SEISMIC UPGRADING OF EXISTING RC ORDINARY MOMENT RESISTING FRAMES USING FRPS

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ABSTRACT

Upgrading of existing RC Ordinary Moment Resisting Frames (OMRF) (in which the ductility capacity of plastic hinges to accommodate the predicted ductility demand is questionable) into a Ductile Moment Resisting Frame (DMRF) is desirable in seismic regions. One possible method of upgrading is relocation of plastic hinges away from the column face toward the beam. This paper presents the results of an experimental study carried out at the University of Queensland in order to examine the capability of CFRP sheets in doing this task. The proposed method called “web-bonded FRP” works by bonding FRP sheets of specific lengths to the sides of the beam’s ends of reinforced concrete beams leading to the joints. In this study, five 1/2.2 scaled-down RC exterior joints of a typical Ordinary Moment Resisting Frame (OMRF) were tested under monotonic/cyclic loads. Two specimens were used as control specimens while the other three were strengthened with CFRP sheets of different lengths. All specimens were loaded under a displacement-controlled regime to failure. The load-deflection and hysteresis curves, modes of failure and ductility gain are presented in this paper. The results show that carbon fibre sheets (when carefully designed) can effectively relocate the plastic hinge away from the face of the column.

KEYWORDS

Carbon fibre sheets, reinforced concrete joints, plastic hinge.

INTRODUCTION

Designing beam column joints is considered to be a complex and challenging task for structural engineers and careful design of joints in RC frame structures is crucial to the safety of the structure. Hence, particular attention to the ductility of the reinforcement within the joint region is necessary. If the joint is not designed properly, the possibility of plastic hinge formation in the columns increases substantially. This is dangerous for two reasons; firstly the collapse mechanism associated with hinges in the columns has a lower ultimate load and secondly the energy absorbance of plastic hinges within the columns is normally less due to reinforcement arrangement and the axial load. Engineers can avoid this when designing Ductile Moment Resisting Frames (DMRFs) by employing the principle of strong-column weak-beam design. According to this design principle, joints are designed in such a way that the joint region and the column remain essentially elastic under the action of high lateral loads such as earthquake and high-pressure winds while the main energy dissipation occurs within the plastic hinges formed in the beams. By controlling the plastic hinge location the beam will perform in a ductile manner. Attempts have been made in the past to develop methods of relocating a plastic hinge away from the column face. Most of the proposed methods for example, Joh et al. (1991) and Paulay and Priestley (1992), however, have been on detailing of reinforcing bars, which can only be utilised in new construction as is shown in Figure 1 (a). In this way stresses in the reinforcement do not exceed the yield stress at the face of the column, while strain hardening develops at the critical section of the plastic hinge. A minimum distance equal to beam height, \( h_b \), or 500 mm, whichever is less, is recommended. Because the nature of shear transfer across the critical section of a plastic hinge is complicated, care must be taken with detailing of the shear reinforcement. As Figure 1 (b) indicates, the beam’s shear force \( V_b \) introduced to region A by the diagonal forces has to be transferred to the top of the beam in region B. This necessitates stirrups extending between these two regions, perhaps supplemented by specially bent top beam bars, to carry the entire shear force.

Many already existing moment resisting frames do not possess correct joint reinforcement detailing as they have been designed based on older codes. A different method which can upgrade the joints of these frames in an efficient and cost effective way is consequently desirable. Smith and Shrestha (2006) carried out a systematic review of experimental research on the FRP-strengthening of RC connections and an evaluation of the
effectiveness of the strengthening schemes. Smith and Shrestha (2006) reported that four different types of FRP-strengthening including Shear Strengthening, Anchorage Strengthening, Shear and Anchorage Strengthening and Plastic Hinge Relocation have been introduced into connections.

This paper presents results of an experimental investigation into the effectiveness of FRP wraps in Controlling the Location of a Plastic Hinge in an OMRF. The proposed method is to stick carbon FRP sheets of specific lengths to the two sides of the beams of a reinforced concrete joint (i.e. web-bonded FRP). Due to the presence of floor slab, the installation of the FRP plates and sheets on the tension face of the beam is hard, whereas using the web-bonded FRP system is easy and practical. The performance of this method has been investigated by the authors and is presented in the following. Other aspects of the investigation can be found in papers (Mahini et al., 2004a, 2004b, 2006). In addition, in another research carried out by the authors (Mahini et al., 2007), it is concluded that the improvement in ductility and performance level of the existing RC frames is better for frames retrofitted with web-bonded FRPs than the steel-braced systems.

EXPERIMENTAL PROGRAM

The prototype structure is a typical eight story residential RC building located in Brisbane, Australia. The controlling design criterion for this structure is the strength required to resist the applied gravity and lateral loads. The prototype was designed as an OMRF, according to the Australian Concrete Code AS3600-2001 with details similar to non-ductile RC frames designed to ACI-318-2002. A scaled-down frame was modelled by the application of the similitude requirements that relate the model to the prototype using the Buckingham theorem (Noor, 1992). Four N12 rebars (\( \phi =12 \) mm) were used for both the column vertical reinforcement and the beam longitudinal reinforcement. R6.5 bars (\( \phi =6.5 \) mm) were used for stirrups at a spacing of 150 mm in both beam and column. A 30 mm concrete cover was considered for the beam and column reinforcements which is about half of the corresponding covers in prototype. An exterior beam-column sub-assembly was then isolated at the first floor level of the scaled down frame. The scaled-down joints were extended to the column mid-height and beam mid-span, corresponding to the inflection points of the bending moment diagram under lateral loading. For specimens that were tested under cyclic loads, the bottom beam reinforcements should be bent toward the joint core similar to the top beam bars; instead, in order to facilitate caging within the joint core, U-bars were used for beam reinforcements. The experimental program consisted of testing five 1/2.2 scale exterior RC beam-column joints. Figure 2 (a) summarizes all the test specimens’ details. All joints consisted of 180 mm wide and 230 mm deep beams with 220 mm \( \times \) 180 mm columns. Yield strength of the main steel reinforcements, N12 was around 500 MPa and the modulus of elasticity was equal to 200 GPa. The stirrups had a yield strength of 382 MPa and a modulus of elasticity of 200 GPa. Ties were also placed in the joints region in accordance with the requirements of AS3600. Additional stirrups and ties and N16 (\( \phi =16 \) mm); threaded rods were placed near the ends of the beam and column in all specimens to ensure that local failure does not occur at the load and support points respectively.

One plain Specimens (CSM0) as a control specimen was constructed and was tested under monotonic load to failure. The Specimen RSM1 was then retrofitted with one ply CFRP sheet at a distance of 350 mm from the column face as is illustrated in Figure 2(a) and was tested under monotonic loads to failure. Carbon Fibre Reinforced Polymer (CFRP) sheets that were used in all experiments were uni-directional. CFRP was an MBrace Fibre sheeting from MBT (2002), which possesses a tensile strength of about 3900 MPa, a modulus of elasticity of 240 GPa and an ultimate tensile elongation of 1.55%. The MBrace Saturant (adhesive) was also used for warping the MBrace sheet using a wet lay up method. The saturant had a minimum tensile strength of 50 MPa.
and a minimum compressive strength of more than 80 MPa. It’s modulus of elasticity was around 3000MPa and its ultimate elongation around 2.5%. The second Specimens RSM2 (see Figure 2 (a)) was retrofitted with three plies of 200 mm CFRP, and was tested under the same loading regime as Specimens RSM1. In order to examine the ability of FRP web-bonded system to retrofit the beam-column joints under cyclic loads, one plain Specimens (CSC1) as a control specimen and the Specimens RSC1 which retrofitted with three plies CFRP sheet at a distance of 200 mm were constructed and tested under cyclic load to failure. The concrete had a compressive strength around 40, 41, 45, 41, and 37 MPa and a modulus of elasticity around 27.6, 30.1, 29.4, 30.9, and 29.7 GPa for plain (CSM0 and CSC1) and retrofitted Specimens (RSM1, RSM2 and RSC1) respectively.

The testing rig was built up into a 3 m by 3 m rigid frame (see Figure 2 (b)). In addition, the joint details for specimens under monotonic and cyclic loads are shown in Figures 2 (c) and 2(d) respectively. The joint was rotated 90° so that the column was parallel and the beam was perpendicular to the ground. The free end of the beam was loaded and the connection between the beam and the actuator was carefully designed in order to ensure that the load could be applied in a push-pull manner. Axial loading of the column was simulated by stretching four high-strength low-elongation steel bars that were placed outside the column by the use of a hydraulic jack while a load cell was utilised to measure the applied load. The column was supported at each end with specially designed supports that ensured their ends were free to rotate but not to translate. In the first three tests, the loading was applied monotonically by an actuator which was capable of applying cyclic loads for the last two specimens in both load and displacement control regimes. The load was applied first in a load control regime and when the load reached the theoretical first yield, subsequent tests were then performed using a displacement control regime from the ductility level of one, then 2, then 3 and continued up until failure.

![Image](image_url)

Figure 2. (a) Specimen’s details; (b) test set-up; (c) joint details for specimens under monotonic loads; (d) joint details for specimens under cyclic loads
EXPERIMENTAL RESULTS AND DISCUSSIONS

The plain control specimen was tested to failure in order to determine its capacity, flexural stiffness, and failure mode. After loading, flexural cracks were developed at the beam-end close to the column face and some cracks were propagated into the joint core and subsequently collapsed (see Figure 3). The retrofitted Specimen RSM1 with only one CFRP sheet on the web exhibited a brittle failure. At ultimate, concrete crushing occurred at the face of the column, which followed by the beam rotation about the fulcrum, and the rupture of CFRP on the tension side. In contrast, Specimen RSM2 exhibited a ductile failure as plastic hinges were relocated from the column face to the cut-off point of CFRP as shown in Figure 3. The beam’s tip load versus displacement curves for Specimens CSM0, RSM1 and RSM2 are shown in Figure 4. Table 1 shows that the maximum strengths and the estimated ductility of all specimens are tabulated. As is seen, the maximum loads recorded in these tests were about 24.64, 24.70, and 21.12; for Specimens CSM0, RSM1 and RSM2 all in kN respectively. As is shown, the maximum load strength recorded for Specimen RSM1 is about that of the plain Specimen CSM0 whereas that for Specimen RSM2 is only 86% of the maximum strength of plain Specimen CSM0. It may relate to the fact that the retrofitting length of this specimen is less than that of specimen RSM1 and therefore the load carrying capacity is reduced. Another evidence indicating the satisfactory behaviour of the retrofitted specimens is their displacements ductility. The displacement ductility factor required of typical structures may vary between 1 for elastically responding structures to 6 for ductile structures. For structures in which ductility is controlled by flexural plastic hinging of members, for instance the retrofitted Specimens RSM1 and RSM2, the displacement ductility is limited by the special manner of failure. In Specimen RSM1 in which the beam end was retrofitted only with one ply of CFRP sheet, the load capacity of the specimen was improved well, although the post peak reduction was not smooth as CFRP ruptured at the column face and the specimen failed by concrete crushing that followed that rupture (see Figures 3, 4). In contrast, as is seen in Figure 3, Specimen RSM2 did not exhibit a sudden failure and the CFRP warp remained intact to failure. The ductility factor for the Specimen RSM2 is equal to 5.7 which is about that of the plain Specimen CSM0.

Table 1

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSM0</td>
<td>24.64</td>
</tr>
<tr>
<td>RSM1</td>
<td>24.70</td>
</tr>
<tr>
<td>RSM2</td>
<td>21.12</td>
</tr>
</tbody>
</table>

Figure 3. Ultimate failure of Specimens CSM0, RSM1, RSM2, CSC1 and RSC1.
Examining the hysteretic behaviour of the Specimen CSC1 showed some pinching, stiffness degradation during the test and strength deterioration in the final two cycles followed by spalling of the concrete and the subsequent full buckling of the steel reinforcements at the column face, as shown in Figures 3 and 4. In Specimen RSC1, the loading continued to the seventh cycle without any debonding of FRP sheets and only then the first signs of debonding were observed at the mid-height of the beam close to the column face. In addition, examining the hysteretic loops of the Specimen RSC1 shows that the energy absorbing capacity is maintained as the maximum loads hold to a healthy plateau up to the end. No considerable pinching, stiffness degradation or strength deterioration was observed in the final two cycles as was the case for the plain RC specimen. This obviously is a result of plastic hinge region being confined by the FRP sheet to some extent. Figures 3 and 4 show the failure mechanisms and beam-tip load versus displacements of Specimens CSC1 and RSC1 respectively and Table 1 contains the estimated displacement ductility factor for these specimens. As is seen, an almost similar displacement ductility were calculated for these specimens whereas the failure of Specimen RSC1 is located further from the beam end and joint core indicating that the plastic hinge in this specimen is relocated from the joint core toward the beam under cyclic loading regime and therefore the performance of the proposed frame can be converted into a ductile manner.

It should be mentioned that in all tested specimens, no considerable cracks occurred beyond the plastic hinge locations within the beam.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading type</th>
<th>No. of CFRP plies</th>
<th>CFRP wrapping length mm</th>
<th>Maximum strength (kN)</th>
<th>Displacement ductility factor</th>
<th>Failure mechanism</th>
<th>Failure location</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSM0</td>
<td>Monotonic</td>
<td>-</td>
<td>-</td>
<td>24.64</td>
<td>5.8</td>
<td>Ductile</td>
<td>Beam end</td>
</tr>
<tr>
<td>RSM1</td>
<td>Monotonic</td>
<td>1</td>
<td>350</td>
<td>24.70</td>
<td>-</td>
<td>Sudden</td>
<td>Beam end</td>
</tr>
<tr>
<td>RSM2</td>
<td>Monotonic</td>
<td>3</td>
<td>200</td>
<td>21.12</td>
<td>5.7</td>
<td>Ductile</td>
<td>Within the beam</td>
</tr>
<tr>
<td>CSC1</td>
<td>Cyclic</td>
<td>-</td>
<td>-</td>
<td>19.52</td>
<td>6.5</td>
<td>Ductile</td>
<td>Beam end</td>
</tr>
<tr>
<td>RSC1</td>
<td>Cyclic</td>
<td>3</td>
<td>200</td>
<td>21.32</td>
<td>6</td>
<td>Ductile</td>
<td>Within the beam</td>
</tr>
</tbody>
</table>

Figure 4. Beam’s tip load vs. displacement for all specimens.
CONCLUSIONS

This paper shows that web-bonded CFRP-retrofitting technique can be used to relocate the beam plastic hinging zone away from the column face in order to upgrade of the existing RC Ordinary Moment Resisting Frames (OMRFs) into a Ductile Moment Resisting Frame (DMRF). Based on the monotonic and cyclic behaviour, it is concluded that: (1) the CFRP-retrofitted specimen developed flexural yielding at the expected critical section. (2) The proposed web-bonded FRP allowed the energy dissipation in the hysteretic behaviour to occur in a ductile manner.

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REFERENCES

ACI318, (2002). Building Code Requirements for Structural Concrete, American Concrete Institute.