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1 **Short and long-term prestress losses in basalt FRP prestressed** 2 **concrete beams under sustained loading**

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12 **Abstract**

13 The favourable mechanical properties of basalt fibre-reinforced polymer (BFRP) bars, such as their
14 excellent strength-to-weight ratio, resistance to corrosion, lower environmental impact and electro-
15 magnetic neutrality, make them attractive for internal reinforcement of concrete elements with
16 specific service requirements. Prestressing has emerged as a possible method for limiting the
17 deflections and cracking of BFRP reinforced flexural RC elements. However, long-term behaviour of
18 such structural members has not yet been investigated extensively. Therefore, this paper aims to give
19 information on long-term behaviour and prestress losses of pretensioned BFRP reinforced concrete
20 beams based on the collected experimental data. The testing programme includes long-term analysis
21 of six BFRP concrete beams pretensioned to different prestress levels; namely, 20%, 30% and 40% of
22 the ultimate tensile capacity of the bars. The monitored testing stages included initial tensioning of the
23 bars, casting and curing of concrete, transfer of prestressing force to the concrete, long-term unloaded
24 phase after transfer, sustained loading application, long-term sustained loading phase, unloading and
25 final destructive testing, conducted in a controlled indoor environment. During all phases of the
26 testing strain levels in the BFRP bars and deflections were continuously monitored. The results of
27 continuous strain monitoring show that the average loss of strain over the period of initial 90 days of

28 unloaded monitoring was 7% of the initial strain. During the following 6 months of monitoring under
29 sustained loading recorded an additional 0.3% reduction on average. The loss was dependent on the
30 initial strain.

31 **Keywords:** prestress losses, BFRP, long-term, FRP internal reinforcement, PT RC beams.

32 **Background**

33

34 One of the suitable solutions for meeting the design requirements and overcoming the issues primarily
35 related to steel corrosion in aggressive environments can be seen in the use of fibre reinforced
36 polymers. Favourable properties of fibre-reinforced polymer (FRP) materials, such as their low
37 density, high tensile capacity, corrosion resistance etc. have made these composites attractive, despite
38 the slightly higher manufacturing cost, due to their lifecycle advantages. The most commonly used
39 types are carbon FRP (CFRP) and glass FRP (GFRP). More recently, basalt FRP (BFRP) has attracted
40 attention. Similarly to GFRP, BFRP reinforcement is more economically competitive than CFRP for
41 structural applications (Zoghi 2013; Kim et al. 2014) (Zoghi 2013; Kim et al. 2014) . The interest in
42 BFRP emerged due to widespread availability of the raw material used for production of fibres, lower
43 environmental impact (Inman et al. 2017; Pavlović et al. 2022)(Inman et al. 2017), as well as
44 comparable or superior mechanical properties to those of GFRP (Wei et al. 2011)(Wei et al. 2011).
45 However, due to the inherent material properties variability of basalt fibres due to their natural origin,
46 or variation of manufacturing process (Atutis et al., 2018a) a larger variation of basalt composite
47 mechanical properties is possible. This can be remedied employing a conservative engineering
48 approach of utilising nominal strength values at the lower boundary of variation.

49 Up to date, applications for BFRP include structural strengthening (Jason et al. 2018; Suon et al.
50 2019; He et al. 2020; Hou et al. 2020; Madotto et al. 2021)(Jason et al. 2018; Suon et al. 2019; He et
51 al. 2020; Hou et al. 2020; Madotto et al. 2021), reinforcing for buried utility tunnels (Meng et al.
52 2020; Zhou et al. 2021)(Meng et al. 2020; Zhou et al. 2021), flexural elements (Crossett et al. 2015;
53 Douglas and Amir 2015; Dal Lago et al. 2017; Younes et al. 2017; Attia et al. 2020)(Crossett et al.
54 2015; Douglas and Amir 2015; Dal Lago et al. 2017; Younes et al. 2017; Attia et al. 2020) and

55 offshore platforms (Younes et al. 2017; Atutis et al. 2019), to name a few. Investigating the bond
56 durability of BFRP bars in concrete has shown their suitability as an alternative to GFRP as internal
57 reinforcement (Ahmed et al. 2015)(Ahmed et al. 2015), although a reduction in bond retention has
58 been noted for BFRP bars embedded in seawater sea-sand concrete (SWSSC) in marine environment
59 (Zhi-Qiang et al. 2018)(Zhi-Qiang et al. 2018).

60 The determination of mechanical properties and performance of BFRP under different environmental
61 conditions and the understanding of corrosion mechanisms is important for the correct prediction of
62 the service life. The material possesses a significantly higher tensile capacity than reinforcing steel,
63 better insulating properties, as well as more economic manufacturing process compared to CFRP and
64 GFRP (Czigány 2005; Scheffler et al. 2009; Wei et al. 2010; Singha 2012; Dhand et al.
65 2015)(Czigány 2005; Scheffler et al. 2009; Wei et al. 2010; Singha 2012; Dhand et al. 2015). The
66 blast resilience of BFRP reinforced elements is also excellent due to the high deformability of the
67 reinforcement (Feng et al. 2017; Gao et al. 2020)(Feng et al. 2017; Gao et al. 2020). It has been
68 demonstrated that BFRP has good resistance to acidic environments (Wu et al. 2014; Gang et al.
69 2015; Li et al. 2016)(Wu et al. 2014; Gang et al. 2015; Li et al. 2016), and high resistance to both
70 simulated and real marine environments (Gang et al. 2015; Lu et al. 2020)(Gang et al. 2015; Lu et al.
71 2020). Alkali resistance has also been investigated by Gang et al. (2015)Gang et al. (2015), noting the
72 simultaneous influence of temperature and stress level on the degradation of the capacity of BFRP
73 bars.

74 One of the main issues regarding the use of FRPs with low longitudinal elastic modulus, such as
75 GFRP and BFRP, as internal reinforcement of flexural elements, are the large deflections occurring at
76 relatively low levels of load. This leads to a poor utilisation of the FRPs' tensile capacity, as designers
77 are forced to increase the reinforcement ratio to satisfy the serviceability limit state (SLS) criteria. To
78 improve the efficiency of the design, prestressing of reinforcement has been adopted as an effective
79 approach in previous research (Thorhallsson et al. 2012; Pearson and Donchev, 2013; Thorhallsson
80 and Gudmundsson, 2013; Mirshekari et al., 2015; Mirshekari et al., 2016)(Thorhallsson et al. 2012;
81 Pearson and Donchev, 2013; Thorhallsson and Gudmundsson, 2013; Mirshekari et al., 2015;

82 Mirshekari et al., 2016). The evidence shows that prestressed BFRP reinforced elements have
83 improved stiffness, approaching or surpassing that of steel reinforced elements with the same
84 reinforcement ratio at levels of prestress over 30% of the ultimate tensile capacity of the bars. Due to
85 the anisotropy of the mechanical properties of BFRP, the anchorage of tendons is also an area of high
86 importance, especially for prestressing applications; issues such as slipping or crushing of the tendons
87 have been reported when using industrial wedge anchorage (Dal Lago et al. 2017; Motwani et al.
88 2020)(Dal Lago et al. 2017; Motwani et al. 2020).

89 GFRP has been used in prestressing in the form of E-glass and polyester resin bars, E-glass and
90 vinylester resin bars and S-glass and epoxy strands (Rossini and Nanni, 2019). Prestressed beams
91 were tested by Atutis, Valivonis and Atutis (2015) and Zawam, Soudki and West (2019) with
92 prestressing levels of 27% and 25% and 40% of the ultimate guaranteed strength respectively. Rossini
93 and Nanni (2019) concluded that GFRP strands could be used with traditional steel anchors in design
94 of elements prestressed to about 40% of the guaranteed tensile strength.

95 The losses of prestress due to creep and stress relaxation were also reported. Zawam, Soudki and
96 West (2019) observed no relaxation for the first 277h for GFRP bars made of continuous glass fibres
97 impregnated in Vinyl-Ester resin. Nkurunziz et al. (2005) tested GFRP bars made of high strength E-
98 glass fibres with vinylester resin in alkaline conditions and de-ionised water and stressed to 25 and
99 38% of the guaranteed tensile strength. The creep test based on 10000h revealed strain increase of 3%
100 for 25% level of prestress and 5% for 38% prestress. Elevated temperature of 60° C caused strain
101 increase of 8% after 114 days for 38% prestress.

102 A smaller number of studies to date has focused on time-dependent properties of the BFRP material
103 and long-term performance. The fatigue performance of BFRP bars in the context of potential
104 application as prestressing tendons has been investigated in (Xin et al. 2016)(Xin et al. 2016),
105 recommending a limit of stress range of 4% f_{tu} and maximum stress of 53% f_{tu} for BFRP bars. In a
106 study of prestressed concrete beams with BFRP bars, Atutis et al. (2018a) concluded that the stress
107 range varying between 55% and 65% f_{tu} can be considered as safe for BFRP bars.

108 Creep behaviour of BFRP has been investigated in (Wang et al. 2014; Sokairge et al. 2020)(Wang et
109 al. 2014; Sokairge et al. 2020), estimating the million-hour creep rupture stress limit between 57.7%
110 and 62.3%. These values can reportedly be improved to up to 63% by preconditioning of the bars via
111 pretensioning for a short duration (Shi et al. 2015)(Shi et al. 2015). A similar conclusion about the
112 reduction of creep rate by short-term pretensioning of bars at low levels of stress, has been
113 demonstrated in a study of creep at low levels of stress, up to 40% f_{tu} (Pavlović et al., 2021),
114 observing also a stabilisation of the deformation after a relatively short time interval. Stress relaxation
115 has been shown experimentally to depend on the initially applied stress, with measured 1000-hour
116 relaxation of close to 6% (Atutis et al. 2018b).(Atutis et al. 2018b).

117 However, long-term behaviour of pretensioned BFRP reinforced concrete elements is insufficiently
118 investigated to date, in particular regarding losses of prestress. The current structural design codes do
119 not provide clear guidance for the calculation of this fundamental design parameter. Information
120 about the prestress losses is also very limited in the available published research, both from
121 experimental (Pavlović et al. 2019b)(Pavlović et al. 2019b) and numerical studies (Atutis and
122 Kawashima 2020)(Atutis and Kawashima 2020). Hence, this paper aims to experimentally investigate
123 the losses of prestress as well as flexural behaviour of pretensioned BFRP reinforced concrete beams
124 via long-term monitoring.

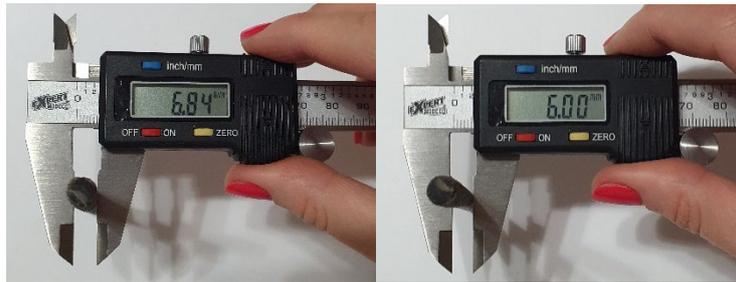
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126 **Methodology**

127 **Samples**

128 The experimental study was based on the investigation of six concrete beam samples. All beams were
129 1200 mm long with a cross section of 130x180 mm. The main (tensile) reinforcement consisted of
130 two 6 mm diameter BFRP bars. The BFRP bars used in this experiment were straight pultruded bars,
131 with surface sand coating, 85_{wr}% fibre fraction (by weight) and epoxy resin. The nominal diameter of
132 6 mm was used in all calculations; the measured diameter was 6.84 mm (standard deviation 0.04 mm)
133 including the sand coating, while the measured diameter after removal of the sand coating was 6.00
134 mm (standard deviation 0.01 mm). The reported measured values were based on average of 5 readings

135 taken using a digital calliper with 0.01 mm precision, as shown in Figure 1. They were characterised
136 by average ultimate tensile capacity of $f_{tu}=1278$ MPa (standard deviation 84 MPa) and longitudinal
137 elastic modulus of $E=48$ GPa (standard deviation 1.4 GPa); this was experimentally determined on a
138 sample of 9 bars, following the procedure given in ASTM D7205/
139 D7205M (ASTM, 2016) for tensile testing of FRP. Ultimate tensile capacity was adopted as 1200
140 MPa for the purpose of all calculations in this paper, which agrees also with the reported minimum
141 capacity provided by the manufacturer.



142

143 *Figure 1 Measurement of the BFRP bar cross section diameter with (left) and without (right) sand coating*

144

144 In addition, 6 mm S275 links were used to prevent shear failure, except in the pure bending zone
145 (middle 300 mm). Top reinforcement of two 6 mm diameter S375 steel bars was also used to
146 construct the reinforcement cage. The top bars were not designed as compression reinforcement due
147 to the expected mode of failure governed by rupture of the bottom BFRP bars, and were provided to
148 keep the shear reinforcement in the correct position.

149

149 The ends of the BFRP bars were connected to threaded steel bars via bonded steel sleeve anchorage
150 (Figure 2), in accordance with the recommendations given in

151

151 ASTM D7205/D7205M (ASTM, 2016), prepared using Weber.tec epoxy structural adhesive. The
152 adhesive was cured at room temperature (20°C) for a minimum of 24h in line with manufacturers

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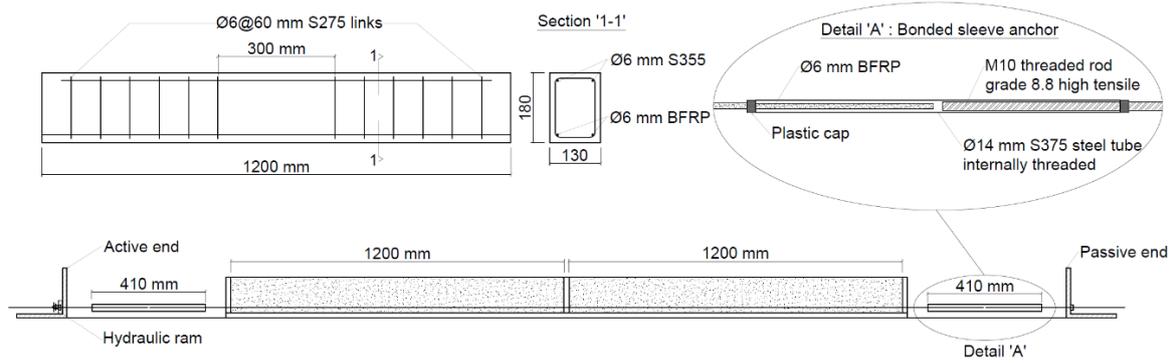
153 guidelines. The threaded steel bars were tensioned using a manual hydraulic ram until the desired
154 prestress level was achieved and then mechanically fastened to steel angle sections securely bolted

155

155 into the reaction floor. All samples were produced in specially prepared formwork with a divider in
156 the middle, which allowed for simultaneous prestressing of two beams.

156

157



158

159

Figure 2 Sample dimensions and prestressing setup

160 After the pretensioning of the bars was completed, all beams were cast using the same concrete mix
 161 (1238 kg/m^3 coarse aggregate, 583 kg/m^3 fine aggregate, 180 kg/m^3 water, 379 kg/m^3 cement). The
 162 curing was done in laboratory conditions for 28 days, after which the threaded bars were released
 163 from the steel angles and the prestress force transferred to the concrete samples. The pairs of samples
 164 were separated by carefully cutting the BFRP bars between them.

165 The samples were cured in controlled indoor laboratory conditions, preventing loss of moisture by
 166 regular spraying of water and provided damp covering sheets. After 28 days curing of the concrete,
 167 the prestress was gradually released from the external anchorage and transferred to concrete by
 168 unscrewing the anchor nuts, whilst clamping the anchor sleeves to prevent torsional effects. The
 169 compressive strength of concrete was tested on the day of prestress transfer, according to BS EN
 170 12390-3:2002 (BSI, 2002). The average of 42 MPa cube strength was obtained on a sample of nine
 171 150 mm cubes, with coefficient of variation $CV=4.5\%$.

172 Testing programme

173 After the release of the prestress, the strain of the BFRP bars was monitored for three months in
 174 unloaded conditions. Then, the samples were loaded using the testing rig (Figure 3), based on
 175 gravitational load and lever-arm principle, with the aim of maintaining a constant load over a
 176 prolonged period of time. The static system corresponds to four-point bending, over a span of 900 mm
 177 and 300 mm distance between the point loads.



178

179

Figure 3. Samples in the sustained loading testing rig

180

The load was applied gradually in steps of approximately 1.5 kN up to the planned level and

181

maintained for 6 months. The samples were subjected to two different levels of loading: 10 kN and

182

14.5 kN as total values monitored by the load cells. The load values were chosen as approximately

183

20% and 30% of the predicted ultimate capacity of the beams, estimated using fib model code EC2

184

based approach for FRP reinforced elements. These values are approximately representative of the

185

permanent service loads for building structures. Both of the loads were below the cracking load

186

estimated using the Response2000 software (Bentz, 2000) as presented in Table 2.

187

The following alphanumeric labelling system was adopted for all samples: $Bn\alpha-L\beta$; where $n=\{20;$

188

$30; 40\}$ depending on the degree of prestress as a percentage of the ultimate tensile capacity of the

189

bars, $\alpha=\{A; P\}$ depending on whether the beams was cast at the active (A) or passive (P) end of the

190

prestressing rig (see Figure 2), and $\beta=\{10; 14.5\}$ depending on the level of externally applied load in

191

kN. The difference between the samples cast at the active (A) or passive (P) end was due to the

192

friction inside the formwork, as the cages and the formwork were assembled prior to the tensioning

193

process. Table 1 details the sample matrix:

194

Table 1. Sample matrix

Sample	Prestress level [% f_{tu}]	Applied sustained load [kN]
B20A-L10	20	10
B30A-L10	30	10
B40A-L10	40	10

B20P-L14.5	20	14.5
B30P-L14.5	30	14.5
B40P-L14.5	40	14.5

^aA = cast at active end; B- cast at passive end. See Figure 1

195 Finally, all samples were unloaded, moved to a bending testing rig and subjected to a quasi-static
 196 four-point bending test until failure, over a span of 900 mm. The test was load controlled at a rate of
 197 0.4 kN/min.

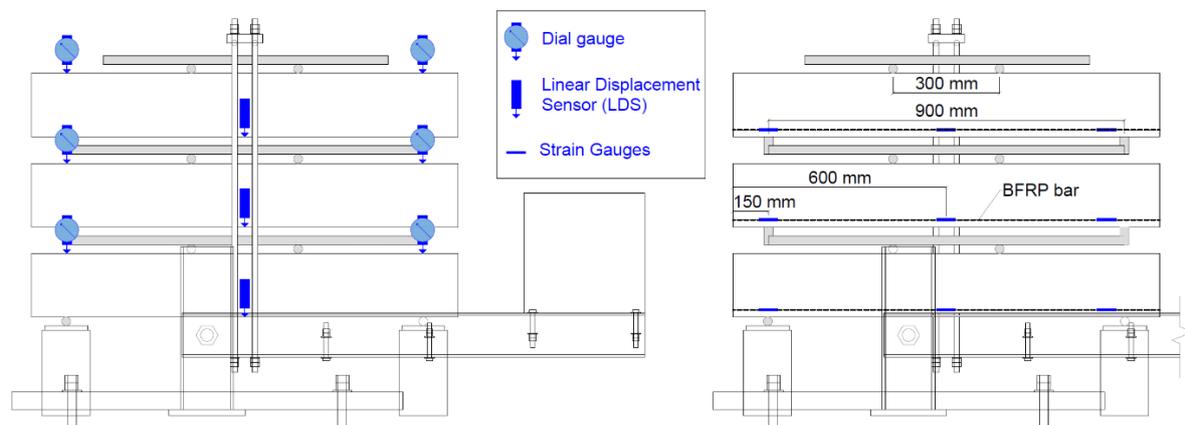
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199 Measuring equipment

200

201 All samples were equipped with 3 Vishay $120 \pm 0.3\% \Omega$ foil strain gauges installed on the BFRP bars,
 202 located at midspan and supports, positioned as shown in Figure 4. Strain was continuously monitored
 203 using the VPG System 8000 data acquisition system at 1 Hz rate.

204



205

206 *Figure 4 Positioning of the measuring equipment*

207

208 During the sustained loading, the samples were equipped with loads cells, linear variable differential
 209 transformers (LVDTs) at midspan connected to the data acquisition system, and dial gauge test
 210 indicators (DTIs) at the supports. The LVDTs used were VPG model HS50, with 1-50 mm stroke,
 211 infinite resolution and $\leq \pm 0.1\%$ accuracy; the DTIs used were Multitoyo Standard Dial Indicators with
 212 0.01 mm precision. The LVDTs and dial gauges were supported by stabilised stands which were

213 placed on the strong floor, adjacent to the testing rig, but with no physical connection to the rig. The
214 environmental temperature was also monitored using a digital thermometer. The potential
215 development of cracks as well as potential slippage of the bars was monitored visually at regular
216 inspections.

217

218 During the final testing until failure, the applied load, displacements and strain on the reinforcing bars
219 were electronically monitored and recorded. The development of cracks was monitored, traced and
220 noted manually.

221

222 **Results and discussion**

223

224 The environmental temperature according to the recorded data was $20\pm 2^{\circ}\text{C}$ throughout all testing
225 phases, with recorded temporary short-term variation of $\pm 5^{\circ}\text{C}$. The relative humidity was also
226 monitored to be within range of $65\pm 3\%$ throughout the entire period. The influence of environmental
227 conditions on the results was therefore considered negligible and was not further analysed.

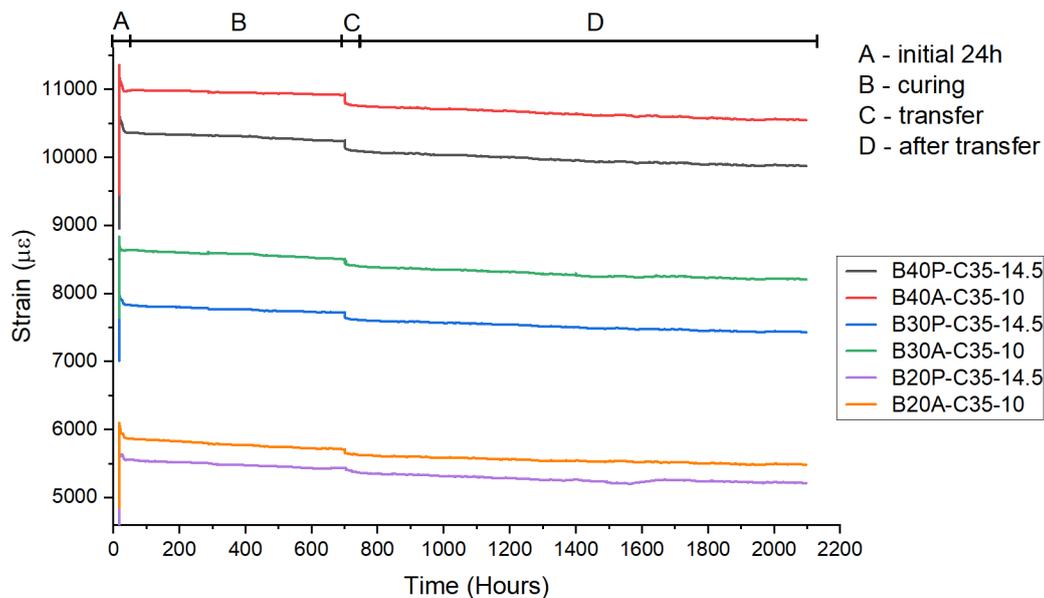
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229 **The phase before the application of the sustained loading - Strains**

230

231 Data acquired during the initial monitoring phase of this experimental programme are presented in
232 this section. Figure 5 presents the recorded strain values measured on the BFRP bars at the midspan of
233 the beams, starting from the pretensioning of the bars, including concrete casting and curing, release
234 of external pretension, resulting in the transfer of the prestressing force to the cured concrete, for a
235 total period of 86 days.

236



237

238

Figure 5 Recorded strain over time at midspan during the unloaded phase

239 As shown in Figure 5 the strain values were decreasing over the entire monitored period. The
 240 observed decrease in strain in the investigated period can likely be attributed to the combined
 241 influence of concrete shrinkage, creep and elastic deformation of concrete upon the release of
 242 prestress.

243 In the initial 24 h from the application of the external prestress (section A), the strain dropped by an
 244 average of 1.86%, at a rate of 0.08% per hour, possibly due to settling of the anchoring devices. Given
 245 that this phase occurs prior to the casting of the concrete, the losses can be recovered by re-adjusting
 246 of the prestress level, which has not been done during these experiments.

247 After the casting of the concrete, the strain decrease was more gradual, at an approximate average rate
 248 of 0.06% per day (section B). This phase represented the process of concrete curing, during which the
 249 losses were largely influenced by the shrinkage of the concrete. The value is dependent on properties
 250 of the concrete and can be limited by prudent design of the concrete mix and an appropriate curing
 251 regime to reduce shrinkage.

252 This was followed by a sudden drop of strain (section C), on average 0.89%, which was a result of the
 253 release of the externally applied prestress and transfer to the cured concrete. As the prestress force is
 254 transferred to the concrete element, the concrete undergoes an instantaneous elastic deformation,

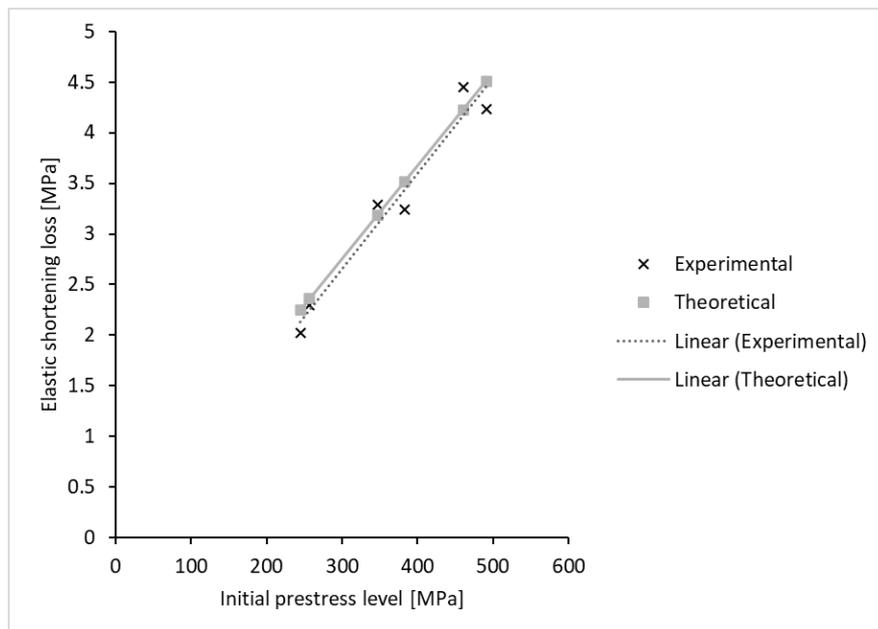
255 which results in shortening of the bars, and thus, loss of prestress. At this stage, the samples with
 256 higher initial level of prestress exhibited a higher loss during prestress transfer.

257 In order to estimate these losses theoretically, a numerical approach in line with Eurocode 2 can be
 258 applied. The instantaneous loss of stress due to an elastic shortening, adjusted for material properties
 259 of BFRP and loading conditions at time of release of prestress (no external load applied) can be
 260 determined by:

$$261 \quad \Delta\sigma_{el} = P_{jack} \times \frac{E_{BFRP}}{E_{cm}} \times \left(\frac{1}{A_g} + \frac{e^2}{I_g} \right) \quad (1)$$

262 where P_{jack} is the prestress force applied by the jack, E_{BFRP} and E_{cm} are the elastic moduli of BFRP
 263 and concrete respectively, A_g is the gross cross-sectional area, e eccentricity of the tendons and I_g
 264 second moment of inertia of the gross cross section. Young's modulus of concrete was considered as
 265 33.34 GPa, calculated using the formula provided in EN1992-1-1 as $E_{cm} = 22 \times (f_{cm} \div 10 \text{ MPa})^{0.3}$.
 266 The results of the calculations are plotted against the experimental results in Figure 6 as a function of
 267 the initial prestress level. A satisfactory correspondence between theoretical and experimental values
 268 can be observed, with average absolute difference of 0.2 MPa.

269



270

271 *Figure 6 Comparison of EC2 based numerical and experimental values of elastic shortening loss.*

272 The final period (section D) was characterised by a steadier rate of decrease in strains of about 0.05%
273 per day. The dominating factors in this phase were rheological properties of concrete and the
274 reinforcing material. At the end of the monitored period, an average total strain decrease by
275 approximately 7.0% was observed for all samples.

276 Applying the approach recommended in EC2 5.10.6(2) (CEN, 1992), the losses of prestress due to
277 creep and shrinkage of concrete can be calculated for each relevant stage, estimating the creep
278 coefficient and shrinkage strain for the given time period and substituting for properties of BFRP in
279 place of prestressing steel. Actual average environmental conditions (temperature, relative humidity)
280 as well as tested concrete strength were used in the calculations to obtain the relevant creep and
281 shrinkage coefficients using standard formulae available in EC2 (CEN, 1992). Figure 7 summarises
282 the calculated prestress losses due to creep and shrinkage after the initial three months, comparing
283 them with average experimentally obtained values at the corresponding stage. Based on this, the
284 existing formulae (adjusted only for properties of BFRP) provide sufficiently accurate prediction of
285 losses at this stage, when compared with experimental values.

286 Theoretically, no creep of concrete is present before the release of prestress, and shrinkage does not
287 depend on the initially applied prestress level, therefore theoretically, the losses in this stage are the
288 same for all samples. During stage D, creep is directly dependent on the level of prestress and is
289 theoretically higher for samples with higher prestress level.

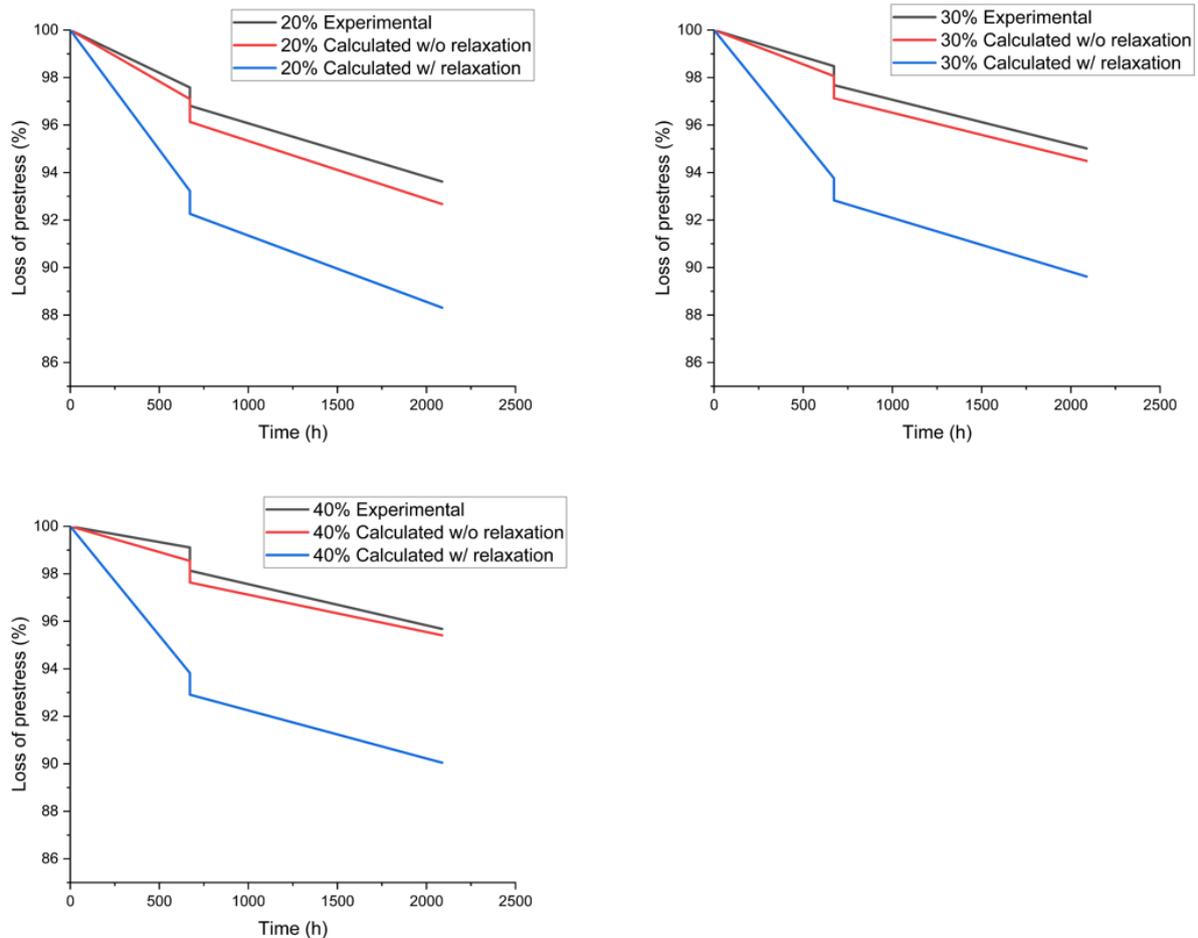
290 Finally, losses of prestress due to relaxation must be also estimated. Using the empirical formula
291 (Equation 2) suggested by Atutis et al. 2018bAtutis et al. 2018b:

$$292 R = (-0.067 \times P^2 + 0.697 \times P + 0.304) \times \ln(t) + 1 \quad (2)$$

293 Where R – relaxation [%]; P – level of prestress [-]; t – time [h]

294 the relaxation losses were calculated and superimposed on the losses due to creep and shrinkage of
295 concrete, and elastic shortening.

296



297

298

Figure 7 Theoretical and experimental decrease of strain during the unloaded testing phase.

299

Experimentally obtained values at relevant phases were also compared with calculated values

300

obtained by using existing formulae available in structural codes, which provides initial confirmation

301

that prestress losses of BFRP reinforced beams can be calculated using a similar approach to steel.

302

Namely, losses due to creep and shrinkage of concrete, as well as elastic shortening can be

303

sufficiently well calculated using EC2 formulae and substituting for material properties of BFRP. Due

304

to the lower longitudinal elastic modulus of BFRP, for the same shrinkage and creep strain and elastic

305

shortening, the losses would be lower than the same type of losses for steel.

306

The formula for estimating relaxation of BFRP currently available in the literature results in losses of

307

similar magnitude as all other types of losses combined. Therefore, in order to take into account all

308

main sources of prestress losses, relaxation of BFRP tendons will need further understanding and

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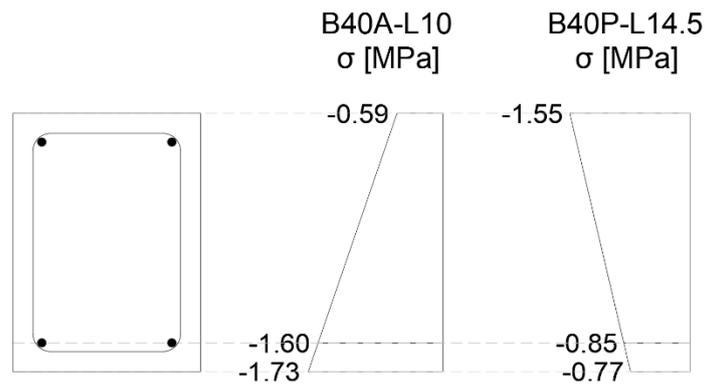
investigation.

310 Sustained Loading - Strains

311 Following the three months of monitoring under unloaded conditions, the samples were transferred to
312 the sustained loading rig and the external load was applied. Strain levels in BFRP bars were acquired
313 and recorded electronically throughout the application of the external load. Samples B20P-L14.5,
314 B30P-L14.5 and B40P-L14.5 were loaded first up to 14.5 kN; the recorded increase in the strain
315 readings was between 76×10^{-6} and 89×10^{-6} at the midspan zone of the bars. Then, samples B20A-
316 L10, B30A-L10 and B40A-L10 were loaded to 10 kN, which resulted in an increase in the midspan
317 strain between 60×10^{-6} and 69×10^{-6} . The strain increased near-linearly with the increase in the load.
318 Figure 9 shows the results of six months of continuous strain monitoring measured at the midpoint of
319 the BFRP bars. The initial point (day 0) of this monitoring period was taken as the day after the
320 sustained loading had been applied. The values indicated as “ Δ Strain” on the vertical axis represent
321 recorded strain readings which were zeroed based on the initial strain at the beginning of this
322 experimental phase. The positive sign in this figure indicates elongation.

323 The variation of strain during the entire period was relatively small, within the range of no more than
324 $\pm 50 \times 10^{-6}$. All samples registered final decrease of the strain in the midspan zone of the bars except
325 for B20A-L10, which exhibited a small increase in the strain readings. The final strain decrease after
326 180 days was below 0.4% of the initial value for all samples, ranging from 0.04% for sample B20P-
327 L14.5, to 0.39% for B40A-L10.

328 To analyse the strain change shown in Figure 9, the monitored period can be divided into three
329 phases. The first, up to approximately 70 days, is characterised by a trend of deformation decrease,
330 indicative of shortening of the bars. After 70 days, until approximately 150 days a slight increase in
331 measured strain is observed for samples B40P-L14.5, B20A-L10 and B30A-L10, indicative of
332 elongation of the bars. The remainder of samples show a tendency to maintain nearly constant
333 deformation. In the last phase, from 150 days until end of monitoring, deformations for all samples
334 remain nearly constant, with only temporary minor fluctuations.

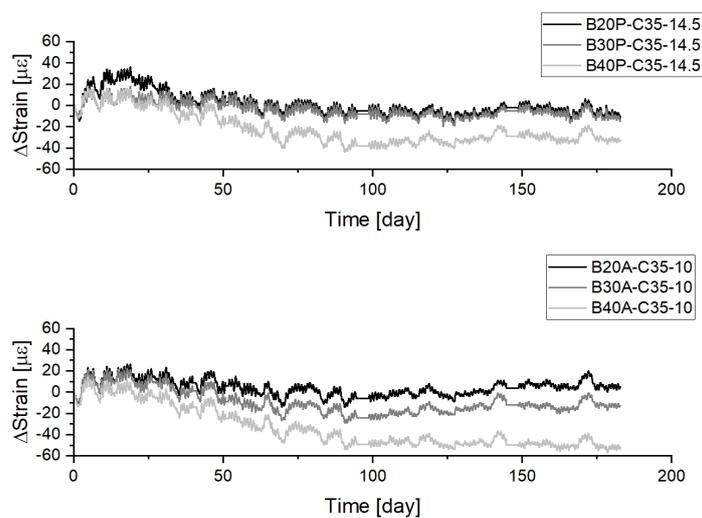


335

336 *Figure 8 Stress profiles for samples B40A-L10 and B40P-L14.5. Negative sign indicates compression.*

337 A possible explanation of any observed shortening of the bars and the corresponding reduction of
 338 strain readings is primarily due to the shrinkage and creep of concrete, under compression induced by
 339 the prestressing. Figure 8 shows the resulting stress profile due to prestress and applied stress profile;
 340 the resulting stress for all samples at the level of the bars was compression, except for samples with
 341 20% prestress level. The greater external load resulted in higher compression at the top fibre (1.55
 342 MPa for sample B40P-L14.5 versus 0.59 MPa for sample B40A-L10) and corresponding reduction of
 343 the compression at the bottom fibre.

344 The deformations due to shrinkage and creep develop throughout the life of the structure; however,
 345 they are the most pronounced in the initial period. The elongation of the bars at later stage can be
 346 explained by increase in the beam curvature due to the loss of stiffness as a result of the prestress
 347 losses.



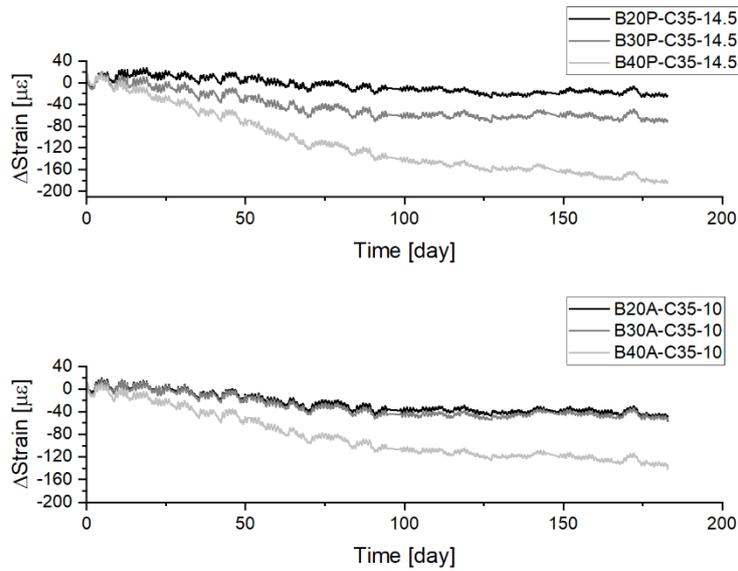
348

349 *Figure 9 Strain variation at midspan during sustained loading*

350 Figure 10 shows the recorded local change of deformation obtained from strain gauges positioned 150
351 mm from the edge of the beam, at the support location, presented in the same way as midspan results.
352 There are two phases that can be defined based on the observed trend for change of deformation. In
353 the beginning, up to approximately 70 days, the local deformation indicates shortening of the bars at a
354 higher rate in comparison with the measured values at midspan. After 90 days, the measured
355 deformations at supports remain nearly constant until the termination of testing, except for samples
356 B40P-L14.5 and B40A-L10, which continued to exhibit shortening, at a slower rate than in the first
357 phase. This can be explained by reduced creep effects as a result of continuous hardening of the
358 concrete.

359 At the supports, the main factor influencing the normal stress distribution is the prestress, as the
360 bending moments from the external load are theoretically zero. Deformational changes at the support
361 locations are, therefore, not subject to direct influence of the external load. The governing factors
362 influencing deformational changes at the supports are shrinkage, and creep of concrete induced by
363 prestressing force, which gives further support to the previous conclusion regarding the behaviour at
364 the midspan. The measured final values of relative deformation change at the support are higher than
365 at the midspan. The creep of concrete is expected to be greater at the supports due to the higher
366 compression stress in this zone at the level of tendon. Consecutively, shrinkage can also be higher,
367 due to the Pickett effect.

368

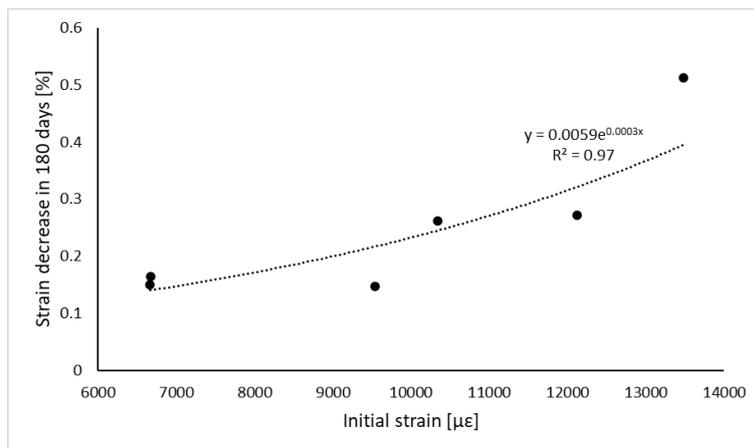


369

370 *Figure 10 Strain variation over time at support under sustained loading (at 10 kN and 14.5 kN)*

371 The measured strain decrease in 180 days at midspan, expressed in percentage of the initial strain
 372 value, was plotted against the initial strain values in order to further examine the relationship between
 373 prestress level and prestress loss (Figure 11). A line of best fit was obtained using the least square
 374 method. As shown in Figure 11, the exponential function approximates the relationship well, with
 375 coefficient of determination $R^2=0.97$. According to this data, the strain decrease over time rises with
 376 initial prestress level, as expected.

377



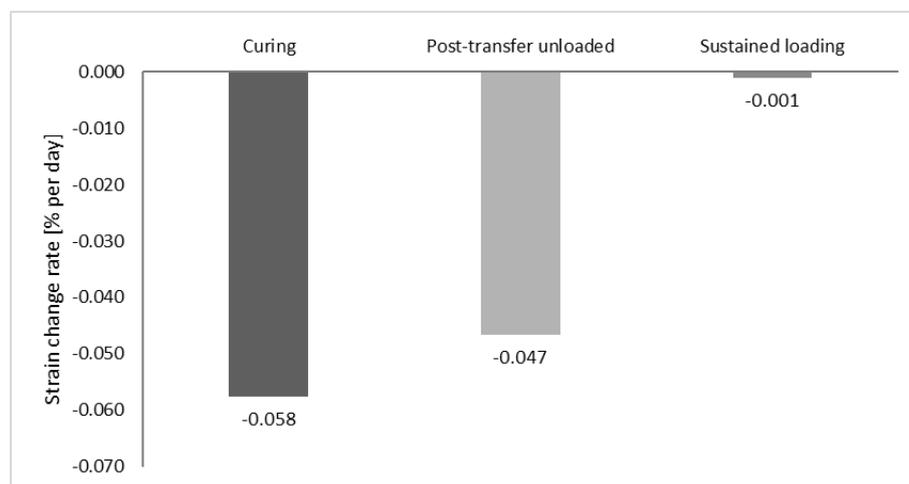
378

379 *Figure 11. Percentage of strain decrease at midspan in 180 days as a function of initial strain*

380 Summary of the strain development

381 During the entire long-term experimental testing programme the strain changes were recorded over
382 the period of 9 months. The recorded changes can be analysed in the following distinct stages: curing
383 of concrete (28 days), prestress transfer (short-term), unloaded stage after the transfer of prestress (59
384 days), application of sustained loading (short-term), loaded stage under sustained load (180 days) and
385 unloading (short-term).

386 The change rate for the long-term periods, expressed as an average change of percentage relative to
387 the initially applied strain per day for all six samples is shown in Figure 12, where a negative value
388 denotes strain reduction. The strain decrease was the most intensive during the curing of concrete,
389 while the rate was lower for the period after the transfer of prestress without external load. During the
390 6-month period under sustained loading, the rate was very low. This could be explained by the known
391 rheological properties of concrete – i.e., shrinkage and creep of concrete being more intensive in the
392 young-age rather than the mature concrete. It should also be noted that the external loading would act
393 beneficially in the sustained loading stage with regards to strain losses, as the external load induced
394 tension in the bars.



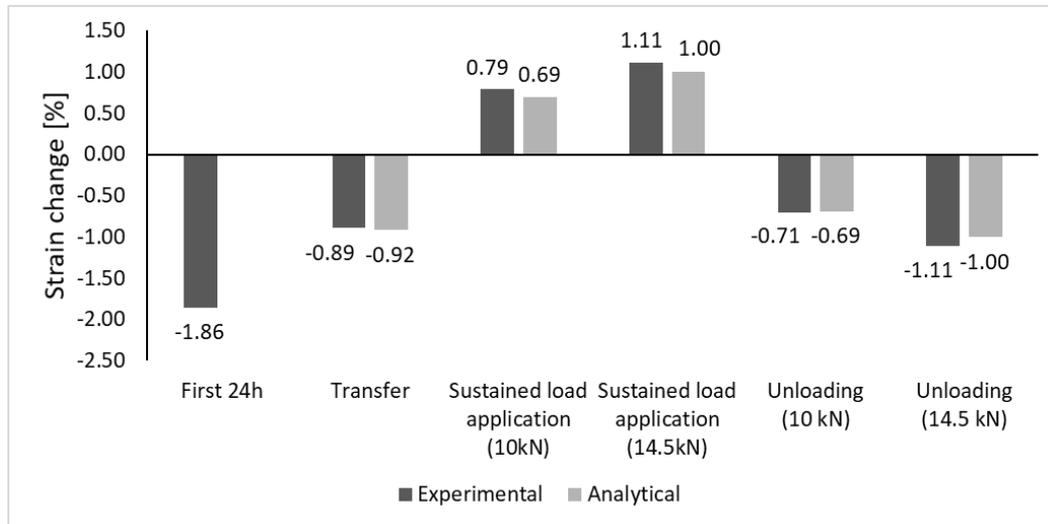
395
396 *Figure 12. Long-term strain change rates for different testing periods*

397

398 As previously noted, four short-term strain changes were also recorded: in the first 24 h from the
399 application of the initial prestress, at prestress transfer, during the load application and during the
400 unloading stage. Figure 13 shows the average total recorded strain change during each of these stages

401 for all six samples, expressed as a percentage of the initially applied prestress, where a negative value
 402 denotes strain reduction, along with analytical predictions for corresponding stages.

403



404

405 *Figure 13. Strain change during short-term testing stages, experimental results and analytical predictions*

406

407 Flexural behaviour under sustained loading

408

409 Instantaneous midspan deflection of a simply supported beam with two equidistant point loads, not
 410 accounting for pre-camber can be determined using:

$$411 \quad \delta = \frac{23 \times P \times l^3}{648 \times E \times I} \quad (3)$$

412 which equals 0.14 mm and 0.20 for 10 kN and 14.5 kN load case respectively. The experimentally
 413 measured instantaneous midspan deflection after the application of the external load was very small,
 414 not exceeding 0.2 mm for all samples. The measured value did not vary significantly with the level of
 415 prestress, which is expected, given that the applied load level falls within the elastic range of the
 416 flexural response. The value is concurrent with both deflections measured during static four-point
 417 bending tests at equal load levels, as well as with simplified numerically estimated values of 0.14 mm
 418 and 0.20 mm.

419 The development of deflections was followed throughout the entire period of sustained loading. No
420 significant change of deformations (midspan deflection) was observed, below ± 0.05 mm in total for
421 all samples, which is close to the precision of the testing equipment.

422 The σ -stress distribution at the midspan of the beam is a resultant of the compression, introduced at
423 the bottom by the prestressed tendons, counteracting the tension induced by the bending moment
424 created by the external load. An increase in downwards deflection (taken as positive) under constant
425 external load occurs as a result of rheological deformation of the material. Similarly, change of
426 deflections in the upwards direction (taken as negative), may occur as the result of rheological
427 deformation due to compressive (prestressing) force. The measured total change of deflections for all
428 samples under the 14.5 kN load, regardless of the prestress level occurred in the positive (downwards)
429 direction, indicating that the external load (applied sustained load) was the dominating force for these
430 samples. However, for all samples under the lower, 10 kN load, regardless of their prestress level, the
431 change of deflections was measured in the negative (upwards) direction, indication that the stress
432 induced by external forces was counteracted in full by the prestress force.

433 In the investigated period, two distinctive phases were observed: from the beginning until
434 approximately 50 days, and from 50 days until the end of testing. Change in the first 50 days of
435 testing occurred at a rate of approximately 8×10^{-4} mm/day for samples under 14.5 kN load, and $-5 \times$
436 10^{-4} mm/day for samples under 10 kN load. This accounted for a much larger change compared to the
437 remainder of the period (50 days to 167 days), during which the deflections changed at a rate of
438 approximately 4×10^{-5} mm and -1×10^{-4} mm, respectively. The much faster development of
439 deflections in the initial one third of the testing period, followed by a much slower rate in the later
440 period, indicates asymptotic tendency for stabilisation of the deformations, evident in the observed
441 relatively short time period (in comparison with the life span of real structures).

442

443 The occurrence of cracks was monitored during regular daily inspections of the samples. No slippage
444 of the bars at the ends was registered. None of the samples had any visible flexural cracks as a result
445 of applied sustained loading for the entire duration of testing of 180 days. This is concurrent with the
446 cracking behaviour under standard quasi-static testing for same applied load.

447

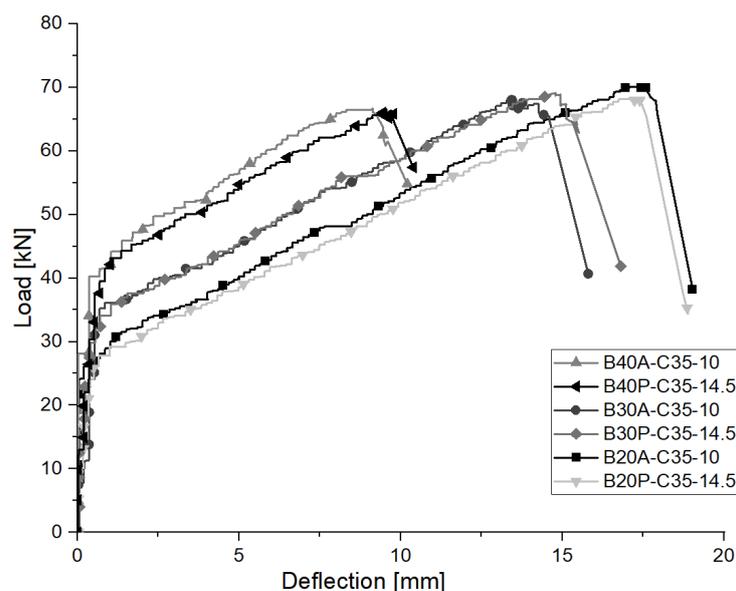
448 Flexural behaviour under final quasi-static loading until destruction.

449

450 After the completion of the sustained loading test, the samples were gradually unloaded and
451 transferred to a reaction frame to be tested under four-point bending until destruction. After
452 unloading, no residual midspan deflections were recorded, indicating full elastic recovery at
453 investigated load levels.

454 Load vs midspan deflection curves for all samples during the final testing are shown in Figure 14. The
455 behaviour of all pairs of samples was similar, indicating that the different levels of sustained loading
456 have no significant influence on the flexural behaviour of the beams in this specific case. The graphs
457 follow an approximately linear pattern up to the point that corresponds to the opening of the first
458 flexural crack. After this point, the angle of the inclination of the load-deflection curve becomes
459 sharper, as the deflections grow more rapidly with the increase in the load, indicating a noticeable
460 change in the stiffness of the element. Similar behaviour continues until reaching the point of
461 maximum load. This is immediately followed by the failure of the beam, which is characterised as
462 brittle, occurring via rupture of the BFRP bars near the midspan for all samples.

463



464

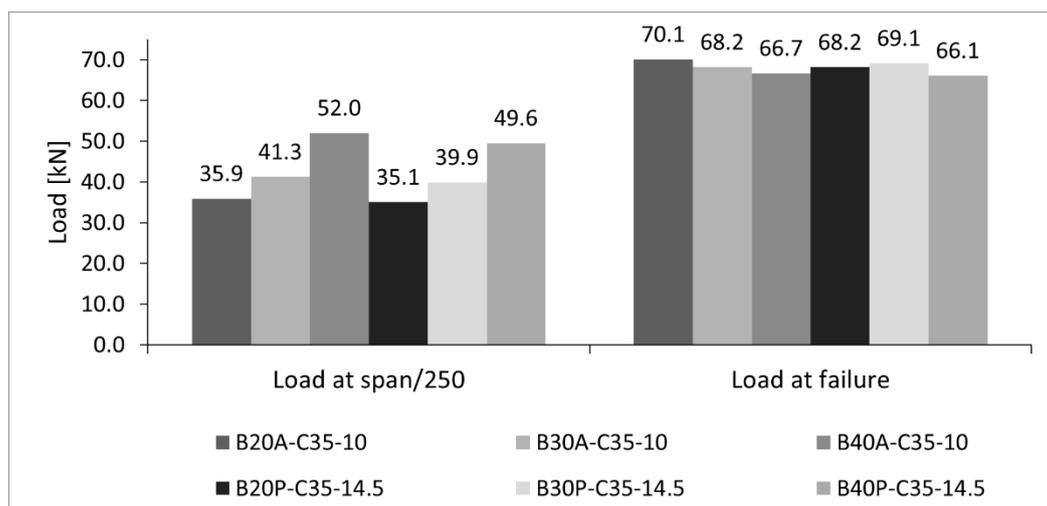
465

Figure 14. Load vs Deflection for all samples during final testing

466 Figure 15 shows a summary of the load results of the final testing. The load at failure is defined as the
 467 maximum load applied to each sample, and ranged between 66.1 kN for B40P-L14.5 to 70.1 kN for
 468 B20A-L10. The average ultimate capacity was 69.2 kN, 68.7 kN and 66.4 kN for pairs of samples
 469 prestressed to 20, 30 and 40% respectively. Based on the results it appears that increasing the
 470 prestress level results in a slightly reduced capacity of the beams. The reduction of the beams'
 471 ultimate capacity for higher levels of prestress is likely related to the higher utilisation of the bars'
 472 ultimate strain capacity introduced by the initial prestress.

473 SLS limit was adopted as span/250 (3.6 mm), as defined by Section 4.4.3.1(5) of Eurocode 2. As
 474 shown in the chart on Figure 15, samples with a higher level of prestressing have increased capacity
 475 according to the SLS deflection criterion.

476



477

478

Figure 15 Summary of SLS and ULS load results of final testing

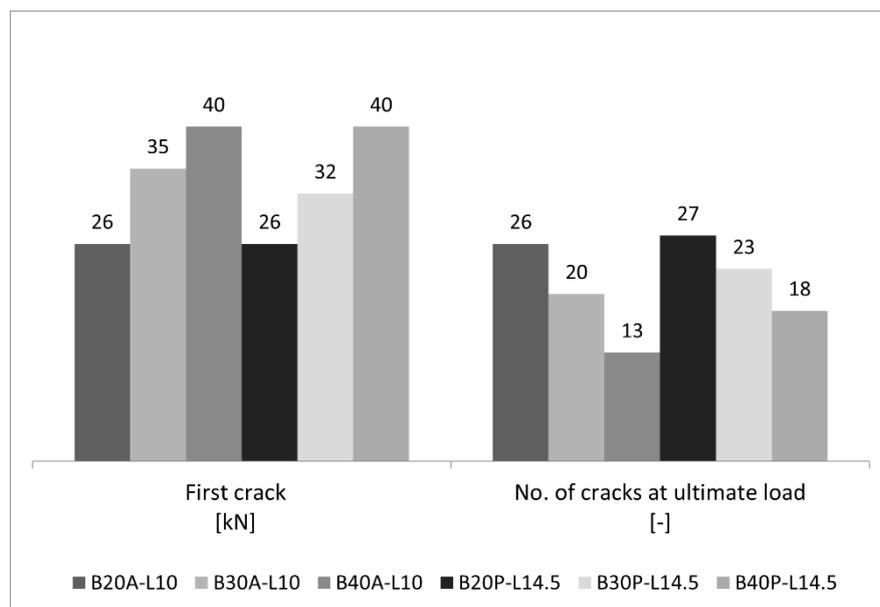
479 There is a clear correlation between the deflection at failure and the prestress level; increasing the
 480 prestress decreases the ultimate deflections of the beam under flexure. The highest deflection at
 481 failure was noted for sample B20A-L10 (17.4 mm), closely followed by B20P-L14.5 (17.2 mm).
 482 Compared to B20A-L10, the deflection at failure was 23% lower for B30A-L10, and 48% lower for
 483 B40A-L10. The failure of all samples was via rupture of bottom bars, as illustrated by Figure 16,
 484 which shows sample B20A-L10 at the end of testing.



485

486 *Figure 16 Sample B20A-L10 at the end of flexural testing until failure, showing rupture of the bottom*
 487 *reinforcement.*

488 The appearance and development of cracks was also monitored during the testing. The first visible
 489 cracks appeared at the bottom of the samples between the applied load points as vertical flexural
 490 cracks. Figure 17 shows the applied load corresponding to the appearance of the first visible crack, as
 491 well as the total number of individual cracks at the bottom of each beam at the end of testing. The
 492 results show that the cracking pattern is related to the prestress level; the higher the prestress load, the
 493 later the first visible crack appears. Total number of cracks is also related to the prestress level; higher
 494 prestress level results in fewer flexural cracks.

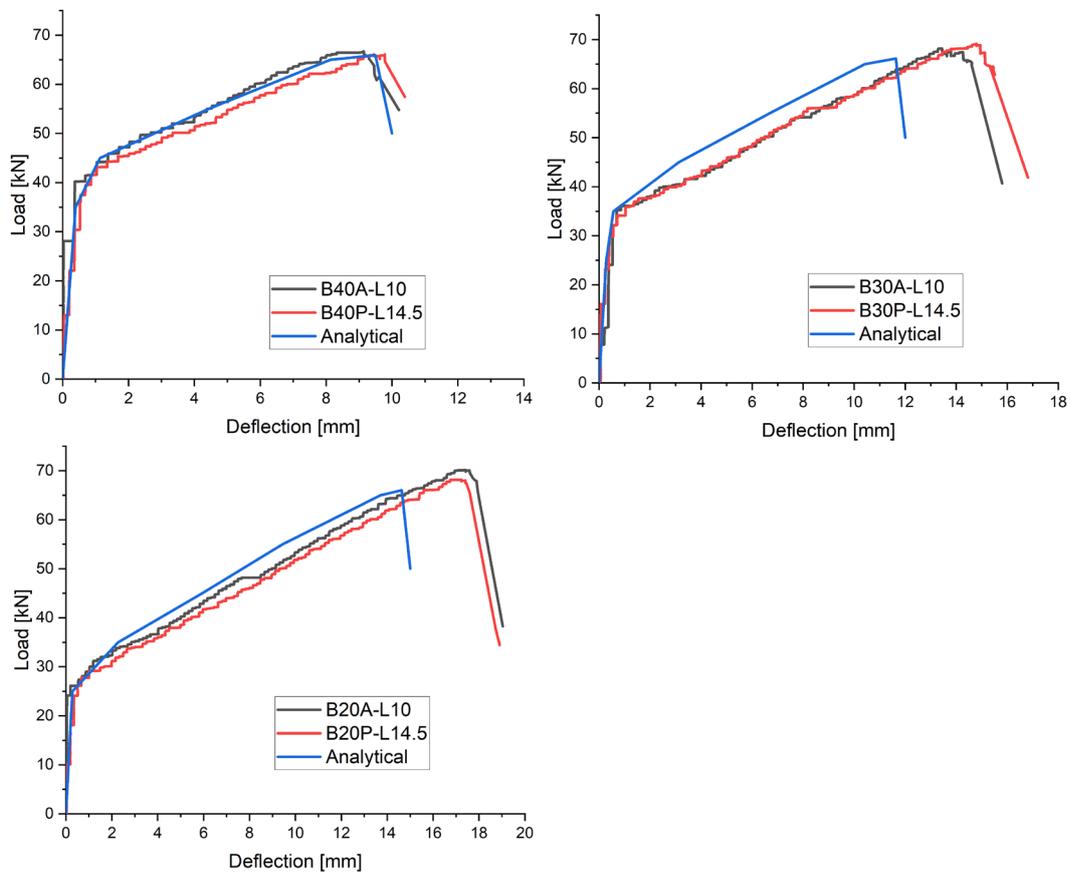


495

496

Figure 17. Crack development during final testing

497 To analytically estimate the flexural behaviour, structural analysis software RESPONSE2000 (Bentz,
 498 2000) was employed to model the sectional response. The material properties of steel, concrete and
 499 BFRP were taken to be the same as explained in the methodology section, consistent with the
 500 materials used in the experiments. The beams' geometry was defined to include all reinforcement:
 501 longitudinal bottom BFRP reinforcement, top high-yield steel reinforcement and shear mild-steel
 502 reinforcement. The prestress level was defined via initial strain, as the theoretical value corresponding
 503 to 20, 30 or 40% of the ultimate strain. The model was developed for the full initial prestress value in
 504 order to assess any deviation of the experimental results from the theoretically predicted behaviour.
 505 The beams were modelled as simply supported, with a moment diagram corresponding to four-point
 506 bending and solved to obtain the moment-curvature response. The obtained diagram was then used to
 507 conduct calculations applying the moment-area approach for estimating deflections.
 508 The resulting load vs midspan deflection curves plotted together with experimentally obtained curves
 509 are shown in Figure 18.



511
 512 *Figure 18 Comparison of experimental and analytical load vs midspan deflection curves.*

513 A summary of main results of destructive testing, with corresponding analytically obtained values is
 514 given in Table 2. Prediction of the cracking load was accurate, whereas the deflection corresponding
 515 to the appearance of the first crack was underestimated for most beams. Furthermore, post-cracking
 516 stiffness for beams prestressed to 30% f_{tu} was overestimated by the analytical model. Nonetheless, the
 517 analytical model resulted in slightly conservative predictions of the ultimate capacity. A potential
 518 explanation for the differences, as well as a possible way for further improvement of the model could
 519 be taking into account the changes of long-term properties of both concrete and reinforcement, which
 520 have not been considered in the model presented here.

521 *Table 2 Summary of main results of destructive testing.¹*

S a m p l e	Crack ing load [kN]	Deflection at first crack [mm]		Ultimate load [kN]		Ultimate deflection [mm]	
		Exp.	Th.	Exp.	Th.	Exp.	Th.
B 2 0 A - L 1 0	2 2 6 5	0.5	0.3	70.1	66.0	17.4	14.6
B 3 0 A - L 1 0	3 3 5 5	0.8	0.6	68.2	66.2	13.4	11.6
B 4 0 A - L 1 0	4 4 0 0	0.7	0.8	66.7	65.9	9.1	9.5

¹ Exp. – Experimental; Th. - Theoretical

B								
2								
0								
P								
-	2	2	0.7	0.3	68.2	66.0	17.2	14.6
L	6	5						
1								
4								
.								
5								
B								
3								
0								
P								
-	3	3	0.9	0.6	69.1	66.2	14.8	11.6
L	2	5						
1								
4								
.								
5								
B								
4								
0								
P								
-	4	4	0.8	0.8	66.1	65.9	9.5	9.5
L	0	0						
1								
4								
.								
5								

522

523 **Conclusions**

524 The paper presented the results of an experimental testing programme which included 6 prestressed
 525 BFRP reinforced concrete beam samples, with varying level of prestress. Continuous monitoring of
 526 strains in the tendons allowed to estimate the losses of strain in comparison with the initial level
 527 throughout the various testing phases.

528 Specifically, based on the initial 3 months of strain monitoring during the unloaded phase of testing
 529 the following can be concluded:

- 530 • The reduction of strain readings during the first 24h (day 0) was the most intensive, on
 531 average of the six samples 1.86% of the initial strain. During concrete curing, from 1 to 28
 532 days, the decrease was on average 1.58% of the initial strain.

- 533 • The instantaneous decrease due to elastic shortening during the transfer of prestress to the
534 concrete was on average 0.86%, 0.89% and 0.91% of the initial strain, for 20%, 30% and 40%
535 prestress level. It corresponded approximately to the values calculated using the EC2 based
536 approach.
- 537 • The losses due to creep and shrinkage of concrete can be estimated using a similar approach
538 to the one used for prestressed steel reinforced elements.
- 539 • The reduction of strain from the moment of transfer of prestress until 87th day was on average
540 2.75%, at a rate of 0.05% per day.
- 541 • The total decrease in strain in this period was on average 7.02%.

542 Based on 6 months period, starting from the application of sustained loading, the following
543 conclusions can be made:

- 544 • During the load application the strains increased by an average of 83×10^{-6} for 14.5 kN load,
545 and an average of 64×10^{-6} for 10 kN load.
- 546 • Change of strain at midspan for the sample with 40% level of prestress up to 70 days from the
547 beginning of the testing resulted in reduction by 0.3%. The change was less pronounced for
548 samples with 20% and 30% levels of prestress. Between 70 days and 180 days, the decrease
549 in strain was very low.
- 550 • The total decrease in strain for the period of 6 months was on average 0.04% of the initial
551 strain for samples with 20%, 0.14% for samples with 30%, and 0.33% with samples with 40%
552 prestress level. The decrease in strain was higher for samples with higher initial strain.
- 553 • The total measured increase in deflections was very small, less than 0.05 mm for all samples.
554 The dominant part of this change occurred within the initial 50 days, with very small
555 additional increase between 50 days and the end of the sustained loading phase.

556 The results of final testing show that:

- 557 • The load at deflection corresponding to the SLS limit of span/250 was higher for samples
558 with higher level of prestress

- 559 • Increase in the prestress level results in delay of the initial crack appearance, as well as in
560 reduction of the total number of cracks at the ultimate load.

561 The conclusions presented here are applicable to the specific setup, materials and geometry of
562 samples used. For further understanding of the influence of parameters other than the prestress level,
563 e.g. bar size effect, it would be beneficial to conduct larger scale experiments of similar type.

564 **Data availability statement**

565

566 All data, models, or code that support the findings of this study are available from the corresponding
567 author upon reasonable request.

568

569 **Acknowledgements**

570

571 The authors would like to express gratitude to MagmaTech Ltd for sponsoring the project by
572 supplying RockBar BFRP reinforcement.

573

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