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Fire testing of concrete beams with fibre reinforced plastic rebar

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Abstract

The behaviour of glass fibre reinforced polymer (GFRP) rebar reinforced concrete beams when exposed to fire is presented in this paper. The experimental programme involved fire tests based on British Standard 476 on two full-scale GFRP rebar reinforced concrete beams with dimensions in cross-section 350 mm×400 mm and 4400 mm total length with a span length of 4250 mm. The beams were designed and constructed according to Eurocode 2 and ACI-440. The purpose of this work was to evaluate the fire resistance of the GFRP reinforced concrete (RC) beams. GFRP rebars with thermoset resin were used for reinforcing beam 1 and GFRP rebars manufactured with thermoplastic resin were used for reinforcing beam 2. Shear reinforcement for beam 1 was GFRP stirrups and for beam 2 steel stirrups were used. Degradation in the flexural capacity due to fire was evaluated and compared. In this study, loaded heating tests were implemented with the aim of collecting basic data for the validation of a model presented in preceding papers by the authors.

Keywords: Glass fibre reinforced polymer rebar; Concrete beams; Failure time; Temperature profile; Fire resistance

1. Introduction

Most building structures must satisfy the requirements of building codes, which relate to the behaviour of those structures in a fire. A measure of fire ratings for buildings refers to the time available in a fire before the structure collapses. The relevant property of the composite rebar is not its flammability or reaction to fire, but rather its ability to continue to sustain loads in an environment of rapidly rising temperatures. The properties of steel at different temperatures are relatively well known, as are the thermal properties of the material, and this allows the modelling of structures with some degree of accuracy to predict a time scale for the ultimate loss of structural integrity. Data is required for glass fibre reinforced plastic (GFRP) rebar in order for similar calculations to be made. GFRP rebars have a wide range of potential applications but their advantages and limitations must be ascertained so they can be used appropriately. UK Building Regulations 2000 [1] has identified the specific requirements for each category of structural element in a building in terms of resistance to collapse (load bearing capacity). The minimum period of fire resistance for the elements of most structures is 90 min.

In this study, GFRP reinforced concrete beams using continuous fibre bars as the main reinforcement were subjected to heating while under load. The three beams in this project were designed based on Eurocode 2 [2] and ACI-440 [3] recommendations and constructed at Queen Mary, University of London. One of these beams was tested as a control in room temperature. This test was carried out for the evaluation of the flexural behaviour of the beam and to choose a sustained load for the fire test. The objective of the study was to determine the fire resistance of GFRP reinforced concrete beams experimentally and to validate predictive models for fire resistance, which have been introduced in previous papers [4,5] by the authors.

2. Test programme

Three reinforced concrete beam specimens were cast, using marine siliceous gravel as coarse aggregate. The dimensions of the beams were 350×400 mm in cross-section, 4400 mm in overall length, and 4250 mm in supported span. The concrete composition for these beams is given in Table 1. In addition, three 100 mm concrete

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Table 1
Concrete composition used for casting the testing beams

Item	kg/m^3
Ordinary Portland cement (OPC)	380
20 mm siliceous aggregate	700
10 mm siliceous aggregate	360
Sand	735
Water	148
Water reducing admixture	(1% of OPC) 3.8

cubes were cast using the same concrete. These gave an average compressive strength of 42 MPa after 28 days of casting.

2.1. Control beam and beam 1

GFRP rebar reinforcements for the control beam and for beam 1 were supplied by Hughes Brothers Inc. Fig. 1(a) illustrates the various GFRP pieces. From left to right; L-shaped #7 (22 mm) used for end rebar, U-shaped #3 (9 mm) used as shear reinforcement by attaching two pieces with plastic coated wires together and a cut piece of #4 (12 mm), used for the main reinforcement. Fig. 1(b) shows the arrangement of the reinforcement in the cross-section of the beam.

2.2. Beam 2

Beam 2 was reinforced by GFRP rebar manufactured by the Dow Chemical Company. This rebar utilised a thermoplastic polyurethane resin matrix. Fig. 2 illustrates the various pieces used for the construction of beam 2. From left to right; L-shaped #4 rebar used for end rebar, steel stirrup, and a cut piece of #4 (12 mm) GFRP rebar, used for the main reinforcement. Tables 2 and 3 give details of the reinforcement specifications used in this work. The reinforcements arrangement shown in Fig. 1(b) is used in beam 2. Steel shear reinforcements were used in beam 2 in order to enhance the shear resistance. The beams were 350 mm width and 400 mm in height with effective depth of 325 mm. The beams reinforced with nine of 12 mm rebars, seven at the tension face and two at the compression face the ratio of GFRP reinforcement 0.89%.

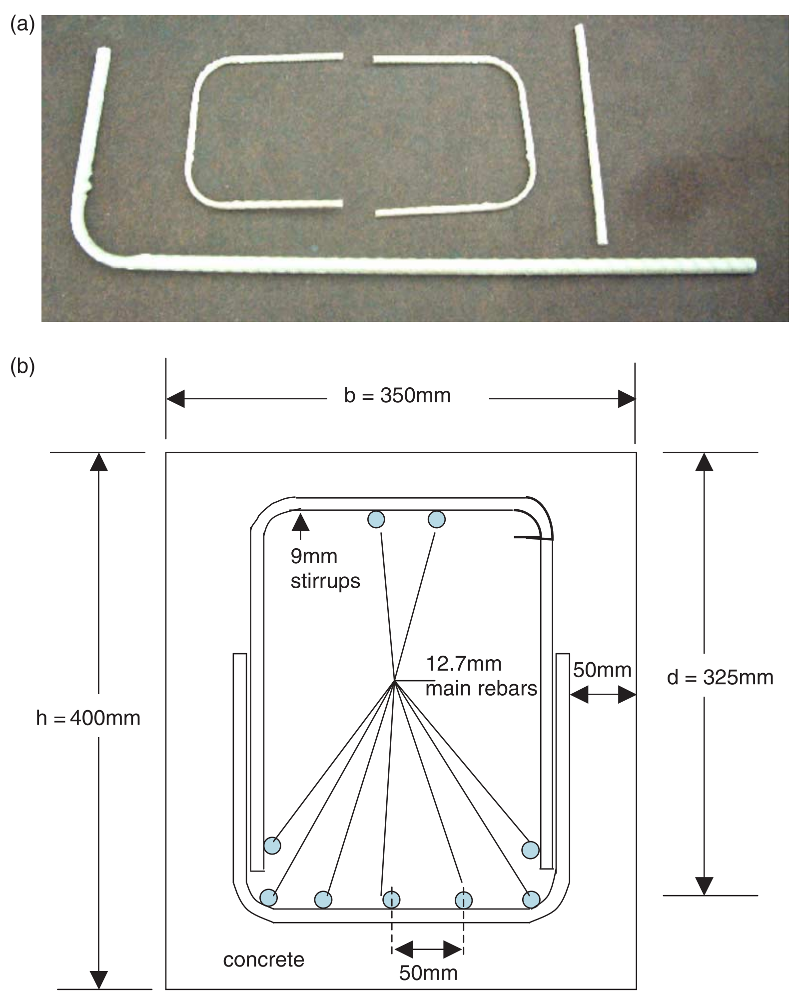

Fig. 1. (a) GFRP rebar shapes used as reinforcement for control beam and beam 1. (b) Beam cross-section with reinforcement arrangement.

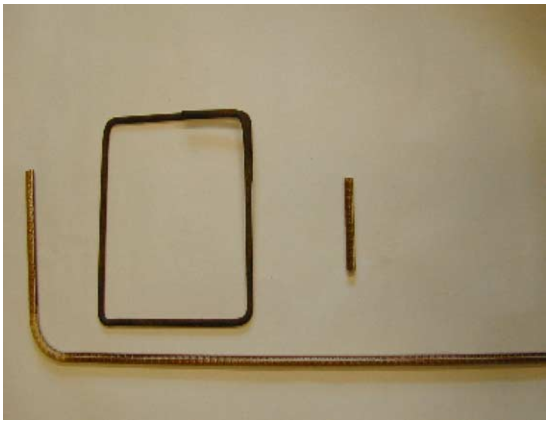

Fig. 2. GFRP rebar shapes and steel stirrup used as reinforcement for beam 2.

3. Testing of the control beam at room temperature

To evaluate the flexural behaviour of the proposed beam, a flexural six point bending test was carried out at room temperature. The beam was subjected to increasing load in increments of 5–10 kN, until final failure. The load was stopped at each increment for 2 min for observation. Load was applied with a hydraulic jack at four points along the beam. Vertical deflection was measured at mid-span using a linear voltage deflection transducer (LVDT). The loading of the beam resulted in the load–deflection curve shown in Fig. 3. The initial linear relationship between load and deflection became non-linear after an applied load of approximately 60 kN. This was accompanied by the onset of cracking in the concrete in tension face. The code of practice specifies a maximum deflection allowable of L/250, which in this case is 17.0 mm, and this was exceeded at a load of 90 kN. A load of 40 kN was selected for the fire test as being below the concrete cracking threshold.

The load was increased to a maximum of 310 kN, which produced a deflection in the beam of 98 mm. After unloading, the beam moved back to the same position as prior to loading. This shows that recovery of the beam is not impeded by plastic deformation of the rebar as would be the case with steel reinforced beams.

4. Fire test programme

4.1. Instrumentation

Specimens for the fire test were monitored with thermocouples. The thermocouples were embedded in the concrete to obtain temperature distribution during the fire tests. The thermocouples used were PTFE insulated K type twisted cable. The ends of the thermocouples were precisely located by placing them in 20×30 mm miniature columns 400 mm long, cast in the formwork prior to pouring the concrete. Thermocouples were also attached to the rebars and stirrups for beams 1 and 2. Fig. 4 shows the thermocouples embedded in concrete at four locations along the beam, A, B, and C. At each location, four thermocouples were embedded: no. 1, 2, 3, and 4, which were 80 mm apart in the section of the beam.

At the locations A, B, and C, thermocouples were also attached to three of the main reinforcements (x,y,z) as shown in Fig. 5. Two thermocouples were also attached to the stirrups at locations A and B. In total, 23 thermocouples were used for each beam: 12 embedded in the concrete (four at position A, four at position B, and four at position C). Nine attached to the rebars at locations is shown in Fig. 5

Table 2
GFRP rebars specifications for the beams tested herein

Specimens	Bar size (mm)	Cross-sectional area (mm^2)	Nominal diameter (mm)	Tensile strength (MPa)[a]	Modulus of elasticity (GPa)[a]
Control and beam 1	9	84.32	9.53	760	40.8
	12	144.85	12.70	690	40.8
	22	382.73	22.23	586	40.8
Beam 2	12	130.69	12.8	≈1000	≈41

The 10 mm steel stirrups used for fabrication in beam 2 has tensile strength of 414 MPa at yield and a Young's modulus of 200 GPa.
[a] Manufacturer data.

Table 3
Beam specimen characteristics

Specimen	Main reinforcement			Tensile main reinforcement	A_t[a] (cm^2)	Tensile reinforcement ratio
	Type of fibre	External profile	Binder			
Control	Glass	Spiral	Vinyl ester	7 of ϕ12	10.14	0.89
Beam 1	Glass	Spiral	Vinyl ester	7 of ϕ12	10.14	0.89
Beam 2	Glass	Molded surface	Polyurethane thermoplastic	7 of ϕ12	10.14	0.89

[a] Total cross-sectional area of tensile main reinforcement.

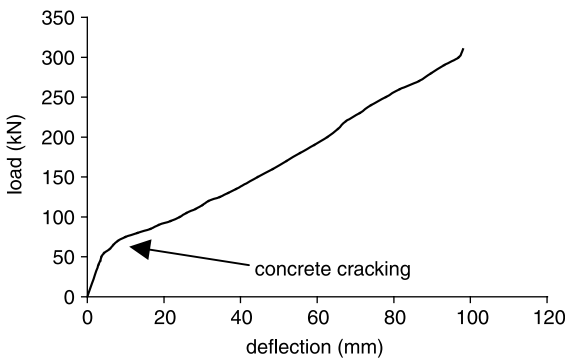

Fig. 3. Load–deflection curve for control beam.

(three at each location to the three rebar). The region where the thermocouples were attached at the mid-points of the applied loads is where the maximum flexural and shear stresses were expected to occur. The experimental set-up for the fire test is shown in Fig. 6.

4.2. Furnace details

Fire testing was conducted at the Building Research Establishment, Watford, UK. Furnace details used in this work are as follows: the internal dimensions of the furnace were 4 m wide × 4 m long × 2 m deep. Two side walls contained the burners. One end wall has a door; the other end wall is modular and can be moved. The flue exit is in the floor at one end of the furnace. The top of the furnace is closed off with a test specimen, or with a set of refractory-lined steel cover slabs. The furnace is lined with 1400 grade insulating brick to comply with British Standard, ISO and EN requirements. Burners were gas-fired nozzle mix burners were used. A total of 20 burners were used, arranged with 10 along two opposite sides of the furnace, approximately 1200 mm above the floor. The loading rig sat above the furnace, running centrally along the length, parallel with the walls containing the burners. The rig provides a span of 4.25 m for the beams. The load was applied hydraulically from above with four points of loading. The maximum test load was approximately 440 kN. The beam specimens were heated on three sides. The furnace temperatures were recorded, monitored and controlled to follow the standard fire curve in accordance with BS 476: Part 20 [6]. The temperatures were measured at 10 points in the furnace near the beam surface.

4.3. Test procedure

About 1 week prior to each fire test, the test beam was taken to the furnace room for instrumentation. At the start of the fire test, 40 kN load was applied by hydraulic jacks at four points (1 m apart) uniformly placed along the beam span. This load was kept constant during each test by load cell monitor located outside the furnace. The deflection was measured at mid-span, using LVDTs. The beam was placed centrally at the roof level of the furnace. The beam was allowed to deflect freely under load. The four point loads were applied to the beam through four pre-made holes in the pre-stressed slab of the roof. The loading rig was the same as that used for testing the control beam. Ten thermocouples

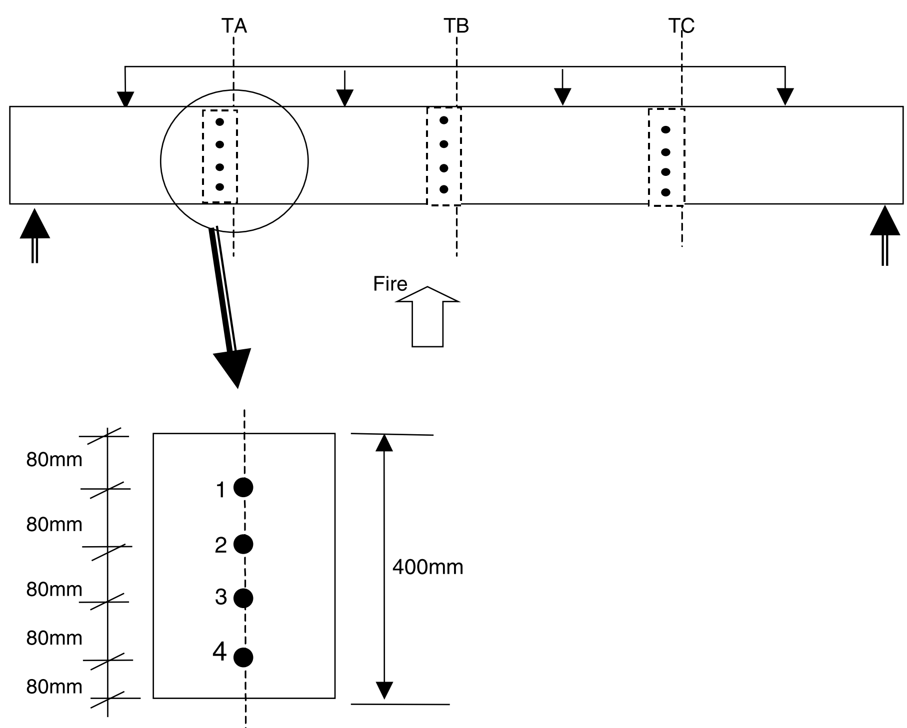

Fig. 4. Thermocouples embedded in concrete. Group of thermocouples at locations A, B, and C in concrete nos. 1–4 at centre line 80 mm c/c.

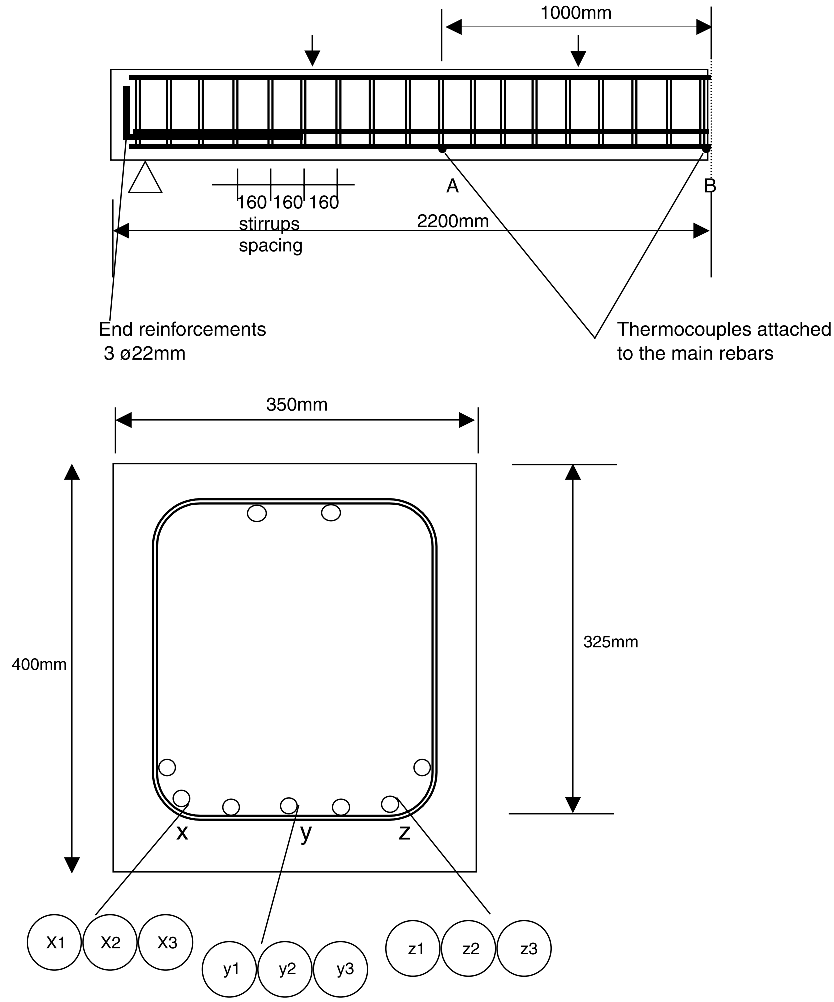

Fig. 5. Specimen configuration and measurement points of temperature on the rebars.

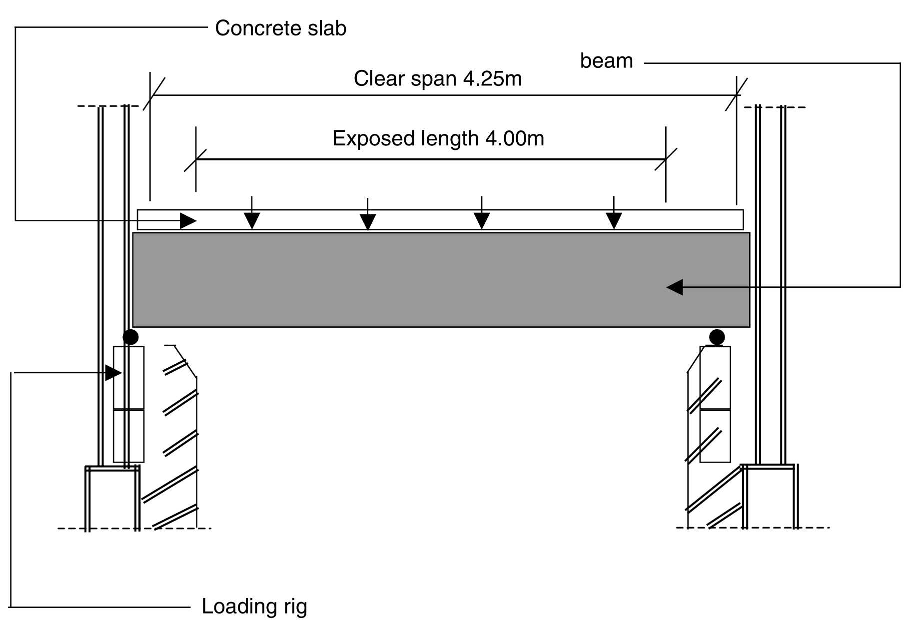

Fig. 6. Fire test set-up.

Table 4
Observation of fire test on beam 1

Time (min)	Observations
0	Test started
12	Small amount of spalling at bottom corners of beam
75	400 mm long crack evident longitudinally along bottom of beam approximately 300 mm in from left hand edge of beam. Flaming from near end on bottom and side of beam
100	Crack has grown to approximately 800 mm long and another has formed 40 mm up the side100 mm long
127	Large chunks have fallen off and a large crack 35 mm wide has formed around the whole perimeter at mid-length. Cracks are showing all over the beam
132	Load removed
140	Large chunk fallen off
143	Test stopped

Table 5
Observation of fire test on beam 2

Time (min)	Observations
0	Test started
14	Small amount of liquid dripping from bottom of beam
25	Small amount of spalling from corners of beam
50	Spalling evident from top of beam
80	300 mm long longitudinal cracks have appeared mid-way up near side of beam at left-hand end. Flickering flame on far side of beam
88	Crack on bottom of beam (tension face) opened approximately 2 mm wide
92	More flaming from other parts of beam and 100 mm long longitudinal cracks appeared mid-length on bottom and near side
94	Rate exceeded for deflection
101	Rate of deflection was exceeded
104	Test stopped

inside ceramic tubes were hung from ceiling of the furnace to measure the temperature near the surface of the beam during the fire test. It was decided that the test should be terminated when the specimen attained a sudden large deflection, or when it was judged by observation as incapable of sustaining the applied load or showed signs of instability, whichever occurred earlier.

5. Fire test results

The sequence of events observed during the fire test on each beam are listed chronologically in Tables 4 and 5, and deflection–time curves are shown in Fig. 7. A slow but steady increase in beam deflection was recorded from outset of the test, but this settled down and the beams were effectively stable after approximately 30 min; at this time the only observed effects were small amount of concrete spalling from the corner of the beams. Cracking in the concrete became evident in both beams after about 75–80 min, which was accompanied by flaming from the crack regions. The beams exhibited an increase in deflection which was identified as failure after 94 min for beam 2 and 128 min for beam 1. The appearance of the beams after failure and cooling to room temperature is shown in Figs. 8 and 9. The mode of failure of beam is via flexural-shear cracks and spalling of the concrete. After the fire testing, the beams were removed from the furnace and it was observed that beam 1 was splitting into two.

Fig. 10 depicts the temperatures measured at the mid-span of the beam in position B, 80 mm from bottom of the beam. The temperature increases with time. Fig. 11 shows the maximum temperature measured at the rebar at each time interval. Fig. 12 shows the average temperature at the stirrups. The data excludes 2–3 readings, which showed sudden, erratic and abrupt changes. This may be attributed to a possible shorting of a PTFE coated wire and the record showing temperature at a location other than the end of the thermocouple.

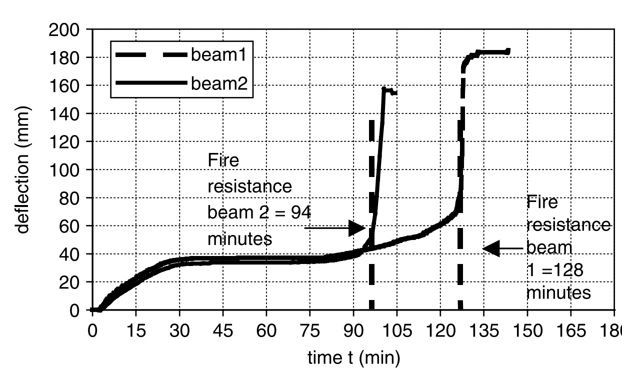

Fig. 7. Heating time–deflection curves for beam 1 and beam 2.

Fig. 8. Rupture at the mid-span of the concrete in beam 1 after fire.

Fig. 9. Failure of beam 2 after fire test inside the furnace shows some flexural cracks and rupture in the main reinforcement.

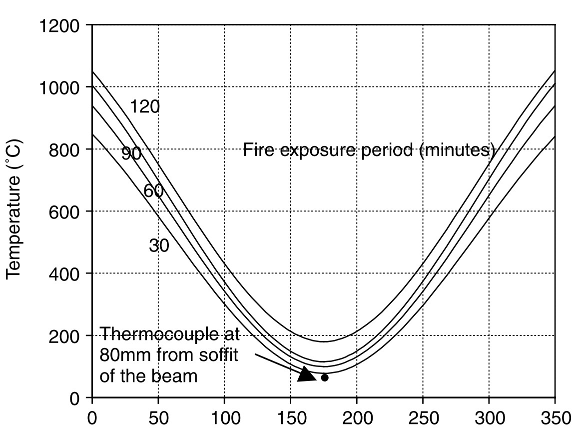

Fig. 10. Temperature profile obtained from the average thermocouples reading in the cross-section of the beam 1, 80 mm from bottom of the beam for different fire exposure periods. The temperature of each side of the beam is an average reading from TC's in furnace on the each side of the beam.

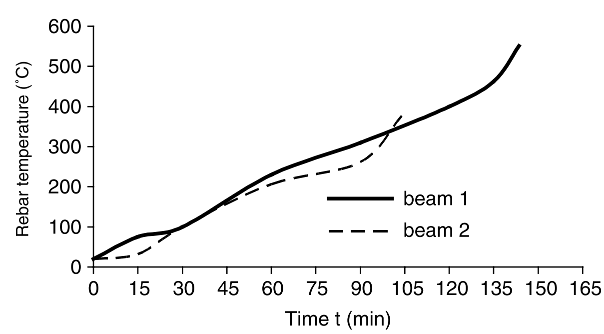

Fig. 11. Heating time/temperature in the rebars for beam 1 and beam 2.

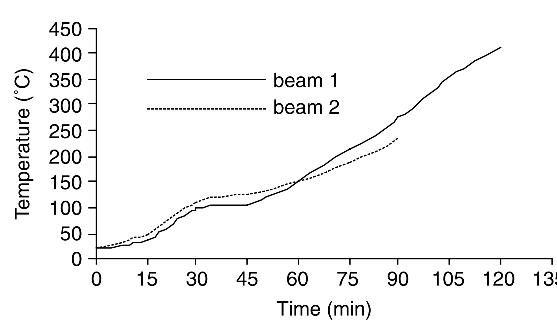

Fig. 12. Average stirrups temperature in beam 1 and beam 2.

Table 6
Results of loaded heating tests

Specimens	Load applied (kN)	Heating time (min)	Failure mode	Failure time (min)	Deflection at centre at end of heating (mm)	Average temperature of bottom reinforcement at end of heating (°C)
Beam 1	40	143	Flexural failure and residual flames emitted from rebar	128	185	462
Beam 2	40	104	Flexural failure, large cracks at tension face of the beam and residual flames emitted from reinforcements	94	157.5	377

6. Discussion

All of the results of the tested beams in comparison are given in Table 6. Average temperature of the bottom reinforcement at the end of heating for beam 1 was 462 °C and for beam 2 was 377 °C. The rise in temperature in a concrete cross-section, in response to high external temperatures, depends on a large number of factors. These factors include the moisture content of the concrete and the chemical composition of the aggregate and cement. Also, the development of temperature in a beam depends on the heating conditions and the heat transfer characteristics of the environment. However, these factors cannot be conveniently evaluated for the purposes of developing a general design rule. The heating time for beam 1 was 143 min with maximum central deflection of 185 mm ≈L/23 and the heating time for beam 2 was 104 min with maximum central deflection of 157.5 mm ≈L/27. The difference may be attributed to the weaker bond strength between the rebar and concrete of beam 2 compared to beam 1. The deflection/time curves for both beam with fire resistance values is depicted in Fig. 7. Sudden deflections in beams 1 and 2 were at 128 and 94 min, respectively. During visual observation subsequent to the completion of heating, the reinforcements were still seen emitting residual flames, this is thought to be due to the high temperature that the fibre bars had reached which aided ignition. The reinforcements which were extracted by chipping away the concrete were found to have undergone thinning and carbonisation due to combustion.

It would appear from the fire tests using full-scale beams and analysis of the samples recovered from the tests, that failure was due to fire penetration through the concrete beam cracks, which developed during testing. This resulted in burning of the matrix of the rebar, which caused interface cracking and de-bonding. This de-bonding resulted in shear cracks along the beam. In beam 1 in which GFRP stirrups was used, the beam split into two pieces after it was taken out of the furnace. Beam 2 was removed from the furnace in one piece, 30 mm expansion was measured and beam was bent at the centre. In both beams spalling of the concrete occurred due to the pressure generated by the conversion of moisture in the surface layer of concrete to steam. Both beams were tested after 38 days of casting. After the fire test, samples of the rebars were collected from the tension face in middle of the beam in order to evaluate the effect of fire on the rebars. The samples were weighed and compared with unexposed samples. The weight of the rebar in beams 1 and 2 had reduced by 22.3 and 33.8%, respectively.

7. Conclusions

The fire resistance rating (load bearing capacity) for beam 1 was 128 min and for beam 2 was 94 min and both beams were tested in accordance with BS 476 Part 21 [7]. 2/3 of cracking load was applied as service load and the deflection was approximately L/23 for beam 1 and L/27 for beam 2. In the criteria for failure under load bearing capacity based on BS 476: Part 20 (Section 10) [6], the deflection is L/20. The building regulations for fire safety recommend that the minimum periods of the fire resistance for the most groups of buildings should be of 90 min. These fire tests results show that concrete beams reinforced with GFRP rebar will meet the fire design requirements for the minimum periods of fire resistance (fire endurance) for the load. A minimum clear concrete cover of 70 mm is recommended for future design of GFRP-RC beams rebars under fire conditions.

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