5

Structural strengthening of concrete beams using prestressed plates

H N G A R D E N A N D G C M A Y S

5.1 Introduction

The aim in prestressing concrete beams may be either to increase the serviceability capacity of the structural system of which the beams form a part or to extend its ultimate limit state. External prestressing can be applied effectively by the use of unbonded tendons or bonded composite plates. In both cases, it is essential that the existing structure can accommodate the mounting of additional elements for the ends of the prestressing components, but the use of tendons usually necessitates the installation of additional deviation supports for tendons to form the longitudinal profile of the tendons. In both cases, it will be essential to investigate fully the magnitude and form of any local stresses which may be induced into the existing structure as a result of the application of additional prestress.

The method of prestressing with externally bonded composite plates combines the benefits of excellent plate durability and structural improvement, due to the plate tension at the greatest possible distance from the neutral axis of the beam. It has been suggested that prestressing with composite plates is a more economical alternative to conventional prestressing methods used in new construction (Triantafillou and Plevris, 1991).

A significant advantage is gained by prestressing in segmental construction (Trinh, 1990). This is due to the compressive strains generated by prestress at the joints between segmental units, these locations experiencing high tensile strain in the absence of prestress. This high tensile strain is associated with high compressive strain at the top of a segmentally constructed member, resulting in failure by concrete crushing. Depending on the prestress magnitude, the likelihood of crushing at joints is either reduced or eliminated. Leeming et al. (1996) reported the ability of externally bonded composite plates to restrict the opening of segmental joints.

The long term shortcomings of conventional prestressing using tendons is that the tendon prestress does not remain at its initial value throughout the life of the structure because various factors bring about prestress losses. The
principal losses are due to the following causes which contribute the proportions shown in brackets to the total prestress loss in post-tensioning construction:

- relaxation of the tendons (5%),
- immediate elastic deformation of the concrete that occurs when prestress is transferred into the beam (2–3%). Tendons that have already been prestressed will experience a loss of prestress due to the shortening of the beam upon the prestressing of subsequent tendons,
- creep and shrinkage of the concrete under the compressive prestress over the service life of the structure (10–20%),
- slippage of the tendons at their end anchorages that occurs when the prestress is transferred into the anchorages,
- friction between the tendon and its duct in post-tensioned members, but this being less of a problem in simply supported beams with fairly flat tendon profiles (1–2%) but more severe in continuous beams with their more profiled tendons (10% or more).

Prestressing with externally bonded plates is also subjected to prestress losses by immediate elastic deformation and long term creep of the concrete, although friction losses do not apply. As with tendon losses due to settlement and deformation at the end anchorages, bonded plates suffer a prestress loss due to the shear transferred through the adhesive and into the concrete by the plate tension. This shear action is sufficient to fracture the concrete even at low prestress levels so it is necessary to install anchorages at the ends of the plate to resist this action.

One of the benefits of applying prestressed composite plates is that the effect of the original internal tendon prestress can be restored. If the prestress loss is severe, the nominal tensile reinforcement will be unable to contain concrete crack widths to within the serviceability limit and the permissible applied load level will be reduced.

The application of externally bonded prestressed plates involves four main stages:

- the prestressing of the plate to that initial force which provides the required long term prestress after losses have been taken into account,
- the bonding of the plate to the concrete member,
- the installation of plate end anchorages that will resist the forces tending to fracture the concrete when the prestress is subsequently transferred into the member and when the member is subjected to an externally applied load,
- the transfer of the plate prestress into the concrete.

The prestressing force is not released until after the plate end anchorages have been installed, in order to ensure that the cover concrete is not...
damaged due to the high shear and peel stresses caused by the plate tension. Investigative work for the ROBUST project, undertaken at the University of Surrey, included interface tests on a concrete/CFRP (carbon fibre reinforced plastic) composite shear specimen in which the applied tensile force was in the plane of the plate. Garden (1997) found that the maximum plate tension that could be transferred into the concrete, before surface delamination of the concrete occurred, was only 6% of the ultimate tensile stress of the plate material. In practice, much higher prestress levels, in the order of at least 25%, will be necessary to achieve a significant improvement in structural stiffness and load carrying capacity of the concrete beam; it has even been suggested that a prestress of as much as 50% of the plate strength may be necessary (Meier et al., 1992)

5.2 Review of previous prestressing studies using composite plates

Triantafillou and Plevris (1991) reported an analytical model developed to describe the maximum achievable level of pretension which can be applied to a composite plate so that the external strengthening system does not fail near the anchorage zones as the pretension is released. It was assumed that failure can occur either by horizontal cracking of the concrete above the fibre reinforced polymer (FRP) plate at the two end zones due to high shear stresses, or by yielding of the adhesive. The analytical model developed is used in a parametric study which suggests that the efficiency of the method is improved by increasing the thickness of the adhesive layer, using a thinner but wider plate of composite material, or by increasing the length over which the plate is bonded, all of these effects theoretically increasing the level of stress which can be applied to the plate, prior to bonding, before failure occurs on release.

Section 2.4.3 of Chapter 2 has given a comprehensive review of the previous prestressing studies using composite plates and it is clear from this review of the work reported in the literature that, whilst this extension of the FRP plate bonding technique is potentially beneficial, given the advantages inherent in prestressing in practice, it remains very much in its infancy. Most of the work carried out to date has merely demonstrated that failure of the system will occur on release of the prestress unless adequate anchorage systems are provided at the plate ends. It has been demonstrated through the programme of beam testing, described in Chapter 4, that for the configurations tested and with shear span/beam depth ratios less than 4.0, failure of a non-prestressed system will occur by separation of the plate from the beam when tested in flexure unless some form of anchorage is provided. It therefore follows that anchorage is an even greater necessity when the plate is given an initial prestress. The provision of anchorages at
the plate ends reduces the shear deformation which occurs within the adhesive layer upon pretension release, thereby reducing the shear stresses transferred to the base of the concrete section and minimising the possibility of premature failure. In the ROBUST investigations, therefore, plate end anchorages were used in all cases so that behaviour on application of external load could be studied, as well as the initial response to pretension release.

5.3 Prestressing technique employed in the laboratory

The technique used to pretension the CFRP plates in the laboratory for both the 1.0m and 2.3m long beams was as follows; the technique was different from that used on site (see Section 5.5).

A aluminium tabs were epoxy bonded at each end and on both faces of the CFRP plate. These tabs were used to provide stress distribution in the end regions and were 2mm thick, 130mm long for both beam sizes and full plate width (80mm for the 1m long beams, 90mm for the 2.3m). Each end of the CFRP plate was then sandwiched between two steel plates 9mm thick which were predrilled and provided a jig into which the tabbed ends of the CFRP plate could be held with pins during drilling. After drilling, 12 6mm diameter bolts were located at each end and tightened to provide frictional as well as shear resistance to the pretressing force to be applied; the CFRP plate with the steel plates bolted on at each end was then loaded into a prestressing frame. The end reaction system is shown in Fig. 5.1.

The sequence of procedures followed in preparing the prestressed beams is shown in Fig. 5.2; methods of gritblasting and cleaning the concrete bond surface, the utilisation of the peel-ply protection on the surface of the composite and the sprinkling of 2mm diameter ballotini to provide the correct thickness of adhesive were similar to those used in Chapter 4 for the non-tensioned plates. After stressing the plate to the required level, the concrete beam was lifted with jacks up to the level of the taut plate, Fig. 5.2(a). A timber plank was then used to hold the beam at the correct height whilst the adhesive cured. Dead weights equivalent to a pressure of around 7.5kN m\(^{-2}\) were applied to the upper face of the plate so that excess adhesive would be extruded, Fig. 5.2(b); in practice, a vacuum bag technique may be necessary to support the external plate during bonding. After the adhesive had cured, steel clamps were installed at each end of the beam to ensure adequate anchorage of the plate ends upon release of the pretension. The load applied to the CFRP plate was then reduced to zero, the pretension being transferred to the beam by the cured adhesive bond layer and the plate was cut through to isolate the beam.
Figure 5.1 End reaction system for prestressing plates.

Figure 5.2 Sequence of procedures in preparing prestressed beams.
GFRP endplates 14mm thick and 40mm long, covering the full width of the CFRP plate were used in the 2.3m and 4.5m beams. These were predrilled and bonded in the correct positions, illustrated in Fig. 5.2(c). After this adhesive had cured and using the glass fibre reinforced plastic (GFRP) endplates as a jig, holes were drilled through the CFRP plate and adhesive into the concrete beam. Steel bolts were then bonded into the holes and allowed to cure, as shown in Fig. 5.2(d); it was necessary for the length of bolt to extend well into the tensile zone of the reinforced concrete beam.

5.4 Results of laboratory tests for concrete beams strengthened with prestressed plates in the ROBUST programme

To illustrate the load/deflection and load/strain responses of reinforced concrete beams strengthened with pretensioned composite plates, typical results from the 2.3m long beams tested under the ROBUST programme will be discussed. The four point load configuration adopted for these beams provided a 400mm long constant moment region in the centre of the beam and each shear span was 450mm long.

The effects of strengthening the 2.3m beams with various plate tensions are summarised in Tables 5.1 and 5.2, whilst the variations in overall member stiffness are given in Table 5.3.

For all the 2.3m beams tested during the project, prestressing the composite plates prior to bonding increased the serviceability load but, as the load was governed by the concrete strength and not by the internal steel, the increase over the unplated case was small and pretensioning the plate

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**Table 5.1 2.3m Prestressing investigation; strengthening effects of plating; \( A_s/bd^4 \) ratio was 1.18% and strength of concrete was nominally 50N mm\(^{-2} \) (prestress as a percentage of the ultimate strength of plate)**

<table>
<thead>
<tr>
<th>Beam</th>
<th>First cracking load (kN)</th>
<th>Increase over unplated (%)</th>
<th>Serviceability load (kN)</th>
<th>Increase over unplated (%)</th>
<th>Yield load (kN)</th>
<th>Increase over unplated (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unplated</td>
<td>13.0</td>
<td>—</td>
<td>47.2</td>
<td>—</td>
<td>76.5</td>
<td>—</td>
</tr>
<tr>
<td>Plated 0%</td>
<td>15.0</td>
<td>15.4</td>
<td>53.3</td>
<td>13.0</td>
<td>100.0</td>
<td>30.7</td>
</tr>
<tr>
<td>Plated 34.7%</td>
<td>22.5</td>
<td>73.1</td>
<td>53.3</td>
<td>13.0</td>
<td>115.0</td>
<td>50.3</td>
</tr>
<tr>
<td>Plated 41.7%</td>
<td>34.0</td>
<td>162.0</td>
<td>53.3</td>
<td>13.0</td>
<td>124.0</td>
<td>62.1</td>
</tr>
</tbody>
</table>

\(^1\)\(A_s\) is the area of tensile steel rebars, \(b\) is the breadth of the beam and \(d\) is the depth of the beam, respectively.
Table 5.2 2.3 m Prestressing investigation; ultimate loads and ductilities; $A_s/bd$ ratio was 1.18% and strength of concrete was nominally 50N mm$^{-2}$

<table>
<thead>
<tr>
<th>Beam</th>
<th>Maximum load carried (kN)</th>
<th>Increase over unplated (%)</th>
<th>Ductility (%)</th>
<th>Proportion of unplated ductility (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unplated</td>
<td>79.9</td>
<td>—</td>
<td>5.15</td>
<td>—</td>
</tr>
<tr>
<td>Plated 0%</td>
<td>125.6</td>
<td>57.2</td>
<td>4.38</td>
<td>85.1</td>
</tr>
<tr>
<td>Plated 34.7%</td>
<td>129.3</td>
<td>61.8</td>
<td>4.10</td>
<td>79.6</td>
</tr>
<tr>
<td>Plated 41.7%</td>
<td>147.8</td>
<td>85.0</td>
<td>5.49</td>
<td>106.6</td>
</tr>
</tbody>
</table>

Table 5.3 2.3 m Prestressing investigation; stiffening effects of plating. $A_s/bd$ ratio was 1.18% and strength of concrete was nominally 50N mm$^{-2}$

<table>
<thead>
<tr>
<th>Beam</th>
<th>Postcracking stiffness (kN mm$^{-1}$)</th>
<th>Increase over unplated (%)</th>
<th>Postyielding stiffness (kN mm$^{-1}$)</th>
<th>Postcracking stiffness retained (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unplated</td>
<td>7.0</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Plated 0%</td>
<td>8.60</td>
<td>22.8</td>
<td>2.34</td>
<td>27.2</td>
</tr>
<tr>
<td>Plated 34.7%</td>
<td>9.23</td>
<td>31.9</td>
<td>2.34</td>
<td>27.0</td>
</tr>
<tr>
<td>Plated 41.7%</td>
<td>9.30</td>
<td>32.9</td>
<td>2.32</td>
<td>26.5</td>
</tr>
</tbody>
</table>

prior to bonding produced little benefit with respect to the serviceability load over the non-prestressed beam for the given concrete strength.

Figure 5.3 shows a typical relationship between the applied load and maximum beam deflection under further tests at the University of Surrey for the ROBUST programme. In this case the plate was initially pretensioned to 25% of its ultimate strength, as indicated. The curves referred to as ‘unanchored’ and ‘bolted’ in this figure describe the absence of plate end anchorage and the use of plate end anchorage bolts, respectively.

In all cases investigated, a considerable increase in the applied load to cause cracking in pretensioned plated beams was apparent and, from the case considered in Fig. 5.3, a 100% increase over the non-prestressed beam was achieved. The initiation and development of both flexural and shear cracking were much less in evidence for the pretensioned cases than for the non-tensioned beams at comparable loads, demonstrating the ability of the pretensioned plate to limit cracking.

For the beams tested during the ROBUST investigations, it was observed that the serviceability load was governed by the strength of the concrete ($\sigma_u$) and by the ($A_s/bd$) ratio. It should be noted that the results presented in Tables 5.1, 5.2, and 5.3 corresponded to values of $\sigma_u$ equal to 50N mm$^{-2}$...
Figure 5.3 Deflection responses to applied load of beams with and without plate prestress (after Garden, 1997).

and values of \((A_s/\text{bd})\) equal to 1.18\% whereas the equivalent values for the results shown in Fig. 5.3 were 50N mm\(^{-2}\) and 0.76\%, respectively. Consequently, the increases in serviceability over the unplated cases were small in the former case and relatively large in the latter.

From Fig. 5.3, the yield load was found to be 14kN for the unplated beam, 21kN for the plated unanchored non-prestressed case and 24kN for the plated bolted non-prestressed cases, giving increases of 33\% and 71.4\%, respectively. Prestressing the plate was again found to increase the yield load; an effective prestress of 25\% produced a value of 31kN, an increase of 121.4\%.

The CFRP prestressing was also found to produce a moderate increase in the maximum load carried by the plated beams compared with the non-prestressed ones. The nominally 50\% prestressed plated case failed by plate tensile fracture, whilst the nominally 25\% prestressed plated beam failed by plate separation due to a vertical shear crack opening of the form described in Chapter 4, Section 4.3. Therefore, the lower the ductility the more brittle the failure, as the collapse mechanism changes from plate separation to plate failure in flexural tension.

A significant benefit of prestressing the plate is that the composite action between the plate and the concrete, at the ends of the plate, improves as the prestress increases. This result is illustrated in Figs. 5.4 and 5.5, in which a comparison is made of the increments in plate tensile strain at various points along the plate, under increasing applied load. Both figures represent beams with bolted plate end anchorage with a shear span/beam depth ratio of less than 3.4. However, it should be stressed that all pretensioned plates
require anchoring at their ends although non-tensioned plates above a shear span/beam depth ratio of 4.0 may not. The increased plate strain at the plate ends indicates an improvement in the ability of the adhesive layer in the vicinity of the plate ends to transfer shear, although the precise mechanism by which this improvement comes about remains the subject of further research.

Figure 5.4 Plate strain distributions with no initial plate prestress (after Garden, 1997).

Figure 5.5 Plate strain distributions with a 40% initial plate prestress (after Garden, 1997).
Figures 5.6 and 5.7 show, for two 4.5m long beams, the strain values at various heights in the plated beams for the non-tensioned and tensioned plates, respectively; the legends in the figures indicate the applied loads. The thick line, marked 0kN in Fig. 5.7, represents the strains in the section after prestress transfer, but before external loading of the beam, for the case in which the plate was prestressed nominally to 50% of its ultimate strength before bonding. Furthermore, when the applied load was 8kN, almost the

![Figure 5.6 Section strains without plate prestress (after Garden, 1997).](image)

![Figure 5.7 Section strains with 50% nominal plate prestress (after Garden, 1997).](image)
full depth of the beam was under a small compressive stress and on subse-
sequently applying external loads greater than 8kN, the neutral axis level
remained lower in the prestressed case than the non-prestressed one. The
high tensile strains, in the prestressed case, at the level of the composite
plate (Fig. 5.7) reflect the prestress tension applied to the CFRP composite
plate before the beam was externally loaded; the values at this level are the
total strains in the composite plate.

A consistent feature of plated beams with end anchorage under flexural
loading is that the curve of applied load against plate strain after yield of the
internal rebars is common for non-prestressed and prestressed members
(Garden et al., 1998); this is illustrated in Fig. 5.8. It is shown that the
non-prestressed member without plate end anchorage exhibited a lower
post-yield stiffness than the non-prestressed beam with the bolted plate
ends. This is a typical result for beams loaded with a shear span/beam depth
ratio equal to or lower than 4.0 due to the stiffening influence of the plate
end anchorages (Garden and Hollaway, 1997). This would indicate that the
post-yield stiffness of a plated beam with end anchorage is not governed by
the plate prestress and that the locus of yield points, with increasing pre-
stress, defines a straight line. These features have been discussed by G arden
(1997) in terms of the material properties and curvatures of plated sections
under the applied load.

During the investigations with non-prestressed plates, it was noticed
that sometimes the adhesive layer suffered cracking through its thickness
and along the interface with the composite plate, due to the propagation of
shear cracks through the depth of the concrete (G arden, 1997; G arden and
Hollaway, 1997). When the plate prestress is of the order of 20% of ultimate, this adhesive cracking can be considerably retarded or even eliminated. In practice, this characteristic would improve the durability of the system by reducing the moisture ingress at the level of the adhesive and its interfaces.

From a study of the number and distribution of shear cracks in the shear spans of beams loaded under symmetrical four point bending, Garden (1997) demonstrated that, for a given applied load with increasing plate prestress, the number of shear cracks reduces in relation to distance from the loading position. This observation reflects the improvement in durability that may be expected with a prestressed composite plate, compared with a non-prestressed one.

5.5 Results of field investigations of concrete beams strengthened with prestressed plates in the ROBUST programme

Two 18.0m long prestressed concrete beams were selected from the same source as those used for the field investigations with non-prestressed plates, presented in Chapter 4. The lower three internal tendons in one of these beams were destressed, as described in Section 4.5, prior to the application of the prestressed plate. A four point bending load test, undertaken by the Royal Military College of Science, using the loading configuration detailed previously in Section 4.5.3, was then undertaken within the elastic range to establish the unplated member stiffness. In the second beam, the three lower tendons were destressed after the beam was plated; the prestressed plates were monitored to determine their increase in strain due to the destressing of the internal tendons.

It was intended that both beams should be strengthened with three prestressed CFRP plates in a single layer, each plate being 90mm wide by 1.0mm thick bonded with Sikadur 31 PBA adhesive. The plates were 6.0m long in the first beam and 15.8m in the second. Table 5.4 summarises the test parameters.

<table>
<thead>
<tr>
<th>Beam no.</th>
<th>Overall plate dimensions (mm)</th>
<th>Free/ Anchored plate ends</th>
<th>Plate length (m)</th>
<th>Maximum unplated loading (kN)</th>
<th>Maximum plated loading (kN)</th>
<th>Monitored during coring</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>270 × 1</td>
<td>A</td>
<td>6.0</td>
<td>62.5</td>
<td>100&lt;sup&gt;1&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>180 × 1</td>
<td>A</td>
<td>15.8</td>
<td>—</td>
<td>83</td>
<td>Yes</td>
</tr>
</tbody>
</table>

<sup>1</sup>Beam failed under core holes by plate debonding.
In order to stress the plates it was necessary to bond a purpose-designed anchor block to each end of the plate. The purpose of anchorage blocks was explained in Chapter 4. The anchor blocks in the 18.0m beams were 530mm by 100mm by 15mm GFRP tabs, illustrated in Fig. 5.9. The anchor blocks were prebonded to the CFRP under factory conditions using the adhesive, 9323-2B/A, manufactured by 3M. The plates were then transported to site and prestressed to 30% of their ultimate tensile load using a prestressing device mounted on and reacting against the concrete beam, as shown in Fig. 5.10. For safety reasons, the prestressing device and the bolted anchorage system were designed to resist a load equivalent to twice the ultimate tensile strength of the layer of three plates.

Once stressed, the plates were pressed against fresh adhesive, spread to a predetermined thickness on the beam soffit, using a temporary timber propping arrangement. This was necessary to accommodate the upward camber present in the beams which were already prestressed under the action of the internal tendons; the propping system remained in place until the adhesive had cured.

This prestressing procedure was the first of its kind to be used successfully under site conditions. The work proceeded satisfactorily in all respects, except for the sudden and unexpected failure during stressing of one plate in the second beam (no. 8). Subsequent examination of the failed plate revealed a weakening due to an overlap between successive peel-ply sheets which were entrapped within the body of the plate during manufacture. Important quality control lessons were learned from this work, of particular relevance to prestressed plates which experience higher local stresses than non-prestressed composites.

The curve of load against deflection for the plated beam, no. 7, is shown in Fig. 5.11, compared with the comparable unplated member. This shows that there was a considerable increase in stiffness as a result of applying the prestressed plates, consistent with the smaller scale laboratory findings. This increase in stiffness correlated well with the upper bound stiffness which was predicted using three-dimensional finite element analysis. Figure 5.11 also shows the load–deflection behaviour of beam 5 which was strengthened with a similar configuration of non-prestressed plates (see Section 4.4); the greater stiffness, arising from the plate prestress, is clearly seen.

Beam 7 achieved an ultimate load of 100kN before failing by plate debonding under one of the core locations (see Fig. 5.12(a) and (b)). It is believed that this premature failure arose as a result of some damage to the two remaining prestressing tendons in the beam, which occurred during the drilling operations to provide fixings for the plate prestressing device.

In beam 8, coring to destress the internal tendons was undertaken after the completion of the plate prestressing and plate strains were monitored as the tendons were destressed. The result from the strain gauge immediately
Figure 5.9 Plan of stressed anchor plate.
beneath the core hole shows the development of tensile strain in the plate as the internal tendon was cut. As coring took place at other locations, these peak strains were redistributed along the length of the plate. There was no sign of any distress to the beam during this coring operation. The beam was then retested and gave the load-deflection response shown in Fig. 5.13. The stiffness of this plated beam exceeded that predicted by the finite element analysis.

5.6 Observations

The prestressing studies reported previously in the literature, together with the findings of the ROBU ST investigative work, lead to a number of significant conclusions regarding the behaviour of beams strengthened with prestressed plates. The more important points may be summarised as follows:

• It has been shown that when bonding composite plates to concrete, the concrete is unable to withstand peeling forces greater than about 5% of the plate strength and, therefore, it is necessary to provide an effective anchorage at the ends of the plate for prestressing forces greater than this value.
Figure 5.11 Load deflection plots for beam 7.
Figure 5.12 Failure mode for beam 7.
Figure 5.13 Load deflection plots for beam 8.
• It is essential that anchorage bolts are well bonded into the concrete to prevent concrete fracture after prestress transfer and during subsequent loading; the bolts must extend beyond the level of the internal tensile rebars.
• Composite action between the plate and concrete is maintained when the plate prestress is transferred to the concrete.
• Since the lower half of the beam is placed in compression by release of the prestress in the plate, flexural cracking will be much less extensive and develop at a later stage under load compared to an identical non-prestressed beam. This effect of controlling cracks is significant in enhancing the durability of the reinforced concrete with respect to corrosion.
• Prestressing will increase the serviceability load of a given member as long as the stress carried by the internal steel is the governing factor; there is little benefit to be gained when the strength of the concrete governs the serviceability load.
• Prestressing increases significantly the applied load at which the internal steel begins to yield compared to a non-prestressed beam.
• Prestressing the plate prior to bonding also affects the mode of failure. The plate puts the lower half of the beam into compression throughout its length, confining the concrete and reducing the extent of shear cracking which could initiate shear or shear step failure. As a result, the failure will generally occur at the adhesive/CFRP interface or within the bottom layers of the concrete; this results in an increase in failure load for cases governed by a failure mode associated with shear.
• For a non-stressed plated beam where the shear span/beam depth ratio is below 4.0, the failure mode is likely to be due to plate separation initiated at a wide shear crack. However, prestressing the plate prevents or reduces the opening of the shear cracks and a flexural failure is more likely.
• Under a given applied load, the position of the neutral axis is lower when the plate is pretensioned prior to bonding. Consequently, more of the concrete is in compression, resulting in a more efficient use of the material. Furthermore, the heights of the tensile cracks are reduced.
• With increasing applied load, a prestressed plate will delay the point at which the number and extent of shear cracks become significant.
• A prestressed plate with a smaller cross-sectional area can achieve the same benefits of strengthening compared to that of a non-prestressed plate (i.e. improved cracking, yield and ultimate loads), thereby reducing the material cost, although the installation costs will be greater.
• The field investigation demonstrated the feasibility of installing externally bonded prestressed plates on site and has opened the way for further trials on bridge decks and other structures. Beams with pre-
stressed plates withstood higher ultimate loads and were stiffer than their counterparts with unstressed plates. The site tests also demonstrated that the provision of prestressed plates can act as a ‘safety net’ against the failure of existing tendons in prestressed concrete elements; non-tensioned plates will provide the same effects, assuming that they are adequately anchored at their ends. This failure may occur as a result of corrosion, the reason why the bridge from which the full size test beams were taken was demolished.

5.7 Concluding remarks

When strengthening a member, the level of prestress that can be applied will be limited by the tensile strength of the plate; tensile failure of the plate should not precede either yielding of the internal steel or compressive failure of the concrete to ensure adequate ductility. However, it should be recognised that, as the level of pretension is increased, so the stiffness of the strengthened member is also increased, thereby reducing the tensile and compressive strains. In this case, the member to be strengthened must have adequate shear capacity for the enhanced ultimate load. This requirement is complicated by the fact that increasing plate pretension provides greater crack containment, which increases the diagonal tension strength of the section and may prevent the ‘peeling off’ failure mode associated with shear ‘steps’; this latter may govern failure at lower prestress levels. From the test programme, it appears that the level of FRP pretension may also have to be limited by the strength of the plate end anchorages, by the horizontal shear strength of the adhesive/FRP plate interface and by the bottom layers of concrete.

5.8 References
