SEISMIC SHEAR CAPACITY OF BRICK MASONRY WALL REINFORCED BY GFRP

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ABSTRACT

By carrying out experiments on eight brick masonry walls with pilasters reinforced by glass fiber reinforced polymer (GFRP), a seismic shear capacity of brick masonry wall structures is studied in detail. First, the interaction coefficient of the pilaster $\psi$, modified coefficient $\delta_{gi}$ statistic coefficient $\beta$ and effective participation coefficient $\zeta$ are determined. Then, based on failure model of brick masonry walls and truss model of fiber reinforced polymer (FRP), the formulae of seismic shear capacity of brick masonry wall with pilaster reinforced by FRP are established, which have an excellent coincidence with the experimental results obtained from the test; Finally, the simplified design formula of seismic shear capacity of brick masonry wall reinforced by FRP has been proposed in this paper. The design formula can be used for further study or for design reference in brick masonry structures.

Keywords: Cyclic tests, glass fiber reinforced polymer (gfrp), reinforcement, brick masonry wall with pilaster, seismic shear capacity

1. INTRODUCTION

The use of new technologies and materials for both restoring and reinforcing masonry structures is technically and economically very interesting. Nowadays, FRP represents a new opportunity to restoring ambition with considerable development in unreinforced masonry strengthening. Schwegler [1-2] was the first to propose and study the use of laminates carbon fiber reinforced polymer (CFRP) as aseismic strengthening elements of masonry structures and developed an analytical model for the in-plane behavior of CFRP-strengthened walls within the framework of stress fields theory. The work of Saadatmanesh [3], Ehsani [4], and Ehsani et al. [5] focused on experimental studies of unreinforced masonry specimens strengthened with epoxy-bonded glass fabrics. Kolsch [6] tested an external reinforcing system for masonry walls incorporating carbon fiber. The specimens were typically able to withstand lateral loads equivalent to the inertial forces generated from 0.3g (gravity) acceleration. Triantafillou [7] studied the mechanical behavior of unreinforced masonry walls strengthened with externally bonded FRP laminates (or fabric strips) using

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simple modeling and obtained the shear capacity of masonry strengthened with FRP laminates. Valluzzi [8] performed a series of compressive tests of masonry panels strengthened by FRP laminates. In this test, two configurations of reinforcing system were investigated: strips as grid arrangement or application of diagonal strips orthogonal to the loaded diagonal. Cecchi et al. [9] proposed a mode for CFRP reinforced masonries by means of homogenization procedures. Stratford et al. [10] studied the influence of reinforcement mode and gave the design equations of shear capacity of FRP-reinforced wall.

In spite of so much previous research, the formulae that have been partly established for calculating shear load-bearing capacity of wall reinforced with FRP are deficient in expressing forms and factors considered which are listed as follows:

a. The formulas (diagonal mode formulas) were just appropriate for the masonry walls reinforced with diagonal sheets, which is helpless for the masonry walls reinforced with horizontal sheets or both diagonal sheets and horizontal sheets.

b. As far as the diagonal mode formulas are concerned, the effect of vertical component of forces provide by diagonal sheets on the shear capacity is neglected.

c. All the formulas that had been established are based on the FRP-reinforced masonry walls with rectangular sections, as for the FRP-reinforced masonry wall with pilaster, no corresponding calculating formula are brought forward.

In order to make up the deficiency of the present formulas, eight pieces of brick masonry walls with pilaster strengthened by GFRP and one piece of normal masonry wall with pilaster were carried out and the corresponding calculating and design formulas based on the truss strengthening system of FRP have been established.

2. EXPERIMENTAL PROGRAM

2.1 Specimen design

To be of possible contrast between this research and others, the main bodies of specimens, which each having pilaster that has a section of 240×240mm and the same height of the main body, are of the nominal dimensions of 1500×750×240mm [see Figure 1].
Figure 1. The concrete dimensions of specimens

Figure 2 shows the testing arrangement. A combination of vertical compression (representing loads from the building above) and in-plane shear load was applied to each specimen.

The vertical pre-stress load was applied through two connected hydraulic jacks. This load was distributed across the top of the specimen by a stiff steel reaction beam. The horizontal load (shear load) was applied to the specimen by a horizontal double-acting jack.

The strength of the specimen mainly relates to the following aspects: the practical compressive strength of brick is equal to 10.15MPa; the compressive strength of mortar is equal to 2.5MPa and the practical compressive strength of mortar of every specimen are shown in table 2; the characteristic properties of single-direction GFRP in Table 1. In the present experimental work, there are nine specimens for which parameters are shown in Table 2.

Strengthening schemes of every specimen are shown in Figure 3 and Table 2.

Table 1. Material Properties of GFRP

<table>
<thead>
<tr>
<th>Practical Tensile Strength (MPa)</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Elongation Ratio (%)</th>
<th>Nominal Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1507</td>
<td>93.75</td>
<td>1.5</td>
<td>0.169</td>
</tr>
</tbody>
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Table 2. Details of the Masonry Specimens

<table>
<thead>
<tr>
<th>Serial Number</th>
<th>Strength of Mortar (MPa)</th>
<th>Reinforcement Mode</th>
<th>Strip Number in Single Surface (mode)</th>
<th>Strip Width (mm)</th>
<th>Area Reinforcement Rate (%)</th>
<th>Volume Reinforcement Rate (%)</th>
<th>Anchorage Mode</th>
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<tbody>
<tr>
<td>GW1</td>
<td>2.67</td>
<td>-----</td>
<td>-----</td>
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<td>-----</td>
<td>-----</td>
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<tr>
<td>GW2</td>
<td>3.55</td>
<td>Horizontal</td>
<td>3 (Horizontal)</td>
<td>100</td>
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<td>0.056</td>
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<tr>
<td>GW3</td>
<td>3.38</td>
<td>Mixed</td>
<td>3 (Horizontal)</td>
<td>50</td>
<td>40.3</td>
<td>0.057</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 (Diagonal)</td>
<td>70</td>
<td></td>
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<tr>
<td>GW4</td>
<td>3.43</td>
<td>Diagonal (uniform width)</td>
<td>4 (Secondary)</td>
<td>75</td>
<td>42.0</td>
<td>0.059</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 (Primary)</td>
<td>75</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GW5</td>
<td>3.15</td>
<td>Diagonal (different width)</td>
<td>4 (Secondary)</td>
<td>60</td>
<td>42.7</td>
<td>0.060</td>
<td>-----</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 (Primary)</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GW6</td>
<td>2.32</td>
<td>Diagonal (different width)</td>
<td>4 (Secondary)</td>
<td>80</td>
<td>56.1</td>
<td>0.079</td>
<td>Anchorage e</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 (Primary)</td>
<td>120</td>
<td></td>
<td></td>
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</tr>
<tr>
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<td>Diagonal (uniform width)</td>
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<td>56.1</td>
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<td>Anchorage e</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>2 (Primary)</td>
<td>120</td>
<td></td>
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<tr>
<td>GW8</td>
<td>2.68</td>
<td>Diagonal (different width)</td>
<td>4 (Secondary)</td>
<td>60</td>
<td>42.7</td>
<td>0.060</td>
<td>Anchorage e</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 (Primary)</td>
<td>90</td>
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<tr>
<td>GW9</td>
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<td>Mixed</td>
<td>3 (Horizontal)</td>
<td>70</td>
<td>56.7</td>
<td>0.080</td>
<td>Anchorage e</td>
</tr>
</tbody>
</table>

Figure 3. Reinforcement scheme of specimens
Before applying the GFRP, the specimen was first cleaned from dust and mortar protrusions manually using a wire brush. In this preparation, attention was focused on cleaning the joints and removing excessive mortar and loose particles from the wall surface. A light layer of epoxy resin was then applied to the using wall a hand held paint roller to the wall surfaces. A mixture of the epoxy resin and silica fume was then used to fill the tooled joints and smooth the prepared surface before applying the GFRP. Prior to application on the walls, the dry fabrics were cut to length and the epoxy was worked into it with a paint roller. In order to minimize trapped air voids, the roller was run several times on the wet fabrics.

2.2 Loading Procedure
Displacement control mode was adopted in this study for the load history. Taking the practical loading state of masonry walls into account, a vertical prestress of $N = 320\text{kN}$ was first applied to the wall, then came to the horizontal load ($P$). The peak value of the horizontal load during the first load cycle is $80\text{kN}$, from the second load cycle, the horizontal load was increased in increments of $20\text{ kN}$, at the same time, the pure control displacement ($\varepsilon$) of the place (in the Figure 4) where is the middle of length but the upside of the every specimen should be aware of, when $\varepsilon$ reach $1\text{mm}$, displacement control mode should be substituted for load control mode and the displacement cycles each has a increment of $1\text{mm}$. The test was stopped when the horizontal load dropped to $85\%$ of the maximal peak value or the specimen breached suddenly.

![Figure 4. Displacement arrangement](image)

2.3 Test Results
Test results for all the specimens are shown in Table 3.
Table 3. Test results for the Masonry Specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>GW1</th>
<th>GW2</th>
<th>GW3</th>
<th>GW4</th>
<th>GW5</th>
<th>GW6</th>
<th>GW7</th>
<th>GW8</th>
<th>GW9</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure mode</td>
<td>Shear compression</td>
<td>Shear slip</td>
<td>Shear slip</td>
<td>Shear compression</td>
<td>Shear slip</td>
<td>De-bonding</td>
<td>Shear compression</td>
<td>Shear compression</td>
<td>Shear slip</td>
</tr>
<tr>
<td>Ultimate load (kN)</td>
<td>215.6</td>
<td>251.8</td>
<td>264.2</td>
<td>254.1</td>
<td>265.0</td>
<td>222.1</td>
<td>242.4</td>
<td>228.0</td>
<td>229.0</td>
</tr>
</tbody>
</table>

3. THEORETICAL EQUATIONS OF SEISMIC SHEAR CAPACITY

3.1 Calculating Mode

The total shear load carried by the strengthened specimen $V_u$ is conventionally split into two components: the shear load carried by an equivalent unreinforced masonry specimen ($V_w$) and the increase in shear capacity due to strengthening ($V_{grf}$), so it can be expressed by the following superposition formula:

$$V_u = V_w + V_{grf}$$

![Figure 5. Calculating mode](image)

3.2 The Shear Contribution Provided by Unreinforced Wall ($V_w$)

According to studies of the Researching Group of Seismic Design Code on Brick Masonry Structure [11] the shear strength $f_s$ of unreinforced wall that is of the shear slip failure mode can be described as
\[ f_v = f_{vo} + 0.4\sigma_v \]  \hspace{1cm} (2)

where \( \sigma_v \) = vertical average normal stress and \( f_{vo} \) = average shear strength on condition that \( \sigma_v = 0 \).

As we can know that when the principle tensile stress of the masonry wall exceed its ultimate tensile strength, tensile cracks occurs. In this paper, a shear compression equation (3) was proposed by adding a statistical coefficient \( \beta \) in the conventional formula, based on the principle tensile stress theory.

\[ \frac{f_v}{f_{vo}} = \frac{1.0}{1.2} \sqrt{1 + \beta \frac{\sigma_v}{f_{vo}}} \]  \hspace{1cm} (3)

In the above Eq. (3), the value of statistical coefficient can be obtained as 2.10 by statistical analysis of correlative data from the many universities. Thus the formula (3) can be written as:

\[ f_v = \frac{f_{vo}}{1.2} \sqrt{1 + 2.10 \frac{\sigma_v}{f_{vo}}} \]  \hspace{1cm} (4)

After the determination of the shear strength formula ((2) or (4)), then come to the calculation of the shear area in which attention should be paid to the pilaster which had been damnified in the area of coupling to meet the need of reinforcement. Considering of the pilaster which weakened the coupling between the pilaster and its main body wall, the interaction coefficient of pilaster \( \psi \) is proposed, thus, the shear capacity of wall with pilaster \( V_{wb} \) can be expressed as

\[ V_{wb} = f_v (A_0 + \psi A_b) \]  \hspace{1cm} (5)

where \( A_0 \) = cross-sectional area of the main body wall and \( A_b \) = cross-sectional area of the pilaster. Substitute the expression (2) and (4) into the expression (5), and the shear capacity of the unreinforced wall with pilaster that is of the shear slip failure mode \( V_{wb-m} \) and that is of the shear compression failure mode \( V_{wb-l} \) both can be obtained as

\[ V_{wb-m} = \left( f_{vo} + 0.4\sigma_v \right) (A_0 + \psi A_b) \]  \hspace{1cm} (6)

\[ V_{wb-l} = \left( \frac{f_{vo}}{1.2} \sqrt{1 + 2.10 \frac{\sigma_v}{f_{vo}}} \right) (A_0 + \psi A_b) \]  \hspace{1cm} (7)
3.2 The Shear Contribution Provided by GFRP Sheet ($V_{gfrp}$)

Three series of reinforcement modes (horizontal, diagonal and mixed) were adopted in this test, which can be seen in Table 2 and in Figure 3. For the three modes, the corresponding formulas of calculating the shear capacity provided by GFRP sheet are as follows:

\[
\begin{align*}
T &= E_{gf} \varepsilon_{gt} t_{gf} b_{gt} \\
T_x &= E_{gf} \varepsilon_{gt} t_{gf} b_{gt} \cos \theta \\
T_y &= E_{gf} \varepsilon_{gt} t_{gf} b_{gt} \sin \theta
\end{align*}
\]

Considering the contribution of vertical component force $T_y$ to the shear capacity, indirect friction coefficient $\mu_{gf}$ is put forward. Then based on the equilibrium theory in mechanics, the shear capacity of a strip GFRP sheet ($V_{gfrp}$) can be obtained as follows.

\[
V_{gfrp} = T_x \mu_{gf} T_y
\]

Namely,

\[
V_{gfrp} = E_{gf} \varepsilon_{gt} t_{gf} b_{gt} (\cos \theta + \mu_{gf} \sin \theta) \quad (8)
\]

Generally, structures are reinforced symmetrically with more than one layer of FRP sheets. In order to help this state, the corresponding calculating Eq. (9) can be obtained by
linear superposition of Eq. (8).

\[ V_{grf-p-x} = \eta m E_{gf} t_{gf} \left[ \sum_{i=1}^{n} \varepsilon_{gfi-x} b_{gfi-x} \cos \theta_{gf} + \delta_{gfi} \mu_{gf} \sin \theta_{gf} \right] \] (9)

In the above expression, \( V_{grf-p-x} \) = shear capacity provided by all the inclined GFRP sheets, \( \eta \) = attenuation coefficient in regard to the number of layer of GFRP sheet, \( m \) = number of layer of GFRP sheet, \( b_{gfis} \) = width of inclined GFRP sheet, \( E_{gf} \) = modulus of elasticity of GFRP, \( t_{gf} \) = nominal thickness of GFRP, \( n_s \) = number of the strips of inclined GFRP sheet which are of the same direction in just one layer, \( \theta_{gf} \) = angle of tensile diagonal to horizontal, \( \delta_{gfi} \) = correction coefficient of \( \mu_{gf} \), \( \varepsilon_{gfi-x} \) = average strain of inclined GFRP sheet.

It was proposed in many technical literatures including some codes that the value of coefficient \( \mu_{gf} \) is 0.4, so in this paper \( \mu_{gf} = 0.4 \). Due to the fact that the increase of shear capacity provided by GFRP is not proportional to the amount of it, the attenuation coefficient \( \eta \) is brought forward and \( \eta = 1.0 \) when the number of layer of GFRP sheet is equal to 1. At the same time, the correction coefficient \( \delta_{gfi} \) is also put forward to take account of the effect of length of GFRP sheet on the shear capacity. It is known that the bigger the angle \( \theta \) is, the bigger contribution of the inclined GFRP sheet to the shear capacity can be obtained, so the correction coefficient \( \delta_{gfi} \) can be defined as follows:

\[ \delta_{gfi} = H_i / H \] (10)

where \( H \) = height of the specimen(wall), \( H_i \) = projection height of inclined GFRP sheet in the direction of height of the specimen (wall).

Provided that the inclined GFRP sheets are not existent in Figure 6, similarly, the shear capacity provided by all the horizontal GFRP sheets \( V_{grf-p-s} \) can also be obtained as:

\[ V_{grf-p-s} = \eta m \sum_{i=1}^{n_s} E_{gf} \varepsilon_{gfis} t_{gf} b_{gfis} \] (11)

where \( n_s \) = number of the strips of horizontal GFRP sheet, \( \varepsilon_{gfis} \) = average strain of horizontal GFRP sheet, \( b_{gfis} \) = width of horizontal GFRP sheet.

The shear capacity of the specimens that are of the mixed reinforced mode can be split into two components: the shear load carried by inclined GFRP sheets \( V_{grf-p-x} \) and that by horizontal GFRP sheets \( V_{grf-p-s} \). Based on the superposition method, the calculating formula for this mixed reinforcement mode can be written as Eq. (12)

\[ V_{grf-p-h} = \lambda_1 V_{grf-p-x} + \lambda_2 V_{grf-p-s} \] (12)

Substituting \( V_{grf-p-x} \) and \( V_{grf-p-s} \) into the above Eq. (12), the concrete expression of \( V_{grf-p-h} \) can be obtained as
\[ V_{\text{gfrp-h}} = \eta m E_g t_{gf} \left\{ \lambda_1 \sum_{i=1}^{n_h} e_{gfrp-h_i} b_{gfrp-h_i} (\cos \theta_{gfrp-h_i} + \delta_{gfrp-h_i} \tan \theta_{gfrp-h_i}) + \lambda_2 \sum_{i=1}^{n_h} e_{gfrp-h_i} b_{gfrp-h_i} \right\} \]  

(13)

where \( V_{\text{gfrp-h}} \) = shear capacity of the specimen that is of the mixed reinforced mode. Taking account of the non-linear superposition between \( V_{\text{gfrp-x}} \) and \( V_{\text{gfrp-s}} \), the working coefficient of inclined GFRP sheet \( \lambda_1 \) and that of horizontal GFRP sheet \( \lambda_2 \) were put forward. In this paper, the value of \( \lambda_1 \) and \( \lambda_2 \) are both assumed 1.0 temporarily.

3.3 Shear Capacity of GFPR-Reinforced Masonry Wall with Pilaster

The theoretical calculations which are appropriate to different reinforcement modes are obtained by combining the Eq. (5), Eq. (9), Eq. (11) and Eq. (12) as follows

\[ V_{u-x} = \zeta (V_{wb} + V_{\text{gfrp-x}}) \]  

(14)

\[ V_{u-s} = \zeta (V_{wb} + V_{\text{gfrp-s}}) \]  

(15)

\[ V_{u-h} = \zeta (V_{wb} + \lambda_1 V_{\text{gfrp-x}} + \lambda_2 V_{\text{gfrp-s}}) \]  

(16)

In the above expressions, \( V_{u-x} \) is shear capacity of masonry wall reinforced with inclined GFRP sheets and \( V_{u-s} \), \( V_{u-h} \) is that with inclined GFRP sheets and mixed GFRP sheets respectively. In terms of the effect coefficient \( \zeta \), \( \zeta = 1.0 \) under low reversed cyclic loading and \( \zeta = 1.1 \) under monotonic loading. For Eq. (14), when \( \lambda_1 = 1, \lambda_2 = 0 \), it is equivalent to Eq. (14) and when \( \lambda_1 = 0, \lambda_2 = 1 \), it is equivalent to Eq. (15), which indicates the Eq. (16) is of the function of Eq. (14) and Eq. (15). Therefore, the Eq. (16) can be acted as the general expression of calculating the shear capacity of the GFRP-reinforced masonry wall with pilaster. Substituting Eq. (6) and Eq. (7) into Eq. (16) respectively, the shear capacity of GFRP-reinforced masonry wall with pilaster with the failure mode of shear slip \( (V_{u-z-1}) \) and that with failure mode of shear compression \( (V_{u-z-1}) \) can be obtained as follows

\[ V_{u-z-1} = \zeta \left[ (f_{vo} + 0.4 \sigma_y)(A_0 + \psi A_b) + \lambda_1 V_{\text{gfrp-x}} + \lambda_2 V_{\text{gfrp-s}} \right] \]  

(17)

\[ V_{u-z-2} = \zeta \left[ \frac{f_{vo}}{1.2} \left( 1 + 2.1 \lambda_1 \frac{\sigma_y}{f_{vo}} \right)(A_0 + \psi A_b) + \lambda_1 V_{\text{gfrp-x}} + \lambda_2 V_{\text{gfrp-s}} \right] \]  

(18)

3.4 Interaction Coefficient of Pilaster \( \psi \)

In Figure 7, \( b \) and \( t \) is the length and the thickness of the wall with pilaster, respectively; \( b_0 \) is the width of the pilaster and \( t_0 \) is the thickness of the pilaster; \( x \) is the width of the conversional cross-section by equivalent lateral rigidity.
It can be seen from the Eq. (5) that the interaction coefficient of pilaster $\psi$ is one of important parameters related to the shear capacity. As it is known that the actual shear area of the specimen in which the pilaster is not damaged is maximum and $\psi$ reaches to the maximum ($\psi_{\text{max}}$), on the contrary, when the pilaster is damaged completed, $\psi$ reaches to the minimum ($\psi_{\text{min}}$). Based on the equivalent lateral rigidity the calculation of $\psi_{\text{max}}$ is as follows:

In order to convert the wall with the pilaster to the wall with the rectangular cross-section but with the same height and length of it, $x$ is needed to meet the conditions that the two walls are of the same lateral rigidity. According to advice of Chen Guoxing et al., the lateral rigidity of masonry wall can be obtained through the Eq. (19) as follows:

$$\frac{1}{K} = \frac{H^3}{12EI} + \frac{\gamma H}{AG}$$

where $A =$ cross-sectional area of wall, $I =$ moment of inertia of cross-section, $K =$ lateral rigidity of wall, $H =$ height of wall, $E =$ modulus of elasticity of masonry, $G =$ shear modulus of masonry, generally, $G = 0.4E$, $\gamma =$ unevenly distributed coefficient of cross-section stress, $\gamma = 1.2$ when cross-section is rectangular.

Assuming that the lateral rigidity of the wall with pilaster is $K_b$ and that of the equivalent wall is $K_w$, then the following Eq. (20) can be established

$$K_b = K_w(x)$$

The value of the $x$ can be obtained by solving for the above Eq. (20), and then the area of the equivalent wall ($A_w$) can be obtained, so the value of $\psi_{\text{max}}$ can be obtained by solving for the above Eq. (19) and Eqs. (21-22)

$$A_0 + \psi A_b = A_w$$

(a) Actual cross-section  (b) Conversional cross-section

Figure 7. Cross-section conversion equivalent lateral rigidity
\[
\psi_{\text{max}} = \frac{\lambda(3 + \delta^2) - tb}{t_0 b_0}
\]

where
\[
\delta = \frac{H}{b}
\]
\[
\lambda = \frac{1}{3 + \frac{H^2}{(t_0 b_0 + tb) + \frac{t_0 b_0^3 + tb_0^3}{(t_0 b_0^3 + tb_0^3)}}}
\]

As for \(\psi_{\text{min}}\), the value of it can be determined by separating the pilaster from the main body wall. Assuming that the lateral rigidity of the separating pilaster is \(K_{b0}\) and the lateral rigidity of the main body wall is \(K_{w0}\), then the calculation of can be obtained by Eq. (19) and Eqs. (23-24).

\[
\psi_{\text{min}} = \frac{K_{b0}}{K_{w0}}
\]

\[
\psi_{\text{min}} = \phi_i \phi_H \phi_b^3
\]

where
\[
\phi_i = \frac{t_0}{t}, \quad \phi_b = \frac{b_0}{b}, \quad \phi_H = \frac{(H^2 + 3b^2)}{(H^2 + 3b_0^2)}
\]

\(\psi_{\text{max}}\) and \(\psi_{\text{min}}\) is corresponding to two extreme state of the wall with pilaster respectively, introducing the burrow rate, the general expression of \(\psi\) can be expressed as follows

\[
\psi = \phi_i \phi_H \phi_b^3 + \left[ \frac{\lambda(3 + \delta^2) - tb}{t_0 b_0} - \phi_i \phi_H \phi_b^3 \right] \left[ 1 - \frac{\sum_{i=1}^N h_{ci}}{H} \right]
\]

where \(h_{ci}\) = height of the \(i\)th hole, \(N=\)total number of hole in pilaster. In this paper, the Eq. (25) are equivalent to the following Eq. (26)
\[
\psi = 0.041 + 0.873 \left[ 1 - \frac{\sum_{i=1}^{N} h_{ci}}{H} \right]
\]

(26)

3.5 Comparison between Test Results and Calculating Results
The comparison between test results and calculated results is shown in Table 4. It can be seen from Table 4 that the agreement between analysis and experiments is quite satisfactory.

Table 4. Comparison between test results and calculated results

<table>
<thead>
<tr>
<th>Data Origin</th>
<th>Serial Number</th>
<th>Failure Mode</th>
<th>Reinforcement Mode</th>
<th>Test Results (kN)</th>
<th>Calculated Results (kN)</th>
<th>Test Result Calculated Results</th>
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<tbody>
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<td>GW1</td>
<td>Shear compression</td>
<td>-----</td>
<td>215.6</td>
<td>213.6</td>
<td>1.01</td>
</tr>
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<td></td>
<td>GW2</td>
<td>Shear slip</td>
<td>Horizontal</td>
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<td>1.09</td>
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<td>GW5</td>
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<td>243.6</td>
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<td></td>
<td>GW7</td>
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<td>229.7</td>
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<td>GW8</td>
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<td>1.01</td>
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<td>GW9</td>
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<td>1.04</td>
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<td>Ref. [12]</td>
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<td>427.7</td>
<td>1.11</td>
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<td>BW10M10-2</td>
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<td>Diagonal</td>
<td>675.0</td>
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<td>0.93</td>
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</table>
3.6 Design Equations of Seismic Shear Capacity

In order to put the Eqs. (17-18) into practice, Eqs. (17-18) should be simplified. To accord with the current seismic code, the Eq. (27) that has been used in the current seismic code to calculate the shear capacity of unreinforced masonry wall is also adopted as design equation in this paper.

\[ V_{wb-d} = \zeta_N f_{vs} (A_0 + \psi A_h) \]  

(27)

where \( V_{wb-d} \) = design shear capacity of unreinforced wall with pilaster; \( \zeta_N \) = direct stress influence coefficient of shear strength of masonry, the value of it is based on the current seismic code; \( f_{vs} \) = design shear strength of masonry.

As for the design shear capacity provided by GFRP sheet, the corresponding calculating equation can be simplified based on the Eq. (13). Generally, FRP sheets are of the same category, arranged symmetrically, parallel to each other and with equidistance. In this test, GFRP sheet is only one layer \( (m = 1) \), so the attenuation coefficient \( \eta = 1.0 \). Currently, it is difficult to study the concrete influence of coupling of reinforcement mode on the shear capacity, so the value of \( \lambda \) is given to 1.0 temporarily. To be convenient, it is assumed that all the FRP sheets in the one specimen are of the uniform strain \( \varepsilon_w \), then it can be expressed as

\[ \varepsilon_w = \xi \varepsilon_u \]  

(28)

where \( \varepsilon_u \) = ultimate strain of fiber-reinforced plastic sheet, \( \xi \) = effective working coefficient of FRP sheet. Another important parameter in the course of simplification is the correction coefficient of \( \mu_{gf} (\delta_{gf}) \)

According to the definition of \( \delta_{gf} \), Eq. (10), not all the GFRP sheets are of the uniform value of \( \delta_{gf} \). To study the compositive effect, the uniform design value of the correction coefficient \( (\delta_{gf}) \) can be calculated as

\[ \delta_{gf} = \frac{\sum_{i=1}^{n_{gf}} (\delta_{gf i} \times n_{gf i})}{n_{gf}} \]  

(29)

where \( n_{gf i} \) = number of the FRP sheets whose \( \delta_{gf i} \) are uniform in one face of the masonry wall, \( n_{gf} \) = total number of the inclined FRP sheets in one face of the masonry wall. For the specimens reinforced by inclined FRP sheets, the values of \( \delta_{gf i} \) of all FRP sheets can form arithmetical progression, so based on this characteristic, Eq. (29) can be further simplified as follows

\[ \delta_{gf} = \frac{n_{gf} + 2}{2n_{gf}} = 0.5 + \frac{1}{n_{gf}} \]  

(30)
It can be seen from the above Eq. (30) that the value of $\delta_{gf}$ decreases with the increase of the value of $n_{gf}$ and when $n_{gf} = 1$, $\delta_{gf}$ reaches the maximum 1.0. If the Eq. (30) is limited, then the Eq. (31) can be obtained as follows

$$\lim_{n_{gf} \to \infty} \delta_{gf} = 0.5$$

The above Eq. (31) shows that when all the surface of one wall is covered with FRP, $\delta_{gf}$ reaches the minimum 0.5. For safety, the design value of is 0.5 regardless of the actual number of the FRP sheets.

Based on the above analysis and simplification, the design equation of shear capacity provided by FRP sheets ($V_{frp-d}$) can be obtained from the Eq. (13) as follows

$$V_{frp-d} = \xi E f E u \left[ \delta_s n_s A_{fs} + \delta_x n_s A_{fx} (\cos \theta + 0.2 \sin \theta) \right]$$

In the above expression, $E_f =$ modulus of elasticity of FRP, $A_{fs} =$ cross-sectional area of inclined FRP sheet, $A_{fs} =$ cross-sectional area of horizontal FRP sheet. $\delta_s$ and $\delta_x$ are both effect coefficient, when the reinforcement mode is horizontal $\delta_s = 1$, other reinforcement mode $\delta_s = 0$; when the reinforcement mode is diagonal $\delta_x = 1$, other reinforcement mode $\delta_x = 0$. The value of $\xi$ is correlative to the area reinforcement rate of FRP, in additional, it is also found in this test to be correlative to reinforcement mode. So based on the different reinforcement mode, corresponding equations to calculate the value of is regressed from test data as follows

$$\xi_s = -0.245 \ln(\rho) - 0.128 \quad (33)$$

$$\xi_x = -0.411 \ln(\rho) - 0.107 \quad (34)$$

where $\xi_s$ is effective working coefficient of horizontal FRP sheet, $\xi_x$ is effective working coefficient of diagonal FRP sheet, $\rho =$ area reinforcement rate of FRP. For mixed reinforcement mode, because of the insufficiency of test data, so the corresponding effective working coefficient can not be obtained in this paper.

Due to the uncertainty of direction of seismic force, one-way seismic force is neglected in seismic design, that is to say, $\zeta = 1.0$ in design equation, thus the overall design equation for FRP-reinforced masonry wall with pilaster can be written as follows

$$V_{u-d} = (V_{wb-d} + V_{frp-d}) / \gamma_{RE} \quad (35)$$

where $V_{u-d} =$ design ultimate shear capacity of FRP-reinforced masonry wall with pilaster, $\gamma_{RE} =$ adjusting coefficient of shear capacity.
4. COMPARISON BETWEEN RESULTS

It can be seen from the Table 5. That the test result is about two times as much as calculated result in design, which implies that the design Eq. (35) can be put into actual use.

<table>
<thead>
<tr>
<th>Serial Number</th>
<th>Reinforcement Mode</th>
<th>Area reinforcement rate</th>
<th>Test Results (kN)</th>
<th>Calculated Results in theory $F_t$ (kN)</th>
<th>Calculated Results in design $F_d$ (kN)</th>
<th>$\frac{F_t}{F_d}$</th>
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<tr>
<td>GW1</td>
<td>------</td>
<td>0</td>
<td>215.6</td>
<td>213.1</td>
<td>97.0</td>
<td>2.2</td>
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<td>Horizontal</td>
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<td>232.6</td>
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<td>112.2</td>
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<td>236.0</td>
<td>104.8</td>
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<td>106.6</td>
<td>2.3</td>
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<td>Diagonal</td>
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<td>224.7</td>
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<td>105.0</td>
<td>2.2</td>
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</table>

5. CONCLUSIONS

An effective method for calculating shear capacity of brick masonry wall with pilaster reinforced by GFRP has been presented in this paper. Based on the results of the experimental program, the following conclusions were reached.

(a) Strip FRP can be easily applied as masonry strengthening. It has been shown in this test that strip FRP strengthening increases the load capacity of masonry subjected to in-plane shear loading.

(b) A statistical coefficient $\beta$ was regressed from the data in this test and other tests and then a shear compression Eq. (4) for masonry structures was obtained based on the principle tensile stress theory.

(c) The concept of the interaction coefficient of pilaster $\psi$ is proposed and the concrete calculating expression (25) was obtained for the first time based on the
equivalent rigidity theory, which solves the difficult problem of calculating the shear area of masonry wall with pilaster. It is also founded in the course of analysis that pilaster is beneficial to the shear capacity but is limit.

(d) From Eq. (30), the correction coefficient \( \delta_{gf} \) decreases with the increase of \( n_{gf} \) when \( n_{gf} = 1 \) (diagonal strengthening), \( \delta_{gf} \) reaches the maximum 1.0. and when \( n_{gf} = \infty \) (the surface of one wall is all covered with FRP), \( \delta_{gf} \) reaches the minimum 0.5.

(e) The effective working coefficient \( \xi \) is not only corrected to the area reinforcement rate but also is corrected to reinforcement mode. The value of \( \xi \) for horizontal reinforcement mode \( (\xi_h) \) is generally smaller than that of for diagonal reinforcement mode \( (\xi_d) \) in the same loading state.

(f) The shear capacity of FRP-reinforced masonry wall with pilaster can be expressed by a simple superposition method and this suggested shear capacity calculation method has good agreement with the test results and enough safety.

The results presented in this paper are based on a limited number of large-scale GFRP-reinforced walls with pilaster tests. More tests can refine the results, but the qualitative conclusions remain valid.

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