FRP RETROFITTED URM WALLS UNDER INPLANE SHEAR – A REVIEW OF AVAILABLE DESIGN MODELS

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SUMMARY

In the last two decades, several seismic retrofitting techniques for masonry structures have been developed and practiced and fibre reinforced polymer (FRP) material has been increasingly used due to their high strength/stiffness to mass ratio and easy application. Although much research has been carried out on FRP strengthening of URM structures, most of them have been experimental studies to investigate the effectiveness of retrofitting techniques rather than the development of a rational design model. In addition, more research has been conducted on FRP retrofitted URM walls under out-of-plane loads where flexural behaviour dominates, the research on the shear strength of FRP retrofitted URM walls has been limited. This paper presents a review of research in this area.

INTRODUCTION

There are many seismic retrofitting methods for URM walls, an extensive review and comparison of various retrofitting techniques is given in Chuang and Zhuge (2005). FRP consists of high resistance fibre impregnated with polymeric resins and have high tensile strength, lightness and corrosion insensitivity. If FRP applies on seismic retrofitting applications, these materials can absorb tensile stress and increase overall stiffness and ductility and bearing capacity. Using FRP for seismic retrofitting applications also have some more advantages like aesthetic, rapid application, durability, low cost of installation, no loss of valuable space and can remain unchanging dynamic properties (mass) for structures. Therefore, these make FRP particularly suitable for seismic retrofitting of URM.

There are three common types of FRP, which are Glass, Aramid, and Carbon Fibre Reinforced Polymers (GFRP, AFRP and CFRP, respectively) and FRP are commercialised in different shapes: rods, tendons fabrics, and laminates. All of them have been successfully used to enhance the strength and ductility for seismic retrofitting of URM structures (Ehsani et al., 1997; Tinazzi et al. 2000; Valluzzi et al 2002; ElGawady et al 2006). However, FRP rods and laminates are most common.

While numerous research has been conducted on FRP retrofitted URM walls under out-of-plane loads where flexural behaviour dominates (Tan and Patoary 2004; Korany and Drysdale 2006), the literature review revealed that the research on the shear strength of FRP retrofitted URM walls has been limited (Ehsani et al. 1997; Valluzzi et al 2002; Stratford et al 2004; ElGawady et al 2006). In addition, similar to concrete, the behaviour of URM walls

under shear is much more complex than that under flexure.

The in-plane shear strength of URM walls could be increased by bonding FRP strips (normally in the diagonal shape (X)) or sheet to the surface of the wall (Figures 1b and 1c), normally only on one side of the wall or using near surface mounted (NSM) FRP bars (Figure 1a).

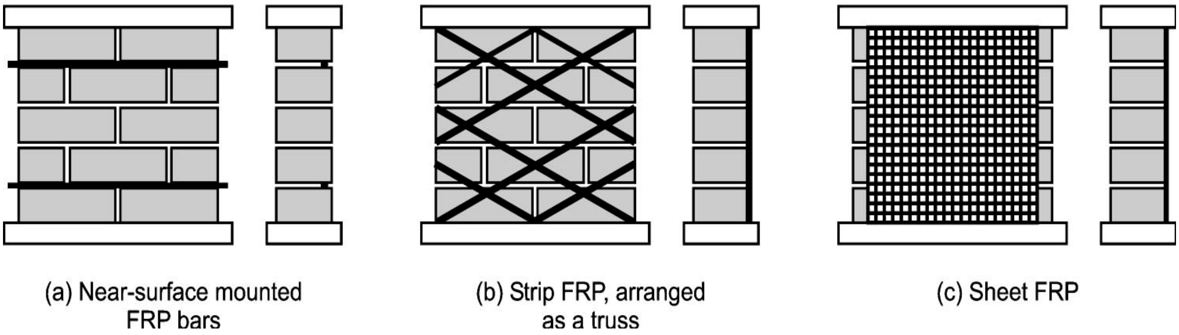
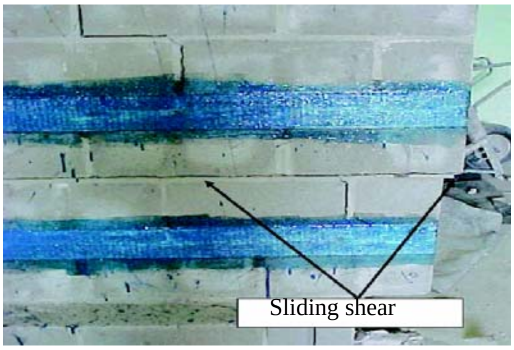


Figure 1. Alternative Approaches to Strengthening Masonry Using FRP (Stratford et al 2004)

For strip FRP (Figure 1b), some researchers also attempted grid configuration or applied FRP either in horizontal or vertical directions (Valluzzi et al 2002, Holberg and Hamilton 2002, Wei et al 2007). When applying FRP strips along horizontal direction only and tested with in-plane loading along the wall diagonal, it was observed the wall failed due to sliding shear along an un-strengthened joint (Figure 2) and therefore this configuration has not been used at the present. When applying FRP strips in the vertical direction (Figure 3), they will act as dowels and the testing results indicated that the shear strength of the wall would be increased (Wei et al 2007). However, all available design models assumed the FRP contribution to be zero if they were placed in vertical direction. Experimental results proved that applying FRP strips along diagonal direction is most effective (Valluzzi et al 2002) and therefore it becomes a common practice.



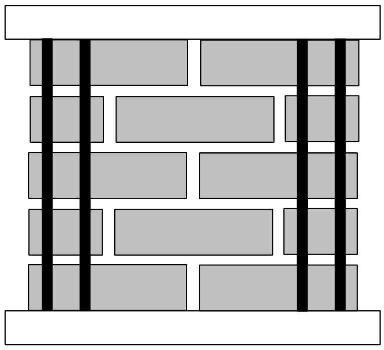


Figure 2: Sliding shear failure for wall strengthened with horizontal GFRP (Nanni and Tumialan 2003)

Figure 3. Vertical FRP configuration

The discussion of this paper will only focus on the surface bonding schemes where FRP are bonded externally to the surface of the wall. Different to shear strengthening of reinforced concrete beams where three schemes were commonly used which including side bonding, U jacketing and wrapping, only side bonding has been used for masonry shear walls. Therefore, the design model will only need to distinguish the fibre distribution (strips or sheets) and orientation.

Triantafillou (1998) tested a series of indicative clay URM walls retrofitted with

unidirectional CFRP fabric strips for the three most common cases of masonry loading: out-of-plane bending with axial force, in-plane bending with axial force, and in-plane shear with axial force. The pertinent results indicated that the in-plane shear capacity of FRP strengthened walls can be quite high, especially in the case of low axial loads. The author's experimentation highlighted the important role of failure through the FRP shearing beneath the bond.

Tinazzi et al. (2000) tested the shear resistance of a masonry wall using the diagonal compression test. Using a specially designed jig the experiment tested various configurations of GFRP on both single and double brick indicative URM walls under quasi-static loading cycles. The results revealed an elastic behaviour until de-bonding of the reinforcement occurred. FRP strengthening completely changed the failure mode of the shear walls, preventing any detrimental sliding of the mortar joints. Similar testing has also been conducted by Valluzzi et al (2002) where FRP was applied either on single or double-side of the wall to study the asymmetrical effect. The results indicated that double-side retrofitting scheme is more effective and the diagonal configuration is more efficient than the grid set up.

The past research also indicated that, for the sake of both economy and mechanical response, unidirectional FRP reinforcement is preferable than two-directional fabrics that cover the whole surface of masonry walls (Triantafillou 1998).

The literature review revealed that the major research have focused on the effectiveness of the retrofitting technique rather than developing a design model. Research work on the development of a design model is very limited (Triantafillou 1998, ElGawady et al 2006). In this paper, experimental observations of failure modes are described first, followed by a comprehensive review of existing design models. The review will provide the necessary background information for the models to be assessed in the companion paper (Zhuge and Teng 2007).

FAILURE OF URM WALLS RETROFITTED WITH FRP UNDER IN-PLANE SHEAR

The strength of the retrofitted URM walls depends on the controlling failure modes. Two failure modes are being common for shear: Shear sliding and diagonal tension cracking. When FRP was bonded to the surface of the wall, compressive crushing type of failure was still observed, but tension or shear failure modes may be prevented by the application of FRP (Hamid et al 2005). Instead, FRP premature debonding or fracture was commonly observed during the testing and in general FRP could not reach its ultimate strength (Ehsani et al 1997, Stratford et al 2004, ElGawady et al 2006). Experimental tests indicated that the failure pattern was affected by the strength, orientation and anchorage length of FRP (Ehsani et al 1997, Hamid et al 2005).

The in plane shear behaviour of URM is influenced by many factors such as the vertical compressive stress, wall aspect ratio etc. The number of factors will increase when FRP is used to retrofit the structure. The experimental results indicated that FRP debonding was the most common case of failure (Ehsani et al 1997, Nanni and Tumialan 2003, Stratford et al 2004, Chuang and Zhuge 2004). Local debonding failure initially occurred near the loading application or/and the reaction points due to stress concentration and the consequence cracks along mortar-break interface. At the ultimate load, the debonding region spread suddenly, roughly along the compressive diagonal (Stratford et al 2004). The debonding failure

normally occurred at the epoxy-brick interface (Valluzzi et al 2002, Chuang and Zhuge 2004), but may also occur beneath the surface of the brick. This type of failure is not a peeling failure (which involves direct tensile stress) as there is no out-of-plane bending. It is a shear delamination failure (Stratford et al 2004).

The testing results also show that grid strip FRP configuration or sheet FRP retrofitted specimens had spread crack patterns, whereas diagonal strip configuration retrofitted specimens had a single splitting crack (Valluzzi et al 2002, ElGawady et al 2006).

The debonding failure of URM walls under in-plane shear was due to the diagonal shear crack in the wall. When this crack has formed, the debonded area of FRP was fast spreading. The FRP strip which was intersecting with the major diagonal crack is vulnerable to debonding failure when the crack becomes wider. As FRP can only carry tensile force, the FRP strip along the compressive diagonal buckled at failure (Figure 4).

Figure 4 Strip FRP debonding and buckling (Chuang and Zhuge 2004)

When a FRP sheet was applied through the whole wall, the failure would be similar. It bucked in the direction of the compressive diagonal as diagonal banding (Figure 5). The fully cracked panel carries shear by a truss mechanism.

Depending on the reinforcement stiffness and bonding area, rupture of the FRP strips has also been observed during experiments (Figure 6) (Valluzzi et al. 2002).

Because debonding is the most common type of failure, strength and stress distribution of bond between FRP and masonry are central aspects especially at the anchorage or at discontinuities such as mortar joints in masonry structures. Cracks and irregularities of the surface could represent weak points for the bond behaviour due to concentration of stresses. For this reason the bond strength in applications on masonry elements is more difficult to prevent due to greater variability and uncertainty of masonry characteristics (Ceroni and Pecce 2006).

One interesting finding from the testing results was that the stress level in FRP at debonding does not increase with bond (anchorage) length if the bond length is sufficient (Stratford et al 2004), same as what was observed for reinforced concrete beams (Chen and Teng 2001).

The testing results also indicated the FRP retrofitting could change the failure mode of the wall (Chuang and Zhuge 2004, Stratford et al 2004, ElGawady et al 2006). For example, if URM wall failed by shear sliding along the mortar-brick interface, the retrofitted wall failed by diagonal shear as FRP increased the vertical compressive stress.

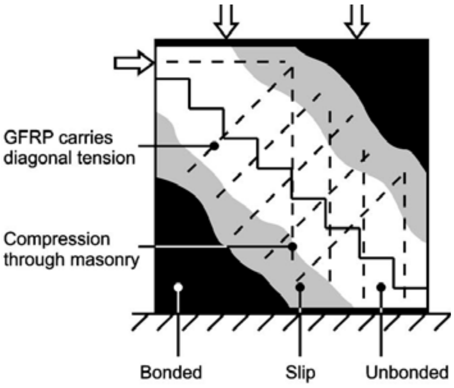


Figure 5 Sheet FRP debonding
(Stratford et al 2004)

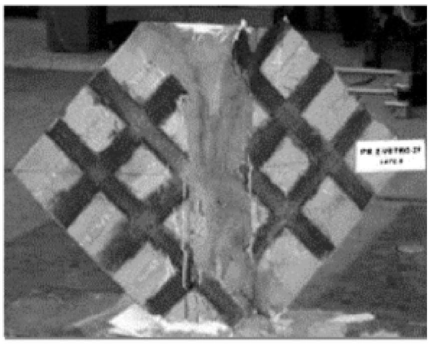


Figure 6 FRP rupture
(Valluzzi et al 2002)

DESIGN MODELS FOR URM RETROFITTED WITH FRP UNDER IN-PLANE SHEAR

For masonry retrofitted with externally bonded FRP, limited design models have been developed (Triantafillou 1998, Triantafillou and Antonopoulos 2000, Nanni et al 2003, ElGawady et al 2006). They are all based on the assumption that the total contribution to shear capacity is the sum of two terms, similarly to reinforced concrete. The first term, V_m accounts primarily for the contribution of uncracked masonry, whereas the second term, V_{FRP} accounts for the effect of shear reinforcement:

$$V = V_m + V_{FRP} \tag{1}$$

V_m may be calculated according to provisions in existing design codes of each country, so the major differences between each available model is attributed to FRP contribution V_{FRP}.

The strength of V_{FRP} will be affected by many factors, such as: the strength of FRP, the orientation of FRP, the anchorage length of FRP and more importantly the strain distribution of FRP. Therefore Triantafillou (1998) argued that it is impossible to accurately predict the FRP contribution to shear strength.

It will be ideal if FRP reaches its tensile strength at failure. However, most experimental data showed that the contribution of the FRP to the shear strength of URM wall is less than its ultimate tensile strength due to premature debonding failure as discussed earlier. Therefore, the ultimate tensile strength of FRP has been replaced by an effective FRP strain for each model. However, different values were adopted by each model. Each of the available models is summarised and reviewed in the following section. A consistent set of notation has been used by all models in order for an easy comparison.

Triantafillou Model

Triantafillou (1998) model is based on classical truss analogy and is only suitable for the case where FRP laminates are in the form of narrow straps (Figure 7). It is assumed that the contribution of vertical FRP is negligible and the only shear resistance mechanism is associated with the action of horizontal laminates (Figure 7). This action is modelled in analogy to the action of stirrups in reinforced concrete beams. V_{FRP} is calculated as follows:

$$V_{FRP} = \rho_h E_{FRP} \left(r \frac{\varepsilon_{FRP,u}}{\gamma_{FRP}} \right) t 0.9d \qquad (2)$$

Where r = reinforcement efficiency factor, depending on the exact FRP failure mechanism (debonding or tensile fracture), ρ_h = FRP area fraction in horizontal direction, $\varepsilon_{FRP,\ u}$ = ultimate tensile strain of FRP, t = masonry wall thickness, d = effective depth, which for masonry walls with several layers of reinforcement, can be taken approximately equal to 0.8*l*, and γ_{FRP} = partial safety factor for FRP in uni-axial tension (1.15, 1.2 and 1.25 for CFRP, AFRP and GFRP respectively). If using the effective FRP strain to replace the ultimate tensile strain, $\varepsilon_{FRP,\ e} = r\varepsilon_{FRP,\ u}$, Equation 2 can be rewritten as:

$$V_{FRP} = \frac{0.7}{\gamma_{FRP}} \rho_h E_{FRP} \varepsilon_{FRP,e} lt \qquad (3)$$

$\varepsilon_{FRP,\ e}$ depends on the FRP development length, defined as that necessary to reach FRP tensile fracture before debonding and is inversely proportional to the FRP axial rigidity ($\rho_h E_{FRP}$). Therefore, as the FRP laminates become stiffer and thicker, debonding dominates over tensile fracture and the effective FRP strain is reduced. The effective FRP strain $\varepsilon_{FRP,\ e}$ in equation 3 can be determined from equation 4, which was developed by the same author through regression of experimental data for concrete members strengthened with FRP in shear:

$$\varepsilon_{FRP,e} = 0.0119 - 0.0205(\rho_h E_{FRP}) + 0.0104(\rho_h E_{FRP})^2 \qquad (4)$$

This model has not been validated by any experimental data. Equation 4 which was based on the testing results for concrete may not be suitable for masonry.

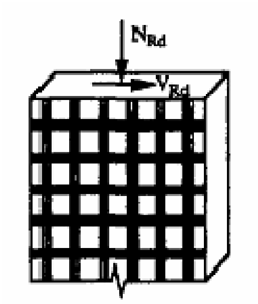


Figure 7. FRP Strengthened Masonry Wall Subjected to in-plane shear (Triantafillou 1998)

Triantafillou and Antonopoulos (TA) Model

Later, the author (Triantafillou and Antonopoulos 2000) proposed an improved model in which the effective FRP strain (equation 4) has been further developed as three expressions to distinguish different failure modes (FRP debonding or rupture) and types of FRP materials (CFRP or AFRP):

Fully wrapped CFRP:

$$\varepsilon_{FRP,e} = 0.17 \left(\frac{f_c^{2/3}}{E_{FRP} \rho_h} \right)^{0.3} \varepsilon_{FRP,u} \qquad (5)$$

Side or U-shaped CFRP jackets:

$$\varepsilon_{FRP,e} = \min\left[0.65\left(\frac{f_c^{2/3}}{E_{FRP}\rho_h}\right)^{0.56}\times 10^{-3}, 0.17\left(\frac{f_c^{2/3}}{E_{FRP}\rho_h}\right)^{0.3}\varepsilon_{FRP,u}\right] \tag{6}$$

Fully wrapped AFRP:

$$\varepsilon_{FRP,e} = 0.048\left(\frac{f_c^{2/3}}{E_{FRP}\rho_h}\right)^{0.47}\varepsilon_{FRP,u} \tag{7}$$

Where f_c = compressive strength of masonry, $\varepsilon_{FRP,u}$ = 0.015 for CFRP, 0.035 for AFRP.

As fully wrapped FRP has never been used to retrofit masonry walls, only equation 6 is relevant to masonry. The effects of orientation of FRP and different retrofitting schemes (using FRP sheet or strip) were not considered in these two similar models. Therefore, they may overestimate the FRP performance of walls with less effective retrofitting schemes as the testing results indicated URM walls retrofitted with diagonal configuration was more than 40% stronger than grid configuration when the same FRP axial rigidity ρE was used (Valluzzi et al 2002). In addition, a more rational approach for the effective (average) strain (stress) at the ultimate limit state would be preferred.

AC 125 (2007) Model

Under section 7.3.2.6 shear strength enhancement, the following formulas have been proposed for rectangular wall sections where fibre bonded on either both sides or one side only. When FRP bonded on both sides of the wall at an angle θ to the member's axis:

$$V_{FRP} = 2t_{FRP} f_j l \sin^2\theta \tag{8}$$

Where t_{FRP} = FRP thickness, l = depth of the wall parallel to the direction of applied shear force, θ = fibre's orientation and $f_j = 0.004E_{FRP} \le 0.75 f_{uj}$ (for completely wrapped on all four sides), f_{uj} = ultimate tensile strength of composite material (MPa).

When FRP only bonded to one side at an angle ≥ 75° to the member axis, nominal shear strength enhancement shall be taken as:

$$V_{FRP} = 0.75t_{FRP} f_j l \sin^2\theta \tag{9}$$

Where $f_j = 0.0015E_{FRP} \le 0.75 f_{uj}$

Equations (8) and (9) are very similar to equation 3 (Triantafillou' model), but the effective strain has been fixed as a constant value of 0.0015 (one side) and 0.004 (fully wrapped). This may not be right as the effective strain is affected by the axial rigidity. Also equation 8 is not applicable as fully wrapped FRP has never been used to retrofit masonry walls. Equation 9 is only applicable to walls strengthened with continuous FRP sheets.

Nanni et al's Model

Nanni et al (2003) model is also based on the truss analogy and in the same form of reinforced concrete:

$$V_{FRP} = \kappa_f \left(\frac{A_{FRP}}{s} \right) f_{fu}^* l \quad (10)$$

Where A_{FRP} = cross-sectional area of FRP shear reinforcement, s = the spacing of reinforcement, l = actual depth of masonry in direction of shear considered and f_{fu}^* = tensile strength of the FRP, κ_f = 0.5 (effective stress in the FRP is 0.5 of the ultimate strength).

Equation 10 is based on the following assumptions:

- Inclination angle of shear cracks is constant and equal to 45°
- Effective strength is reached in all reinforcement intersected by the diagonal crack
- FRP reinforcement carries all the shear demand.

As equation 10 is purely based on reinforced concrete beams, effective strain, FRP orientations and retrofitting schemes (using FRP sheet or strip) were not considered.

Stratford et al's Model

Stratford et al (2004) model is also based on the truss analogy (Fig 8), but is only applied to FRP sheet not for strips. It is assumed that the debonded FRP is linear elastic and the strain capacity must be greater than ε_1:

$$\varepsilon_1 = \frac{\delta_1}{l \cos\theta} \quad (11)$$

Where *l* = unbonded length of FRP which is determined by the anchorage arrangements, δ_1 the deflection capacity of the URM.

The above equation only provides a lower bound solution and could not be used for the ultimate limit state design.

The authors also proposed that any fixed anchorage of the FRP to the masonry must not fail under a load of:

$$V'_{F1} = \frac{w t_f E_{f\theta}}{l} \delta_1 \quad (12)$$

Where V'_{F1} = load carried through the FRP at failure of masonry, w = effective width of FRP, $E_{f\theta}$ = stiffness of FRP, t_f = thickness of FRP.

The model was not calibrated with any experimental data and it seems too superficial to apply for practical design.

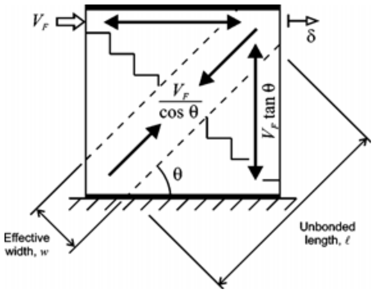


Figure 8. Truss mechanism for load through debonded strengthening (Stratford at al. 2004)

Based on the truss analogy, the diagonal component of the truss is at 45° to the vertical. Therefore if additional FRP is applied to the masonry the vertical force will increase and the

failure of masonry changes from slipping to crushing (Stratford et al 2004). Therefore, there is an upper limit to the useful amount of FRP. However, this has not been considered by the existing models.

CONCLUSIONS

In this paper, a review of existing research on the design models for shear strengthening of URM walls with externally bonded FRP has been presented. Each existing design model found in the literature was reviewed and compared. Some interesting conclusions and observations are summarised as follows:

The research work in this area has been very limited and most of them have been concentrated on the assessment of the effectiveness of the retrofitting schemes through experimental testing. There are a few design models have been proposed which are all based on the truss analogy and in a similar form of shear strength design for RC beams. However, none of them has been calibrated with experimental results. Therefore, the accuracy of the models could not be assessed. FRP rupture and debonding are two different failure modes, none of the existing models treatment them separately. In addition, experimental testing proved that retrofit schemes would have significant effect on the ultimate strength of the wall. However, none of the models reflects such difference. Some models only apply to FRP sheet retrofitting.

Although all existing models reflect the fact that FRP will not reach its ultimate tensile strength at failure, different values were proposed for the effective strain (stress) without a satisfactory explanation. A more rational approach for the effective strain (stress) at the ultimate limit state would be preferred.

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