Numerical modelling of CFRP-retrofitted RC exterior beam-column joints under cyclic loads

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ABSTRACT: This paper presents the results of a study on the capability of nonlinear quasi-static finite element modelling in simulating the hysteretic behaviour of CFRP-retrofitted RC exterior beam-column joints under cyclic loads. In this study, several unstrengthened and CFRP-strengthened specimens tested by the first and third authors at the University of Queensland, Australia are modelled using ANSYS. Concrete in compression is defined by the modified Hognestead model and anisotropic multi-linear model is employed for modelling the stress-strain relations in reinforcing bars while anisotropic plasticity is considered for the FRP composite. Both concrete and FRP are modelled using solid elements whereas space link elements are used for steel bars. Perfect bond between materials is assumed in this modelling as no considerable de-bonding occurred in the experiments. The specimens are then loaded using a step by step load increment procedure to simulate the cyclic loading regime employed in testing. In this analysis procedure, an automatically reforming stiffness matrix strategy is used in order to simulate the actual seismic performance of the RC concrete after cracking, steel yielding and concrete crashing during the push and pull loading cycles. The results show that the hysteresis simulation for both unstrengthened and FRP-strengthened specimens is satisfactory and therefore suggest that the numerical model can be used as an inexpensive tool in the design of FRP reinforcement levels to improve the hysteretic performance of RC beam-column joints under cyclic loads.

1 INTRODUCTION

Hysteretic behaviour of structure under cyclic loads can be used a means for evaluating the behaviour of structures under actual earthquakes. The hysteretic behaviour can generally be divided into two groups: The narrow low strength hysteretic curve with limited energy dissipation capacity and ductility, and the wide high strength hysteretic curve with more ductility and more energy dissipation.

Many existing reinforced concrete (RC) frames buildings located in seismic zones are deficient to withstand moderate to severe earthquakes. Insufficient lateral resistance along with poor detailing of members and joint reinforcement are the main reasons for inadequate seismic performance of these structures. 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 weak-beam strong-column principle (Mahini & Ronagh, 2007a). In recent years, several researchers used externally bonded FRP composites in order to increase the shear, flexural and anchorage capacity of RC frame
joints. For example, Smith and Shrestha (2006) carried out a systematic review of experimental research on the FRP-strengthening of RC connections and evaluated the effectiveness of these strengthening schemes. They 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. To the best of the authors’ knowledge, no tests have been performed on using externally bonded FRP to relocate the plastic hinge further along the beam away from the column face. In this regard, Mahini et al (2004a) tested an exterior beam-column joint before and after reinforcement with CFRP web-bonded under cyclic loads. Mahini et al (2004a) concluded that the plain and FRP retrofitted connection have similar strength and ductility, but the reinforced connection has better energy dissipation due to the shifting of the plastic hinge away from the column face. Mahini & Ronagh (2004b, 2006, 2007) also showed that the web-bonded CFRP system (when carefully designed) could restore the strength and ductility of damaged joints and improve the brittle failure of beam-column joints into a ductile manner by relocating the potential plastic hinge away from the column face towards the beam. Similar studies, but with other methods (such as the use of headed reinforcing bars) have been conducted on relocating plastic hinges away from the column faces (Chutarat & Aboutaha 2003).

Numerical modelling is also used to investigate into the hysteretic behaviour of RC beam-column joints. Recently, Parvin & Shahong Wu (2007) conducted a numerical analysis to investigate the effect of ply angle on the improvement of shear capacity and ductility of beam-column joint strengthened with CFRP wraps under combined axial and cyclic loads. The finite element analysis study entailed profiling the behaviour of three beam-column joint that were strengthened through the CFRP wrapping with various ply angle configuration. Parvin & Shahong Wu (2007) indicated that four layers of wrapping placed successively at ±45° ply angles with respect to the horizontal axis is the most suitable upgrade scheme for improving shear capacity and ductility of beam-column connections under combined axial and cyclic loads.

In this paper, two exterior beam-column connections tested by Mahini et al (2004a) are modelled using nonlinear finite element method, and the hysteretic curves of these connections are extracted and compared with experiments.

2 EXPERIMENTAL MODELS

2.1 Details of the test specimens

In this paper, two RC beam-column connection before and after FRP retrofitting tested by Mahini et al (2004a) under cyclic loads are numerically evaluated as shown in Figure 1. The specimens represented a scaled down exterior connection of a typical RC residential building designed according to AS3600 (2001). All connections consisted of 180 mm wide and 230mm deep beams with 220 mm × 180 mm columns and the width of the columns and beams are equal. All beams are reinforced with high-strength 12 mm diameter (N12) longitudinal reinforcing steel bars, with two bars in the top and two bars in the bottom of the beam. All columns were reinforced with four N12 reinforcing bars, with one bar positioned in each corner of the column. The beam stirrups and column ties are 6.5 mm bars at 150 mm centres. Ties were also placed in the connection region in accordance with the earthquake loading requirements of AS3600 (2001). Additional stirrups and ties were placed near the ends of the beam and columns in all specimens to ensure local failure would not occur at the load and support points respectively. The proposed retrofitting system consisted of three plies of web-bonded CFRP sheet. All layers were unidirectional sheets and were applied in a length of 200 mm from the column face on the beam-end with fibre directions parallel to the longitudinal beam axis as shown in Figure 1.

2.2 Material properties

The concrete compressive strength; elastic modulus and splitting strength (see Table 1) were determined on the day of testing each connection. Yield strength of the N12 reinforcing steel bars and R6.5 mm stirrups and ties were also tested as being 500 MPa and 380 MPa respectively.
Carbon fibres used had a thickness of 0.165 mm with a maximum elongation of 1.55% and a tensile strength of 3900 MPa according to the manufacturer's specifications.

Table 1. Mechanical properties of concrete

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength (MPa)</th>
<th>Modules of elasticity (GPa)</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSC1</td>
<td>40.73</td>
<td>30.17</td>
<td>3.29</td>
</tr>
<tr>
<td>RSC1</td>
<td>36.44</td>
<td>29.7</td>
<td>3.62</td>
</tr>
</tbody>
</table>

3 FINITE ELEMENT MODELING

Three-dimensional nonlinear finite element models for the beam-column connections were developed using ANSYS. To model the characteristics of concrete, the solid 65 element was used. This element is capable of simulating the cracking and crushing of concrete. The William-Vank criterion was used for fracture modelling in concrete. This model is able to account for the cracking and crushing of concrete in compression and tension respectively. Some important parameters to perform the failure envelope in the model are the compressive strength of concrete, the modulus of the rupture, and the shear transfer coefficients for open and closed cracks. Also for modelling the compressive strength of concrete, the Hongestad model was used. In addition, to model the longitudinal reinforcement and the FRP composites, Link 8 and Solid 45 elements were used, respectively. In order to model the FRP composites, an anisotropic material called ANISO was employed. To model the material in both compression and tension and in any direction of X, Y and Z a bi-linear stress-strain curve was used. A multi-linear isotropic stress-strain curve was also considered for the stress-strain curve.

As no debonding was observed during the previous tests performed by the authors (Mahini et al. 2004a), perfect bond between materials and concrete is considered so that adjacent elements have common nodes. In Figure 2, a typical finite element model is shown. Cyclic loads are applied with a step by step strategy in a displacement control regime similar to the Tests. Each cycle is modelled in a load step and each load step is divided into a number of sub steps. At the initial step of the analysis, because the section is uncracked and the solution is linear, a lower number of sub-steps are considered. However, at the cracking load and the final cycles, more...
load steps and sub-steps are utilised. An automatically reforming stiffness matrix is employed in order to simulate cracking and crushing of the concrete and steel yielding during cyclic loading.

4 COMPARISON BETWEEN EXPERIMENTAL AND FINITE ELEMENT MODEL

4.1 Specimen CSC1

In the experimental study reported by Mahini et al (2004a), the plain specimen is loaded in two phases. The first phase includes cycles that cause cracking and the second phases include cycles that cause first yielding of the longitudinal reinforcement followed by failure of the joint. In the analytical modelling, this specimen is loaded similar to the experimental study. As is shown in Figures 3 and 4, the plastic hinge was formed at the face of the column similar to the experimental observation. In Figure 5, the hysteresis curves of the specimen obtained from experimental and analytical studies are shown. As is seen, the experimental and numerical hysteresis curves are reasonably similar. The differences between the two could even become less distinguishable if a finer mesh is used in the numerical simulation.

Examining the hysteresis curves shows that in the analytical solution, the yield stress is reached at the displacement of 5 mm, where the maximum displacement and load are about 30.19 mm and 19.47 KN, respectively. In the experimental result, the displacement at yield and the maximum displacement are equal to 5 mm and 26.6 mm, respectively with maximum load of 19.51 KN, as shown in Figure 5.
Figure 4. Observed failure mechanism of specimen CSC1 (Mahini et al. 2004a)

Figure 5. Comparison between the obtained and the experimental hysteresis curves for plain specimen CSC1

4.2 Retrofitted specimen RSC1

Specimen RSC1 is subjected to the same loading regime as specimen CSC1. Figure 6 shows the final failure of the specimen in which the plastic hinge formed beyond the cut–off point of the CFRP. In fact, CFRP reinforcement causes the plastic hinge moving from the face of column toward the beam. This can also be seen from experimental observations (Figure 7) Comparison between the hysteresis curves obtained from the analytical and experimental show that the energy absorption is maintained as the maximum load is held to the end. The curves show that the first yield load of the analytical curve is equal to the experimental one (16 KN) and so is the maximum load (22 KN) as shown in Figure 8.

Figure 6. Failure mechanism of RSC1 obtained from numerical modelling
5 CONCLUSIONS

Based on the comparative modelling presented in this study, it is concluded that both hysteresis curves of the plain and retrofitted specimens are close to their experimental counterparts. This gives confidence to the design engineers and researchers in using finite element modelling for evaluating the cyclic performance of RC joints. Most effective retrofitting schemes can be easily found using the low cost finite element models similar to that presented in this study. The conclusion here is only valid to the peak load, as concrete's strain softening cannot be modelled by ANSYS.

REFERENCES

AS3600, 2001, Concrete Structures, Standards Australia, Homebush Bay, Australia.