EXPERIMENTAL INVESTIGATION ON THE DUCTILITY OF BEAMS PRESTRESSED WITH FRP

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A perceived problem with the use of FRP materials for prestressing beams is their linear elastic behaviour; this results in a brittle structural failure. However, by using the plastic capacity of concrete and by enhancing it, the ductility of structures can be increased. To test this idea, an experimental programme was conducted that included simply supported rectangular prestressed concrete beams. Some beams were prestressed using steel tendons and others using Aramid Fibre Reinforced Plastics (AFRP) tendons. For the compression zone of the AFRP beams three types of concrete were used: normal concrete, steel-fibre reinforced concrete (FRC) and concrete confined using an AFRP spiral. The beams were loaded and unloaded at regular intervals to determine the energy dissipation taking place. The results show that ductile behaviour can be achieved using FRP prestressed concrete sections, provided that over-reinforced beams with confined concrete are used.

INTRODUCTION

FRP reinforcement exhibits elastic behaviour up to failure, without the typical yield plateau of steel. The behaviour of concrete beams prestressed by FRP tendons is non-linearly elastic up to failure. As a result there have been questions concerning the ductility of such structures.

As far as the ductility of FRP structures is concerned, much research has been concentrated on finding ductility measures equivalent to the ones existing for steel. It is, however, important to focus on what ductility means and what are the requirements for a structure to behave in a ductile manner.

When designing a structure, two issues are of concern:
(a) To make sure the design model is safe.
(b) To ensure that the structure should give warning of failure and should not fail explosively or catastrophically.

The first requirement means that the structure should have enough rotation capacity to allow the use of the plasticity theorems. In seismic regions it should also be able to dissipate energy. The second requirement
means that the structure at failure should exhibit strain hardening behaviour and only a small amount of elastic energy can be released suddenly.

In order to improve the ductility of FRP reinforced and prestressed beams, the following measures have been suggested:

(a) The use of partially bonded FRP;
(b) The use of hybrid FRP with a bilinear stress strain curve;
(c) The use of compression reinforcement;
(d) The step layering of the FRP's to have a progressive failure;
(e) Improvements in the strain capacity of the concrete.

This paper looks at how the improved strain capacity of concrete can be used and how it influences the beam behaviour.

TEST PROGRAMME

The experimental programme consisted of tests on beams in flexure. Single span beams were tested by applying a load at each of the third-span points (Figure 1).

![Figure 1 Beam loading arrangement](image)

These beams were used to study the ductility behaviour and the localisation effects due to the different reinforcement and concrete properties. The beams were under- or over-reinforced and were of different beam sizes.

By changing the percentage of reinforcement the beam can be forced to fail in an under- or over-reinforced manner.

Two different sizes of beams were tested. The properties of the beams are described in Table 1, in which $f_{cu}$ is the cube strength at the day of testing. Figure 2 shows the beam cross section.

![Figure 2 Beam cross section](image)

The tendons were made from two materials, steel and Aramid FRP. Two types of steel tendons were used. For the small beams and for the under reinforced large beam, 5mm prestressing wire was used. For the over-reinforced large beam, a seven-wire 15mm strand was used (Table 2).

To facilitate prestressing, the AFRP tendons, were connected to a threaded mild steel bar by means of a coupler made from a mild steel tube. The AFRP rod was placed inside the coupler tube and the gap filled with expansive cement. The inside of the tube was threaded on one side. This
The concrete mix was designed so it was possible to transfer the prestress at day 3 and for a cube strength of 50 MPa at testing at day 5.

To confine the concrete, Aramid FRP spirals developed by Leung were used. These spirals are made of aramid fibres with an epoxy resin. Following Leung experiments, a spiral pitch of 20 mm was used.

For the Fibre Reinforced Concrete (FRC), a percentage of 0.5% of fibres by volume was used. The fibres used were hooked-end steel fibre with a 30mm length and a 0.50mm diameter. With this percentage of fibres, a similar behaviour to the confined concrete was expected.

All beams were designed not to fail in shear. The over-reinforced section had a high flexural strength, so shear failure could occur before the flexural capacity was reached. Because the flexural capacity was close to the shear capacity, the over-reinforced AFRP beams were reinforced with stirrups in the shear span. Closed stirrups made of 8mm bars at 150 mm spacing were used. No stirrups were used in the constant moment region in order not to further confine the concrete.

**TEST RESULTS**

During testing, the load was measured using a load-cell and deflections using displacement transducers.

**Load deflection Response**

The load deflection curves for the steel prestressed beams are shown in Figure 3. Figure 4 shows the load-deflection curves for the AFRP prestressed beam and also that for the under-reinforced steel prestressed beam which is deemed to illustrate the desired beam behaviour.
For the beams with FRC, there were further, more closely spaced cracks. The number of cracks in the constant moment region and their spacing are represented in Table 3.

<table>
<thead>
<tr>
<th>Beam Name</th>
<th>No. of Cracks in constant moment region</th>
<th>Average crack spacing (mm)</th>
<th>Maximum crack spacing (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oc-St-UR-L</td>
<td>3</td>
<td>188</td>
<td>213</td>
</tr>
<tr>
<td>Oc-St-UR-S</td>
<td>2</td>
<td>115</td>
<td>115</td>
</tr>
<tr>
<td>Oc-St-OR-L</td>
<td>4</td>
<td>218</td>
<td>231</td>
</tr>
<tr>
<td>Oc-St-OR-S</td>
<td>2</td>
<td>149</td>
<td>149</td>
</tr>
<tr>
<td>Oc-Ar-UR-L</td>
<td>3</td>
<td>257</td>
<td>276</td>
</tr>
<tr>
<td>Oc-Ar-OR-L</td>
<td>4</td>
<td>228</td>
<td>248</td>
</tr>
<tr>
<td>Cc-Ar-OR-L</td>
<td>4</td>
<td>192</td>
<td>203</td>
</tr>
<tr>
<td>FRc-Ar-OR-L</td>
<td>5</td>
<td>138</td>
<td>147</td>
</tr>
</tbody>
</table>

**Ductility Measures**

To measure ductility, two different indexes were used. The first was proposed by Mufti\(^3\) and relates to the rotation capacity of the beam. This factor takes into account the increase in moment as well as the increase in curvature or deflection, since unlimited increase in curvature is more beneficial if the moment of resistance also increases. The authors assumed that the beam behaved elastically if the concrete had a compressive strain lower than 0.001. This J-factor can be defined as:

\[
J - \text{factor} = \frac{P_{ult}}{P_{0.001}} \times \frac{\delta_{ult}}{\delta_{0.001}}
\]

where:

- \(P_{ult}\) = Ultimate load capacity;
- \(P_{0.001}\) = Load for a concrete compressive strain of 0.001;
- \(\delta_{ult}\) = Ultimate deflection at mid-point; and
- \(\delta_{0.001}\) = Mid-point deflection for a concrete comp. strain of 0.001

The second index was proposed by Naaman and Jeong\(^6\) and takes into account the elastic and inelastic energies. They argued that large deflection
prior to failure does not necessarily imply good ductility as an explosive behaviour can occur. The following measure of ductility was proposed:

$$
\xi = \frac{1}{2} \left( \frac{E_{\text{tot}}}{E_{\text{elastic}}} + 1 \right)
$$

(2)

where

- $E_{\text{tot}}$ = Total energy stored in the beam; and
- $E_{\text{elastic}}$ = Elastic energy stored in the beam;

In Table 4, the values of these two indexes are presented.

<table>
<thead>
<tr>
<th>Beam Name</th>
<th>$P_{0.001}$</th>
<th>$\delta_{0.01}$</th>
<th>$P_{\text{ult}}$</th>
<th>$\delta_{\text{ult}}$</th>
<th>$J$-factor</th>
<th>$\xi$</th>
<th>Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oc-St-UR-L</td>
<td>23.6</td>
<td>1.3</td>
<td>46.4</td>
<td>30.9</td>
<td>46.7</td>
<td>2.15</td>
<td>Yielding/Compression</td>
</tr>
<tr>
<td>Oc-St-UR-S</td>
<td>8.5</td>
<td>0.5</td>
<td>16.2</td>
<td>19.4</td>
<td>73.9</td>
<td>2.39</td>
<td>Yielding/Compression</td>
</tr>
<tr>
<td>Oc-St-OR-L</td>
<td>38.4</td>
<td>2.4</td>
<td>70.0</td>
<td>21.3</td>
<td>16.2</td>
<td>1.38</td>
<td>Flexural/Shear</td>
</tr>
<tr>
<td>Oc-St-OR-S</td>
<td>12.7</td>
<td>0.7</td>
<td>26.3</td>
<td>9.5</td>
<td>28.1</td>
<td>1.32</td>
<td>Concrete Compression</td>
</tr>
<tr>
<td>Oc-Ar-UR-L</td>
<td>13.5</td>
<td>1.2</td>
<td>23.9</td>
<td>29.5</td>
<td>43.5</td>
<td>1.49</td>
<td>Tendon</td>
</tr>
<tr>
<td>Oc-Ar-OR-L</td>
<td>35.3</td>
<td>2.1</td>
<td>76.4</td>
<td>30.6</td>
<td>31.5</td>
<td>1.33</td>
<td>Concrete Compression</td>
</tr>
<tr>
<td>Cc-Ar-OR-L</td>
<td>34.0</td>
<td>1.8</td>
<td>58.7</td>
<td>50.3</td>
<td>48.2</td>
<td>1.66</td>
<td>Confining spiral</td>
</tr>
<tr>
<td>FRe-Ar-OR-L</td>
<td>36.6</td>
<td>2.1</td>
<td>73.5</td>
<td>23.4</td>
<td>23.4</td>
<td>1.34</td>
<td>Concrete Compression</td>
</tr>
</tbody>
</table>

DISCUSSION

The beams prestressed with AFRP had lower energy dissipation ratios, $\xi$, than the ones prestressed with steel. Among the AFRP prestressed beams, the beam with spirally confined concrete had a higher energy dissipation ratio and a higher $J$-factor; it is also the one with the most explosive behaviour. The energy dissipation ratio, $\xi$, therefore did not measure the explosiveness of failure.

The over-reinforced beams all have roughly the same energy dissipation ratio, however they have quite different $J$-factor values and hence redistribution capacities (Figure 5). Energy dissipation therefore is not a measure of the redistribution capacity.

![Figure 5 Energy dissipation ratio Vs. J-factor](image)

From the results, the smaller beams have a higher redistribution capacity than the larger ones. Among the over-reinforced beams, the AFRP prestressed beams have higher redistribution capacity. The under-reinforced AFRP beam and the beam with confined concrete both have a similar $J$-factor to the large under-reinforced steel beam.

CONCLUSIONS

The following conclusions can be drawn from this study:
(a) Failure of under-reinforced AFRP prestressed beams is sudden but with a large deflection capacity;
(b) The use of confinement in the compression zone of AFRP prestressed beams enhances the ductility behaviour but final failure is explosive; and
(c) Energy dissipation is a requirement for the structure only in seismic regions. It says nothing about the redistribution capacity of the beam nor the explosiveness of collapse.

ACKNOWLEDGMENTS

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REFERENCES