PLASTIC HINGE RELOCATION IN RC JOINTS USING FLANGE-BONDED FRP SHEETS

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ABSTRACT

To increase the seismic performance of an existing RC frame, retrofitting beam-column joints with FRP can be very effective. Such retrofitting is generally carried out with the aim of relocating the plastic hinge away from the face of the column so that a more ductile failure may be achieved. The present paper reports on the results of experimental and numerical studies carried out in order to assess the ability of CFRP sheets to relocate plastic hinges away from the joint and into the beam in intermediate ductility RC frames. For this purpose, two full-scale RC exterior joints are constructed and tested under monotonic loads. Examination of the retrofitted specimen after testing showed that the retrofitting method successfully shifted the plastic hinge away from the face of the column. Appropriate detailed finite element models of the test specimens are also developed and nonlinear pushover analyses are carried out to further study the effects of some problem variables such as the length and thickness of FRP retrofitting sheets. Results of the experimental and numerical investigations show that the FRP flange-bonded retrofitting configuration investigated is able to increase the strength capacity of the joint by 23% which constitute a large increase in capacity. The ability of the investigated retrofitting scheme to relocate the plastic hinge is shown to be dependent on the thickness of the FRP overlay.

Keywords: Seismic retrofitting; Exterior RC joint; FRP; pushover analysis; plastic hinge.

1. INTRODUCTION

In an RC frame, to increase the seismic performance of beam-column joints, different FRP retrofitting schemes may be adopted including; web-bonded and flange-bonded configurations. The vast amount of research conducted on this subject in the last decade has been dedicated to the web-bonded scheme. However, Web-bonded retrofitting schemes suffer from being impractical in the actual 3D frames due to the presence of cross beams and the integrated slab at the joint. In a numerical study, Parvin and Granata (2000) investigated three different flange-bonded configurations; differing only in the use of fibre wraps to avoid debonding. They noted that a large increase in the joint moment capacity, prior to formation of plastic hinge, is achieved through flange-bonded FRP overlays. Later, Granata and Parvin (2001) experimented with FRP retrofitted joints. They confirmed their earlier findings regarding the increased moment capacities of the joints. Another early work is due to Mosallam (2000) dealing specifically with repairing damaged RC joints with epoxy resins and FRP laminates. He concluded that the repair scheme increases the joint’s strength beyond its undamaged capacity. Later, Ghobarah and Said (2002) evaluated RC joints rehabilitated with different FRP composites, including U-shaped and X-shaped configurations, concluding that all the selected schemes were successful in changing the non-ductile shear failure mode of the joint into a ductile flexural mode. Pantelides et al. (2000) also tested half-scale beam-column joints retrofitted by FRP overlays and reported substantial increase in strength and improved performance regarding ductility and drift. In another experimental study, Attari

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et al. (2010) evaluated the effect of fiber reinforced composites on strengthening beam–column joints. They concluded that a combination of carbon and glass fiber reinforced polymers could improve the shear strength and ductility of deficient beam–column joints.

Karayannis and Sirkelis (2007) combined epoxy resin injection with FRP jacketing to rehabilitate RC beam-column joints. Their experiments showed that the use of epoxy resin can even restore the strength of a large-scale damaged joint and that application of FRP sheets can provide substantial further improvements on both load-carrying capacity and ductility. They also noted that failure in the retrofitted joints occurred outside the retrofitted area. Antonopoulos and Triantafillou (2003) also tested a number of scaled joints strengthened with different FRP configurations. The test results were dominated by partial or full debonding of composites. However, their tests showed increases in the strength from 65% to 85% and enhancement in the energy dissipation capacity from 50% to 70%, as well as, increases in the stiffness up to around 100%. Mostofinejad and Talaeeitaba (2006), on the other hand, developed numerical models for nonlinear analysis of RC joints retrofitted with FRP overlays. Their results showed that the numerical model matched the experimental work with good accuracy.

Mahini and Ronagh (2007) tested scaled beam-column RC joints web-bonded with FRP with the aim of relocating the plastic hinge away from the column face. Their experimental studies showed that the FRP web-bonding scheme can restore/upgrade the integrity of the joint, keeping/upgrading its strength, stiffness and ductility, as well as, shifting the plastic hinge from the column face further into the beam. The effectiveness of using web-bonded FRPs on plastic hinge relocation has also been reported by Smith and Shrestha (2006). Later, in an experimental study, Balsamo et-al. (2005) evaluated the seismic behaviour of a full-scale RC frame repaired using CFRP laminates. They indicated that the repaired frame had a large displacement capacity without exhibiting any loss of strength, while providing almost the same energy dissipation capability of the original frame. In a numerical study, Niroomandi et al. (2010) also carried out a study on the effectiveness of FRP web-bonding of joints in relocating the plastic hinge and in enhancing the seismic performance level of an RC frame. They compared the results of retrofitted joint at web by FRP sheets with those obtained from retrofitting the same frame using a steel X-bracing scheme (Maheri & Akbari, 2003) and found that both retrofitting schemes have comparable abilities to increase the frame behaviour factor, R; the former comparing better on the ductility component and the latter on the over strength. Later, similar studies conducted on joint web-bonded scheme by Hadigheh et al. (2013, 2014) indicated similar results.

Web-bonded schemes, constituting full wrapping of FRP around members, has practical limitations in real 3D RC frames due to the presence of slab and cross beams. Considering these limitations, Zarandi and Maheri (2015) introduced a new flange-bonded RC joints retrofitting scheme which can be applied to both 2D and 3D frames and also allows for the presence of slab in the joint area. They carried out numerical studies to investigate the effect of their flange-bonded scheme in relocation the plastic hinge into the beam. Their results showed that the flange-bonded scheme is more superior than the web-bonded scheme regarding capacity, ductility and the performance level, as well as, cost.

In the present paper, the flange-bonded scheme proposed by Zarandi and Maheri (2015) is further studied both experimentally and numerically with the aim of relocating the plastic hinge location away from the joints core into the beam.

## 2. TESTS SETUP AND RESULTS

### 2.1 Test Setup

A typical exterior beam-column joint of a four-storey moderate ductility RC Moment Resisting Frame (MRF) was considered for investigations. The frame was designed according to ACI code of practice (2008). Two identical full-scale specimens of the selected joint were then constructed based on the design details. One of the specimens (CIS1) was considered as a control specimen and the other specimen (RIS1) was retrofitted with the FRP flange-bonded scheme. Dimensions and reinforcement details of the two specimens are shown in Figure 1. The joint beam and columns have cross-section dimensions of 300×300 mm. The longitudinal reinforcement used in the columns were 8 ɸ16 bars corresponding to a 1.78% reinforcement ratio. The transverse reinforcement in the column was ɸ 10 closed rectangular ties, spaced at 120 mm inside the joint and for 450 mm immediately above and
below the joint and spaced at 190 mm for the rest of the column height (See Figure 1). The beams
length was 1500 mm from the face of the column at its free end with a cross-section of 300×300 mm.
All beams are reinforced with, two, 20 mm diameter bars at the top and at the bottom of the beam. The
transverse reinforcement of the beam was $\phi$ 8 rectangular stirrups starting at 50 mm from the face of
the column. The stirrups were spaced at 50 mm for the first 600 mm from the face of the column and
then at 120 mm for the remaining 900 mm, ending at 50 mm from the free end of the beam. The
average yield strength of reinforcing steel bars and ties are 400 MPa and 340 MPa, respectively;
obtained from direct tensile strength tests. The compressive strength of concrete was measured at 25.3
MPa, using standard cylinder test data.

![Figure 1. Dimensions and reinforcement details of the selected exterior RC joint.](image)

The RC column was subjected to a constant axial load of 225 kN, prior to application of the incrementally-
applied, quasi-static, pushover loading of the beam tip, to simulate the actual gravity load in the column,
obtained from the frame’s analysis. This load continued to be applied throughout the pushover testing. As
it is shown in Figure 3, to be able to use the best capabilities of the loading frame and the loading
actuators, joint specimens were rotated 90° so that the column was parallel and the beam was
perpendicular to the ground. Other details of the test set-up are also shown in Figure 3. To simulate the
inflection points at the centers of the upper and lower columns, the joint’s column is supported at each end
with specially designed supports that ensured their ends are free to rotate. This special hinge support is
designed to simulate the real performance at the inflection points as well as enabling the constant load to
be applied to the column. The test frame is equipped with a hydraulic actuator with the capacity of 200 kN
and maximum travel of $\pm$150 mm and a hydraulic jack with a maximum capacity of 300 kN.
The measured loads were the column’s axial load and the load applied to the tip of the beam. Also, displacements at the tip of the beam, as well as at mid-length of the beam were measured using linear variable displacement transducers (LVDT) located at the said positions (See Figure 4). A third LVDT was used to control the rigid body motion of the joint. Strains in the reinforcing steel and concrete at different locations during the tests were also extracted using electrical resistance strain gauges, bonded to the steel bars at pre-prepared locations for smooth contact. The locations of strain gauges on the longitudinal and transverse reinforcement bars in the beam and column and concrete strain gauges are shown in Figure 4.
2.2 Test Results

Each specimen was in turn subjected to monotonically increasing static pushover load to failure and the data from the load cells, LVDTs and strain gauges were recorded through a data logger. The recorded beam tip load versus tip displacement of the non-retrofitted control joint specimen (CIS1) is plotted in Figure 5. As it can be seen, the response is almost linear up to about 13 kN, after which flexural cracks developed in the beam near the face of the column. Nonlinear response of the joint continued with formation of other flexural cracks up to the peak load of 49.8 kN, considered as the ultimate capacity of the joint. The ultimate load corresponded to a displacement of about 91.8 mm. A look at the force-displacement capacity curve of this specimen shows a somewhat ductile behaviour post the yield point.

The retrofitted specimen RIS1 was tested under the same loading regime as CIS1. First flexural cracking of the beam section appeared at a beam tip load of 14 kN, at the cut-off point of the warp (loading step 1, marked in Figure 6) about 300 mm away from the face of column. This load is only marginally higher than the load causing the first crack in the control join CIS1. Soon after this first crack, additional flexural cracks developed at the end of the beam adjoining the column face (loading step, marked 2 in Figure 6). With increasing pushover load, flexural cracks developed in other parts of the beam and the crack at the face of the column further widened. At higher loads, diagonal shear cracks also developed at the end of the beam, however, no significant cracking was observed in the joint’s core (column section). The force-displacement capacity curve for the retrofitted specimen is compared with that of the control joint in Figure 5. The final load recorded for this specimen was 61.4 kN which, when compared with the ultimate capacity of the non-retrofitted control specimen (CIS1), shows an increase in strength of about 23%. The test results show that, although a respectable increase in strength was achieved due to this retrofitting scheme, formation of the major flexural crack at the face of the column indicates that relocation of the plastic hinge away from the column face and towards the end of the wrap did not occur. Indeed, at the end of the test, it was observed that the wrapping around the column close to the face of the beam had experienced some rupture indicating inadequacy of the 5-layer FRP configuration in confining the critical location of column-beam interaction.

The ultimate load, $P_u$, maximum displacement, $\Delta_u$, and maximum strains recorded for the steel reinforcement and concrete surface for the two specimens are compared in Table 1. The strains listed...
in this Table also clearly show that the plastic hinge occurred at the column face and not at the end of the FRP overlay.

![Figure 5. Experimental force-displacement pushover capacity curves of control joint (CIS1) and retrofitted joint (RIS1)](image)

![Figure 6. Final crack pattern for specimen RIS1](image)

Table 1. The strength properties of the tested joint specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Bar Strain</th>
<th>Max Concrete Strain</th>
<th>P_a (kN)</th>
<th>Δ_max (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column face</td>
<td>End of FRP</td>
<td>Column face</td>
<td>End of FRP</td>
</tr>
<tr>
<td>Control (CIS1)</td>
<td>6.57E-03</td>
<td>_</td>
<td>-3.51E-03</td>
<td>_</td>
</tr>
<tr>
<td>Retrofitted (RIS1)</td>
<td>5.89E-03</td>
<td>_</td>
<td>-3.26E-03</td>
<td>_</td>
</tr>
</tbody>
</table>

3. NUMERICAL RESULTS

In this section, finite element numerical models of the test specimens are developed and their accuracy verified using the experimental data. ANSYS software (2010) is used for this purpose. In the nonlinear model, both geometric and material nonlinearities are taken into consideration.
3.1 Numerical models

The eight-noded solid element, Solid65, was used to model the concrete. This element is capable of handling plasticity, creep, cracking in tension and crushing in compression. As for the failure criterion, the five-parameter William-Varnak model was used (Ansys, 2010). This model is able to account for the cracking of concrete in tension and crushing of concrete in compression. Furthermore, it uses a smeared crack model. The longitudinal and transverse reinforcements were modeled using two-noded link elements (ANSYS element link-8). An eight-noded solid element, Solid185, is also used for the steel plates at the beam and column hinge supports in order to prevent any cracking caused by stress concentration at these locations. A three dimensional, eight nodded solid-shell element (SOLSH190) was also used for the FRP sheets. The element possesses the continuum solid element topology at each node.

The nonlinear pushover problem was solved using the modified Newton-Raphson method. Also, for the reinforced concrete solid elements, convergence criteria was based on displacement and the convergence tolerance limit was kept at the ANSYS program default value.

In the FE model, the Hognestad model was used for the stress-strain curve of concrete. To define concrete in ANSYS, two shear transfer coefficients, $\beta_t$ and $\beta_c$, need to be introduced for open cracks and closed cracks. Both coefficients have values between 0 and 1, with 0 representing a smooth crack (complete loss of shear transfer) and 1 representing a rough crack (no loss of shear transfer). In this study shear transfer coefficient, $\beta_t$, is taken as 0.25 for open cracks. Furthermore, the shear transfer coefficient, $\beta_t$, of 0.99 was used for closed cracks, as are recommended by Chansawat et al. (2009).

3.2 Numerical Evaluation of Control joint CIS1

Nonlinear pushover analysis of the control specimen was carried out on the model discussed above. The beam tip load-displacement curve extracted from the nonlinear analysis is compared with that obtained from the experiments in Figure 7. Following the expected initial discrepancy owing to initial experimental adjustments, the numerical and experimental curves follow a similar trend, although the numerical solution shows a somewhat stiffer response. The stiffer numerical response is expected as the FE analysis usually produces a higher bound solution. The discrepancy in the numerical and experimental models are also attributed to actual bond slippage between the concrete and reinforcing bars, not modelled in the numerical analysis. However, the numerical analysis appears to have relatively accurately predicted the ultimate capacity, erring by only 10%, indicating the reliability of the developed FE model and analysis.

![Figure 7. Comparison of the numerical and experimental load-displacement curves for control specimen CIS1](image-url)
3.3 Numerical Evaluation of the Retrofitted Joint

The same nonlinear pushover protocol as the control specimen CIS1 was also applied to this model. The force-displacement curve obtained for the tip of the beam of this specimen is compared with the experimental pushover capacity curve in Figure 8. Again, a close trend can be seen in the numerical and experimental capacity curves of this specimen, numerical curve showing a stiffer model. The maximum load evaluated for the numerical model is about 8% higher than the experimental ultimate capacity.

![Comparison of numerical and experimental load-displacement curves for the retrofitted specimen RIS1](Figure 8)

3.4 The Effect of FRP Overlay Thickness on Relocation of Plastic Hinge

It was noted that although the tested flange-bonded retrofitting scheme with 5 layers of FRP overlay resulted in a large increase in the joint capacity, it failed to relocate the plastic hinge away from the column face towards the end of overlay. Relocation of the plastic hinge away from the column face is considered as a useful event in deficient RC joints. To investigate the effect of FRP overlay thickness on possible relocation of the plastic hinge in this joint, a parametric numerical investigation was carried out in which three different overlay thicknesses comprising of 5, 7 and 9 layers of overlay, corresponding respectively to thicknesses of 0.825 mm, 1.155 mm and 1.485 mm, were considered. Nonlinear pushover analyses were performed on the joint with these configurations. The state of strain in the tensile reinforcements of the beam was used to determine the position of the plastic hinge in each model. The location of the first reinforcement yielding, constituting the main location of plastic hinge, can be deduced from a plot of strain variation in tensile steel reinforcement along the length of the beam. Such plots for the three different overlay thicknesses are compared in Figure 9. As is seen in this figure, for models with 5 and 7 layers of FRP overlay, the maximum tensile strain in the reinforcement occurs near the face of the column, indicating that relocation has not occurred. However, in the model with 9 layers of overlay, the maximum strain in the steel bar occurs at the end of the FRP overlay which shows a successful relocation of the plastic hinge with increasing the thickness of overlay to 9 layers. The strain variation in concrete at the ultimate load for the RC joint before retrofitting and after retrofitting with 5 layers and 9 layers of FRP overlay are shown in Figure 10. This figure also illustrates plastic hinge relocation due to the retrofitting scheme with 9 layers of overlay.
4. CONCLUSIONS

Results of the experimental and numerical investigations as described in this paper lead us to conclude that:

a) The joint FRP flange-bonded retrofitting configuration investigated was able to increase the strength capacity of the joint by 23% which constitute a large increase in capacity.
b) The numerical models developed were capable of predicting the pushover response relatively accurately, erring by 10% for the control specimen and 8% for the FRP retrofitted joint.
c) In a moderate ductility moment resisting RC frame, the plastic hinge in the beam may occur very close to the column face and extend into the joint itself. This is an undesirable situation which may result in brittle joint failure. The ability of the investigated flange-bonded FRP retrofitting scheme to relocate the plastic hinge was shown to be dependent on the thickness of the FRP overlay. While a 9-layer FRP overlay resulted in successful relocation of the plastic hinge away from the column face, thinner overlays failed to relocate the plastic hinge.
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