Concrete Beams Strengthened with Externally Bonded FRP Plates

By Hamid Rahimi and Allan Hutchinson

Abstract: The structural behavior of reinforced concrete beams strengthened with adhesively bonded fiber-reinforced plastics (FRP) is presented. The experimental work included flexural testing of 2.3-m-long concrete beams with bonded external reinforcements. The test variables included the amount of conventional (internal) reinforcement and also the type and amount of external reinforcement. For comparison, some of the beams were strengthened with bonded steel plates. Theoretical analyses included 2D nonlinear finite-element modeling incorporating a “damage” material model for concrete. In general there were reasonably good correlations between the experimental results and nonlinear finite-element models. It is suggested that the detachment of bonded external plates from the concrete, at ultimate loads, is governed by a limiting principal stress value at the concrete/external plate interface.

Introduction

The plate bonding technique is now established as a simple and convenient repair method for enhancing the flexural performance of concrete structures and, in particular, bridge beams and slabs. The advantages of this technique are that the work can be carried out while the structure is still in use and it is economical compared to other methods.

The earliest reported examples of plate bonding were those carried out in South Africa and France (Fleming and King 1967; Bresson 1971). Since then the plate bonding technique has received a lot of attention and today it is used in all types of structures for repair and rehabilitation (Baker and Chester 1992; Mays and Hutchinson 1992; Allan and Bird 1986). Traditionally, steel plates were used but there are problems associated with them as external reinforcement for existing concrete structures, including the need for careful surface preparation of the steel prior to bonding, uncertainty regarding adhesive bond durability and potential corrosion at the steel/adhesive interface, awkward manipulation of heavy steel plates necessitating restrictions on plate length and the need for many lapped joints, the need for anchor bolts, and maintenance painting.

Fiber-reinforced plastic (FRP) materials do not suffer from corrosion problems, and most of their mechanical and physical properties are better than those of steel. Swiss researchers pioneered work on the use of FRP as a replacement for steel in plate bonding applications (Meier and Kaiser 1991), and numerous researchers have since investigated the structural behavior of concrete elements with bonded FRP reinforcements [e.g., Taerwe (1995), El-Badry (1996), and Meier and Bett (1997)].

Experimental research programs into the flexural behavior of steel and FRP-plated beams are reported by Ladner and Weder (1981), Macdonald (1978), Jones et al. (1980, 1982), Van Gemert and Maeschuck (1983), Swamy et al. (1987), Saadatmanesh and Ehsani (1991), Ritchie et al. (1991), Chajes et al. (1994), and Sharif et al. (1994). Theoretical aspects of the plate-bonding technique are given by Roberts (1989), Roberts and Haji-Kazemi (1989), Oehlers and Moran (1990), Hamoush and Ahmad (1990), Wei et al. (1991), Rostasy (1993), Ziraba et al. (1994), and Hussain et al. (1995). Limited studies of the behavior of plated concrete beams using finite-element (FE) methods are given by Mays (1993), Triantafilou and Plevris (1990), Hamoush and Ahmad (1990), and Ziraba et al. (1994). The most comprehensive study of an FRP strengthening system was undertaken in the United Kingdom during the ROBUST project (Hollaway and Leeming 1999) in which all aspects of materials, design, and analysis were addressed. The substance of this paper formed part of the ROBUST Project.

Many research studies investigating the performance of concrete structures with bonded external composite materials have ignored the problems associated with adhesion aspects and appropriate surface treatments for adhesive bonding. The optimum properties of adhesives required for plate bonding applications is not known, but recommendations by Mays and Hutchinson (1988) are generally followed, at least in the United Kingdom, and they are incorporated in the U.K. Department of Transport (DTF) guidelines BA 30/94 (DTF 1994). There is also a lack of understanding of the requirements of FRP materials for plate bonding applications.

The overall aim of the research was to investigate the suitability of FRP for externally bonded reinforcement of concrete structures subjected to flexural loading. Aspects of adhesive bonding technology, composite materials, and numerical modeling were used and applied to plate bonding technology. Detailed nonlinear FE analysis was carried out to identify some of the variables that have a significant effect on the performance of the strengthened beams and also to understand the failure mechanisms associated with such structures.

Experimental Program

Concrete Test Beams

The mix was similar to that used previously in U.K. concrete research programs (Embsmore and Mays 1990), providing a compressive strength of about 50 N/mm² after 28 days. The constituent parts by weight were 1/1.6/2.9/0.45 [ordinary Portland cement/sand/aggregate (20 mm down to 5 mm)/water]. Beam experimental variables included their type and condition (uncracked and precracked), the type and thickness of external reinforcement, and the plate-end geometry of external reinforcement.

The design of the reinforced concrete (RC) beams was based on the following criteria (also Appendix I):

- Limit the beam length to around 2 m, for practical reasons.
- Obtain beam dimensions, under four-point bending, that give a shear span-to-depth ratio of at least 6 while main-
TABLE 1. Details of Conventional (Internal) and External Reinforcements for RC Beams

<table>
<thead>
<tr>
<th>Beam type (1)</th>
<th>Conventional Reinforcements</th>
<th>External reinforcement* (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(2)</td>
<td>(3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(4)</td>
<td></td>
</tr>
<tr>
<td>A 2T10 (A_/bd = 0.65%)</td>
<td>2T10 (A_/bd = 0.42%)</td>
<td>CFRP plates (0.8 and 1.2 mm)</td>
</tr>
<tr>
<td>B 2T10 (A_/bd = 0.65%)</td>
<td>2T10 (A_/bd = 0.42%)</td>
<td>CFRP plates (0.4 and 1.2 mm)</td>
</tr>
<tr>
<td>C 2T16 (A_/bd = 1.68%)</td>
<td>2T10 (A_/bd = 0.42%)</td>
<td>CFRP plates (1.8 mm)</td>
</tr>
</tbody>
</table>

Note: Shear links were placed only within shear spans of beams.

*Plate thickness.

Gaining an adequate constant movement region so that the beam can undergo sufficient bending deflection under the load before failure.

- Allow for various amounts of conventional tensile reinforcement without creating overreinforced sections. Naturally, only underreinforced beams require strengthening.

The beams were 2,300-mm long \( \times \) 200-mm wide \( \times \) 150-mm deep (Fig. 1), and they were typically 4–6 months old at the time of strengthening. Three types (A–C) were used, comprising the internal and external reinforcements identified in Table 1. Glass FRP (GFRP), carbon FRP (CFRP), and mild steel external reinforcement were used. Two replicate beams of Type A (A8 and A9) were incrementally loaded to 20 kN, then unloaded, to cause cracking in the concrete and yielding of the tensile rebars. This preload represented 80% of the ultimate strength of unplated Type A beams, resulting in some permanent flexural deformation. These beams were externally strengthened with 0.8-mm CFRP plates.

### External Reinforcing Materials

Prepreg tapes with unidirectional fiber reinforcement were chosen for their versatility and ease of shaping for experimental purposes. Some real strengthening applications have employed prepreg materials but, in general, unidirectional pultruded composite plates are more commonly used (Meier et al. 1992; Hollaway and Leeming 1999). The prepreg materials selected were manufactured by Hexel Composites of Duxford, U.K.:

- GFRP—Fibredux 913GE 50%, a unidirectional glass fiber/epoxy resin system with around 50% fiber volume content.
- CFRP—Fibredux 920Cx (BG/B) T800H (12K)5 40%, a unidirectional carbon fiber/epoxy resin system with around 40% fiber volume content.

These materials required curing at 120°C to achieve full strength, using a conventional vacuum bagging procedure. The composite specimens were subsequently tested in accordance with Composites Research Advisory Group (CRAG) test procedures (Curtis 1988), resulting in the characteristic properties shown in Table 2.

The FRP plates were 1,930-mm long, similar to the clear distance between the supports (1,950 mm), and 150-mm wide, which covered the distance between the tensile rebars (140 mm). The thickness of the CFRP laminates was varied from 0.4 mm (2-ply) to 1.2 mm (6-ply). Due to the relatively low modulus of GFRP, the chosen thickness of these laminates was 1.8 mm (12-ply).

For comparison, four beams (two each of Types B and C) were strengthened with 3-mm-thick mild steel plates of the same lengths and widths of the composite plates. This thickness was based on the required bending stiffness and also on practical considerations such as availability and the need to choose a minimum thickness to enable grit blasting without significant distortion.

### Surface Preparation Techniques

All concrete test beam surfaces were grit blasted with 180-mesh alumina at an average pressure of 207 kPa (30 psi) in a
TABLE 2. Properties of Materials Used

<table>
<thead>
<tr>
<th>Property</th>
<th>Concrete</th>
<th>GFRP composite</th>
<th>CFRP composite</th>
<th>Sikadur 31PBA adhesive</th>
<th>Internal steel reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m$^3$)</td>
<td>200</td>
<td>2,200</td>
<td>1,500</td>
<td>1,500</td>
<td>7,800</td>
</tr>
<tr>
<td>Young's modulus (GN/m$^2$)</td>
<td>25</td>
<td>36</td>
<td>127</td>
<td>7</td>
<td>210</td>
</tr>
<tr>
<td>Shear strength (MN/m$^2$)</td>
<td>6</td>
<td>80$^a$</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Tensile strength (MN/m$^2$)</td>
<td>3</td>
<td>1,074</td>
<td>1,532</td>
<td>25</td>
<td>575</td>
</tr>
<tr>
<td>Compressive strength (MN/m$^2$)</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>0.4</td>
<td>—</td>
</tr>
<tr>
<td>Fracture energy (kJ/m$^2$)</td>
<td>0.02</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>—</td>
</tr>
<tr>
<td>Elongation at break (%)</td>
<td>0.15</td>
<td>3.1</td>
<td>1.21</td>
<td>0.7</td>
<td>&gt;20</td>
</tr>
<tr>
<td>Coefficient of thermal expansion (10$^{-6}$/°C)</td>
<td>10</td>
<td>8</td>
<td>—0.8</td>
<td>30</td>
<td>11</td>
</tr>
</tbody>
</table>

Note: GFRP and CFRP properties quoted in fiber direction.

$^a$Interlaminar shear strength.

pressure-fed recirculating machine, then blown with clean air to remove dust. The resultant concrete surface was characterized by a uniformly abraded surface with exposed small- to medium-sized pieces of aggregate.

Surface preparation of the composite plates was accomplished by stripping off a clean, scrubbed, nylon peel-ply layer molded into one surface during composite fabrication. This type of treatment leaves a clean and uniformly rough surface finish that contributes toward mechanical interlocking with an adhesive. The selection of peel-ply material, its weave pattern, and the processing conditions were based on research work at Oxford Brookes University, Oxford, U.K., by Wingfield (1993).

Mild steel plates were degreased by wiping with trichloroethane, grit blasted with grade 180 alumina at an average pressure of 207 kPa, then blown with clear air to remove dust. The grit-blasting operation caused warping of the steel plates; accordingly, the backs of the plates were also grit blasted to restore flatness.

**Adhesive Material and Curing Procedure**

Sikadur 31 plate bonding adhesive, a two-part, room-temperature curing epoxy adhesive was selected. Mechanical tests using small-scale specimens showed that its adhesion to concrete, steel, and FRP materials was good (Hutchinson and Quinn 1999).

For the concrete beams, a vacuum bag technique was used to apply a uniform pressure of 1 atm (100 kPa) to the composite plates. Tack tape was adhered to the perimeter of the bonding area, the paste epoxy adhesive was troweled into position, the FRP plate was applied, the backing paper from the top surface of the tack tape was removed, and vacuum bag material was then adhered to the surface. A vacuum breach unit was then attached to the area and connected to a pump. A resultant bondline thickness of 2 mm was achieved over the entire bond area without the use of physical spacers or microspheres.

Electrical resistance strain gauges were attached to the steel rebars, external surfaces of reinforcement plates, and concrete to measure the variation of strain along the length of the test beams. Strains down the depth of the concrete beams, at mid-span, were measured using a mechanical extensometer applied across targets that had been bonded to the concrete surface. Linear variable differential transducers were employed to provide automatic recording of beam deflections by means of a computerized data-logging system.

**Flexural Test Procedure**

The beams were tested in four-point bending, being simply supported on a pivot bearing on one side and a roller bearing on the other, over a span of 2,100 mm. Identical bearing pads were placed at the loading points on top of the beams. A spreader I-beam resting on top of these provided a system for load distribution.

Load was applied, by means of a hydraulic jack, in increments of 5 kN throughout the tests. At each load increment, observations of crack development on the concrete beams were noted. Fabrication, surface treatments, and testing of specimens were carried out under ambient laboratory conditions [20 ± 3°C and 50% relative humidity (RH)], unless otherwise stated.

**ANALYSIS OF EXTERNALLY STRENGTHENED BEAMS**

**Ultimate Moment of Resistance of Beams**

The following assumptions were made as recommended in BS 5400: Part 4 [British Standards Institution (BSI) 1990] (also Appendix I): Full bond exists between the rebars and the surrounding concrete—i.e., strain compatibility is maintained throughout; the ultimate concrete compressive strain was 3,500 $\mu$e; and the self-weight of the beam was ignored. Fig. 2 shows the cross section of a plated beam. In the following
analyses the contribution of the adhesive layer is neglected. Most standard procedures for concrete design recommend that the following criteria should be met for the effective contribution of links toward the shear loading capacity of concrete beams [e.g., BSI (1990) and Kong and Evans (1987)]: (1) The distance between the centers of the links should be less than the effective depth of the beam—i.e., $S_i < d$; and (2) the amount of links should be considered according to the following relationship for the web resistance in shear:
\[
A_s \geq \frac{0.2b S_s}{f_{yw}}
\]  
(1)

For Type A beams, the link spacing was at 150-mm centers within the shear span, which contravenes the first criterion, because it was intended to study the behavior of beams containing insufficient shear reinforcement.

**FE Analysis of Plated Beams**

Closed-form solutions for the analysis of plate-bonded structures are normally based on linear elastic models for which stress and strain values can be calculated globally. Such theories cannot usually cope with problems where gross material and geometric nonlinearity prevail. It is however, desirable to be able to predict effects such as stress and strain variations within an RC beam with external reinforcement while undergoing nonlinear changes.

The nonlinear behavior of concrete under load is frequently dominated by progressive cracking, resulting in localized failure. In FE analysis, various procedures have been adopted for predicting cracking in concrete and these fall broadly into two main methods, namely, the discrete crack approach and smeared crack formulation.

With the discrete crack concept, the position and direction of crack growth within the model is predefined. The LUSAS FE analysis system (LUSAS 1994) was used in this research because it incorporates the smeared crack concept that has been used widely for predicting the nonlinear behavior of concrete (Rots and Blaauwendraad 1989). In this continuum approach, cracks are simulated as local discontinuities that are distributed within the entire FE model. Relative displacements of crack surfaces are represented by crack strains, and the constitutive behavior of cracked concrete can be modeled in terms of a stress-strain relationship. The smeared crack strategy, however, tends to spread crack formation over the entire structure so that it is incapable of predicting local fracture.

More recently, damage models have become popular techniques for simulating various nonlinearity effects in materials. The LUSAS program additionally incorporates an isotropic damage model, based on a publication by Oliver et al. (1990), which simulates the nonlinear behavior of concrete by means of a scalar variable called the damage or degradation parameter. These isotropic damage models do not incorporate “fudge factors” such as the shear retention factor used in the smeared crack models, and they allow for degradation in both tension and compression. The damage threshold is analogous to the yield point in an elastoplastic analysis. In a damage analysis, the value of the damage threshold $r_0$ influences degradation of the elastic modulus matrix, where $r_0$ is calculated using (2) (LUSAS 1994)
\[
r_0 = ((\sigma'_u)/(E_0))^{0.5}
\]  
(2)

where $\sigma'_u$ = uniaxial tensile strength of concrete; and $E_0$ = undamaged Young’s modulus. The damage accumulation function for the Oliver model is given by
\[
G(r) = 1 - (r/r_0)\exp[A(1 - (r/r_0))]
\]  
(3)

For the case of no damage, $G(r_0) = 0$. The parameter $r$, is the current damage threshold, and a means of computing $A$ has been postulated for the Oliver damage model
\[
A = [[[((\alpha E_0)/(\sigma'_u)) - 0.5]]^{-1}
\]  
(4)

A typical FE model developed for analysis of the RC beams is shown in Fig. 3, although Hutchinson and Rahimi (1993) used a much finer mesh discrimination to estimate the plate-end stress concentrations. Comparison of the results of these analyses showed that the same predictions were obtained with the relatively coarse mesh. Moreover, the results from the experimental studies show that the largest stress concentrations do not necessarily occur at the plate ends and this zone was not generally the site for the initiation of failure.

The beams were completely symmetrical about the midspan; thus, half the beams were modeled and a symmetry condition was imposed along the boundaries. For 2D analysis, concrete beams were modeled mostly with four- or eight-noded quadrilateral isoparametric elements. The internal rebars were modeled with two- or three-noded bar elements that were smeared onto the concrete elements, although shear reinforcement (vertical links) was not incorporated into the models. For beams with an adhesive layer, three- or six-noded triangular elements were used in the transition zones to reduce the size of elements toward the bond-line area (Fig. 3). The adhesive layer and external reinforcement were modeled with a single row of four- or eight-noded elements. For modeling purposes, the adhesive layer was square ended, whereas in real situations an adhesive fillet often forms at the edge of bonded joints. The adhesive was assumed to behave in an elastic manner.

**DISCUSSION OF EXPERIMENTAL AND ANALYTICAL RESULTS**

The properties of the materials used, derived from standard test procedures, are given in Table 2. Tensile testing showed that the unidirectional CFRP and GFRP composite materials exhibited linear elastic behavior up to failure. The tensile strength of CFRP material was unaffected after exposure to hot/humid conditions (40°C and 95% RH) for over a year, while the tensile strength of GFRP was reduced by 30–40% (Rahimi 1997).

Hutchinson and Hollaway (1999) showed that there was no deterioration in the performance of bonded single lap joints made with the same materials. The locus of failure of these joints was mostly cohesive within the adhesive even after 1
Concrete pull-off tests were used to measure adhesion to concrete, and this always resulted in failure of the concrete substrates; indeed, freeze-thaw cycles apparently caused an increase in pull-off strength.

**Strength of Test Beams**

It can be seen from Fig. 4 and Table 3 that all beams with bonded external reinforcement performed significantly better than the control (unplated) beams, in terms of strength and stiffness. Clearly the strength of plated beams is influenced by the original stiffness of the beams and by the type and amount of external reinforcement. In turn, strength depends on the limiting stress-strain properties of the constituent materials, the properties of the interface between the steel rebar and concrete, and the adhesive joint between the concrete and external reinforcement. For the three types of unplated beams tested (A-C), the strength was limited to the maximum compressive fiber strain in the concrete, which was around 3,500 με. This was validated experimentally, showing concrete crushing at ultimate load levels.

Fig. 4 shows that an almost twofold increase in the strength of Type B beams was obtained with a very thin (2-ply and 0.4-mm) CFRP bonded plate. The effectiveness of bonded external reinforcement becomes more apparent when comparing the performance of a 2-ply CFRP-plated Type B beam with an unplated Type C control beam. The amount of conventional reinforcement used in Type C beams was 1.68%, compared with 0.65% in Type A/B beams. Thus even with a relatively small amount of bonded external reinforcement added to Type B beams, comparable performance is obtained with a beam containing a relatively high percentage of conventional reinforcement.

Fig. 5 shows that beam strengths for Type A/B do not vary linearly with the amount of external reinforcement and there is a diminishing return in beam strength as the amount of external reinforcement is increased. It should also be noted that all Type A/B beams failed prematurely in the sense that no concrete crushing occurred at ultimate load levels.

For a given load, the GFRP plates (12-ply and 1.8-mm thick), used on the Type B and C beams, strained by the same amount as a 0.5-mm CFRP plate. The ultimate loading capacity of the Type B GFRP-plated beams was 60 kN, compared with 29 kN for the control beams. It should be noted that the elastic modulus of the CFRP used was around 120 GPa, compared with 36 GPa for GFRP. The strain to failure of GFRP is higher than CFRP, although its ultimate strength is somewhat lower.

The behavior and strength of the steel-plated beams was strongly affected by yielding of the steel plates; as a result, the proportional increase in strength with the steel plates was less than that obtained with the FRP plates. Beam Types B and C failed prematurely without reaching their ultimate load-carrying capacity. Failure was initiated in the Type B beams as a result of transverse curling of the plates at midspan, whereas in Type C beams it was as a result of plate-end peeling.

Theoretical predictions of strength were based on strain compatibility methods and FE analysis. The procedure in BS 5400: Part 4 was used for calculating the ultimate moment of resistance of both plated beams and unplated beams. These calculations showed reasonably good correlations with experimental results for Type C beams (Table 3). However, closed-form solutions overestimated the strength of Type A/B beams, which generally failed prematurely.

The FE predictions of the ultimate load $P_{\text{ult}}$ for the test beams are given in Table 3. These were particularly sensitive to the concrete tensile strength value used and were based on limiting the maximum concrete compressive fiber strain to 3,500 με. In the FE models, a single, more accurate numerical solution for beam strength was obtained with a concrete tensile strength of 1.5 MPa. Due to uncertainty regarding concrete material behavior, Fig. 6 shows a family of numerical curves to indicate the load-deflection characteristics of beams (FE analysis often inherently overestimates the stiffness of structures). In general, predicted numerical solutions for beam strengths were within 20% of the experimental results. The only exceptions were the steel-plated beams, which failed prematurely for different reasons. In particular, steel delamination of Type B beams, which was due to anticlastic deformation, could not be modeled with the 2D FE techniques employed.

The lines of best fit in Fig. 5 for each type of test beam indicate that the strength of plated and unplated beams can be related to each other using the following general expression:

$$P_{\text{plated}} = P_{\text{unplated}} + mA_p$$

where $m = f(A_s, b, d, E_p, E_s \text{concrete}, f_c, f_s, \ldots)$; and $A_p = \text{cross-}$

**FIG. 4. Load-Deflection Behavior of Concrete Beams with Bonded External Reinforcement**

48 / JOURNAL OF COMPOSITES FOR CONSTRUCTION / FEBRUARY 2001
### TABLE 3. Load-Deflection Data for Concrete Test Beams

<table>
<thead>
<tr>
<th>Beam number (1)</th>
<th>Type of external reinforcement and plate thickness</th>
<th>Cross-sectional area of plate (mm²) (3)</th>
<th>Closed form solutions for ultimate load (kN)</th>
<th>FE predictions for ultimate load $P_{ult}$ (kN)</th>
<th>Experimental Results</th>
<th>Ultimate load $P_{ult}$ (kN)</th>
<th>Midspan deflection (mm)</th>
<th>Type of failure* (9)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>None</td>
<td>—</td>
<td>50.6</td>
<td>28</td>
<td>—</td>
<td>26.2</td>
<td>48.3</td>
<td>CC</td>
</tr>
<tr>
<td>A2</td>
<td>None</td>
<td>—</td>
<td>50.6</td>
<td>28</td>
<td>—</td>
<td>26.2</td>
<td>60</td>
<td>CC</td>
</tr>
<tr>
<td>A3</td>
<td>None</td>
<td>—</td>
<td>50.6</td>
<td>28</td>
<td>—</td>
<td>26.6</td>
<td>49.7</td>
<td>CC</td>
</tr>
<tr>
<td>A4</td>
<td>CFRP laminate, 4-ply (0.8 mm)</td>
<td>120</td>
<td>50.6</td>
<td>78.9</td>
<td>65.9</td>
<td>61.9</td>
<td>32</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A5</td>
<td>CFRP laminate, 4-ply (0.8 mm)</td>
<td>120</td>
<td>50.6</td>
<td>78.9</td>
<td>65.9</td>
<td>63.2</td>
<td>31.3</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A6</td>
<td>CFRP laminate, 6-ply (1.2 mm)</td>
<td>180</td>
<td>50.6</td>
<td>90.4</td>
<td>77.7</td>
<td>59.4</td>
<td>23.1</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A7</td>
<td>CFRP laminate, 6-ply (1.2 mm)</td>
<td>180</td>
<td>50.6</td>
<td>90.4</td>
<td>77.7</td>
<td>70.6</td>
<td>27.9</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A8</td>
<td>CFRP laminate, 4-ply (0.8 mm), beams preloaded to 20 kN before plating</td>
<td>120</td>
<td>50.6</td>
<td>78.9</td>
<td>65.9</td>
<td>65.2</td>
<td>31.9</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A9</td>
<td>CFRP laminate, 4-ply (0.8 mm), beams preloaded to 20 kN before plating</td>
<td>120</td>
<td>50.6</td>
<td>78.9</td>
<td>65.9</td>
<td>63.9</td>
<td>33</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A10</td>
<td>CFRP laminate, 4-ply (0.8 mm), plate extended to under supports</td>
<td>120</td>
<td>50.6</td>
<td>78.9</td>
<td>65.9</td>
<td>67.5</td>
<td>41.2</td>
<td>S/C/P</td>
</tr>
<tr>
<td>A11</td>
<td>CFRP laminate, 4-ply (0.8 mm), plate extended to under supports</td>
<td>120</td>
<td>50.6</td>
<td>78.9</td>
<td>65.9</td>
<td>69.4</td>
<td>40</td>
<td>S/C/P</td>
</tr>
<tr>
<td>B1</td>
<td>None</td>
<td>96.2</td>
<td>28</td>
<td>29.2</td>
<td>54.6</td>
<td>29.2</td>
<td>54.6</td>
<td>CC</td>
</tr>
<tr>
<td>B2</td>
<td>None</td>
<td>—</td>
<td>28.4</td>
<td>40.4</td>
<td>—</td>
<td>28.4</td>
<td>40.4</td>
<td>CC</td>
</tr>
<tr>
<td>B3</td>
<td>CFRP laminate, 2-ply (0.4 mm)</td>
<td>60</td>
<td>92.6</td>
<td>62.2</td>
<td>53.6</td>
<td>55.2</td>
<td>38.7</td>
<td>C/P</td>
</tr>
<tr>
<td>B4</td>
<td>CFRP laminate, 2-ply (0.4 mm)</td>
<td>60</td>
<td>92.6</td>
<td>62.2</td>
<td>53.6</td>
<td>52.5</td>
<td>38.1</td>
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<tr>
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<td>92.6</td>
<td>90.4</td>
<td>80.2</td>
<td>69.7</td>
<td>30.3</td>
<td>C/P</td>
</tr>
<tr>
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<td>CFRP laminate, 6-ply (1.2 mm)</td>
<td>180</td>
<td>92.6</td>
<td>90.4</td>
<td>80.2</td>
<td>69.6</td>
<td>28.3</td>
<td>C/P</td>
</tr>
<tr>
<td>B7</td>
<td>GFRP laminate, 12-ply (1.8 mm)</td>
<td>270</td>
<td>92.6</td>
<td>72.6</td>
<td>57.6</td>
<td>59.1</td>
<td>33.3</td>
<td>C/P</td>
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<tr>
<td>B8</td>
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<td>270</td>
<td>92.6</td>
<td>72.6</td>
<td>57.6</td>
<td>61.6</td>
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<td>B9</td>
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<td>92.6</td>
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<td>91.1</td>
<td>62.7</td>
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<td>51.6</td>
<td>—</td>
<td>58.5</td>
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<td>87.1</td>
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<td>104.1</td>
<td>82.4</td>
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Note: Average cube strength varied from 54 to 69 N/mm² and tensile splitting strength varied from 3 to 3.8 N/mm².

*CC = concrete crushing; S/C/P = concrete shear failure followed by cover separation and plate detachment; C/P = cover failure followed by plate detachment; PP = plate-end peel off; St = steel plate yield.

FIG. 5. Variation of Beam Strength with Type and Amount of External Reinforcement
FIG. 6. Comparisons of Numerical Predictions, Using Different Values of Concrete Tensile Strength $\sigma_t$, with Experimental Results for Type B CFRP-Plated Beam

sectional area of external reinforcement (mm$^2$) with a CFRP plate modulus of around 120 GPa.

A simple linear interpolation shows that the rate of increase in beam strength is the same in both types of beams; i.e., the gradient $m$ is approximately 0.28. This empirically derived relationship is dependent upon many parameters and should not generally be used unless either sufficient data for the lower and upper bound limits are available for the strength of the beams under study or the mode of beam failure remains unchanged.

The data given in Fig. 4 and Table 3 are those for test beams with square-ended plates. The effect of plate-end geometry, reported elsewhere (Hutchinson and Rahimi 1993), showed that the strength of the beams was not influenced greatly by scarifying and tapering. However, for these particular beams, failure did not initiate at the plate ends. Concrete shear failure dominated, which is intrinsically related to the initiation of failure within the concrete cover.

Two replicate beams (A10 and A11) were strengthened with CFRP laminates bonded along the full length of the soffit of the beams, going underneath the supports. Their failure loads were marginally higher than for those beams with plates terminating before the supports, and they failed in shear, which initiated near the point loads. Triantafillou and Plevris (1990) obtained a significant improvement in beam strength by extending plates under the supports for their particular experiments.

Table 3 shows that there was no reduction in the load-carrying capacity of the CFRP-plated test beams as a result of preloading. The average strength of these beams (A8 and A9) was around 65 kN, compared with 63 kN for the equivalent undamaged beams, and they ultimately failed within the concrete cover.

**Stiffness of Test Beams**

The stiffnesses of the test beams shown in Fig. 7 were obtained from the experimental and numerical load-deflection curves. Fig. 4 showed that bonded external reinforcement does...
not contribute greatly to an increase in stiffness in the elastic range of the beams. However, the stiffnesses of such beams is significantly enhanced in the postcracking region compared to unplated beams, and there may be a linear relationship between the beam stiffness in the postcracking region and the amount of external reinforcement. There may also be a critical beam stiffness level for these systems, as evidenced by the knee in the experimental results. The discrepancy between the experimental and numerical solutions is probably due to errors arising from the manual estimation of stiffness parameters.

The modulus of steel is nearly twice that of the CFRP used here, and it has been shown that this results in beams possessing relatively high stiffness values. The stiffness of these beams is reduced as the steel begins to yield, whereas the high strength of the composites helps to maintain a constant beam stiffness level up to relatively high loads (Fig. 4).

**Variation of Strain in Tensile Rebars, External Plates, and Concrete**

The largest strains in the tensile rebar and external plates occurred at midspan of the beams, as expected. The highest recorded strains were associated with 2-ply CFRP-plated beams, which showed an average strain of around 1.2% (12,000 µε). This figure is similar to the ultimate experimental tensile strain for these composites (Table 2). The lowest strain of almost 5,500 µε was for the 6-ply CFRP-plated beams (A6 and A7). Transverse strain readings across the width of the composite plates were negligible.

Strain in the tensile rebars, unlike the external reinforcement, increased dramatically after yielding had taken place. This behavior is not uncommon because rebar strain increases sharply adjacent to cracks in concrete (Scott and Gill 1984). Fig. 8 shows the predicted variation of strain along the rebars and external plate. At a point along the tensile rebars, 150 mm from the point loads within the beam shear span, high strains were associated with increasing load [Fig. 8(a)]. A similar pattern was found for Type C beams in which a plastic zone in the tensile rebars formed 100 mm from the point loads.

**Variation of Shear Stress along Concrete/External Reinforcement Interface**

The changes in stress along the concrete/bonded external reinforcement interface are shown in Fig. 9. Shear stress plots are employed by many researchers [e.g., Jones et al. (1988) and Chajes et al. (1996)], the most common method of estimating interface shear stress being based on experimental strain measurements along the reinforcement plates (Appendix II). The shear stress varies significantly along the external plates, with peak values occurring near the plate ends [Fig. 9(b)].

Numerical predictions based on nonlinear FE models of shear stress variation along the plates are shown in Fig. 9(a). At low load levels, the peak stresses occur at the plate ends, as expected, but with an increase in load, the profile changes and the location of the peak shear stress depends on the type and amount of external reinforcement. The classical elastic theories of Volkersen (1938) and Goland and Reissner (1944) cannot cope with material nonlinearities such as cracking and damage phenomena in concrete. As such, they always predict large maximum stresses at the plate ends, followed by a plateau region in which the stresses are relatively low.

Fig. 10 shows numerical and calculated average interface shear stresses for three Type A/B CFRP-plated beams (B3, A5, and B6). At ultimate load levels, the peak stress values are higher for beams with thicker plates, demonstrating that the use of an average shear stress concept for beam strength predictions may be misleading for these cases.

Roberts (1989) provided an analytical method for predicting peak shear and peel stresses. In general, Roberts’s solutions for shear stresses are higher than for both the experimental

![Predicted Variation of Strain with Applied Load for Type A/B Beams (Only Half of Beams Modeled) along: (a) Tensile Steel Rebar; (b) External Plates](image-url)
results and the numerical predictions. Further, Roberts's method of analysis inherently predicts that peak shear and peel stresses will occur at the plate ends, which is erroneous.

**Failure Modes of Test Beams**

The types of beam failure are summarized in Table 3. No adhesion failures took place at the bonded interfaces. Plate detachment, when it occurred, resulted typically in a layer of adhesive and cement paste still being attached to the FRP surface. Failure modes fell broadly into two groups, those that initiated within the constant moment zone and those that seemed to occur within the shear span of the beams. The mode of failure was found to be dependent on the type of concrete beam and also on the type and amount of external reinforcement. In contrast to the typical failure patterns reported for steel-plated beams, plate-end peeling was not a primary failure mechanism for any FRP-plated beams.

The control (unplated) beams started developing flexural cracks in the constant moment region at relatively low load levels (4–6 kN). These cracks initiated at the locations of the shear links and then gradually increased in both number and height with an increase in load. As the steel rebars began to yield, the beams started to deflect at a greater rate, with corresponding crack widening.

The failure of strengthened Type A/B beams was characterized by either classical shear failure of the beams or concrete cover failure/plate debonding at ultimate load levels. It should be remembered that the Type A beams were relatively under-reinforced in shear, but although some exhibited shear failure, their failure loads were above the theoretical values.

FIG. 9. Shear Stress Distribution at Concrete/External Reinforcement Interface—Comparison of: (a) Numerical Predictions; (b) Experimental Results

FIG. 10. Comparison of Theoretical and Experimental Shear Stress Averaged along Concrete/External Reinforcement Interface for Three Type A/B Beams

B3 (2-ply CFRP-plated) | A5 (4-ply CFRP-plated) | B6 (6-ply CFRP-plated)
fillou and Plevris (1990) proposed that the plate-debonding failure load is proportional to the shear stiffness $\Sigma \,(GA)$ of the constituent elements of the beams. They addressed various aspects of plate bonding and postulated that debonding happens when the ratio of the vertical crack opening $v$ to the horizontal crack opening $w$ reaches a critical level, i.e.

$$P \approx \frac{v}{w} \Sigma \,(GA)$$  (6)

where $P = \text{applied load}$; $\Sigma \,(GA) = \text{total shear stiffness of beams}$, which includes the contributions from both internal reinforcement and external reinforcement; and the parameter $(v/w),_c = \text{characteristic property of the system}$. These authors proposed that a parametric study should be conducted by varying the amount of internal and external reinforcements to obtain the characteristic properties of a plate-adhesive-concrete joint to give the constant of proportionality in (6).

The shear load in a plate-bonded beam is probably carried by the full depth of concrete and not limited to the area of the tensile rebars. Moreover, aggregate interlocking will play an important contribution to shear stress transfer in a cracked beam. Type B beams had sufficient shear reinforcement to prevent premature shear failure; thus, there was no classical shear failure in these beams. Shear cracks appeared at relatively high load levels, but they did not extend to the compression face and no concrete crushing took place.

The failure pattern of the Type B beams changed as the plate thickness increased from 0.4 mm (2-ply CFRP) to 1.2 mm (6-ply CFRP). There were fewer cracks in the 2-ply CFRP-plated beam, and these did not develop as far as the plate ends: also there was no concrete cover failure.

Extensive nonlinear FE modeling was carried out to establish the underlying causes of failure (concrete shear and crushing mechanisms were ruled out). Principal stresses combine the effect of both direct stress and shear stresses and therefore provide a useful parameter upon which to base a failure criterion. The principal stress variation along the concrete/adhesive interface is shown in Fig. 11. The experimental findings and FE models show that the failure of plated beams is initiated by concrete crushing within the constant moment region or by debonding/concrete cover failure within the shear span. The FE analyses show that there is a limiting principal stress.
value of 1.7 MPa at the concrete/plate interface, beyond which failure is likely to occur just within the concrete. [The tensile strength of the concrete is much less than that of the adhesive (Table 2).] Numerical predictions showed that all Type A/B plated beams failed by this mechanism. Failure of the 6-ply CFRP-plated beams also initiated within the shear span.

The modes of failure in composite-plated Type B beams, depicted schematically in Fig. 12, indicate the likely progression of damage for 2-ply and 6-ply CFRP reinforcement. The proportion of internal tensile steel in Type C beams was 1.68%, whereas Type B had only 0.65% steel. Consequently, these beams exhibited reduced deflection and ductility. Concrete crushing was followed by plate delamination in the CFRP-plated and GFRP-plated Type C beams. In most cases, the concrete cover within the constant moment region also detached along the line of internal reinforcement.

The steel-plated beams behaved differently from the composite-plated beams, the fundamental difference being yielding of the steel. Large transverse strains across the bonded steel plates, due to Poisson’s contraction, led to pronounced curvature. Steel plates on Type B beams began to debond from the edges of the plates, at relatively high loads, leading to large deflections that caused concrete crushing at midspan. Steel plates on the Type C beams developed slightly smaller transverse strains across the plates. These beams failed ultimately by plate debonding that appeared to initiate at the plate ends.

CONCLUSIONS

Based on the particular program of work undertaken involving flexural testing of the 2.3-m concrete beams described, the following conclusions were reached:

- The peel-ply surface treatment technique for plate bonding applications represents a novel approach, and the particular system adopted exhibited good adhesion characteristics both in the short term and when joints were subjected to environmental aging. It represents a very practical method of surface preparation for the on-site treatment of large composite plates. By contrast, the grit-blasting operation for the steel plates proved to be time-consuming and required treatment of both sides of the steel plates to reduce distortion.
- The stiffness and strength of the beams strengthened with composite plates was substantially increased. The ultimate load-carrying capacity of the beams increased by as much as 230% over their unplated counterparts.
- The concrete beams that had been preloaded before bonding had an equivalent performance to the other beams. This indicates the effectiveness of the plate bonding technique in repair situations.
- The magnitude of the performance increase was influenced by the composition of the concrete beams (i.e., tensile and shear reinforcement) and also by the type and amount of external reinforcement. In general, the strength and stiffness of the beams increased with the modulus and amount of applied external reinforcement. For every type of plate-bonded beam, there is a limiting point beyond which no further increase in beam strength can be obtained. The ultimate load-carrying capacities of plated beams depend largely on the characteristics of the cover concrete.
- The locus of failure in Type B beams reinforced with thin laminates was in the cover concrete close to the point loads within the shear span of the beams. However, with an increase in plate thickness, the locus of failure moved toward the plate ends. Thus shear and normal (peel) stresses at the plate ends increase with an increase in plate modulus and thickness.
- Nonlinear FE modeling is a powerful predictive tool for the analysis of externally strengthened concrete beams. In general, the “damage” model for concrete seemed to perform reasonably well in predicting the salient features of composite-plated beam behavior. The numerical predictions were sensitive to the value of concrete tensile strength, but a value of 1.5 MPa provided good correlation with the experimental results.
- The plate debonding/concrete-cover failure mechanism is governed by a limiting principal stress value at the concrete/external-plate interface. The exact location of this value depends on a number of factors, including plate material thickness. For the nonlinear models using a tensile strength value of 1.5 MPa for concrete, this limiting principal stress was found to be 1.7 MPa.
- The present 2D nonlinear model was inadequate for predicting anticlastic deformation of the steel-plated beams due to large transverse strain (necking) across the plates.

APPENDIX I. PROCEDURE FOR CALCULATION OF ULTIMATE MOMENT OF RESISTANCE OF TEST BEAMS

The following equilibrium conditions were assumed for calculation of the ultimate moment of resistance of control (unplated) beams (Fig. 2):

\[ F_u + F_w = F_{co} \]  \hspace{1cm} (7)

where

\[ F_u = f_u \cdot A_u \]  \hspace{1cm} (8)

\[ F_w = f_w \cdot A_w \]  \hspace{1cm} (9)

\[ F_{co} = 0.6f_{co} \cdot bx \]  \hspace{1cm} (10)

The size and amount of conventional (rebar) tensile reinforcements selected were, for Types A/B beams, \( A_u = 157 \text{ mm}^2 \) (two 10-mm-diameter bars), and for Type C beams, \( A_u = 400 \text{ mm}^2 \) (two 16-mm-diameter bars).

Under equilibrium conditions and using compatibility for unplated beams

\[ F_u + \varepsilon_{co} ((30 - x)/x)E_c \cdot A_w = 0.6f_{co} \cdot bx \]  \hspace{1cm} (11)

The ultimate moment of resistance of an unplated beam is given by

\[ (M_u)_{unplated} = F_o(x - x/2) - F_o(x - d') \]  \hspace{1cm} (12)

Similarly for plated beams, under equilibrium conditions

\[ F_p + F_u = F_w + F_{co} \]  \hspace{1cm} (13)

where

\[ F_p = f_p \cdot A_p \]  \hspace{1cm} (14)

where \( f_p \) = tensile strength of external plate; and \( A_p \) = cross-sectional area of reinforcement.

The ultimate moment of resistance of a plated beam can then be calculated using

\[ (M_u)_{plated} = F_o(x - x/2) - F_o(x - d') + F_p(d_p - d) \]  \hspace{1cm} (15)

APPENDIX II. METHOD OF CALCULATION OF SHEAR STRESS VALUES AT CONCRETE/EXTERNAL PLATE INTERFACE USING STRAIN GAUGE READINGS FROM PLATE

Consider longitudinal forces in a short length \( \Delta \lambda \) of an external plate. A change in strain \( \Delta \varepsilon \) over this length corresponds to a difference in load given by
\[ \Delta F = E b t \Delta \epsilon \] (16)

where \( E \) = modulus of elasticity of the external plate; \( b \) = plate width; \( t \) = plate thickness; and \( \Delta F \) = difference in load.

By equilibrium of forces, the force in the external plate must be balanced by the shear force exerted by resin. Thus

\[ \Delta F = \tau \Delta L b \] (17)

where \( \tau \) = shear stress at the interface.

Now by equating (1) and (2), one has

\[ E b t \Delta \epsilon = \tau \Delta L b \] (18)

\[ \tau = \Delta N / \Delta L E t \] (19)

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APPENDIX III. REFERENCES


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APPENDIX IV. NOTATION

The following symbols are used in this paper:

- \( A \) = cross-sectional area of member;
- \( A_s \) = area of tension reinforcement;
- \( A_v \) = area of both legs of link;
- \( a_v \) = shear span;
- \( b \) = width of beam;
- \( d \) = effective depth;
- \( E \) = modulus of elasticity;
- \( F \) = design load;
- \( f \) = stress, strength;
- \( f_{cc} \) = concrete compressive stress at compressive face;
- \( f_{cm} \) = characteristic cube strength of concrete;
- \( f_y \) = characteristic strength of reinforcement;
- \( G \) = shear modulus;
- \( M \) = bending moment;
- \( NA \) = neutral axis;
- \( P \) = applied load;
- \( S_v \) = longitudinal spacing of shear links;
- \( V \) = shear force;
- \( V_{ult} \) = ultimate shear load-carrying capacity;
- \( x \) = neutral axis depth;
- \( \alpha \) = modular ratio \( E_s/E_c \);
- \( \gamma \) = partial safety factor;
- \( \varepsilon \) = strain;
- \( \rho \) = tension steel ratio; and
- \( \phi \) = bar size.

Subscripts

- \( c \) = concrete;
- \( cc \) = concrete compression;
- \( ct \) = concrete tension;
- \( p \) = plate;
- \( st \) = steel rebar; and
- \( ult \) = ultimate.