Design Guidelines for Masonry Structures: Out of Plane Loads

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Synopsis: Unreinforced masonry (URM) walls are prone to failure when subjected to out-of-plane loads caused by seismic loads or high wind pressure. Fiber Reinforced Polymers (FRP) in the form of laminates or grids adhesively bonded to the masonry surface with epoxy or polyurea based resins; or FRP bars used as Near Surface Mounted (NSM) reinforcement bonded to the masonry using epoxy or latex modified cementitious pastes, have been successfully used to increase flexural and/or shear capacity of URM walls. However, the practical application of FRPs to strengthen masonry structures is only limited to few research projects due to the limited presence of specific design guidelines. This paper describes provisional design guidelines for the FRP strengthening of masonry walls subject to out of plane loads. The proposed design methodology offers a first rational attempt for consideration by engineers interested in out-of-plane upgrade of masonry walls with externally bonded FRP systems.

Keywords: bar shapes; design; epoxy- or cementitious-based paste; FRP grids; FRP laminates; masonry; NSM FRP bars; out-of-plane; polyurea

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RESEARCH SIGNIFICANCE

A design methodology for the FRP strengthening of un-reinforced masonry walls subject to out of plane loads is presented. Non-Bearing and bearing walls are studied taking into account the influence of the boundary conditions. Different types of FRP strengthening are investigated: Glass Grid Reinforced Polyurea (GGRP), FRP laminates, and Near Surface Mounted (NSM) FRP bars.

INTRODUCTION

Unreinforced masonry (URM) walls are prone to failure when subjected to overstress caused by out-of-plane and in-plane loads. Externally bonded FRP laminates have been successfully used to increase the flexural and/or the shear capacity of the strengthening of unreinforced masonry (URM) walls subjected to overstresses (Schwegler and Kelterborn, 1996; Hamilton and Dolan, 2001; Tumialan et al., 2001). The effectiveness of Near Surface Mounted (NSM) FRP bars to increase both strength and ductility of URM walls subject to out-of-plane loads was proven by several researchers (Hamid, 1996; Galati et al., 2004). A field application on flexural strengthening with NSM FRP bars of cracked URM walls in an educational facility in Kansas City - Missouri, showed effectiveness and practicality of such technique (Tumialan et al., 2003). Glass Grid Reinforced Polyurea (GGRP) was successfully used by Yu et al., (2004), to strengthen URM walls subject to both, out-of-plane and in-plane loads.

This paper presents design guidelines for the strengthening of masonry structures strengthened either with Glass Grid Reinforced Polyurea (GGRP), FRP laminates, and Near Surface Mounted (NSM) FRP bars. Such design guidelines are based upon the present available literature and design codes. Both, non-bearing and bearing walls will be presented as well as strength limitations due to arching action.

The design procedures presented in this paper are based on principles of equilibrium and compatibility and the constitutive laws of the materials, and they written in the form of a design guideline in order to facilitate its immediate use.

DESIGN PHILOSOPHY

Strength design methodology

The design of FRP reinforcement for out-of-plane and in-plane loads is based on limit state principles. The design process for masonry walls requires investigating several possible failure modes and limit states (CNR-DT 200, 2004).

In this paper the strength design approach of reinforced masonry members is adopted, to assure consistency with the Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/ TMS 402-02) and with other ACI document on masonry (ACI 530.1-02/ACSE 6-02/TMS 602-02 "Specification for Masonry Structures', ACI 530-02/ASCE 5-02/TMS 402-02 "Commentary on Building Code Requirement for Masonry Structures", ACI 530.1-02/ASCE 6-02/TMS 602-02 "Commentary on Specification for Masonry Structures").

The strength reduction factors given in Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/ TMS 402-02) together with the load factors given in ASCE 7-98 "Minimum Design Loads for Building and Other Structures" are used, unless otherwise noted.

DESIGN MATERIAL PROPERTIES

The materials considered in this paper are the masonry and the FRP system. The masonry material properties should be obtained from the Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/ TMS 402-02) or equivalent codes or as provided by the producers. For the FRP system, the materials properties are those provided by the manufacturers.

FRP Design Material Properties

The FRP material is considered linear elastic up to failure. The material properties guaranteed by the manufacturer should be considered as initial values that do not include the effects of long-term exposure to the environment. Because long-term exposure to various environments can reduce the tensile strength and creep rupture and fatigue endurance of the FRP system, the material properties used in design equations should be reduced based on the type and level of environment and loads exposure.

Equations (1) to (2) give the tensile properties that should be used for the design, taking into account the environment exposure. The design strength, f_{fu}, should be determined, according to ACI 440.2R-02 as:

$$f_{fu} = C_E f_{fu}^* \tag{1}$$

where: C_E is the environment reduction factor summarized in Table 1, and f_{fu}^* is the guaranteed tensile strength of FRP provided by the manufacturer. The design rupture strain should be determined as:

$$\varepsilon_{fu} = C_E \varepsilon_{fu}^* \tag{2}$$

where ε_{fu}^* is the guaranteed rupture strain of the FRP system. The design modulus of elasticity is assumed to be the same as the value reported by the manufacturer: $E_f = E_{f,ave}$.

Reduction for debonding at ultimate -- FRP debonding can occur if the force in the FRP cannot be sustained by the interface of the substrate. In order to prevent debonding of the FRP, a limitation should be placed on the strain level developed in the laminate. The debonding of FRP in flexure or shear is accounted through a parameter k_m. The effective design strength and strain, f_{fe} and ε_{fe}, of the FRP should be considered as:

$$f_{fe} = k_m f_{fu} = k_m C_E f_{fu}^* \tag{3}$$

$$\varepsilon_{fe} = k_m \varepsilon_{fu} = k_m C_E \varepsilon_{fu}^* \tag{4}$$

Table 2 summarizes values for k_m based on test results on un-reinforced masonry (URM) walls strengthened with GGRP, FRP laminates and NSM FRP bars (Tumialan et al., 2003-a, Galati et al.2004). It should be noted that in the case of GGRP it is reasonable to conservatively assume k_m = 0.65 as for the case of FRP laminates applied on puttied masonry.

Reductions for creep rupture at service -- Walls subjected to sustained load such as retaining or basement walls, creep rupture considerations need to be taken into account (ACI 440.2R-02, 2002). In such cases, for serviceability check, the designed admissible tensile stress, $f_{f,s}$, should not exceed the values presented in Table 3.

Masonry

Most masonry materials exhibit nonlinear behavior in compression, and a negligible tensile strength disregarded in the present guideline. The stress distribution for the part of masonry in compression should be determined from an appropriate nonlinear stress-strain relationship or by a rectangular stress block suitable for the given level of strain in the masonry. The stress block has dimensions $\gamma f_m'$ and γd. Expressions for β_1 and γ are given in equation (5) (Tumialan et al., 2003-a).

$$\beta_1 = 2 - \frac{4\left[\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right) - \tan^{-1}\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right)\right]}{\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right)\ln\left(1+\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right)^2\right)} \text{ and } \gamma = 0.90\frac{\ln\left(1+\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right)^2\right)}{\beta_1\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right)} \tag{5}$$

where $\varepsilon_m^{'} = \frac{1.71 f_m^{'}}{E_m}$ and $\tan^{-1}\left(\frac{\varepsilon_m}{\varepsilon_m^{'}}\right)$ is computed in radians. The strength and the modulus of elasticity of the masonry can be computed as recommended in the Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/ TMS 402-02) as $E_m = 700 f_m^{'}$, for clay masonry and $E_m = 900 f_m^{'}$, for concrete masonry. The maximum usable strain, ε_{mu}, at the extreme compressive side is assumed to be 0.0035 (in./in.) for clay masonry and 0.0025 (in./in) for concrete masonry. When masonry crushing failure occurs the parameters β_1 and γ can assume the values shown in Table 4.

DESIGN PROCEDURE

General considerations

The failure of masonry panels for out-of-plane loads could be due by earth pressure, seismic loads, dynamic vibrations, verticality flaw, wind pressure, and by arch thrust (CNR DT, 2004, Tumialan, 2003-a).

The failure modes of URM walls strengthened with FRP systems and subject to out-of-plane loads can be summarized as follow:

- FRP debonding: due to shear transfer mechanisms at the interface masonry/FRP, debonding may occur before flexural failure. Debonding starts from flexural cracks at the maximum bending moment region and develops towards the support. Since the tensile strength of masonry is lower than that of the resin, the failure typically occurs in the masonry for walls strengthened with FRP laminates or GGRP (Tumialan, 2003-a, Hamilton, 2001).

  In the case of NSM FRP strengthening, since after cracking the tensile stresses at the mortar joints are taken by the FRP reinforcement, cracks can develop in the masonry units oriented at 45^{o} or in the head mortar joints. Some of these cracks follow the embedding paste and masonry interface causing debonding and subsequent wall failure (Galati et al., 2004). In the case of smooth rectangular NSM FRP bars, the failure mode can be due to the sliding of the bar inside the epoxy (Galati et al., 2004). Finally, if deep grooves are used, debonding can also be caused by splitting of the embedding material (Galati et al., 2004).

- Flexural failure: after developing flexural cracks primarily located at the mortar joints, a failure can occur either by rupture of the FRP reinforcement or masonry crushing (Tumialan, 2003-a, Tumialan, 2003-b). Typically, flexural failure of masonry strengthened with FRPs is due to compressive crushing in walls strongly strengthened. FRP rupture is less desirable than masonry crushing being that the latter more ductile (Triantafillou, 1998). Both failure modes are acceptable in governing the design of out-of-plane loaded walls strengthened with FRP systems provided that strength and serviceability criteria are satisfied.

- Shear failure: cracking starts with the development of fine vertical cracks at the maximum bending region. Thereafter, two types of shear failure could be observed: flexural-shear or sliding shear. The first type is oriented at approximately 45°, and the second type occurs along bed joint, near the support, causing sliding of the wall at that location. The crack due to flexural-shear mode cause a differential displacement in the shear plane, which often results in FRP debonding (Tumialan, 2003-a, Hamoush, 2002).

The recommendations given in this section are only for members of rectangular cross-sections with strengthening applied to one side, as the experimental work has almost exclusively considered members with this shape and the FRP strengthening assumed to work only in tension, not in compression.

General Assumptions

The following assumptions and limitations should be adopted:

- The strains in the reinforcement and masonry are directly proportional to the distance from the neutral axis, that is, a plane section before loading remains plane after loading.
- The tensile strength of masonry is neglected.
- There is no relative slip between external FRP reinforcement and the masonry, until debonding failure.
- The wall can be assumed to behave under simply supported conditions (i.e. arching mechanism is not present).

The FRP design strength is adjusted for the effects of environmental exposure by means of the coefficient C_E as defined in ACI440.2R-02, and for the effects of debonding by the parameter k_m.

Flexural behavior of non-load bearing walls

The ultimate strength design criterion states that the design flexural capacity of a member must exceed the flexural demand (Eq. (6)).

$$\phi M_n \geq M_u \tag{6}$$

where ϕ is the strength reduction, which should be taken as 0.7 (Tumialan et al. 2003-a), and M_n and M_u represent the nominal and ultimate moment, respectively.

Computations are based on force equilibrium and strain compatibility. The distribution of strain and stress in the FRP reinforced masonry for a rectangular cross-section under out-of-plane load is shown in Figure 1, where the value of the stress block parameters γ and β_1 associated with a parabolic compressive stress distribution are given in Eq. (5).

The general equations to evaluate the nominal moment capacity, M_n, for a strip of masonry are given as:

$$\left(\gamma f_m^{'}\right)\left(\beta_1 c\right) b = A_f f_f \tag{7}$$

$$M_n = \left(\gamma f_m^{'}\right)\left(\beta_1 c\right) b \left(d - \frac{\beta_1 c}{2}\right) \tag{8}$$

$$\frac{\varepsilon_m}{c} = \frac{\varepsilon_f}{d-c} = \frac{\varepsilon_m + \varepsilon_f}{d} \tag{9}$$

where d is the distance from extreme compression fiber to the centroid of the tension reinforcement.

Failure mode -- The flexural capacity of FRP strengthened masonry subject to out-of-plane loads is dependent on whether the failure is governed by masonry crushing or FRP debonding or rupture. The failure mode can be determined by comparing the FRP reinforcement ratio for a strip of masonry to the balanced reinforcement ratio, defined as the ratio where masonry crushing and FRP debonding or rupture occurs simultaneously. The FRP reinforcement ratio for a strip of masonry is computed as:

$$\rho_f = \frac{A_f}{b\,t} \tag{10}$$

then, according to equilibrium and compatibility, the balanced reinforcement ratio is:

$$\rho_{fb} = \gamma \beta_1 \frac{f_m^{'}}{f_{fe}} \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{fe}} = \gamma \beta_1 \frac{f_m^{'}}{f_{fe}} \frac{E_f \varepsilon_{mu}}{E_f \varepsilon_{mu} + f_{fe}} \tag{11}$$

If the reinforcement ratio is below the balanced ratio ($\rho_f < \rho_{fb}$), FRP rupture or debonding failure mode governs. Otherwise, when $\rho_f > \rho_{fb}$, masonry crushing governs.

Nominal flexural capacity -- *Masonry crushing failure*: When $\rho_f > \rho_{fb}$, the failure is initiated by crushing of the masonry, and the stress distribution in the masonry can be approximated with a rectangular stress block defined by the parameters β_1 and γ that in this case assume the values shown in Table 4.

According to 440.1R-03 and based on the equations (7) to (9), the following equations can be derived:

$$M_n = \left(\gamma f_m^{'}\right) a\, b \left(d - \frac{a}{2}\right) = A_f f_f \left(d - \frac{a}{2}\right) \tag{12}$$

$$a = \beta_1 c = \frac{A_f f_f}{\gamma f_m^{'} b} \tag{13}$$

$$f_f = E_f \varepsilon_{mu} \frac{\beta_1 d - a}{a} \tag{14}$$

Substituting a from Eq. (13) into Eq. (14) and solving for f_f gives:

$$f_f = \left(\sqrt{\left(\frac{E_f \varepsilon_{mu}}{2} \right)^2 + \frac{\gamma \beta_1 f_m^{'}}{\rho_f} \frac{d}{t} E_f \varepsilon_{mu}} - \frac{E_f \varepsilon_{mu}}{2} \right) \leq f_{fe} \tag{15}$$

The nominal flexural strength can be determined from Eq. (12), (13) and (14). Based on compatibility, the stress level in the FRP can be found from Eq. (15), and needs to be less or equal to f_{fe}.

FRP debonding or rupture: When $\rho_f < \rho_{fb}$, the failure of the wall is initiated by rupture or debonding of the FRP, and the equivalent stress block depends on the maximum strain reached by the masonry. In this case, an iterative process should be used to determine the equivalent stress block. The analysis incorporates four unknowns: the masonry compressive strain at the failure ε_m, the depth to the neutral axis c, and the parameters β_1 and γ.

Once the value of the four parameters have been found, the flexural capacity can be computed as shown in Eq. (16):

$$M_n = A_f f_{fe} \left(d - \frac{\beta_1 c}{2} \right) \tag{16}$$

For this type of failure, the upper limit of the product $\beta_1 c$ for balanced conditions is equal to $\beta_1 c_b$. Therefore, a simplified and conservative calculation of the nominal flexural capacity of the member can be based on Eq. (17) and (18):

$$M_n = A_f f_{fe} \left(d - \frac{\beta_1 c_b}{2} \right) \tag{17}$$

$$c_b = \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{fe}} \right) d \tag{18}$$

Flexural behavior of load bearing walls

The ultimate strength design criterion states the design capacity of a member subject to flexural and axial load should be:

$$\frac{P_u}{\phi P_n} + \frac{M_u}{\phi M_n} \leq 1 \tag{19}$$

Again, ϕ is conservatively assumed to be 0.7 for flexure and/or axial loads. Computations are based on force equilibrium and strain compatibility. The geometry of the un-cracked cross-section is given in Figure 2. The distribution of strain and stress in the FRP reinforced masonry for a rectangular section under out-of-plane and axial loads are shown in Figure 1.

The nominal axial strength, P_n, for the masonry strip of width b should be evaluated according to the Building Code Requirements for Masonry Structures (ACI

530-02/ASCE 5-02/TMS 402-02), and shall not exceed the values given in Eq. (20) or Eq. (21).

(a) For members having $\frac{h}{r} \leq 99$:

$$P_n = 0.80\left[\gamma f_m^{'} A_n\right]\left[1-\left(\frac{h}{140r}\right)^2\right] \tag{20}$$

(b) For members having $\frac{h}{r} > 99$:

$$P_n = 0.80\left[\gamma f_m^{'} A_n\right]\left(\frac{70r}{h}\right)^2 \tag{21}$$

where, in this paragraph, r is the minimum radius of gyration of the uncracked cross-section of width l (Figure 2), A_n is the net cross-section area of the masonry strip of width b, and h the effective height of wall.

Using the stress distribution for a masonry section subject to flexural and axial load, the general equations of equilibrium and compatibility, written relative to the center of gravity, G, are given as:

$$\left(\gamma f_m^{'}\right)\left(\beta_1 c\right)b - P_u = A_f f_f \tag{22}$$

$$M_n = \left(\gamma f_m^{'}\right)\left(\beta_1 c\right)b\left(t - \frac{\beta_1 c}{2}\right) + A_f f_f\left(d - \frac{t}{2}\right) \tag{23}$$

$$\frac{\varepsilon_m}{c} = \frac{\varepsilon_f}{d-c} = \frac{\varepsilon_m + \varepsilon_f}{d} \tag{24}$$

The moment M_n can be also evaluated relative to the FRP reinforcement (Eq.(25)) or to the center of compression of the masonry (Eq.(26)).

$$M_n = \left(\gamma f_m^{'}\right)\left(\beta_1 c\right)b\left(d - \frac{\beta_1 c}{2}\right) - P_u\left(d - \frac{t}{2}\right) \tag{25}$$

$$M_n = A_f f_f\left(d - \frac{\beta_1 c}{2}\right) + P_u\left(\frac{t}{2} - \frac{\beta_1 c}{2}\right) \tag{26}$$

Failure mode -- The flexural capacity of a FRP load bearing wall is dependent on failure mode. The failure mode can be determined by comparing the FRP reinforcement ratio (Eq. (10)) to the balanced reinforcement ratio Eq. (27).

$$\rho_{fb} = \frac{f_m^{'}}{f_{fe}}\left[\gamma\beta_1 \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_{fe}} - \frac{P_u}{b\,t\,f_m^{'}}\right] = \frac{f_m^{'}}{f_{fe}}\left[\gamma\beta_1 \frac{E_f \varepsilon_{mu}}{E_f \varepsilon_{mu} + f_{fe}} - \frac{P_u}{b\,t\,f_m^{'}}\right] \tag{27}$$

If the reinforcement ratio is below the balanced ratio ($\rho_f < \rho_{fb}$), FRP rupture or debonding failure mode governs. Otherwise, ($\rho_f > \rho_{fb}$) masonry crushing governs.

Nominal flexural capacity -- *Masonry crushing failure:* When $\rho_f > \rho_{fb}$, the failure is initiated by crushing of the masonry, and the stress distribution in the masonry can be approximated with a rectangular stress block defined by the parameters β_1 and γ that in this case assume the values shown in Table 4. Based on equations (20) to (26), the following expressions can be derived:

$$M_n = \left(\gamma f_m'\right) a\, b\left(d-\frac{a}{2}\right) - P_u\left(d-\frac{t}{2}\right) = A_f f_f\left(d-\frac{a}{2}\right) + P_u\left(\frac{t}{2}-\frac{a}{2}\right) \tag{28}$$

$$a = \beta_1 c = \frac{\left(A_f f_f + P_u\right)}{\gamma f_m' b} \tag{29}$$

$$f_f = E_f \varepsilon_{mu}\frac{\beta_1 d - a}{a} \tag{30}$$

Considering equations from (28) to (30), in the case of masonry crushing, the following values for f_f and c can be obtained:

$$f_f = \left(\sqrt{\left(\frac{E_f\varepsilon_{mu}}{2} - \frac{P_u}{2A_f}\right)^2 + \left(\frac{\gamma\beta_1 f_m'}{\rho_f}\frac{d}{t} - \frac{P_u}{A_f}\right)E_f\varepsilon_{mu}} - \left(\frac{E_f\varepsilon_{mu}}{2} + \frac{P_u}{2A_f}\right)\right) \le f_{fe} \tag{31}$$

$$c = \frac{a}{\beta_1} = \frac{\rho_f t}{\beta_1\gamma f_m'}\left[\sqrt{\left(\frac{E_f\varepsilon_{mu}}{2} - \frac{P_u}{2A_f}\right)^2 + \frac{\beta_1\gamma f_m'}{\rho_f}\frac{d}{t}E_f\varepsilon_{mu}} - \left(\frac{E_f\varepsilon_{mu}}{2} - \frac{P_u}{2A_f}\right)\right] \tag{32}$$

FRP debonding or rupture: When $\rho_f < \rho_{fb}$, the failure of the wall is initiated by debonding or rupture of the FRP, and the equivalent stress block depends on the maximum strain reached by the masonry. In this case, an iterative process should be used to determine the equivalent stress block. The analysis incorporates four unknowns given the value of P_u: the masonry compressive strain at failure ε_m, the depth to the neutral axis c, and the parameters γ and β_1. Solving for this system of equations may be laborious.

Alternatively, according to the Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02, section 3.2.2) values of β_1 and γ equal to 0.80 can be assumed. Therefore, the following simplified equations can be used:

$$M_n = \left(0.80 f_m'\right)\left(0.80\,c\right)b\left(d-\frac{0.80\,c}{2}\right) - P_u\left(d-\frac{t}{2}\right) \tag{33}$$

$$c = \frac{\rho_f t}{0.80^2 f_m'}\left[\sqrt{\left(\frac{E_f\varepsilon_{mu}}{2} - \frac{P_u}{2A_f}\right)^2 + \frac{0.80^2 f_m'}{\rho_f}\frac{d}{t}E_f\varepsilon_{mu}} - \left(\frac{E_f\varepsilon_{mu}}{2} - \frac{P_u}{2A_f}\right)\right] \tag{34}$$

$$f_f = E_f\varepsilon_{mu}\frac{d-c}{c} \le f_{fe} \tag{35}$$

Shear Limitations

The nominal moment calculated for flexural behavior should be compared and, if necessary, limited by the one associated with shear failure. In fact, if a large amount of FRP is applied, the failure can be controlled by shear instead of flexure. The theoretical shear capacity of the FRP strengthened masonry should be evaluated according to the Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02). The shear strength capacity should exceed the shear demand, as shown in (36):

$$\phi V_n \geq V_u \tag{36}$$

Due to the fact that the FRP system is only bonded onto the masonry surface, its contribution can be neglected, and the nominal strength becomes:

$$V_n = V_m \tag{37}$$

The shear strength provided by the masonry, V_m, shall be computed using equation (38) for non-load bearing walls, and equation (39) for load bearing walls. The value of $\frac{M}{V\,t}$ need not be taken greater than 1.0.

$$V_m = \left[4.0 - 1.75\left(\frac{M}{V\,t}\right)\right] A_n \sqrt{f_m'} \tag{38}$$

$$V_m = \left[4.0 - 1.75\left(\frac{M}{V\,t}\right)\right] A_n \sqrt{f_m'} + \frac{P}{4} \tag{39}$$

where M is the maximum moment at the section under consideration, V is the corresponding shear force, t the thickness of the masonry, A_n the net cross-section area of the masonry strip of width b, f_m' the specified compressive strength of masonry and P is the axial load.

The nominal shear capacity, V_n, shall not exceed the following limits:

(a) When $\frac{M}{V\,t} \leq 0.25$:

$$V_n \leq 6 A_n \sqrt{f_m'} \tag{40}$$

(b) When $\frac{M}{V\,t} \geq 1.00$

$$V_n \leq 4 A_n \sqrt{f_m'} \tag{41}$$

(c) For values of $\frac{M}{V\,t}$ falling in the range 0.25 to 1.00, a linear interpolation can be used to determine the limiting value of V_n.

STRENGTHENING LIMITATIONS DUE TO ARCHING ACTION

When a wall is built between supports that restrain the outward movement, membrane compressive forces in the plane of the wall, accompanied by shear forces at the supports, are induced as the wall bends. The in-plane compression forces can delay cracking. After cracking, a so-called arching action can be observed. Due to this action, the capacity of the wall can be much larger than that computed assuming simply supported conditions. Experimental works (Tumialan et al., 2003-b, Galati, 2002-a, Galati, 2002-b, Carney, 2003), have shown that the resultant force between the out-of-plane load and the induced membrane force could cause the crushing of the masonry units at the boundary regions.

The arching mechanism must be considered in the quantification of the upgraded wall capacity to avoid overestimating the contribution of the strengthening. Three different modes of failure have been observed in walls exhibiting the arching mechanism:

- Flexural failure (i.e. rupture of the FRP in tension or crushing of the masonry)
- Crushing of masonry at the boundary regions
- Shear failure.

Figure 3 illustrates a comparison between the load-deflection curves obtained in the case of simply supported walls and walls with the end restrains, tested under four point bending (Galati et al., 2002-a). A significant influence of the boundary conditions in the wall behavior is observed. Figure 3 shows that the increase in the ultimate load for walls strengthened with 3 in. and 5 in. wide GFRP laminates were about 175 and 325%, respectively. If the wall is restrained (i.e. arching mechanism is observed) the same effectiveness of the FRP reinforcement is not observed because crushing of the masonry units at the boundary regions controls the wall behavior. In this case, the increase in the out-of-plane capacity for strengthened specimens with 3 and 5 in. wide GFRP laminates was about 25%. It is to be stressed that capacity of an unstrengthened URM wall with end restrains is far superior to that of an identical simply supported wall with FRP strengthening.

Design Procedure

When a non load bearing wall is built solidly between supports capable of resisting an arch thrust with no appreciable deformation or when walls are built continuously past vertical supports (horizontal spanning walls), the lateral load resistance of the wall can benefit from the arching action if height to thickness ratio is less than 20. In such cases, the ultimate strength design criteria states the design ultimate load capacity of a member should be:

$$\phi q_n \geq q_u \tag{42}$$

where $\phi = 0.6$, and q_n and q_u have dimensions kN/m. The design procedure for unstrengthened and strengthened walls is presented herein. The design procedure presented herein allows determining the nominal resisting uniform force, q_n, for both unstrengthened and strengthened URM walls. The resisting force for loading conditions other than the uniform pressure can be derived from q_n.

The resisting force, Q_n, for a concentrated load at mid-height of the wall is given by equation (43):

$$Q_n = \frac{q_n h}{2} \tag{43}$$

where h is the height of the wall. For a triangular distribution, the maximum resisting pressure $\bar{q}_n$ can be determined using the following equation:

$$\bar{q}_n = \frac{q_n}{2} \tag{44}$$

Unstrengthened Masonry Walls -- Analysis may be based on a three-hinge arch, when the bearing of the arch thrust at the supports and at the central hinge should be assumed as 0.1 times the thickness of the wall, as indicated on Figure 4. If chases or recess occur near the thrust-lines of the arch, their effect on the strength of the masonry should be taken into account (Eurocode 6 Sec. 6.3.2).

The arch thrust should be assessed from knowledge of the applied lateral load, the strength of the masonry in compression, the effectiveness of the junction between the wall and the support resisting the thrust, and the elastic and time depending shortening of the wall. The arch thrust may be provided by a vertical load (Eurocode 6 Sec. 6.3.2). The resisting force, q_n, per width b of wall is given by equation (45):

$$q_n = 0.58 f_m' b \left(\frac{t}{h} \right)^2 \tag{45}$$

where b, t and h are the width, thickness and height of the wall, respectively. If the clamping force per width b of the wall, C, is needed, it can be easily computed using equation (46).

$$C = 0.58 f_m' \frac{bt}{10} \tag{46}$$

Strengthened Masonry Walls -- In addition to the general assumptions presented in the first part of the paper, the wall is also assumed cracked at mid-height, and that the two resulting segments can rotate as rigid bodies about the supports. With reference to Figure 5, the resisting force per unit area of wall is given by equation 8.44:

$$q_n = \frac{8}{h^2} \left(\gamma_1 \beta_{11} w_m b_1 f_m' a_c + A_f f_f a_f \right) \tag{47}$$

where h is the height of the wall, A_f is the area of FRP reinforcement, w_m is the width of the wall, γ and β_1 define the stress block. The additional subscripts 1 or 2 for γ and β_1 has been used to single out the corresponding section. Finally, a_f and a_c define the arm of both the force in the FRP and of the clamping force, respectively. For small values of rotation of the wall θ, a_f and a_c can be determined as follows:

$$a_f = d - \frac{\beta_{12} \left(\varepsilon_{m2} \right) b_2}{2} \tag{48}$$

$$a_c = a_f - \frac{\beta_{11} \left(\varepsilon_{m1} \right) b_1}{2} \tag{49}$$

where b_1 and b_2 represent the bearing widths at the supports and at mid-height, respectively. It is important to notice that γ and β_1 are functions of the maximum compressive strain at the considered cross-section (ε_{m1} or ε_{m2}), as expressed in equations 7.5 and 7.6 in Chapter 7.

Equation 8.42, when accounting for equations 8.43 and 8.44, contains five unknowns: ε_{m1}, ε_{m2}, b_1, b_2, and f_f. Such unknowns can be determined using the procedure based on compatibility and equilibrium equations presented herein (Galati, 2002-a; Tumialan et al., 2003-b).

The free-body diagram shown in Figure 5 (b) can be derived analyzing the top segment of the masonry wall depicted in Figure 5 (a). From the equilibrium of forces in the vertical direction, the following relationship can be drawn:

$$C_2 = C_1 + T_f \tag{50}$$

where C_1 and C_2 are the clamping forces at top and mid-height of the wall, respectively, T_f is the force in the FRP strengthening. Considering the stress block distribution, the clamping forces by wall strip width, w_m, acting on the restrained ends of the wall can be calculated as:

$$C_1 = \gamma_1 \beta_{11} w_m b_1 f_m' \tag{51}$$

$$C_2 = \gamma_2 \beta_{12} w_m b_2 f_m' \tag{52}$$

where the additional subscripts 1 and 2 for γ and β_1 have been used to single out the corresponding cross-section. The tensile force developed by the FRP laminate is:

$$T_f = A_f f_f = A_f E_f \varepsilon_f \tag{53}$$

Based on equations (51) to (53), equation (50) can be re-written as:

$$\gamma_2 \beta_{12} w_m b_2 f_m' = \gamma_1 \beta_{11} w_m b_1 f_m' + A_f E_f \varepsilon_f \tag{54}$$

Equation (54) expresses the equilibrium of the forces. The compatibility of deformations is expressed with the following two equations (Tumialan et al., 2003)

$$t - b_1 - b_2 = \frac{h}{2} \cdot \frac{1-\cos\theta}{\sin\theta} \cong \frac{1}{16} \frac{h^2}{b_1} \varepsilon_{m1} \tag{55}$$

$$\frac{b_2}{b_1} = \frac{\varepsilon_{m2}}{\varepsilon_{m1}} \tag{56}$$

Moreover, assuming that the deformation of the FRP occurs in an unbonded length, l_b, the strain in the FRP can be estimated using the equation:

$$\frac{\frac{t-b_2}{b_2} h \varepsilon_{m1}}{16 l_b} = \frac{\frac{t-b_2}{b_1} h \varepsilon_{m2}}{16 l_b} \tag{57}$$

To date, there is no experimental basis for the determination of l_b. Based on experimental observations (Tumialan et al., 2003-b) a suggested value for l_b is equal to 1.5 in (37.5 mm).

Given the failure mode (i.e. set the maximum value for ε_{m1} or ε_{m2} or ε_f), equations 8.49 to 8.52 can be iteratively solved for the remaining four unknowns out of the five (ε_{m1} , ε_{m2} , b_1 , b_2 , and f_f). Comparing the results of the first iteration with the ultimate values of ε_{m1}, ε_{m2} and ε_f, the actual failure mode of the wall can be determined and, therefore, a second iteration will give the actual value of all the unknowns.

Shear Limitations

The design shear strength for walls for which the arching action cannot be neglected, shall be in accordance to equation (38) after setting the ratio $\frac{M}{V\,t} = 1.00$.

CONCLUSIONS

The design methodology proposed in this paper offers a first rational attempt for consideration by engineers interested in out-of-plane upgrade of masonry walls with externally bonded FRP systems. Both, the case of bearing and not load bearing walls, and of infill and load-bearing walls for which the arching action cannot be neglected were considered. Additional experimental work as well as a reliability analysis is needed in order to determine a more comprehensive set of design factors.

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Table 1 -- Environment Reduction Factors

Exposure condition	Fiber type	Environment reduction factor C_E
Masonry, interior exposition	Carbon	0.95
	Glass	0.75
	Aramid	0.85
Masonry, exterior exposition	Carbon	0.85
	Glass	0.65
	Aramid	0.75
Masonry, aggressive environment	Carbon	0.85
	Glass	0.50
	Aramid	0.70

Table 2 -- k_m Factors for Different Strengthening Systems

Strengthening System	Limitations	Resin Type	k_m
GGRP	-	Polyurea	0.65
FRP Laminates	If putty is used	Epoxy	0.65[(1)]
	If putty is not used	Epoxy	0.45[(1)]
NSM FRP Bars	FRP rectangular bars, Groove having the same height of the bar and width 1.5 times the one of the bar	Epoxy	0.65[(2)]
	FRP circular bars, Square groove 1.5 times the diameter of the Bar[(4)]	Epoxy	0.35[(2)]
	FRP circular bars, Square groove 2.25 times the diameter of the Bar	Epoxy / LMCG[(3)]	0.55[(2)]

(1) Based on Tumialan et al., 2003-a
(2) Based on Galati et al. 2004
(3) Latex Modified Cementitious Grout
(4) Latex Modified Cementitious Grout can not be used with a standard square groove having dimensions 1.5 times the diameter of the bar

Table 3 -- $f_{f,s}$ for Different Fiber Types

Fiber Type		
Carbon	**Glass**	**Aramid**
$0.55 f_{fu}$	$0.20 f_{fu}$	$0.30 f_{fu}$

Table 4 -- Stress Block Patameters β_1 and γ

Parameter	Concrete	Clay
β_1	*0.805*	*0.822*
γ	*0.853*	*0.855*

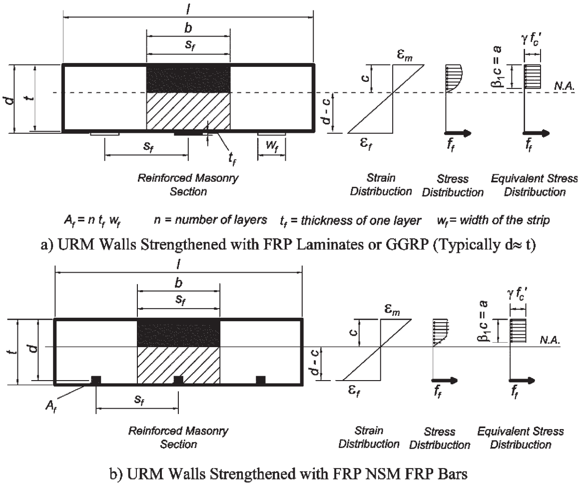

Figure 1 — Internal strain and stress distribution for a horizontal rectangular section of a strip of masonry under out-of-plane loads, without axial compression

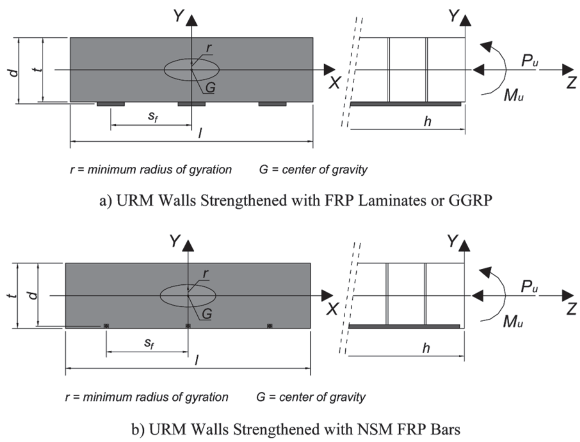

Figure 2 — Geometric parameters of the uncracked section under out-of-plane loads with axial compression

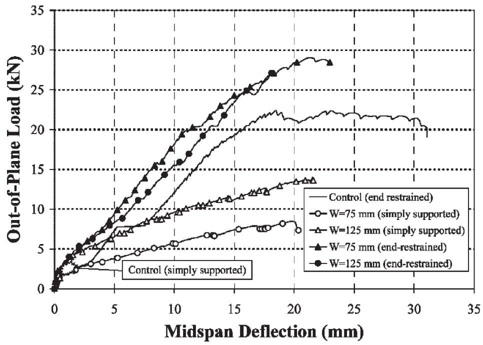

Figure 3 — Comparison between Simply Supported and End-Restrained Walls

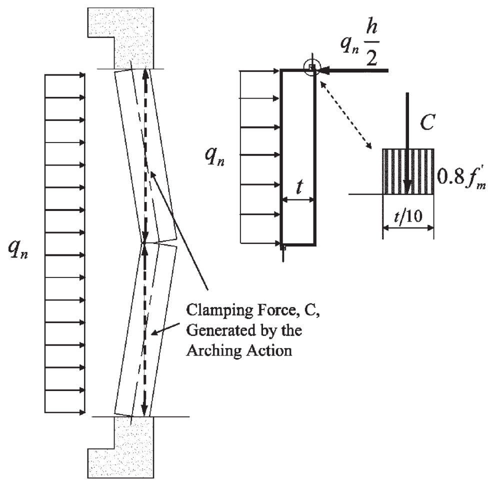

Figure 4 — Design of Rigid Arching Walls

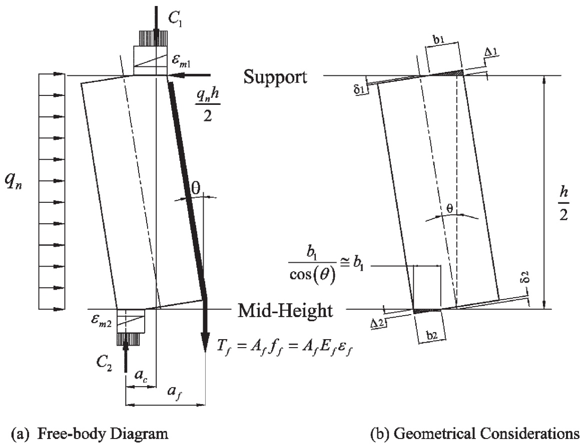

Figure 5 — Half Part of Analyzed Wall