Construction and Building Materials 25 (2011) 838–850

Contents lists available at ScienceDirect

Construction and Building Materials

journal homepage: www.elsevier.com/locate/conbuildmat

Behavior of FRP-confined concrete after high temperature exposure

Yousef A. Al-Salloum [a], Hussein M. Elsanadedy [b,*,1], Aref A. Abadel [a]

[a] Department of Civil Engineering, King Saud University, Riyadh, Saudi Arabia
[b] Specialty Units for Safety & Preservation of Structures at King Saud University, Riyadh, Saudi Arabia

ARTICLE INFO

Article history:
Received 8 July 2009
Received in revised form 20 June 2010
Accepted 29 June 2010
Available online 23 July 2010

Keywords:
Fiber reinforced polymers
Concrete columns
High temperature
Rehabilitation
Strengthening

ABSTRACT

This paper presents the results of an experimental program to investigate the effect of high temperature on the performance of concrete externally confined with FRP sheets. For this purpose, a two-phase experimental program was conducted. In the first phase, 42 standard 100 × 200 mm concrete cylinders were prepared. Out of these specimens, 14 cylinders were left unwrapped; 14 specimens were wrapped with one layer of CFRP sheet; and the remaining 14 specimens were wrapped with one layer of GFRP sheet. Some of the unconfined and FRP-confined specimens were exposed to room temperature; whereas, other cylinders were exposed to heating regime of 100 °C and 200 °C for a period of 1, 2 or 3 h. After high temperature exposure, specimens were tested under uniaxial compression till failure. The test results demonstrated that at a temperature of 100 °C (a little more than the glass transition temperature (T_g) of the epoxy resin), both CFRP- and GFRP-wrapped specimens experienced small loss in strength resulting from melting of epoxy. This loss of strength was more pronounced when the temperature reached 200 °C. In the second phase of the experimental program, three 100 × 100 × 650 mm concrete prisms were prepared and then overlaid by one layer of CFRP and GFRP laminates for conducting pull-off strength tests as per ASTM D4541 – 09. The objective of this testing was to evaluate the degradation in bond strength between FRP and concrete substrate when exposed to elevated temperature environments. One prism was exposed to room temperature whereas the other two specimens were exposed to heating regime of 100 °C and 200 °C for a period of 3 h. It was concluded that a significant degradation in the bond strength occurred at a temperature of 200 °C especially for CFRP-overlaid specimens.

1. Introduction

Research initiatives around the world during the past two decades have documented the behavior of externally bonded fiber reinforced polymers (FRPs) for strengthening reinforced concrete (RC) structures. In these applications, FRPs are bonded to the exterior of RC structures, typically using an epoxy resin saturant/adhesive, to provide additional tensile or confining reinforcement, which supplements that provided by the internal reinforcing steel. Sufficient research and implementation has now been conducted for the development of various design codes and guidelines for the application of FRPs in conjunction with concrete structures [1–4]. Numerous studies have shown that circumferential wraps of FRP on the exterior of reinforced concrete columns can significantly increase the strength and ductility of these members [5–7]. Hence, FRP applications have been widespread in repair and restoration of reinforced concrete columns in existing bridges. The application of FRP wraps in buildings, however, has been hindered due to uncertainties regarding their behavior in fire. Most FRPs are susceptible to combustion of their polymer matrix, potentially resulting in increased flame spread and toxic smoke evolution. In addition, commonly used polymer matrices and adhesives rapidly lose strength and stiffness above their glass transition temperature (T_g). The critical T_g threshold, which depends on the specific polymer matrix constituents, among other factors, typically varies from 65 to 82 °C for externally bonded systems [1]. Thus, if left unprotected in fire, FRPs may ignite, supporting flame spread and toxic smoke evolution [8], and may rapidly lose mechanical and/or bond properties [9]. This may raise concerns as to the fire performance of FRP-strengthened reinforced concrete columns in buildings, where fire is one of the primary design considerations. To date, information in this area is extremely scarce, and a great deal of further work is required to fill all the gaps in knowledge. The purpose of this paper is to fill some of the gaps in understanding the performance of the fire endurance of structures strengthened with FRPs.

A limited number of studies exist on the behavior of FRP-strengthened concrete members under fire conditions. Deuring [10] conducted a fire test program which demonstrated that rectangular RC beams strengthened in flexure with externally bonded

* Corresponding author. Tel.: +966 505428970.
E-mail address: elsanadedy@yahoo.com (H.M. Elsanadedy).
1 On leave from the Department of Civil Engineering, Helwan University, Cairo, Egypt.

0950-0618/$ - see front matter © 2010 Elsevier Ltd. All rights reserved.
doi:10.1016/j.conbuildmat.2010.06.103

Table 1
Summary of test specimens of phase 1.

Exposure time (h)	No. of test specimens								
	Temperature = 100 °C			Temperature = 200 °C			Room temperature		
	Unconfined specimens	Confined specimens		Unconfined specimens	Confined specimens		Unconfined specimens	Confined	
		CFRP	GFRP		CFRP	GFRP		CFRP	GFRP
1	2	2	2	2	2	2	2	2	2
2	2	2	2	2	2	2			
3	2	2	2	2	2	2			
Total No. of specimens = 42									

Table 2
Proportion of ingredients used for concrete mix of phase 1.

Ingredients	Quantity (for 1 m^3 of concrete mix)
Cement	350 kg/m^3, Type !
Silica sand	585 kg/m^3
Washed sand	195 kg/m^3
10 mm aggregate (3/8″)	315 kg/m^3
20 mm aggregate (3/4″)	735 kg/m^3
Free water	175 kg/m^3
Admixture (super plasticizer)	0.67 L

Table 3
Properties of FRP systems.

Property	CFRP system	GFRP system
Thickness of layer (mm)	1.0	1.3
Ultimate tensile strength (MPa)	846	552
Ultimate tensile strain	1.1%	1.9%
Tensile modulus of elasticity (MPa)	77,280	27,600

Table 4
Properties of adhesive.

Adhesive property	Value
Tensile strength (MPa)	71.6
Tensile modulus of elasticity (GPa)	1.8620
Tensile strain at break (mm/mm)	0.0525
Glass transition temperature (°C)	88.00

Fig. 1. Oven with concrete cylinders ready for heating.

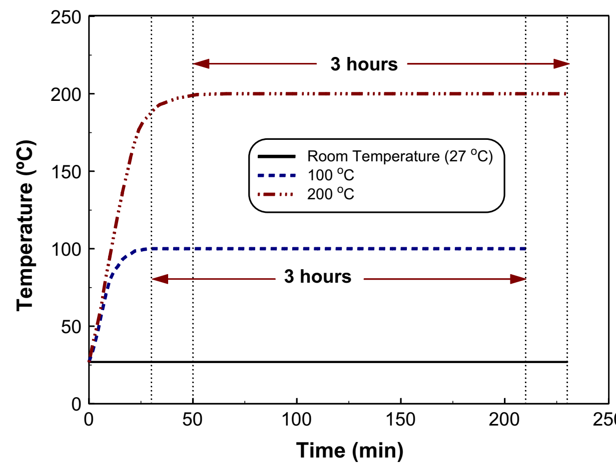


Fig. 2. Time–temperature curves used in this study.

Fig. 3. Instrumentation layout and testing machine.

CFRP strips, but without supplemental fire insulation, experienced loss of interaction between the CFRP and the concrete substrate as early as 20 min into the ISO 834 standard fire test, while FRP-strengthened beams protected with supplemental fire insulation schemes (consisting of mechanically fastened insulating boards) displayed lower temperatures at the concrete/adhesive interface and lost interaction only after about 1 h of fire exposure. Blontrock et al. [11] studied the effect of the supplemental fire protection thickness, configuration, length and method of adhesion on the fire performance of concrete beams strengthened in flexure with externally bonded CFRP strips.

Chowdhury et al. [12] studied experimentally the fire performance of FRP-wrapped reinforced concrete circular columns. Fire tests were conducted on two columns, one of which was tested without supplemental fire protection, and the other column was protected by a supplemental fire protection system applied to the exterior of the FRP-strengthening system. The primary objective of these tests was to compare the fire behavior of the two

FRP-wrapped columns and to investigate the effectiveness of the supplemental insulation system. The column specimens were fire-tested in the full-scale column furnace at the National Research Council of Canada (NRC), Ottawa. This test furnace was designed to expose the column specimens to a standard time–temperature fire curve and to subject the columns to sustained concentric axial load during the fire test, as prescribed by ASTM E119 [13] or CAN/ULC S101 [14]. The results showed that, although FRP systems are sensitive to high temperatures, satisfactory fire endurance ratings can be achieved for reinforced concrete columns that are strengthened with FRP systems by providing adequate supplemental fire protection. In particular, the insulated FRP-strengthened column was able to resist elevated temperatures during the fire tests for at least 90 min longer than the equivalent un-insulated FRP-strengthened column. However, even though the second column was thermally insulated, the temperature at the FRP/concrete interface surpassed the glass transition temperature of the FRP, which was 71 °C for the CFRP system used, at about 34 min into the fire test. This indicates that the used insulation system was probably not able to protect the FRP system, which is widely thought to degrade at temperatures beyond its glass transition temperature. However, even though the FRP-strengthening system was presumed to have been rendered ineffective by the end of the fire tests, the loss of strength of the two columns was significantly different. The un-insulated column failed under the sustained load after 210 min of exposure and its tested strength was lower than the factored design strength of an equivalent unwrapped column. This indicates that FRP-wrapped column, without supplementary fire protection, had experienced significant loss of strength during the fire exposure. Yet, the insulated column failed after 300 min of fire exposure at a load 59% higher than the factored strength of an equivalent unwrapped column. Clearly, the insulated FRP-wrapped column failed at a higher applied load than the un-insulated column because the supplemental fire protection system used was able to maintain low temperatures in the concrete and reinforcing steel during the fire tests, thus enabling the concrete and steel to retain most of their room temperature strength during the fire endurance tests.

When reinforced concrete members are strengthened using FRPs, the ultimate strength of the members increases, thus allowing for higher service load to act on the members. During a fire event, the ultimate strength of the FRP-strengthened reinforced concrete member would decrease with increasing temperature and failure of the member would occur when the ultimate strength drops below the applied service load. Therefore, to ensure fire safety, the strength of the reinforced concrete member retrofitted with FRP must remain greater than the strengthened service loads for the required duration of a fire event [15].

Fig. 4. Failure of the unconfined specimens.

Table 5
Results of unconfined specimens.

Average compressive strength (MPa)			
Temperature	Exposure time (h)		
	1 h	2 h	3 h
Room	38.8	38.8	38.8
100 °C	38.2	38.1	37.0
200 °C	36.1	35.3	34.2

Green et al. [16] has carried out full-scale fire tests on five FRP-wrapped reinforced concrete columns under full service load, in accordance with ASTM E119 [13]. Four 400 mm diameter 3810 mm long circular columns strengthened with carbon FRP wraps have been tested. In addition, a single 400 mm square 3810 mm long column strengthened with glass FRP wraps has been fire tested. Four of the five columns were protected with supplemental fire insulation systems applied to the exterior of the FRP wraps. The two fire protection systems were developed specifically for this application by industry partners, and consisted of spray-applied cementitious mortars with specialized fillers and coatings. Good thermal protection was provided by the supplemental insulation systems, although the recorded FRP temperature exceeded its T_g relatively early in the fire exposure, even for well insulated systems. At temperatures above T_g, the strength, stiffness, and bond properties of the FRP deteriorated. The amount of deterioration has not been quantified due to a lack of appropriate material properties at such high temperatures. The temperatures of the reinforcing steel and concrete inside the insulated columns remained significantly less than 400 °C for the full duration of the fire exposure. This indicated that the columns likely retained their full unconfined strength for the full duration of the fire exposure, since temperatures less than 400 °C were not considered structurally significant for concrete or reinforcing steel. Thus, even if the contribution of the FRP was ignored, the columns would have adequate strength to resist loads expected during a fire event because of the thermal protection of the supplemental insulation. In conclusion, these fire tests have demonstrated that supplemental insulation systems can be used to provide effective fire protection for both circular and square FRP-wrapped reinforced concrete columns. Circular and square FRP-confined columns protected with appropriate thicknesses of these systems were capable of achieving satisfactory fire endurance ratings, in excess of 4 h, even when the T_g of the FRP system was exceeded early in the test. Clearly, this effect is due to the fact that the pre-existing unconfined concrete column, which was subjected to service loads only during fire, was protected by the supplemental insulation system, and experienced only mildly increased internal temperatures which do not significantly decrease its capacity.

Cleary et al. [17] investigated the residual strength of GFRP-confined concrete cylinders exposed to high temperatures. In their study, the GFRP-wrapped concrete cylinders were exposed to a range of elevated temperatures, cooled to ambient temperature, and then loaded in compression to failure. The average compressive strength of the unconfined cylinders was about 40 MPa. The cylinders were wrapped with two continuous layers of GFRP, which were made with a polymer resin that had a high glass transition temperature of 121 °C. The FRP wrap was extended 50 mm beyond the completion of the second layer of FRP. Wrapping the cylinders with GFRP increased the strength by about 255%. The

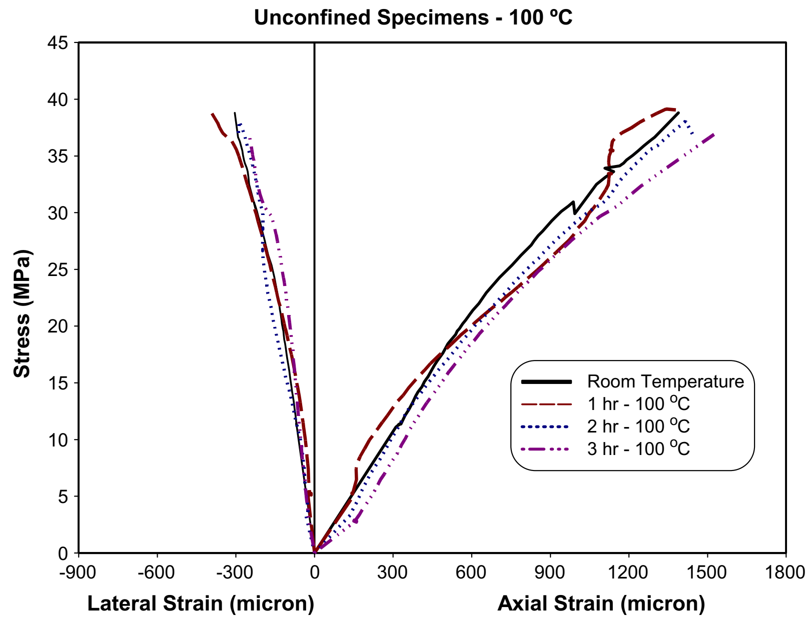


Fig. 5. Stress–strain curves of unconfined specimens exposed to 100 °C.

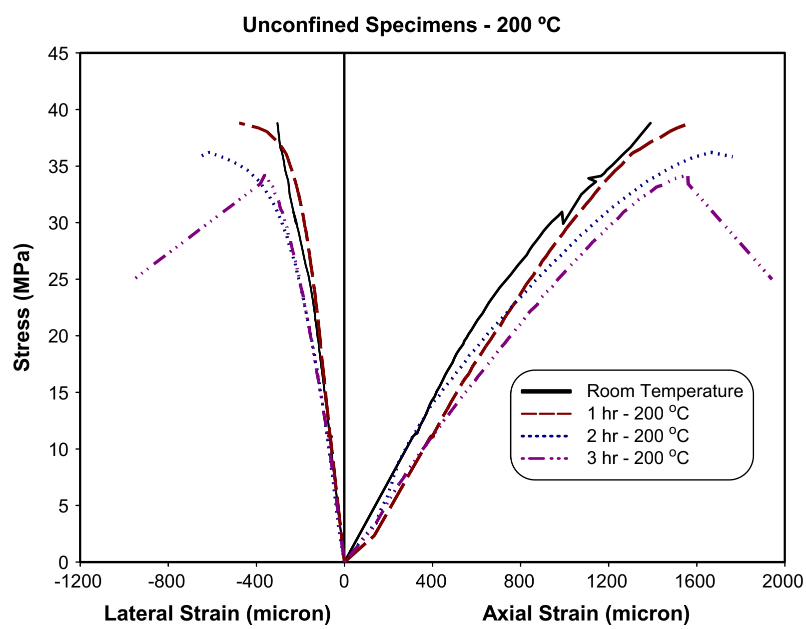


Fig. 6. Stress–strain curves of unconfined specimens exposed to 200 °C.

GFRP-confined cylinders lost about approximately 2%, 4%, 13% and 18% of their initial room temperature ultimate strength when they were exposed to temperatures of about 120 °C, 135 °C, 150 °C and 180 °C, respectively. However, the degradation of axial strength due to heating was reduced after treating the cylinder with an epoxy-based fireproofing coating and paint; the axial strength of the GFRP-confined cylinders decreased by about only 3% and 10% at about 150 °C and 185 °C, respectively. The mode of failure of the GFRP-confined cylinders changed with increasing temperature. At lower temperature, fiber-dominated failure modes were observed, whereas, resin-dominated failure modes were observed at higher temperatures.

Saafi and Romine [18] studied the effect of fire on concrete cylinders confined with GFRP and found that the specimens heated at temperature equal to $2T_g$ for a period of 3 h exhibited significant reduction in axial compressive strength and ductility compared to those subjected to temperature equal to $0.5T_g$. Delamination and separation of the GFRP jackets were also observed. It was also found that the response of GFRP-wrapped concrete cylinders depends mainly on the fire resistance of epoxy resin. At a temperature equal to or higher than the glass transition temperature T_g of the resin, GFRP jackets experienced severe damage resulting from creep and melting of epoxy. The damage was more pronounced after 3 h of fire exposure. Specimens heated at temperature equal to $2T_g$ for a period of 3 h exhibited a significant reduction in strength and ductility and delamination occurred. Even though a reduction in strength of 50% was observed after 3 h of exposure, the residual strength is still higher than the service strength.

Although it was found that the behavior of FRP-strengthened concrete structures at normal temperature is satisfactory, little information regarding the behavior of FRP-strengthened concrete members at high temperatures is available. This study examines the effect of elevated temperature environments on performance of concrete cylinders confined/wrapped with FRP-strengthening system. For this purpose, unconfined as well as FRP-confined concrete cylinders were prepared and then exposed to room temperature and heating regimes of 100 °C and 200 °C for a period of 1, 2 or 3 h. After being exposed to high temperatures, cylinders were tested under uniaxial compression till failure.

2. Previous research on bond between FRP and concrete under elevated temperature

The mechanical behavior of FRP composites at elevated temperatures depends to large extent on the behavior of the polymer resin matrix/adhesive. Polymer resins soften at temperatures in the region of their glass transition temperature, T_g, thus limiting the transfer of stress between the fibers [19] or to any substrate to which they are bonded. Blontrock et al. [11] stated that the strength and stiffness of FRP composites start degrading rapidly at temperatures close to the glass transition temperature of their constituent polymer resin. Furthermore, both epoxy- and polyester-based composites will quickly ignite when they are exposed to fire, typically at temperatures in the range of 300–400 °C [9]. Thus, the mechanical properties of FRP composites will considerably and irreversibly deteriorate due to combustion of the polymer resin at these temperatures [20]. As reported by Bisby [9], studies have shown that carbon fibers experience little to no change to their tensile strength up to temperatures of more than 1000 °C [21], thus demonstrating more resistance to high temperature than glass fibers, which (similar to mild steel reinforcement) lose 50% of their original tensile strength above 550 °C [22].

The bond between FRP laminates and concrete is also an important issue because bond is often the limiting parameter when strengthening concrete structures with FRP [23]. Studies by Chen and Teng [24] have shown that, under room temperature conditions and without specific preventative measures, premature failure due to debonding of FRP laminates is the most common type of failure in FRP-strengthened concrete beams and slabs. External application of FRP laminates requires them to develop and transfer high shear forces through the interface between the adhesive or polymer resin and the concrete substrate. The bond properties between concrete and FRPs deteriorate rapidly with increasing temperature, and this could eventually lead to delamination or debonding of the FRP and the ensuing loss of interaction between the FRP and the concrete [25].

Leone et al. [26] have conducted an experimental program to determine what effect elevated service temperatures have on the bond performance between FRP reinforcement and concrete members. To that extent specimens have been tested under double-face pure shear test and at different test temperatures: 20, 50, 65 and 80 °C. A comparison between un-conditioned (20 °C) and conditioned specimens was performed to evaluate the degradation caused by the thermal exposure. Three types of FRP reinforcement were used in that study: CFRP sheets and laminates and GFRP sheets. It was found that the bond stress–slip curves with increasing service temperatures show a similar qualitative shape even if a significant variation of some relevant parameters was observed. The initial slope of the ascending branch of the curves drops when the temperature increases. The maximum bond stress decreases with service temperatures above the glass transition temperature of the adhesive. In particular, at 80 °C, shear stress decreases by 54% in the case of CFRP sheets, 72% for GFRP sheets and 25% for CFRP laminates with respect to the room temperature. It was also observed that the type of failure changes with increasing test temperature. Specimens tested at $T = 50$ °C show cohesion failure within the concrete. With increasing the temperature ($T = 80$ °C), an adhesion failure at the interface was observed. At temperature similar to or higher than T_g, the adhesion strength of the adhesive drops below that of the concrete, causing the bond failure at the FRP reinforcement–adhesive interface.

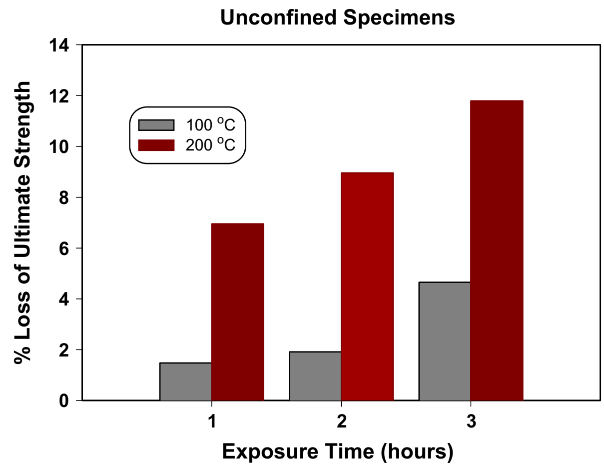


Fig. 7. Percentage loss of average compressive strength for unconfined specimens due to high temperature exposure.

Fig. 8. Failure of the CFRP-confined specimens.

Table 6
Results of CFRP-confined specimens.

Average compressive strength (MPa)			
Temperature	Exposure time (h)		
	1 h	2 h	3 h
Room	95.4	95.4	95.4
100 °C	94.7	94.5	90.5
200 °C	77.5	74.5	69.4

3. Experimental program for phase 1 (testing of cylinders)

3.1. Test specimens

To study the effect of high temperature on FRP-strengthened concrete, 42 small-scale concrete cylinders of size 100 × 200 mm were cast. Concrete specimens were then cured for 28 days and after the curing, they were de-molded. Out of the 42 specimens, 14 cylinders were left unwrapped; 14 specimens were wrapped with

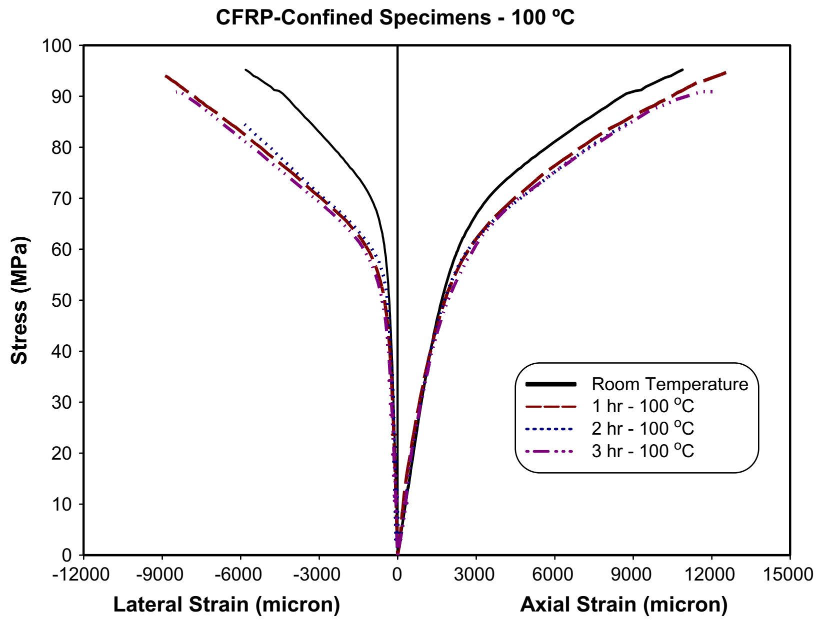


Fig. 9. Stress–strain curves of CFRP-confined specimens exposed to 100 °C.

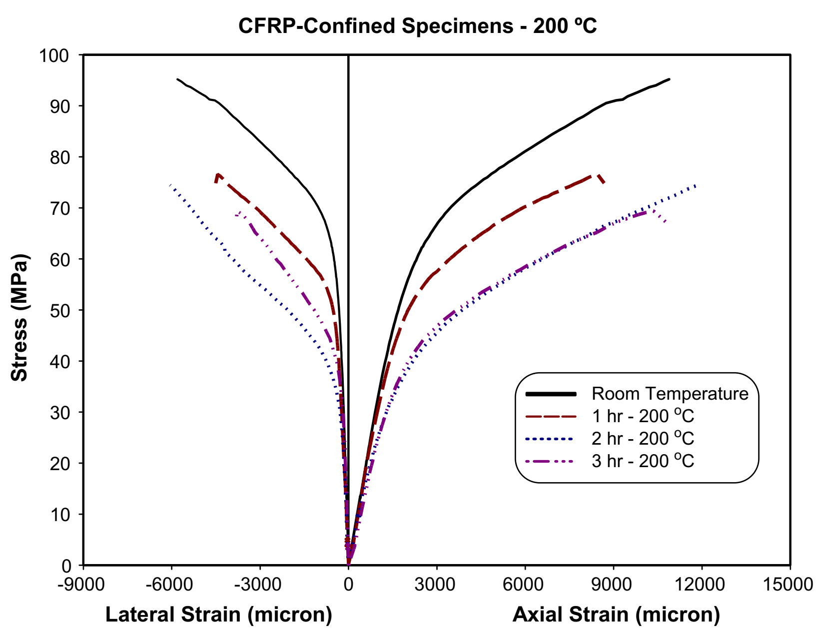


Fig. 10. Stress–strain curves of CFRP-confined specimens exposed to 200 °C.

one layer of CFRP sheet; and the remaining 14 specimens were confined with one layer of GFRP sheet. Before confining the concrete with FRP sheets, voids and deformities on the surface of the concrete specimens were filled using the gypsum paste. The two-component epoxy system, consisting of resin and hardener, was thoroughly hand–mixed for at least 5 min before use. The one layer of CFRP sheet was then applied directly onto the surface of the specimens providing unidirectional lateral confinement in the hoop direction. The same procedure was used for GFRP-confined specimens. All specimens were stored at room temperature for at least 7 days to ensure enough time for curing of epoxy. Before testing, all specimens were exposed to a specified temperature for the duration of 1–3 h. A summary of test specimens with considered temperatures and exposure time is shown in Table 1. Prior to placing the specimens in the oven and then loading onto testing machine, the ends of the jacket were ground and smoothed to remove any uneven edges.

3.2. Material properties

The concrete used in this phase of study had specified 28-day strength of 38 MPa. The quantities of ingredients used in the concrete mix were as shown in Table 2. The properties of CFRP and GFRP systems were determined using five test coupons for each system. The coupons were tested in tension to failure in accordance with ASTM D3039 [27]. The properties of CFRP and GFRP systems are shown in Table 3. The saturating resin used to impregnate the reinforcing fibers and bonding the FRP sheets to the surface of cylinders was of two-component cold-curing type. The ASTM D-638 Type 1 test [28] was used to determine the ultimate tensile strength and the elastic modulus of the adhesive and the ASTM D3418 –08 [29] was used to evaluate the glass transition temperature (T_g) of the adhesives used in this investigation. The results are shown in Table 4 along with other properties of the adhesive.

3.3. Test procedure

As mentioned earlier, some cylinders were exposed to different heating regimes for different durations. For this purpose, a small-scale electrical oven, with internal dimensions 75 × 60 × 50 cm and as shown in Fig. 1, was used. The time–temperature curves for this oven and used in this study are presented in Fig. 2. After each heating exposure, the specimens were cooled at room temperature for a period more than 24 h.

Prior to testing, all specimens were instrumented with two horizontal strain gages mounted at mid-height, 180° apart, on the concrete and jacket surface to measure the lateral strains. To measure axial strain, each specimen was attached to a compressometer as shown in Fig. 3, fitted with two LVDTs that were mounted on two round sleeves (180° apart) around the specimen. The sleeves were attached to the specimen with pin-type support in order not to affect the dilation of the specimen. The wires of the strain gages, the load cell, and the LVDTs were attached to a data acquisition system to record the readings during the experiment. Each specimen was subjected to uniaxial compression till failure.

4. Discussion of test results for phase 1

This section discusses the experimental test results and observations noted from the testing of unconfined and FRP-confined specimens. The experimental results are presented in terms of compressive strength and characteristic stress–strain diagrams. Summary of the test results are presented in the form of tables and figures.

4.1. Unconfined specimens

Unconfined specimens represent 100 × 200 mm concrete cylinders without FRP jacket. Before testing, these specimens were exposed to room temperature and heating regimes of 100 °C and 200 °C for a period of 1, 2 or 3 h. Thereafter, unconfined specimens were tested and the axial stresses in addition to axial and lateral strains were recorded. The performance of cylinders under axial load was found to be consistent and failure was characterized by shearing and splitting of concrete, as shown in Fig. 4. Cracks and concrete spalling were also observed on the surfaces of the specimens. Table 5 reports the compressive strength for unconfined specimens tested in this study. It should be noted that the values of compressive strength presented in Table 5 are average of two specimens. Figs. 5 and 6 illustrate the experimental stress–strain diagram for the unconfined specimens exposed to temperatures of 100 °C and 200 °C, respectively. These diagrams show that stress–strain behavior is consistent but magnitudes of strains are increased due to increase of temperature from 100 °C to 200 °C. Presented in Fig. 7 is percentage loss of average compressive strength for unconfined specimens due to high temperature exposure compared with specimens at room temperature. It is clearly shown that at a specific elevated temperature (i.e. 100 or 200 °C), loss in concrete strength increases with exposure time. This can be attributed to more deterioration in concrete strength with longer exposure to elevated temperature. In addition, it is also indicated that rate of loss in strength with time is more pronounced for 200 °C than 100 °C. However, for both elevated temperatures, if exposure time is up to only 1 h, loss of strength is not significant. This leads to the conclusion that if temperature elevates due to fire, concrete will start losing its strength with time very fast. However, if exposure to elevated temperature is controlled within 1–2 h (by extinguishing fire, for example), the loss in strength is not substantial.

4.2. CFRP-wrapped specimens

This section represents concrete cylinders wrapped with one layer of CFRP sheet and exposed to the room temperature and temperatures of 100 °C and 200 °C for 1, 2 or 3 h. After heat exposure and before conducting compressive strength testing, visual inspection of the specimens subjected to the heating regime of 100 °C did not show any severe damage; whereas specimens subjected to the heating regime of 200 °C showed some deterioration of the CFRP surface due to epoxy deterioration.

Two specimens of each type were tested and the axial stress, axial and lateral strains were recorded. The performance of these cylinders under axial load was found to be consistent. Cracking noises were heard minutes prior to failure, indicating the start of stress transfer from the dilated concrete to the jacket. The failure was gradual and explosive in nature, characterized by crushing of concrete followed by rupture of the CFRP jacket, which primarily took place in the middle portion of the specimen. Also, the failure mechanism of the wrapped cylinders exposed to high temperature was

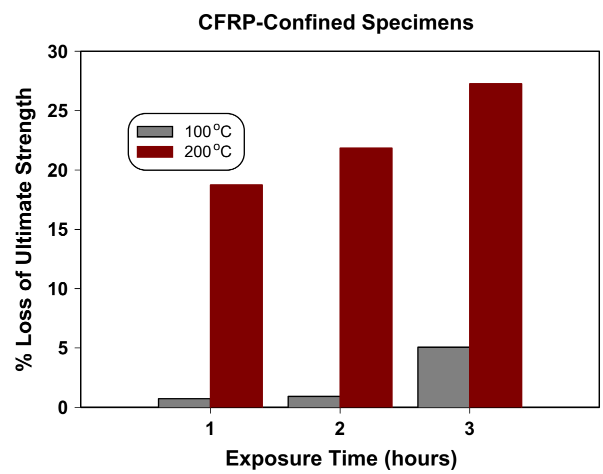


Fig. 11. Percentage loss of average compressive strength for CFRP-confined specimens due to high temperature exposure.

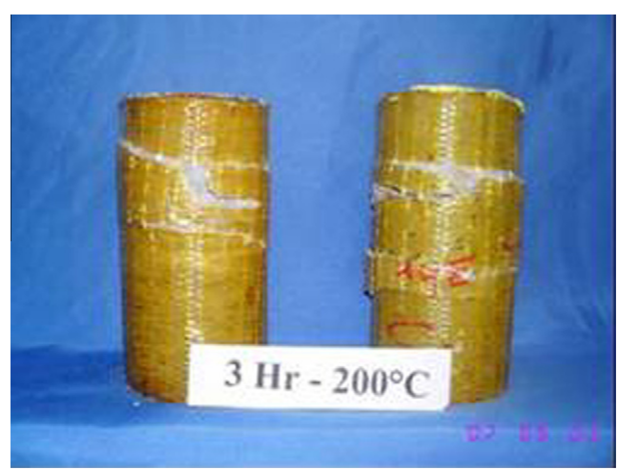


Fig. 12. Failure of the GFRP-confined specimens.

Table 7
Results of GFRP-confined specimens.

Average compressive strength (MPa)			
Temperature	Exposure time (h)		
	1 h	2 h	3 h
Room	69.6	69.6	69.6
100 °C	66.7	65.2	61.7
200 °C	61.1	58.8	58.0

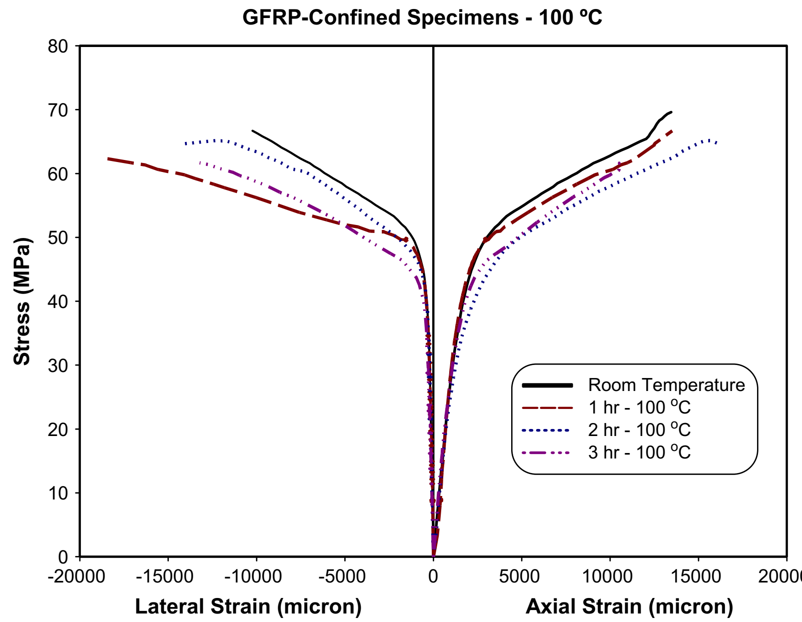


Fig. 13. Stress–strain curves of GFRP-confined specimens exposed to 100 °C.

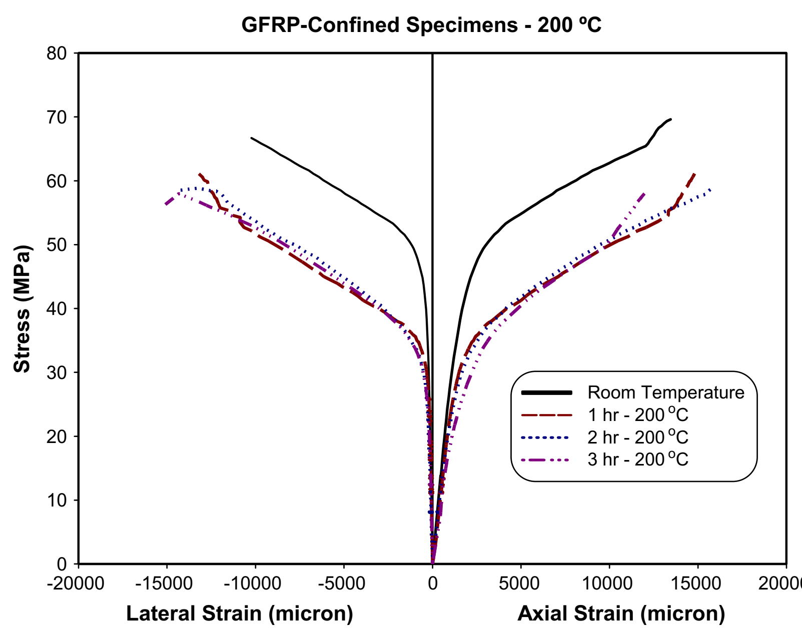


Fig. 14. Stress–strain curves of GFRP-confined specimens exposed to 200 °C.

very sudden and much more explosive (Fig. 8), compared to that of the unexposed (i.e. room temperature) specimens.

Table 6 presents the compressive strength for CFRP-wrapped specimens tested in this study. It should be noted that the values of compressive strength shown in Table 6 are average of two specimens. Figs. 9 and 10 show the stress–strain diagrams for the CFRP-confined specimens at room temperature, 100 °C and 200 °C respectively. These diagrams show that the nature of stress–strain variation is not significantly affected by the temperature rise. Yet, at 200 °C, the strains are more in magnitude than those for 100 °C exposure specimens. This may be attributed to softening of FRP or reduced confinement of concrete at higher temperature. Fig. 11 shows a comparison of the loss of compressive strengths, for the CFRP-confined concrete cylinders subjected to the two heating regimes, compared with specimens at room temperature. It is again illustrated that at given temperature, confined concrete strength decreases with time. However, rate of loss in strength is more pronounced if elevated temperature is substantially higher than the glass transition temperature. Therefore, in order to avoid any serious consequences, the elevated temperature should not go much beyond glass transition temperature of the polymer matrix. In case the elevated temperature is to be allowed to go up to 200 °C (approximately 2.5 times the glass transition temperature), the ultimate load should be kept at least 25% less than its corresponding load at room temperature. It is also worth to mention that the elevated temperature can be endured by the CFRP sheets for a limited time only, and therefore, efforts should be made to reduce the elevated temperatures within few hours (e.g. by extinguishing the fire) as at high temperatures, loss in strength with time is very dramatic.

4.3. GFRP-wrapped specimens

This section represents concrete cylinders wrapped with one layer of GFRP sheet and exposed to a temperature of 100 °C and 200 °C for 1, 2 or 3 h. Two specimens of each type were tested and the maximum axial stress, axial strain and lateral strain were recorded. Fig. 12 shows failed specimens after testing. From the figure, it can be seen that the concrete inside the specimen was disintegrated, and the jacket ruptured in small continuous strips. Also, the failure mechanism of the wrapped cylinders exposed to high temperature was very sudden and much more explosive, compared to that of room-temperature specimens. Table 7 lists the compressive strength for GFRP-wrapped specimens tested in this study. It should be noted that the values of compressive strength shown in Table 7 are average of two specimens. Figs. 13 and 14 illustrate the experimental stress–strain curves for the GFRP-confined specimens at room temperature and subjected to 100 °C and 200 °C respectively. Fig. 15 shows a comparison of the loss of compressive strengths for the GFRP-confined concrete cylinders subjected to the two heating regimes, compared with specimens at room temperature. The trend of loss in compressive strength is the same as that found in CFRP-confined specimens. However, at 200 °C of exposure, strength degradation in CFRP-confined specimens was more severe than that of GFRP-confined specimens.

5. Comparison of test results of phase 1 with other experimental data

Figs. 16 and 17 show the relationship between the exposure time and the compressive strength enhancement ratio (ratio between compressive strength of FRP-confined specimens at certain temperature exposure, f'_{cc}, and unconfined compressive strength at room temperature, f'_{co}). It is demonstrated that as the exposure time increases, the strength enhancement ratio decreases, especially for specimens exposed to a temperature of 200 °C.

For unconfined concrete at 200 °C, it was observed that the loss of the compressive strength during 3 h of exposure was about 12% as seen in Figs. 7 and 17. However, in the test carried out by Chowdhury et al. [12], it was observed that even through the temperature at the concrete surface for the insulated column went up to approximately 400 °C at 300 min, the column still maintained a constant axial deformation value under the sustained applied load. The difference between unconfined specimens tested in this research and columns tested by Chowdhury et al. [12] is that the rate of temperature increase was much severe in this study (temperature increased to 200 °C in 50 min and was maintained constant for the next 3 h), however, in the study by Chowdhury et al. [12], the temperature at concrete surface gradually increased to 400 °C in 300 min. This leads to the conclusion that the rate of temperature increase plays an important role in the performance of concrete under elevated temperature regimes. For the same target temperature, as the rate of temperature rise increases the loss in concrete strength becomes more pronounced.

It is obvious that even though test specimens in this study were exposed to heating regimes with temperature as high as 2.5 times T_g sustained for 180 min, the ultimate capacity of the FRP-strengthened specimens was at least 149% of that for unconfined concrete. This means that as long as the temperature at the FRP level is still under the decomposition limit of the epoxy polymer matrix, the FRP is deemed effective (but with less efficiency) in contact-critical applications. The same conclusion was reached by Foster and Bisby [30] as it was stated that for FRP strengthening applications that are not bond critical, much smaller thicknesses of supplemental insulation may be allowable for FRP-strengthened concrete members (and their residual performance may be much better than cur-

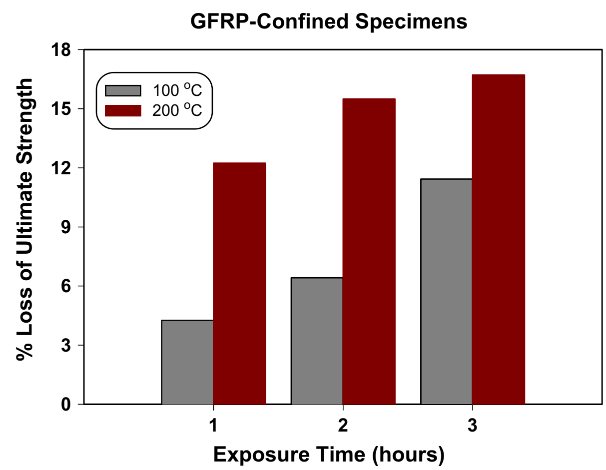


Fig. 15. Percentage loss of average compressive strength for GFRP-confined specimens due to high temperature exposure.

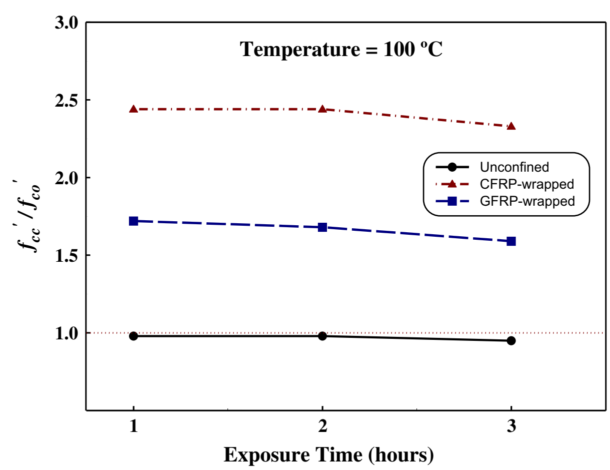


Fig. 16. Relationship between the exposure time of 100 °C and the compressive strength enhancement ratio.

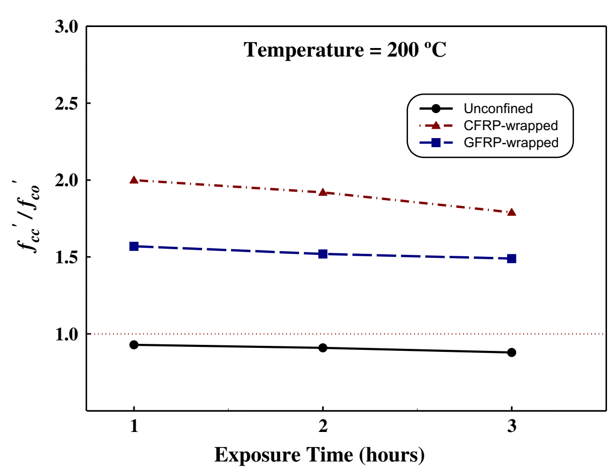


Fig. 17. Relationship between the exposure time of 200 ºC and the compressive strength enhancement ratio.

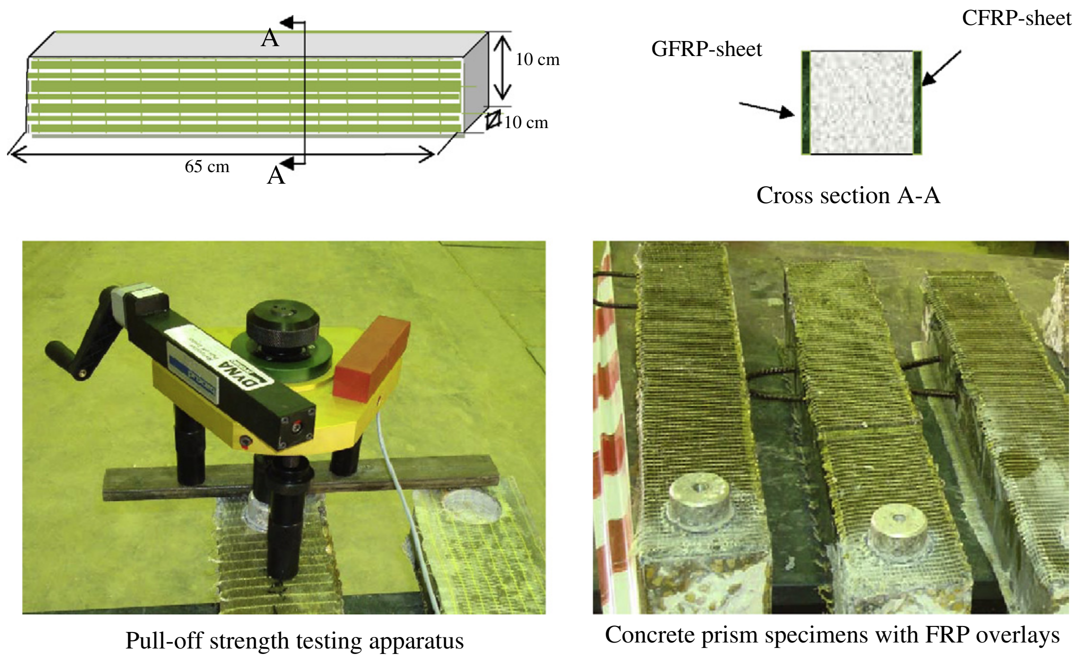
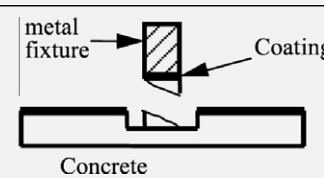
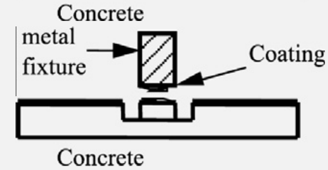
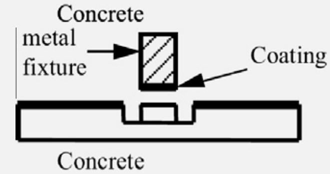
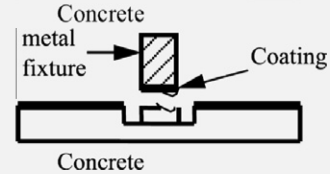
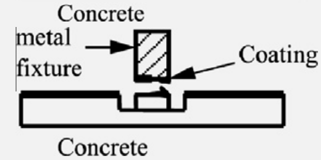


Fig. 18. Concrete prisms for pull-off tests.

Fig. 19. Concrete prisms in the oven ready for heating.

Table 8
Most common failure types in pull-off tests [32].

Failure type	Shape of failure
Type 1, concrete failure	
Type 2, coating failure	
Type 3, bonding interface failure	
Type 4, bonding + concrete failure	
Type 5, bonding + coating failure	

rently thought). Contrary to this conclusion, Chowdhury et al. [12] mentioned that if the temperature at the FRP level exceeds the glass transition temperature (T_g), it can be conservatively assumed that the FRP wraps are rendered structurally ineffective.

6. Experimental program for phase 2 (pull-off tests)

6.1. Test specimens and procedure

In order to evaluate the deficiency in bond strength between FRP and concrete substrate when exposed to elevated temperature environments, the second phase of the experimental program was conducted. In this phase, pull-off strength tests were carried out on FRP-strengthened specimens as per ASTM D4541 – 09 [31]. In this regard, three $10 \times 10 \times 65$ cm concrete prisms were prepared. One face of each prism was overlaid by a single layer of CFRP sheet whereas the opposite face was covered by one layer of GFRP laminate as seen in Fig. 18. A ready-mix concrete with specified 28-day strength of 25 MPa was utilized in this phase. It should be outlined that even though the concrete strength used in casting the prisms is lower than that used for the cylinders, the objective of conducting this phase will not be affected. The goal is to evaluate the degradation in the bond strength at the FRP-concrete interface due to high temperature exposure and as long as the same concrete strength was maintained for the prisms in this phase, a good

Table 9
Results of pull-off tests performed on specimens.

Exposure time and temperature	Average bond strength (MPa)		% loss of bond strength		Mode of failure
	CFRP	GFRP	CFRP	GFRP	
Room	1.16	1.18	–	–	Type 1 (concrete failure)
3 h – 100 °C	1.07	1.17	8.2	0.84	Type 1 (concrete failure)
3 h – 200 °C	0.74	0.95	36.2	20.3	Types 1 and 4 (bonding and/or concrete failure)

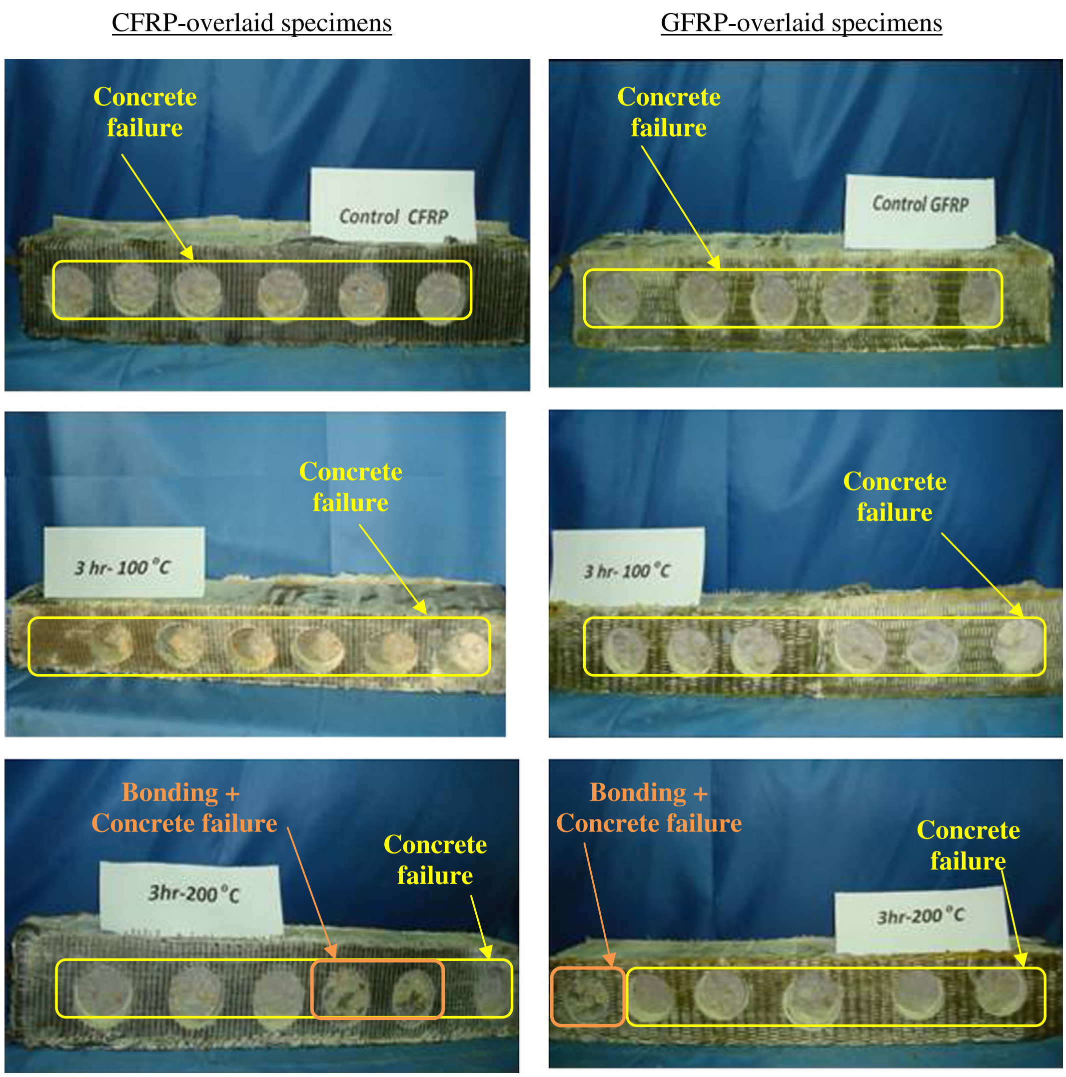


Fig. 20. Mode of failure for concrete prisms.

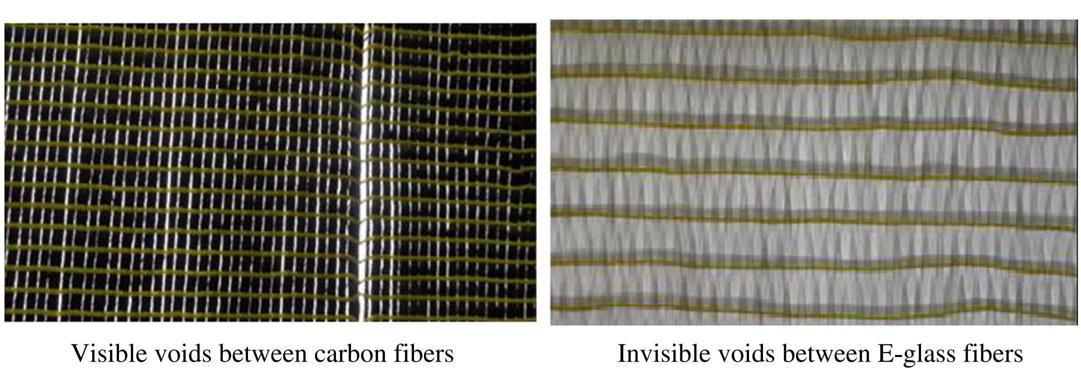


Fig. 21. Close-up picture of carbon and E-glass fibers used in this study.

representation of bond strength degradation can be obtained. It should be also noted that the FRP composites used in this phase were the same as those used for wrapping the cylinders in phase 1 of the experimental program. One of the prisms was used as a control specimen, where it was exposed to the room temperature; the second prism was heated to a temperature regime of 100 °C for a period of 3 h and the third specimen was subjected to a temperature of 200 °C for the same 3 h period. For this purpose, a small-scale electrical oven, with internal dimensions 75 × 60 × 50 cm and as shown in Fig. 19, was used. The time–temperature curve shown in Fig. 2 was followed for heating the specimens.

After the exposure to elevated temperature, the two specimens were allowed to cool down to the room temperature. For each FRP-strengthened face of the prisms, six pull-off tests were carried out. The pull-off test was performed by securing a loading fixture (dolly) perpendicular to the surface of the coating with an adhesive. After the adhesive was cured, a testing apparatus, as shown in Fig. 18, was attached to the loading fixture and aligned to apply tension normal to the test surface. The force applied to the loading fixture was then gradually increased and monitored until either a plug of material is detached, or a specified value is reached. When a plug of material is detached, the exposed surface represents the plane of limiting strength within the system. The nature of the failure was qualified in accordance with the percent of adhesive and cohesive failures, and the actual interfaces and layers involved. The pull-off strength was computed based on the maximum indicated load and the original surface area stressed. During the pull-off tests, different failure modes may be observed. Summary of all possible modes of failure in a pull-off test, as mentioned in reference [32], are listed in Table 8.

6.2. Discussion of test results

The results for the pull-off tests carried out on the concrete prisms bonded with two types of FRP sheets (GFRP and CFRP) and exposed to room and elevated temperatures of 100 °C and 200 °C for 3 h have been reported in this section. Table 9 depicts a summary of results for the pull-off tests carried out in this study. The bond strength tabulated in Table 9 is the average bond strength for the six pull-off tests carried out as described earlier for each case.

6.2.1. CFRP-overlaid specimens

The average bond strength for the prism bonded with CFRP sheet and exposed to room temperature was 1.16 MPa, whereas the average bond strength for the two prisms exposed to elevated temperatures of 100 °C and 200 °C for 3 h was 1.07 and 0.74 MPa respectively. For the CFRP-overlaid prism specimens exposed to temperatures of 100 °C and 200 °C, the percentage reduction in the average bond strength compared with the control specimen was 8.2% and 36.2%, respectively. The mode of failure for all the six pull-off tests on control specimen was concrete failure due to the tension during the pull-off test (Type 1 failure in Table 8). This type of failure is the most desired failure type because it proves that the bonding strength between FRP and concrete is higher than the tensile strength of concrete. It was also found that, the mode of failure did not change for the CFRP-overlaid specimen exposed to 100 °C for a period of 3 h. However, for the case of CFRP-overlaid prism exposed to 200 °C for a period of 3 h, it was observed that in case of two pull-off tests, bonding along with concrete failure (Type 4 failure in Table 8) occurred. This type of failure might have occurred as a result of the epoxy resin melting at the elevated temperature. Fig. 20 shows the failure modes obtained during the pull-off tests for the prisms overlaid with FRP sheets and exposed to different temperatures.

6.2.2. GFRP-overlaid specimens

From the results summary in Table 9, it was observed that, the average bond strength for the concrete prism overlaid with GFRP sheet and exposed to room temperature was 1.18 MPa, whereas, the average bond strength for the two GFRP-overlaid specimens exposed to elevated temperatures of 100 °C and 200 °C for a period of 3 h, decreased to 1.17 and 0.95 MPa, respectively. For the GFRP-overlaid specimen exposed to 100 °C, the percentage decrease in the average bond strength was negligible (0.84%). However, a percentage reduction of 20.3% was observed for the GFRP-overlaid specimen exposed to 200 °C for a period of 3 h. Comparing the failure modes of GFRP-overlaid specimens, it was found that the specimens exposed to room temperature and 100 °C had concrete failure in all the six pull-off tests for each case. Only two pull-off tests out of the six for the GFRP-overlaid specimen exposed to the temperature of 200 °C for 3 h had bonding failure along with concrete failure (Type 4 failure in Table 8). The remaining four tests resulted in concrete failure.

From the above discussion and the results of the pull-off tests carried out, it was observed that the reduction in bond strength was higher at elevated temperatures in case of CFRP-overlaid specimens compared with the GFRP-overlaid specimens. This could be attributed to the fact that, carbon fiber sheet has visible voids in between the fibers as seen in Fig. 21. These voids may cause the epoxy to be directly exposed to the elevated temperatures thereby resulting in the reduction of the epoxy-matrix strength. However, in case of GFRP sheet, the absence of voids and the close-knit structure of the fibers may protect the epoxy resin from the adverse effects of the elevated temperatures.

7. Conclusions

Based on the present experimental study, the following conclusions may be drawn:

- FRP materials used as externally bonded reinforcement for concrete structures are sensitive to the effects of elevated temperatures. FRPs experience degradation in strength and bond at temperatures exceeding glass transition temperature (T_g) of the polymer matrix.
- In order to avoid any serious consequences, the elevated temperature should not go much beyond FRP glass transition temperature. In case the elevated temperature is to be allowed to go up to 200 °C (approximately 2.5 times the glass transition temperature), the designed ultimate load for FRP-strengthened members should be kept at least 25% less than its corresponding value at room temperature.
- At elevated temperature, rate of loss of compressive strength with time is very high in FRP-confined specimens. Therefore, best efforts should be made to control the temperature within few hours in order to avoid a very high loss of strength.
- The rate of temperature increase plays an important role in the performance of concrete under elevated temperature regimes. For the same target temperature, as the rate of temperature rise increases the loss in concrete strength becomes more pronounced.
- For FRP material used as external strengthening in contact-critical applications (such as confinement of concrete columns) and exposed to high temperature environments, it was found out that as long as the temperature at the FRP level is still under the decomposition limit of the epoxy polymer matrix, the FRP is deemed effective (but with less efficiency).
- From the results of the pull-off test conducted in this study, it was concluded that a significant degradation in the bond strength between the FRP and concrete substrate occurred at a temperature of 200 °C. In addition, the reduction in bond

strength was higher in case of CFRP-overlaid specimens compared with the GFRP-overlaid specimens. This lies in line with the results of the FRP-confined cylinders exposed to elevated temperature environments where the loss in concrete strength was more pronounced for the case of 200 °C especially for CFRP-confined cylinders.

Acknowledgements

The authors gratefully acknowledge the support received from the Saudi Arabian Basic Industries Corporation (SABIC) to conduct this research study. Thanks are also extended to the Director and employees of the Research Center at the College of Engineering, King Saud University for their administrative efforts.

References

[1] ACI. 440.2R-02. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures. Farmington Hills, Michigan: American Concrete Institute; 2002.
[2] CSA. S806-02. Design and construction of building components with fibre-reinforced polymers. Mississauga (Canada): Canadian Standards Association; 2002.
[3] ISIS Design Manual No. 4. Strengthening reinforced concrete structures with externally-bonded fibre reinforced polymers. Winnipeg (Canada): Intelligent sensing for innovative structures; 2001.
[4] Technical Report 55. Design guidance for strengthening concrete structures using fibre composite materials. Blackwater, Camberley, Surrey: The Concrete Society; 2000.
[5] Shahawy M, Mirmiran A, Beitelman T. Tests and modeling of carbon wrapped concrete columns. Composite Part B 2000;31:471–80.
[6] Xiao Y, Wu H. Compressive behavior of concrete confined by carbon fiber composite jackets. J Mater Civil Eng 2000;12(2).
[7] Jin XN. Experimental study on the mechanical properties of axially symmetrical confined concrete. Ph.D. thesis. School of Civil Engineering, Harbin Institute of Technology; 2002.
[8] Apicella F, Imbrogno M. Fire performance of CFRP-composites used for repairing and strengthening concrete. In: Proceedings of the 5th ASCE materials engineering congress, Cincinnati, Ohio; 1999. p. 260–6.
[9] Bisby LA. Fire behaviour of FRP reinforced or confined concrete. Ph.D. thesis. Department of Civil Engineering, Queen's University, Kingston, Canada; 2003.
[10] Deuring M. Research report EMPA No. 148'795. Swiss Federal Laboratories for Materials Testing and Research, Dubendorf, Switzerland; 1994.
[11] Blontrock H, Taerwe L, Vandevelde P. Fire tests on concrete beams strengthened with fiber composite laminates. Third Ph.D. symposium, Vienna, Austria; 2000. 10p.
[12] Chowdhury EU, Bisby LA, Green MF, Kodur VKR. Investigation of insulated FRP-wrapped reinforced concrete columns in fire. Fire Saf J 2007;42:452–60.
[13] ASTM. ASTM E119-01. Standard methods of fire test of building construction and materials. West Conshohocken (PA): American Society for Testing and Materials; 2001.
[14] ULC. CAN/ULC-S101-M04. Standard methods of fire endurance tests of building construction and materials. Scarborough (Ont.): Underwriters' Laboratories of Canada; 2004.
[15] Kodur V, Bisby LA, Green MF. FRP retrofitted concrete under fire conditions. J Concr Int 2006;28(12):37–44.
[16] Green MF, Bisby LA, Fam AZ, Kodur VKR. FRP confined concrete columns: Behaviour under extreme conditions. J Cem Concr Compos 2006;28:928–37.
[17] Cleary DB, Cassino CD, Tortorice R. Effect of elevated temperatures on a fiber composite to strengthen concrete columns. J Reinf Plast Compos 2003;22(10):881–95.
[18] Saafi M, Romine P. Effect of fire on concrete cylinders confined with GFRP. In: Proceedings of the 2nd international conference on durability of fibre reinforced polymer (FRP) composites for construction (CDCC '02), Montreal, PQ, Canada; June 2002. p. 512–21.
[19] Bank LC. Properties of FRP reinforcements for concrete. Fiber-reinforced-plastic (FRP) reinforcements for concrete structures: properties and applications. Elsevier Science Publishers; 1993. p. 59–86.
[20] Mouritz AP. Post-fire flexural properties of fibre-reinforced polyester, epoxy and phenolic composites. J Mater Sci 2002;37(7):1377–86.
[21] Bakis CE. FRP reinforcement: materials and manufacturing. Fiber-reinforced-plastic (FRP) reinforcements for concrete structures: properties and applications. Elsevier Science Publishers; 1993. p. 13–58.
[22] Sen, R. Investigation of S-2 glass/epoxy strands in concrete. In: International symposium of fiber reinforced polymer reinforcement for reinforced concrete structures. ACI, Detroit, Michigan; 1993. p. 15–33.
[23] Ueda T, Dai J. Interface bond between FRP sheets and concrete substrates: properties, numerical modeling and roles in member behaviour. Prog Struct Eng Mater 2007;7(1):27–43.
[24] Chen JF, Teng JG. Anchorage strength models for FRP and steel plates bonded to concrete. J Struct Eng, ASCE 2001;127(7):784–91.
[25] Gamage JCPH, Wong MB, Al-Mahaidi R. Performance of CFRP strengthened concrete members under elevated temperatures. In: Proceedings of the international symposium on bond behaviour of FRP in structures (BBFS 2005). International Institute for FRP in Construction, Hong Kong; 2005. p. 113–8.
[26] Leone M, Matthys S, Aiello MA. Effect of elevated service temperature on bond between FRP EBR systems and concrete. J Compos: Part B 2009:85–93.
[27] ASTM. ASTM E119-01. ASTM D3039/D3039M – 08. Standard test method for tensile properties of polymer matrix composite materials. West Conshohocken (PA): American Society for Testing and Materials; 2008.
[28] ASTM. ASTM D638 – 08. Standard test method for tensile properties of plastics. West Conshohocken, PA: American Society for Testing and Materials; 2008.
[29] ASTM. ASTM D3418 – 08. Standard test method for transition temperatures and enthalpies of fusion and crystallization of polymers by differential scanning calorimetry. West Conshohocken (PA): American Society for Testing and Materials; 2008.
[30] Foster SK, Bisby LA. High temperature residual properties of externally bonded. FRP systems. In: Proceedings of the 7th international symposium on fiber reinforced polymer reinforcement for reinforced concrete structures (FRPRCS-7), SP-230-70; 2005. p. 1235–52.
[31] ASTM D4541 – 09. Standard test method for pull-off strength of coatings using portable adhesion testers. West Conshohocken (PA): American Society for Testing and Materials; 2009.
[32] Liu J, Vipulanandan C. Tensile bonding strength of epoxy coatings to concrete substrate. J Cem Concr Res 2005;35:1412–9.