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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).

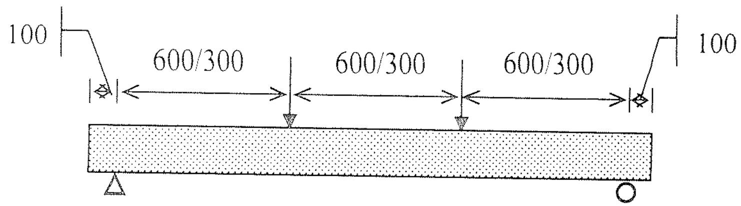

Figure 1 Beam loading arrangement

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.

Table 1 Details of test beams

Beam Name	*b (mm)*	*h (mm)*	A_s *(mm²)*	*Prestress (kN)*	f_{cu} *(MPa)*
Oc-St-UR-L	106	200	58.9	68.3	57.0
Oc-St-UR-S	63	128	19.6	22.3	50.9
Oc-St-OR-L	106	200	137.4	155.5	52.4
Oc-St-OR-S	63	128	39.3	47.9	50.9
Oc-Ar-UR-L	106	200	25.2	41.1	56.7
Oc-Ar-OR-L	106	200	120.7	105.7	48.5
Cc-Ar-OR-L	106	200	120.7	107.7	51.7
FRc-Ar-OR-L	106	200	120.7	106.3	49.3

b – Beam width; h – beam depth; A_s – Area of prestressing tendons

Note: Each beam is identified by a series of letters. The first set of letters indicates the concrete type, the second set represents the tendon material, the third set of letters identifies the type of failure, and the final the size of the beam. The key is as follows:

Oc-ordinary concrete	Cc-Spirally confined concrete	FRc –Fibre reinforced concrete
St – Steel tendon	Ar – AFRP tendon	
UR – Under-reinforced	OR – Over-reinforced	
S – Small beam	L – Large beam	

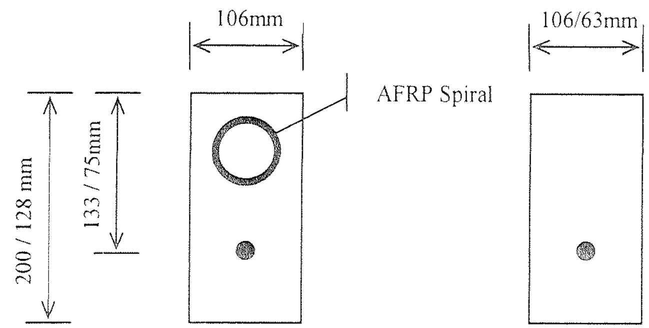

Figure 2 Beam cross section

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).

T-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 technique has successfully been used to stress FRP rods by Harada[1] and Less[2,3].

Table 2 Material properties of the tendons

Material	E_t (MPa)	ϕ (mm)	A_s (mm^2)	P_{ult} (kN)
Steel wire	200,000	5	19.6	34.4
Steel 7 wire strand		15	137.4	245.0
AFRP	54,000	4	12.6	22.7
		14	122.7	227.0

E_t - Modulus of elasticity of tendon
ϕ - Tendon diameter
P_{ult} – Manufacturer's assured load for tendon

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[4] were used. These spirals are made of aramid fibres with an epoxy resin. Following Leung[4] 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.

All the beams behaved in essentially the same way up to failure. The behaviour was linear elastic up to cracking, followed by elastic behaviour with reduced stiffness.

The over-reinforced beams had a higher cracked stiffness due to the larger area of reinforcement. The AFRP beams had a smaller cracked stiffness as the reinforcement had a lower Young's modulus.

For the steel under-reinforced beams, the cracked behaviour was followed by a plateau as a result of the yielding of the steel. The AFRP under-reinforced beam did not have this plateau, as the tendon material was elastic up to failure; the tendon snapped leading to the collapse of the beam.

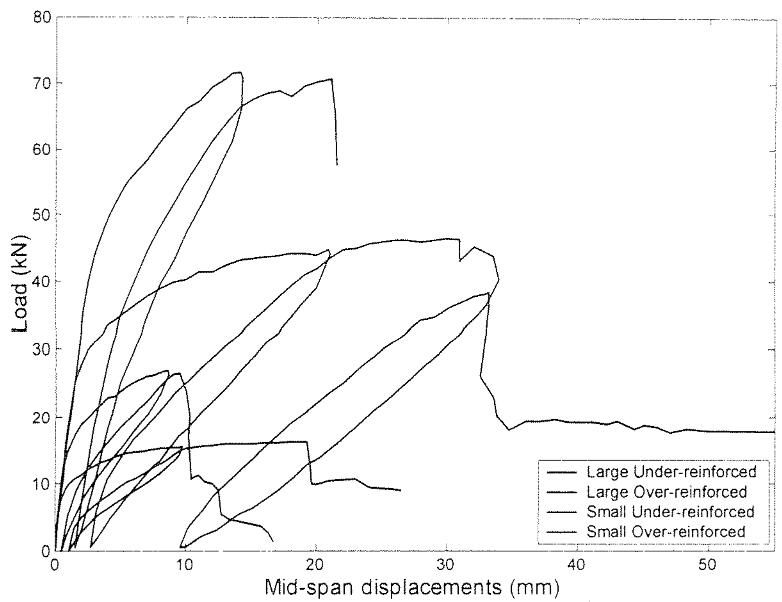

Figure 3 Load-deflection for Steel prestressed beams

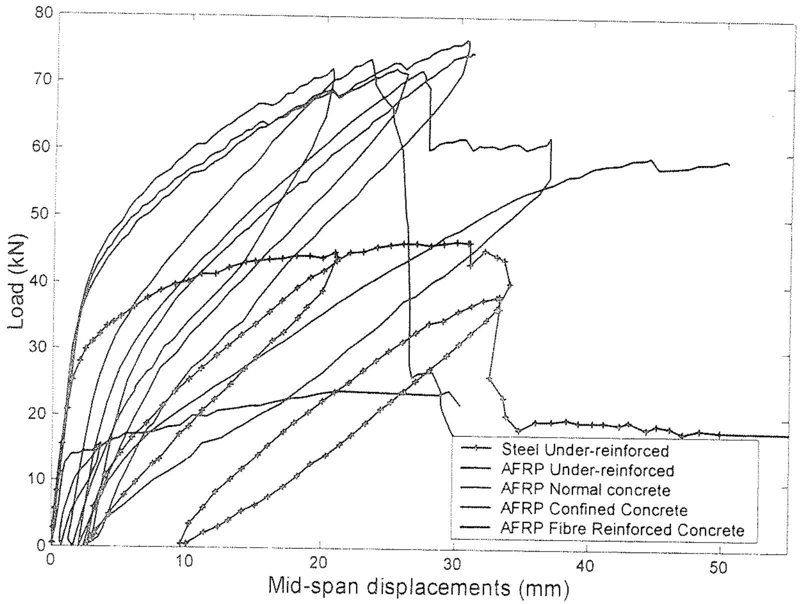

Figure 4 Load-deflection for steel under-reinforced beam and for AFRP over-reinforced concrete with FRC and spirally confined concrete

Apart from the under-reinforced AFRP beam and the over-reinforced AFRP beam with confinement, the final collapse was normally governed by the crushing of the concrete. In the under-reinforced beam failure was sudden due to snapping of the tendon. In the over-reinforced beam with confinement, final failure was quite explosive due to the snapping of the confining spiral. This failure was preceded by a drop in load capacity that resulted from spalling of the unconfined concrete that surrounded the spiral.

The use of the Steel Fibre Reinforced concrete did not change the beam behaviour up to the peak. The failure however was very gentle by crushing of the concrete, which allowed the post peak descending branch to be logged.

Crack Pattern

The governing cracks were mainly flexural and evenly distributed along the beam. Only for the large steel over-reinforced beam did a flexural-shear crack determine collapse.

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 3 Crack behaviour

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

Ductility Measures

To measure ductility, two different indexes were be used. The first was proposed by Mufti[5] 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:

$$J - factor = \frac{P_{ult}}{P_{0.001}} \times \frac{\delta_{ult}}{\delta_{0.001}} \tag{1}$$

where:

P_{ult} = Ultimate load capacity;
$P_{0.001}$ = Load for a concrete compressive strain of 0.001;
δ_{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:

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

where

E_{tot} = Total energy stored in the beam; and
$E_{elastics}$ = Elastic energy stored in the beam;

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

Table 4 Ductility indexes and failure modes for the beams tested

Beam Name	$P_{0.0001}$	$\delta_{0.001}$	P_{ult}	δ_{ult}	J-factor	ξ	Failure
Oc-St-UR-L	23.6	1.3	46.4	30.9	46.7	2.15	Yielding/ Compression
Oc-St-UR-S	8.5	0.5	16.2	19.4	73.9	2.39	Yielding/ Compression
Oc-St-OR-L	38.4	2.4	70.0	21.3	16.2	1.38	Flexural / Shear
Oc-St-OR-S	12.7	0.7	26.3	9.5	28.1	1.32	Concrete Compression
Oc-Ar-UR-L	13.5	1.2	23.9	29.5	43.5	1.49	Tendon
Oc-Ar-OR-L	35.3	2.1	76.4	30.6	31.5	1.33	Concrete Compression
Cc-Ar-OR-L	34.0	1.8	58.7	50.3	48.2	1.66	Confining spiral
FRc-Ar-OR-L	36.6	2.1	73.5	23.4	23.4	1.34	Concrete Compression

DISCUSSION

The beams prestressed with AFRP had lower energy dissipation ratios, ξ, 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, ξ, 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.

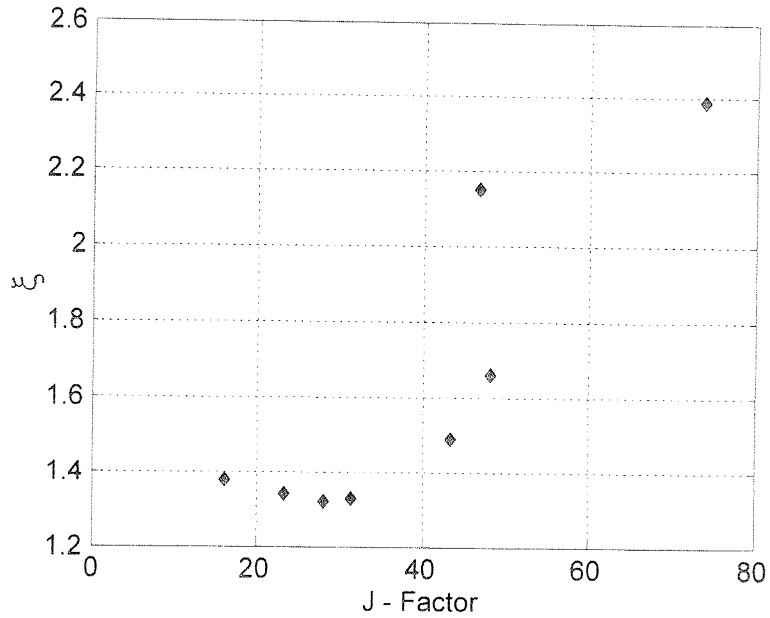

Figure 5 Energy dissipation ratio Vs. J-factor

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.

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