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PRORAČUN ZIDANIH KONSTRUKCIJA OJAČANIH FRP MATERIJALIMA

Rezime:

Zidane konstrukcije su sklone različitim oštećenjima koja su posledica preopterećenja, grešaka pri projektovanju ili izvođenju, problema trajnosti, incidentnih dejstava. U zavisnosti od stepena oštećenja, može doći do kolopsa konstrukcije. Ovo ukazuje na potrebu za razvojem efikasnih i pristupačnih tehnika ojačanja zidanih konstrukcija, kako bi se postiglo poboljšanje mehaničkih karakteristika, kao i veća pozdanost konstrukcija. Uzimajući u obzir prednosti i nedostatke tehnika ojačanja, polimeri ojačani vlaknima - FRP materijali su pokazali veliku efikasnost i lakoću primene pri ojačanju zidanih konstrukcija. U ovom radu su prikazane smernice za proračun i numerički primer ojačanja zida FRP materijalima za opterećenje u ravni.

Ključne reči: zidane konstrukcije, ojačanje, FRP, proračun, granično stanje nosivosti, naprezanje u ravni

DESIGN OF FRP STRENGTHENED MASONRY STRUCTURES

Summary:

Masonry structures are prone to different types of damages caused by overloading, design or construction errors, durability problems, accidental loading. Depending on the extent of damages, failure of masonry structures can occur as a consequence. This indicates a need to develop efficient and affordable strategies for strengthening masonry structures, thus achieving improvement of their mechanical characteristics and providing greater reliability. Considering advantages and disadvantages when choosing materials for reinforcement, Fibre Reinforced Polymers (FRP) have proven to be very efficient and easily applicable solution. In this paper, design guidelines and numerical example for strengthening masonry walls with FRP materials for in-plane loading are presented.

Key words: masonry, strengthening, FRP, design, ultimate limit state, in-plane loading

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1. INTRODUCTION

Masonry structures represent the oldest man-made buildings. More often than not, these structures were built without adequate design guidelines, solely based on the then-existing experience. The need for their restoration and strengthening became evident over the years from various reasons, such as: structural problems and damages caused by changing loads and exposure conditions, accidental loads, inadequate design, errors in construction, floods, foundation settlement, deterioration resulting from abrasion, fatigue, chemical exposure, weathering, inadequate maintenance etc.; current codes and standards requirements; structure's change of use; durability problems; increased life-span demands for old structures.

Many different strengthening techniques were developed and tested over the past century. Nowadays, both techniques which use traditional and innovative materials and solutions are employed depending on the unique requirements appearing in each individual situation. When selecting adequate strengthening technique following principals should be considered: minimal intervention level, reversibility, non-invasiveness, durability, compatibility with original structure and cost.

Due to low weight, high stiffness and tensile strength, corrosion resistance, a wide range of available shapes and sizes and possibility of application on irregularly shaped surfaces, FRP (Fibre Reinforced Polymer) materials are increasingly employed for reinforcement of variety of structures, including masonry. Strengthening solutions include application of FRP materials in form of plates, bars, unidirectional and bidirectional sheets installed on the masonry members by adhesion or by mechanical anchorage devices. Selection of the adequte FRP material (carbon, glass or aramid fibres) should into account physical and chemical properties of the masonry. Primarily, externally bonded FRP plates have been used, with near surface mounted FRP bars becoming increasingly popular due to anchoring or aesthetic advantages.

Flexural strengthening of unreinforced masonry walls using different types of FRP plates and sheets (Figure 1a) was experimentally and theoretically analysed in papers by *Hartley et al* [1], *Hamilton III & Dolan* [2], *Yousuf & Tarek* [3] and *Ayman* [4]. Shear strengthening of masonry walls with FRP plates (Figure 1b) was investigated by *Valluzzi et al* [5], *Motavalli* [6] and *Marcari et al* [7]. Flexural and shear strengthening of masonry walls using FRP bars (Figure 1c) is presented in papers by *Nanni & Gastavo* [8] and *Turco et al* [9]. All mentioned researches and numerous others proved the effectiveness of interventions, that is, significant increase in flexural and shear capacity depending on the number and position of FRP reinforcement.


(a) *(b)* *c)*

Figure 1 – FRP strengthening of masonry: FRP plates – flexural strengthening (a); FRP plates – shear strengthening (b); FRP bars – flexural and shear strengthening (c)

Adequate design approach for strengthening masonry walls includes analysing the existing ultimate limit state for appropriate loads and developing optimal strengthening solutions that would satisfy different criteria presented in each situation. In this paper, design of FRP strengthened masonry wall subjected to in-plane vertical and horizontal loading is presented in accordance with the recommendations given in CNR-DT 200 R1/2013 [10].

2. DESIGN OF FRP STRENGTHENED MASONRY MEMBERS

2.1. GENERAL DESIGN PRINCIPLES

General expression for structural safety is also employed for the design of FRP strengthened masonry members:

$$E_{\mathrm{d}} \leq R_{\mathrm{d}} \tag{1}$$

where: E_{d} - required design resistance calculated for design loads according to the current building code; R_{d} - design resistance.

Design resistance for the specific type of load effect being considered (e.g. flexural, shear, etc.) can be calculated as:

$$R_{\mathrm{d}} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot R\left\{X_{\mathrm{d,i}}; a_{\mathrm{d,i}}\right\} \tag{2}$$

where: γ_{Rd} - partial factor accounting for uncertainties in the assumed model; $X_{\mathrm{d,i}}$ - design values of the material properties; $a_{\mathrm{d,i}}$ - nominal values of the geometrical parameters. Value of partial factor γ_{Rd} depends on the resistance model, according to [10]: bending/combined bending and axial load $\gamma_{\mathrm{Rd}} = 1.0$; shear/torsion $\gamma_{\mathrm{Rd}} = 1.2$; confinement $\gamma_{\mathrm{Rd}} = 1.1$.

Design value of the considered FRP material property is:

$$X_{\mathrm{d}} = \eta \cdot \frac{X_{\mathrm{k}}}{\gamma_{\mathrm{m}}} \tag{3}$$

where: η - conversion factor accounting for special design problems; X_{k} - characteristic value of material property, γ_{m} - partial factor for materials.

Recommendation for partial factor of FRP materials $\gamma_{\mathrm{m}} = \gamma_{\mathrm{f}}$ value is equal to 1.1 for ultimate limit states. In the case of FRP debonding, value of $\gamma_{\mathrm{m}} = \gamma_{\mathrm{f,d}}$ can be selected in a range of 1.2 to 1.5, depending on the probability of failure due to debonding, according to [10].

Conversion factor η_{a} takes into account degradation of mechanical properties of FRP materials as a consequence of environmental conditions such as alkaline environment, moisture, extreme temperatures, thermal cycles, freeze-thaw weathering, and UV radiations. Environmental conversion factor η_{a} depends on fiber/resin type and exposure conditions, with values ranging from 0.5 to 0.95, according to recommendations given in [10], but with further analysis of material properties and environmental conditions are advised.

Continuous stress or cyclic loading cause degradation of mechanical properties of FRP materials due to creep, relaxation, and fatigue. This is taken into account through conversion factor η_{l} depending on fiber/resin type and loading mode, with values ranging from 0.5 to 0.8, according to [10].

2.2. DESIGN ASSUMPTIONS

Design recommendations are based on limit states. For ultimate limit states analysis, there are two possible approaches depending on the type of structural analysis: nonlinear or linear. If linear elastic models or simplified schemes are used, which consider balanced distribution of stresses that satisfy equilibrium conditions, but not necessarily compatibility of strains, the resulting stress on each structural member should be verified. With the assumption that a plane section before loading remains plane after loading, the design criteria is met when design shear forces and bending moments due to the applied loads are less than the corresponding design shear and flexural capacities.

Masonry exhibits brittle behaviour when subjected to tensile loading, hence the corresponding tensile strength is negligible compared to its compressive strength. For design purposes, it is acceptable to neglect the tensile strength of masonry. In most cases stress-strain relationship for masonry under compression uniaxial loading can be considered as parabolic up to design strength of f_d, and design strain of ε_m1. Design strength is equal to f_md for strains between $\varepsilon_\mathrm{m1} \leq \varepsilon \leq \varepsilon_\mathrm{mu}$ and it is zero for strains larger than ε_mu. Unless experimental data is available, masonry ultimate design strain is equal to 3.5‰. Masonry shear strength is important when considering wall subjected to lateral forces and it depends on the applied axial load.

In order to enhance flexural and/or shear capacity of masonry walls using FRP reinforcement, FRP materials has to be always in tension. FRP materials have a linear elastic behaviour up to the point of failure. FRP stress should be determined as a product of modulus of elasticity and calculated strain. Maximum design strain for FRP is calculated as follows:

$$\varepsilon_\mathrm{fd} = \min\left\{\eta_\mathrm{a}\frac{\varepsilon_\mathrm{fk}}{\gamma_\mathrm{f}};\varepsilon_\mathrm{fdd}\right\} \tag{4}$$

where: ε_fk - FRP characteristic strain at failure; ε_fdd - maximum FRP strain due to intermediate debonding; η_a - environmental conversion factor, γ_f - partial factor for FRP.

2.3. DEBONDING STRENGTH

The bond between FRP reinforcements and structural masonry elements is of great importance, as debonding (loss of adhesion) can lead to a brittle failure. If tensile strength of the adhesive used to install FRP reinforcement is larger than that of the substrate, debonding can occur at the interface between the adhesive and masonry.

Two failure modes are considered if FRP is applied in isolated straight strips: plate/sheet end debonding or intermediate crack debonding. Shear debonding is characteristic for FRP anchorage sections and it can cause rip-off failure, that is, removal of a significant portion of brick, usually when shear stresses are combined with normal tensile stresses. Bond capacity can be increased by using mechanical anchorage devices.

Optimal bond length, which corresponds to the minimal bond length able to carry maximum anchorage force, can be estimated as:

$$l_\mathrm{ed} = \max\left\{\frac{1}{\gamma_\mathrm{Rd}\cdot f_\mathrm{bd}}\sqrt{\frac{\pi^2\cdot E_\mathrm{f}\cdot t_\mathrm{f}\cdot \Gamma_\mathrm{Fd}}{2}};150\mathrm{mm}\right\} \tag{5}$$

where: E_f - FRP modulus of elasticity in the fibre direction; t_f - FRP thickness; Γ_{Fd} - design value of specific fracture energy; γ_{Rd} - corrective factor ($\gamma_{Rd} = 1.5$ for brick masonry [10]); f_{bd} - design bond strength between FRP and masonry:

$$f_{bd} = \frac{2 \cdot \Gamma_{Fd}}{s_u} \tag{6}$$

If experimental data are unavailable, value of interface slip at full debonding s_u is equal to 0.4 mm for brick masonry [10].

Design value of the specific fracture energy is calculated as:

$$\Gamma_{Fd} = \frac{k_b \cdot k_G}{FC} \sqrt{f_b \cdot f_{bt}} \tag{7}$$

where: k_b - geometrical corrective factor; k_G - corrective factor depending on the type of masonry ($k_G = 0.031$ mm for brick masonry and wet lay-up FRP systems [10]); FC - confidence factor; f_b and f_{bt} - average compressive and tensile strength of masonry units, respectively (it can be taken that $f_{bt} = 0.10 \cdot f_b$). For pre-cured FRP systems, values of k_G used for wet lay-up FRP systems should be reduced to 40%.

If experimental data are unavailable, k_b can be calculated as:

$$k_b = \sqrt{\frac{3 - b_f / b}{1 + b_f / b}} \tag{8}$$

where: b - the strengthened element width; b_f - FRP width.

The value of b can be calculated as a sum of FRP width b_f and width of bond strength distribution area b_d (Figure 2).

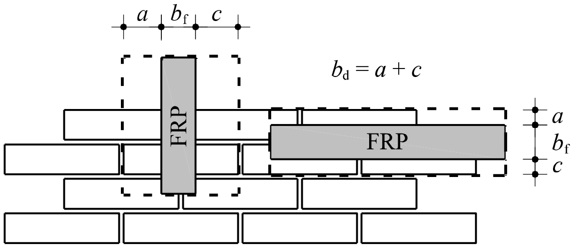


Figure 2 – Bond strength distribution for regular shaped masonry units

When first masonry layers are included in debonding and the bond length is longer or equal to the optimal bond length, design debonding strength is equal to:

$$f_{fdd} = \frac{1}{\gamma_{f,d}} \sqrt{\frac{2 \cdot E_f \cdot \Gamma_{Fd}}{t_f}} \tag{9}$$

where $\gamma_{f,d}$ is partial factor of FRP materials for debonding. In case of masonry walls with mortar joint distance smaller than the optimal bonding length, design debonding strength in Eq (9) should be reduced to 85% of its value.

Intermediate debonding strength can be determined by limiting debonding strength of FRP to the design value:

$$f_{\mathrm{fdd,2}} = \alpha \cdot f_{\mathrm{fdd}} \quad \left(1{,}0 \leq \alpha \leq 2{,}0\right) \tag{10}$$

Maximum FRP strain due to intermediate debonding is calculated as:

$$\varepsilon_{\mathrm{fdd}} = \frac{f_{\mathrm{fdd,2}}}{E_{\mathrm{f}}} \tag{11}$$

2.4. DESIGN PROCEDURES FOR FRP STRENGTHENED MASONRY WALLS

FRP strengthening is employed in order to increase load-carrying capacity and ductility of masonry walls for in-plane or out-of-plane loading.

In-plane failure of masonry walls is one of the most frequent types of failure for masonry structural members. This failure mode is primarily a consequence of seismic actions. Masonry walls subjected to in-plane loading should be analysed for:

- in-plane combined bending and axial load;
- shear force.

2.4.1. In-plane combined bending and axial load

Vertical FRP reinforcement, symmetrically and adequately bonded on both wall sides, can be employed to enhance combined bending and axial load-carrying capacity. This FRP reinforcement should be anchored to the foundation and floor by mechanical anchorage devices.

FRP strengthened members subjected to combined bending and axial loading should be designed as follows:

$$M_{\mathrm{Ed}} \leq M_{\mathrm{Rd}}\left(N_{\mathrm{Ed}}\right) \tag{12}$$

where M_{Ed} - design moment; M_{Rd} - flexural capacity of the strengthened member considering design axial force N_{Ed}.

The analysis of FRP strengthened members can be performed using strain compatibility and force equilibrium methods. Two types of failure mechanisms can be observed, depending on whether CFRP maximum tensile strain $\varepsilon_{\mathrm{fd}}$ or masonry maximum compressive strain $\varepsilon_{\mathrm{mu}}$ is reached. The maximum masonry and FRP stain should be calculated as explained in section 2.2. Compressive stress-strain relationship for masonry is assumed to be rectangular with a uniform compressive stress of f_{d}, distributed over an equivalent compression zone, which may be taken with a depth of 0.8 times the actual compression zone.

2.4.2. Shear force

Shear capacity of masonry walls can be enhanced by adding FRP reinforcements to both sides with fibres parallel to shear direction. In the vertical direction, FRP should be placed in order to ensure a formation of the truss mechanism as an element in tension.

When formation of truss mechanism is ensured, design shear capacity V_{Rd} of the FRP strengthened masonry wall is calculated as a sum of masonry contribution $V_{\mathrm{Rd,m}}$ and FRP contribution $V_{\mathrm{Rd,f}}$ up to the maximum value of $V_{\mathrm{Rd,max}}$, inducing failure of the compressed strut of the truss:

$$V_{\mathrm{Rd}} = \min\left\{V_{\mathrm{Rd,m}} + V_{\mathrm{Rd,f}}; V_{\mathrm{Rd,max}}\right\} \tag{13}$$

with

$$V_{\mathrm{Rd,m}} = x \cdot t \cdot f_{\mathrm{vd}} \tag{14}$$

$$V_{\mathrm{Rd,f}} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot 0.6 \cdot d \cdot \left(E_{\mathrm{f}} \cdot \varepsilon_{\mathrm{fd}}\right) \cdot 2 \cdot t_{\mathrm{f}} \cdot \frac{b_{\mathrm{f}}}{p_{\mathrm{f}}} \tag{15}$$

$$V_{\mathrm{Rd,max}} = 0.3 \cdot f_{\mathrm{d,h}} \cdot t \cdot d \tag{16}$$

where: x - distance from the neutral axis to the extreme compression fibre; t - masonry wall thickness; d - distance between compression side of masonry and centroid of FRP flexural reinforcement; f_{vd} - design shear strength of masonry; $f_{\mathrm{d,h}}$ - design compressive strength of masonry parallel to mortar joints; E_{f} - FRP modulus of elasticity in the fibre direction, $\varepsilon_{\mathrm{fd}}$ - FRP ultimate strain, t_{f} - FRP thickness; b_{f} - FRP width; p_{f} - FRP spacing measured orthogonally to fibre direction; γ_{Rd} - partial factor for shear.

If angle of friction of mortar joints φ is less than 45°, the value of $V_{\mathrm{Rd,f}}$ should be reduced by factor equal to cotg(90°-φ). The angle φ can be obtained from compressive and tensile strength of mortar.

Masonry walls can also be shear strengthened using a couple of diagonal FRP applied symmetrically to both sides of the wall.

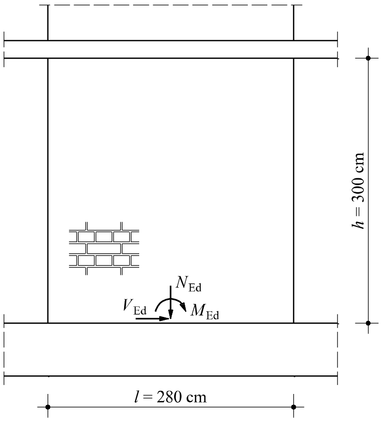
3. EXAMPLE OF FRP STRENGTHENED MASONRY DESIGN

In this chapter, numerical examples of FRP strengthening of masonry wall are provided. It is assumed that FRP strengthening is necessary to ensure earthquake resistance.

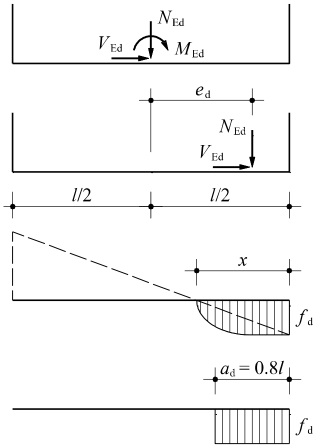
Masonry wall is made from solid clay bricks and general purpose mortar. Wet lay-up CFRP with monodirectional fibres is used for reinforcement. Wall loading (seismic design situations): $N_{\mathrm{Ed}} = 240.3$ kN, $M_{\mathrm{Ed}} = 361.8$ kNm, $V_{\mathrm{Ed}} = 142.2$ kN.

Design data:

Wall dimensions: $l/h/t = 280/300/25$ cm
Compressive strength of masonry units: $f_{\mathrm{b}} = 15$ N/mm^2
Compressive strength of masonry mortar: $f_{\mathrm{m}} = 2.5$ N/mm^2
Characteristic compressive strength of masonry: $f_{\mathrm{k}} = 3.86$ N/mm^2
Characteristic initial shear strength of masonry, under zero compressive stress: $f_{\mathrm{vk0}} = 0.2$ N/mm^2
CFRP thickness: $t_{\mathrm{f}} = 0.165$ mm
CFRP width: $b_{\mathrm{f}} = 100$ mm
CFRP modulus of elasticity: $E_{\mathrm{f}} = 230$ GPa
CFRP ultimate strain: $\varepsilon_{\mathrm{fk}} = 0.0175$



3.1. COMBINED AXIAL AND BENDING MOMENT CAPACITY



Design bending capacity of the wall M_{Rd}, for known design axial force N_{Ed}, is:

$$M_{Rd}(N_{Ed}) = f_d \cdot t \cdot a_{d,\min} \cdot e_{d,\max} = \frac{N_{Ed} \cdot l}{2} \cdot \left(1 - \frac{N_{Ed}}{t \cdot l \cdot f_d}\right)$$

Design value of compressive wall strength:

$$f_d = \frac{f_k}{\gamma_{M,a}} = \frac{3.86}{2.0} = 1.93\ \text{N/mm}^2 = 0.193\ \text{kN/cm}^2$$

Partial factor for material (seismic design situations):
- for units of Category II, any mortar and level of control 5

$$\gamma_{M,a} = 2/3 \cdot \gamma_M = 2/3 \cdot 3.0 = 2.0 > 1.5$$

Design bending capacity of the wall:

$$M_{Rd}(N_{Ed}) = \frac{240.3 \cdot 280}{2} \cdot \left(1 - \frac{240.3}{25 \cdot 280 \cdot 0.193}\right) = 27654\ \text{kNcm}$$

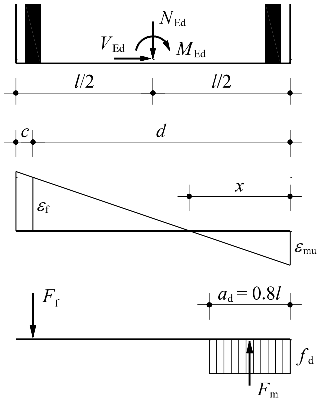
Design condition: $M_{Ed} \le M_{Rd}(N_{Ed})$

$M_{Ed} = 361.8\ \text{kNm} > M_{Rd}(N_{Ed}) = 276.5\ \text{kNm} \rightarrow$ The equation is not satisfied.

Design of FRP for combined axial and bending moment

Vertical CFRP sheets are applied over the entire wall height, on both internal and external side, with a distance of 10 cm from the wall edges. Mechanical anchoring devices are used at the end portions of the masonry wall.

It is assumed that failure mechanism is masonry crushing (strain equal to ε_{mu} = 0.0035). Assuming the linearity of strain distribution, FRP strain value can be calculated as follows:



$$\varepsilon_f = \frac{(l - c - x)}{x} \cdot \varepsilon_{mu} \le \varepsilon_{fd}$$

Design ultimate FRP strain:

$$\varepsilon_{fd} = \eta_a \cdot \frac{\varepsilon_{fk}}{\gamma_f} = 0.95 \cdot \frac{0.0175}{1.1} = 0.0151$$

Position of neutral axis, x, is determined using translational equilibrium equation along the wall length as follows:

$$F_m - F_f - N_{Ed} = 0$$

with $F_m = f_d \cdot t \cdot a_d = f_d \cdot t \cdot 0.8 \cdot x$

$$F_f = \frac{1}{\gamma_{Rd}} \cdot \sigma_f \cdot 2 \cdot A_f = \frac{1}{\gamma_{Rd}} \cdot E_f \cdot \varepsilon_f \cdot 2 \cdot t_f \cdot b_f$$

$$0.193\cdot 25\cdot 0.8\cdot x-\frac{1}{1.0}\cdot 230\cdot 10^{2}\cdot\frac{\left(280-10-x\right)}{x}\cdot 0.0035\cdot 2\cdot 0.0165\cdot 10-240.3=0\ \Rightarrow x=79\ \text{cm}$$

FRP strain:

$$\varepsilon_{\mathrm{f}}=\frac{\left(280-10-79\right)}{79}\cdot 0.0035=0.0085<0.0151$$

Internal forces:

$$F_{\mathrm{m}}=0.193\cdot 25\cdot 0.8\cdot 79=304.5\ \text{kN}$$

$$F_{\mathrm{f}}=2\cdot 230\cdot 10^{2}\cdot\frac{\left(280-10-79\right)}{79}\cdot 0.0035\cdot 0.0165\cdot 10=64.2\ \text{kN}$$

Bending capacity of the strengthened member can be calculated using the following rotational equilibrium equation:

$$M_{\mathrm{Rd}}\left(N_{\mathrm{Ed}}\right)=F_{\mathrm{m}}\cdot\left(\frac{l}{2}-\frac{0.8\cdot x}{2}\right)+F_{\mathrm{f}}\cdot\left(\frac{l}{2}-c\right)$$

$$=304.5\cdot\left(\frac{280}{2}-\frac{0.8\cdot 79}{2}\right)+64.2\cdot\left(\frac{280}{2}-10\right)=41367\ \text{kNcm}$$

Design condition: $M_{\mathrm{Ed}}\leq M_{\mathrm{Rd}}\left(N_{\mathrm{Ed}}\right)$

$$M_{\mathrm{Ed}}=361.8\ \text{KNm}<M_{\mathrm{Rd}}\left(N_{\mathrm{Ed}}\right)=413.7\ \text{kNm}$$

3.2. SHEAR CAPACITY

Design compressive stress perpendicular to the shear direction, based on average vertical stress over compressed part of the wall that is providing shear resistance:

$$\sigma_{\mathrm{d}}'=\frac{N_{\mathrm{Ed}}}{t\cdot x}=\frac{240{,}3}{25\cdot 79}=0.122\ \text{kN/cm}^{2}=1.12\ \text{N/mm}^{2}$$

Characteristic shear strength of masonry:

$$f_{\mathrm{vk}}=\min\left\{\begin{array}{l}f_{\mathrm{vk0}}+0.4\cdot\sigma_{\mathrm{d}}'=0.2+0.4\cdot 1.12=0.687\ \text{N/mm}^{2}\\ 0.065\cdot f_{\mathrm{b}}=0.065\cdot 15=0.975\ \text{N/mm}^{2}\end{array}\right\}=0.687\ \text{N/mm}^{2}$$

Design shear strength of masonry:

$$f_{\mathrm{vd}}=\frac{f_{\mathrm{vk}}}{\gamma_{\mathrm{M,a}}}=\frac{0.678}{2.0}=0.339\ \text{N/mm}^{2}=0.034\ \text{kN/cm}^{2}$$

Design shear resistance:

$$V_{\mathrm{Rd}}=f_{\mathrm{vd}}\cdot t\cdot x=0.034\cdot 25\cdot 79=67.8\ \text{kN}$$

Design condition: $V_{\mathrm{Ed}}\leq V_{\mathrm{Rd}}$

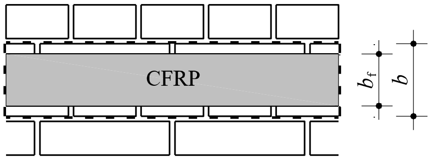
$V_{\mathrm{Ed}}=124.2\ \text{kN}>V_{\mathrm{Rd}}=67.58\ \text{kN}\rightarrow$ The equation is not satisfied.

Design of FRP for shear

Horizontal CFRP sheets are applied, throughout the entire height of the wall, symmetrically to both sides, with a distance between sheets equal to 50 cm.

Width of the strengthened element:

$b = 2 \cdot 65 + 10 = 140$ mm



Geometrical corrective factor:

$$k_b = \sqrt{\frac{3 - b_f / b}{1 + b_f / b}} = \sqrt{\frac{3 - 100/140}{1 + 100/140}} = 1.155$$

Corrective factor dependent on the type of masonry: $k_G = 0.031$ mm

Factor of confidence (level of knowledge 3): $FC = 1.0$

Design value of the specific fracture energy:

$$\Gamma_{Fd} = \frac{k_b \cdot k_G}{FC} \sqrt{f_b \cdot f_{bt}} = \frac{1.155 \cdot 0.031}{1.0} \cdot \sqrt{15 \cdot 1.5} = 0.170 \text{ N/mm}$$

Interface slip at full debonding: $s_u = 0.4$ mm

Design bond strength between FRP and masonry:

$$f_{bd} = \frac{2 \cdot \Gamma_{Fd}}{s_u} = \frac{2 \cdot 0.170}{0.4} = 0.85 \text{ N/mm}^2$$

The optimal bond length:

$$l_{ed} = \frac{1}{\gamma_{Rd} \cdot f_{bd}} \sqrt{\frac{\pi^2 \cdot E_f \cdot t_f \cdot \Gamma_{Fd}}{2}} = \frac{1}{1.5 \cdot 0.85} \sqrt{\frac{3.14^2 \cdot 230 \cdot 10^3 \cdot 0.165 \cdot 0.170}{2}} = 140 \text{ mm}$$

$\min l_{ed} = 150 \text{ mm} \rightarrow l_{ed} = 150$ mm

Design debonding strength of FRP (mortar joints distance less than l_{ed}):

$$f_{fdd} = \left(\frac{1}{\gamma_{f,d}} \sqrt{\frac{2 \cdot E_f \cdot \Gamma_{Fd}}{t_f}} \right) \cdot 0.85 = \left(\frac{1}{1.2} \sqrt{\frac{2 \cdot 230 \cdot 10^3 \cdot 0.170}{0.165}} \right) \cdot 0.85 = 487.3 \text{ N/mm}^2$$

Design debonding strength of FRP (intermediate debonding):

$f_{fdd,2} = 2 \cdot f_{fdd} = 2 \cdot 487.3 = 974.6 \text{ N/mm}^2$

Maximum strain of FRP before debonding:

$$\varepsilon_{fdd} = \frac{f_{fdd,2}}{E_f} = \frac{974.6}{230 \cdot 10^3} = 0.0042$$

Maximum design strain allowed to the FRP system:

$$\varepsilon_{fd} = \min \left\{ \eta_a \cdot \frac{\varepsilon_{fk}}{\gamma_f} ; \varepsilon_{fdd} \right\} = \min\{0.0151;\ 0.0042\} = 0.0042$$

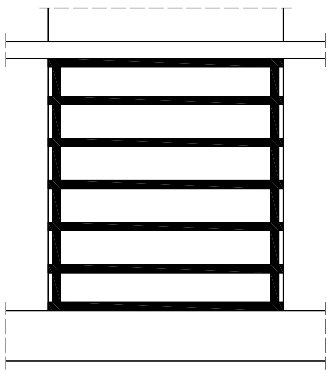
Masonry contribution to shear capacity:

$$V_{\mathrm{Rd,m}} = f_{\mathrm{vd}} \cdot t \cdot x = 0.034 \cdot 25 \cdot 79 = 67.8\ \mathrm{kN}$$

FRP contribution to shear capacity:

$$V_{\mathrm{Rd,f}} = \frac{0,6 \cdot d \cdot E_{\mathrm{f}} \cdot \varepsilon_{\mathrm{fd}} \cdot 2 \cdot t_{\mathrm{f}} \cdot b_{\mathrm{f}}}{\gamma_{\mathrm{Rd}} \cdot p_{\mathrm{f}}} =$$

$$= \frac{0.6 \cdot (280-10) \cdot 230 \cdot 10^2 \cdot 0.0042 \cdot 2 \cdot 0.0165 \cdot 10}{1.2 \cdot 50} = 86.8\ \mathrm{kN}$$



Design compressive strength of the masonry parallel to the mortar joints:

$$f_{\mathrm{d,h}} \approx 0.5 \cdot f_{\mathrm{d}} = 0.5 \cdot 0.193 = 0.096\ \mathrm{kN/cm}^2$$

Maximum value of shear capacity of the FRP strengthened masonry wall:

$$V_{\mathrm{Rd,max}} = 0.3 \cdot f_{\mathrm{d,h}} \cdot t \cdot d = 0.3 \cdot 0.096 \cdot 25 \cdot (280-10) = 195.2\ \mathrm{kN}$$

Design shear capacity of the strengthened masonry:

$$V_{\mathrm{Rd}} = \min\left\{V_{\mathrm{Rd,m}} + V_{\mathrm{Rd,f}}; V_{\mathrm{Rd,m}}\right\} = \min\left\{67.8 + 86.8 = 154.6\ \mathrm{kN};\ 192.5\ \mathrm{kN}\right\} = 154.6\ \mathrm{kN}$$

Design condition: $V_{\mathrm{Ed}} \leq V_{\mathrm{Rd}}$

$$V_{\mathrm{Ed}} = 124.2\ \mathrm{kN} < V_{\mathrm{Rd}} = 154.6\ \mathrm{kN}$$

4. CONCLUSION

As masonry represents heterogeneous and anisotropic material with structural behaviour governed by mechanical properties of its components (units and mortar) and bond between them, failure modes in masonry are usually very complex and pronounced. FRP materials are successfully employed for masonry strengthening and repair. This reinforcement should be designed to meet serviceability, durability and strength requirements.

Although FRP strengthening is not a novel technique, there is a lack of standardized design procedures. In this paper, guidelines given in the document for the design and construction of externally bonded FRP systems for strengthening existing structures, within the framework of the Italian regulations, are presented. The most problematic part of this design procedure is the assessment of the bond quality between FRP reinforcement and masonry (maximum FRP strain before debonding). The given equations were calibrated based on a limited database of experimental results. However, for the application of this strengthening technique in everyday engineering practice, it is necessary to conduct more extensive experimental and theoretical research in order to define equations that would describe the behaviour FRP strengthened masonry as realistically as possible, while being easily applicable.

REFERENCES

[1] Hartley, A., Mullins, G., Sen, R.: Repair of concrete masonry block walls using carbon fiber, In: Proceedings of the Second International Conference on Advanced Composite

Materials in Bridges and Structures, Canadian Society of Civil Engineering, Ottawa, 795–802, 1996.

[2] Hamilton III H.R., Dolan C.W.: Flexural Capacity of Glass FRP Strengthened Concrete Masonry Walls, Journal of Composites for Construction, 5(3): 170–178, 2001.

[3] Yousuf A., Tarek A.: Load capacity of concrete masonry block walls strengthened with epoxy-bonded GFRP sheets, Journal of Composite Materials, 39(19):1719–44, 2005.

[4] Ayman M.: Out-of-plane flexural behaviour of unreinforced red brick walls strengthened with FRP composites, Composites: Part B 38: 559–574, 2007.

[5] Valluzzi M.R., Tinazzi D., Modena C.: Shear behaviour of masonry panels strengthened by FRP laminates, Construction and Building Materials, 16(7):409–16, 2002.

[6] Motavalli M.: Seismic Assessment and Retrofitting of Shariati Museum in Tehran, FRP Strengthening of Masonry, Fibre Composites, FS07, EMPA Switzerland, University of Tehran, Iran, 2005.

[7] Marcari G., Manfredi G., Prota A., Pecce M.: In-plane shear performance of masonry panels strengthened with FRP, Composites Part B: Engineering. 38(7-8):887-901, 2007.

[8] Nanni A., Tumialan G.: Strengthening of Masonry Walls with FRP Bars, Composites Fabricator Magazine, March, 2002, Arlington, VA.

[9] Turco V., Secondin S., Morbin A., Valluzzi M.R., Modena C.: Flexural and shear strengthening of un-reinforced masonry with FRP bars, Composites Science and Technology 66:289–296, 2006.

[10] CNR-DT 200 R1/2013: Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures - Materials, RC and PC structures, masonry structures, Italy: National research council - Advisory Committee on Technical Recommendations for Construction, 2014.