Available online at www.sciencedirect.com

SCIENCE DIRECT®

Cement & Concrete Composites 27 (2005) 864–874

Cement & Concrete Composites

www.elsevier.com/locate/cemconcomp

Variation of strength and stiffness of fibre reinforced polymer reinforcing bars with temperature

Y.C. Wang [a], V. Kodur [b,*]

[a] *Manchester Centre for Civil and Construction Engineering, UMIST, Manchester M60 1QD, United Kingdom*
[b] *Institute for Research in Construction, National Research Council, Ottawa, Canada K1A 0R6*

Received 3 August 2004; accepted 24 March 2005
Available online 14 June 2005

Abstract

This paper presents the results of tensile mechanical properties of FRP reinforcement bars, used as internal reinforcement in concrete structures, at elevated temperatures. Detailed experimental studies were conducted to determine the strength and stiffness properties of FRP bars at elevated temperatures. Two types of FRP bars namely: carbon fibre reinforced polyester bars of 9.5 mm diameter and glass fibre reinforced polyester bars of 9.5 mm and 12.7 mm diameter were considered. For comparison, conventional steel reinforcement bars of 10 mm and 15 mm diameter were also tested. Data from the experiments was used to illustrate the comparative variation of tensile strength and stiffness of different types of FRP reinforcing bars with traditional steel reinforcing bars. Also, results from the strength tests were used to show that temperatures of about 325 °C and 250 °C appear to be critical (in terms of strength) for GFRP and CFRP reinforcing bars, respectively. A case study is presented to illustrate the application of critical temperatures for evaluating the fire performance of FRP-reinforced concrete slabs.

Keywords: Fibre reinforced plastics; Mechanical properties; Elevated temperatures; Fire resistance; Tests; Critical temperature; Strength; Stiffness

1. Introduction

Fibre-reinforced polymer (FRP) materials have recently received a great deal of interest in the civil engineering research community. An important feature of FRP composites is their extremely high corrosion resistance. This makes them suitable for use in structures subjected to severe environmental exposure. Applications for FRP reinforcing bars as internal reinforcement in concrete structural members include parking garages, multi-storey buildings and industrial structures. In many of these applications, provision of appropriate fire resistance is one of the major design requirements.

However, very little research has been conducted on the behaviour of FRP reinforced concrete structures under fire conditions [1–3]. With advances in fire engineering, methods of calculating the fire resistance of a structure are now well established. These methods can be applied for evaluating fire resistance of FRP reinforced concrete structures provided the mechanical properties of FRP is available [4]. A review of literature indicates there is limited information on the mechanical properties of FRP reinforcement at elevated temperatures [1,2,5,6] and only a few studies of the behaviour FRP-reinforced concrete elements [4,7–10]. Many of these studies have recommended further research to characterise the mechanical properties of FRP at elevated temperatures.

To develop information on strength and stiffness degradation for FRP at elevated temperatures, as well as to obtain critical temperatures of FRP composite materials, a joint research program between the National Research Council, Canada (NRC) and the University

* Corresponding author. Tel.: +1 613 993 9729.
E-mail address: venkatesh.kodur@nrc.ca (V. Kodur).

0958-9465/$ - see front matter © 2005 Elsevier Ltd. All rights reserved.
doi:10.1016/j.cemconcomp.2005.03.012

of Manchester, UK, was initiated. As part of this project, experimental studies of the tensile mechanical properties of steel and FRP reinforcing bars at elevated temperatures were undertaken. Results from these experimental studies are presented in this paper, to illustrate the comparative variation of tensile strength and stiffness of different types of FRP bars with traditional steel reinforcement. Also, a case study is presented to illustrate the application of the mechanical properties for evaluating the fire performance of FRP-reinforced concrete slabs.

2. Research significance

The fire performance of a structural element depends, in part, on the properties of the materials from which the structural element is made [3]. When a structural member is subjected to a defined temperature-time exposure during a fire, this exposure will cause temperature rises in the member. Increased temperatures will degrade the materials mechanical properties and weaken the structural element. The availability of information on high-temperature mechanical properties and temperature distributions permits a mathematical approach to predicting the performance of building components exposed to fire.

As is the case for most engineering materials, the mechanical properties of FRP materials deteriorate when they are exposed to fire. One of the major concerns with FRP reinforcing bars is the potential for severe loss of strength and stiffness at elevated temperatures. Yet there is very little information available in the literature on the variation of strength and stiffness of FRP with temperature [1,3].

Furthermore, in reinforced and prestressed concrete structural members, fire resistance is often defined based on the critical temperature attained in reinforcing or prestressing bars. While the critical temperatures for reinforcing and prestressing steel are well established there is no information on the critical temperatures of FRP rebars [3]. In this paper, data from strength tests is applied to derive critical temperatures for glass and carbon FRP reinforcing bars. These critical temperatures, as well as mechanical properties, can be used as input in computer programs for modelling the fire behaviour of concrete structural members reinforced with FRP bars.

3. Experimental studies

3.1. General

The mechanical property tests were conducted in a specially built test rig at the Manchester Centre for Civil and Construction Engineering of the University of Manchester, UK. The test specimens were prepared at NRC and sent to the University of Manchester for testing.

3.2. Test specimens

Three types of reinforcement; namely carbon FRP, glass FRP, and steel, were considered in this study. The FRP specimens were cut from 9.5 mm diameter carbon fibre reinforced polymer (CFRP) bars, and 9.5 mm or 12.5 mm diameter glass fibre reinforced polymer (GFRP) bars. Two sizes of steel reinforcing bars, 10 mm and 15 mm diameter, were also tested. The reinforcing bars, obtained by NRCC from manufacturers, were prepared to specimens 1.35 m in length and then shipped to the University of Manchester.

3.3. Test apparatus

The elevated temperature tests were conducted in a purpose-built facility. The test rig consisted of two portions: one for the preparation of the test samples, and the other for loading at elevated temperatures. The loading facility is shown in Fig. 1 and consists of a reaction frame, a hydraulic loading jack, and an electrically heated kiln.

Fig. 2 shows a schematic view of the test set up. The test assembly, including the reinforcing bar to be tested and the anchorage system, was passed through the electrically heated kiln and clamped to the reaction frame at both ends through a pair of clamping brackets at each end (shown in Fig. 1). Load was applied using a hydraulic jack as shown in Fig. 2. Based on the dimensions shown, the tensile force in the specimen was calculated to be twice the applied load in the hydraulic jack.

3.4. Test set-up

An important consideration in testing FRP rods is to ensure that failure occurs in the test specimen, and not at the anchorage. Because of the low gripping resistance

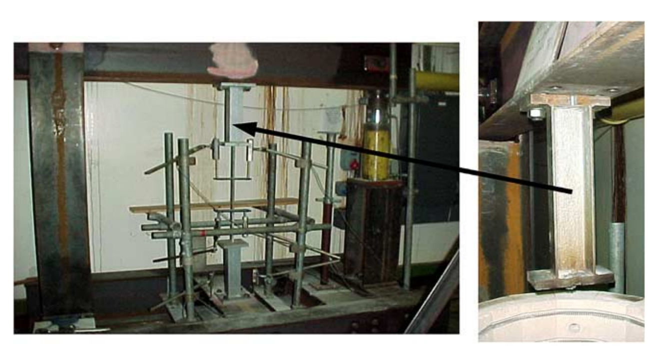

Fig. 1. Elevated temperature test facility.

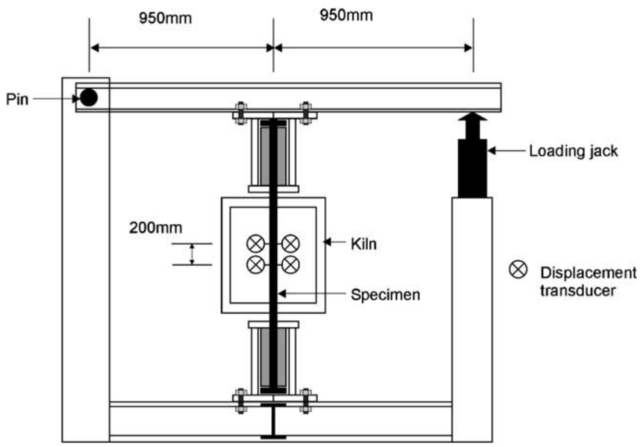

Fig. 2. Schematics of test rig.

of FRP rods on their surface, the design of an effective anchorage system was the most important issue in developing the test method. The ACI standard [11] does not have any specific recommendation on the anchorage system, other than giving information on how to use the test results to calculate the failure stress and the elastic modulus.

Different types of anchorage systems have been tried by a large number of researchers. So far, the most reliable method is to use expansive cement confined by a circular steel tube [12,13]. When curing, chemical reactions take place in the expansive cement and pressure is generated due to confinement by the circular steel tube. If the FRP rod is surrounded by expansive cement, the pressure in the cement will allow the FRP rod/cement interface to develop sufficient friction resistance to transfer failure from the anchorage position at the end of the test specimen to within the specimen length. Important parameters in the design of an adequate anchorage system, when using expansive cement, are the length of the anchorage and the size of the steel tube. For this study, the Bristar 150 expansive cement was used. The nominal dimensions of the steel tubes were 48.3 mm in outer diameter, 3 mm in thickness and 350 mm length.

The Bristar manufacturer's literature for this expansive cement gives a maximum safe operation temperature of 20 °C. However, it was found that at temperatures of 18 °C and 19 °C in the laboratory, this cement produced the blown-off phenomenon and preparation of the test samples had to be aborted until temperature in the laboratory was lower. At a cooler temperature of about 15 °C, the expansive cement performed satisfactorily.

Another important issue is to ensure that the FRP specimens are aligned vertically and centrally in the anchorage tubes. This was achieved through the fabrication of a wooden frame. Fig. 3 shows five test specimens being prepared in the wooden frame. The base of this

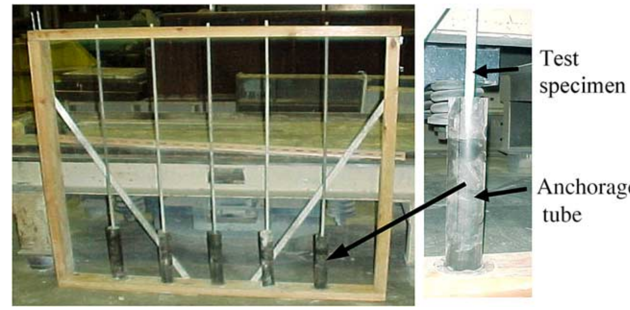

Fig. 3. Wooden frame for the preparation of test samples.

wooden frame was fitted with five carefully machined steel housing caps for the anchorage steel tubes into which the expansive cement was poured. A hole with a diameter just bigger than the diameter of the test sample was drilled in the middle of each steel cap to allow the test bar to pass through. After cleaning, the anchorage steel tube was placed vertically into the steel cap. Silicone gel was then injected around the anchorage steel tube to secure it to the wooden base and to prevent leakage of the expansive cement. Once the silicone gel was hardened, a test specimen was pushed through the central hole in the steel cap.

To ensure the vertical alignment of the test sample, the top of the wooden frame had five holes with a diameter just slightly larger than the specimen diameters so that the specimens fitted snugly to the wooden frame.

Once the expansive cement hardened in the anchorage tubes at the base of the wooden frame, the specimens were turned around for casting at the other ends.

3.5. Test procedure

All tests were conducted under steady-state conditions. After each specimen was fixed in place, as shown in Fig. 2, the kiln temperature was raised to the appropriate value and held constant for approximately 30 min to allow the test specimen to achieve a uniform temperature. The test specimen was then loaded to failure at constant temperature.

Four displacement transducers were attached to each of the test specimens; two were placed at two locations 200 mm apart within the kiln to record the axial deformations (see Fig. 2). At each location, the two transducers, which were outside the kiln, were fixed, using a pair of ceramic rods, to a set of metal brackets mounted on the test specimen inside the kiln. Measurement by the displacement transducers began at the same time as loading, and started after thermal expansion of the test specimen had occurred. Therefore, the displacement transducers measured the mechanical deformations of the test specimen, including creep, but excluding thermal expansion. However, because the loading period was

Table 1
Summary of test parameters for reinforcing bars

Specimen type	Number of tests @ temperature (°C)
9.5 mm GFRP (SG)	3@20, 2@100, 2@250, 3@350, 2@500
12.7 mm GFRP (BG)	2@20, 2@100, 2@200, 2@300, 2@400, 2@500
9.5 mm CFRP (C)	2@20, 2@100, 2@200, 4@400, 2@600
10 mm steel (SS)	2@20, 2@100, 2@200, 2@400, 3@600
15 mm steel (BS)	2@20, 2@100, 2@300, 2@500, 2@700

relatively short, it is unlikely that creep strains were significant.

Typically, a batch of five specimens were cast on a Thursday or Friday and then tested in the early part of the following week. This was necessary to allow the expansive cement to develop and maintain sufficient confinement pressure.

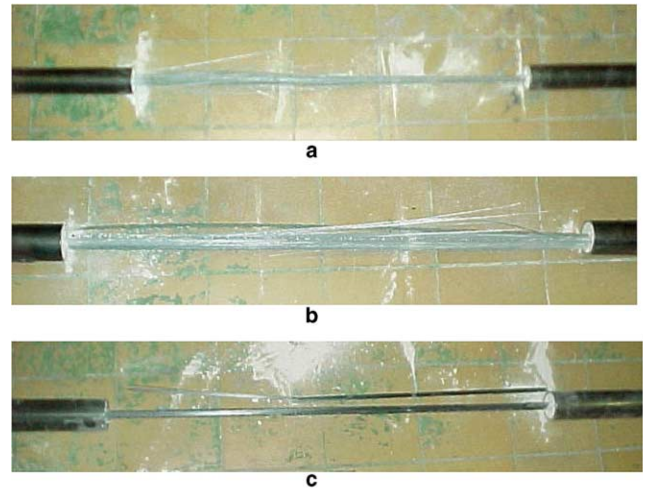

Fig. 4. Failure patterns of different types of FRP reinforcing bars at ambient temperature: (a) 9.5 mm GFRP bar, (b) 12.7 mm GFRP bar, and (c) 9.5 mm CFRP bar.

3.6. Recording of results

As part of the experimental studies, 57 tensile strength tests were carried out on FRP and steel specimens. Each type of specimen (steel, GFRP, or CFRP) was tested at five different temperatures, except for the 12.7 mm diameter GFRP bars, which were tested at six temperatures. Because all materials display variability in their mechanical properties, each test was repeated at least once. To generate as much information as possible, test temperatures for the GFRP and steel reinforcing bars of two different diameters were arranged so that they covered as many temperatures as possible for each type of material. Also, visual observations were taken though out the duration of the test. Table 1 gives a summary of test parameters for the various specimens. During the tests, temperatures inside the kiln and on the specimen were continuously measured using thermocouples.

4. Results and discussions

4.1. Test observations

The following observations were made during the strength tests on various specimens:

1. In all tests, the specimens were at approximately the same temperature as the kiln when loaded to failure.
2. All GFRP and CFRP bars failed by fracture within the specimen length. Failure did not occur in the anchorage. Fig. 4(a)–(c) illustrates the observed failure mode of GFRP and CFRP reinforcing bars at ambient temperature. At elevated temperatures, specimens have lower strength so that failure was also in the specimen length as at ambient temperature. As an example, Fig. 5(a)–(e) shows the failure pattern of 9.5 mm GFRP bars at different temperatures. This observed mode of failure, by fracture within the specimen length, validates the experimental set-up.
3. No fracture of the steel reinforcing bar was observed in any of the tests. This is because the test set-up could not allow for sufficient extension of the steel bars to break them. Nevertheless, all steel bars were loaded well into the effective yield range. The effective yield stress is defined as the steel stress at 0.2% plastic strain (the 0.2% proof stress).
4. The displacement transducer system performed well for the majority of tests. It failed to record data for the smaller diameter GFRP and CFRP bars at temperatures above 350 °C. This is because the resin in these reinforcing bars burned away, and the displacement clamp system disengaged from the test specimens.
5. In one test on GFRP reinforcing bars the resin ignited when the temperatures were about 500 °C (see Fig. 6).

4.2. Variation of stress–strain curves

Figs. 7–9 present typical recorded stress–strain relationships for the 9.5 mm GFRP, 12.7 mm GFRP and 9.5 mm CFRP reinforcing bars. The other GFRP and CFRP bars had similar linear stress–strain relationships until specimen fracture. For the steel bars, the stress–strain relationships were linear during the initial pre-yield phase. Tests performed at different temperatures indicated reproducible results.

In Figs. 7–9, a linear equation is fitted to the predominant linear portion of each curve. The slope to this

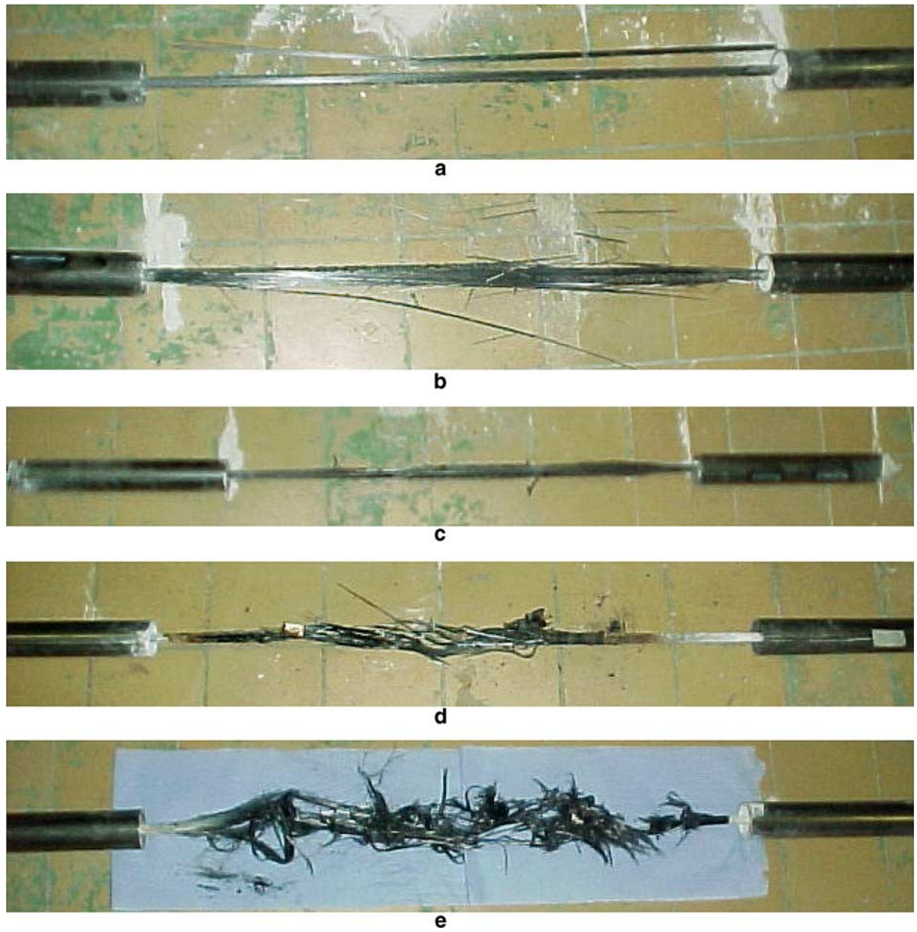

Fig. 5. Failure patterns of 9.5 mm CFRP reinforcing bars at different temperatures: (a) 20 °C, (b) 100 °C, (c) 200 °C, (d) 400 °C, and (e) 600 °C.

Fig. 6. Burning of resin at 500 °C, diameter 12.7 mm GFRP specimen.

linear equation may be taken as the Young's modulus of the FRP bar. The recorded stress–strain relationships show a short period of erratic behaviour during the initial stage of loading. This is clearly a result of the lack of fit in the test system. The origin of the stress–strain relationship may be taken as the intercept of the proposed line and the horizontal axis.

4.3. Variation of strength

Data from the experimental studies (recorded stress–strain relationships) was used to determine the average failure strength for each type of reinforcement at different temperatures. The nominal test temperature was used, this temperature being very close to the actual specimen temperature. For the GFRP and CFRP bars, all test specimens fractured and the recorded failure strength was used. For the steel bars, 0.2% proof stresses were used.

Fig. 10 shows variation of normalized strength with temperature, for all five types of specimens. In the figure, strength retention factor is defined as the ratio of strength at elevated temperature to that at ambient temperature. The measured strength was calculated based on the nominal diameter.

The strength variation trends in Fig. 10 indicate that there are difference between results for the 9.5 mm and 12.7 mm GFRP bars at low temperatures, with the 9.5 mm giving high strengths. The higher degradation of strength in the 12.7 mm diameter bars could partly be attributed to higher resin content in these bars. At low elevated temperatures, the resin was damaged first

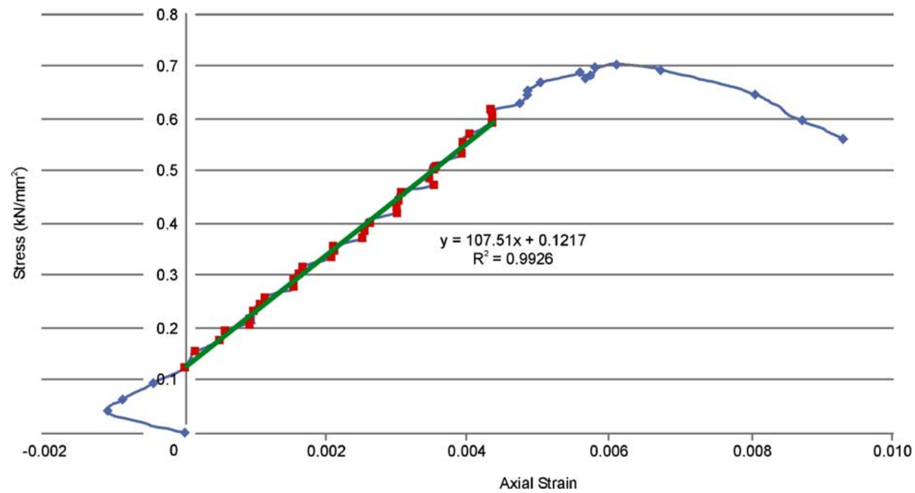

Fig. 7. Stress–strain relationship for CFRP (9.5 mm diameter) bar at 200 °C.

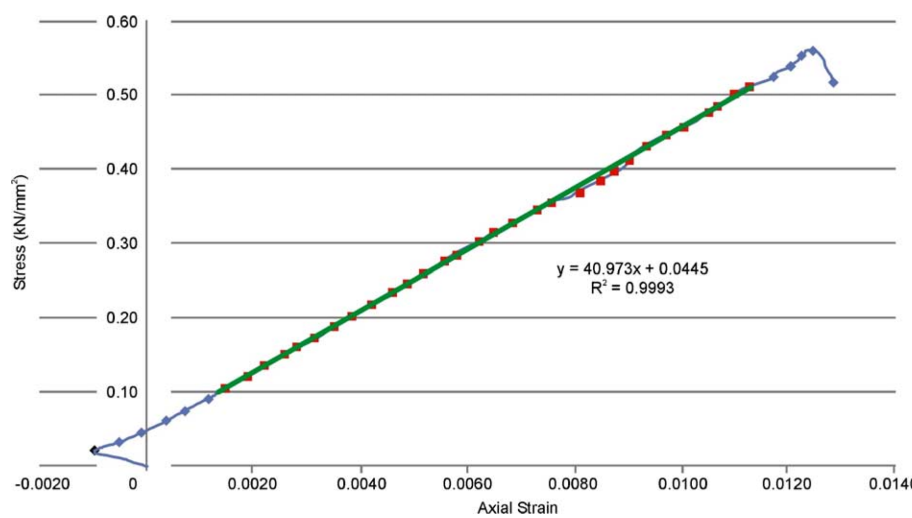

Fig. 8. Stress–strain relationship for GFRP (9.5 mm diameter) bar at 100 °C.

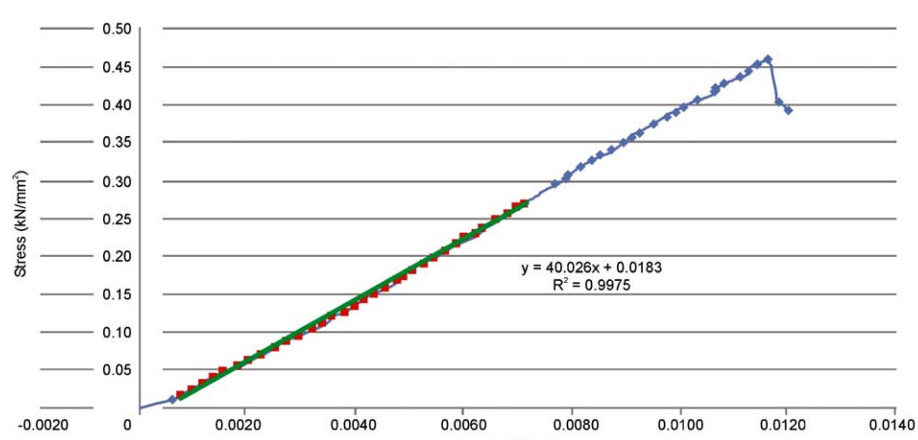

Fig. 9. Stress–strain relationship for GFRP (12.7 mm diameter) bar at 100 °C.

and lost its strength, affecting the 12.7 mm bars more than the 9.5 mm bars.

It can also be seen that the reductions in strength for CFRP bars are greater than those for GFRP bars. This

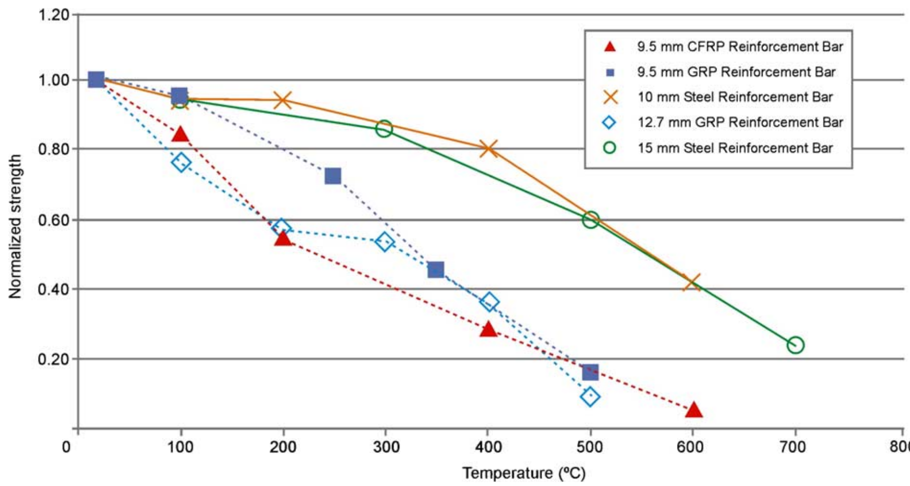

Fig. 10. Variation of strength with temperature for FRP and steel rebars.

is expected because CFRP bars have much greater strengths at ambient temperature. At elevated temperatures, the resin becomes more influential so that the absolute strengths of GFRP and CFRP bars become closer. This means that the relative reductions of CFRP bars are greater. A review of literature indicates that the current strength variation follows a similar trend to that obtained by Blontrock et al. [5] and that the data is in good agreement for both CFRP and GFRP composites. In both cases there is an almost linear reduction in their strengths with temperature, with the strength of CFRP bars reduced to zero at about 500 °C and GFRP bars reduced to zero at about 550 °C. In comparison, steel bars reduce their strengths at a lower rate than FRP composite bars. Also the variation of strength in steel reinforcing bars of two different diameters are very similar.

Fig. 11 compares results from the current study with those obtained by Saafi [10] for CFRP bars. It can be seem that the data from Saafi [10] predicts slightly higher strength and modulus at some lower temperatures, but in general, the proposed relationships are conservative. A similar comparison for GFRP bars also indicated that the proposed relationships of Saafi [10] predict conservative strength.

4.4. Variation of elastic modulus

Test data (recorded stress–strain relationships) was used to determine the elastic modulus for each type of reinforcement at different temperatures. As indicated previously, the nominal test temperature was used, this temperature being very close to the actual specimen temperature. The elastic modulus was taken as the slope of a straight line fitted to the linear portion of the recorded stress–strain relationship for each specimen, as can be seen in Figs. 7–9.

Fig. 12 shows the variation of elastic modulus, with temperature, for the five specimens. The modulus retention factor is defined as the ratio modulus of elasticity

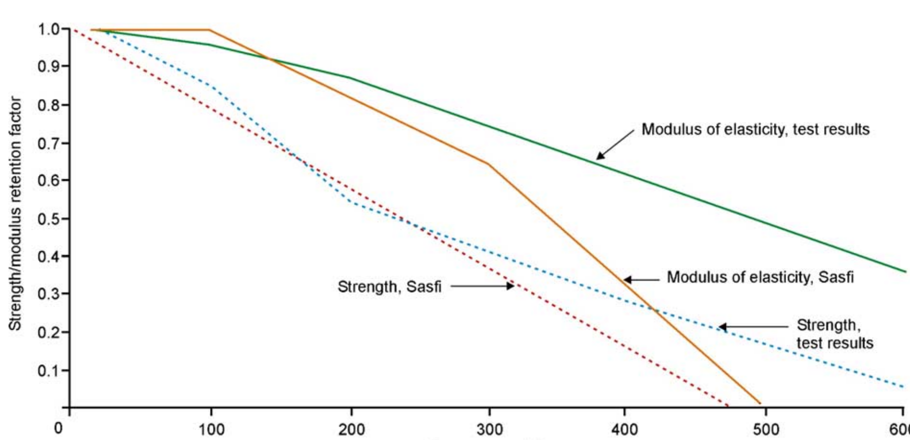

Fig. 11. Comparison of strength and elastic modulus for CFRP bars.

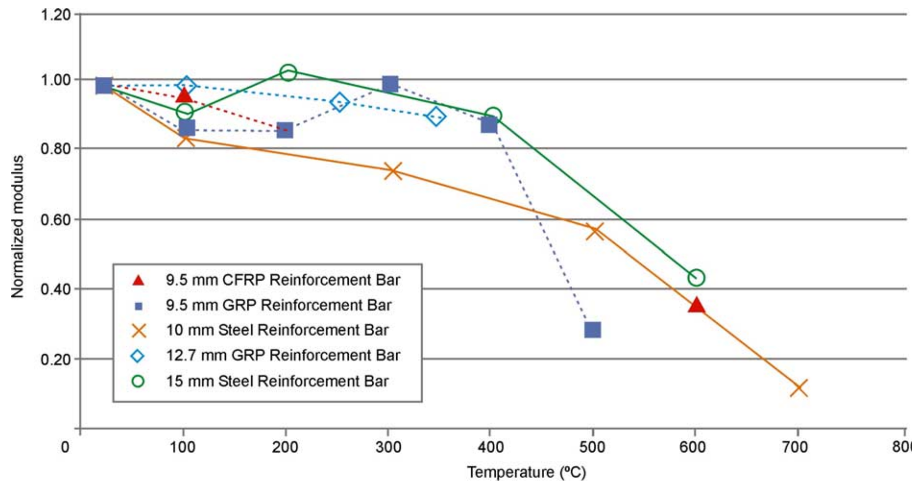

Fig. 12. Variation of elastic modulus with temperature for FRP and steel rebars.

value at elevated temperature to that at ambient temperature.

Fig. 12 shows that, GFRP bars suffer very little loss in their Young's modulus up to approximately 400 °C, retaining about 90% of its ambient temperature value. After this temperature, GFRP bars suffer a drastic reduction in their elastic modulus, with Fig. 12 showing less than 30% of its ambient temperature value at 500 °C. Results are not available for CFRP bars at the crucial temperature of 400 °C due to collapse of the displacement measuring device. At temperatures up to 200 °C, the results of CFRP bars are similar to those of GFRP bars. In contrast, steel reinforcement bars experience a steady decline in their Young's modulus after 200 °C.

Table 2 gives the recorded strength and modulus of elasticity for each test specimen. The nominal test temperature was used, this temperature being very close to the actual specimen temperature. For completeness, Table 2 also includes results for the steel reinforcing bars. From Table 2, it can be seen that different duplicate tests on the same type of specimen at the same temperature produced similar results (<10% difference) at low temperatures (<350 °C). The difference becomes greater at higher temperatures. This is clearly a result of decomposition of the resin. The fibres used to make the specimens are long fibres, but not continuous. Therefore, when the resin decomposes, the mechanical behaviour of the fibre composite becomes more influenced by the variable bond behaviour between the fibres and the decomposing resin. At temperatures above 350 °C, Figs. 10 and 12 show that both carbon and glass fibre composites still have a high level of average tensile strength and elastic modulus. However, results of individual specimens in Table 2 indicate a large variability of strengths at the same temperature of above 350 °C.

Table 2
Variation of tensile strength and of elastic modulus of FRP reinforcing bars with temperature

Specimen type	Diameter (mm)	Test temperature (°C)	Strength (MPa)	Elastic modulus (MPa)
Glass fibre	9.5	20	561.95	41.02
		20	560.49	41.11
		100	559.04	40.97
		100	511.85	40.91
		250	398.59	37.77
		250	410.93	40.21
		350	286.78	N/A
		350	257.01	N/A
		350	215.63	37.24
		500	114.71	N/A
		500	61.71	N/A
Glass fibre	12.7	20	608.56	N/A
		20	572.81	N/A
		20	578.50	42.68
		100	436.72	34.32
		100	457.84	40.03
		200	331.50	34.39
		200	340.44	39.24
		300	326.62	35.47
		300	304.69	50.14
		400	237.66	35.66
		400	182.41	39.53
		500	40.62	11.75
		500	64.59	N/A
Carbon fibre	9.5	20	1260.00	121.41
		20	1280.00	126.50
		100	993.21	105.10
		100	1283.62	132.73
		200	702.79	107.51
		200	763.05	107.27
		400	371.73	N/A
		400	387.70	N/A
		600	132.86	44.68
		600	9.44	N/A

N/A: result not available.

4.5. Critical temperature

In reinforced concrete structural members, fire resistance is often defined based on the critical temperature attained in reinforcing or prestressing bars. Generally, minimum concrete cover thickness requirements are to be ensured so that the temperature in the reinforcement does not reach this limiting critical temperature for the required fire resistance duration. The critical temperature is defined as the temperature at which the reinforcement loses approximately 50% of its strength, and can no longer support the applied load. For reinforcing steel, the critical temperature is 593 °C, while for prestressing steel it is 426 °C [3,14]. In the absence of any structural fire endurance tests on loaded FRP reinforced concrete slabs, the critical temperature concept can also be used for concrete slabs and beams reinforced with FRP bars. However, this requires the determination of appropriate critical temperatures for FRP reinforcing materials.

Data from the strength tests discussed above was applied to derive critical temperatures for the GFRP and CFRP reinforcing bars. Based on the data presented in Fig. 10, it can be seen that the GFRP and CFRP bars lose approximately 50% of their room-temperature strength at about 325 °C and 250 °C, respectively. Hence, based on these test results, the critical temperatures for GFRP and CFRP reinforcing bars are approximately 325 °C and 250 °C, respectively. Also from Fig. 10, it can be seen that the steel reinforcing bars lose 50% of its strength at about 580 °C which agrees well with findings in literature [3,14].

The critical temperature, as well strength and stiffness properties, can be used as input in more elaborate computer programs for modelling the fire behaviour of concrete structural members reinforced with FRP bars. It should be noted that the critical temperature concept indicates 50% loss of strength in a member, often taken as failure from fire resistance point of view, and does not point to actual failure of a member.

5. Design case study

In recent years, there has been significant effort in developing design guidance for the design of concrete structures reinforced with FRP bars. A number of design standards provide detailed guidance for the design and analysis of concrete structures reinforced with FRP bars [11,15,16]. However, there is very limited information on fire resistance design in these standards. The fire resistance of structural members reinforced with FRP rebars can be determined similar to other structural members, either by testing or calculation.

The recent edition of CSA standard "Design and construction of building components with fibre reinforced polymers" [15] provides detailed guidelines for evaluating the fire endurance of structural members reinforced with FRP bars. In this standard there are a series of design charts that provide guidance on the required cover to FRP reinforcement for a particular overall slab thickness, critical temperature of reinforcement, aggregate type, and fire resistance rating.

The design charts in the CSA standard were developed based on numerical studies of concrete slabs reinforced with FRP rebars [9]. The numerical procedure is based on one-dimensional heat transfer analysis to

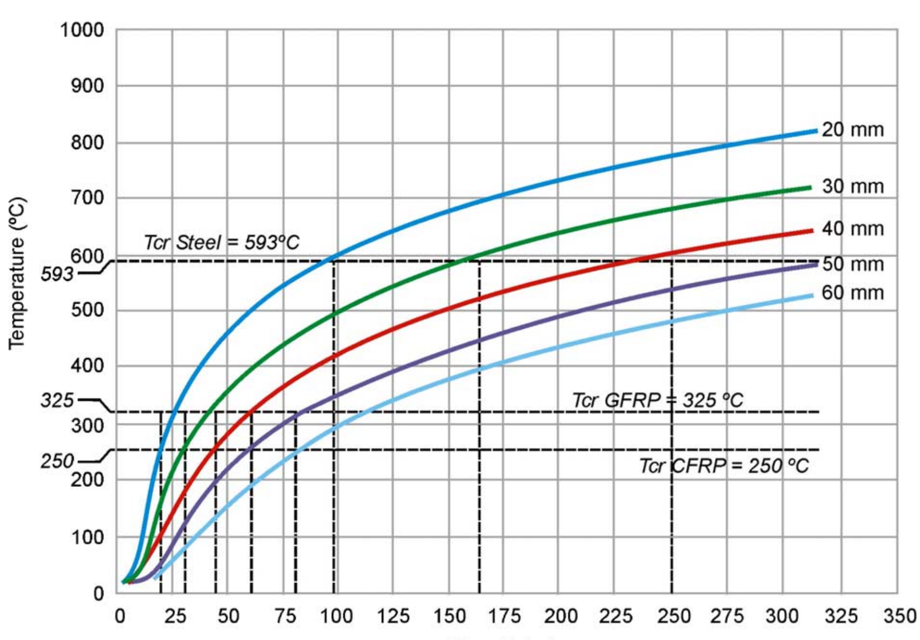

Fig. 13. Design chart for 180 mm thick carbonate aggregate concrete slab.

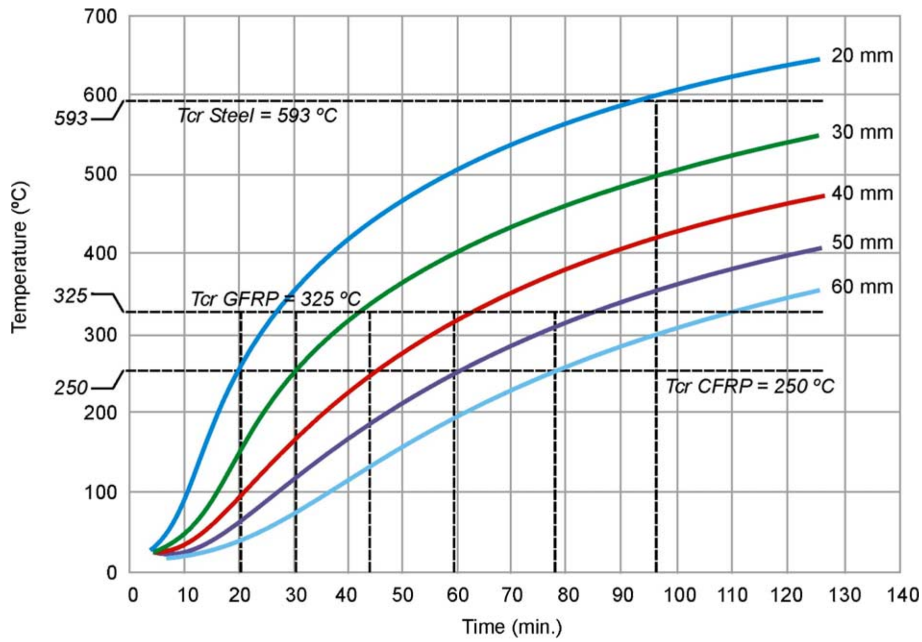

Fig. 14. Design chart for 120 mm thick carbonate aggregate concrete slab.

determine the temperature distribution over the cross-section of the slab. In the model, the failure of the slab is considered to occur when the temperature in the reinforcement reaches the critical temperature or the unexposed surface temperature exceeds the limiting criterion as specified in standards, such as CAN/ULC-S101 [17] and ASTM E119 [18]. It is assumed that the reinforcing bars in concrete structures will not make a significant difference in the temperature profiles. The model has been validated with the limited test data available in the literature [9]. Using the model, the temperature distribution across the thickness of the slab, as a function of time, and the fire resistance of the slab can be computed.

Two examples of these design charts for estimating fire resistance of FRP-reinforced concrete slabs are shown in Figs. 13 and 14. These graphs can be used for evaluating fire endurance of concrete slabs provided the slab properties and critical temperatures of reinforcing bars are known. The critical temperature for GFRP and CFRP reinforcing bars is 325 °C and 250 °C, respectively, while for steel rebars it is 593 °C. As an illustration, a 180 mm thick carbonate aggregate concrete slab reinforced with GFRP rebars and with a concrete cover of about 40 mm will have about 65 min fire endurance. For a similar slab with CFRP rebars the fire endurance is 45 min. If the slab were reinforced with steel rebars the fire endurance would be about 240 min.

The design charts can also be used to determine the required concrete cover to the reinforcement for a desired fire resistance rating and a known reinforcement critical temperature. For instance, for a 120 mm thick FRP-reinforced concrete slab (of carbonate aggregate type) with a desired 1-h fire resistance rating, a CFRP bar or grid with a critical temperature of 250 °C would require a concrete cover of about 50 mm, while for a GFRP bar or grid with a critical temperature of 325 °C, the required concrete cover would be about 40 mm. For conventional steel reinforcement the slab, a concrete cover of about 20 mm will provide a fire resistance of 90 min. Thus, The critical temperature of the reinforcement has a key effect on the fire resistance of reinforced concrete slabs. Also, substantially larger concrete cover thicknesses are required for FRP-reinforced concrete slabs to achieve the same level of fire endurance as steel reinforced ones.

6. Conclusions

Based on the experimental studies presented in this paper, the following conclusions can be drawn:

1. A temperature of approximately 350 °C is critical for the elastic modulus of FRP reinforcing bars. Below this temperature, FRP composite bars retain a high level (90%) of their original stiffness.
2. Strength variation of FRP bars are almost linear up to about 350 °C, giving about 45% and 35% of their original ambient temperature strengths for GFRP and CFRP composite bars.
3. The critical temperatures for loss of strength, based on a 50% strength reduction criterion, are 325 °C and 250 °C for GFRP and CFRP bars, respectively. For reinforcing steel the corresponding critical temperature is 580 °C.

4. The critical temperature of the reinforcement has significant effect on the fire resistance of reinforced concrete members. For typical concrete slabs with glass or carbon FRP rebars, fire resistance of 25–35% of those obtained using conventional steel reinforcement can be expected.
5. The critical temperature information developed in this study will facilitate the use of simple design charts for evaluating fire resistance of concrete members reinforced with FRP bars. Also, the strength and stiffness properties can be used as input in computer models for evaluating the fire behaviour of concrete structural members reinforced with FRP bars.

Acknowledgements

This research is part of a collaborative research project between the National Research Council of Canada (NRC) and the University of Manchester, UK. The authors would like to acknowledge the contributions of Mr P.M.H. Wong of the University of Manchester who conducted the tests. Financial supports were provided by the Public Works and Government Services Canada (PWGSC) and the University of Manchester in the form of a PhD scholarship to Mr P.M.H. Wong.

References

[1] Kodur VKR, Baingo D. Fire resistance of FRP reinforced concrete slabs. RC Internal Report No. 758, National Research Council of Canada, Ottawa, ON; 1998. p. 37.
[2] Bisby LA, Green MF, Kodur VKR. Fire behaviour of FRP-wrapped reinforced concrete columns. Structural faults and repair—2001, London, England; July 4–6, 2001, CD-ROM.
[3] Kodur VR, Harmathy TZ. Properties of building materials. In: DiNenno PJ, editor. SFPE handbook of fire protection engineering. 3rd ed. Quincy, MA: National Fire Protection Association; 2002. p. 1.155–81.
[4] Kodur VKR. Fire resistance requirements for FRP structural members. In: Proceedings of the annual conference of the Canadian society for civil engineering. Regina, Saskatchewan; 1999. p. 83–95.
[5] Blontrock H, Taerwe L, Matthys S. Properties of fiber reinforced plastics at elevated temperatures with regard to fire resistance of reinforced concrete memerbs. In: Dolan CW, Rizkalla SH, Nanni A, editors. Proceedings of 4th international symposium on fiber reinforced polymer reinforcement for reinforced concrete structures. ACI special publication, vol. 188. p. 44–54.
[6] Kumahara S, Masuda Y, Tanano H, Shimizu A. Tensile strength of continuous fiber bar under high temperatures. In: Nanni A, Dolan C, editors. Proceedings of 1st international symposium on fiber reinforced polymer reinforcement for concrete structures. ACI special publication, vol. 138. p. 731–42.
[7] Bisby LA, Williams BK, Green MF, Kodur VKR. Studies on the fire behaviour of FRP reinforced and/or strengthened concrete members. In: Proceedings of the 2nd international conference on the durability of composites for construction. Montreal, PQ; May 29–31, 2002. p. 405–17.
[8] Nakagawa H, Kobayashi M, Suenaga T, Ouchi T, Watanabe S, Satoyama K. Application of three-dimensional fabric reinforced concrete to building panels. In: Nanni A, Dolan C, editors. Proceedings of 1st international symposium on fiber reinforced polymer reinforcement for concrete structures. ACI special publication, vol. 138. p. 211–32.
[9] Kodur VKR, Baingo D. Evaluation of fire resistance of FRP-reinforced concrete slabs. In: Interflam '99, 8th international fire science and engineering conference. Edinburgh, Scotland, UK; 1999. p. 928–37.
[10] Saafi M. Design of FRP reinforced concrete beams under fire conditions. In: Teng JG, editor, Proceedings of FRP composites in civil engineering, vol. 2. Hong Kong; 2001. p. 1235–44.
[11] ACI 440.1R-40. Guide for the design and construction of concrete reinforced with FRP bars. American Concrete Institute, Farmington Hills, MI; 2001.
[12] Harade T, Idemitsu T, Watanabe A, Khin M, Soeda K. New FRP tendon anchorage system using highly expansive material for anchoring. In: Modern prestressing techniques and their applications, Proceedings vol. II, FIP'93 symposium. Japan Prestressed Concrete Engineering Association; 1993. p. 711–8.
[13] Lees JM, Gruffydd-Jones B, Burgoyne CJ. Expansive cement couplers, A means of pre-tensioning fibre-reinforced plastic tendons. Construct Build Mater 1995;9(6):413–23.
[14] Lie TT. Structural fire protection. American Society of Civil Engineers Manuals and Reports on Engineering Practice No. 78. ASCE, New York; 1992.
[15] CSA, CAN/CSA-S806. Design and construction of building components with fibre reinforced polymers. Canadian Standards Association, Ottawa, Ont., Canada; 2002.
[16] Clarke JL. Structural design of polymer composites EUROCOMP design code and handbook. UK: The European Structural Polymeric Composites Group, E&FN Spon; 1996.
[17] CAN/ULC. Standard Methods of fire endurance tests of building construction and materials. CAN/ULC-S101-M89, Underwriters' Laboratories of Canada, Scarborough, Ont., Canada; 1989.
[18] ASTM. Standard methods of fire test of building construction and materials. In: Test method E119-98. American Society for Testing and Materials, West Conshohocken, PA, USA; 1998.