# Fiber-Reinforced Polymer-Strengthened/Reinforced Concrete Structures Exposed to Fire: A Review

**Chrysanthos Maraveas,** Technical Director; **Konstantinos Miamis, Apostolos A. Vrakas,** C. Maraveas Partnership, Structural Engineering Department, Athens, Greece. Contact: c.maraveas@maraveas.gr

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## Abstract

Fire resistance of concrete members reinforced or strengthened with fiber-reinforced polymer (FRP) materials is an extremely crucial area that needs to be investigated before implementing FRP composites in buildings and other structures vulnerable to fire. A review on the fire performance of FRP materials and FRP-strengthened/reinforced concrete members is presented. The review includes the mechanical properties of various types of FRP materials at elevated temperatures (based on experimental results). The behavior of bond adhesive epoxies and the bond of FRP reinforcement are investigated under elevated temperature effects. Experimental (fire test) results and numerical studies for unprotected concrete members strengthened or reinforced with FRPs (beams, columns and slabs), including failure patterns, as well as respective results for insulated concrete members, together with the type of insulation materials are also presented. The measured evolution of temperature at distinct locations within the cross section of the tested element and the role of insulation are discussed. Factors that significantly influence the fire resistance of FRP-strengthened or FRP-reinforced systems are investigated and preliminary guidelines for the efficient design of such systems in a fire environment are provided.

**Keywords:** elevated temperatures; fire; fiber-reinforced polymers; strengthening; reinforcing; concrete members.

## Introduction

The retrofitting of damaged structures or the rehabilitation of old construction in an efficient and cost-effective way has always been a challenge for the civil engineering community. Recent research has indicated that the use of new construction materials and techniques may provide viable solutions regarding this issue. A state-of-the-art technology that has seen an increasing number of field applications during the past two decades involves the use of fiber-reinforced polymers (FRPs) in the retrofit of reinforced concrete (RC) members. The development of relevant codes and specifications[1–6] verifies this observation. Characteristic field applications are a 90 000-m$^2$ three-story parking garage in Pittsburgh, Pennsylvania, USA,[7] in which the double tee beams were strengthened with FRPs, and the reinforcing of six cement silos with FRP bars in Boston, Massachusetts, USA.[7] Common practice suggests that FRPs be used either in the form of reinforcing bars, placed inside the concrete section (reinforcing with FRP materials), or as sheets/plates, placed around the member in the form of externally bonded reinforcement (strengthening with FRP materials). Epoxy resins have been used as adhesives to provide sufficient bond between FRPs and concrete surface.

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This newly developed technology provides a wide range of advantages over traditional rehabilitation methods. Most notable are the ease of installation, fast on-field construction and limitation of the disturbance due to works. Moreover, the high resistance to corrosion, certain mechanical characteristics (high strength and durability) and the lightweight properties of FRP materials explain their rapidly increasing use, contrary to conventional materials. However, their low resistance to elevated temperature effects has limited their application to structures in which they are not expected to experience severe temperature rise (e.g. bridges and naturally ventilated parking structures). This observation has motivated researchers to investigate the behavior of FRPs subjected to fire. This paper presents a review of the research work done up to date regarding the mechanical properties of FRPs at elevated temperatures, fire experiments on FRP-strengthened/reinforced concrete elements as well as preliminary fire design guidelines.

## Fire Definition

In most experiments presented in this paper, the ASTM E119[8] or the ISO-834[9] standard fire curves were used to evaluate the fire resistance of RC members incorporating FRPs. Both curves are suitable for fire testing of structural elements. In these standard fires, temperature increases rapidly during their early stages and stabilizes at around 1200°C after prolonged exposure (5 h). In very few tests, the fire exposure was not according to these curves. Instead, an arbitrary slow rate of heating was adopted because the target temperature was low.

## Properties of FRP Materials at Elevated Temperatures

### *Thermal Properties at Elevated Temperatures*

Some research regarding the thermal properties of FRP materials at elevated temperatures has been done in recent years. However, raw experimental data are very scarce. In one source,[10] the change in specific heat and thermal conductivity of heated carbon (CFRP) specimens with aerospace applications is given for temperatures up to 1000°C. Similar plots (for temperatures up to 500°C) for glass (GFRP) bars have also been presented by others.[11] Another experimental curve showing the variation of thermal conductivity with temperature (from ambient conditions to 720°C) has been reported in the literature.[12] The same study provides some data on specific heat but for a very limited temperature range

(150–300°C). These are not given here. The collected information is presented graphically in *Fig. 1*.

### *Mechanical Properties at Elevated Temperatures*

During the past two decades, numerous researchers have investigated the effect of elevated temperature on the mechanical properties of FRP materials. The major focus of recent and ongoing research related to FRPs is the variation of their ultimate tensile strength and modulus of elasticity with temperature. Attention has also been given to the reduction in the bond strength between FRPs and concrete at elevated temperatures. The most common FRP applications involve materials such as CFRP, GFRP and aramid (AFRP). Before presenting the various mechanical properties of FRPs under fire effects, it is important to define the glass transition temperature $T_g$, in which the matrix of the FRP reduces in stiffness and strength by transforming into a soft, rubbery material.[13] This temperature marks a significant increase in the viscosity of the material and is usually measured by differential scanning calorimetry ($T_g$ is the temperature around which the polymer matrix undergoes a change in its specific heat capacity) or from dilatometric data (measurement of the thermal strain and determination of $T_g$ as the temperature that marks a change in the coefficient of thermal expansion). A summary of the collected data regarding the mechanical properties of FRP materials at elevated temperatures, together with their sources (some references contain results from more than one study), is given in *Table 1*. Tests on

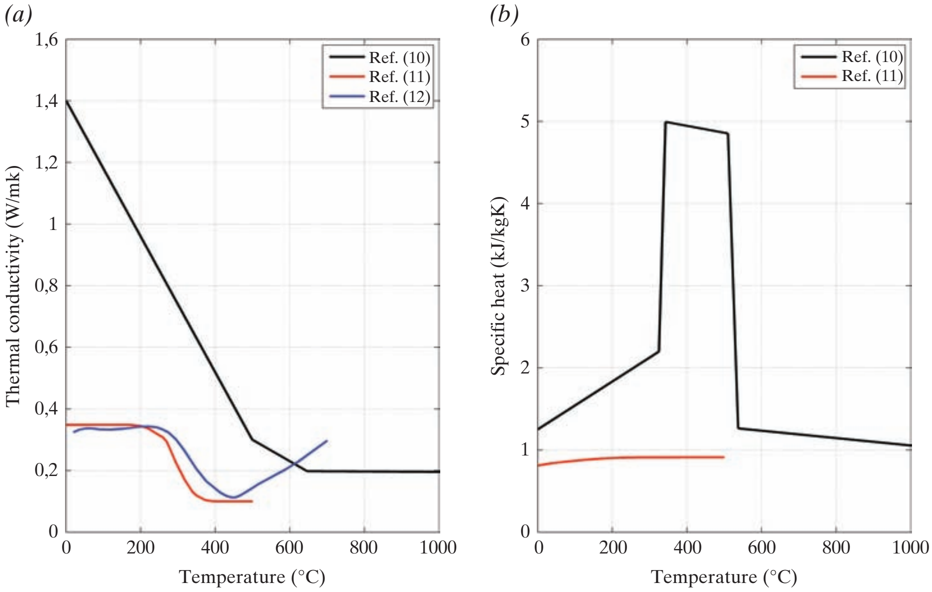


*Fig. 1: Variation of (a) thermal conductivity and (b) specific heat of FRP materials with temperature*

| Reference | Type | Tensile strength | | | | Elastic modulus | | | | No. of sources |
|---|---|---|---|---|---|---|---|---|---|---|
| | | No. of tests | $T_{min}$ (°C) | $T_{max}$ (°C) | $T_{inc}$ (°C) | No. of tests | $T_{min}$ (°C) | $T_{max}$ (°C) | $T_{inc}$ (°C) | |
| [10] | CFRP | 101 | 0 | 450 | — | 75 | 0 | 400 | — | 15 |
| | GFRP | 47 | 0 | 400 | — | 15 | 20 | 350 | — | |
| | AFRP | 38 | 20 | 400 | — | 32 | 20 | 350 | — | |
| [14] | GFRP | 24 | 20 | 500 | 100 | 17 | 20 | 100 | | From author |
| | CFRP | 10 | 20 | 600 | 150 | 7 | 20 | 150 | | |
| [15] | CFRP | 35 | 16 | 200 | 40 | — | — | — | — | From author |
| [16] | CFRP | 24 | 50 | 700 | 50 | — | — | — | — | From author |
| [17] | CFRP | 12 | 20 | 200 | 50 | 12 | 20 | 200 | 50 | From author |
| [18] | GFRP | 30 | 10 | 500 | 50 | 19 | 10 | 300 | 50 | From author |
| [19] | GFRP | 14 | 20 | 350 | 50 | 14 | 20 | 350 | 50 | From author |
| [20] | GFRP | 32 | 200 | 350 | 50 | 23 | 200 | 350 | 50 | From author |
| [21] | GFRP | 8 | 25 | 325 | 50 | — | — | — | — | From author |
| [22] | GFRP | 35 | 5 | 270 | 20 | — | — | — | — | 1 |
| [23] | GFRP | 30 | 20 | 200 | 40 | 30 | 20 | 200 | 40 | From author |
| [24] | GFRP | 56 | 20 | 400 | — | 6 | 20 | 250 | — | 2 |
| | AFRP | 24 | 20 | 400 | — | 24 | 50 | 300 | — | |
| | CFRP | 47 | 20 | 450 | — | 31 | 20 | 300 | — | |
| | | **Bond strength** | | | | **Residual bond strength** | | | | |
| [25] | Bond | 53 | 20 | 280 | — | — | — | — | — | 3 |
| [26] | Bond | 10 | 20 | 200 | 50 | — | — | — | — | From author |
| [27] | Bond | — | — | — | — | 70 | 20 | 335 | 20 | From author |
| [28] | Bond | — | — | — | — | 6 | 20 | 200 | 100 | From author |
| [29] | Bond | — | — | — | — | 8 | 20 | 80 | 20 | From author |
| | | **Tensile strength** | | | | **Residual tensile strength** | | | | |
| [27] | Epoxy | — | — | — | — | 48 | 20 | 250 | 50 | From author |
| [30] | Epoxy | 36 | 20 | 175 | 25 | — | — | — | — | From author |

*Table 1: Mechanical properties of FRP materials at elevated temperatures: summary of the experimental data found in the literature*

mechanical properties were carried out at steady-state conditions. *Table 1* also includes the minimum ($T_{min}$) and maximum ($T_{max}$) temperature at which specimens were exposed in every experimental program. The increase in temperature exposure ($T_{inc}$) among consecutive tests is also given, when information is available.

### *Tensile Strength and Modulus of Elasticity of FRPs at Elevated Temperatures*

The tensile strength of FRP materials is crucial when determining the load bearing capacity of FRP-reinforced/strengthened members. Experimental data showing the reduction in tensile strength of CFRP, GFRP and AFRP with temperature from 15 sources have been reported in the literature.[10] In an attempt to evaluate the response of FRP-RC members subjected to fire, researchers[14] measured the tensile strength of CFRP and GFRP rods at elevated temperatures up to 700°C. The ultimate tensile strength of CFRP sheets at elevated temperatures has also been measured.[15–17] Researchers have also investigated the effect of temperature on the tensile strength of GFRP rebars[18–22] and GFRP sheets.[23] Furthermore, relevant experimental data on GFRP, CFRP and AFRP materials from another two sources have been presented.[24] *Figure 2* summarizes the above-mentioned results for the three studied FRP materials. In all graphs, the range of $T_g$ is presented, according to information given in the relevant sources, along with its mean value (dotted line). It should be noted that $T_g$ for AFRP materials is based on reported values relating to aramid fibers.[24]

The modulus of elasticity of FRP materials plays an important role in the design, as it determines the stiffness and deflection characteristics of FRP-reinforced/strengthened members. Its variation with temperature has also been studied by a wide range of researchers. Experimental results regarding its variation in CFRP,[10,14,17,24] GFRP[10,14,18–20,23] and AFRP[10,24] specimens have been reported meticulously in the literature. *Table* 1 shows the tested temperature range in the different experimental series. A relevant plot of the assembled data is also presented in *Fig. 2*.

### *Bond Strength of FRPs and Tensile Strength of Epoxy Resins at Elevated Temperatures*

Temperatures can compromise the bond between the concrete and FRP, rendering the FRP strengthening/reinforcing of the member ineffective. In a relevant literature review,[25] data from three sources showing the degradation of the bond strength between FRP bars and concrete with temperature are presented. Researchers[26] also measured the bond strength of CFRP wires embedded in concrete at elevated temperatures and reported the results. Other researchers[27] carried out bond pull-off (direct tension) tests on concrete specimens strengthened with CFRP and GFRP sheets after being heated to temperatures up to 350°C and measured the residual bond strength between the two materials. In a similar experimental work,[28] the tested specimens were exposed to temperatures up to 200°C. The residual bond strength between GFRP bars and concrete has also been investigated[29] through pull-out testing of FRP-reinforced specimens heated up to 80°C. A synopsis of the gathered experimental data is presented in *Fig. 3*.

In concrete members externally strengthened with FRPs, FRP materials are bonded to the concrete with adhesive resins, such as epoxy. A sound fire-resistant design should account for possible reductions in the tensile strength of the adhesive material with increasing temperature, as this could lead to

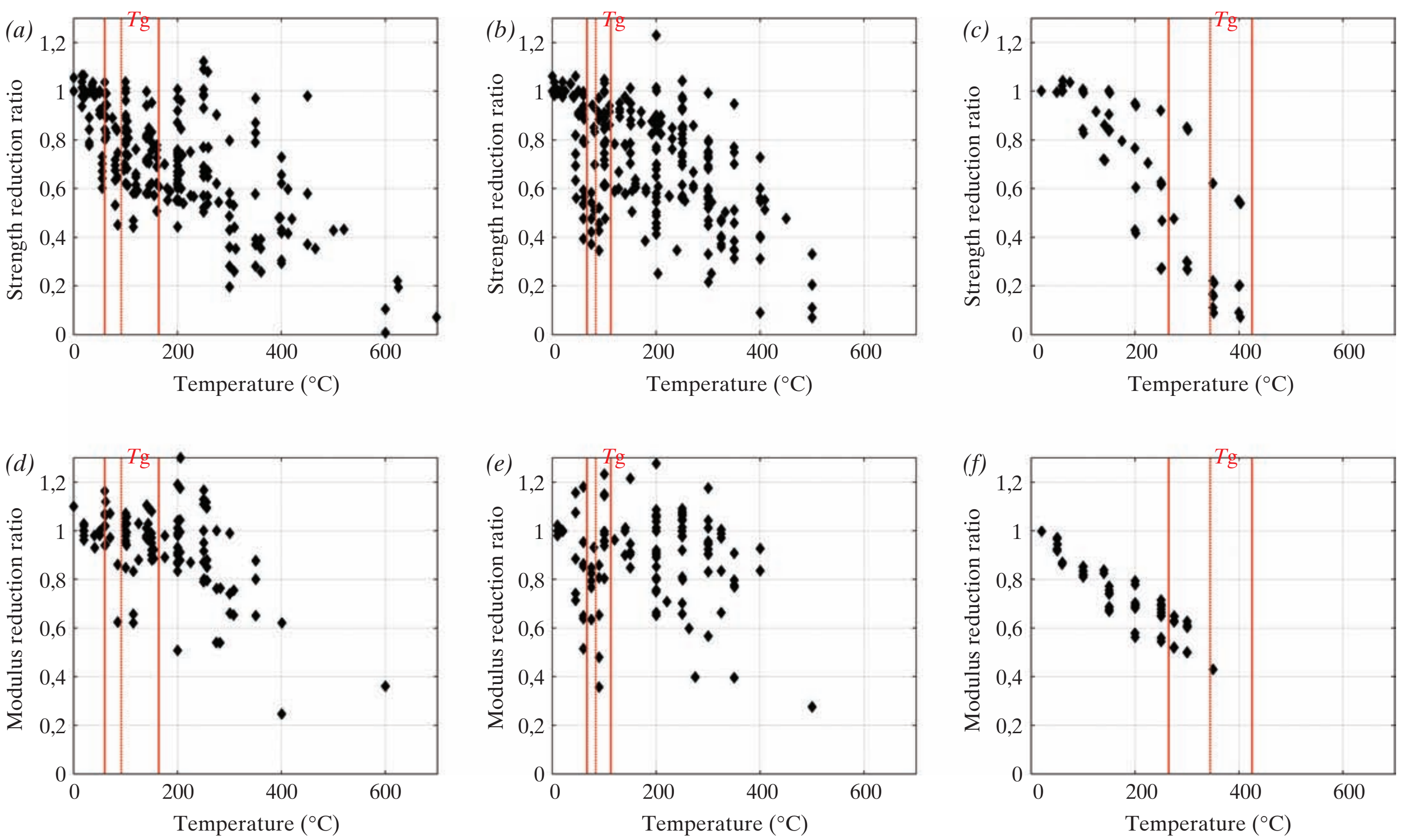


*Fig. 2: Variation of (a) CFRP, (b) GFRP and (c) AFRP ultimate tensile strength with temperature and variation of (d) CFRP, (e) GFRP and (f) AFRP modulus of elasticity with temperature*

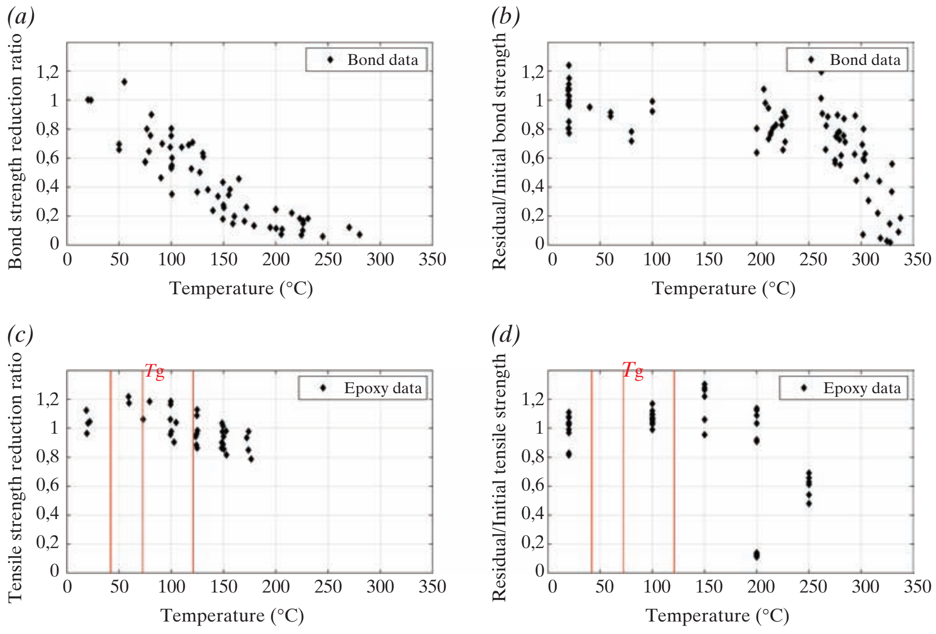


*Fig. 3: Variation of (a) bond strength and (b) residual bond strength of FRPs and concrete with temperature and variation of (c) tensile strength and (d) residual tensile strength of epoxies with temperature*

complete loss of bond between FRP and concrete. Tensile tests of epoxy coupons at elevated temperatures up to 170°C have been reported in the literature[30] (*Fig. 3c*). Other studies presented experimental data[27] on the residual strength of epoxy specimens heated up to 250°C (*Fig. 3d*).

### Discussion and Research Needs

Contrary to the mechanical properties of FRPs, their thermal properties at elevated temperatures have not been studied thoroughly, because FRP applications usually have small thickness and are not expected to alter the thermal response of the overall cross section. The curves proposed in the literature display considerable differences. The sudden rise in specific heat between approximately 300 and 520°C reported for CFRPs is not observed for the remaining data. The steep reduction in the thermal conductivity of CFRP specimens at low temperatures (up to 400°C) is not observed for the tested GFRPs, in which a decrease is observed approximately between 250 and 400°C. These discrepancies can possibly be attributed to the different nature of the carbon and glass fibers, but further research on the thermal properties of FRPs is required before drawing solid conclusions.

Most experiments regarding the mechanical properties of FRPs have been conducted for the temperature range of 20–600 or 20–500°C. Some researchers have limited their experimental investigation to temperatures as low as 200°C (*Table* 1). The test data confirm the expected degradation of the tensile strength with temperature. The majority of the reported results fall within 50–400°C. For temperatures lower than 200°C, the reduction ratio generally varies from 1 to 0,55 for CFRPs, from 1 to 0,40 for GFRPs and from 1 to 0,70 for AFRPs. Because of the variability in the data, the effect of the glass transition temperature cannot be clearly determined. However, it marks, in most cases, the initiation of the tensile strength degradation for the tested FRP materials. The tensile strength reduces further for the temperature range of 200–400°C, with reductions as high as 80% (around 300°C) being reported. This observation raises doubts regarding the capability of FRPs to maintain their design load in this temperature range. The limited experimental results for temperatures >500°C highlight the need for further testing in that temperature region. Such tests could also provide information on the temperature that causes complete loss of strength, which is up to date not clearly determined. The large scatter in the data can be attributed to the high variability in the composition of the polymer matrix and fibers of the tested specimens. The high strength of carbon fibers at elevated temperatures is not reflected on the retained strength of heated CFRPs, as the properties of the polymer matrix usually dictate the reduction in the mechanical properties.[25]

The effect of temperature on the modulus of elasticity of FRPs is not so profound. With the exception of very few data points, the elastic modulus does not reduce below 60% of its initial value when heated up to 400°C. Especially for low temperatures (below 250°C), the stiffness of CFRP and GFRP materials may, in some cases, increase up to 20% with respect to its room temperature value. Furthermore, the glass transition temperature does not indicate a major change in the elastic modulus of the tested CFRP specimens, while large scatter in GFRP data is observed around that temperature. The $T_g$ of aramid fibers (as shown in *Fig. 2c and f*) has been reported to fall within the region of 300°C, where considerable deterioration of the AFRP material has already occurred. The data once again display considerable scatter, except for AFRP materials where a steady decline with temperature can be observed. This could possibly be attributed to the nature of aramid. However, the experimental results found in the literature are extremely limited for temperatures higher than 400°C, suggesting that further research work is needed to evaluate the variation of the modulus of elasticity beyond 400°C.

Relevant data (*Fig. 3a*) confirm rapid loss of bond strength even at temperatures as low as 100–200°C.[25] This phenomenon occurs due to changes in the microstructure of the polymer matrix of FRPs[25] at their interface with concrete, as fibers are more resistant to elevated temperatures.[26] The data do not exhibit large scatter and show a reduction at a steady rate. The reported results suggest an average reduction of 40% at 100°C, with the bond strength dropping below 20% of its initial value around 200°C. Therefore, the force transfer mechanism between FRP and concrete can be seriously weakened for this temperature region, an observation that poses serious design considerations. The post-fire bond strength, however, experienced a less severe reduction for the temperature range of 200–300°C, with measurements showing that 60–100% of the initial bond strength was maintained. Low temperatures have a minor effect on the bond strength (approximately 15% reduction) when the specimen is allowed to cool down. The gathered data suggest that, practically, there is no bond strength remaining beyond 300°C. Moreover, the experimental data are limited to FRP-reinforcing schemes, with no attention being given to externally bonded systems. The need for a thorough experimental investigation on this research object has already been reported.[25]

Experimental data on the tensile strength of epoxy resins are few and limited to temperatures lower than 170°C. Below 100°C, the data show an increase in the ambient temperature strength up to 20%. On the contrary, minimal reduction in strength (approximately 15%) is observed for temperatures ranging between 100 and 170°C. The variation of tensile strength for higher temperatures should be investigated by testing. The results will also provide crucial information on the temperature region around which epoxies cannot provide sufficient bond (due to complete loss of strength). Reported data on the residual strength of epoxy specimens heated up to 150°C show approximately the same variation with temperature. When specimens were exposed to 250°C, their strength after cooling down reduced to 50–70% of its initial value. Results referring to higher temperatures should also be reported in future work. The reported $T_g$ range does not seem to affect the tensile ("hot" or residual) strength of epoxies.

Information regarding the post-fire mechanical properties of FRP materials is extremely limited. Researchers[31] measured the residual strength of CFRP and GFRP coupons (sheets) after high-temperature exposure. Further research work is required, however, for useful conclusions regarding the post-fire mechanical properties of FRPs to be drawn.

## Thermal Response of Members Incorporating FRPs

Another important issue that should be discussed is the temperature evolution in members that incorporate FRP materials. A clear distinction is made between FRP-reinforced and FRP-strengthened elements, because their thermal response is not expected to be similar.

### *FRP-Reinforced Elements*

*Experimental Studies*

In most experiments, the temperature at distinct locations of the tested member was monitored by thermocouples and reported as a function of time. For example, Ref. [32] provides a temperature profile for a section at the midspan of a representative CFRP-reinforced beam that was tested (*Fig. 4a*). Similar information regarding another beam has been presented in an experimental program elsewhere.[33] In another research project,[34] two concrete members with GFRP reinforcement were tested and the temperature evolution in the rebars was measured.

Time–temperature curves are reported for six slabs reinforced with GFRPs.[11] Measurements of the thermal field at different locations and the temperature variation along the depth of section are included in the same study (*Fig. 4b*). A similar experimental program has been reported in the literature,[35] providing, however, little information on the thermal response of the specimens. Detailed experimental data are given by other researchers,[36] who recorded the temperature evolution at several locations and cross-sectional depths of the tested CFRP and GFRP slabs (*Fig. 4c*).

## Numerical Analysis Studies

Several researchers attempted to simulate the thermal response of FRP-RC elements. In beam elements, a semi-empirical equation for calculating the temperature at the FRP rebar–concrete interface has been proposed.[37]

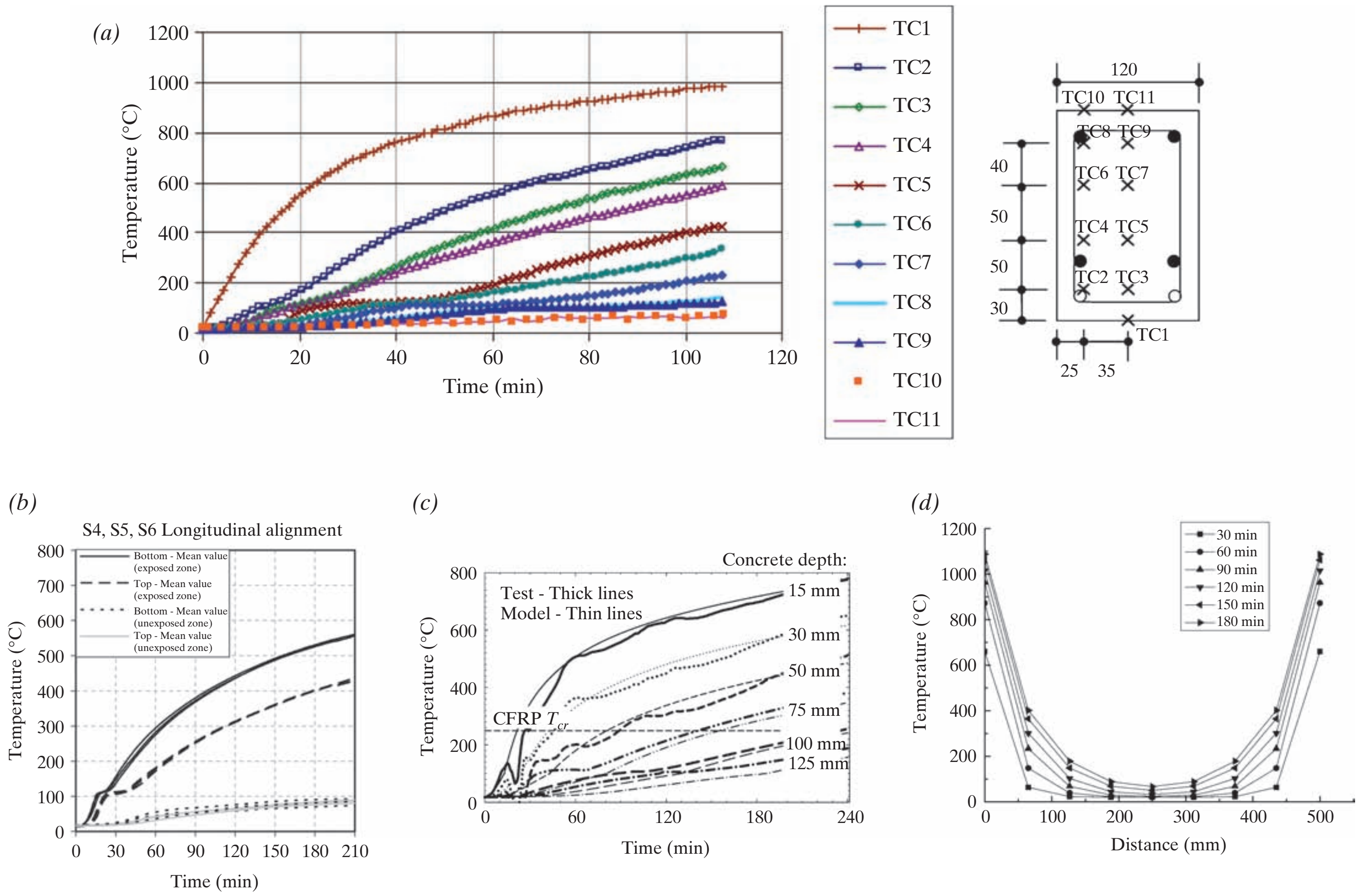


*Fig. 4: Temperature evolution in FRP-reinforced (a) beams,[32] (b) slabs,[11] (c) slabs[36] and (d) columns[38]*

The same study[37] also proposed a two-dimensional (2D) finite element model to predict the temperature distribution along the depth of the beam's cross section and compared the results with data from a relevant experimental program.[34]

The temperature profile of FRP-reinforced columns (*Fig. 4d*) heated from all sides has also been simulated using the finite element method (FEM).[38] The analysis involved three-dimensional (3D) models incorporating eight-node solid elements. The mechanism of heat transfer to the surface of the column included both radiation and convection. The influence of concrete cover on the temperature evolution of the FRP bars was also investigated. Similar work was also reported by others.[39]

Another study[40] created a one-dimensional (1D) heat transfer model (based on the explicit finite difference method) to determine the temperature profile in slabs heated from below and compared the numerical results with experimental data.[36] In this model, the influence of FRP reinforcement on the heat transfer mechanism was ignored. In other cases, 2D solid elements were employed to perform a heat transfer analysis of the cross sections of slabs.[11]

*Discussion and Research Needs*

Experimental results describing the thermal response of FRP-RC members found in the literature are scarce. No data referring to FRP-reinforced columns exist and, therefore, relevant experiments should be carried out to determine their temperature profile when exposed to fire. The experimental results on beams confirm the expected decrease in temperature toward the interior of the cross section and the severe temperature gradient between top and bottom fiber.[32] More importantly, they show that the temperature at the FRP bars might exceed 600°C. This observation is crucial for the design, as it suggests loss of the strength of the bars and structural failure. Moreover, it highlights the significance of cover. More testing of beams with varying cover thicknesses should be performed for its role in the thermal response to be quantified. Future work should also include monitoring the temperature throughout the length of the member. Measurements of the thermal field in slabs also show the existence of a temperature gradient between top and bottom surface. The rebars display once again considerable temperature rise (more than 500°C), suggesting that the used cover was not sufficient. Its importance in the fire resistance of slabs with FRP reinforcement has already been emphasized.[36] Moreover, the experimental studies show that temperature evolution in the slab does not depend on the nature of the FPR material (*Fig. 4*).

Even though reasonable, numerical analysis results referring to columns with FRP reinforcement should be verified by testing. The 2D finite element analysis model found in the literature[37] was proven to underestimate the temperature evolution in FRP-reinforced beams.[37] On the contrary, the semi-empirical equation proposed in the same study matches the test results accurately.[37] More work is required, however, for solid conclusions to be drawn. Numerical results regarding slabs follow the experimental data very closely. Some improvement in the derived 1D heat transfer model[40] could be achieved by taking into account the presence of the FRP bars. Furthermore, all proposed numerical analyses have to be compared with extensive testing (existing and future) from different sources for their validity to be confirmed.

## FRP-Strengthened Elements

FRP materials are well suited for repairing or strengthening concrete members. Especially in applications where the material is wrapped around a column, it can provide adequate confinement and, therefore, enhance the behavior of the member in terms of ductility and strength. This type of application, however, is extremely susceptible to fire, as the FRP material is directly exposed to elevated temperatures. For this reason, in practice, FRP-strengthened elements are commonly insulated via sprayed-on applications or boards of insulating materials. Characteristic materials commonly used as fire protection and their thermal properties are given in the literature.[41,42]

*Experimental Studies*

FRP-strengthened members subjected to fire are usually insulated to prevent the fast temperature rise in the FRP wrap. In a relevant experimental project,[43] a total of 18 such specimens (six columns and 12 beams) were tested and the temperature evolution, on both sides of the insulating surface, was recorded. Temperature measurements of FRP-strengthened columns with insulation exposed to a furnace temperature of 900°C for 30 min have also been reported.[44] The effect of fire protection in reducing temperature has also been noted in experiments by other researchers,[45] who tested four insulated and one unprotected FRP-strengthened columns (*Fig. 5a*). More detailed information regarding the temperature profiles for three of these columns is given elsewhere.[46]

Others[47] recorded the evolution of temperature at the midspan of insulated

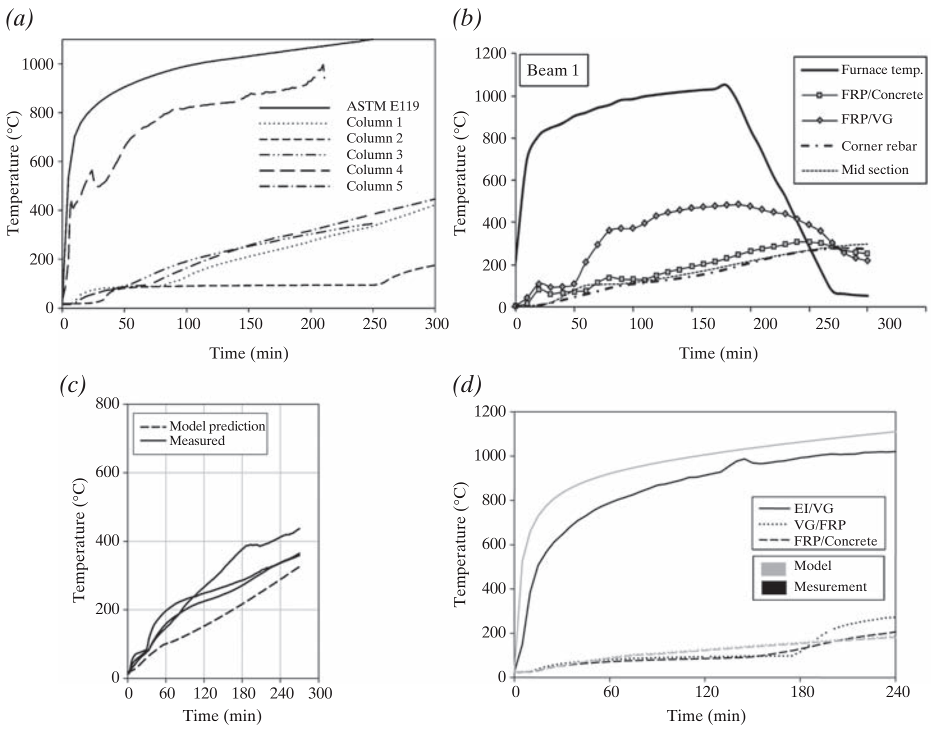


*Fig. 5: Temperature evolution in FRP-strengthened (a) columns,[45] (b) beams,[47] (c) beams[50] and (d) slabs[53]*

beams (*Fig. 5b*). Significant work on the thermal response of FRP-strengthened beams with fire protection has also been done in other research programs.[41,48–51] Temperature measurements at various cross-sectional depths and positions were reported. Temperature profiles of unprotected beams are also given in the literature.[41,52]

The thermal field of FRP-strengthened slabs has also been investigated experimentally.[53] In the specific research project, four unloaded slab specimens with varying insulation were exposed to fire and the temperature evolution was monitored at distinct locations. Loss of insulation was reported for one test.[53] Similar work was done for ten CFRP-strengthened slabs[54] and information on the thermal field was given.

*Numerical Analysis Studies*

Analysis of the thermal response of FRP-strengthened members has also been the object of research in recent years. A heat transfer analysis model for circular columns based on the finite difference method has been developed.[55] In this model, the cross section is discretized in circular layers (the existence of reinforcement is neglected) and the temperatures are calculated assuming elemental energy balance. Similar work was done for rectangular FRP-wrapped columns with insulation.[56] A nonlinear finite element procedure to predict the temperature distribution in FRP-wrapped columns subjected to fire has also been proposed.[57]

The FEM has been employed to describe the thermal response of insulated beams with FRP strengthening,[42,47] and information regarding the meshing of the simulated beams and the governing heat transfer equations was published. Information referring to relevant 2D[41,48] as well as 3D heat transfer finite models[49,58] is given in the literature. Others[50] created a 2D heat transfer model incorporating the finite difference method to predict the temperature profile of such beams and compared analysis results with their experimental data (*Fig. 5c*).

An explicit finite difference model based on 1D heat propagation (which includes material variation with temperature) has been derived for modeling the thermal behavior of slabs strengthened with FRP systems.[53] Numerical results from this model have been compared with experimental data[53] (*Fig. 5d*).

*Discussion and Research Needs*

Experimental data confirm the rapid increase in temperature in the FRP for members with no fire protection. *Figure 5a* shows that the temperature increased at a much higher rate for the unprotected column (Column 4).[45] Furthermore, temperatures exceeding 800°C[45,52] in the FRP system have been recorded for nonprotected members. On the contrary, FRP temperature did not exceed 400°C in insulated elements, even after prolonged (more than 5 h in certain cases) exposure to fire. Still, these temperatures are well above $T_g$ (where deterioration of the FRP can be noted). In other research,[41] even though insulation delayed the debonding of CFRP in beams,[41] loss of the strengthening system could not be prevented. Further research is required to determine the effectiveness of the insulating materials and their application thickness. More testing of real-scale members should also be carried out, especially for slabs and columns, as the tests to date are few.

Comparison with experimental data shows that the cross-sectional temperature distribution of FRP-confined columns can be adequately described by the current thermo-numerical models. However, future work should include numerical studies of 3D FEM column models to address issues of heat propagation along the height of the member. Numerical results referring to beams generally follow the measured temperatures. All proposed approaches (2D and 3D FEM analyses and 2D finite difference method) describe the heat propagation problem well and capture the effect of insulation in reducing the temperature of the FPR layer accurately. Numerical studies regarding the thermal response of FRP-strengthened slabs are limited. Even though test data are in agreement with numerical results, further testing is required to confirm their validity. 3D FEM models might also be appropriate to simulate the spatial variation of temperature in slabs strengthened with FRPs. Moreover, comparison of the current numerical results should be extended to a wider range of experimental data (including future work).

## Structural Response of Members Incorporating FRPs

The need to provide adequate fire resistance and establish a threshold temperature, under which an FRP-strengthened/reinforced structural system can sustain service load, has motivated researchers to carry out fire experiments on concrete members (i.e. beams, columns and slabs) incorporating FRP materials. The relevant tests found in the literature are summarized in *Table 2*.

In an attempt to describe the structural response of RC members incorporating FRPs, the load ratio of each member (defined as the applied load divided by the member capacity at room temperature) is plotted against the temperature at failure. The reported failure temperatures refer to the FRP–concrete interface at the middle section of the element. Some researchers measured the ultimate load for the tested specimens by loading

| Reference | Type | No. of experimental data | | Failure load determination |
|---|---|---|---|---|
| | | FRP-reinforced | FRP-strengthened | |
| [28] | Columns | — | 12 | Experiment |
| [45] | Columns | — | 5 | Per ACI 440[2] |
| [59] | Columns | — | 2 | Per ACI 440[2] |
| [60] | Columns | — | 3 | Per ACI 440[2] |
| [61] | Columns | — | 4 | Per ACI 440[2] |
| [44] | Columns | — | 10 | Experiment |
| [62] | Columns | — | 2 | Per ACI 440[2] |
| [32] | Beams | 5 | — | Per ACI 440[2] |
| [41] | Beams | — | 5 | Per ACI 440[2] |
| [34] | Beams | 2 | — | Per ACI 440[2] |
| [48] | Beams | — | 1 | Per ACI 440[2] |
| [63] | Beams | — | 1 | Per ACI 440[2] |
| [11,64] | Slabs | 6 | — | Experiment |
| [35] | Slabs | 3 | — | Per ACI 440[2] |
| [65] | Slabs | — | 7 | Experiment |
| [54] | Slabs | — | 5 | Per ACI 440[2] |

*Table 2: Fire tests of RC members reinforced/strengthened with FRPs found in the literature*

a control specimen to failure. When such information was not available, it was calculated according to ACI-440[2] (*Table 2*). These plots show the effect of temperature on the load-bearing capacity of FRP-reinforced/strengthened elements exposed to fire. In terms of design, they can be used to assess the survivability of such members in different fire situations or can provide insight regarding the maximum allowable temperature of the FRP materials for a given load level.

*Fig. 7: Failure of FRP-reinforced beam[34]*

### *FRP-Reinforced Elements*

*Experimental Studies*

No tests on FRP-reinforced columns have been reported in the literature. Researchers mainly focus on reinforcing the cross section of beams with FRP bars. In a relevant experimental program,[32] five experiments on FRP-reinforced beams, with the FRP bars being placed at different locations within the cross section, were carried out. In two tests, steel rebars were used in conjunction with the FRP reinforcement. Failure temperatures as well as the time–displacement curves obtained by heating the beams were reported. Beams with GFRP reinforcement have also been tested,[34] with the evolution of deflection, until failure, due to increase in temperature being measured. The load ratio of the tested beams is plotted against the failure temperature (*Fig. 6*).

Most of the tested beams reported in the literature failed in flexure. Such a failure mode, where the beam ruptured at midspan (*Fig. 7*), has been reported.[34] Obvious is also the spalling of concrete at the edges.[34] Other experimental results[32] suggest similar failure patterns.

Another study[40] also refers to a research program that involved ten fire tests of FRP-reinforced slabs and reports that failure did not occur until temperature at the reinforcement had reached 500°C. Furthermore, results from an experimental program comprising six simply supported concrete slabs reinforced with GFRP rebars, which were tested under four point bending in fire conditions, have been reported.[11,64] The ultimate load, fire rating and the failure mode for each slab are provided. Other experimental series[35] involved one-way simply supported slabs reinforced with GFRP bars, which were subjected to two-point loading. Three specimens were heated to 100°C and were loaded to failure after cooling. Detailed information regarding the crack patterns at failure are provided. *Table 3* summarizes the results referring to FRP-reinforced slabs.

| Reference | Slab specimen | Temperature at failure (°C) | Load level at failure | Failure mode |
|---|---|---|---|---|
| [11,64] | S1 | 710 | 0,55 | Pull out |
| [11,64] | S2 | 560 | 0,50 | Pull out |
| [11,64] | S3 | 420 | 0,60 | Pull out |
| [11,64] | S4 | 500 | 1,00 | Bar rupture |
| [11,64] | S5 | 500 | 0,85 | Bar rupture |
| [11,64] | S6 | 500 | 1,00 | Bar rupture |
| [35] | SG 13-6-1B | 100 | 0,68 | Shear |
| [35] | SG 13-6-2B | 100 | 0,66 | Shear |
| [35] | SG 16-4-1B | 100 | 0,75 | Shear |

*Table 3: Experimental results of FRP-reinforced slabs at elevated temperatures*

*Numerical Analysis Studies*

Despite the lack of experimental data, researchers modeled the structural response of FRP-reinforced columns subjected to fire via 3D FEM analyses. In an axially loaded column model,[38] eight-node solid elements with the appropriate (temperature dependent) mechanical properties were used. No slip was allowed in the FPR–concrete interface. The evolution of the top displacement with temperature was calculated. Others[39] simulated the structural response of a heated half-scale column with 3D FEM modeling. Their results included vertical displacement calculation at the top of the column and stress profiles after different fire exposure times. Numerical studies regarding FRP-reinforced slabs are limited to flexural strength calculation of the cross section[66] after fire exposure from below. The proposed methodology involves division of the section into layers. Afterward, an iteration procedure is followed until the moment resistance is calculated.

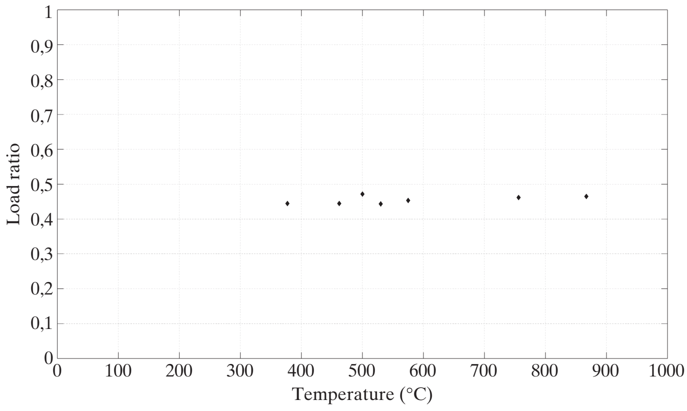


*Fig. 6: Plot of load ratio of tested FRP-reinforced beams versus failure temperature*

*Discussion and Research Needs*

Experimental results show that beams loaded below 50% of their ambient temperature capacity usually fail at temperatures ranging from 500–650°C. High-temperature failures (around 800°C) correspond to heavily FRP-reinforced cross sections, which are not representative of typical FRP reinforcing. Therefore, beams with adequate cover could resist temperature effects under service load (as they are typically designed with a safety factor of two). Moreover, the fire resistance of beams subjected to higher load ratios should be investigated by testing. However, the load ratio might not be sufficient to describe the behavior of concrete beams incorporating FRPs. Other parameters, such as the loading type (point or uniform) and the anchorage length of the FRP reinforcement, may be critical in determining the fire resistance and should be investigated by further testing.

FPR-reinforced columns have not been tested to date. Future experimental work could contribute in the structural fire design of such elements. Fire testing of slabs incorporating FRPs is limited. Despite being able to sustain the applied loads for temperatures ranging between approximately 400 and 700°C, results show that more than one failure modes are possible. Rapture of the FRP bars was only observed when the slabs were subjected to high load ratios and sufficient anchorage length unexposed to fire was provided,[64] while pull-out of the bars occurred for typical service load conditions (load ratio around 50%). Shear failure was observed in another series of experiments,[35] but this should probably be attributed to poor fabrication of the specimens (small width and high longitudinal reinforcement ratio) and did not occur as a result of temperature exposure (temperature at the GFRP bars did not exceed 100°C). Future testing on a wider range of slab specimens is required to ensure proper understanding of their structural response to elevated temperatures.

It should be noted that numerical analysis studies on FRP-reinforced members subjected to fire are few. Despite some FEM modeling pertaining to columns, the absence of numerical analyses of beams is notable. Further numerical investigation of these elements, possibly via the FEM, should be conducted. The analysis of slabs should also involve 3D FEM modeling of full-scale specimens, because spatial temperature distribution might affect the overall structural response of the system. Comparison with relevant experimental data (current and future) is also necessary to assess the validity of numerical methods for FRP-reinforced members.

## FRP-Strengthened Elements

### *Experimental Studies*

Testing of FRP-strengthened elements has been the object of research in current years. More specifically, one square and four circular columns, wrapped externally with FRPs, were tested under fire effects and the failure temperature and load was reported.[45] Also, their design capacity was calculated according to ACI 440.[2] Based on that, the load ratio was calculated. Other researchers[59] conducted a similar experimental program, which involved two circular columns (one of which was insulated), and reported relevant results. More recent research[62] involved fire testing of one rectangular and one circular column with fire protection. Results relating the load ratio and the failure temperature of the studied columns are summarized in *Fig. 8a*.

Other researchers[60,61] measured the residual strength of columns with externally bonded FRPs after being subjected to elevated temperatures. The tests involved three rectangular[60] and four circular columns.[61] In another research program,[44] half-scale CFRP-strengthened columns with insulation were exposed to fire and then loaded to failure. Small-scale (100 mm × 200 mm) concrete cylinders were tested[28] in compression after being heated to 100 or 200°C. *Figure 8b* plots the reduction in strength against the maximum temperature exposure for these experiments. The initial (unheated) strength was calculated according to ACI440,[2] except for the half-scale and small-scale specimens, whose strength at room temperature was measured by compression tests.[28]

FRP strengthening is also suitable for the retrofit of beams. Tests of four CFRP-strengthened RC beams[47] have also been reported in the literature. However, the beams did not reach failure due to the existence of insulation and low loading level (below 50% according to the study). On the contrary, CFRP-strengthened RC beams tested by other researchers[41] were loaded to failure. Relevant experimental results on FRP-strengthened beams are also reported in the literature.[48,63] The increase in midspan deflection of two CFRP-strengthened beams exposed to fire has also been measured.[49] *Figure 8c* shows the correlation between the load ratio and the failure temperature of the studied beams.

Most researchers provide information on the failure modes of FRP-strengthened RC members in their work. In the experimental program[45] discussed earlier in this section, the insulated columns failed in a "non-violent manner by apparent crushing of the concrete core", with the insulation being intact until failure. Concrete spalling and complete loss of cover in certain regions was observed for the unprotected column,[59] with no deformation of the longitudinal and transverse reinforcement being obvious[59] (*Fig. 9a*). *Figure 9b* also shows the failure of uninsulated columns wrapped with FRPs after exposure to elevated temperatures,[60] which shows complete debonding of the fibers at one end. The authors state that failure of the FRP wrap was sudden and accompanied by concrete crushing.[60] This type of failure was confirmed by similar experiments[61] (*Fig. 9c*). These failure patterns resemble the typical failure of axially loaded

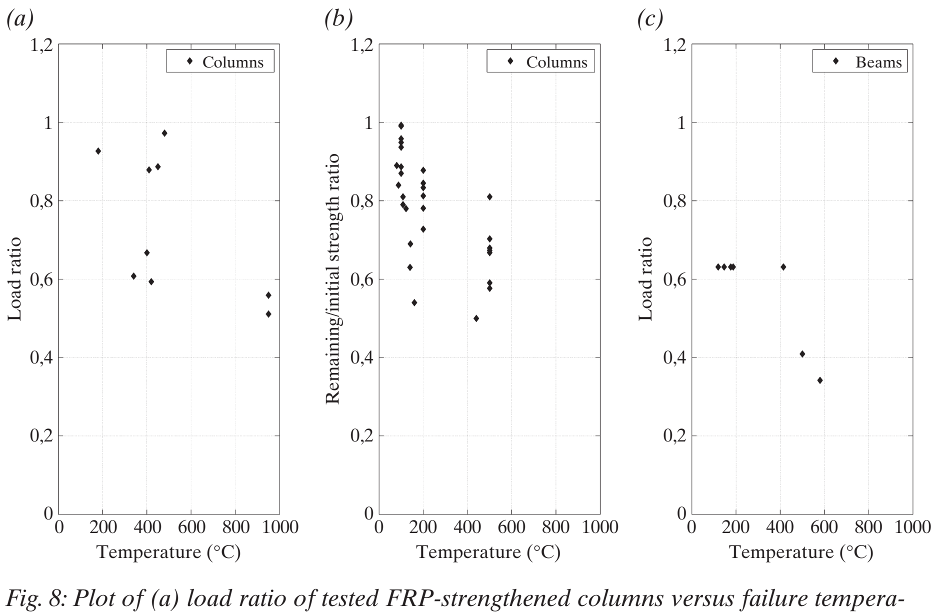


*Fig. 8: Plot of (a) load ratio of tested FRP-strengthened columns versus failure temperature, (b) ratio of residual by initial strength of tested FRP-strengthened columns versus maximum temperature during heating and (c) load ratio of tested FRP-strengthened beams versus failure temperature*



Fig. 9: Failure of (a) FRP-reinforced column,[59] (b) rectangular column with FRP wrap,[60] (c) circular column with FRP wrap[61] and (d) FRP-strengthened beam[41]

Fig. 10: View of the compartment after one of the Dalmarnock fire tests[68]

(nonslender) columns at room temperature, once the FRP was debonded. It should also be noted that slender columns are expected to buckle, while short stocky columns are expected to fail in compression.

In FRP-strengthened beams tested by other researchers,[41] failure (*Fig. 9d*) occurred by tensile rupture of the reinforcement after debonding of the FRP strengthening system.[41] The observed failure modes suggest typical failure of a RC beam at elevated temperatures, after detachment of the FRP strengthening.

FRP strengthening of RC slabs has also been investigated to some extent. A research project including five one-way FRP-strengthened slabs exposed to fire was conducted.[54] The slabs were heated to failure, while structural loading was maintained constant. Detailed results regarding the deflection history of the slabs with temperature increase are given. In other research,[65] near surface mounted strips were bonded to loaded slabs that were heated to failure. Midspan deflection during heating was recorded. *Table 4* summarizes results from both experimental programs, as well as the reported failure modes.

The effect of fire on FRP-strengthened slabs was investigated in two "real" compartment fire tests (the Dalmarnock fire tests).[67,68] The significance of these tests is that they simulated natural fire conditions and showed that loss of bond is the expected failure mode in real fire situations too. In these experiments, the 150-mm concrete slab was strengthened by six strips of externally bonded FRPs, two of which were left unprotected. Despite high gas temperatures in the compartment[68] (750–900°C) after flashover, the temperature in the bondline between the two materials exceeded 300°C only for the unprotected strips.[68] However, fire caused the FRP plates to be completely detached from the concrete surface even when protected by intumescent coating.[67] *Figure* 10 shows a picture of the compartment after fire in one of the tests.

*Numerical Analysis Studies*

Numerical studies regarding the structural response of FRP-strengthened members subjected to fire have also been the topic of recent research. One study[59] presented an analysis method in which the cross section of circular columns is divided into annular elements. Once the temperature-dependent mechanical properties have been selected (according to the thermal profile of the section), the axial capacity is calculated via strain compatibility and equilibrium. A similar methodology has been followed for rectangular columns.[56] Others[57] used the fiber element method to discretize the cross section of FRP-wrapped columns and calculated the load capacity via strain compatibility. The approach included

| Reference | Slab specimen | Temperature at failure (°C) | Load level at failure | Failure mode |
|---|---|---|---|---|
| [65] | E-100-1 | 94 | 0,58 | Bond failure |
| [65] | G-100-1 | 91 | 1,00 | Bond failure |
| [65] | E-200-1 | 185 | 0,58 | Bond failure |
| [65] | E-200-2 | 166 | 0,58 | Bond failure |
| [65] | E-200-3 | N/A | 0,58 | Bond failure |
| [65] | G-200-1 | 179 | 0,77 | Bond failure |
| [65] | G-200-2 | 197 | 0,77 | Bond failure |
| [54] | Slab 6 | 400 | 0,36 | Loss of FRP bond followed by steel yielding |
| [54] | Slab 7 | 400 | 0,36 | Loss of FRP bond followed by steel yielding |
| [54] | Slab 8 | 400 | 0,36 | Loss of FRP bond followed by steel yielding |
| [54] | Slab 9 | 400 | 0,36 | Loss of FRP bond followed by steel yielding |
| [54] | Slab 10 | 500 | 0,37 | Loss of FRP bond followed by steel yielding |

Table 4: Experimental results of FRP-strengthened slabs at elevated temperatures

geometrical nonlinearities and creep effects. The same study provided results of axial force–moment interaction curves after different fire exposure times for rectangular and circular columns and compared them with test data of RC columns.

An approach to simulate the structural behavior of heated FRP-strengthened beams via generation of moment–curvature relationships at distinct time intervals has been proposed in the literature.[47] The beam is divided to segments, for which separate calculations are conducted, and accounts for all appropriate strains (thermal, creep and transient in concrete), as well as the slip at the FRP–concrete interface. FEM has also been applied to simulate the structural response of FRP-strengthened beams. In relevant work,[49] eight-node solid and shell elements with the appropriate thermo-mechanical properties were used to simulate concrete and FRP, respectively. In other 3D simulations, the possible bond failure[58] of the FRP or bond-slip models were included.[22]

### *Discussion and Research Needs*

Relevant experimental work[45,59] has shown that FRP-strengthened columns with insulation can, in some cases, achieve a fire rating higher than 5 h. The gathered experimental data indicate that columns loaded up to 60% of their room temperature strength can sustain the load for FRP temperatures up to 500°C. This can be attributed to the cover, which limited temperature rise in steel reinforcement at even lower levels and prevented strength loss of the RC section. The two columns (*Fig. 7a*) that failed at high temperatures (around 900°C) were unprotected. This phenomenon has been explained[59] by stating that concrete spalling did not occur until late in the test due to heavy confinement of the section. The data referring to the residual strength of columns display considerable scatter, especially around the region of 200°C. While most columns maintained 80–100% of their initial strength for FRP temperatures up to 200°C, reductions up to 50% were also observed. It should be noted, however, that data with low residual strengths are part of an experimental series,[44] in which the RC section was lightly reinforced (in terms of longitudinal and lateral reinforcement) and most of the initial strength depended on the FRP system. These data correspond to experiments in which insulation was not effective enough and FRP bond was lost, leading to a great compromise of the residual strength. However, the current experimental database regarding heated columns incorporating FRPs should be expanded to provide a better understanding of their structural response. Current data suggest that load ratio alone might not be adequate to determine their failure temperature. Other parameters (temperature variation at lateral and longitudinal steel reinforcement and size effects of the tested specimens) should be investigated in future work.

Experimental studies on FRP-strengthened beams show that their initial load bearing capacity reduced to approximately 60% even at temperatures lower than 200°C, due to loss of the FRP system. Despite this observation, typical service loads for beams, which correspond to approximately 50% of their capacity, could be sustained for low temperatures. In one experiment,[41] the beam failed under the same load ratio at a temperature of 420°C. This was attributed[41] to low temperatures of the FRP at the anchorage zones, which delayed the debonding at midspan. Higher failure temperatures are associated with lower load ratios, because the strength of steel rebars degrades considerably at temperatures higher than 500°C.[69] However, the data are few and future testing should be carried out. Emphasis should be given on the effect of catenary action in delaying the failure of FRP-strengthened beams.

FRP-strengthened slabs failed at very low temperatures (200°C or lower), due to debonding of the FRP[65] for load ratios exceeding approximately 0,6. When the load ratio was lower (around 35%), failure temperatures from 400 to 500°C were achieved. When the FRP increases the strength of the RC section significantly (Ref. [54] reported a 51% increase for the tested specimens), the ability of the slab to carry loads at elevated temperatures, after FRP debonding, is greatly compromised. Testing to date focuses only on one-way slabs. Future research should also include some experimental work on two-way specimens, particularly when the transverse and longitudinal direction properties of the FRP material are different. In such cases, the effectiveness of the strengthening system in sustaining two-way load distributions should be investigated.

Numerical analyses yield reasonable results, but their comparison with test data is very limited. Future experimental work could provide an opportunity to validate these approaches. Researchers[59] have included the effect of confinement in FRP-wrapped columns, which is significant in terms of strength calculations. Numerical studies on beams seem to be complete after bond-slip models have been incorporated in simulations. On the contrary, relevant studies on slabs have not been conducted so far, and future research is needed to evaluate their response when heated.

## Preliminary Guidelines

The need for an adequate fire design of RC members reinforced/strengthened by FRPs has already been emphasized.[7] As mentioned in Introduction section, relevant codes used in practice nowadays are limited to design issues (such as ultimate strength, durability and ductility issues) at ambient temperature. The current fire design philosophy suggests that the initial (before FRP strengthening) nominal strength of the member be sufficient to carry the load during fire. This approach has been proposed by ACI Committee 440[2] and ignores the contribution of FRP strengthening at elevated temperatures. Contrary to this, some studies[13] mention that FRPs can be included in the calculation of the member's strength, provided that their temperatures are kept below a certain temperature. The same study[13] states that this temperature must fall within the range of 100–300°C.

Despite numerous suggestions, controversy still exists regarding the design load in a fire situation. According to ACI Committee 440,[2] the service load assumed for fire design includes the unfactored dead and full live loads, making the specific design approach rather conservative. On the other hand, Eurocode 1[70] proposes the use of a portion of live load in fire design. Moreover, it has been stated[71] that the existing loads in buildings during a fire event are usually <50% of the loads that will cause failure. Clearly, the need to establish a rational loading level for structural fire design arises.

Performance-based methods, which involve calculation of strength at elevated temperatures or even more sophisticated coupled thermal/structural analyses, have been suggested[72] as the most appropriate fire safety design approach for FRP-strengthened/reinforced concrete structures.

Based on the literature reviewed, future codes and provisions related to the fire design of FRP-reinforced/strengthened RC members should include the guidelines listed below:

- FRP manufacturers should certify their products via fire testing and provide the fire resistance rating and $T_g$ of unprotected FRP materials. They should also develop new certified insulation systems. Authorities should publish instructions and standards for such tests.
- Future codes should include tabulated data (similar to those in ENV 1992-1-2[73]) regarding the effect of the cover on the fire resistance of FRP-reinforced RC members.
- Simple rules for determining the fire resistance of members incorporating FRPs (according to the increase in strength provided by the FRP system and its effectiveness at elevated temperatures) could be determined.
- A simplified method for calculating the fire resistance of FRP-strengthened members based on the insulation material and its thickness should be included in future codes.
- A critical temperature (most possibly in the region of $T_g$), above which the FRP material shall be considered ineffective in strength calculations, must be determined. When the temperature of the FRP exceeds this temperature (e.g. 400°C), the fire resistance of the member should be determined by taking into account only the RC section.
- Emphasis must be given to the bond strength between concrete and FRPs by adopting stringent rules with regard to the anchorage length and the properties of adhesive materials (epoxies) at elevated temperatures. The use of materials that could provide a better bond between FRP and concrete, instead of epoxies, should be investigated.
- A limiting temperature/fire duration for the FRP system, beyond which it should be repaired or replaced, has to be determined.
- A rational estimation of the design load in fire situations has to be incorporated in fire safety codes.

### *Recommended Fire Design Procedure*

A fire design procedure for RC structures incorporating FRP materials should include the following steps:

- The structural elements that can carry the design loads without FRP strengthening/reinforcing can be exempt from structural adequacy checks in fire design situations.
- In modern RC construction, structures are designed with adequate redundancy. Elements not critical in the collapse prevention mechanism should be identified and designed for lower fire ratings. Thus, a cost-efficient structural fire design could be achieved.
- The insulation/cover thickness should then be calculated for structural elements critical in fire design, according to the required fire resistance rating. The calculations should account for minimal increase in the temperature of FRP materials (i.e. up to 100–200°C). The estimation of cover/insulation could be done via mathematical formulas given in relevant codes and literature or simplified thermal analysis of the cross section.
- In construction projects of major importance or structures where a sound fire design is vital, the insulation/cover thickness should be obtained by 3D thermal analysis. The overall response of the structure should be simulated by appropriate modeling and a coupled thermal/structural analysis. The proposed method may require longer calculation times but can lead to an economic design through optimization of the cover/insulation.

## Summary and Conclusions

This paper presented a review on the behavior of concrete structural elements strengthened/reinforced with FRP materials under elevated temperature effects. After presenting experimental data regarding the variation of their properties with temperature, the behavior of RC members incorporating FRPs in fire was studied. Once the fire exposure was defined, their thermal and structural response was determined through experimental and numerical analysis studies. Information on the existing preliminary guidelines for fire design was also provided and new ones were proposed.

Experimental results on the mechanical properties of FRPs at elevated temperatures have shown that the reduction in bond strength is the critical parameter in determining the response of FRP-strengthened/reinforced members exposed to elevated temperatures. Relevant information on the failure modes of such members verifies this. Despite being sensitive to fire, the application of such systems in rehabilitating damaged concrete members should not be restricted to structures in which fire safety is not of primary concern. The use of insulation has proven to be an effective measure against temperature rise in the FRP material, thus allowing for this strengthening technique to become applicable in construction where fire poses a serious threat. The publication of relevant codes and standards that will address all issues pertaining to a sound fire safety design of concrete structures strengthened/reinforced with FRPs is deemed necessary.

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