DESIGN OF FRP RETROFITTED MASONRY UNDER OUT-OF-PLANE BENDING

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ABSTRACT

The severity of damage possible in unreinforced brick masonry (hereafter termed 'masonry') construction subjected to high levels of out-of-plane loading has been well demonstrated in recent times. Due to the large global building stock of masonry structures, it is essential that efficient methods for retrofit of masonry structures be developed. The use of efficient fibre-reinforced polymer (FRP) strips has been shown to improve the load-carrying and displacement capacities of masonry sections subjected to out-of-plane loading. This paper presents principles for design of masonry elements strengthened with vertically oriented FRP strips and subjected to out-of-plane bending. Design considerations are given, along with recommendations based on experimental observations. Design variables discussed include retrofitting technique (i.e. externally bonded or near-surface mounted), FRP material (i.e. carbon or glass) and FRP placement (i.e. relative to mortar joints). A design methodology for masonry retrofitted with vertical FRP strips with intermediate crack (IC) debonding as the failure mode is also presented.

KEYWORDS

FRP, masonry, out-of-plane bending, strengthening, design.

INTRODUCTION

Background

Inadequate out-of-plane bending strength of walls near the tops of buildings has been identified as of one of the governing weak links in the seismic load path for unreinforced brick masonry (hereafter termed 'masonry') buildings (e.g. Klopp and Griffith 1998). For this failure mode, the top storeys are generally critical due to the combination of increased earthquake induced accelerations (and therefore out-of-plane loading) with building height and reduced vertical compressive stress (e.g. Priestley 1985). This has been demonstrated in the analysis of structural damage after significant earthquake events, e.g. Whittier Narrows, California earthquake in 1987 (e.g. Deppe 1988; Moore *et al.* 1988). Recent catastrophic earthquake events in Italy (Pescaro, April 2009) and China (Sichuan, May 2008) have further demonstrated the severity of damage possible in masonry construction and again highlighted the need for retrofit of masonry structures. Efficient fibre-reinforced polymer (FRP) retrofitting technologies were initiated with reinforced concrete (RC) structures and have been adopted for use with masonry construction. Research (e.g. Oehlers and Seracino 2004) indicates that intermediate crack (IC) debonding is the most ductile, and therefore preferred, failure mechanism for FRP strengthened flexural members.

The application of externally bonded (EB) and near-surface mounted (NSM) FRP reinforcement to the tension face of masonry sections under flexure has been shown to effectively increase the maximum strength and displacement capacity. This paper discusses key design considerations for FRP retrofitted modern clay brick masonry based on experimental observations (i.e. FRP retrofitting technique, material and placement). A design methodology for masonry retrofitted with vertical FRP strips with IC debonding as the design failure mode is presented. The methodology uses the generic IC debonding model developed by Seracino *et al.* (2007) for RC and subsequently modified for use with masonry by Yang (2006).

MASONRY RETROFITTED WITH VERTICAL FRP STRIPS

Vertical Bending Moment Capacity

Consider a masonry wall supported along all four edges. If the wall length is sufficient compared to its height then, under out-of-plane bending, the central strip of the wall is predominantly under vertical bending and first cracking will occur along a horizontal bed joint at wall mid-height (Figure 1). The Australian Masonry Code, AS 3700 (Standards Australia 2001), uses the virtual work method for the design and analysis of masonry walls under out-of-plane loading and subject to two-way bending. The method assumes that this initial horizontal crack occurs at such a low level of applied loading that the vertical bending moment capacity, M_{cv}, does not contribute significantly to the flexural resistance of the wall when it reaches its maximum strength. This has provided the impetus for the application of vertical FRP strips to strengthen such walls, which has been shown to improve the vertical bending capacity and thus the maximum wall capacity (e.g. Willis *et al.* 2009).

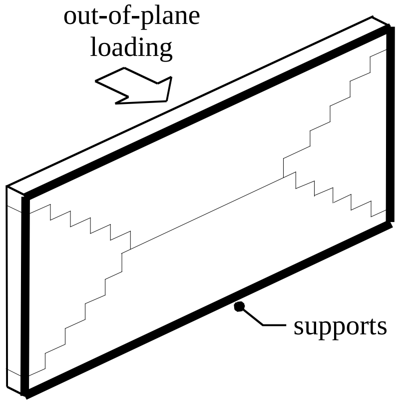


Figure 1. Wall supported along all four edges (Griffith *et al.* 2005)

The remainder of this paper discusses the use of vertically oriented FRP strips to strengthen masonry walls under out-of-plane bending. It is recognised that the use of horizontal FRP strips is most effective in the resistance of in-plane shear forces. Walls where this failure mode is critical are typically at the ground level in multi-storey buildings, which is the opposite of out-of-plane bending where top storey walls are generally critical. Discussion of this retrofitting arrangement is beyond the scope of this paper.

Prediction of IC Debonding Resistance

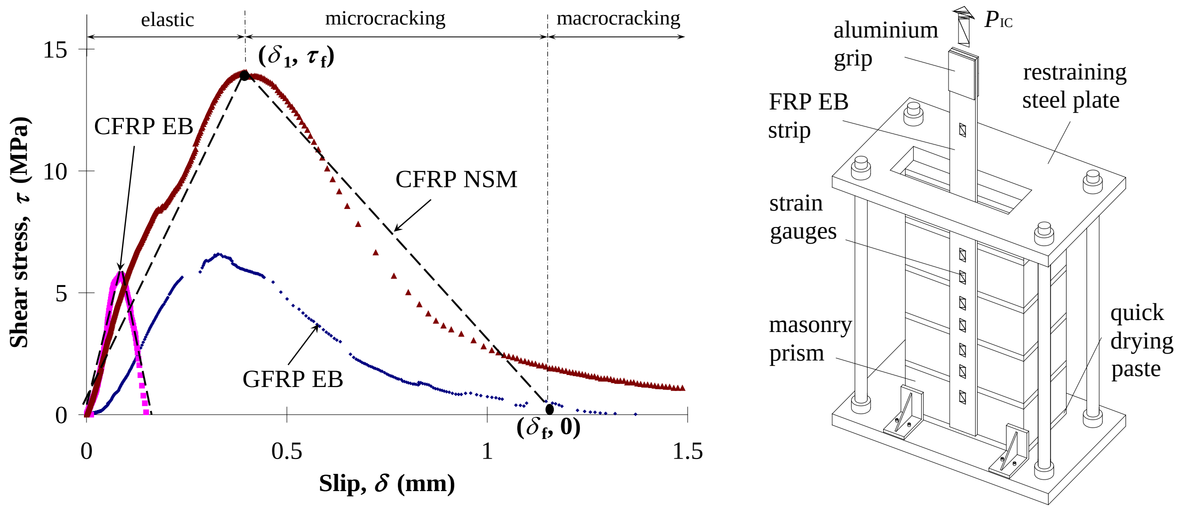
The axial force in the FRP strip required to cause the onset of IC debonding, P_{IC}, is given by Equation (1) developed by Seracino *et al.* (2007). This IC debonding force, P_{IC}, is a function of the area under the shear stress, τ, versus local slip curve, δ, i.e. the fracture energy, G_f. This curve (Figure 2(a)) is commonly referred to as the bond-slip relationship and may be determined experimentally using the monotonic pull test (Figure 2(b)). In Equation (1), τ_f and δ_f are defined in Figure 2(a), L_{per} is the failure perimeter (units of mm) and $(EA)_p$ is the flexural rigidity of the FRP strip.

To remove reliance on bond-slip data, Seracino *et al.* (2007) used regression to express Equation (1) in terms of readily available material parameters, namely the compressive strength of the concrete, f_c. Modification of the model for use with masonry by Yang (2006) replaced f_c with the lateral modulus of rupture, f_{ut}, of a brick unit using a material transformation based on theory by MacGregor (1988). The resulting generic model for use with masonry is given by Equation (2) and its accuracy has been verified against 29 pull tests (Yang 2006) (where b and t are the width of thickness of the FRP strip, respectively).

$$P_{IC} = \sqrt{\tau_f \delta_f}\sqrt{L_{per}(EA)_p} = \sqrt{2G_f}\sqrt{L_{per}(EA)_p} \tag{1}$$

$$P_{IC} = 0.988\varphi_f^{0.263}\left(\frac{f_{ut}}{0.53}\right)^{0.6}\sqrt{L_{per}(EA)_p} = 1.45\varphi_f^{0.263} f_{ut}^{0.6}\sqrt{L_{per}(EA)_p} \tag{2}$$

where $L_{per} = 4 + b$ for EB; $L_{per} = 4 + 2b + t$ for NSM and $\varphi_f = \dfrac{1}{2+b}$ for EB; $\varphi_f = \dfrac{1+b}{2+t}$ for NSM



(a) Bond-slip relationship (b) Pull test set-up

Figure 2. Bond-slip relationship and pull test (Yang *et al.* 2006)

Table 1 indicates the key parameters needed for design of vertical reinforcement of masonry walls in bending for the most commonly used FRP materials, i.e. carbon (CFRP) and glass (GFRP). In terms of FRP material and retrofitting technique, NSM CFRP strips are more efficient than the EB retrofitting technique for masonry due to the increased fracture energy, G_f, indicated in Figure 1(a).

Table 1. Typical values for design parameters for FRP retrofitted masonry (all units of N/mm^2)

Parameter	Carbon FRP	Glass FRP
Modulus of elasticity of FRP, E_p	165 x 10^3	70 x 10^3
Tensile strength of FRP, f_{up}	2700	3400
Lateral modulus of rupture of the brick unit, f_{ut}	3.5	
Modulus of elasticity of masonry, E_m	3500	

Design for Vertical Bending

The design process for a masonry wall strengthened with vertical FRP strips is based on the aim that the preferred wall behaviour will be governed by IC debonding of the FRP strip rather than any of the other more brittle failure modes. These include FRP rupture, horizontal bending failure of masonry between vertical FRP strips, or masonry crushing. The methodology presented is generic for either FRP material (CFRP or GFRP) therefore final selection is up to the discretion of the design engineer. The critical design parameters for the FRP retrofitting scheme include: (i) material (i.e. the modulus of elasticity of the FRP, E_p); (ii) cross-sectional dimensions (i.e. the width, b, and thickness, t, giving a cross-sectional area, A_p); and, (iii) horizontal spacing between vertical FRP strips (i.e. wall design strip width, s). The FRP cross-section and the spacing between strips determine the reinforcement ratio.

The usual flexural theory design assumptions for the cross-sectional analysis of an FRP reinforced section (as illustrated in Figure 3) are:

(1) plane section remains plane after bending;

(2) full composite action exists between the FRP strip and the masonry interface (i.e. no slips or opening-up at the interface); and,

(3) the tensile resistance of the masonry is neglected (i.e. the FRP does not contribute to strength until the masonry has fully cracked).

The design process for earthquake (or wind) loading depends on the horizontal acceleration (or wind pressure) that the wall is subject to in the out-of-plane direction. For a given load demand, w_d, the required flexural strength (i.e. vertical moment demand, M_d) for a wall spanning vertically between simple supports at its top and bottom edges is given by

$$M_d = \frac{w_d \cdot h^2}{8} \quad (3)$$

which can be written in terms of the design inertial acceleration (demand), a_d, as

$$M_d = \frac{a_d \cdot t_m \cdot s \cdot \gamma \cdot h^2}{8} \tag{4}$$

where

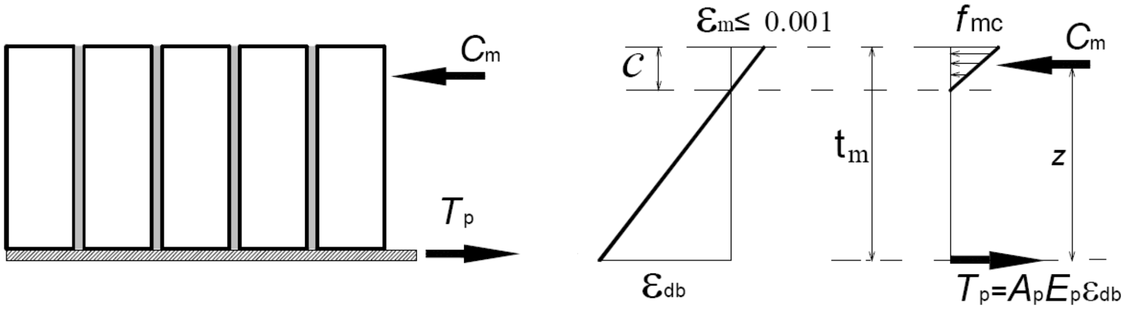
a_d = demand acceleration in terms of the acceleration due to gravity, g; (that is, units of 'g', i.e. acceleration normalised by the acceleration of gravity)

h = clear span of the masonry wall in the vertical direction (i.e. height of the masonry wall);

s = horizontal spacing between the vertical FRP strips;

t_m = thickness of the masonry wall; and,

γ = specific weight of the masonry.



(a) FRP retrofitted masonry section (b) Stress and strain profiles

Figure 3. Design for IC debonding (EB shown) (Willis *et al.* 2007)

Step 1: Select the horizontal spacing, s, of the vertical FRP reinforcement so that the strengthened masonry wall does not fail in horizontal bending between the strips due to the inertial load caused by a_d. To do this, calculate the horizontal bending capacity of the masonry wall, M_{ch}, (e.g. using AS 3700 to determine M_{ch} for a unit length of wall in the vertical direction in units of kNm/m) and then choose a strip spacing, s, such that

$$\frac{8M_{ch}}{s^2} > w_d = a_d \cdot t_m \cdot \gamma \tag{5}$$

In effect, rearrange Equation (5) and calculate 's' such that the following is satisfied

$$s^2 < \frac{8M_{ch}}{a_d \cdot t_m \cdot \gamma} \tag{6}$$

Step 2: Assume a cross-section for the FRP (i.e. choose b and t) and then calculate its IC debonding capacity, P_{IC}, given by Equation (2). Check that P_{IC} is less than the tensile rupture capacity of the FRP strip, $P_{rupture}$ (= f_{up} A_p). If not, adjust the FRP cross-section, usually by increasing the thickness, t, and start over at Step 2.

Step 3: Solve for the neutral axis location, c, using Equation (7) to satisfy axial force equilibrium (where C_m is the masonry compressive force, T_p is the FRP tensile force and ε_m is the masonry strain) and Equation (8) corresponding to plane sections remaining plane (where ε_{db} is the FRP debonding strain).

$$P_{IC} = T_p = C_m \quad \text{gives} \quad c = \frac{P_{IC}}{\frac{1}{2} \cdot \varepsilon_m \cdot E_m \cdot s} \tag{7}$$

$$\varepsilon_m = \frac{c}{\left(z - \frac{2}{3}c\right)} \varepsilon_{db} \tag{8}$$

where the lever arm, $z = t_m + \frac{t}{2} - \frac{c}{3}$ for EB; $z = t_m - \frac{b}{2} - \frac{c}{3}$ for NSM

For simplicity, it is assumed that for the EB case, t_m is negligible compared to t, and for the NSM case the level of embedment is also small compared to t. Therefore the neutral axis location, c, may be approximated by substituting Equation (8) into (7) to give:

$$c = \frac{-\alpha + \sqrt{\alpha^2 + 4 \cdot \alpha \cdot t_m}}{2} \qquad \left(\text{use } z \cong t_m - \frac{c}{3} \text{ to give } \varepsilon_m = \frac{c}{t_m - c} \varepsilon_{db}\right) \tag{9}$$

where $\alpha = \dfrac{2 \cdot P_{\mathrm{IC}}}{\varepsilon_{\mathrm{db}} \cdot E_{\mathrm{m}} \cdot s}$ and $\varepsilon_{\mathrm{db}} = \dfrac{P_{\mathrm{IC}}}{(EA)_{\mathrm{p}}}$

Step 4: Check that the masonry compressive stress is less than its capacity, f_{mc}, using Equation (8). If not, return to Step 2 and decrease the FRP cross-section. Otherwise, continue to Step 5.

Step 5: Calculate the vertical bending capacity of the FRP reinforced section, M_{cv}, using Equation (10) to check if its capacity is greater than the demand. If not, then go back to Step 2 and increase the FRP cross-section, and/or decrease the FRP spacing and restart the procedure. If OK, then the design is complete.

$$M_{\mathrm{cv}} = P_{\mathrm{IC}} \cdot z > M_{\mathrm{d}} \tag{10}$$

Vertical Strip Location

Vertically oriented NSM FRP strips can either be placed through the brick units or the perpend joints. Due to aesthetics and ease of placement it is likely that in most applications the strip would be run through the perpend joints. However, it should be noted that positioning vertical NSM FRP strips through the perpend joints can cause a reduction in bond strength in the order of 10% (Yang *et al.* 2006). However, such level of reduction may not be deemed significant given the beneficial effects of the relative ease of placement (i.e. cutting through half the amount of brick units) and the reduced aesthetic impact.

Verification of Design Procedure

Full scale wall tests conducted by Yang (refer Willis *et al.* 2009) were designed to have approximately equal probability of three failure modes occurring, i.e. (1) horizontal bending failure; (2) vertical IC debonding failure; and, (3) FRP rupture. Three walls were retrofitted using EB strips (with varying strip spacing, cross-section and FRP material), and all behaved in a similar manner (refer Figure 5). Wall 1 (CFRP) failed by IC debonding, while Walls 2 and 3 (GFRP) failed by horizontal bending between the FRP strips. The maximum FRP strain, ε_{max}, for each retrofitted wall test was within 10% of the experimental debonding strain obtained from pull tests. For Wall 1 (CFRP), ε_{max} was approximately 30% of the rupture strain, while for Walls 2 and 3 (GFRP) ε_{max} was less than 90% of the rupture strain. Figure 4 indicates the load-displacement behaviour for each wall test and demonstrates the relative increase in strength possible from the design procedure presented in this paper. The load-carrying capacity of the unstrengthened wall (specimen 'Control A') was approximately doubled. It should be noted that the strengthened walls all had peak capacities in excess of 8 kN/m^2. For the 110 mm thick walls tested here, this corresponds to an equivalent inertial acceleration capacity of nearly 4*g*, i.e. four times the acceleration of gravity.

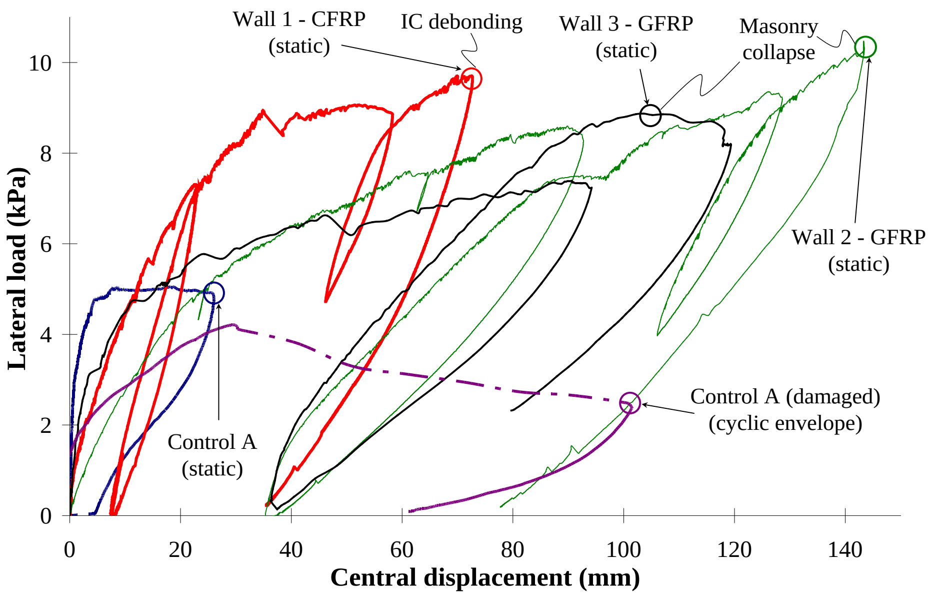


Figure 4. Load-displacement comparison (Willis *et al.* 2009)

CONCLUDING REMARKS

The following observations and conclusions are presented for this paper:

(1) A design methodology for masonry retrofitted with vertical FRP strips with IC debonding as the failure mode was presented. The design procedure was verified using the results of three full scale wall tests for externally bonded glass and carbon FRP applications. From this it appears that it is possible to design FRP retrofits for unreinforced masonry walls that can be optimised to use the minimum FRP material required in terms of both cross-section and maximum spacing.

(2) There was little observable difference in the strength of the three walls as there should not have been given that they were all designed to fail at similar loads. However, the displacement capacity of the GFRP retrofitted walls which both failed by horizontal bending of the masonry between the vertical FRP strips was substantially greater (approximately double) than the displacement at failure for the CFRP retrofitted wall.

(3) Many questions remain regarding the behaviour of FRP strengthened walls including the performance of NSM FRP and the effects of in-plane shear deformations on flexural FRP application, particularly EB FRP where it is oriented in its strong direction and may be more prone to premature debonding.

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