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High Temperature Properties of Fiber Reinforced Polymers and Fire Insulation for Fire Resistance Modeling of Strengthened Concrete Structures

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Abstract

Temperature dependent properties of materials are a crucial input for evaluating fire resistance of a structural member through calculation methods. While considerable amount of data is available on the high temperature properties of concrete and reinforcing steel, only limited information is available on the temperature dependent properties of FRP and fire insulation. This paper reviews the currently available high temperature property test data and associated relations for FRP and highlights the influence of different property relations on the fire resistance predictions of FRP-strengthened concrete structural members. Further, the influence of temperature dependent thermal properties of fire insulation on fire resistance prediction of FRP-strengthened member is also evaluated. Results from the analysis indicate that use of different property relations for FRP significantly alters the predicted fire response as well as the fire resistance of the strengthened concrete members. Results from the numerical studies further indicate that thermal properties of fire insulation have significant influence on fire resistance of FPR-strengthened RC members. Based on these numerical studies recommendations are made for

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using appropriate property relations for FRP and fire insulation in evaluating fire resistance of FRP-strengthened concrete members.

Keywords: FRP, high temperature properties, numerical modeling, fire resistance

1. Introduction

In recent years, fiber reinforced polymers (FRP) are used for strengtheneing and retrofitting of reinforced concrete (RC) structural members due to the numerous advantages of FRP including, high strength, light weight, and corrosion resistance, over traditional materials such as, steel plates (Bakis et al., 2002). FRP are used either as an externally bonded (EB) reinforcement or near surface mounted (NSM) reinforcement to enhance the load bearing capacity (strengthening) of a struture or to repair a damaged structure (retrofitting). Although FRP exhibits excellent properties at ambient conditions, the properties at elevated temperatures are quite poor. For instance, the strength and stiffness properties of FRP degrade significantly even at modest temperatures (less than 200°C) due to glass transitioning of polymer matrix. Owing to the poor properties of FRP at elevated temperatures, fire resistance of FRP-strengthened member is a major concern in building applications. To prevent the rapid degradation in strength and bond properties of FRP, and to increase the fire resistance of strengthened structural member, various researchers (Blontrock et al., 2000; Williams et al., 2006, Kodur and Ahmed, 2010) have recommended to apply a layer of external fire insulation.

Structural members, when used in building, are required to satisfy fire resistance requirement as specified in building codes (IBC, 2015). Therefore, when a RC member is strengthened with FRP, fire resistance of the strengthened RC member must be evaluated. In last two decades, the use of numerical models for evaluating the fire resistance of structural member has increased significantly. Accurate fire resistance assessment of FRP-strengthened RC structural member using numerical models, require knowledge of high temperature thermal and mechanical properties of the constituent materials i.e., concrete, reinforcing steel, FRP, and insulation.

A good amount of reliable data on high temperature properties and associated temperature dependent relations of concrete and reinforcing steel is available in the literature and design codes or standards. However, relatively limited data is available on high temperature properties for FRP in codes and standards or open literature (Firmo et al., 2015a). Similar to FRP, the information on high temperature thermal properties of insulation is also limited. In most cases, only the room temperature thermal properties of the insulation are provided by the manufacturer.

The available information on high temperature strength and stiffness properties of FRP is based on the limited material property tests available in the literature. These tests are carried out on FRP materials with different type and volume fraction of fibers and polymers, which significantly influence the high temperature properties. Moreover, at present there are no standardized test methods for evaluating the properties of FRP at elevated temperatures. Thus, previous researchers have adopted different parameters such as, type of specimen, heating rate, and load levels during the tests on FRP.

Owing to the dissimilar FRP materials and testing conditions, there exists a wide variation in the available data on the high temperature material properties of FRP. Similar to strength and stiffness properties, the data available on the temperature induced bond degradation is also limited, that too, over a smaller temperature range and thus there exists a wide variation in available temperature induced bond degradation data. Therefore, there are limited number of bond stress-slip relations describing the interfacial bond behavior at elevated temperatures. Since use of different property relations may result in varying fire resistance prediction, it is important to critically examine and arrive at the most suitable high temperature material properties of FRP for fire resistance evaluation.

The lack of information on high temperature thermal properties of insulation and the wide variation in limited amount of data available on high-temperature properties of FRP presents a challenge in evaluating realistic fire resistance of FRP-strengthened RC structural member. To address this concern, the available high temperature property relations for FRP in literature are reviewed in the present study and the influence of the available relations, on fire resistance prediction, is highlighted by undertaking numerical studies on FRP-strengthened RC beams. Further, the effect of temperature dependent thermal properties of fire insulation on fire resistance prediction is also evaluated through numerical studies. Based on these studies recommendations are made for use of relevant FRP and insulation property relations for fire resistance evaluation of FRP-strengthened RC structural members.

2. High Temperature Material Properties of FRP and Insulation: State-of-the-Art

Fire response of FRP-strengthened RC structures is governed by thermal, mechanical, and deformation properties of constituent materials as well as by the level of bond developed at FRP-concrete interface. The thermal properties govern the temperature rise and associated thermal gradients that develop within the section, whereas mechanical properties determine the extent of fire induced degradation in capacity and stiffness of the structural member. Deformation properties control the extent of deformation in concrete members incorporating FRP, while the bond properties determine the level of stress transfer from concrete to FRP.

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Temperature dependent thermal, mechanical, and deformation properties of concrete and steel rebars have been extensively studied in the literature (Phan and Carino, 1998; Phan and Carino, 2003; Kodur and Sultan, 2003; Cheng et al., 2004; Kodur et al., 2004; and Naus, 2006). Further, constitutive/empirical relations defining the temperature variation of these properties are also reported in various documents, such as Eurocode 2 (2004) and ASCE manual (1992). However, the information available regarding the temperature dependent properties of FRP and insulation is rather limited. A review of the information available on these properties is presented in following sections.

2.1 Thermal Properties of FRP

The thermal properties required for evaluating temperature rise in a fire exposed FRP-strengthened RC member include thermal conductivity, specific heat and density of concrete, steel reinforcement, FRP, and insulation. There is limited test data on thermal conductivity and specific heat of FRP. This is attributed to lack of research and specific instrumentation required to handle the complex nature of chemical reactions taking place in FRP at high temperatures. Moreover, there are no empirical relations currently available describing thermal properties of FRP as a function of temperature. Figure 1 (a) and (b) shows thermal conductivity and specific heat of FRP as a function of temperature, compiled using limited published data from the tests carried out on FRP used in aerospace and automobile applications (Griffis et al., 1981; Scott and Beck, 1992; Miller and Weaver, 2003; and Sweeting and Liu, 2004).

It can be seen from the Figure 1 (a) and (b) that both thermal conductivity and specific heat of FRP varies almost linearly with increase in temperature and there exists a wide variation in the available test data. A closer examination of Figure 1 (a) shows that the temperature variation of thermal conductivity as reported by Griffis et al. (1981) is contradictory to trends reported by other researchers. The variation in the reported test data is attributed to different polymer matrix, as well as different type, orientation, and volume fraction of fibers in the FRP systems used in tests. On the contrary, the similar trends are reported for specific heat of FRP at elevated temperature. The sharp increase followed by a plateau between 350-510°C is attributed to the consumption of additional heat due to thermal degradation (decomposition) of polymer matrix. Additionally, it can be seen from Figure 1 (a) and (b) that except for Griffis et al. (1981), most of the test data is available only up to a small temperature range (less than 200°C). This is because the tests were terminated once the polymer matrix started burning due to thermal decomposition. Therefore, there is lack of data on variation of thermal properties of FRP at temperature above the decomposition temperature of polymer matrix.

2.2 Mechanical Properties of FRP

Mechanical properties required for fire resistance of FRP-strengthened RC structural member include tensile strength and elastic modulus of FRP as well as strength of FRP-concrete interfacial bond. A review of literature indicates that high-temperature strength properties of FRP have been studied more widely than the thermal properties of FRP. In these studies, the strength properties were measured either during exposure to a specific constant temperature level or after exposure to elevated temperature level and then cooling down the to room temperature.

(a) Strength and Elastic modulus

Figure 2 (a) shows the reported test data on the tensile strength and elastic modulus of two different types of FRP, namely carbon FRP (CFRP) and glass FRP (GFRP) as a function of temperature. The figure is compiled using the limited test data available on strength tests of CFRP

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and GFRP at elevated temperature, reported by, Kumahara et al. (1993), Blontrock et al. (1999), Sumida et al. (2001), Zhou (2005, referred from Dai et al., 2014), Wang et al. (2007), Cao et al. (2009), Cao et al. (2011), Chowdhury et al. (2011), Wang et al. (2011), Yu and Kodur (2014), Hawileh et al. (2015), Nguyen et al. (2016 and 2018), Hamad et al. (2017), and Hajiloo et al. (2018). For clear comparison, the tensile strength and elastic modulus values of FRP composites at different temperatures are normalized to that at ambient temperature.

It is evident from the figure that there is a significant variation in the test data available on the high-temperature strength properties of FRP. These variations can be attributed to factors, such as, test procedures, heating rates as well as specific polymer type, orientation and volume fraction of fibers. Therefore, the plotted data cannot be directly compared/ extrapolated to other FRP materials. However, the plotted data is a representative of the mechanical behavior of typical FRP systems used in strengthening of RC structural members.

It can be seen from Figure 2 (a), the strength properties of FRP deteriorate significantly with increasing temperature. In general, the strength and elastic modulus of FRP experience a steep reduction at temperature close to glass transition temperature (T_g) of the polymer matrix, followed by a gradual reduction during the decomposition of polymer. Even after glass transition and decomposition of polymer matrix FRP are able to retain a considerable fraction of their ambient temperature tensile properties as fibers (i.e., carbon or glass) are able to retain much of their strength at high temperature. However, with increase in temperature beyond 500°C, the strength and elastic modulus of FRP once again experience a steep reduction due to oxidation of fibers and is more intense in the temperature range of 600-800°C. Additionally, it can be seen that FRP loses nearly 50% of its strength in the temperature range of 250°-350°C, which is much more rapid as compared to loss in strength of steel rebars.

Apart from the test data, relations defining the variation of strength and elastic modulus of CFRP and GFRP, with temperature are available in literature and are shown in Figure 2 (b) and (c), respectively. These relations are proposed by (Saafi (2002), Bisby et al. (2005), Wang et al. (2007), Wang et al. (2011), Dai et al. (2014), Yu and Kodur (2014), Nguyen et al. (2016), and Nguyen et al. (2018) for FRP. Majority of these relations define the temperature dependent variation in strength properties of CFRP, while relatively few relations define the variation in strength properties of GFRP. Additionally, most of these relations are in form of semi-empirical equations and express the variation of strength and stiffness of FRP up to 800°C, whereas some relations are in the form of reduction factors providing percentage degradation in ambient temperature strength and stiffness of FRP at different temperature level.

Saafi (2002) proposed linear/ bilinear relations for expressing degradation of strength and stiffness of CFRP and GFRP at elevated temperature. These relations are based on the tests carried out on CFRP and GFRP rebars reported by Blontrock et al. (1999). The strength and stiffness degradation relation for CFRP are given as:

$$\frac{f_T}{f_{20°\mathrm{C}}}=\begin{cases}1; & 0\le T\le 100\\ 1.267-0.00267T; & 100\le T\le 475\\ 0; & 475<T\end{cases}$$
$$\frac{E_T}{E_{20°\mathrm{C}}}=\begin{cases}1; & 0\le T\le 100\\ 1.175-0.00175T; & 100\le T\le 300\\ 1.625-0.00325T; & 300\le T\le 500\\ 0; & 500<T\end{cases} \quad (1)$$

where, f and E denotes the strength and stiffness of CFRP.

Bisby et al. (2005) compiled the test data available until then on the high temperature strength properties and proposed a sigmoid function to define the variation in strength and stiffness of CFRP and GFRP. This function is given as:

$$\frac{f_T}{f_0} = \left(\frac{1-a}{2}\right)\tanh\left(-b\left(T-c\right)\right)+\left(\frac{1+a}{2}\right) \tag{2}$$

where, f_T is the strength property in question (strength or elastic modulus) at temperature T; f_0 is the value of the strength property at ambient temperature; a, b, and c are the constants derived from least-squares regression analysis, and their values depends on the type of fiber (i.e., carbon or glass). The constant a describes the residual value for the strength property in question, whereas, b and c describe severity of property degradation and central temperature. The sigmoid function i.e., EQ. [2] does not consider the effect of T_g of the polymer, which significantly Most of the test data compiled by Bisby et al. (2005) was from the tensile strength tests on prefabricated FRP rebars, which has slightly higher T_g than the FRP sheets.

The sigmoid function Eq. [2] was further modified by Dai et al. (2014) to account for T_g of the polymer matrix and is given as:

$$\frac{f_T}{f_0} = \left(\frac{1-a}{2}\right)\tanh\left(-b\left(\frac{T}{T_g}-c\right)\right)+\left(\frac{1+a}{2}\right) \tag{3}$$

Wang et al. (2011) proposed a relation for defining degradation in strength of CFRP with temperature, by fitting the strength test data of CFRP to the Chen et al. (2006) model proposed for structural steel and is given as:

$$\frac{F_{u,T}}{F_{u,normal}} = A - \frac{(T-B)^n}{C} \tag{4}$$

where, A, B, C, and n are constants the values for which varies with temperature. Yu and Kodur (2014) proposed a semi-empirical relation for expressing the variation of strength and stiffness of CFRP strips and rods with temperature by fitting the test data to the hyperbolic function proposed by Gibson et al. (2006). The relations proposed by Yu and Kodur for strength and stiffness of CFRP strips at elevated temperatures are given as:

$$\begin{aligned} f_{\text{strip}}(T) &= 0.56 - 0.44\tanh\left(0.0052(T-305)\right) \\ E_{\text{strip}}(T) &= 0.51 - 0.49\tanh\left(0.0035(T-340)\right) \\ f_{\text{rod}}(T) &= 0.54 - 0.46\tanh\left(0.0064(T-330)\right) \\ E_{\text{rod}}(T) &= 0.51 - 0.49\tanh\left(0.0033(T-320)\right) \end{aligned} \tag{5}$$

Nguyen et al. (2018) calibrated a three-degree polynomial function to illustrate the strength reduction of CFRP. The function was calibrated using the test data generated by the authors from the elevated temperature tests on wet lay-up CFRP coupons and is given as:

$$\frac{\sigma_{u,T}}{\sigma_{u,20°\text{C}}} = K_0 - K_1\left(\frac{T-T_m}{T_{\text{max},10\%}}\right)^3 - K_2\left(\frac{T-T_m}{T_{\text{max},10\%}}\right) \tag{6}$$

where, T_{m} is the mechanical glass transition temperature, $T_{\text{max},10\%}$ is failure temperature at 10% of the stress ratio, K_0 is the coefficient (ranging from 0.4 to 0.6), K_1 and K_2 are calibrated coefficients.

As is the case with test data, there also exists a significant variation in the empirical relations for strength properties of both CFRP and GFRP. These variations are due to the large variation in the test data used to compile the respective relations. The variations are also due to the different parameters (such as, T_g of adhesive, stress ratio, failure load etc.) considered and technique used for deriving the respective relations. In some cases, the strength degradation relations are nearly same, but the stiffness degradation relations are significantly different. For instance, the strength degradation in CFRP, as predicted by the generalized sigmoid function proposed by Bisby et al. (2005), is nearly same as that predicted by the specific relations for CFRP strips and CFRP rods proposed by Yu and Kodur (2014). However, the stiffness degradation in CFRP, as per the relation proposed by Bisby (2005) is significantly different from the degradation predicted using Yu and Kodur (2014) relations, during the entire temperature range up to 600°C.

The property relations proposed by Wang et al. (2011) and Dai et al. (2014) describe rapid reduction (up to 60%) in the strength and elastic modulus of FRP at temperatures below 200°C, whereas the relations by Saafi (2002), Bisby et al. (2005), Yu and Kodur (2014), and Nguyen et al. (2018) (only for strength) consider a gradual reduction. Furthermore, the relation proposed by Dai et al. (2014) consider the strength and elastic modulus to be constant beyond T_g of polymer matrix used in FRP, whereas the other models predict continuous reduction in strength and elastic modulus until 600-700°C. This is attributed to the fact that the relations derived by Dai et al. (2014) assumes that the FRP-concrete interface would lose the bond-capacity beyond 200°C completely and therefore the contribution of FRP would be completely lost.

Another difference in the available relations, is the behavior of elastic modulus predicted by Nguyen et al. (2016) and Nguyen et al. (2018) relations, which shows an increase in the elastic modulus with increase in temperature (Figure 2 (c)). This behavior contradicts the behavior predicted using other relations and is attributed to the method used for preparing CFRP coupons (wet lay-up), the test method and the resulting test data used for the deriving the relation.

(b) FRP-concrete interfacial bond

The reported test data available on bond strength of the FRP-concrete interface is plotted as function of temperature in Figure 3 (a). Majority of these tests (Blontrock, 2003; Klamer et al., 2005; Gamage et al., 2006; Wu et al., 2005; Leone et al., 2009; Klamer, 2009; Firmo et al., 2015b) are carried out on concrete specimens strengthened with FRP using EB technique. Whereas, relatively little test data (Palmieri et al., 2011; Burke et al., 2013; Yu and Kodur, 2014b; and Firmo et al., 2015c) is available on the elevated temperature bond behavior of concrete strengthened with FRP using NSM technique. It can be seen from the figure that a large variation exists in the available test data on bond strength of FRP-concrete interface. Further, it can be seen that the bond strength decreases very rapidly with rise in temperature and diminishes after about 120-150°C.

The bond between FRP and concrete is critical for stress transfer from concrete to FRP, and therefore, influences the response of FRP-strengthened RC structures under fire exposure. Hence, the temperature induced degradation in interfacial bond must be duly accounted for evaluating fire resistance of FRP-strengthened RC structures. In many previous studies, researchers adopted a perfect bonding between FRP and concrete, as it is simplest approach to account for bond behavior. However, this approach leads to stiffer response and thus, higher fire resistance prediction which is unrealistic (Ahmed and Kodur, 2011). Alternatively, others adopted a crude approach in which the FRP-concrete interfacial bond was assumed to be completely lost once temperature at the interface exceeded the T_g of adhesive. This approach, on the other hand provides too conservative response (Kodur and Yu, 2013) and again unrealistic.

A better approach to account for bond behavior is through temperature dependent bond stress-slip relation, which can be expressed as a bilinear curve or as a single exponential curve. While several bond stress-slip relations are available in literature for modeling FRP-concrete interfacial bond at ambient temperature, very few relations are available for modeling bond behavior at elevated temperature. Due to the lack of information on bond stress-slip relations at elevated temperature, several researchers have used ambient temperature bond stress-slip relations, together with bond strength degradation, for fire resistance analysis of FRP-strengthened RC beams.

At present, the widely accepted bond stress-slip relations available in the literature are in the form of bilinear model proposed by Lu et al. (2005) and exponential curve model proposed by Dai et al. (2013) and are shown in Figure 3 (b). The model proposed by Lu et al. (2005) describe the bond behavior at ambient temperature, whereas the model proposed by Dai et al. (2013) describe the bond behavior at ambient and elevated temperatures, as shown in Figure 3 (b).

It can be seen from the Figure 3 (b) that there exists a significant difference in the behavior described by different bond stress-slip relations, specifically in the ascending branch of the curve before peak bond-strength is reached. The difference can be attributed to the test data and the parameters considered in deriving the relations. A closer examination of the figure indicate that the relation proposed by Dai et al. (2013) provide a stiffer response as compared to Lu et al. (2005) relation. Further, the relation by Lu et al. reaches a zero bond stress after reaching specific slip value indicating complete loss of bond. Whereas the relation by Dai et al. being an exponential curve does not lead to zero bond stress indicating that the bond is never completely lost.

2.3 Deformation Properties of FRP

Deformation properties such as thermal expansion and creep strain determine the extent of deformation in concrete members incorporating FRP. The deformation properties in FRP depend on the type and orientation of fibers as well type of polymer. Like thermal and mechanical properties, the deformation properties of FRP in longitudinal and transverse direction is dominated by fibers and polymer matrix, respectively (Bank, 1993). Limited information is available on the variation of deformation properties of FRP at elevated temperatures; extending only up to 200°C, as the polymer starts melting beyond this temperature. It has been reported that commercially available fibers (except aramid fibers) do not creep significantly and have very small value of thermal expansion until 200°C (Mallick, 1988). Therefore, temperature effect on creep and thermal expansion properties can be neglected for unidirectional FRP sheets/laminates used in strengthening applications (Rahman et al., 1993).

2.4 Thermal Properties of Fire Insulation

Thermal properties of fire insulation required for fire resistance evaluation of FRP-strengthened RC members include thermal conductivity and heat capacity. These thermal properties depend on the type and composition of the insulation material used such as, calcium silicate board, gypsum wall board, Rockwool, or spray applied fire resistive materials (SFRM). Even a small variation in the composition of fire insulation can lead to significant changes in the thermal properties.

Limited test data is available on temperature dependent thermal properties of insulation material. Moreover, in most cases the properties are evaluated up to a lower temperature range less than 400°C. Figure 4 (a) and (b) show the plots of available data on thermal conductivity and heat capacity of different types of fire insulation, as a function of fire exposure time. The fire insulation materials considered for compiling the figure include, insulation boards (Promatect H, Promatect L 500, Promasil), and cementitious SFRMs (CAFCO 300, Carboline Type 5-MD, Tyfo WR-AFP, TB tunnel fire proofing). The insulation boards comprise of different proportions of calcium silicate, gypsum and vermiculite as main components, whereas the cementitious materials consist of different proportions of cellulose, Portland cement, gypsum, and vermiculite as main components. The properties of insulation boards are based on the test and analytical studies reported by Bai et al. (2010), López et al. (2013), and Firmo et al. (2015d). Whereas the properties of cementitious materials are based on the tests carried out by Kodur and Shakya (2013) on different SFRMs. Bai et al. (2010) and Kodur and Shakya (2013) also proposed relations for defining the variation of thermal properties of insulation material with temperature. However, these relations are specific to the type of insulation material with specific composition and hence, cannot be used as a general relation for all fire insulation materials.

It can be seen from the figure that the thermal properties of all the insulation materials vary significantly with increase in temperature. Further, it can be seen that even for similar type of insulation material i.e., board or cementitious material, the variation in thermal properties with temperature is significantly different. Initially, the thermal conductivity of cementitious insulation materials decreases in the temperature range of 100-200°C due to the evaporation of free moisture present in them. However, with increase in temperature the thermal conductivity of these cementitious material increases significantly. This can be attributed to the crystallinity of gypsum at elevated temperature. In case of insulation boards, the thermal conductivity increases continuously with increase in temperature at a constant rate, however, the increase is very small and, in some cases, negligible (Promatect L 500).

Similar to thermal conductivity, the heat capacity of the cementitious materials varies significantly with temperature as compared to insulation boards. Initially the heat capacity of the cementitious materials increases with increase in temperature. This is attributed to the evaporation of free moisture present in the insulation. The heat capacity remains almost constant or decreases at a very slow rate until 400°C and then start increasing at gradually. The increase in heat capacity beyond 500°C is be due to the release of chemically bound water present in the ingredients of the cementitious material. In case of insulation boards, the heat capacity increases at a faster rate until 100°C after which the change in heat capacity of the insulation boards is negligible. The increase in the heat capacity in until 100°C is attribute to evaporation of moisture present in the boards. These temperature dependent variations in thermal properties of fire insulation are to be duly accounted for in analysis for realistic fire resistance predictions in FRP-strengthened RC members.

3. Numerical Studies

To illustrate the effect of different thermal, mechanical, and bond degradation property relations of FRP on fire resistance predictions in FRP-strengthened RC members, numerical studies are carried out on a set of five RC beams. These RC beams are strengthened by EB CFRP sheets/laminates and are protected with appropriate fire insulation. For each beam ten different analysis cases, C1 to C10, were considered, and in each case the variation of specific FRP property on fire resistance predictions was evaluated. Results from the analysis are then compared to the measured fire resistance of strengthened beams to draw inferences on the effect of different property relations on fire resistance predictions. More details about the beams and the analysis approach are given in the following sections.

3.1 Beams for Analysis

The beams selected for the analysis are taken from fire tests undertaken in the literature by different researchers. The beams are designated as B1, B2, B3, B4, and B5. Beam B1 is the one tested by Blontrock et al. (2000); B2 is tested by Gao et al. (2010); B3 is tested by Firmo and Correia (2015); B4 is tested by Dong et al. (2016); and B5 is tested by Zhang et al. (2018) for fire resistance evaluation. Details of design parameters, dimensions, material properties, and loading for the selected beams are summarized in Table 1.

All selected beams have rectangular cross-section and are strengthened externally with one layer of CFRP sheet. The layout and type of insulation on each beam is shown in Table 1. Beams B1, B2, B4, and B5 are exposed to ISO 834 standard fire at the bottom and two sides, whereas beam B3 is exposed to ISO 834 fire only at the bottom surface. All the beams have simply supported end conditions, with four-point loading on beams B1 and B3, and six-point loading on beams B2, B4, and B5. The load ratio applied to the beams is defined as the ratio of applied load to the room temperature strengthened capacity of the beams and is summarized in Table 1.

3.2 Analysis Method

The selected beams were analyzed using a macroscopic finite element based numerical model initially developed by Kodur and Ahmed (2010) and later modified by Kodur and Bhatt (2018). This model accounts for temperature induced degradation in thermal and mechanical properties of concrete, steel reinforcement, FRP, including bond, as well as fire insulation and different failure limit states for tracing fire response of FRP-strengthened RC beam over the entire range of loading till failure of the beam. Fire resistance analysis was carried out in several incrementing time steps until failure is attained in the beam. At each time step, the temperature rise in cross-section, as well as deflection and moment capacity, of the beam are evaluated through a numerical procedure, which is briefly described herein.

Discretization: Fire resistance analysis starts by discretizing the given beam into 20-30 segments along its length [as shown in Figure 5(c)], and discretizing the cross-section of each segment into a mesh of rectangular elements [as shown in Figure 5(d)]. The cross-section at the middle of each segment is assumed to represent the overall behavior of the segment.

Fire temperature: Following the discretization of the beam, fire temperatures are computed at the beginning of each time step, using the time-temperature relation of relevant fire exposure. In the current analysis, to simulate fire conditions similar to the fire tests on these beams, ISO 834 standard fire exposure was consisdered as the fire scenario. The time-temperature relation for ISO 834 (2002) standard fire is taken as:

$$T_f = T_0 + 345 * \log_{10}(8t + 1) \tag{7}$$

where, T_f is temperature of fire (°C); T_0 is the initial temperature (°C); t_h is time (hours).

Thermal analysis: In this step, the temperature distribution within the beam cross-section was determined through a finite element based heat transfer analysis, using the high-temperature thermal properties of constituent materials (concrete, steel rebars, FRP, and insulation). At each time step, a heat balance is established in each element of the discretized beam to compute the temperature rise in the section, which is then provided as an input to the structural analysis.

Structural analysis: The reduced sectional moment capacity and increasing deflection of the beam with increasing fire exposure time was evaluated. The moment capacity of the beam was evaluated taking into account temperature dependent degradation in mechanical properties of constituent materials. The increasing deflection of the beam was evaluated through a direct stiffness analysis using the secant stiffness of each segment of the beam, which was computed from the moment-curvature (M- κ) relations generated for each segment during the sectional analysis (Kodur and Bhatt, 2018).

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Fire resistance time: At each time step, the increasing deflections and reduced moment capacity were utilized to determine the failure state of the beam. The failure is said to occur when the reducing moment capacity of the beam falls below the bending moment due to applied loading or the when mid-span deflection of the beam exceeds $L^2/400d$, and the rate of deflection exceed $L^2/9000d$ (mm/min) limit over one minute interval where, L = span length of the beam (mm), and d = effective depth of the beam (mm). The time at which any of the applicable failure limit state is exceeded, is taken as fire resistance of the beam.

Material properties: During the analysis, the temperature dependent degradation in thermal and mechanical properties of concrete and reinforcing steel, FRP and supplementary fire insulation were incorporated in the model. For concrete and steel rebars, the temperature dependent property relations defined in Eurocode-2 (2004) and ASCE manual (1992) are incorporated in the model. A preliminary analysis was carried out with three different combination of property relations for concrete and steel rebars, namely, (i) ASCE manual property relations for both concrete and steel, (ii) Eurocode-2 relations for both concrete and steel, and (iii) ASCE manual relations for concrete and Eurocode-2 relations for steel rebars. Response parameters (deflections, capacity at various fire exposure time, as well as failure time) predicted from the analysis with combination (iii) were closer to the experimentally measured data. Therefore, for the analyses in this paper, the temperature dependent property relations for concrete as per ASCE manual (1992) and the property relations for steel rebars as per Eurocode-2 (2004) were used in the model.

The temperature dependent variation in thermal conductivity and heat capacity of fire insulation was incorporated in the model depending upon the type of insulation used for thermal protection of the specific beam in the fire test and is shown in Figure 4 (a) and (b), respectively.

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The FRP property relations incorporated in the analysis are varied based on the specific analysis case under consideration. These property relations are summarized in Table 2 for cases C1 and C2, in Table 3 for cases C3 to C6, and in Table 4 for cases C7 to C10.

3.3 Results from Numerical Studies

Fire resistance analysis was carried out using the above described numerical model for all the five beams (B1, B2, B3, B4, and B5) in cases C1 to C10. The effect of specific FRP property relation on the fire resistance prediction in each case as well as results from the analysis in these cases are discussed in following sections.

3.3.1 Effect of thermal properties of FRP

Cases C1 and C2 are analyzed to evaluate the effect of temperature dependent thermal properties of FRP on fire resistance prediction. In case C1 the thermal properties of FRP are considered to be constant throughout fire exposure, whereas in case C2 the thermal properties of FRP are considered to vary with increase in temperature. Same strength and interfacial bond properties of FRP are considered in both these cases and are summarized in Table 2. Additionally, the fire resistance time and the failure limit state exceeded for each of the five beams are also compared in Table 2. Detailed results from the analysis of beam B1 in both these cases are selected to illustrate the effect of thermal properties of FRP on fire resistance prediction.

Figure 6 compares the thermal response of beam B1 predicted in cases C1 and C2 by plotting the temperature rise in rebar and FRP-concrete interface, as a function of fire exposure time. Additionally, the temperature rise measured at these locations during the fire test is also shown in the figure. It can be seen from Figure 6 that the temperature rise in rebar and FRP-concrete interface predicted in both the cases C1 and C2, is nearly identical. The lack of influence of thermal properties of FRP on predicted sectional temperatures can be attributed to the smaller cross-section of FRP sheet as compared to the cross-section of concrete. Further, it can be seen from the figure that the temperature rise predicted in both the cases follow trends similar to that of measured temperatures. However, the predicted temperatures are slightly higher than the measured temperature. This may be attributed to the thermal properties of fire insulation incorporated in the model which may be slightly different from the actual properties. Further, there exists a plateau in the temperature rise measured at the FRP-concrete interface at 100°C, which may be attributed to the evaporation of free water in the insulation. The model does not account for such evaporation and therefore, no plateau is predicted by the model.

Figure 7 compares the structural response of beam B1 predicted in analysis cases C1 and C2. Figure 7 (a) compares the degradation in moment capacity of beam B1 as predicted in both the cases, whereas Figure 7 (b) compares the deflection predicted in both the cases with the measured deflections. It can be seen from the Figure 7 that the predicted degradation of moment capacity and increase in deflections in both the cases (with and without varying thermal properties of FRP) are identical. Additionally, it can be seen from the Figure 7 (b) that the deflections predicted in both the cases are in close agreement with the measured values. The identical moment capacity degradation and deflection rise in both cases C1 and C2 is attributed to the nearly identical temperature rise within the cross-section in both the cases. Moreover, beam B1 fails at 108 minutes of fire exposure by exceeding the deflection limit state in both the cases (refer to Table 2), indicating that the beam has same fire resistance in both the cases. Similar results are obtained for other four beams (B2 to B5) analyzed in cases C1 and C2. From these results it can be clearly

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concluded that thermal properties of FRP, do not affect temperature rise and thus the fire resistance predictions in a fire exposed FRP-strengthened member.

3.3.2 Effect of Strength and Stiffness Reduction Factors for FRP

The effect of different property relations, defining variation of strength and elastic modulus, on fire resistance predictions, is analyzed for different beams through cases C3, C4, C5, and C6. In these cases, the strength and stiffness property reduction factors are assumed to vary as per the relations specified by Bisby et al. (2005), Wang et al. (2007), Dai et al. (2014), and (Nguyen et al. 2018)), respectively. Moreover, to prevent the complete loss of FRP contribution due to temperature induced bond degradation, perfect bond is assumed at the FRP-concrete interface. Further, during the analysis of the beams (B1 to B5) in cases C3 to C6, the temperature rise in the cross-section of a particular beam is identical to the temperature rise measured during the fire test of respective beam.

Fire resistance time as well as the failure limit state exceeded in each beam analyzed for each of these cases are compared in Table 3. The structural response from the analysis of beam B4 in cases C3 to C6 is used to illustrate the effect of different strength and stiffness reduction factors. Since the beam fails in strength limit state, only the degradation in moment capacity with fire exposure time is compared in Figure 8. It can be seen from the figure that the predicted degradation of moment capacity in all the cases is significantly different throughout the fire exposure duration.

The capacity of the beam in case C5 (when Dai et al. strength model for FRP is used) decreases rapidly in the initial stages of fire exposure and then continue to decrease a slower rate but does not fall below the moment due to applied loading, indicating no strength failure. This is attributed to the fact that the Dai et al. model considers no reduction in strength of FRP beyond the T_g of polymer matrix. Further, the moment capacity of the beam degrades at a faster rate in case C4 (using Wang et al. model) as compared to cases C3 (using Bisby et al. model) and C6 (using Nguyen et al. model). However, the moment capacity in case C4 falls below the applied loading at a much later time (thus higher fire resistance) as compared to cases C3 and C6. This is attributed to the faster reduction in FRP strength in cases C3 and C6 after 350°C and 500°C, respectively, as compared to reduction in strength of FRP in case C4 (*cf*. Figure 2 (b)).

Overall, the fire resistance predicted in cases C4 and C5 are much higher than the fire resistance predicted in cases C3 and C6. Similar results are obtained from the analysis of beams B1, B2, B3, and B5 in cases C3 to C6, and predicted fire resistance are summarized in Table 3. Based on these analysis results, it can be concluded that the temperature dependent strength relations for FRP proposed by Bisby et al. (2005) and Nguyen et al. (2018) provide a conservative estimate of fire resistance, as compared to relations proposed by Wang et al. (2007) and Dai et al. (2014).

3.3.3 Effect of Bond-Slip Relations for FRP-Concrete Interface

The effect of different FRP-concrete bond degradation relations on fire resistance prediction in EB FRP-strengthened RC beams is analyzed in cases C7, C8, C9 and C10. The specific bond degradation relation as well as the strength and stiffness reduction factors considered in each of these cases are summarized in Table 4. Additionally, the fire resistance predicted by the model and the failure limit state exceeded in the analysis of each beam are also summarized in Table 4.

The fire resistance predicted in case C7 (with no bond degradation) for all the beams is significantly higher than the measured values as well as the fire resistance predicted in other cases C8 to C10. On the contrary, the fire resistance predicted in case C10 (with no bond after T_g) is significantly lower than measured values. This is attributed to the fact that during the analysis in case C7, perfect bond is considered between FRP and concrete, whereas, during the analysis in case C10 it is considered that contribution of FPR is completely lost after interface temperature exceeds T_g. Thus, neglecting FRP-concrete bond degradation (case C7) provides an unrealistic estimate of the fire resistance, and neglecting strength and stiffness contribution of FRP beyond T_g (case C10) provides an over conservative estimate of fire resistance.

In cases C8 and C9, although the predicted fire resistance is close to the measured values, the deflection response predicted in case C9 (non-linear bond-slip relation) is closer to measured values than the deflection response predicted in case C8 (with bilinear bond-slip relation). Therefore, the use of Dai et al. (2013) bond-slip relations lead to better fire resistance predictions.

To further illustrate the effect of different bond degradation relations on fire resistance predictions structural response predicted from the analysis of beam B4 in cases C7 to C10 is compared in Figure 9 (a) and (b). It can be seen from the Figure 9 (a) that the capacity of the beam decreases gradually at a similar rate in cases C7, C8, and C9 for a major part of the fire exposure duration, whereas the capacity in case C10 decreases abruptly after 40 minutes of fire exposure and falls below the moment due to applied loading. This is attributed to the complete loss of strength and stiffness contribution from FRP after the temperature at the FRP-concrete interface exceeds the T_g of adhesive. The capacity of the beam B4 falls below the moment due to applied loading after 244 minutes of fire exposure in case C7, however, in cases C8 and C9 the capacity does not fall below the applied loading due to rapid increase in deflection leading to failure.

Similarly, it can be seen from Figure 9 (b) that deflection predicted in case C9 is in close agreement with the measured deflection response, whereas the deflection in cases C7 and C8 is much stiffer than the measured values. The stiffer response in case C7 is attributed to the perfect bond between FRP and concrete, whereas the stiffer response in case C8 is attributed to the higher bond-strength required for the initiation of debonding at the FRP-concrete interface. Due to early strength failure of the beam in case C10, the deflection is predicted only up to 40 minutes of fire exposure and is in close agreement with the measured values. This confirms that bond-slip model proposed by Dai et al. (2013) yields a better assessment of fire resistance in EB FRP-strengthened RC beams.

4. Effect of Varying Thermal Properties of Fire Insulation

FRP-strengthened concrete structural members are provided with a layer of fire insulation to increase the effectiveness of FRP for a longer time and to improve the fire resistance of the FRP-strengthened RC member. Thus, fire resistance of FRP-strengthened RC members is also dictated by thickness as well as thermal properties of fire insulation. As discussed in previous section, the thermal properties of fire insulation vary significantly with increase in temperature (Figure 4), and therefore, must be accounted in the fire resistance analysis. However, most often, constant (room temperature) thermal property of fire insulation (without considering temperature dependent variation) is utilized in fire resistance analysis of FRP-strengthened RC members. Such a design consideration can lead to inaccurate fire resistance assessment. To illustrate the effect of thermal properties of insulation, on fire resistance predictions, the aforementioned beams B1, B2, B3, B4, and B5 are analyzed in two different cases namely, C11 and C12.

In case C11, only room temperature properties of the insulation are considered i.e., the variation of thermal properties with temperature is neglected. Whereas, in case C12, the temperature induced variation in thermal properties of fire insulation is incorporated in the model. The thermal, mechanical, and deformation properties of concrete and steel are utilized as per

Eurocode 2 (2004). Further, the thermal, strength and bond degradation properties of FRP considered in the analysis in each case is are summarized in Table 5. Additionally, fire resistance time predicted from the analysis of all the beams in cases C11 and C12 are summarized in Table 5. It can be seen from the table that the fire resistance of all beams in case C11 is significantly higher than the fire resistance predicted in case C12.

Detailed results from thermal and structural analysis of beam B5 are used to illustrate the difference in response in these cases (C11 and C12). Figure 10 compares the temperature rise in rebar and FRP-concrete interface of beam B5 with the temperature rise at respective locations measured in test. It can be seen from the figure that the overall trends of temperature rise predicted in both the cases are similar to the trends measured in the test. However, the temperature rise predicted in case C11 are significantly lower than measured values, whereas the temperature rise predicted in case C12 are slightly higher than the measured values. The lower temperatures predicted in case C11 can be attributed to constant thermal properties of insulation used in the analysis which alters the heat transfer within the section thereby, providing an in-accurate temperature rise prediction. The higher temperature rise predicted in case C12 is attributed to the increase in thermal conductivity and decrease in heat capacity of the fire insulation at high temperature (refer to Figure 4), which increases the heat propagation within the cross-section, thereby increasing temperatures.

To compare the structural response of beam B5 predicted from the analysis in cases C11 and C12, the degradation in moment capacity and progression of deflection are plotted as a function of fire exposure time in Figure 11 (a) and (b), respectively. It can be seen from the Figure 11 (a) that although the degradation of moment capacity of beam in both the cases follow similar trends, the

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capacity decreases at a slower rate in case C11, as compared to that in case C12. Similarly, it can be seen from Figure 11 (b) that the deflection in case C12 match well to the measured values, whereas deflection in case C11 are stiffer as compared to the test data. This is attributed to the slower temperature rise in the beam in case C11, resulting from not accounting temperature induced effect on thermal properties of fire insulation, which in turn reduces the degradation of strength properties of the constituent materials. Further, the fire resistance time predicted in case C11 is 284 minutes which is significantly higher than the measured fire resistance of 199 minutes. Hence, neglecting the temperature dependence of thermal properties of fire insulation would lead to an unrealistic prediction of fire resistance in FRP-strengthened RC structural members.

5. Design Recommendations

Thermal, mechanical and deformation properties of concrete, steel, and FRP as well as thermal properties of fire insulation have a significant influence on fire resistance predictions of FRP-strengthened RC members. The thermal, mechanical, and deformation properties of concrete and reinforcing steel has been extensively studied in the literature and reliable temperature dependent property relations defining the variation of these properties with temperature are available in codes and standards. Based on recommendation from previous studies (Kodur et al., 2008 and Kodur et al., 2010), and based on the results from current analysis, for fire resistance prediction of FRP-strengthened RC members, the temperature dependent variation in properties of concrete and steel rebar can be accounted using the relations specified in ASCE manual (1992) and Eurocode-2 (2004), respectively.

There are considerable variation and significant discrepancies in the reported test data on the high temperature properties of FRP. Consequently, there are large variations in the available thermal, mechanical and bond degradation property relations available in literature. Results from the analyses infer that temperature dependent variation in thermal properties of FPR do not affect the fire resistance prediction of FRP-strengthened RC members. Hence, high temperature thermal properties of FRP can be neglected for fire resistance analysis of FRP-strengthened members.

Temperature dependent strength and stiffness relationd proposed by Wang et al. (2007) and Dai et al. (2014) tend to overestimate the fire resistance of FRP-strengthened, whereas the relations proposed by Bisby et al. (2005) and Nguyen et al. (2018) provide a reasonable estimate of fire resistance, with Nguyen et al. (2018) being more conservative than the Bisby et al. (2005) model. Therefore, it is recommended to use the relations proposed by Bisby et al. (2005) for defining the variation of strength and stiffness of FRP with temperature. These relations are summarized in Table 6.

Neglecting the temperature induced bond degradation (i.e., perfect bond), or assuming complete loss of FRP-concrete bond once the interface temperature exceeds adhesive T_g will lead to unrealistic assessment of fire resistance in FRP-strengthened RC members. Further, the use of bond stress-slip relations (models) proposed by Dai et al. (2013) and Lu et al. (2005) yield a closer estimate of fire resistance prediction, however, the deflection predicted by Dai et al. (2013) model are in close agreement with measured values as compared to the deflections predicted by Lu et al. (2005) model. Thus, the temperature dependent bond-slip relations proposed by Dai et al. (2013) yield better fire resistance prediction in FRP-strengthened RC structural members. These bond-slip relations and details of the relevant parameters are summarized in Table 6.

The analyses further indicate that neglecting the temperature variation of thermal properties of insulation will lead to unrealistic (higher than measured) assessment of fire resistance. Hence, the temperature dependent thermal properties of fire insulation must be considered in the fire resistance evaluation of an insulated FRP-strengthened RC structural member. The relevant temperature dependent thermal conductivity and heat capacity of commonly used insulation materials are summarized in Table 7 and Table 8, respectively.

6. Conclusions

Based on the comparative numerical study, the following conclusions are drawn regarding on the influence of high-temperature properties of fiber reinforced polymer (FRP) on fire resistance of FRP-strengthened RC members:

- Thermal properties of FRP do not affect sectional temperature rise and thus fire resistance of the FRP-strengthened RC members. Hence, the temperature dependence of thermal properties of FRP can be neglected in fire resistance modeling of FRP-strengthened structures.
- Significant variation exists in the available test data on strength and elastic modulus, as well as bond strength, of FRP at elevated temperature. The relations used for degradation of strength, stiffness, bond of FRP have significant influence on fire resistance predictions in FRP-strengthened RC structural members.
- Neglecting the temperature inducted bond degradation or assuming complete loss of FRP-concrete bond at interface temperature exceeding adhesive T_g, leads to an unrealistic estimate of fire resistance. The non-linear bond-slip relation proposed by Dai et al. (2013) provides a better assessment of fire resistance of FRP-strengthened RC structural members.
- High temperature thermal properties of insulation significantly alter sectional temperature predictions and thus fire resistance values in an FRP-strengthened RC member. Hence,

temperature dependent variation of thermal properties of fire insulation must be accounted for in fire resistance assessment of FRP-strengthened RC structural members.

- Based on the analysis, the temperature dependent strength and non-linear bond stress slip relations for carbon FRP recommended in this paper can give better predictions for deflection moment capacity degradation, and failure time of FRP-strengthened RC structural member exposed to fire.

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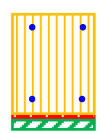
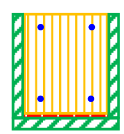
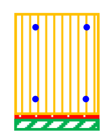
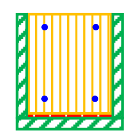
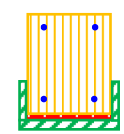
Tables

Table 1: Geometrical and material property details of beams selected for analysis

Parameter/Property			**B1**	**B2**	**B3**	**B4**	**B5**
Tested by			Blontrock et al. (2000)	Gao et al. (2010)	Firmo and Correia (2015)	Dong et al. (2016)	Zhang et al. (2018)
Clear span (m)			3.0	6.0	1.5	5.2	4.4
Cross-section: width × depth (mm × mm)			200 × 300	200 × 500	100 × 120	200 × 500	250 × 400
Steel rebar diameter	Top (mm)		2-10 ϕ	2-12 ϕ	2-6 ϕ	2-12 ϕ	2-12 ϕ
	Bottom (mm)		2-16 ϕ	2-16 ϕ	2-6 ϕ	2-16 ϕ	2-22 ϕ
Concrete cover thickness (mm)			25	20	10	20	25
Compressive strength concrete (f_c' , MPa)			47	23	37	30.7	40
Steel	Yield strength (f_y, MPa)		591	375	470	372	363
	Elastic Modulus (E_s, GPa)		205	200	193	210	210
FRP	Type		CFRP	CFRP	CFRP	CFRP	CFRP
	Thickness × width (mm × mm)		1.2 × 100	0.334 × 200	1.4 × 20	0.334 × 200	0.167 × 250
	Modulus of Elasticity (E_f, GPa)		165	160	189	260	200
	Tensile strength (f_u, MPa)		2800	4030	2076	4030	3455
	Glass transition temperature (T_g, °C)		65	73	47	73	85
Insulation	Material type		Promatect H	CS board	Promatect L 500	CS board	SFRM
	Thickness (mm)		25	40	25	40	10
	Density of insulation (kg/m^3)		870	244.7	450	250	500
	Thermal conductivity (W/m K)		0.175	0.0603	0.09	0.061	0.125
	Specific heat (J/ kg°C)		840	790	815	740	1036
	Scheme/ config-uration	Concrete					
		FRP					
		Insulation					
		Steel rebar					
Fire exposure			ISO 834	ISO 834	ISO 834	ISO 834	ISO 834

Load applied (kN)	2 × 40.6	4 × 30.2	2 × 6.1	4 × 33	4 × 25.5
Load Ratio (%)	45	52.2	36	63	50

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Table 2: Summary of property relations considered and fire resistance predicted in cases C1 and C2

Parameter		**Cases evaluating effect of thermal property variation of CFRP**	
		C1	**C2**
Thermal properties of CFRP		Constant w.r.t. temperature	Varying with temperature
Strength and elastic modulus reduction factors		Bisby et al. (2005)	
Relation to define bond between CFRP and concrete		Dai et al. (2013)	
Thermal properties of insulation		**Varying with temperature**	
Fire resistance of beams (minutes) and failure limit state S=strength criterion D=deflection criterion	B1	108	108
		D	D
	B2	112	112
		S+D	S+D
	B3	138	138
		S	S
	B4	152	152
		S+D	S+D
	B5	196	196
		D	D

Table 3: Summary of property relations considered and fire resistance predicted in cases C3, C4, C5, and C6

Parameter		**Cases evaluating effect of different strength and elastic modulus relations of FRP**			
		C3	**C4**	**C5**	**C6**
Thermal properties of CFRP		Varying with temperature			
Strength and elastic modulus reduction factors		Bisby et al. (2005)	Wang et al. (2007)	Dai et al. (2014)	Ngyugen et al. (2018)
Relation to define bond between CFRP and concrete		Perfect bonding			
Thermal properties of insulation		**Varying with temperature**			
Fire resistance of beams (minutes) and failure limit state S=strength criterion D=deflection	B1	136	144	320	108
		S	S	S	S
	B2	184	112	180	144
		S	S	S	S
	B3	336	164	>400	278

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criterion NF=No failure		S	S	NF	S
	B4	244	296	>320	248
		S	S	NF	S
	B5	204	200	228	196
		S + D	S	S	S

Table 4: Summary of property relations considered and fire resistance predicted in cases C7, C8, C9, and C10

Parameter		**Cases evaluating effect of different bond degradation relations**			
		C7	**C8**	**C9**	**C10**
Thermal properties of CFRP		Varying with temperature			
Strength and elastic modulus reduction factors		Bisby et al. (2005)			
Relation to define bond between CFRP and concrete		Perfect bonding	Lu et al. (2005)	Dai et al. (2013)	No bond after interface temperature exceeds T_g of adhesive
Thermal properties of insulation		**Varying with temperature**			
Fire resistance of beams (minutes) and failure limit state S=strength criterion D=deflection criterion	B1	136	108	108	108
		S	D	D	D
	B2	184	100	112	48
		S	S+D	S+D	S
	B3	336	135	138	58
		S	D	D	S
	B4	244	144	152	40
		S	D	D	S
	B5	204	196	196	196
		S + D	D	D	D

Table 5: Summary of property relations considered and fire resistance predicted in cases C11 and C12

Parameter	**Cases evaluating effect of thermal properties of insulation**	
	C11	**C12**
Thermal properties of CFRP	Varying with temperature	
Strength and elastic modulus reduction factors	Bisby et al. (2005)	
Relation to define bond between CFRP and concrete	Dai et al. (2013)	

Thermal properties of insulation		Constant w.r.t. temperature	Varying with temperature
Fire resistance of beams (minutes) and failure limit state S=strength criterion D=deflection criterion	B1	120	108
		S	D
	B2	208	112
		S	S+D
	B3	160	138
		S	S
	B4	196	152
		S	S+D
	B5	284	196
		S+D	D

Table 6: Recommended relations for defining degradation of strength, stiffness and bond properties of FRP

Property	**Proposed by**	**Relation**
Strength	Bisby et al. (2005)	$\frac{\sigma_T}{\sigma_0} = \left(\frac{1-a}{2}\right)\tanh\left(-b\left(T-c\right)\right) + \left(\frac{1+a}{2}\right)$ $a = 0.1;\ b = 5.83\times10^{-3};\ c = 339.54$
Elastic modulus	Bisby et al. (2005)	$\frac{E_T}{E_0} = \left(\frac{1-a}{2}\right)\tanh\left(-b\left(T-c\right)\right) + \left(\frac{1+a}{2}\right)$ $a = 0.05;\ b = 8.68\times10^{-3};\ c = 367.41$
Bond stress-slip	Dai et al. (2013)	$\tau_{f,T} = 2G_{f,T}B_T\left(e^{-B_T\delta} - e^{-2B_T\delta}\right)$ $\frac{G_{f,T}}{G_{f0}} = \frac{1}{2}\tanh\left[-c_2\left(\frac{T}{T_g} - c_3\right)\right] + \frac{1}{2}$ $\frac{B_T}{B_0} = \frac{\left(1-d_1\right)}{2}.\tanh\left[-d_2\left(\frac{T}{T_g} - d_3\right)\right] + \frac{\left(1+d_1\right)}{2}$ $G_{f0} = 0.308\beta_w^2\sqrt{f_t}$ $\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}}$ $c_2 = 3.21;\ c_3 = 1.31;$ $d_1 = 0.485;\ d_2 = 14.1;\ d_3 = 0.877;$ $B_0 = 10.4\left(\text{for 15 MPa} \le f_c' \le 50\ \text{MPa}\right)$

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Table 7: Thermal conductivity of different insulation materials at elevated temperatures

Tempe -rature	Promatect H	Promatect L 500	Promasil 950	CAFCO 300	Carboline Type 5-MD	Tyfo WR AFP	TB tunnel fireproofing
(°C)	(W/m°C)	(W/m°C)	(W/m°C)	(W/m°C)	(W/m°C)	(W/m°C)	(W/m°C)
20	1.00	1.00	1.03	0.07	0.12	0.19	0.10
100	1.12	1.01	1.15	0.08	0.09	0.19	0.11
200	1.24	1.05	1.31	0.06	0.06	0.14	0.09
300	1.35	1.01	1.47	0.06	0.06	0.14	0.09
400	1.35	1.02	1.63	0.10	0.07	0.13	0.14
500	1.35	0.94	1.79	0.16	0.07	0.17	0.21
600	1.35	1.73	1.95	0.20	0.07	0.20	0.28
700	1.35	1.75	2.11	0.25	0.08	0.23	0.34

Table 8: Heat capacity of different insulation materials at elevated temperatures

Tempe -rature	Promatect H	Promatect L 500	Promasil 950	CAFCO 300	Carboline Type 5-MD	Tyfo WR AFP	TB tunnel fireproofing
(°C)	(MJ/°C m^3)	(MJ/°C m^3)	(MJ/°C m^3)	(MJ/°C m^3)	(MJ/°C m^3)	(MJ/°C m^3)	(MJ/°C m^3)
20	0.38	0.71	0.19	1.20	0.90	0.31	0.86
100	0.60	0.81	0.21	1.63	1.62	0.38	1.16
200	0.69	0.82	0.21	1.32	1.21	0.41	0.94
300	0.72	0.81	0.21	1.33	1.12	0.41	0.95
400	0.73	0.80	0.21	0.43	0.76	0.33	0.31
500	0.73	0.78	0.20	0.54	1.13	0.44	0.39
600	0.74	0.77	0.20	0.49	1.53	0.45	0.35
700	0.72	0.75	0.20	0.39	1.62	0.39	0.28

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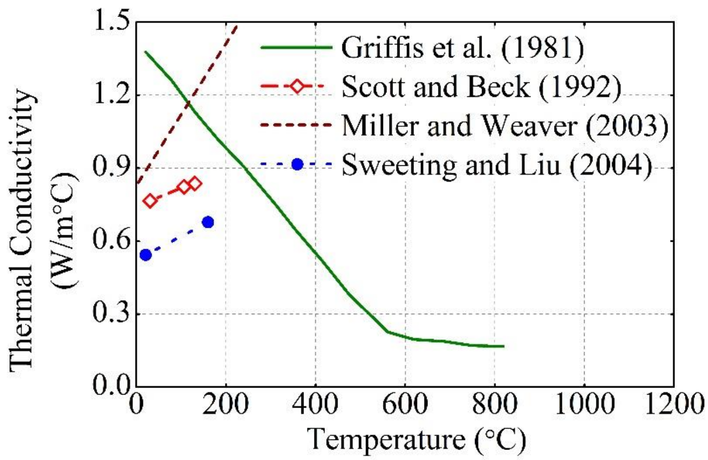
Figures



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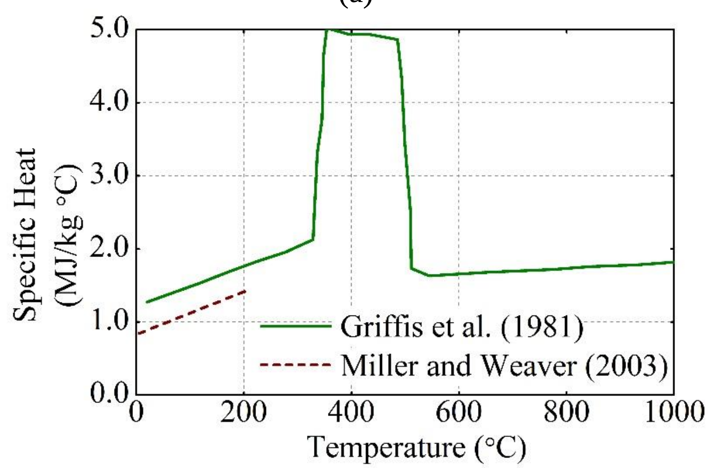

(b)

Figure 1: Thermal properties of FRP at elevated temperature: (a) Thermal conductivity and (b) specific heat

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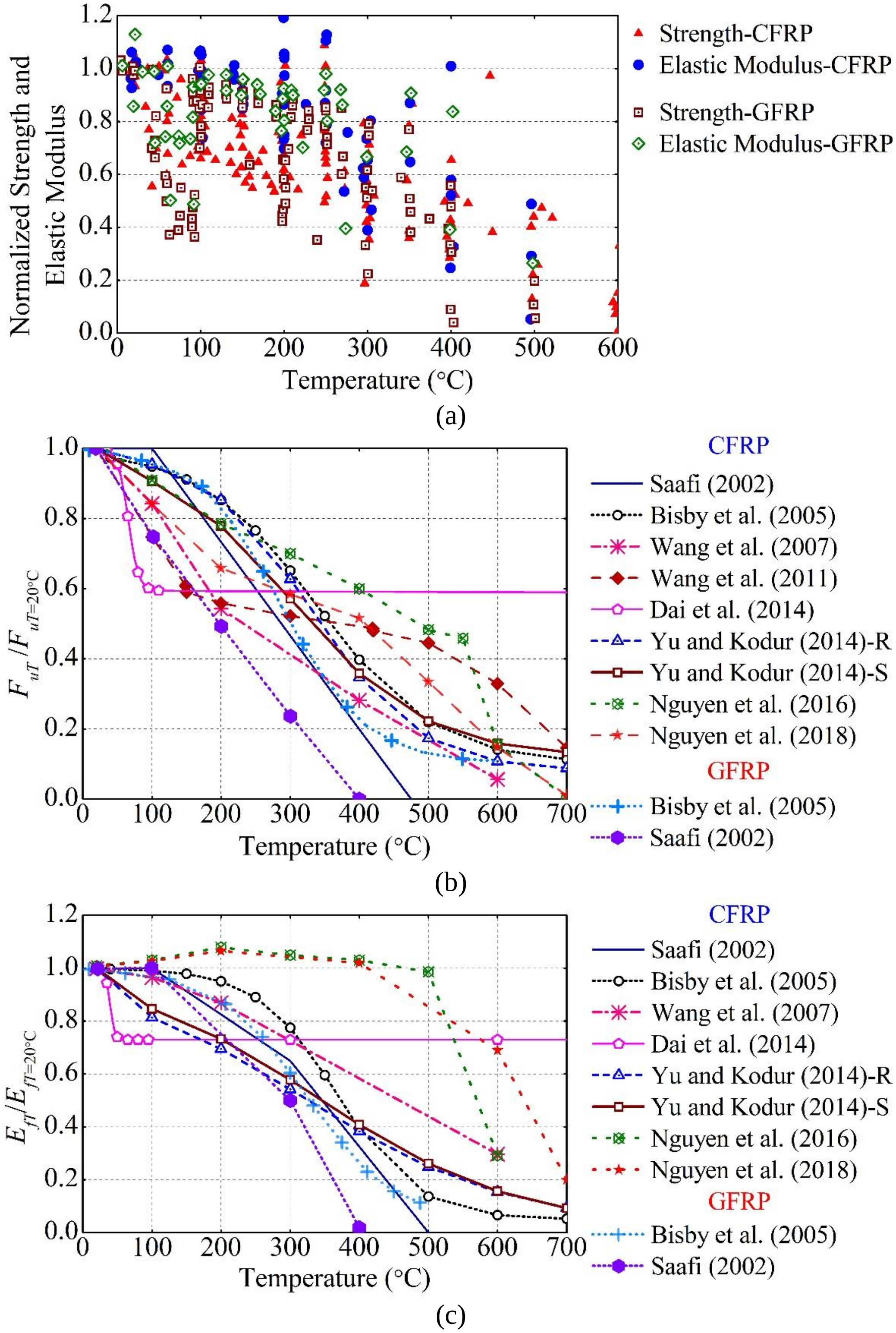

Figure 2: Normalized strength and elastic modulus properties of CFRP and GFRP at elevated temperatures: (a) test data (b) relations for tensile strength (c) relations for elastic modulus

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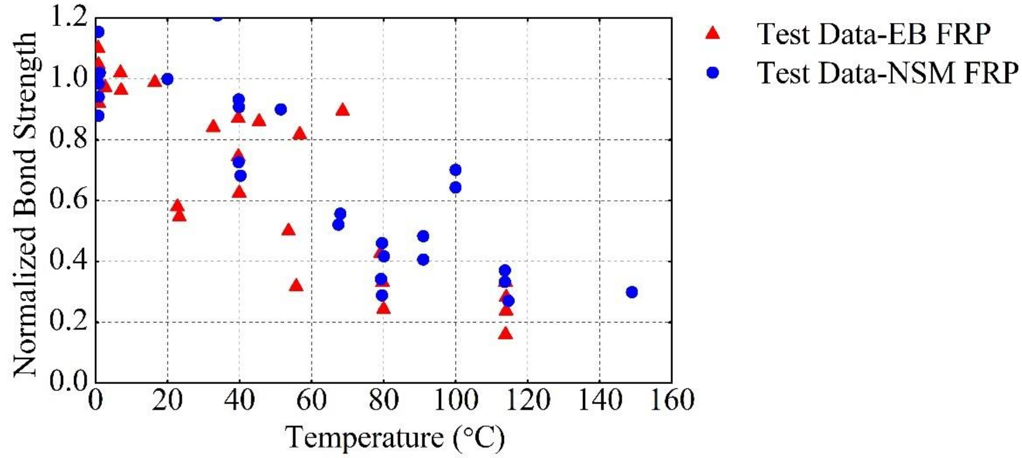
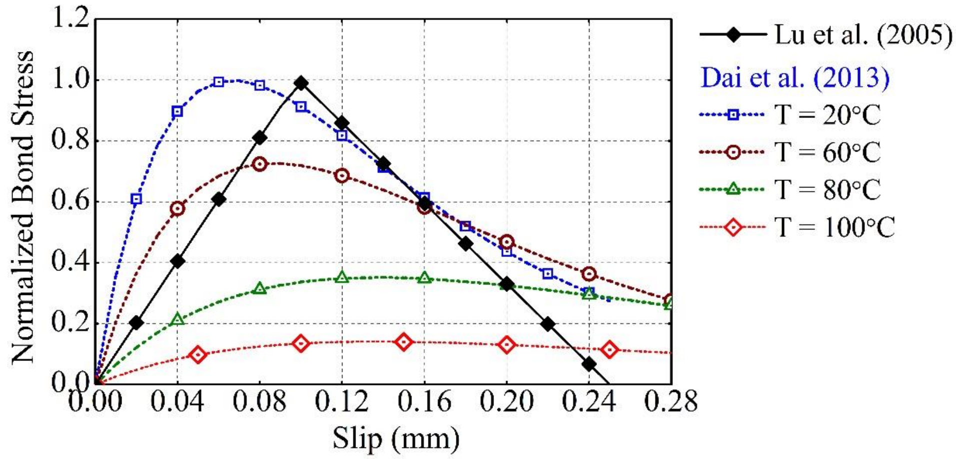
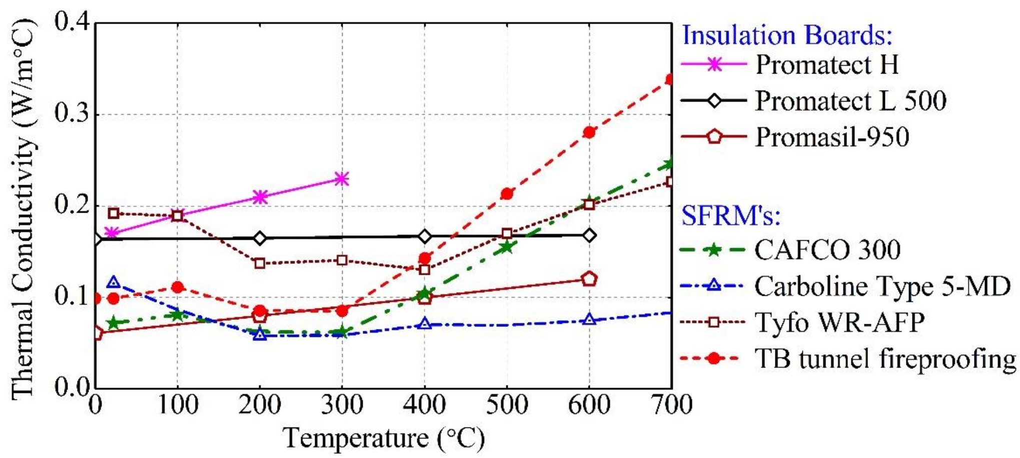


(a)



(b)

Figure 3: Normalized bond strength of EB and NSM CFRP at elevated temperature: (a) test data (b) bond stress-slip relations



(a)

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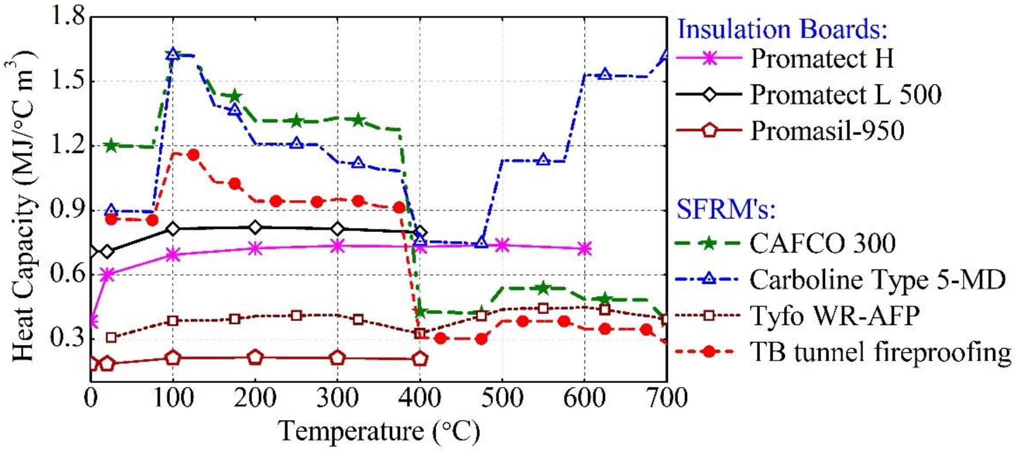

(b)

Figure 4: Thermal properties of different fire insulation materials at elevated temperature: (a) thermal conductivity and (b) heat capacity

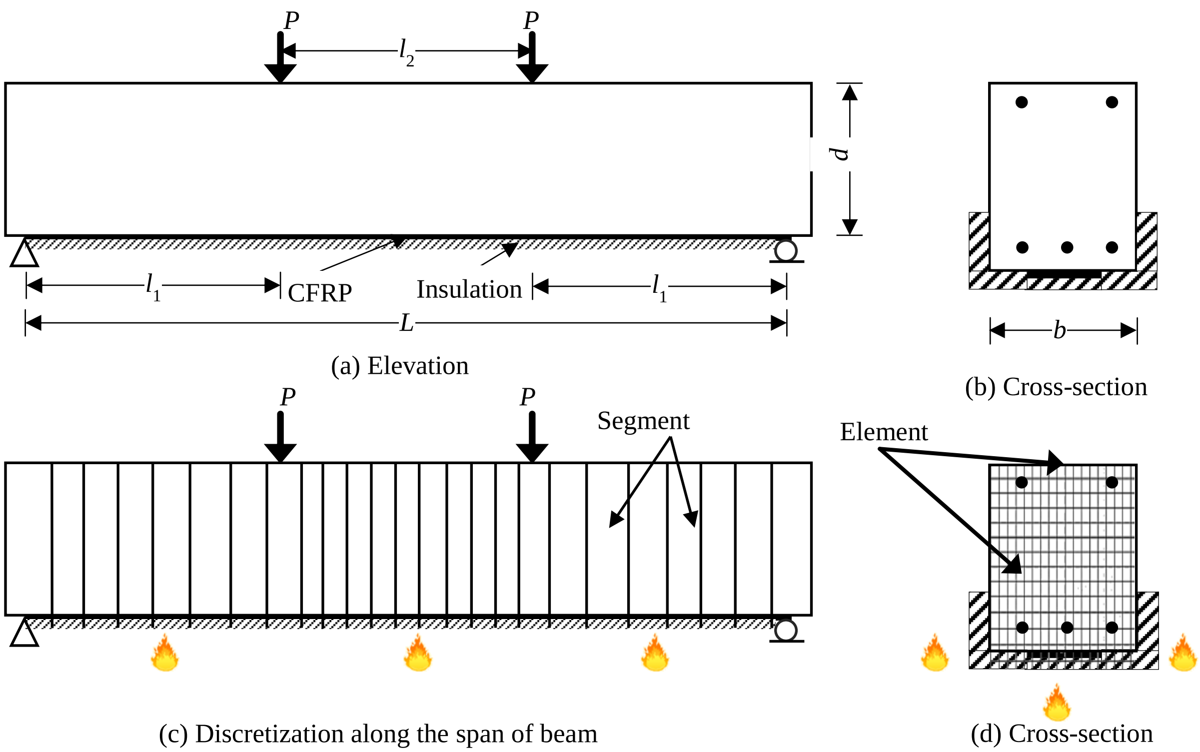

Figure 5: Typical beam layout and discretization of beam into segments and elements

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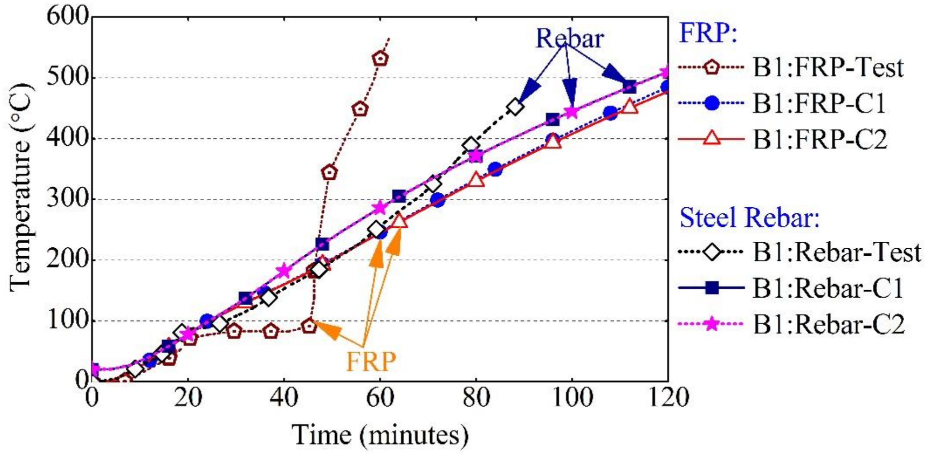
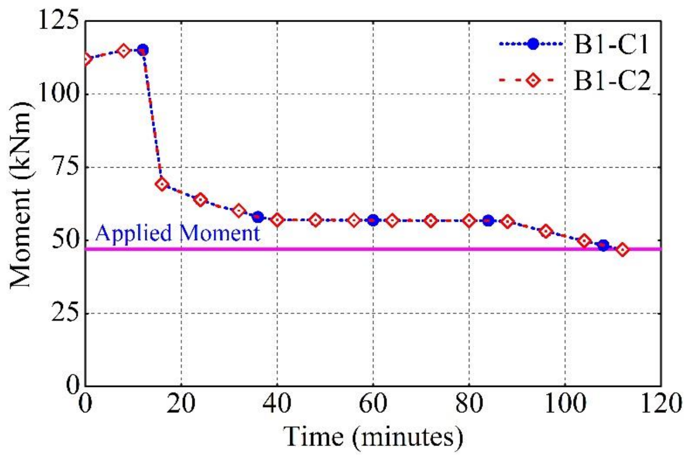


Figure 6: Effect of temperature dependence of thermal properties of FRP on temperature rise in steel rebar and FRP-concrete interface in beam B1



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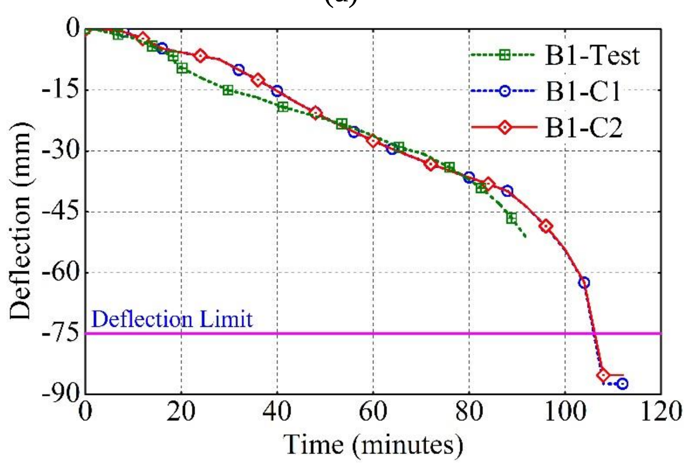


(b)

Figure 7: Effect of temperature dependence of thermal properties of FRP on degradation of moment capacity and deflection in beam B5: (a) degradation of moment capacity and (b) deflection

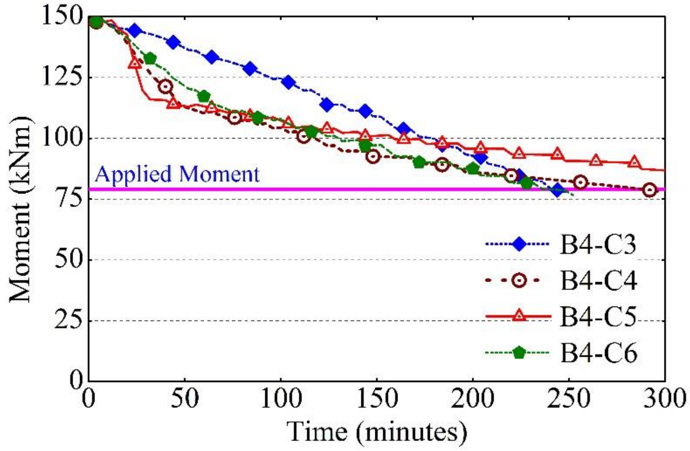
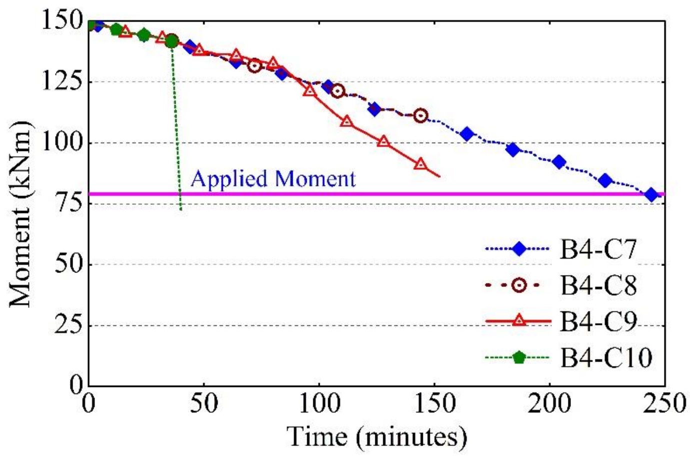


Figure 8: Effect of different temperature dependent strength and elastic modulus relations of FRP on degradation in moment capacity of beam B4



(a)

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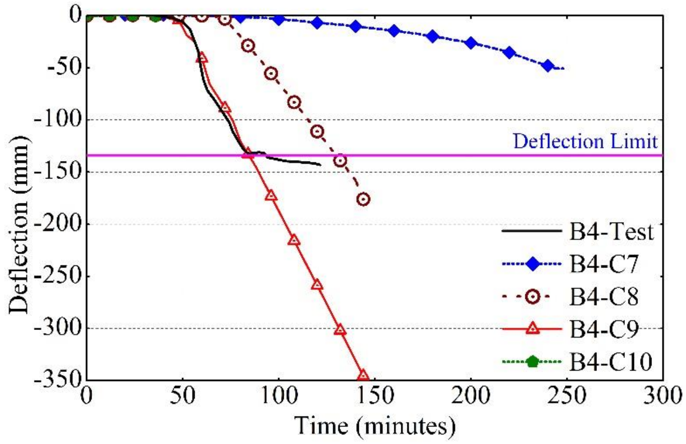

(b)

Figure 9: Effect of different bond-slip relations on degradation in moment capacity of beams and deflection of beam B4: (a) degradation of moment capacity and (b) deflection

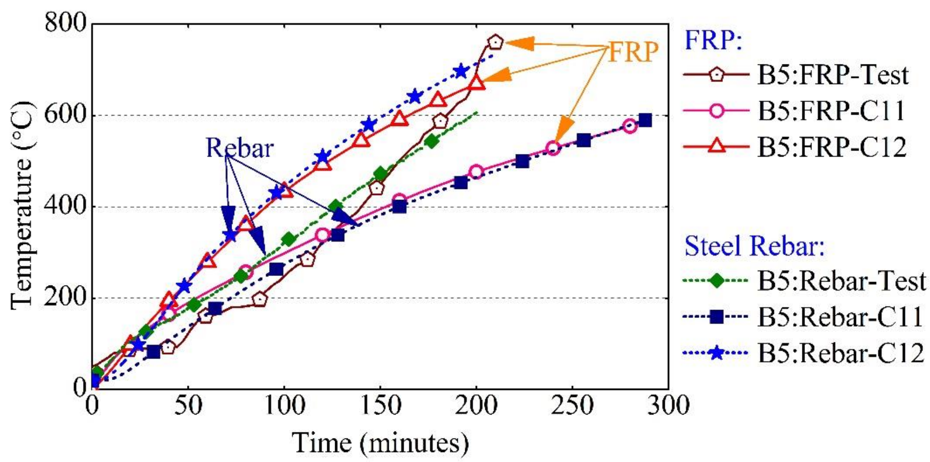

Figure 10: Effect of temperature dependence of thermal properties of fire insulation on temperature rise in steel rebar and FRP-concrete interface in beam B5

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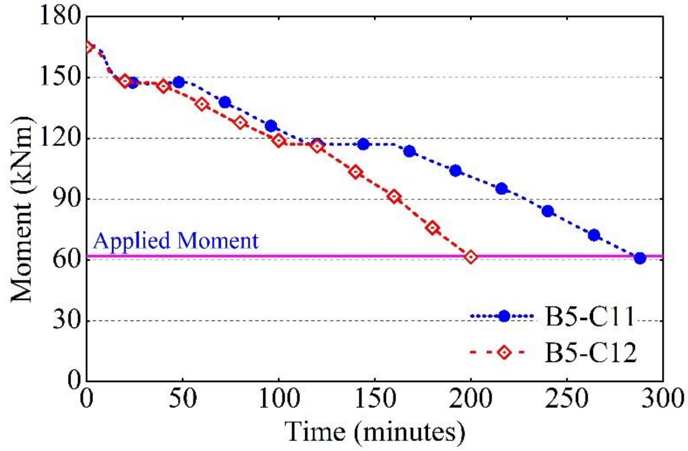
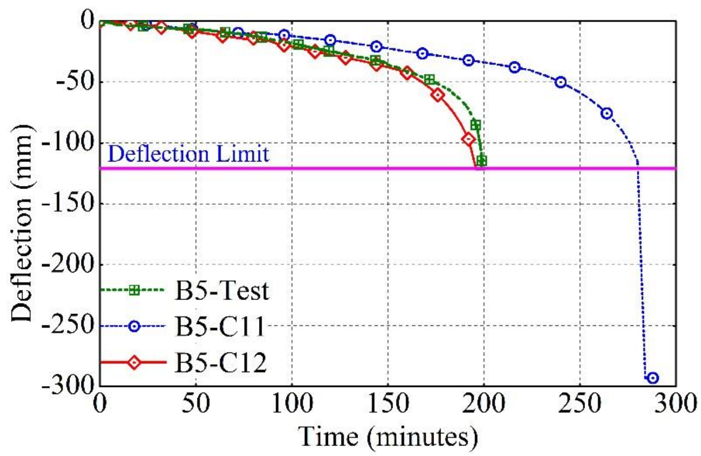


(a)



(b)

Figure 11: Effect of temperature dependent thermal property variation in fire insulation on: (a) degradation of moment capacity and (b) deflection of beam B5