REINFORCED CONCRETE SQUAT WALLS RETROFITTED WITH CARBON FIBER REINFORCED POLYMER

Shyh-Jiann HWANG¹, Tsung-Chih CHIOU² and Yaw-Shen TU³

ABSTRACT

For seismically insufficient buildings, to retrofit the RC partition walls using the Carbon Fiber Reinforced Polymer (CFRP) is of particular interest at the present juncture in Taiwan after Chi-Chi Earthquake. This paper describes theoretical and experimental studies related to the seismic retrofits of the RC frames containing walls using CFRP materials. Three “as built” RC frames with or without walls and two “retrofitted” RC frames with walls had been tested under simulated seismic actions. Experimental observations and theoretical analyses indicated that the shear resisting mechanism of the RC squat walls can be modeled as the struts and ties, and that shear strength of squat wall can be reasonably predicted by the softened strut-and-tie model. The test results of the retrofitted squat walls indicated that the CFRP with sufficient end anchorage is an effective retrofitting measure.

Keywords: carbon fiber reinforced polymer; compressive softened theory; reinforced concrete; seismic retrofit; shear strength; squat wall; strut-and-tie.

INTRODUCTION

In the literature, two major types of methods can be distinguished with regard to the modeling of distributed plasticity or spread of plasticity for steel structures, including plastic-hinge-based methods and plastic zone methods (also referred to as distributed plasticity or spread of plasticity methods). Methods of the former type include the conventional hinge-by-hinge analysis procedures, which concentrate the effects of member plastification using zero-length plastic-hinges located at the ends of beam-column elements. Although very simple, they often overestimate the ultimate strength of structures, especially for structures whose members are bent about their weak axes. Recently, there have been several modified plastic-hinge models being proposed with the aim of improving the accuracy. These include the notional-load plastic-hinge (Liew et al. 1994), tangent-modulus plastic-hinge (White 1993), and refined plastic-hinge methods (Liew et al. 1993). A detailed discussion of these three methods was given in White et al. (1993). According to this reference, the refined plastic-hinge method is the most accurate one. However, the above three methods were all developed for the calculation of the in-plane strength of structures; three-dimensional behaviors such as flexural-torsional effects cannot be accounted for.

Since these partition walls are detailed with temperature reinforcement only, which might result in insufficient strength. The common practice in Taiwan now is to tear down the partition walls then to recast with the RC structural walls. This retrofit scheme is time consuming and causes tremendous inconvenience to the residents. Seeking for the other alternative is of ever-increasing expectancy.

The repair of understrengthed or damaged reinforced concrete members by the external bonding of Carbon Fiber Reinforced Polymer (CFRP) laminates is becoming increasingly popular in the construction industry. The use of CFRP laminates for this application offers several desirable attributes, such as high strength, resistance to corrosion, light weight, and ease of handling. Retrofitting the RC partition walls by the bonding of CFRP might be a feasible solution of the aforementioned problem.

This paper describes theoretical and experimental studies related to the seismic retrofits of the RC frames containing walls using CFRP materials. Experimental tests were conducted on wall specimens that were subjected to reversed cyclic inelastic deformations representative of earthquake loadings. Theoretical analyses, based on the softened strut-and-tie model (Hwang and Lee 2002) and the ACI 318 building code (ACI 2002), were performed to evaluate the shear strength of the walls.

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EXPERIMENTAL PROGRAM

Test Specimens

In all, five large-scale isolated specimens, one frame and four walls, were tested. The test specimens are identified as PF, WF-12, WF-12-FV, WF-12-FHV, and WF-15. The dimensions and reinforcing details are given in Fig. 1. In order to focus attention on the failure behavior of walls, the ductile detailing requirements per ACI 318 building code (ACI 2002) was adopted for the design of frames for all specimens.

Figure 1  Geometry and Reinforcement Detail

Specimen PF was a pure frame, which intended to draw a comparison between frame and wall. The details of Specimen PF, shown in Fig. 1 (a), were used repeatedly for the frames of the other wall specimens. The test wall of Specimen WF-12 represented approximately 0.6 scale model of a prototype partition wall in a building. The test wall was 12 cm thick with 30 × 50 cm boundary elements. The overall length of the wall
was 350 cm and the height was 155 cm. The vertical and horizontal reinforcement in the wall of Specimen WF-12 was 0.2 percent of the wall cross section [Fig. 1 (b)], which corresponded with the area of shrinkage and temperature reinforcement.

Specimens WF-12-FV and WF-12-FHV had the same existing wall dimensions as WF-12 but retrofitted with CFRP laminates. Specimen WF-12-FV was strengthened with four layers of CFRP laminates, two layers for each side of wall. The fibers of CFRP laminates for Specimen WF-12-FV were placed in the vertical direction and the thickness of each layer is 0.1375 mm. Total of eight layers of CFRP laminates were bonded to Specimen WF-12-FHV, four layers in the horizontal direction and four layers in the vertical direction. The end anchorages of the CFRP laminates were carefully considered and revealed in Fig. 2.

![Figure 2: Detail of End Anchorage of CFRP](image)

Specimen WF-15 contains a structural wall of thickness of 15 cm. The vertical and horizontal reinforcement in the wall of Specimen WF-15 was approximately 0.5 percent of the wall cross section [Fig. 1 (b)], which provided a threshold of the qualified wall behavior.

The test specimens were cast vertically in timber molds.

Table 1 summarizes the material properties.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete</th>
<th>Steel</th>
<th>CFRP (MRL-T7-250)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wall &amp; Frame</td>
<td>Base</td>
<td>#3</td>
</tr>
<tr>
<td></td>
<td>$f'_c$ (MPa)</td>
<td>$f'_c$ (MPa)</td>
<td>$f_y$ (MPa)</td>
</tr>
<tr>
<td>PF</td>
<td>22.8</td>
<td>23.4</td>
<td>—</td>
</tr>
<tr>
<td>WF-12</td>
<td>21.0</td>
<td>23.4</td>
<td>446</td>
</tr>
<tr>
<td>WF-12-FV</td>
<td>22.1</td>
<td>23.4</td>
<td>446</td>
</tr>
<tr>
<td>WF-12-FHV</td>
<td>23.6</td>
<td>23.4</td>
<td>446</td>
</tr>
<tr>
<td>WF15</td>
<td>22.6</td>
<td>23.4</td>
<td>446</td>
</tr>
</tbody>
</table>
Test Setup and Procedure

The overall test setup is shown in Fig. 3. Horizontal load was applied with three double-acting servo-controlled actuators, 1000-kN capacity each. The top beams of the test specimens were clamped with two steel beams to simulate the mechanism that the lateral load is transferred from the strong diaphragm (Fig. 3). The footing was tied down to the test floor with 8 post-tension rods, and the horizontal movement of the footing was further restrained by two end reaction blocks. Fig. 4 presents the photo of the test setup in this study. The specimens were subjected to a reversed cyclic loading based on displacement control. The loading pattern for the specimens consisted of two cycles at lateral drift ratios, as shown in Fig. 5.
Applied horizontal forces were measured by calibrated load cells. Linear potentiometers were used to measure displacements. Strains in the reinforcement and CFRP were measured by means of electrical resistance strain gages.

**OBSERVED BEHAVIOR**

Horizontal force-displacement hysteretic response histories and final damage patterns for all specimens are shown in Fig. 6. All the test specimens were failed in shear. The failure mode of Specimen PF was the flexural shear failure of columns near the base. The failure mode of Specimens WF-12, WF-12-FV and WF-12-FHV was the diagonal compression failure in the wall web due to shear. Specimen WF-15 was failed due to the interface shear.

![Load versus Deflection Responses and Final Damage Patterns](image)

**Figure 6  Load versus Deflection Responses and Final Damage Patterns**
Table 2 reports the peak measured strengths, $P_{\text{test}}$, as well as the calculated flexural strengths, $P_y$ and $P_n$. Only the peak horizontal force of PF exceeded the calculated nominal flexural strength $P_n$. The maximum force-carrying capacities developed by other wall specimens were less than the calculated yield strength $P_y$, indicating insufficient shear strengths.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Measured $P_{\text{test}}$ (kN)</th>
<th>Calculated $P_y$ (kN)</th>
<th>$P_n$ (kN)</th>
<th>$P_{\text{SST}}$ (kN)</th>
<th>$P_{\text{test}}/P_{\text{SST}}$</th>
<th>$P_{\text{ACI}}$ (kN)</th>
<th>$P_{\text{test}}/P_{\text{ACI}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PF</td>
<td>740.1</td>
<td>539</td>
<td>668.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>WF-12</td>
<td>1664</td>
<td>2789</td>
<td>3048</td>
<td>1352</td>
<td>1.23</td>
<td>912</td>
<td>1.82</td>
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<tr>
<td>WF-12-FV</td>
<td>2215</td>
<td>3149</td>
<td>4692</td>
<td>1797</td>
<td>1.23</td>
<td>925</td>
<td>2.4</td>
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<tr>
<td>WF-12-FHV</td>
<td>2669</td>
<td>3171</td>
<td>4670</td>
<td>1861</td>
<td>1.43</td>
<td>1368</td>
<td>1.95</td>
</tr>
<tr>
<td>WF-15</td>
<td>2122</td>
<td>3378</td>
<td>4260</td>
<td>2088</td>
<td>1.02</td>
<td>1671</td>
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<tr>
<td><strong>MEAN</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>1.23</strong></td>
<td><strong>1.86</strong></td>
</tr>
<tr>
<td><strong>COV</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>0.12</strong></td>
<td><strong>0.22</strong></td>
</tr>
</tbody>
</table>

Fig. 7 shows the envelopes of load-deflection response for all specimens. By comparing Figs. 7 (a) and (b), it is clearly shown that the partition wall could greatly enhance the stiffness and the strength of a RC frame. As revealed in Fig. 7, the retrofitted specimens WF-12-FV and WF-12-FHV developed much higher strengths than the as-built walls WF-12 and WF-15. The test results indicated that the retrofit of RC partition walls by the external bonding of CFRP laminates is quite effective.

**Figure 7  Envelopes of Load-Deflection Response**

**ASSESSMENT OF WALL SHEAR FAILURE**

**Softened Strut-and-Tie Model**

A rational model for determining the shear strength of shear walls for seismic resistance has been developed by Hwang et al. (2001). The proposed model, called the softened strut-and-tie model, is based on the concept of struts and ties and derived to satisfy equilibrium, compatibility, and constitutive law of cracked reinforced concrete.

Three strut-and-tie load paths (Hwang et al. 2001) are proposed to model the force transfer within the wall, and they are the diagonal, horizontal, and vertical mechanisms as depicted in Fig. 8. The diagonal mechanism is a diagonal compression strut. The vertical mechanism includes one vertical tension tie and two steep compression struts, and the horizontal mechanism is composed of one horizontal tension tie and two flat compression struts. For diagonal compression failure, the shear strength of the wall is defined as the concrete compressive stress on the nodal zone reaching its capacity. The softened strut-and-tie model was found to reproduce the available test results of shear walls from the literature with reasonable accuracy (Hwang et al. 2001).
In order to facilitate the routine design, a simplified approach of the softened strut-and-tie model was also developed (Hwang and Lee 2002). It is found that the simplified model is a useful and practical tool for determining the shear strength of the walls failing in diagonal compressions. More details of the simplified softened strut-and-tie model are presented elsewhere by Hwang and Lee (2002).

### Evaluation of Test Results

It was reported that the walls with a barbell or flanged section have a strength significantly higher than that of a rectangular section (Hwang et al. 2001). The higher strength for the wall with boundary elements is attributed to the improved end conditions of its diagonal strut provided by the compression boundary element. Web crushing usually occurs in the compressive struts that intersect the compression boundary element at the wall base. Therefore, load carried by crushed struts can be transferred to higher or lower struts depending on the stiffness of boundary element (Oesterle et al. 1984).

In light of above argument, the damage patterns of the boundary elements were clearly documented as shown in Fig. 9. The depth of the diagonal strut $a_y$ of the softened strut-and-tie model is then defined as

$$a_y = \sqrt{a^2 + b^2}$$  \hspace{1cm} (1)

where $a$ is the depth of the compression zone at the base of the wall, and $b$ is the depth of the compression zone provided by the boundary element. In this paper, $a$ was determined by the sectional analysis for the stage when the extreme tensile steel reaches yielding, and $b$ was estimated as (Hwang et al. 2004).

$$b = \frac{1}{5} \frac{h_b}{H_w} (7b_h h_b + \ell_n t_w) \leq \frac{H_w}{2}$$  \hspace{1cm} (2)

where $h_b$ is the depth of the boundary element; $b_h$ is the width of the boundary element; $H_w$ is the clear height of wall web measured from the reaction beam to the wall base; $t_w$ is the thickness of wall web; and $\ell_n$ is the clear length of the wall web in the direction of the shear force applied.

Note: $b = \text{measured value}$

(calculated value by Eq (2))
The angle of inclination $\theta$ of the diagonal compression strut becomes flatter due to the participation of the boundary element. According to Fig. 10, the value of $\theta$ is defined as

$$
\theta = \tan^{-1}\left(\frac{h - \frac{b}{3}}{\frac{l}{3}}\right)
$$

(3)

![Figure 10  Angle of Inclination for Strut-and-Tie Modeling](image)

The shear strengths of test walls were calculated using the softened strut-and-tie model ($P_{SST}$ in Table 2; Hwang and Lee 2002) and the ACI 318 method ($P_{ACI}$ in Table 2; ACI 2002). The average strength ratio ($P_{test} / P_{SST}$) for the softened strut-and-tie model is 1.23 with a coefficient of variation (COV) of 0.12. The mean value and the coefficient of variation of the test-to-calculated shear strength ratio by ACI equations were found to be 1.86 and 22%, respectively (Table 2). As seen in Table 2, the softened strut-and-tie model (Hwang and Lee 2002) predicts the failure shears more accurately than the equation of the ACI 318 building code (ACI 2002).

CONCLUSIONS

The test results of the retrofitted squat walls indicated that the CFRP with sufficient end anchorage is an effective retrofitting measure. Experimental observations and theoretical analyses indicated that the shear resisting mechanism of the RC squat wall can be modeled as the struts and ties, and that the shear strength of squat wall can be reasonably predicted by the softened strut-and-tie model.

As an improvement to current wall retrofit design methodology, it is recommended that the softened strut-and-tie model be used to assess the shear strength of RC walls, and that the insufficient RC partition walls be retrofitted by using CFRP materials.

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REFERENCES


