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Review Article

A Review of Performance of Fibre Reinforced Polymer Strengthened Structures Under Fire Exposure

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Fibre reinforced polymers (FRPs) have been rapidly gaining popularity across a variety of civil engineering applications. Over the decades, most of the applications have been for structures where no fire exposure is considered such as bridges and parking lots. For applications in confined spaces, performance under fire is an important property to be considered for any material, due to which, FRP strengthening systems in buildings have not seen widespread use. Due to scarce research in the field, the fibre reinforced polymers do not have sufficient documentation of bond properties and mechanical characteristics at elevated temperatures required for use in buildings. This review consolidates the existing research on the mechanical and bond behavior at high temperature of the constituent materials of FRPs. The paper also discusses experimental results and numerical studies conducted by various researchers for insulated as well uninsulated FRP strengthened concrete members (beams, columns, slabs) at elevated temperatures. Available design guidelines have been discussed. Finally, recommendations for future research are also discussed.

Keywords: Fibre; CFRP; GFRP; Fire Exposure; NSM; EBR; Concrete; Glass Transition Temperature

Introduction

In the last decade,several researches have been conducted on strengthening of reinforced concrete (RC) structures using fiber-reinforced polymers (FRPs). FRPs have been rapidly gaining acceptance across a variety of civil engineering applications. Majority of applications have been for structures where fire performance is not in consideration such as bridges. Although there is a wider market for application in high-rise buildings, garages, factories and parking lots where fire safety is a key concern (Kodur and Baingo, 1998).

A wide range of advantages over traditional rehabilitation methods are provided by this newly developed technology, as it provides easy installation, faster on-field construction and limited disturbance due to construction. Because of its high resistance to corrosion, strength and durability, lightweight characteristics, FRP materials explain their increasing use, contrary to traditional materials (Maraveas *et al*., 2012).

The review includes the mechanical characteristics of different types of FRP materials at elevated temperatures which are based on experimental data. Experimental studies for insulated and uninsulated concrete members strengthened with FRPs (using two different techniques) including respective results from various researchers are presented. The measured variation of temperature at distinct locations within the cross-section of the tested elements and the role of insulation are also discussed.

Application of FRPs is done by the use of one of the two prominent techniques for reinforcement, externally bonded reinforcement (EBR) and near surface mounted (NSM). Both have several advantages and disadvantages over each other. In externally bonded FRPs, the fabrics or sheets are wrapped around the structural members thus increasing the confinement and axial strength. Also it increases the flexural, shear and torsion strengths of the concrete which leads to efficient design of the concrete members. In NSM method, the

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reinforcement is embedded in grooves cut on the structural member to be strengthened and filled with a binding agent such as epoxy paste or cement grout. However, during fire, FRP system (externally-bonded or NSM) is likely to lose structural integrity after a short time during a standard fire (Foster and Bisby, 2008).

Application Techniques

Externally Bonded Reinforcement (EBR)

Composites have found their applications as strengthening materials of reinforced concrete (RC) elements such as beams, slabs, columns etc. where conventional strengthening techniques have failed or been proved to be disadvantageous. In externally bonded reinforcement, the FRP sheets are epoxy-bonded to the external surfaces (e.g., tension zones) of beams and slabs. This technique is simple and effective as far as both cost and mechanical performance are concerned and also does not lead to any corrosion unlike other alternatives such as steel. The externally bonded FRPs provide substantial increase in strength (axial, flexural, shear, torsional) and ductility without much affecting the stiffness and incurring any undesirable weight. The range of applicability of EBR in RC structures is increasing constantly (Fib bulletin 14, 2001).

Near Surface Mounted (NSM)

Near surface mounted (NSM) FRP reinforcement has lately appeared as a promising technology for shear and flexure and strengthening of concrete structures. Near surface mounted (NSM) FRPs is becoming more widely recognized for its efficiency, effectiveness, and ease of application. In these applications, grooves are cut into the concrete cover of a RC member, typically using a diamond cutting disc, and an FRP bar, strip, or tape is inserted, along with an epoxy adhesive. Although, recently, cementitious grouts or adhesives have been attempted to be used in place of the epoxy. In both cases, the primary objective is to strengthen the concrete with well anchored tensile reinforcement. Shear or flexural strengthening can be provided by either of the techniques used for strengthening for reinforced concrete members (Barros and Fortes, 2006).

Material Properties at High Temperature

Kumahara *et al*. (1999) suggested that two types of performance against fire are extremely important (i) Performance against initial fire. (ii) Performance after the fire was extinguished. Performance against initial fire includes: flammability, which is related to smoke and fume-generating properties, which has a direct impact on the safe evacuation of the inhabitants' ability to safely evacuate a building. For the post fire performance the heat-insulating, flame resisting, and smoke barrier properties of separating members play a significant role in the safety (Bisby and Green, 2005).

An important parameter to be considered for FRPs is their glass transition temperature. The Glass transition temperature is the temperature above which the performance of an FRP drops dramatically due to loss of its modulus properties. The polymer transitions from a hard, glassy material to a soft rubbery material. Glass transition temperature (T_g) for anFRP material typically lies between 50-90°C (Foster and Bisby, 2008). Thus, for the integrity of any structural application, it is of paramount importance to make sure the design service temperature never reaches the glass transition temperature. Furthermore, at temperatures above 400°C FRPs are susceptible to combustion. Unprotected FRPs may ignite leading to the spread of flames and evolution of smoke. The volatile parts produced by decomposition of the polymer matrix may react with available oxygen which may release more heat and increase the temperature (Green, 2004). After the fire, the FRP materials can suffer failures such as delamination, cracking, charring, melting, and buckling making its structural integrity questionable.

Thermal properties at elevated temperatures

Thermal Expansion

Another deformation property that plays a prominent role in the fire behavior of structural members is thermal expansion. The coefficient of thermal expansion (CTE) represents the change in unit length of a material due to unit temperature rise or drop. Due to differential thermal expansion internal pressure develops in FRP which leads to spalling of concrete when FRPs are used as internal reinforcement. Also, the development of shear stresses occur within the adhesive layer when FRPs are used in external applications. At room temperature, the CTEs of composites are generally higher than the CTEs of steel (Mallick, 2007). The transverse thermal expansion of FRP reinforcements for concrete is different from that of concrete without FRPs. Differential expansion can also result in cracking of the concrete which can severely limit the structural safety of FRP-reinforced structures in fires.

Table 1: Thermal conductivities of various unidirectional FRPs and building materials (Mallick, 2007)

Material	Thermal Conductivity (W/m-°C)	
	Longitudinal	Transverse
Glass/Epoxy	3.46	0.35
Aramid/Epoxy	1.73	0.73
High Modulus Carbon/Epoxy	48.44-60.55	0.87
Ultra-High Modulus Carbon/Epoxy	121.1-129.8	0.04
Boron/Epoxy	1.73	1.04
Aluminum	138.4-216.3	
Steel	15.57-46.71	
Epoxy	0.346	

Table 2: CTEs of various unidirectional FRPs and building materials (Mallick, 2007)

Material	Coefficient of Thermal Expansion (10^{-6}/ °C)	
	Longitudinal	Transverse
Glass/Epoxy	6.3	19.8
Aramid/Epoxy	-3.6	54
High Modulus Carbon/Epoxy	-0.09	27
Ultra-High Modulus Carbon/Epoxy	-1.44	30.6
Boron/Epoxy	4.5	14.4
Aluminum	21.6-25.2	
Steel	10.8-18	
Epoxy	54-96	

Thermal Conductivity

Thermal conductivity, represents the capacity of a material to conduct heat. For composite materials, it is a function of the fibre type, orientation, volume fraction and laminate configuration. As, in civil engineering applications unidirectional composites are used, the longitudinal thermal conductivity is controlled by fibres and the transverse thermal conductivity is controlled by the matrix. Some typical values of thermal conductivities for various FRP materials at room temperature are given in Table 1. Figure 1 plots the thermal conductivity and specific heat of different CFRP materials reported in the literature.

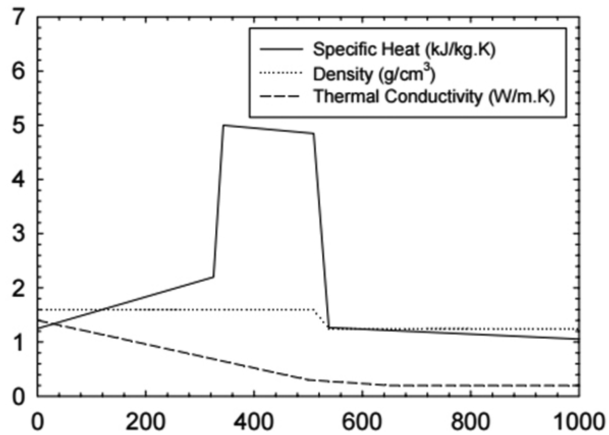

Fig. 1: Variation of transverse thermal conductivity, density, and specific heat with temperature for carbon/epoxy FRP (Bisby and Green, 2005)

Specific Heat

Specific heat is the rate of heat transfer through a material. It is extremely difficult to determine the variability of specific heat with temperature due to the complex chemical reactions that take place in an FRP at elevated temperature. Griffis *et al*. (1984) reported the variation in specific heat as shown in Fig. 1 to be used for heat transfer calculations in a carbon/epoxy FRP.

Mechanical Properties at Elevated Temperatures

During the past two decades, many researchers have studied the effect of high temperature on the mechanical properties of FRP materials. It is well established that the strength of materials decreases with increased temperature. Deterioration in mechanical properties in the case of FRP reinforcement for concrete structures is critical to the fire performance of the material, since decrease in elastic modulus and strength during a fire could lead to large deflections, loss of reinforcement, and finally the collapse of structure.

Tensile Strength

The tensile strength of FRP materials is important when we need to determine the load bearing capacity of FRP reinforced/strengthened members. Sen *et al.* (1993) investigated the tensile strength of various types of GFRPs at varying temperatures and concluded that glass fibres lose about 50% of their original tensile strength above 550°C. Carbon fibres displayed much higher resistance to high temperatures. They do not experience any change in their tensile strength for temperatures greater 1000°C.

Gates (1991) investigated the variation in the longitudinal, transverse, and shear moduli of a carbon FRP and a carbon thermoset FRP in the temperature range of 23°C to 200°C. The results suggested that changes in the transverse and shear moduli of carbon FRPs were more visible at elevated temperatures than those observed for the longitudinal modulus, which showed no significant changes up to 200°C. It is important to note that the glass transition temperature quoted in the study was 220°C, which is unusually high for an FRP composite. Gates (1991) also studied the stress-strain relationship for the CFRPs at elevated temperature and found that the tensile strength reduced by about 40-50% at 125°C and 80-90% at 200°C, which indicates very high strength degradation at temperatures lesser than matrix T_g.

Kodur and Baingo (1998) surveyed the literature on FRP properties at high temperature, and suggested relationships for the change in tensile strength of FRP materials with increasing temperature. Figure 2 shows the strength versus temperature relationships as reported in the study as compared with curves for concrete and steel, and illustrates that FRP is highly sensitive to elevated temperatures (Williams *et al.*, 2008).

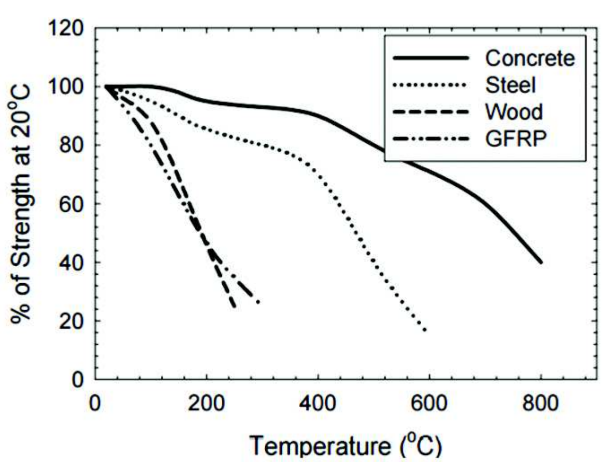

Fig. 2: Strength-temperature curve for concrete, steel, timber and FRP (Williams *et al.*, 2008)

Chowdhury *et al.* (2011) conducted a detailed study of the thermal and mechanical behaviour of FRP materials at high temperature. The results showed that the epoxy FRP material tested experienced 50% loss in tensile strength, 30% decrease in tensile elastic modulus, and 60% decrease in FRP to FRP bond strength at temperatures 15°C below the T_g of its resin matrix. The loss of strength was attributed to the loss of load-sharing between the individual fibres.

Wang *et al.* (2011)studied the tensile strength of CFRP strips at temperatures ranging from room temperature up to 700°C, in both steady-state and transient conditions. For both the series the CFRP strips retained 60 % of the room temperature tensile strength in the range 20 to 150°C and decreased from 45% to less than 10% of the room temperature tensile strength in the range 450°C to 700°C. Between 150°C and 450°C a gradual reduction in strength took place, with about 50% of strength at ambient temperature at 300°C. The results obtained from the transient state series were slightly higher yet similar to those of the steady-state series.

Creep

Creepis the tendency of a solid material to move slowly or deform permanently under the influence of mechanical stresses. It occurs as a result of long-term exposure to stresses that are high yet below the yield strength of the material. Creep can become a critical factor in the design of FRP-reinforced concrete as excessive long-term deflections can lead to unserviceable structures or to the rupture of FRP reinforcement. Thermally accelerated creep may be severe and could lead to large deformations and failure which may lead to uncertainty over the use of FRPs at high temperatures (Bisby and Green, 2005).

In general, creep strain of FRPs increases with the rise of temperature and is greatly dependent on the matrix material. Also, the orientation of the fibre influences the temperature dependence of the creep characteristics of FRPs. (Mallick, 1988) found that creep in the fibres governs creep in the composite, if the fibres are in the loading direction.

Bond Properties at Elevated Temperature

The bond strength between FRPs and concrete is an important factor because it is the critical parameter when concrete is strengthened with FRPs as the bond transfers loads through the shear stresses which arise in the matrix or adhesive layer, from the FRP to the concrete (Bisby and Green, 2005).

Katz, (1999) studied the effect of elevated temperature on the bond properties of FRP bars in concrete at high temperatures by performing pullout tests with six different types of surface textures. They found that up to temperatures of 100°C, there was no notable drop of bond strength, but after 100°C, the bond strength fell rapidly. Interestingly, at temperatures of 200°C to 220°C, the bond strength dropped to 10% of the original value.

Smoke Toxicity

During a fire, polymers generally produce large amounts of very dense, sooty, black smoke and some components such as carbon monoxide that are toxic to humans. In addition to fire resistance, it may be necessary to test FRP products for evolution of smoke and toxins. Smoke toxicity is most critical in cases where the FRP is applied at the outside of concrete member. While charring of matrix in internally reinforced FRP may not be a serious problem due to the presence of the concrete cover, , it may generate smoke that could escape through cracks which are developed in the concrete at high temperatures (Kodur and Baingo, 1998).

Experimental Studies

EBR Strengthening

Foster and Bisby (2008) conducted fire tests on externally bonded FRP systems to study (i) The residual ultimate tensile strength, failure strain and elastic modulus of well anchored unidirectional FRP members. (ii) The residual FRP to FRP bond in pure shear mode II as it would be important in applications where the contact between FRP to FRP lap splices is critical for performance, e.g., column wrapping and (iii) The residual FRP-to-concrete bond under direct tension Mode I and in pure shear Mode II.

The Thermogravimetric analysis (TGA) curves provided in Fig. 3 show that the glass fiber fabric retained almost all of its initial mass till 800°C, while both carbon fiber fabrics had lost about 10% of their initial mass at those temperatures. None of the fiber fabrics lost any notable mass at temperatures below 400°C. Both of the epoxy resin systems lost 90% of their mass at 800°C, with 80–90% of this loss taking place between 300°C and 400°C.

For accomplishing the required fire exposures the specimens were kept in an electric furnace and were heated to a preset maximum temperature at the rate of 10°C/min. The temperature was held constant for 3 hours at the maximum temperature and then allowed to cool to room temperature to simulate the fire exposure under a standard fire. After cooling the required tests were conducted on the specimen.

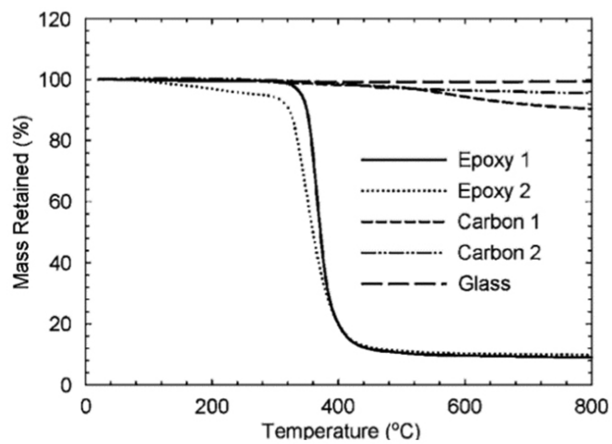

Fig. 3: Results of thermogravimetric analysis of various material components for the FRP strengthening systems (Foster and Bisby, 2008)

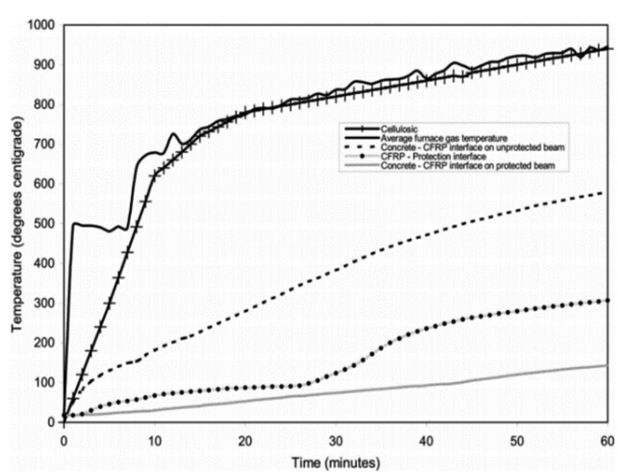

Fig. 4: Cellulosic time/temperature curve adapted from BSI 1987 compared with temperature measurements at various locations on the specimens (Barnes and Fidell, 2006)

The experiment concluded that residual tensile strength of externally bonded FRP systems experienced greater than 50% reductions at temperatures approaching thermal decomposition of epoxy polymer matrices. For temperatures greater than 100°C above T_g, the FRP systems were able to maintain about 80% of their tensile strength. Thus practically exposure temperature up to 200°C may be permissible. The FRP to FRP bond in mode II and FRP to concrete bond in mode I retained 80% of their strength at 150°C greater than T_g, i.e., at approximately 250°C.

Barnes and Fidell (2006) conducted tests to investigate the behavior of carbon FRP (CFRP) strengthened RC beams under fire exposure. The test program consisted of a total of 24 RC beams with across-sectional area of 100x150mm strengthened with CFRP plate of 100mm width and 1mm thickness, then insulated by applying one layer of 15-20 mm thick cementitious (cement/gypsum) based fire insulation. To study the usefulness of insulation and mechanical bolting of CFRP plate, testing was done under standard fire conditions (British Standard Institute BSI 1987). The beams were exposed to the fire for 1 hour without applying any loads and later exposed to 4 point bending loads till the failure of beams was attained. Figure 4 shows the temperature measured at various locations on the specimen.

The test results indicated that the adhesive bond between concrete and CFRP plate was destroyed by the fire due to the loss of resin component of the CFRP although the carbon fibers were still intact. It was concluded that the fire protection given was not sufficient to keep the temperatures below T_g at the FRP-concrete interface beyond 30-45 min. The matrix in CFRP plate endured temperatures only till 310°C, while the carbon fibers lasted up to 950°C.

Williams *et al*. (2008) studied the fire performance of two large-scale RC T-beams flexurally strengthened with CFRP sheets due to which the strength(flexural)was predicted to increase by 15%. The CFRP sheets were protected with vermiculite/ gypsum cementitious insulation, applied on the bottom and lateral faces of the beams with thicknesses of 25 mm (Beam 1) and 38 mm (Beam 2) respectively. The beams were loaded to 48% of their predicted ambient strength and the ends of the beams were axially restrained during the tests. Both test specimens achieved 240 minutes of fire endurance. However, in both the cases adhesive T_g (quoted as 93°C) was exceeded between 16 to 36 minutes in Beam 1 and between 55 to 57 minutes in Beam 2.

Adelzadeh *et al*. (2012) further extended this study with tests on two additional T-beams but with insulation of40 mm cement-based, spray-applied mortar on their lateral and bottom faces and loaded up to 71% of their theoretical ambient ultimate capacity before being subjected to the ASTM E119 fire. The results obtained were similar to those by (Williams *et al*., 2008)despite exceeding the adhesive T_g (quoted as 50-60°C) in less than 30 min. None of the beams failed and both beams achieved 240 min of fire endurance. This high fire endurance was because of the insulation system maintaining low temperatures. But flexural strength provided was low as the RC beams originally guaranteed that they would not collapse even if the CFRP was lost. Axial restraint on the beams' ends might also have enhanced their fire endurance (Firmo *et al*., 2015).

Ahmed and Kodur (2010) tested 4 loaded RC beams strengthened with CFRP sheets due to which the flexural strength was predicted to increase by 50%. The beams were exposed to either the ASTM E119 curve or a design fire deûned using Eurocode 1 for a typical office compartment. All beams were 4-point loaded upto half of their theoretical capacity. The beams were simply supported at the ends having an unsupported span of 3.66m. For studying the influence of cooler anchorages, CFRP was retrofitted along the whole unsupported length of two beams, whereas only the central length (2.44 m)that was directly exposed to fire was retrofitted with CFRP for the remaining two.

The results show that insulated FRP-strengthened RC beams could sustain both types of fire exposures for more than 3 hours. This was due to the presence of insulation, which played a vital role in keeping the temperature low in the rebars for the entire duration of the fire test. The steel reinforcement maintained nearly full strength (flexural capacity) for the test duration as rebars do not lose any significant strength up to 400°C. The CFRP debonded from the concrete between 20 to 25 min of fire exposure when the temperature in the concrete-CFRP interface reached around the adhesive T_g (given as 82°C). The axial restraint forces that were induced by fire, reduced the deformability of the strengthened beams. None of the beams failed in terms of fire resistance and no differences were reported in terms of time of FRP debonding.

Firmo *et al*. (2012) performed 6 fire resistance tests on RC beams with CFRP strengthened strips and loaded to half of their theoretical ambient temperature strength. The soffits of the beams were insulated with either CS (Calcium Silicate) boards or vermiculite/perlite cementitious mortar. The thickness of the insulation was 25-40mm while the anchorage zones were additionally thermally insulated by the furnace walls. Therefore the ends of the strengthening systems, were not directly exposed to the ISO 834 fire. This method of insulation allowed the heated length of the CFRP strip to transform into a "cable" which was fixed at the anchorage zones, where the CFRP-concrete bond remained relatively undamaged (Fig. 5).

The CFRP system failed when one of the anchorage zones lost its bond strength, and it was associated with the average temperature in these areas crossing the adhesive T_g which was quoted as 55°C. Fig. 6 shows the mid-span deûection plots with time for all beams in the experiment tested by (Firmo

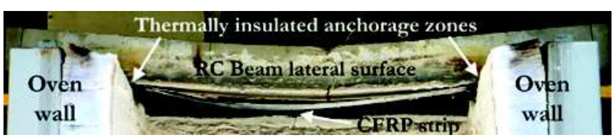

Fig. 5: CFRP strip detached from the RC beam only at the central zone e "cable" behavior (Firmo *et al.*, 2012)

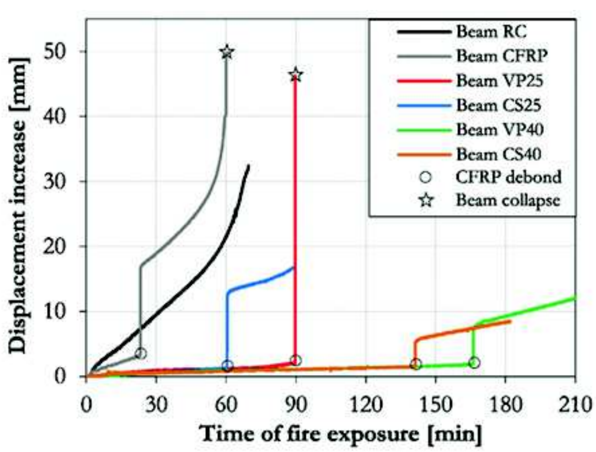

Fig. 6: Increase of midspan deflection vs. time, beams tested by (Firmo *et al.*, 2012)

et al.,, 2012). It is identified from Fig. 6 that the bond failure of the CFRP system is seen by a sudden rise in mid-span deflection due to the reduction in stiffness. The strategy for protection leads to a considerable increase of the fire endurance of the strengthening system, with the increase varying between 23min and 167 min for different insulation conditions.

Ahmed and Kodur, (2011) conducted tests to measure the effect of insulation on FRP reinforced beams under fire and to address previous shortcomings in research to gain test data for validating numerical models. For the test, four rectangular RC beams were designed as per ACI 318 specifications to represent typical beams in buildings. The RC beams having a compressive strength of 55 MPa on the day of the test, were strengthened with FRP sheets (2 mm thick and 203 mm width) to increase the flexural strength capacity by 50%. A two-component epoxy material having a glass transition temperature (T_g) of 82°C was used. FRP was applied on the entire unsupported length on two of the beams to study the influence of anchorage zone and the other two were retrofitted with FRP sheets to evaluate the effect of debonding FRP-RC beams. The beams were applied with a vermiculite-based insulation (VG insulation) and epoxy coating. For the first two beams, the fire comprised of a growth phase followed by a cooling phase while other two beams were tested under standard fire. The beams were subjected to two point loads of approximately 50% of the strengthened beam's nominal capacity.

The temperature at beam cross sections increased throughout the test duration for beams B3 and B4 as they were exposed to the standard fire and in beams B1 and B2 temperatures first increased to a maximum value and then started to decrease. The decrease in temperature can be due to the cooling phase in the fire. Due to development of cracks, the temperature increased in the FRP and resulted in localized burning of epoxy since T_g for epoxies is low (82°C).

Due to the formation of a protective char layer as a result of the pyrolysis the temperature at the FRP concrete interface increased slowly. The average rebar temperature remained below 400°C for the complete test duration which resulted in a minimal loss in strength of rebar and thus the steel reinforcement maintained full strength capacity for the test. This led to achieving high fire resistance in these beams. Therefore, the test implied the conclusion that an effective insulation scheme in FRP-strengthened RC beams is critical for achieving good fire resistance.

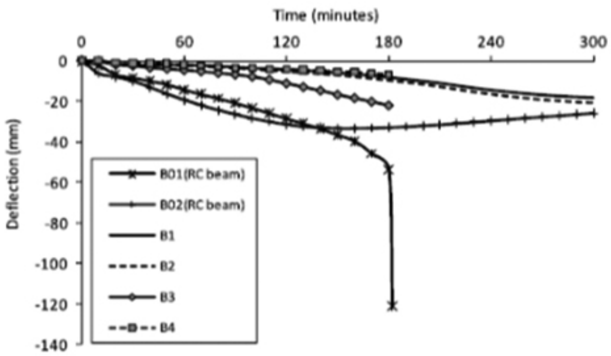

Fig. 7: Time deflection curve for the four beams (Ahmed and Kodur, 2012)

NSM Strengthening

Yu and Kodur, (2014)tested 4 full-scaleNSM strengthened RC T-beams with CFRP strips by exposing it to ASTM E119 fire, loaded in a 4-point bending conûguration. The test parameters included the load level (50-65% of the predicted ambient capacity), the presence of a 25 mm thick U-shaped fire insulation system and the effect of the axial restraint. The structural performance of the 4 NSM FRP strengthened beams, under the given conditions, is compared in Fig. 7. It was found that the CFRP strips debonded from the central zone of the beam instantly during the fire, but continued to transmit tension due to the cool anchorages. As a result, NSM FRP system retained its structural effectiveness during the full 210 min of ûre exposure in all cases.

Palmieri *et al*. (2012) evaluated the ûre behavior of twelve beams with a span of 3.15 m, tested under fire exposure with two fire test series of six beams each. Out of the twelve beams two were unprotected and un-strengthened beams and 10 were insulated and strengthened RC beams. The insulated beams were strengthened in flexure with NSM-FRP strips or rods (GFRP/CFRP) bonded to the concrete with either epoxy adhesive, with T_g quoted as between 62°C and 65°C, and a high T_g epoxy (82°C). The insulation comprised of five different materials with

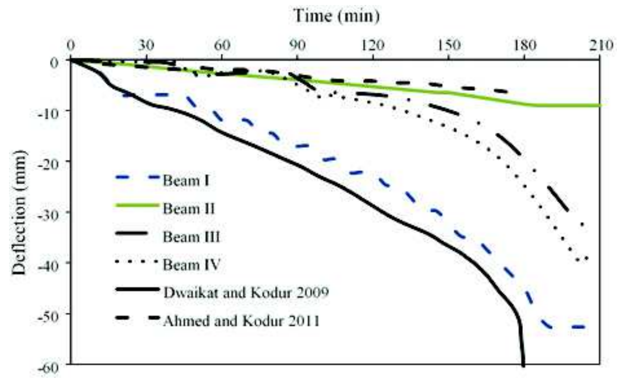

Fig. 8: Comparison of mid-span deflections of NSM FRP strengthened RC beams with unstrengthened RC beam and external FRP strengthened RC beam (Yu and Kodur, 2014)

uniform thicknesses of 20-100 mm applied in a U-shaped manner along the length of the beams. Initially the beams were loaded only up to a service load (which depending on the FRP type varied between 37-54% of their ultimate load) and later were then subjected to the ISO 834 fire. All beams were able to take the applied load without failure for 2 h under fire exposure. The adhesive T_g was exceeded initially but it did not have any visible effect on the system bonding.

Burke, (2008)tested thirteen reinforced concrete slab specimens, 11 of which were flexurally strengthened with a single strip of CFRP tape NSM strengthening system with adhesive system (either epoxy or cementitious grout). Six of the slab strips were tested at room temperature under monotonic load to failure and rest seven under sustained load with increasing surface temperature till failure. For these tests, the beams were loaded at room temperature initially under load control at a rate of 2 kN/min till 20kN. After reaching 20 kN load the heating commenced using a heat blanket up to a temperature of either 100ºC or 200ºC.

The test concluded that the epoxy adhesive used provided superior bond performance as compared to the cementitious grout adhesive used for NSM-FRP strengthening systems for RC members. It may be possible for NSM strengthened FRP systems to endure fire for several hours even for the cases when the FRP needsenough strength to resist service loads. The test implied further conclusions such as the performance at high temperature of NSM CFRP strengthening systems can be improved considerably by using a cementitious grout adhesive and also the grout adhesive used was capable of withstanding 4 hours of fire.

Discussion

Fire Resistance of FRP-strengthened RC Members

From various studies, it is seen that the loss of effectiveness of the strengthening system does not entirely cause failure of the strengthened member under a sustained service load. Even if the pre-existing structural element is over-designed in terms of ûre resistance and lower strength increase is provided by the FRP system, the member may still be able to carry the applied load for the required duration (Firmo *et al*., 2015). Also, in most of the investigations described above, the FRP systems provided low-strength increase comparatively, which explains the long duration of fire endurance obtained in many cases. When supplementary fire insulation was applied to the FRP systems it delayed the loss of effectiveness of the FRP in some cases. Also, the load-carrying capacity of the pre-existing member remained preserved for even longer durations of fire exposure. Note that the fire protection also provides thermal insulation to both concrete as well as the steel reinforcement.

Relative Performance of EBR and NSM Systems

Firmo *et al*. (2012)concluded that the NSM strengthening systems comprising CFRP strips provides superior retention to RC beams under exposure to ûre. For similar insulation schemes, not only the critical temperatures in the anchorages but also the fire endurance obtained are considerably higher with an NSM system. The credit for this behavior goes to (i) the thermal insulation provided by the NSM's partial embedment in concrete for the mechanical confinement conferred by the surrounding concrete and (ii) the better bonding performance. A comparison between the fire performance of EBR and NSM strengthening systems as given in (Firmo *et al*., 2015) is presented in Fig. 9 a) average temperature along the anchorage length; b) ûre resistance of the strengthening system.

Design Guidelines

The guidelines which are currently prevalent for fire design of FRP-strengthened RC beams, which is based on standard fire tests, specifies the use of fire insulation materials as a method to achieve desired fire resistance. Provisions in current codes and standards recommend that the FRP strengthening system should be considered completely ineffective during the event of a fire. Based on test results and parametric studies, the following guidelines are recommended for enhancing fire response of FRP-strengthened RC beams.

General Fire Design Recommendations

Both FRP and polymer adhesives lose their strengthening properties at elevated temperatures. Thus, the available guidelines advise special provisions are needed for FRP design. If no special provisions are taken, the strengthening system will quickly lose its contribution to overall strength during a fire. For no fire protection to be applied, the element must meet the fire resistance rating required all by itself.

The structural elements which can carry the design loads without any FRP strengthening or reinforcement can be exempt from load capacity checks in fire design situations. Since, in modern RC construction, structures are designed with adequate redundancy.

Structural elements which are not critical in the collapse prevention mechanism of a building should be identified and designed for lower fire ratings in order to achieve a cost-efficient structural fire design.

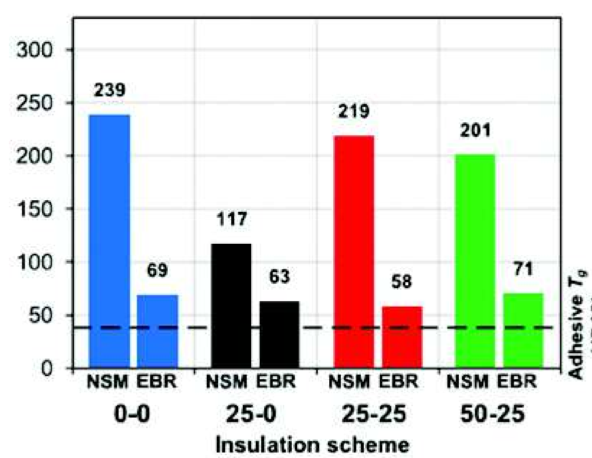

Fig. 9A: Average temperature along the anchorage length versus insulation thickness in mm (Firmo *et al*., 2015)

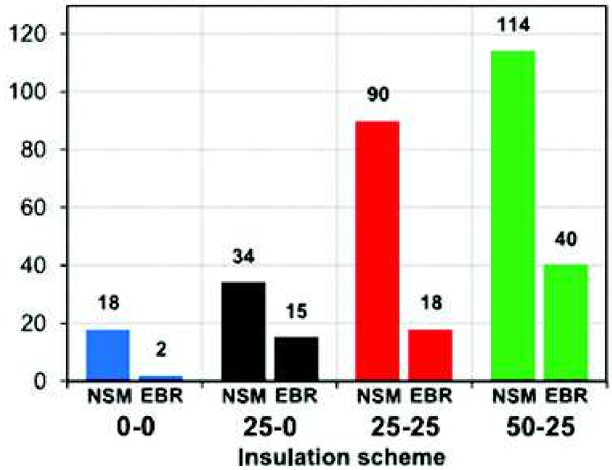

Fig. 9B: Average failure time versus insulation thickness in mm (Firmo *et al*., 2015)

The insulation/cover thickness should be calculated only for structural elements critical in fire design according to the required fire resistance rating. It should account for the minimal rise in the temperature of FRP materials (up to 100-200°C) as these will be rendered ineffective during a fire.

On large scale construction projects where a sound fire design is important, the insulation/cover thickness must be obtained by 3D thermal analysis. This method may need longer calculation times but can lead to an economic design by optimization of the cover/insulation (Maraveas *et al*., 2012).

FRP Strengthening System and Insulation

The current fire design philosophy which was proposed by ACI Committee 4402 suggests that the strengthening system should be designed in such a way that the initial (prior to FRP strengthening) nominal strength of the member stand sufficient to carry the load during thefire. This approach exempts the contribution of FRP strengthening at elevated temperatures. In other words, in the event of a fire, the influence of the EBR systems should be presumed to be lost unless it can be established that the FRP temperature remains below the lowest T_g of the components of the strengthening system.

Effect of the FRP strengthening system in a fire situation is accounted only when insulation systems are in use. Fib bulletin 14, (2001)recommends that the fire resistance should be determined using a "refined calculation method" which is based on a thermal analysis and then subsequently by mechanical analysis. It further recommends that the protection layer be kept under the required dimensions by keeping the adhesive within a certain limited temperature like between 50°C and 100°C. A minimum insulation of 25mm thickness to avoid debonding of FRP can be used for most applications, as recommended byYu and Kodur, (2014).

Load Combinations

When the fire insulation materials are not used, fib bulletin 14, (2001) recommends that the safety of the unstrengthened element should be verified by taking accidental load combination of the strengthened element. For this verification, only partial material safety factors ($\gamma_m = 1.0$) are used and the partial load safety coefficients and combination factors defined in Eurocode are taken into account. Such a combination gives

$$(R_n)_{existing} \geq 1.0DL + 0.9\,LL + 0.5\ \mathrm{TL} \tag{1}$$

Where $(R_n)_{existing}$ is the nominal resistance of the unstrengthened member at an elevated temperature, DL is the dead load effect, LL is the permanent live load effect, and TL is the other transient live load effect on the strengthened structure.

A more restrictive recommendation given by ACI 440.2R-08 than the one given above (Eurocode), is that the nominal strength at high temperature must be higher than the strengthened service load on the member defined as follows:

$$R_n \geq 1.0DL + 1.0\ \mathrm{LL} \tag{2}$$

It is recommended by the design codes to limit the amount of strengthening in order to prevent disastrous failure of the concrete member due to the damage of the strengthening system. ACI 440.2R-08 recommends that the capacity of the existing member that is to be strengthened allows resisting a substantial fraction of the future load defined as follows:

$$(\phi R_n) \geq 1.1DL + 0.75\ \mathrm{LL} \tag{3}$$

Where ϕ is the strength reduction factor for ambient conditions. Therefore, only 25% to 100% strengthening is permissible for FRP strengthened members depending on the ratio between live loads and dead loads, and the resistance factors used in the design.

The recent edition of ASCE 7-05 suggests a reduction in load levels by lowering live load factor from 0.5, and taking 1.2 factor for dead load. Under such load levels, an FRP-strengthened beam will have higher fire resistance since the beam can sustain the loads for a longer duration under lower load levels. Therefore, accounting for relevant and realistic load factors through rational fire resistance calculations can yield higher fire resistance. Thus, this factor should be considered for realistic fire safety assessment of FRP-strengthened RC beams.

Conclusions

Although many studies have been conducted in this area, the information for fire performance of FRP structures is limited, especially under realistic fire and loading conditions. In this study, it is seen that if the structural element is over-designed in terms of fire resistance and the strength increase due to the FRP system is low, the structural member may still be able to carry the service load.

Strengthening of structures with NSM FRP reinforcement technique offers more significant advantages over the technique of strengthening structures with externally bonded FRP reinforcement. Also, coating the FRP materials in case of externally bonded systems with fire insulation materials can significantly improve the fire performance of the structure but may lead to increased costs. There is also a need to develop an easier method to calculate to design fire resistance and thickness of insulation material of FRP strengthened member.

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