach has the twofold advantage to avoid the use of interface elements for simulating the behavior of mortar layers interposed between masonry blocks and, moreover, it allows avoiding an introduction of coupled interfaces for considering the interaction between shear and normal stresses which, differently for planar substrates, assume a relevant role on the bond mechanism of curved substrates.

Homogenized masonry model

A homogenized model of the masonry material has been considered to analyze the masonry arches. In particular, two different strategies have been adopted, one numerical (Model 1) and one analytical (Model 2), by considering in both cases the elementary cell depicted in Figure 1. It is constituted by two half masonry blocks and an interposed layer of mortar. In both cases, homogenization has been performed by considering a linear elastic behavior for blocks and a nonlinear behavior for mortar in tension. Taking into account the results of tensile tests on mortar specimens [11], a bilinear law has been assumed for the mortar material considered (see Figure 1). In Model 1, a refined FE discretization of the elementary cell is assumed. To determine the homogenized uniaxial stress-strain behavior, a simple load condition is analyzed, consisting of two opposite forces applied along the vertical free edges of the semi-blocks and monotonically increased during the FE computations (performed with the commercial code TNO-DIANA [13]). The Σ_{xx}-ε curve derived from the homogenization process has been then introduced in the homogenized model of the masonry material.

Model 2, see Figure 1 (bottom), is semi-analytical and based on the same holonomic procedure proposed in [14]. It assumes bricks elastic and joints reduced to interfaces with a piecewise-linear relationship between normal stress and jump of displacement.

Writing compatibility along the straight line passing through nodes 1, 2, 3 and 4 and remembering that the behavior of the bricks is elastic, the following equations can be written:

$$\begin{aligned} &U_n^0 - U_n^2 = \frac{\sigma_n H}{E_b} \\ &U_n^3 = \frac{\sigma_n H}{E_b} \\ &\Rightarrow U_n^0 - \Delta_n = 2\frac{\sigma_n H}{E_b} \end{aligned} \tag{1}$$

where E_b is brick elastic modulus, U_n^0 is the imposed displacement on node 1 (equal to $U_n^0 = E_{nn}(2H + e_v)$, E_{nn}: homogenized strain), σ_n the homogenized stress, H the brick semi-height, Δ_n the mortar interface jump of displacement.

The homogenized stress strain curve can be graphically determined as the point of intersection between the last of Eq. (1), which is a straight line in the $\sigma_n - \Delta_n$ plane, and the interface constitutive relation $\sigma_n = f^I(\Delta_n)$.

Both models, provided that the constitutive behavior of the joints is the same, provide the same homogenized result, which is shown in Figure 1 for the arches analyzed after.

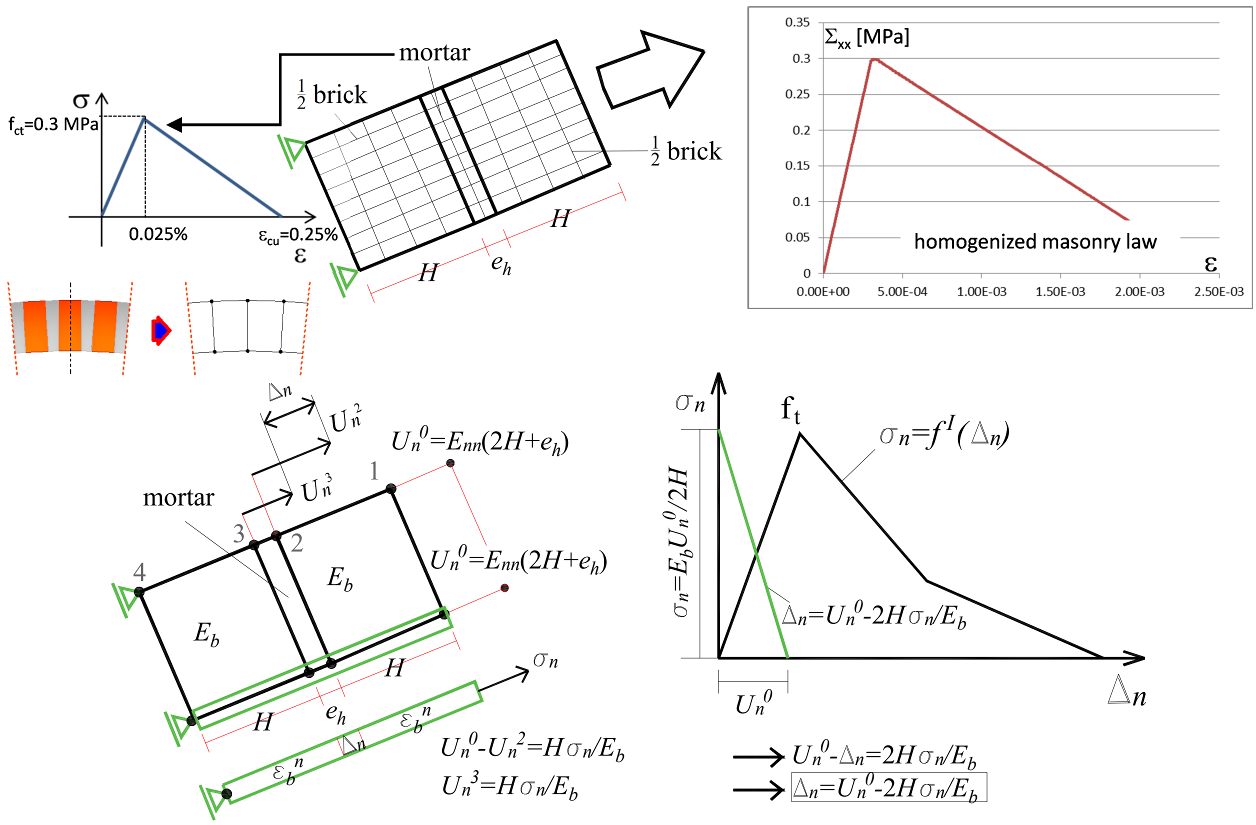

Figure 1: Schematization of the homogenization process used for the masonry material. Top: discretization into FEs. Bottom: analytical approach.

Then, considered the homogenized law, a smeared crack model is assumed for the masonry material by selecting the option *total strain rotating crack* [13] and imposing a linear elastic behavior in compression.

In order to have a further insight into the numerical approach proposed, the results obtained at a structural level with the total strain model in DIANA [13] are compared also with those obtained with a Concrete Damage Plasticity model (CDP) available in ABAQUS [15] where however the reinforcement is assumed perfectly bonded to the surface and with equivalent mechanical properties that take into account the possible delamination.

FRP-strengthening model

Regarding the strengthening system, the proposed approach considers simple elastic truss elements for simulating the reinforcement and zero-thickness interface elements introduced for the layer interposed between the masonry and the reinforcement. Particular attention has been devoted to the modeling behavior of the interface. Indeed, since the curvature of the masonry substrate induces normal stresses which affect the shear strength of the interface, the approach proposed by Grande and Milani [12] has been here considered. Starting from the shear strength τ^0 evaluated according to Grande et al. [2] for planar substrates, the simplified formula proposed in [12] has been used for evaluating the bond strength τ_b (Figure 2):

$$\tau_b=\tau^0 e^{-\alpha \chi \tan(\Psi)} \tag{2}$$

where χ is the curvature geometry of the arch, Ψ is the friction angle (a value of $tan(\Psi)=1.41$ has been assumed), and α is a coefficient assumed equal to 55 [12].

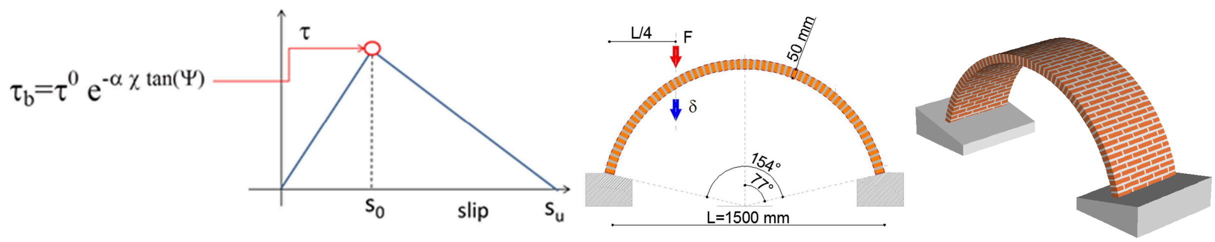

Figure 2: Derived shear stress-slip law for the FRP/masonry interface component (left) and masonry arches analyzed to benchmark the approach proposed.

Case studies

The case studies considered are one un-strengthened and one FRP-strengthened at the intrados semi-circular masonry arch, see Figure 2. Experimental data are collected from existing literature available and for further details the reader is referred to [10], where results concerning both experimental tests and numerical analyses are at disposal.

FE modeling and numerical analyses

Taking into account the proposed modeling strategy and the parameters reported in Table 1 [11], the FE models and the numerical analyses have been developed through the commercial code TNO-DIANA [13]. In particular, the following finite elements have been selected:

- four-node quadrilateral isoparametric plane stress elements based on linear interpolation and Gauss integration, assuming a 2 × 2 integration scheme (labeled as Q8MEM in DIANA), for both the masonry;
- two-node directly integrated (1-point) truss elements (labeled as L2TRU in DIANA), for the reinforcement;
- zero-thickness elements based on linear interpolation and considering a 3-point Newton–Cotes integration scheme (labeled as L8IF in DIANA), for interfaces interposed between the masonry and the reinforcement.

Table 1: Parameters accounted for the numerical FE analyses.

Homogenized Masonry	**FRP**	**FRP/masonry interface**
Young's modulus $E_b=1400\ MPa$	Young's modulus $E_f=80000\ MPa$	Bond strength $\tau_b=1.0\ MPa$
Poisson's coefficient $\nu=0.2$	Poisson's modulus $\nu=0.2$	Slip at the end of the ascending branch $s_0=0.02mm$
Tensile strength $f_{ct}=0.30\ MPa$	equivalent thickness $t_f=0.299mm$	Slip at the end of the descending branch $s_u=0.6mm$
Mode I fracture energy $G_{ft}=0.005\ N\ mm/mm^2$	FRP width $b_f=100\ mm$	Shear stiffness $G_e=51\ N/mm$

Nonlinear static analyses have been carried out by applying an incremental load at the loaded section of the arch and using an iterative linear method coupled with a convergence norm criterion imposed on both displacements and forces.

Un-strengthened arch

The results concerning the un-strengthened arch are shown in Figure 3 in terms of force-displacement curve and in Figure 4 in terms of principal stresses at peak load.

From the plots it emerges a good approximation of the experimental evidences both in terms of peak load and post-peak behavior particularly in the case of the numerical model developed through ABAQUS: a linear behavior characterizes the first load steps; a progressive stiffness degradation occurs before the attainment of the pre-peak load; a remarkable softening behavior characterizes the post-peak stage. At the same time, examining the principal stresses at the peak load (Figure 4) it is possible to derive information on the pattern of hinges which results in agreement with the one emerged from experimental tests. Moreover, considering the same load step, numerical analyses underline a maximum value of compressive principal stresses of about 1 MPa at the extrados of the loaded zone of the arch.

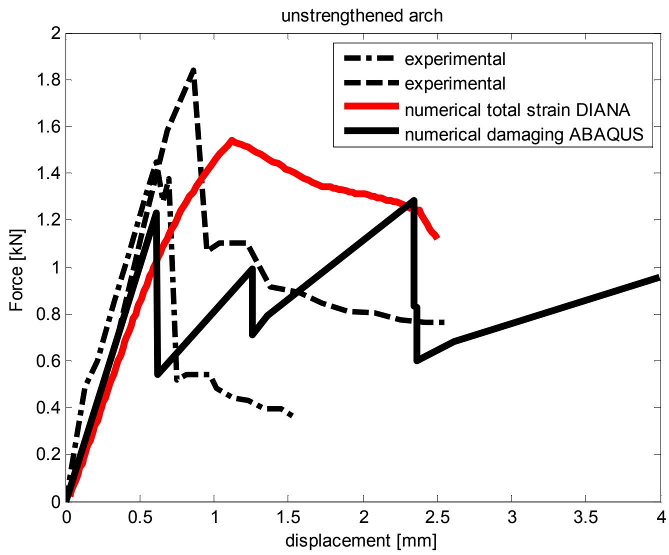

Figure 3: Un-strengthened arch: experimental and numerical Force-displacement curves.

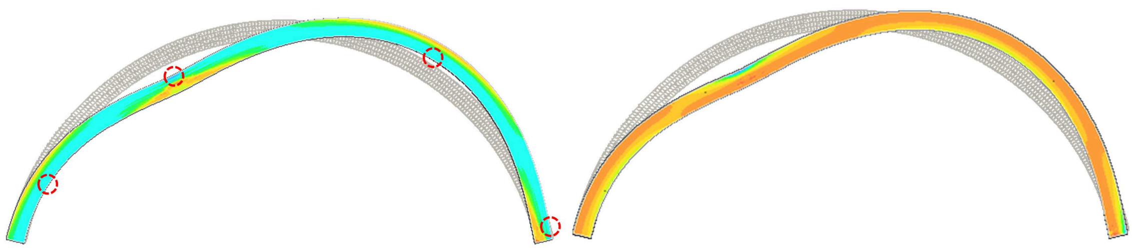

Figure 4: Un-strengthened arch: maximum principal stresses and pattern of hinges at the peak load (left); minimum principal stresses at the peak load (right).

FRP-strengthened arch

The results concerning the FRP-strengthened arch are shown in Figure 5, together with the experimental curves. Additional results concerning principal stresses and the shear stress at the FRP/masonry interfaces are also reported in Figure 6 and Figure 7.

The comparison between the experimental and numerical force-displacement curves underlines the capability of the proposed approach to furnish a good approximation of the experimental behavior of the arch accounting for the contribution of the reinforcement: after a first drop of the load, a post-peak ductile behavior characterizes both the experimental and the numerical response. Nevertheless, the numerical curve shows a post-peak behavior with a global resistance slightly lower than the experimental one.

By examining the distribution of both minimum and maximum principal stresses at the first peak load (Figure 6), i.e. before the drop of the curve occurs (phenomenon also evident during experimental tests), it is interesting to notice a concentration of tensile stresses in the same sections of the un-strengthened arch where the hinges form. Nevertheless, examining the principal compression stresses, numerical analyses show values of the principal compression stresses significant greater than the ones emerged in the case of the un-strengthened arch. This is a consequence of the presence of the reinforcement which contributes to resist tensile forces at the intrados of the arch.

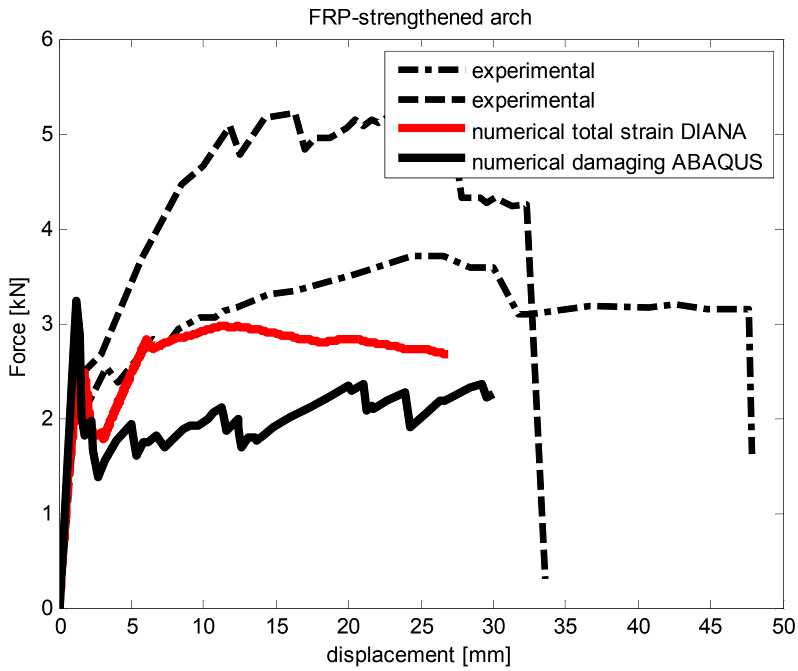

Figure 5: FRP-strengthened arch: experimental and numerical force-displacement curves.

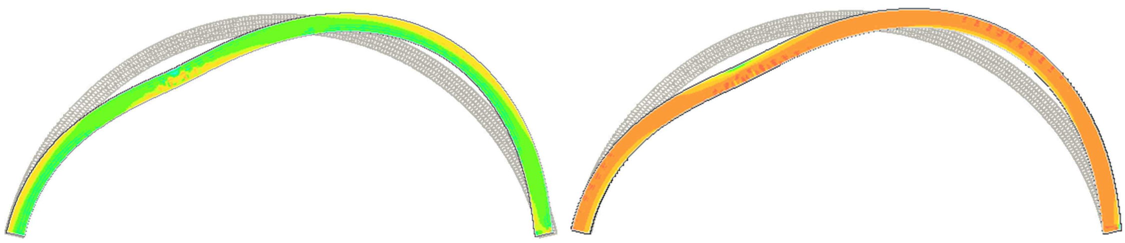

Figure 6: FRP-strengthened arch: maximum principal stresses (left) and minimum principal stresses (right) at the first peak load: F=2490 N.

In the case of the FRP-strengthened arch the global behavior is strongly affected by the local bond mechanism of the strengthening system. In Figure 7 the shear stresses at the FRP/masonry interface emerged from the numerical analyses at two different load steps are reported. In particular, the stresses are depicted through triangular symbols whit a dimension proportional to the stress value. From the figure it is interesting to notice that the de-bonding starts at the base of the arch where a free-end debonding phenomenon occurs due to the absence of anchorage devices. This phenomenon does not significantly affect the global response of the arch but it leads to a progressive increase of the interface shear stresses at the loaded zone of the arch. Here, the de-bonding of the reinforcement, together with the progressive damage of the masonry material, significant affects the global response of the arch.

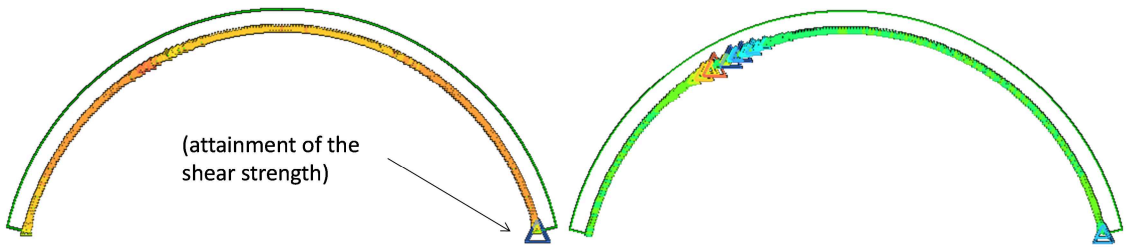

Figure 7: FRP-strengthened arch: shear stress at the FRP/masonry interface: pre-peak stage F=1100 N (left); first peak load F=2490 N (right).

Conclusions

The present paper concerns the modeling of curved masonry structures strengthened with FRPs. In particular, a simple modeling approach based on both the homogenization of masonry and on the use of zero-thickness interface elements shear deformable only has been proposed for simulating the behavior of masonry arches derived from the current literature. Particular attention has been given to the derivation of the tau-slip laws of the interfaces by considering the approach proposed by Grande and Milani [12], where the geometric curvature is explicitly considered for taking into account the effect of normal stresses on the interface shear strength.

The results obtained from numerical analyses have underlined the ability of the proposed approach in predicting the experimental behavior of the FRP-strengthened arch by also underlining the role of the local stress transferring mechanism at the interface level.

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