Pre-stressing using BFRP bars: an experimental investigation on a new frontier of precast FRSCC

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ABSTRACT: Basalt fibre reinforced polymer (BFRP) bars are proposed as a pre-stressed reinforcement for precast concrete elements because of their enhanced resistance to aggressive agents which makes them far more corrosion resistant than steel. Moreover, the lower elastic modulus enables a limit on the instantaneous and time-dependant losses and makes their application to pre-stressed precast concrete particularly promising. A full scale experimental investigation has been carried out in the framework of the EU funded research project Eirocrete at Banagher Precast Concrete Ltd on a precast concrete voided lightweight floor slab pre-stressed with basalt reinforced polymer bars and made with fibre reinforced self-compacting concrete (FRSCC). The experiment was aimed at verifying the correct functioning of the pre-stressing system, typically employed for steel tendons, the time-dependant behaviour of the beam during service loading and its resistance, allowing to obtain information about reliability and robustness of this technology.

KEY WORDS: Basalt Fibre Reinforced Polymer (BFRP) bars; Pre-stressing; Steel-free beams; fibre reinforced self-compacting concrete (FRSCC).

1 INTRODUCTION

Extensive research has been performed worldwide in the recent decades with the aim of providing solutions for the substitution of steel reinforcement in concrete members. The main reasons are durability and reduction of carbon emission. Composite materials have been successful in this quest. Among the composites, the production of basalt fibre reinforced polymer (BFRP) bars has been advanced (see Fiore et al. [1]) and its application to concrete has been experimented by Tharmarajah et al., Zhang et al. [2, 3] and applied in practice (Taylor et al. [4]). It has also been used pre-stressing in timber by McConnell et al. [5].

BFRP bars are characterised by a high tensile strength, ranging from 920 to 1650 MPa, and a relatively low elastic modulus, ranging from 45 to 59 GPa (Crossett et al. [6]). These particular features make their application in pre-stressed concrete promising, since their resistance is lower but comparable with that of traditional pre-stressing steel, while having an elastic modulus about four times smaller. This implies that both elastic and long-term pre-stressing losses occurring in pre-stressed concrete elements due to the shortening of the member will be relevantly reduced with the use of BFRP bars. The use of FRP bars in pre-stressed concrete has been studied by Dolan et al. and Zhou [7,8] and extended to BFRP bars by Stoll et al. and Crossett et al. [9,6]. Crossett et al. [10] also studied the BFRP-concrete load transfer mechanism with reference to sandblasted bars.

A challenge for the application of pre-stressing composite bars is the temporary mechanical anchorage prior to the release of the load. All fibre composite materials are characterised by orthotropic mechanical behaviour, with a better performance when the load is applied in the direction of the fibres, while the strength is typically much lower when the load is applied orthogonally with respect to the fibres. The typical anchorage devices used for steel tendons are based on wedges that apply strong lateral pressures to the bars. Solutions to be applied to fibre composite bars have been proposed by Crossett et al., Al-Mayah et al., Carvelli et al. and Schmidt et al. [6, 11, 12, 13] with reference to FRP bars. This paper investigates the structural behaviour of a steel-free pre-stressed slab member designed and tested under the research project IAPP-Eirocrete funded by the European Commission within FP7. The 10m long fibre reinforced self-compacting concrete (FRSCC) slab member was pre-stressed with BFRP bars and had GFRP shear links. Polypropylene fibres were added at 4 kg/m³ of concrete to improve service performance. The design concept and the technical features of the element are presented. The manufacturing of the element is described in detail, highlighting the occurrence of some unexpected issues. The experimental results of a 3-point loading test on the element are reported. Comments about the reliability and robustness of BFRP pre-stressed concrete are finally provided with reference to both its structural behaviour and its technology challenges in an established industry using steel.

2 STRUCTURAL DESIGN OF THE SLAB MEMBER

A voided light-weight section was used due to its popularity in the UK market. Since the cross section, shown in Figure 1, is not symmetrical along the vertical axis, the positioning of the pre-stressing BFRP bars has been studied in such a way to minimise torsional effects. The element has been designed under the assumptions of its use as a roof element, for which...
the additional dead loads other than self-weight, as coming from waterproofing and thermal insulation layers, may be considered negligible. The design condition adopted, without a particular reference to a snow load, is the serviceability limit state, in particular with the goal of having a positive evolution of camber, similar to traditional pre-stressed elements with steel tendons. This meant that the achievement of a pre-camber and a controlled rise in service over the time of the release is generally a design criterion to be pursued.

Due to the use of an innovative material in the beam, the simplified formulation for the estimation of pre-stressing losses provided by Eurocode 2 (EN 1992-1-1:2005) [14], based on the linearisation of the long-term interaction among the phenomena influencing deformability, including viscoelastic member shortening, concrete shrinkage and reinforcement relaxation, was not used. A novel semi-analytical procedure was developed based on the explicit interaction (as Equation 1) with de-coupling of the longitudinal and transversal deformation whilst taking into account the shortening evolution profile to assess the pre-stressing losses. The C45/55 concrete behaviour curves and mechanical properties provided in [14] were used. The average mechanical properties of BFRP (rupture stress $f_t = 900$ MPa and elastic modulus $E_m,b = 56$ GPa) have been considered as previously tested by [3 & 5]. In the design stage, relaxation of the BFRP bars has been neglected. Further details are given in the FP7 deliverable reports with EU Commission.

$$v(t, t_0) = v_e(t) + \int_{t_0}^{t} v_e(t) \dot{\phi}(t, t_0) \, dt \quad (1)$$

12 BFRP bars, placed at 40 mm from the soffit for bond stress distribution in the concrete, were designed for 500 MPa pre-stress, which corresponds to ~50% rupture stress. Figure 1(a) shows the layout including the design of the shear links.

![Figure 1](image1.png)

**Figure 1.** Slab beam cross section: (a) designed, (b) realised.

The maximum bending moment was calculated using mechanical non-linear moment-curvature relationship for this cross-section, including pre-stress losses at 58 days from release of the pre-stressing. BFRP bars were modelled with a linear elastic behaviour. The Sargin curve provided in the ModelCode 10 [15] was used for concrete. The effect of the polypropylene fibres was taken into account in the post-cracking residual tensile stress tests in accordance with EN14617 [16]. The calculated bending moment vs curvature relationship is shown in Figure 2. A maximum resistance equal to 550 kNm was predicted. The post-cracking field is characterised by a sudden load loss followed by a much softer branch which inclination is due to the elastic elongation of the pre-stressing bars. Failure of the BFRP bars in tension is expected. The post-failure residual bending strength of the diagram is due to the residual tensile contribution of the non-pre-stressed bars and the Polypropylene fibres. The design shear load has been calculated considering the contributions of the self-weight and of the predicted maximum mid-span point load to be applied during the test.

The shear reinforcement was 45° inclined 6 mm diameter straight GFRP bars. As they are corrosion resistant, there is no need to keep a minimum clear concrete cover so just a few millimetres cover was used. This also increased the anchorage, which was effective within less than 10 diameters due to the sandblasting surface treatment of the bars. Two shear reinforcement trusses have been designed to be inserted in the ribs. 4 BFRP bars with 12 mm of diameter were placed 2 at the bottom and 2 at the top of the section to fix the transverse bars. These bars were not pre-stressed. Their contribution is included in the moment-curvature diagram.

![Figure 2](image2.png)

**Figure 2.** Non-linear bending moment vs curvature relationship.

Figure 3(a) shows details of the beam, including shear reinforcement. It can be observed that three transversal stiffening ribs at the quarter span, were detailed and formed by cutting the polystyrene blocks which formed the voids. The upper and lower slab have been provided with distributed transversal GFRP reinforcement with 6 mm diameter bars.

In order to investigate the experimental anchorage length of the BFRP bars, a series of optical sensors were fixed to 3 BFRP bars at locations shown in Figure 3(c). Details of the application of optical sensors are reported in Taylor et al. [17] and Mokhtar et al. [18].
3 MANUFACTURING OF THE SPECIMEN

The beam element was cast at the factory of Banagher Precast Concrete. The whole process is illustrated in the following, highlighting the challenges that occurred.

The element was cast in a longer pre-stressing bed. Thus, an inner timber mould was built (Figure 4). The BFRP bars had been ordered with a length of 12 m, which was too short for this bed which had a length of 15 m. In order to solve this, steel couplers were applied externally with respect to the inner mould (Figure 5) and coupled bars of 18 m were used to accommodate the hydraulic jack for pre-stressing and enable anchoring with traditional steel tendons wedges on the opposite side.

Traditional wedges used for pre-stressing steel were chosen, despite the literature survey on the scarce efficiency of those retainers due to the strong concentration of transversal stress in the bars. Nevertheless, several pre-tests performed at Banagher were positive. The problems in literature mainly refer to testing equipment, associated with the objective to break the rebar, while in this study the temporary restraint was for a pre-stress level of 50% of the rupture strength. The optical sensors were installed prior to tensioning at positions calculated taking into account the expected elongation of the cables. Table 1 summarises the mix composition of the FRSCC which has been developed from previous laboratory test [19] with w/b of 0.45 and containing polypropylene fibres (38 mm length) and self-healing admixture (SH). SCC was made with limestone powder (LSP) and ground granulated blast furnace slag (GGBS). The slump flow was 700 mm, J-ring (Pj) was 18 mm and V-funnel time was 20 s.

The BFRP rebars were stressed with a hollow core hydraulic jack with an automatic wedge pushing system, as used for steel tendons. Figure 6 shows the operator (Peter Deegan) stressing each of the bars which were pulled from the same side with an initial stress of 500 MPa. This corresponded to a load of 56.5 kN, and an elongation of 151 mm. The operation was successful and the elongation was checked for each cable and was as predicted. After about 15 minutes from stressing, the first tensile failure of one rebar occurred at the position of the wedge, due to the lateral pressure exerted by it. Within the following 30 minutes, two additional similar failures occurred. A detail of the delamination failure is shown in Figure 7. Figure 8 shows one bar after failure.
The cast was successfully performed and uniform dispersion of fibres was obtained except for some isolated clots which were removed by hand or with the help of a rake. After having poured the lower flange, the shear resisting trusses were inserted in correspondence of the ribs (Figure 9).

The moment-curvature diagram associated with the updated reinforcement layout is shown in Figure 2.

4 TEST SETUP

The slab member (shown in Figure 10 after removal from the mould) was simply supported on top of concrete blocks placed at the edges. 10 mm thick timber slats have been placed between the member and the supporting blocks to distribute the load. The load was applied at mid-span by a +300 mm stroke mono-directional accurately calibrated hydraulic jack with a 50 T capacity used in a steel reaction frame. A steel box beam was placed between the jack and the slab. Figure 11 shows the test rig.

Three digital displacement transducers were placed at the soffit of the mid-span section, one at the centre and two near the beam side face edges. Two additional dial gauges were placed at about 800 mm from the support to measure any settlement at the support region. Load was applied incrementally and in cycles as described in the next section.
5 EXPERIMENTAL RESULTS

The loading was applied in cycles repeated twice at increasing load amplitudes, in order to investigate the crack propagation effect at each load step. A first load amplitude of 25 kN (corresponding to a bending moment of 159 kNm including the contribution of the self-weight) was performed to investigate the pre-cracking behaviour, then the load was increased to first crack formation. The following load amplitudes were gradually increased up to failure.

Based on the updated section (Figure 1b) and on the mean material properties, a failure point load of 145 kN (460 kNm) was predicted. The test results are summarised in Figure 12. The load history (Figure 12a) shows that the beam has attained the expected load, and that it did not fail even at the maximum load reached during the last cycle, equal to 155 kN (484 kNm).

Load vs displacement results (Figure 12b) show that first cracking occurred at an applied load of 70 kN (272 kNm), which was aligned with what expected (280 kNm).

The initial stiffness was maintained up to ~25 kN, due to the effect of the pre-stressing force, after which the behaviour softens with a stiffness degradation depending on the extent of the cracks. Since the BFRP bars are fully elastic, the residual deflection, after large load cycles was very small.

Displacement data from the last load cycle was not recorded as the transducers were removed for safety. An approximated deflection of 150 mm is estimated to have been attained.

The large deflection of the beam at the +120 kN cycle is observable in Figure 13. The crack pattern at the +120 kN cycle is reported in Figure 14. The numbers indicate the pressure level of the pump in bars at the formation of the crack. A factor of 2.33 was calibrated to convert to kN.

A good distribution of cracks allowed to keep the crack opening small. The formation of the cracks at each location typically occurred with a vertical major one with superposing cracks forming at a space equal to the fibre length and converging towards the vertical one.
6 CONCLUSIONS

The manufacturing of a 10 m long steel-free pre-stressed beam with BFRP longitudinal bars, GFRP shear-resisting bars and fibre reinforced SCC showed an interesting technological solution for developing corrosion resistant pre-cast concrete elements even if the pre-stressing of the BFRP bars is limited to half of their rupture stress. Traditional wedging anchorage systems, used for steel, caused the failure of 25% of the bars due to breakage of the outer fibres. A novel anchorage device was developed in lab tests [6] which protects the fibres and allows better stress distribution.

The 3-point loading test showed a satisfactory performance of the element, with an efficient elastic performance of the pre-stressing reinforcement even at a load larger than the predicted resistance, corresponding to mid-span deflections of ~1/70 of the span at peak load. Negligible residual deflection and crack closing were observed after unloading. The element was not taken to failure due to the attainment of an unexpected over-resistance. It has been left under a sustained cracking load in order to investigate the self-healing performance.

The crack pattern of the member showed a very good capacity of crack distribution and corresponding low mean crack opening, which is mainly attributable to the polypropylene fibres.

The shear-resisting truss consisting of GFRP non-bent bars inclined at 45° behaved satisfactorily even in the post-cracking phase.

Based on the observed performance in the reported test, the authors believe that this technological solution is reliable and robust and its application to pre-stressed elements promising, provided that a suitable anchorage system of the BFRP bars during production is adopted.

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