Flexural Retrofitting of Concrete Bridge Beams Using CFRP Fabrics

Huy Pham and Riadh Al-Mahaidi
Department of Civil Engineering, Monash University, Melbourne

SYNOPSIS

Fibre reinforced polymers (FRP) have been used recently to strengthen RC structures. With current improvement in the quality of adhesives for construction, bonding between FRP and concrete is generally satisfactory. However, concrete cannot take high shear stresses. Therefore, FRP still tend to delaminate or debond from the structures. To study this debonding failure mechanism, a testing program including eight rectangular reinforced concrete beams is carried out. Testing shows two type of debonding: end debond and mid-span debond. Two theoretical models are then developed and verified with a relatively large database. It has been showed that the models are able to predict the ultimate debonding loads. For more inside understanding of the delamination phenomenon, a non-linear finite element model cooperated smeared cracks is also constructed. The model proves to be able to represent the beams' behaviour well, giving similar crack patterns and loading curves.

1 INTRODUCTION

There are two main reasons for strengthening bridges: deterioration caused by environmental factors and increase in traffic load. As a consequence, in the Unite States, almost 40% of highway bridges are classified as deficient and in need of rehabilitation or replacement (1). In Australia, more than 50% of bridges were built prior to 1948 (2). Most of these bridges are deemed to be inadequate, particularly those already damaged.

One method to overcome the problem is to strengthen these bridges. There are a number of solutions to do that: adding members, adding supports, increasing member cross sections, post-tensioning or bonding external laminates. Out of all such methods, laminate bonding has received much attention as a way to meet financial, time and disruption requirements. It is mostly used to increase load bearing capacity for reinforced concrete members. The principle of this method is to externally bond fibre reinforced polymer laminates on concrete members and they take tension as internal steel reinforcement. A reinforced concrete beam can be strengthened to increase flexural, shear or torsional capacity.

Bonding characteristics between FRP and concrete have been studied by several research groups. Most of the studies base on shear-lap tests, in which FRP are adhered on a concrete block and pulled on one end. Some of the recent papers on this topic are by Chen and Teng (3), Kanakubo et al. (4), Dai and Ueda (5) and Ulaga et al. (6).

Reinforced concrete beams bonded with steel plates or FRP laminates for flexural strengthening have been investigated by numerous researchers. The most commonly used experimental set-up is four-point bending of a simply supported beam. The most common failure mode observed is debonding of FRP from its end or from a shear crack. Table 1 below lists the studies and the number of retrofitted beams failed by the debonding mode found in literature up to date.

***Table 1*: Experimental studies on FRP retrofitting for flexural strengthening**

Reference	No. of beams	Reference	No. of beams
This study	6	Garden et al. (7)	10
Pham and Al-Mahaidi (8)	2	Ahmed and Van Gemert (9)	7
Arduini et al (10)	3	Beber et al. (11)	6
Gao et al (12)	3	David et al. (13)	4
Fanning and Kelly (14)	4	Hau (15)	3
Zarnic and Bosiljkov(16)	1	Tumialan et al. (17)	3
Rahimi and Hutchinson (18)	6	Pornpongsaroj and Pimanmas (19)	3
Ritchie et al. (20)	5	Kishi et al. (21)	8
Nguyen et al. (22)	4	Takahashi and Sato (23)	2
Quantrill et al. (24)	3	Valcuende and Benlloch (25)	4

Although, there has been significant amount of research on this topic, the failure mechanisms are still not fully understood and a simple design method is still not ascertained. To address these issues, in this study, an experimental program is implemented and modelled using non-linear finite element method. The data is then used to develop and validate two simple theoretical predicting models.

2 EXPERIMENTAL SET-UP

2.1 Experiment overview

A total of eight RC beams were constructed. Two were control beams and six were retrofitted with two plies of carbon fibre reinforced fabrics (CFRP) 1000 mm long and 100 mm wide. All of the beams were tested in four-point bending with the span of 2300 mm and the shear span of 700 mm. Two identical beams were manufactured for each configuration (Beam a and b). Three configurations were labelled S1, S2 and S3. The reinforcement for S1 is shown in Figure 1. For a parametric study, the stirrup spacing in S2 was reduced to 90 mm and the tension reinforcement in S3 included only 2 N12 bars.

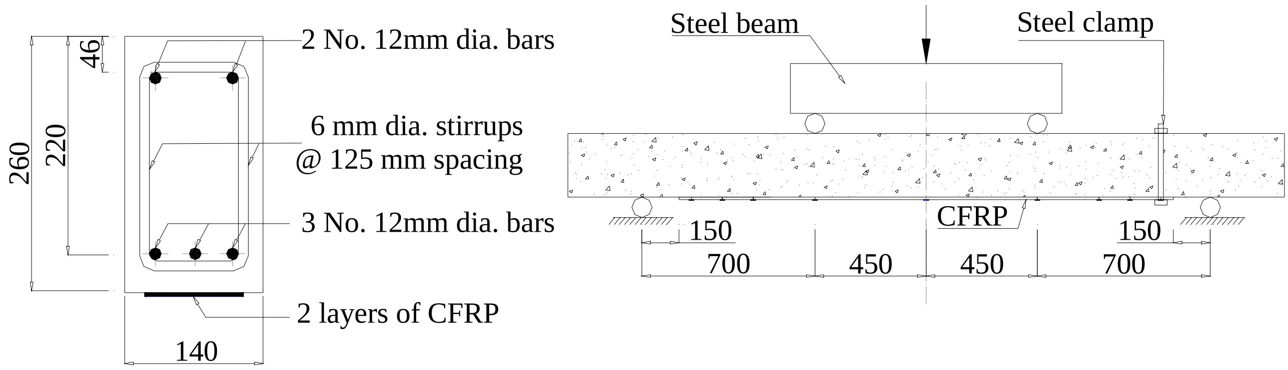


Figure 1: Typical beam dimensions (Beam S1)

The concrete material used was ready-mix concrete. The concrete strength on the day of the first test was 48 MPa (82 days). The properties of steel and CFRP reinforcement were also measured and listed in Table 2.

Table 2: Mechanical properties of materials used

Material	E (MPa)	f_{sy} (MPa)	f_t (MPa)
Steel (N12)	192000	504	599
Steel (N10)	204000	334	483
Steel (N6)	238000	423	576
CFRP fabrics	213500	-	3900*
Adhesive	>3500*	-	> 120*

* given by the manufacturer

2.2 Specimen preparation and instrumentation

The process of applying CFRP to concrete involved two main steps: surface preparation and bonding. The concrete surface was prepared using high-pressure water jet to remove a thin layer of the paste to expose the coarse aggregates. This method proved to be efficient to provide a relatively rough surface to improve mechanical bonding. Before bonding MBrace, all loose particles and dust were removed with an industrial vacuum cleaner. To ensure maximum bond, MBrace Primer was applied on the surface thoroughly with a brush.

Bonding operation was carried out about 30min to 1 hour after the application of the primer. Bonding operation included resin under-coating, carbon fibre sheet application and resin over-coating. To allow for epoxy impregnation, a period of five minutes was allowed between subsequent applications. The beams were left to cure at room temperature for at least seven days before testing.

All of the beams were instrumented with strain gauges and LVTDs. Crack propagation was captured using a digital video recorder and a high speed video recorder.

3 EXPERIMENTAL RESULTS

3.1 Failure modes

3.1.1 Control beams

The two control beams failed by typical steel yielding followed by concrete crushing. Both beams showed wide flexural cracks at mid-span. These cracks extended to the compressive area. Concrete crushing happened underneath the load point.

3.1.2 Strengthened beams

All strengthened beams failed by steel yielding followed by debonding of CFRP. The load deflection responses for the beams are plotted in Figure 2. Three regions can be clearly identified corresponding to when the concrete is not cracked, when the concrete is cracked and when the steel reinforcement yields. As expected, after the yielding of steel, major increase in tension was transferred to FRP.

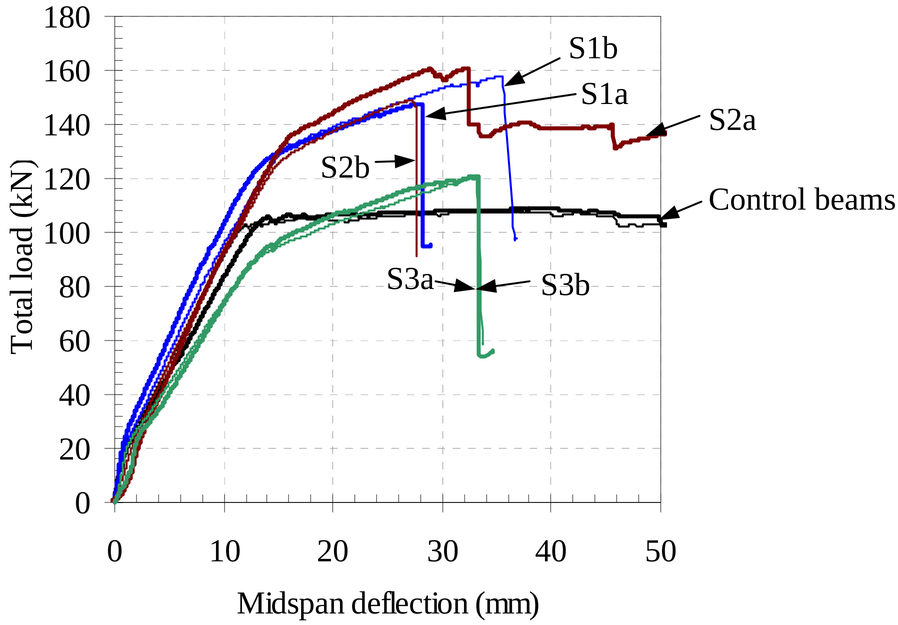


Figure 2: Loading curves

Observing the cracking patterns and the progress of separation of CFRP from the RC beams, it was clear that CFRP was separated due to high shear stress level developed in the concrete cover layer. The shear stress induced the inclined portion of the flexure-shear cracks below tension reinforcement level. Debonding was initiated by the opening at the crack tips. There were two critical cracks, which resulted in two distinct types of debond. End debond was resulted from the shear crack terminated at the end of FRP. It propagated along the tension steel level. Mid-span debond occurred at the tip of the flexure-shear crack near the load point. CFRP was delaminated along a layer about 1-2 mm above the bond surface.

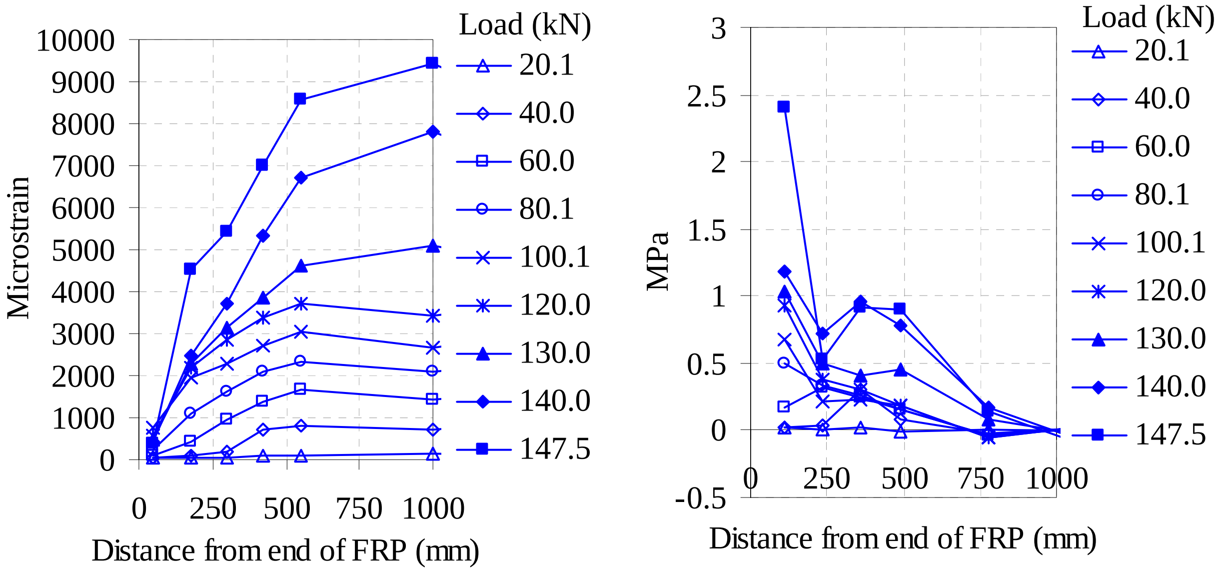
Beam S1a and S1b failed by a combination of mid-span debond and end debond. Concrete was fractured simultaneously from the unclamped end of FRP and from the tip of a wide flexural shear crack near middle of shear span. Stress was transferred along and cracking propagated toward the middle.

Beams S2a and S2b failed by mid-span debond. A wide flexural shear crack under the load point initiated delamination of FRP from concrete. For S2a, that happened on the clamped side. Delamination propagated toward the clamp location. The FRP was held by the clamp. The load level stayed at 140 kN after dropping down from the maxium value of 161 kN. Since the clamp was prestressed, the friction due to the interlocking of aggregate under the clamp plate was very high. As load was increased, severe crushing of concrete occurred under load point. More rotation was observed. The upper part of the beam tried to rotate to the right but it was held back by the FRP. As the tensile force increased to a very high level, the entire concrete cover on the right side was sheared off. For S2b, delamination occurred on the unclamped side. The ultimate load was slightly less than that in S2a.

Beams S3a and S3b had less tension reinforcement, which was indicated by the lower stiffness as in Figure 2. Beam S3a failed by mid-span debond first appeared on the clamped side. S3b failed by combination of mid-span and end debond on the unclamped side.

3.2 Strain distribution

The variations in longitudinal strain and bond stress in CFRP at different load levels for S1a are plotted in Figure 3. The strain drops from the maximum value under the load point to zero at the end of the plate. The stress is derived from the FRP strain distribution. The stress distributions for all six retrofitted beams indicate that delamination generally occurs at a shear stress level around 0.85 MPa.



a) Strain distributions

b) Interfacial shear stress distributions

Figure 3: Strain and shear stress distributions at different load levels

4 NUMERICAL MODELLING

4.1 FE idealisation

DIANA 8, a versatile FEA package, was used in this study. Plane stress elements with 20 x 20 mm mesh were used. The FRP was modelled using beam elements assuming perfectly elastic up to rupture. Line reinforcements were used. They were connected to concrete through interface elements. Plasticity for reinforcement was based on the Von Mises yield criterion. Eight-noded quadrilateral isoparametric elements were used to model concrete. The concrete width near tension reinforcement level was reduced according to the amount of reinforcement present (Figure 4b).

An available smeared crack model based on total crack strain was utilised for concrete cracking. In this model, the stress was evaluated as a function of the strain in the directions of the cracks. The strain and stress in the element coordinate system were related to strain and stress in the crack direction by the strain transformation matrix *T*. The deterioration of the material due to cracking or crushing depended on different levels of strains and was monitored by six internal damaged variables to ensure no damage recovery is possible. Tensile behaviour was described with a linear softening curve. The stress strain curve for compressive behaviour, however, depended on the effect of lateral confinement or lateral cracking, which was modelled using Thorenfeldt curve and Vecchio and Collin reduction factors respectively (26).

Two total crack strain models were used: *fixed crack model* and *rotating crack model*. In the first model, the stress-strain relationship was evaluated in a coordinate system, which was fixed upon cracking. In the second one, the coaxial stress-strain concept was used, in which the stress-strain relationship was evaluated in the principal directions of the strain vector. To model delamination of concrete cover, rotating crack model was used for the concrete below the tension reinforcement. The other part of the beam was modelled with fixed crack model.

4.2 Correlation between experiments and finite element analysis

The load versus midspan deflection curves from numerical models are compared with the corresponding experimental ones in Figure 4a. The curves match well for all three distinct regions: concrete uncracked, concrete cracked and steel yielding. The predicted ultimate loads predicted by FE models are very close to the experimental results.

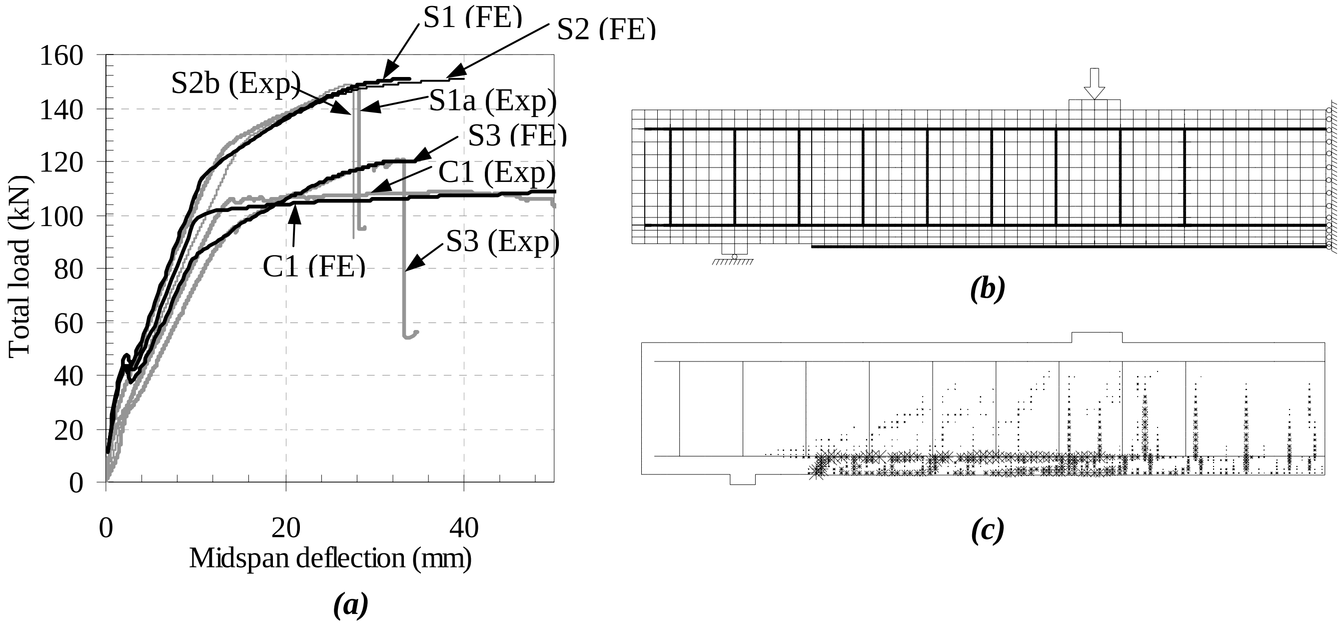


Figure 4: FE analysis: (a) Loading curves; (b) Mesh and boundary condition; (c) Crack patterns

Similar crack development was observed in FE models (Figure 4c). At first, flexural cracks appeared near mid-span. As loading was increased, inclined flexural shear cracks could be seen in the shear span. Debond cracks were visible after steel had yielded. They opened wider as load was increased more. Shear cracks extended to the load point were also seen but these cracks did not open wider due to the presence of the stirrups. As the load level was close to the ultimate value, the curve started to level off. The slip of the concrete cover below tension reinforcement due to high axial stress in FRP could then be seen in the model. It represented debonding type of failure. Due to the nature of smeared crack models, the curve descended gradually instead of experiencing a sudden drop.

5 THEORETICAL MODELS

5.1 Model derivation

Figure 5 explains the failure mechanisms of the two types of debonding. End debond is the result of the tensile force $F_{f,0}$ pulling on the end of FRP. This leads to a high shear stress level at the weakest layer near tension reinforcement level, inducing a longitudinal debonding crack there. For mid-span debond, the situation is different. The concrete tooth between two adjacent flexure-shear cracks is subjected to a tensile force, which is the difference between $F_{f,i}$ and $F_{f,i+1}$. The pulling force $F_{f,i+1} - F_{f,i}$ causes the peeling crack. Once this peeling crack propagates, tension is transferred to the concrete tooth on the right side of the shear crack. That causes progressive delamination towards the centre of the beam.

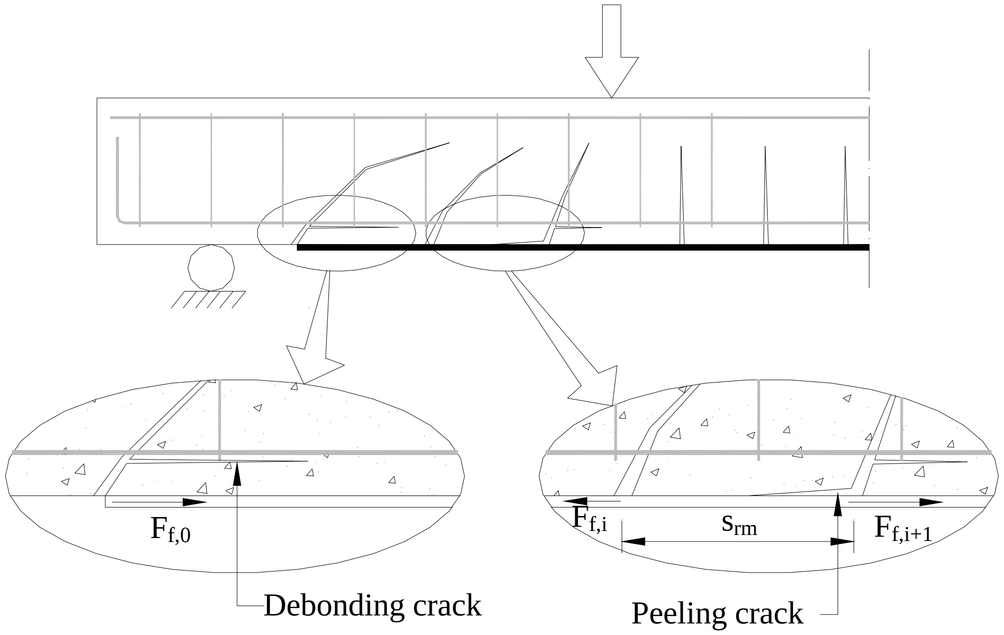


Figure 5: Failure mechanisms

Based on the observation of the failure mechanisms above, it is proposed that the debond capacity of retrofitted beams are controlled by two failure criteria. End debond occurs when the force $F_{f,0}$ is large enough so that the shear stress at the concrete layer near tension reinforcement exceeds the concrete shear strength. Mid-span occurs when the force difference $F_{f,i+1} - F_{f,i}$ is large enough to cause peeling of FRP from concrete. The later situation is similar to a simple shear test, in which FRP is bonded on a concrete block and pulled on one end until failure.

From our experiments, it has been found that beam theory can be used to predict the maximum strain level in FRP accurately. It has also been approved that FRP strains vary from a maximum value under the load point to zero at the end of FRP, following a curve curving upwards. For the sake of simple and conservative calculation, it is possible to assume that the curve is linear.

The followings are recommended:

For end debond:

Concrete shear strength = 0.3 f_{ctm} where f_{ctm} is direct concrete tensile strength.
FRP strain level is calculated using strain compatibility and equilibrium conditions. Hognestad's formula (27) is adopted for concrete stress-strain curve in compression.

For mid-span debond:

The maximum force that FRP can take is recommended by Chen and Teng (3) as

$$P = 0.427\beta_f \beta_L \sqrt{f_{cm}}\, b_f L_e \qquad (1)$$

where

$$L_e = \sqrt{\frac{E_f t_f}{\sqrt{f_{cm}}}} \tag{2}$$

$$\beta_f = \sqrt{\frac{2 - b_f / b_c}{1 + b_f / b_c}} \tag{3}$$

$$\beta_L = \begin{cases} 1 \ \text{if} \ L \geq L_e \\ \sin(\pi L / 2L_e) \ \text{if} \ L < L_e \end{cases} \tag{4}$$

b_c is the concrete web width; b_f, t_f and E_f are CFRP width, thickness and elastic modulus; L is the bond length and f_{cm} is the concrete compressive strength.
Concrete tooth spacing is calibrated with the experimental result based on the formulation recommended by Eurocode 2 (28)

$$s_{rm} = s_{rm0}(1.1 - 0.12 s_{rm0}) \tag{5}$$

$$s_{rm0} = 50 + 0.25 k_1 k_2 \frac{A_{c,ef}}{\sum u + b_f} \tag{6}$$

where c is the concrete cover; k_1 and k_2 are coefficients dependant on the bond properties and the distribution of tensile stress within the section; $\sum u$ is the sum steel bar perimeters and $A_{c,ef}$ is the effective concrete area.

5.2 Verification

As shown in Figure 7, the two models proposed above predict the failure loads for those beams accurately. As expected, the model of end debond gives a slightly conservative results.

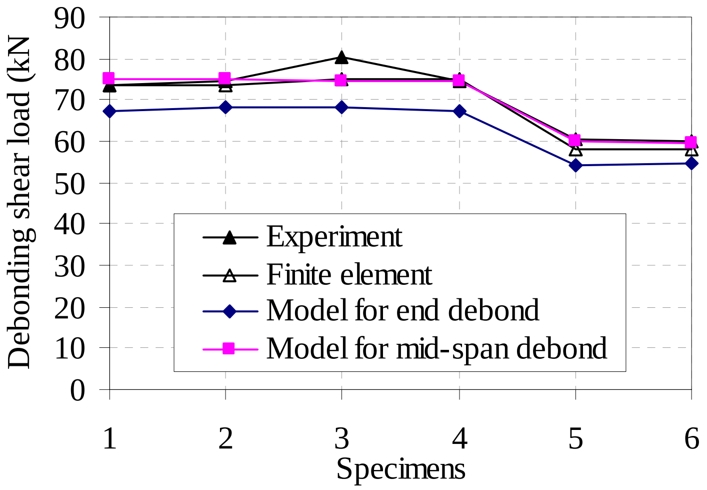


Figure 6: Comparison between the theoretical models and experimental results

The proposed theoretical model was also verified with the beams listed in Table 1. The result is shown in Figure 8. For the beams failed by end debond, the average predicted failure loads are 80% of the experimental results. For the beams failed by mid-span debond, the ratio of predicted value to the actual results is 1.03.

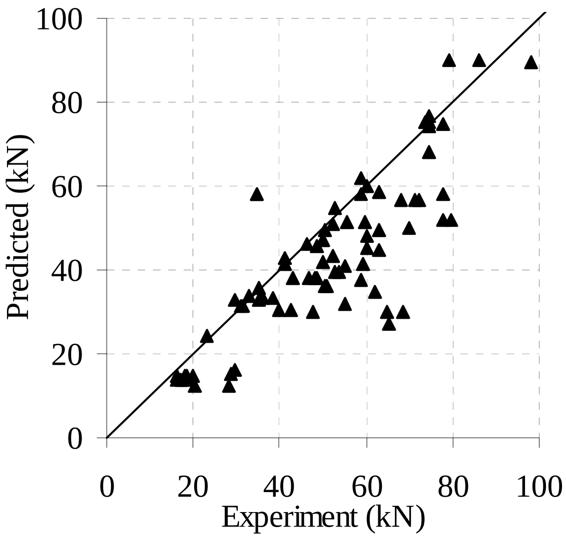


Figure 7: Verification of the proposed models

6 CONCLUSION

An investigation into the behaviour of reinforced concrete beams strengthened with CFRP fabrics has been presented. Two brittle failure modes were observed: debonding from the end of FRP and debonding from a flexure-shear crack. These failures occurred due to high shear stress levels developed either at the end of CFRP or at a tip of a flexure-shear crack. The non-linear FE model based on smeared cracks has shown its capability to represent the retrofitted beams' behaviour. The two theoretical models were developed based on the actual experimental observation and recognised equations. They have been proved to be reliable.

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