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Performance of RC T-Beams Externally Strengthened with CFRP Laminates under Elevated Temperatures

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ABSTRACT

This paper presents a numerical study that investigates the performance of reinforced concrete (RC) T-beams externally strengthened with carbon fibre reinforced polymer (CFRP) plates when subjected to fire loading. A finite element (FE) model is developed and a coupled thermal-stress analysis was performed on a RC beam externally strengthened with a CFRP plate tested by other investigators. The spread of temperature at the CFRP-concrete interface and reinforcing steel, as well as the mid-span deflection response is compared to the measured experimental data. Overall, good agreement between the measured and predicted data is observed. The validated model was then used in an extensive parametric study to further investigate the effect of several parameters on the performance of CFRP externally strengthened RC beams under elevated temperatures. The variables of the parametric study include applying different fire curves and scenarios, different applied live load combinations as well as the effect of using different insulation schemes, types and thicknesses. Several observations and conclusions were drawn from the parametric investigation. It could be concluded that successful FE modeling of this structural member when exposed to thermal and mechanical loading would provide a valid economical and efficient alternative solution to the expensive and time consuming experimental testing.

Keywords: Finite element; thermal-stress analysis; CFRP; Fire; Elevated temperatures; ASTM E119; ; T-beams; Strengthening

INTRODUCTION

Carbon Fibre reinforced polymer (CFRP) material became recently as one of the state-of-the-art materials in rehabilitating and strengthening reinforced concrete (RC) structural members due to its superior properties in terms of high strength to weight ratio, resistance to corrosion, lightweight, and ease of installation [1, 2]. However, one of the shortcomings of such material is its poor performance when externally attached to the soffit of the beam in the event of fire. This is due to the complex natural composition of the composite matrix and low transition glass temperature, T_g of about 65°C for most CFRP materials used in strengthening. Furthermore, the FRP mechanical properties (stiffness and strength) are known to experience large degradation when subjected to elevated temperature [3-7]. Such large losses of strength and stiffness in the externally bonded strengthening system would cause localized failures that might initiate a progressive collapse in the strengthened structural member. Thus, further research studies are warranted in this field to examine the behavior of FRP strengthening systems when used as retrofitting supplements in buildings and subjected to thermal loading effects [8].

In an event of fire, fire can develop fully rapidly and reach very high temperature levels. Such high temperature levels are sufficient to cause significant damage to the externally bonded

FRP strengthening materials along with their associated bonding adhesives [5, 9- 10]. Thus, engineers and designers should take into account the severity and complexity of a possible fire event (scenario) in the analysis and design of such strengthening systems. So far, there have been few experimental programs conducted in the previous years on the fire performance of externally strengthened RC beams with FRP laminates. This is due to the expensive experimental set-ups and shortage of specialized facilities and trained personnel [11]. Few researchers investigated the performance of RC structural members strengthened with FRP laminates under elevated temperatures [12-17]. Deuring [12] performed fire tests on six RC beams that were externally strengthened with different systems. The experimental matrix in his program consisted of a control unstrengthened specimen, a strengthened specimen with an adhesive bonded steel plate, and another four specimens strengthened with CFRP plates. Two of the CFRP plated beams were tested without insulation and two were protected with insulating plates of different thicknesses. It was shown that the unprotected FRP-strengthened beam specimen achieved a fire endurance of 81 minutes. On the other hand, the identical beam specimen with the CFRP protected with 40 mm calcium/silicate insulation, achieved a fire endurance of 146 minutes. Deuring [12] experimental program showed the need for insulating the externally bonded FRP system since the bond action between the CFRP and concrete materials was lost within the first few minutes of fire exposure for the unprotected beam specimens.

Blontrock et al. [14] tested ten RC beams strengthened with CFRP plates and protected with different insulation boards. In this experimental program, several insulation parameters were investigated, including insulation board thickness, length, location, and bonding method. Initially, two RC beams were tested at ambient temperatures monotonically up to failure to serve as a benchmark. One of the RC beams was unstrengthened while the other was strengthened with a CFRP plate attached to the beam's soffit. In addition, two unprotected and unstrengthened beams as well as six strengthened and protected beams were loaded to full service load and tested under the standard ISO834 [18] fire curve exposure. It was observed that the best fire endurance can be achieved if U-shaped fire protection insulation is applied to both the soffit and vertical sides of the beams.

Williams et al. [3] conducted in another experimental program, two full size experiments on insulated RC T-beams (Beam 1 and Beam 2) strengthened with CFRP plates attached to the soffit of the beams under a sustained uniformly distributed service load and exposed to the ASTM E119 [19] standard fire curve. The RC beams were identical but the Vermiculite/Gypsum (VG) insulation thickness was varied between 25 mm and 38 mm. The objective of their study was to evaluate the performance of the strengthened beams and demonstrate that the CFRP strengthening system can maintain its structural integrity if provided with sufficient insulation thickness. The strengthened RC beams were able to stand the ASTM E119 fire exposure up to 4 hours. The results of this experimental program indicated that a properly insulated system can maintain the FRP and reinforcing steel temperatures below a certain critical temperature value that preserved their structural integrity.

Hawileh et al. [20] developed a 3D nonlinear finite element (FE) model that was based on the work of Williams et al. [3] using the FE software, ANSYS 11.0 [21]. The developed FE model was able to accurately predict the fire performance of the CFRP externally strengthened RC beam that was protected with a 25 mm thick VG insulation layer and tested under the ASTM E119 [19] fire curve. The predicted FE thermal and mechanical results were in close agreement with the measured experimental data by Williams et al. [3].

This study aims to predict the fire performance of strengthened RC beams with externally bonded CFRP plates under several design scenarios. The developed FE model was first validated further by predicting the experimental results of another strengthened RC T-beam specimen (Beam 2) with a CFRP plate and protected with a 38 mm VG insulation layer [3]. The developed FE model accounts for the temperature-dependent constituent material properties of the different materials associated with the CFRP strengthened RC beam. A coupled thermal-stress simulation environment is performed using the commercial FE software, ANSYS 11.0 [21]. The accuracy of the analysis was checked by comparing the predicted temperature results and mid-span deflection history from the FE model against the measured test data of Williams et al. [3]. A parametric study was then performed to investigate the effect of applying different fire curves and scenarios, different applied live load combinations, as well as the effect of using different insulation schemes, types and thicknesses on the fire performance of strengthened RC beams. Several conclusions and observations were drawn and discussed. Successful FE modeling of such highly nonlinear problems would provide a valid insight on the behavior of strengthened RC structural members under fire loading with a tremendous reduction in the associated costs and time of conducting experimental programs.

THERMAL –STRESS ANALYSIS METHODOLOGY

The general methodology used in performing thermal-stress analysis is itemized and explained in the following steps:

1. Developing a 3D FE model of the tested beam specimen. The FE model integrates the same geometric features, appropriate material properties (concrete, reinforcement steel, CFRP

plate, and insulation), loading set-up and boundary conditions. Hence, two FE models consisting of thermal and structural elements are required to enable the coupled thermal-structural analysis.

2. Applying thermal loads (in terms of temperature-time curves) to the beam's soffit and vertical side surfaces. The applied temperature-time curve is based on the average measured temperature in the testing facility.
3. Validate the resulting temperature distribution of the thermal FE model by comparing the predicted and measured temperature at various key points within the RC beams cross sections.
4. Applying a sustained uniformly distributed load at the top face of the structural beam model. This will be the first load step in the structural analysis that will be followed by applying nodal temperature loads from the results of the thermal analysis at specified time loads (load steps and sub-steps) from Step 2 above.
5. Compare the predicted and measured mid-span deflection for the entire fire exposure to evaluate the performance of the developed FE model.

FINITE ELEMENT MODEL

Geometry and element types

The presented FE model has the same geometry, configuration and dimensions of the second beam specimen (Beam 2) that has an insulation thickness of 38 mm and tested by Williams et al. [3]. The RC T-beam was designed with typical dimensions as those used in regular buildings to represent actual fire scenarios. In addition, the beam was loaded up to its service load conditions while exposed to the ASTM E119 fire curve. The tested RC beam had a depth, flange width and

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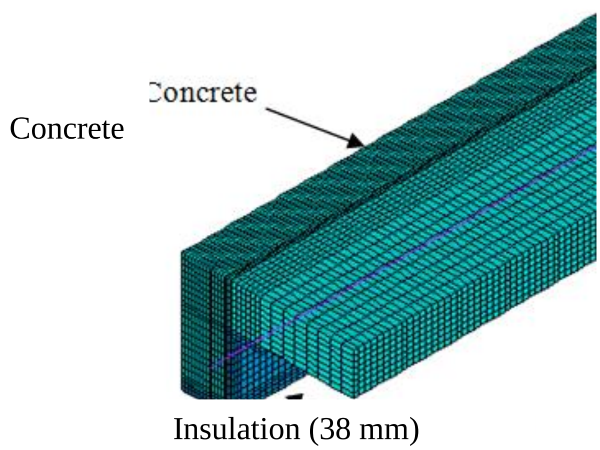
web thickness of 400, 1220 and 300mm, respectively. Furthermore, the total length of the beam was 3900 mm. It should be noted that the length of the members were chosen to satisfy the ASTM E119 standard [19], which states that the length of the member should be more than 3660 mm. The main steel flexural reinforcement was two 20 mm diameter bars. The soffit of the T-beam was strengthened with a CFRP plate that has a thickness and width of 1.2 and 100 mm, respectively. The insulation system covered the exterior face of FRP along with the three sides of the T-beam web were insulated with a 38 mm layer of VG insulation. The VG insulation was extended to a distance of 125 mm into the bottom underside surfaces of the flange, along the entire beam length. Clear concrete cover to the stirrups was 40 mm. Due to the symmetry of the loading, boundary conditions, and materials, a quarter model was built and analyzed using ANSYS [21]. Figure 1 shows the different views of the FE quarter model as well as the locations of the thermocouples used in the experimental program.



(a) Isoperimetric view

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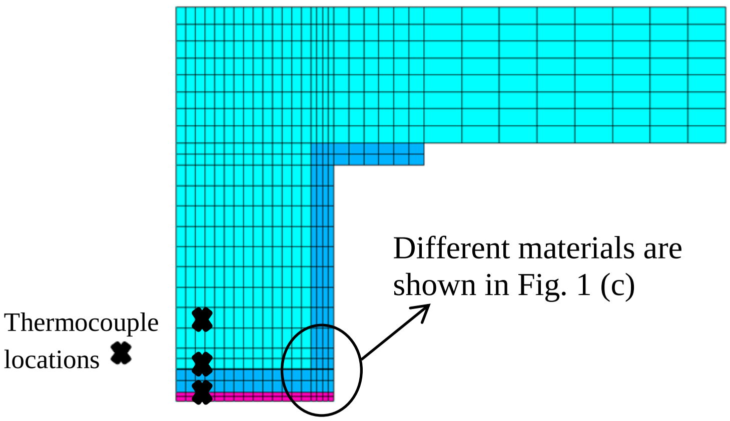
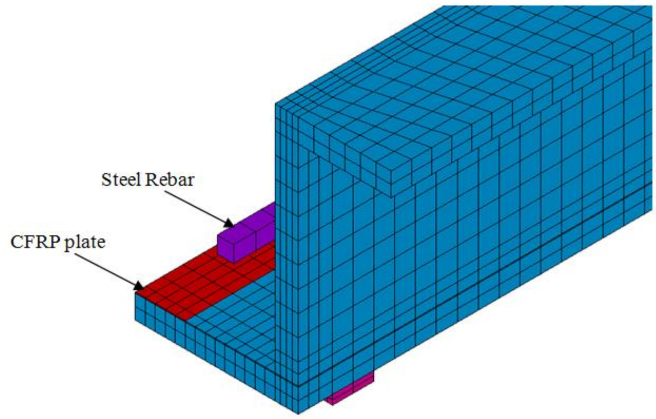


(b) Cross sectional view



(c) Different materials used in the Quarter model

Figure 1. Different views of the FE quarter model

Various thermal and structural element types were used from the ANSYS elements library [21] in the development of the present FE model. The thermal elements used were SOLID70 and LINK33 [21] and the structural elements used in the stress analysis were SOLID65, SOLID46, SOLID45, and LINK8 [21]. Mesh sensitivity analysis [10] was performed to obtain the optimum element size shown in Fig. 1. Based on the results of the mesh sensitivity analysis, the total number of elements in the adopted model is 55000.

Since modeling such phenomenon requires thermal and structural modeling, the thermal elements (used in the thermal simulation) were converted to structural elements in the stress analysis as follows:

- For the concrete material, the thermal 3D SOLID70 element is converted to the 3D structural element SOLID65 (3-D 8-Node Reinforced Concrete Solid).
- For the CFRP material, the thermal SOLID70 element is converted to the 3D composite structural element SOLID46 (3-D 8-Node Layered Structural Solid).

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- For the insulation material, the thermal SOLID70 element is converted to the 3D structural element SOLID45 (3-D 8-Node Structural Solid).
- For the steel reinforcement material, the thermal LINK33 element is converted to structural element LINK8 (3-D 2-Node Structural bar).

Material Properties at Room and Elevated Temperature

In order for the FE model to accurately predict the behavior of the tested RC beam, temperature-dependent thermal and mechanical material properties are required as inputs for the thermal-stress analysis. The developed FE model takes into account the different nonlinear material properties of the materials included in the beam specimen. The nonlinearity is mainly due to the natural compositions of such materials and the changes they experience when exposed to elevated temperature. Table 1 provides a list of the mechanical and thermal properties for concrete, steel, CFRP, and insulation materials at room temperature. On the other hand, the normalized stiffness (modulus of elasticity), variation of concrete compressive strength, steel reinforcement ultimate strength, thermal conductivity, and specific heat with elevated temperatures for the different materials needed for the FE simulation are shown in Figs. 2-6.

The behavior of the concrete and steel materials was assumed to follow the proposed equations and charts provided in Eurocode [22]. Still, little research has been published on the mechanical and thermal properties of FRP materials. Griffis et al. [23] performed tests on a carbon/epoxy FRP used in aerospace applications, while Park et al. [24] studied the thermal and mechanical properties of gypsum board at high temperature. In this analysis, material properties proposed by Blontrock et al. [25] and Bai et al. [26] are adopted.

Table 1 Mechanical and thermal material properties at room temperature [3, 27]

Material	E_{xo} (MPa)	E_{yo} (MPa)	E_{zo} (MPa)	K_o (W/mm.K)	C_o (J/kg.K)	μ_{xo}	$\mu_{y,z}$	α_{xo} (1/°C)	ρ_o (Kg/mm^3)
Concrete	30200	-	-	2.7×10^{-3}	722.8	0.20	-	6.08×10^{-6}	2.40×10^{-6}
Steel bars	210000	-	-	5.2×10^{-2}	452.2	0.30	-	6.00×10^{-6}	7.86×10^{-6}
CFRP	228000	10000	10000	1.3×10^{-3}	1310	0.28	0.0122	-0.90×10^{-6}	1.60×10^{-6}
VG Insulation	2100	-	-	2.5×10^{-4}	1654	0.30	-	1.70×10^{-5}	2.69×10^{-7}

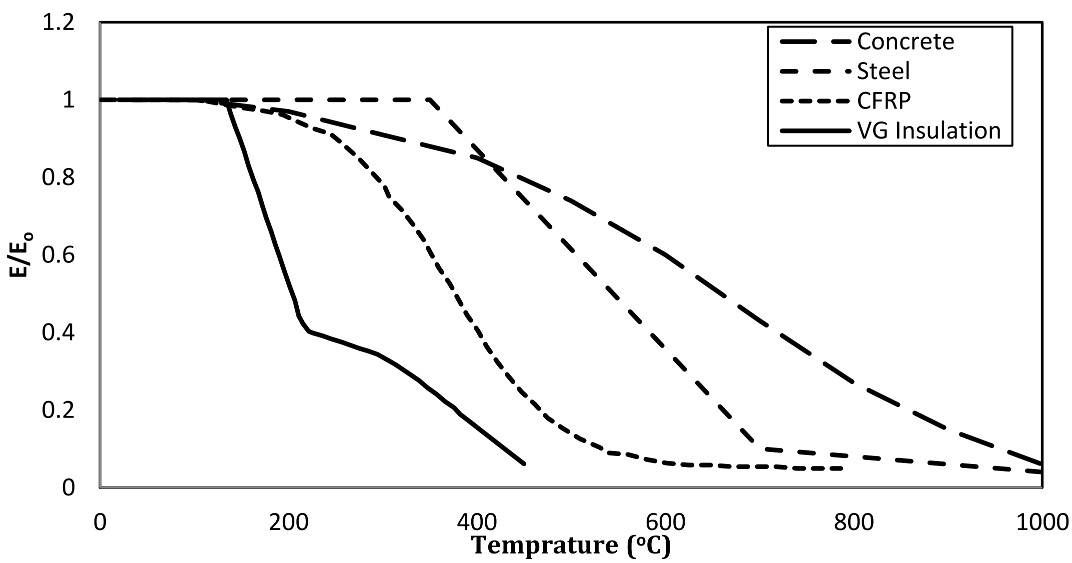

Figure 2. Variation of normalized stiffness with temperature [3]

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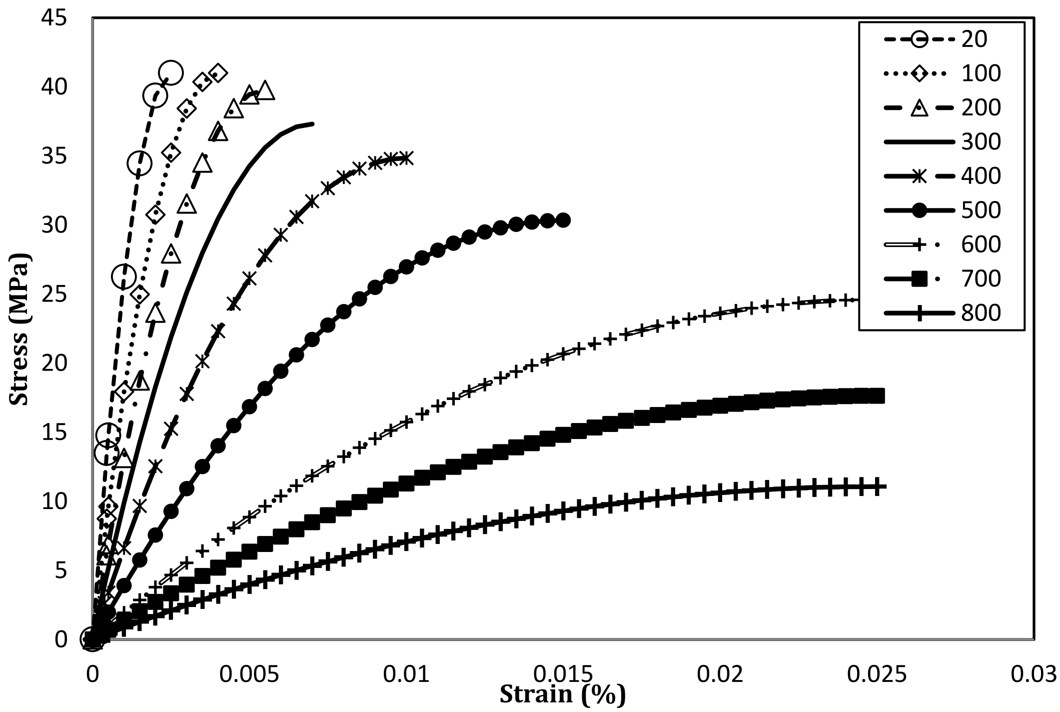

Figure 3. Variation of concrete compressive strength with temperature [22]

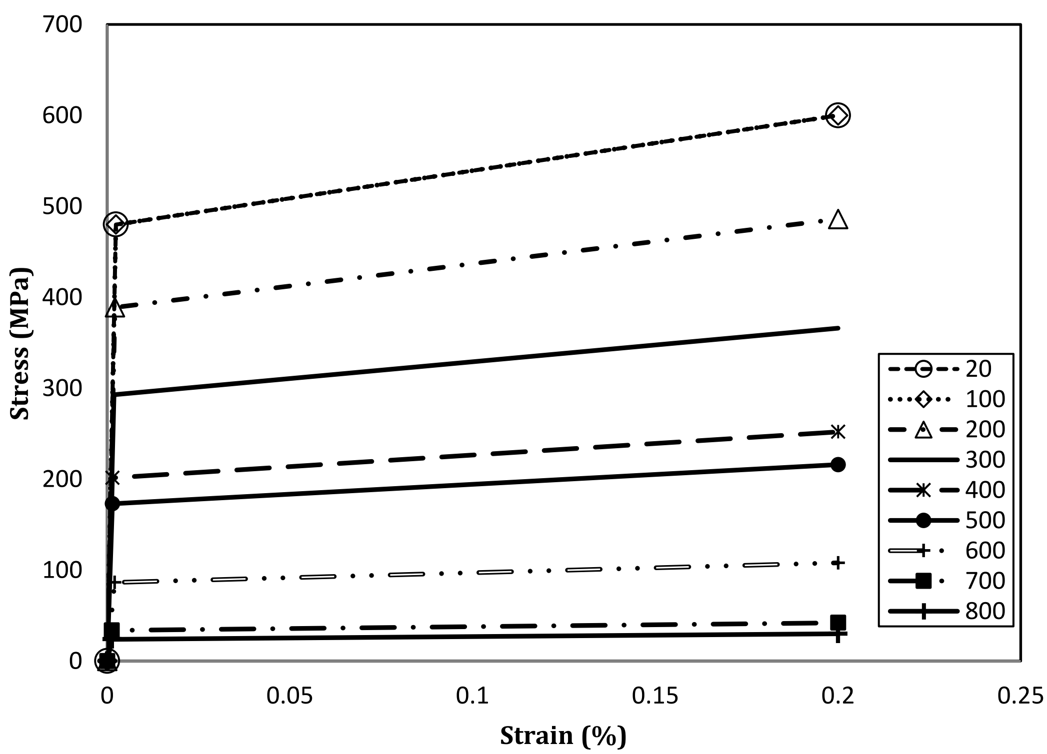

Figure 4. Variation of steel reinforcement strength with temperature [22]

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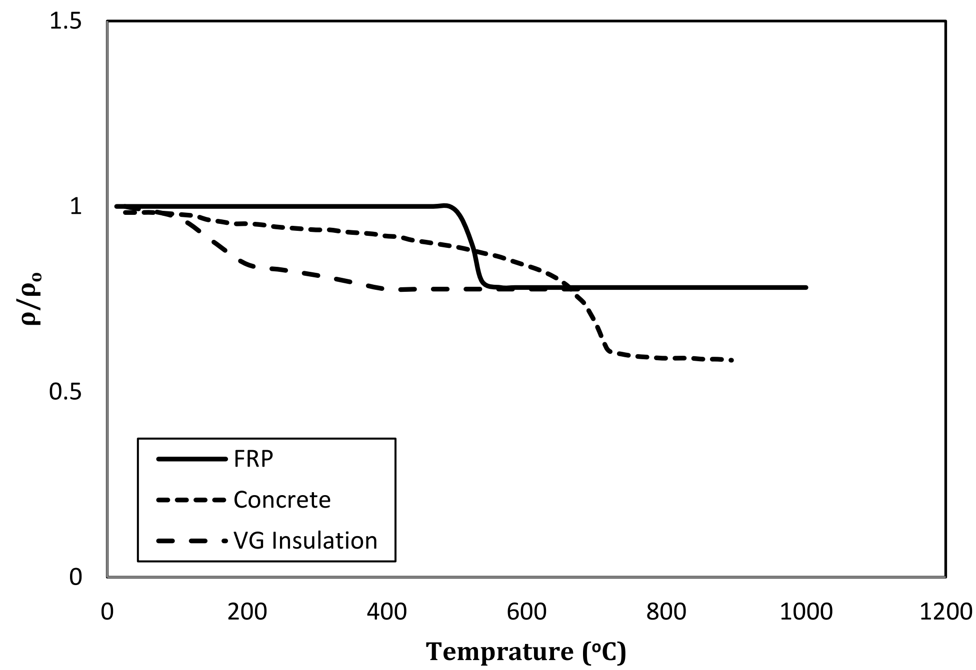
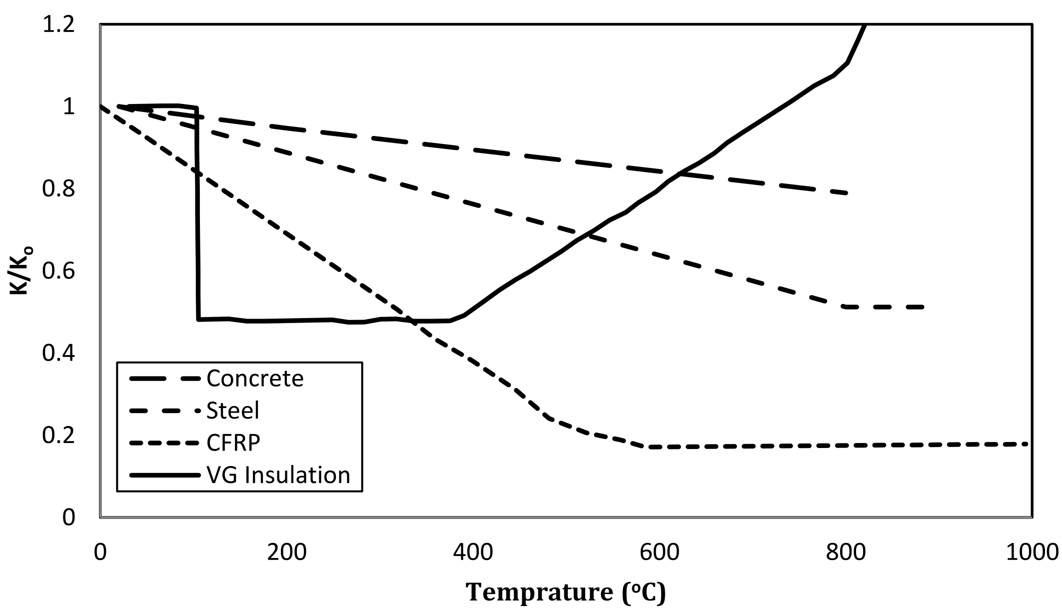
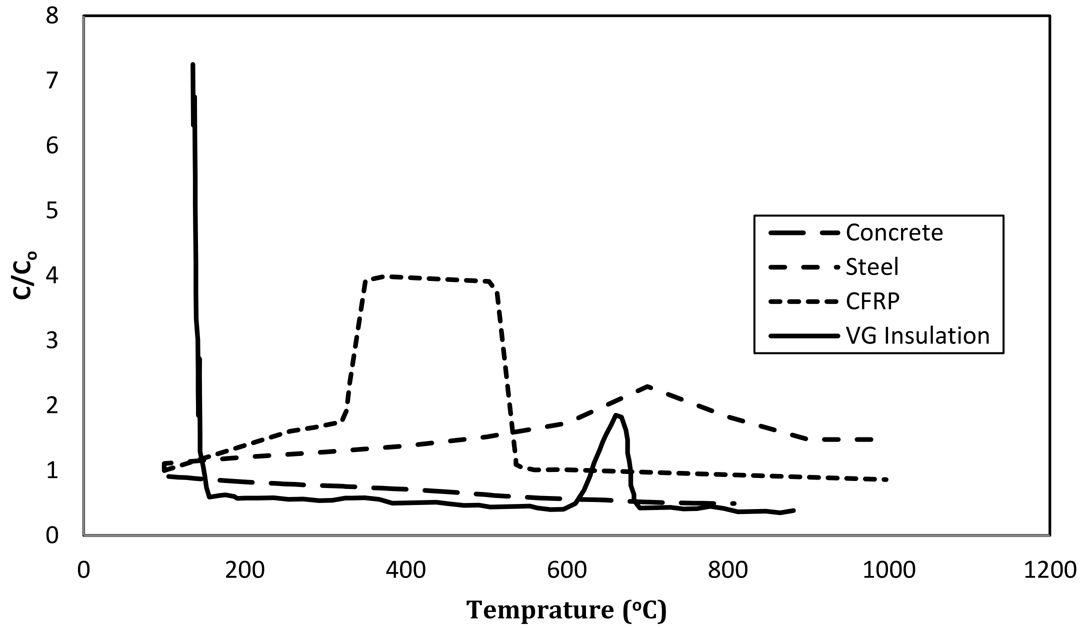


Figure 5. Variation of normalized density with temperature [4]



(a) Thermal conductivity



(b) Specific heat

Figure 6. Variation of normalized thermal conductivity and specific heat with temperature [4]

Compressive multi-linear stress-strain curves as a function of elevated temperatures were used to simulate the compressive behavior of the concrete material. Figure 3 shows the temperature dependent compressive stress-strain curves produced according to the reduction factors given by Zhou and Vecchio [27]. The concrete tensile rupture stress is taken as $0.6\sqrt{f_c'}$ where f'_c is the ultimate compressive strength of concrete. After the concrete material reaches its tensile peak rupture stress, a tensile stiffness multiplier of 0.6 is used to simulate a sudden drop of the tensile stress to 60% of the rupture stress, followed by a linearly descending segment to zero stress at a strain value of six times the strain corresponding to the concrete tensile stress.

The nonlinear behavior of the concrete material was based on the nonlinear constitutive concrete material model of Willam and Warnke [28]. This model takes into account the nonlinearity of concrete in tension by allowing the concrete elements to crack upon reaching their ultimate tensile strength and to crush once they reach their maximum compressive strength.

Moreover, the open and close crack shear transfer coefficients, β_t and β_c are supplementary parameters required for the concrete constitutive material model [28]. Typical shear transfer coefficients are taken as zero when there is a total loss of shear transfer representing a smooth crack and 1.0 when there is no loss of shear transfer representing a rough crack [21]. The values of β_t and β_c in the presented developed model are assumed to be 0.4 and 0.7, respectively.

Loads, Boundary Conditions & Convergence of Nonlinear Solution

The beams were exposed to the thermal ASTM E119 fire curve in a full scale furnace at the Fire Risk Management laboratory of the Canadian Institute for Research in Construction's (IRC). The ASTM E119 fire curve could be presented using the following equation [19]:

$$T = 750[1 - e^{-3.79553\sqrt{t}}] + 170.41\sqrt{t} + T_o \qquad (1)$$

where,

T_o is the initial room temperature take as 25 °C

t is the time in hours

The thermal analysis started by applying the ASTM E119 temperature-time curve to the nodes of the bottom and sides of the FE model.

In the structural analysis phase, the symmetrical boundary conditions were simulated by inserting roller restrains at each node located perpendicular to each plane of symmetry. Then, a sustained uniformly distributed load of 34 kN/m was applied to the beam's top face of the structural model. This mechanical constant loading was simulated in the FE model by applying a pressure of 0.0557 MPa to the top face of the beam.

In this study, automatic time stepping option is turned on to predict and control the load step sizes. At the end of each load step, convergence is achieved by the Newton-Raphson equilibrium iterations. The convergence of the thermal solution is achieved when the temperature difference at each node between the equilibrium iterations is less than 0.5 °C (32.9 °F). As for the structural simulations, the solution is also converged by the Newton-Raphson equilibrium iterations. In addition, a force convergence tolerance limit value of 0.1 (typical range 0.05 to 0.2) was used to achieve convergence of the solution [10, 20].

RESULTS AND DISCUSSION

In order to examine the validity of the developed FE model, the results were compared with the measured experimental data of the tested two beam specimens (Beam 1 and Beam 2) conducted by Williams et al. [3]. In a previous study [20], the authors validated their model by correlating the predicted FE results with the obtained experimental data of Beam 1. The results of Beam 1 will be also summarized in this section. It should be noted that the difference between the tested RC beam specimens (Beam 1 and Beam 2) was in the thickness of the insulation layer. Beam 1 had an insulation thickness of 25 mm, while Beam 2 had an insulation thickness of 38 mm. In this study, another FE model is created in order to gain extra validation and confidence and the predicted results were compared with the obtained data of Beam 2. In both models, the temperature variation at different locations with the beam’s cross section, mid-span deflection, and time to failure are compared with the obtained experimental data. Then, the validation FE model of the first beam (Beam 1) was extended herein into a parametric study in which the effect of applying

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different fire curves and scenarios, different applied live load combinations as well as the effect of using different insulation schemes, types and thicknesses are investigated.

FE Model Validation:

Thermal Validation:

The thermal analysis of the tested T-Beams was compared against the thermal measured experimental data of Beam 1 and Beam 2, respectively. The thermocouples were located at the VG insulation/CFRP interface, CFRP/concrete interface, and at the flexural steel reinforcement as shown in Fig. 1 (b). Figure 7 shows a comparison between the measured and predicted FE temperatures for Beam 1 (Fig. 7a) and Beam 2 (Fig. 7b) at the different locations within the cross section of the tested beams. Figure 8 shows a similar comparison at the steel reinforcement location. It is clear from Figs. 7 and 8 shown that there is a good agreement between the measured and predicted temperature results at those locations during the entire fire exposure. Also, it can be seen from Fig. 7 that the FE results underestimates the temperature at the VG/CFRP interface. The measured temperature at the VG/CFRP interface seems to significantly increase after 75 min. The deviation between the predicted and measured temperature at the VG/CFRP interface can be explained by either a slight movement of the thermocouple or an initiation of some major cracks in the VG insulation near the embedded thermocouple [3]. Those cracks allowed the fire to penetrate near that location. Furthermore, it is noticed from Fig. 7 that the temperature at the VG/CFRP interface faced some disturbance in the first hour of fire exposure. This is due to the evaporation of water moisture from the VG insulation which is caused by the increase of thermal energy provided from the fire. Such phenomena may not be accurately simulated in the available FE packages due to its highly nonlinear characteristics and limitation of the elements capabilities.

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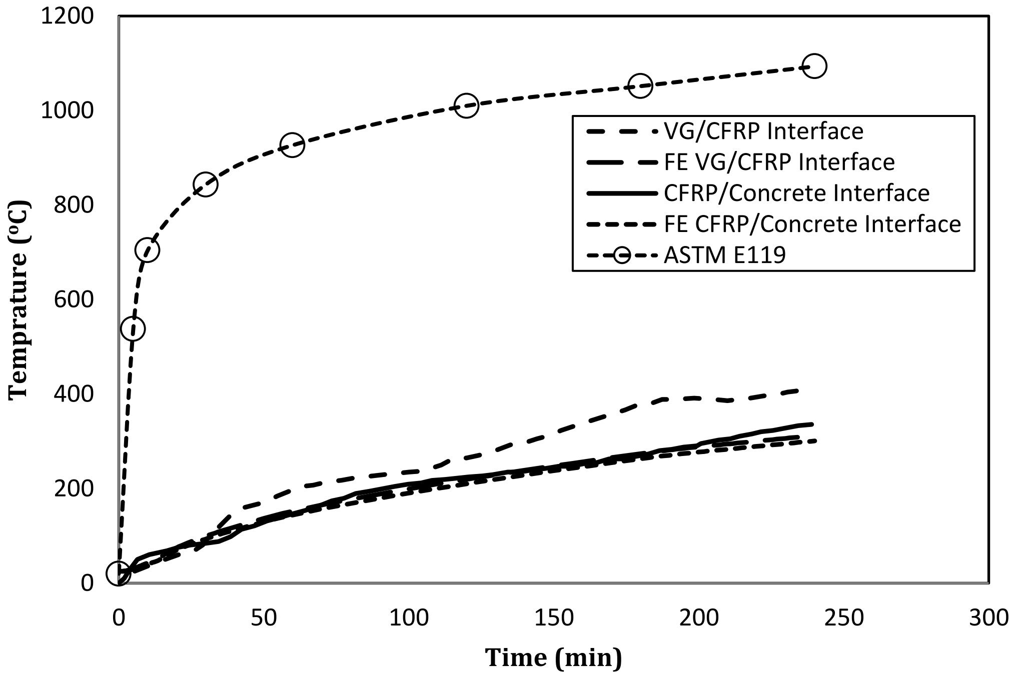
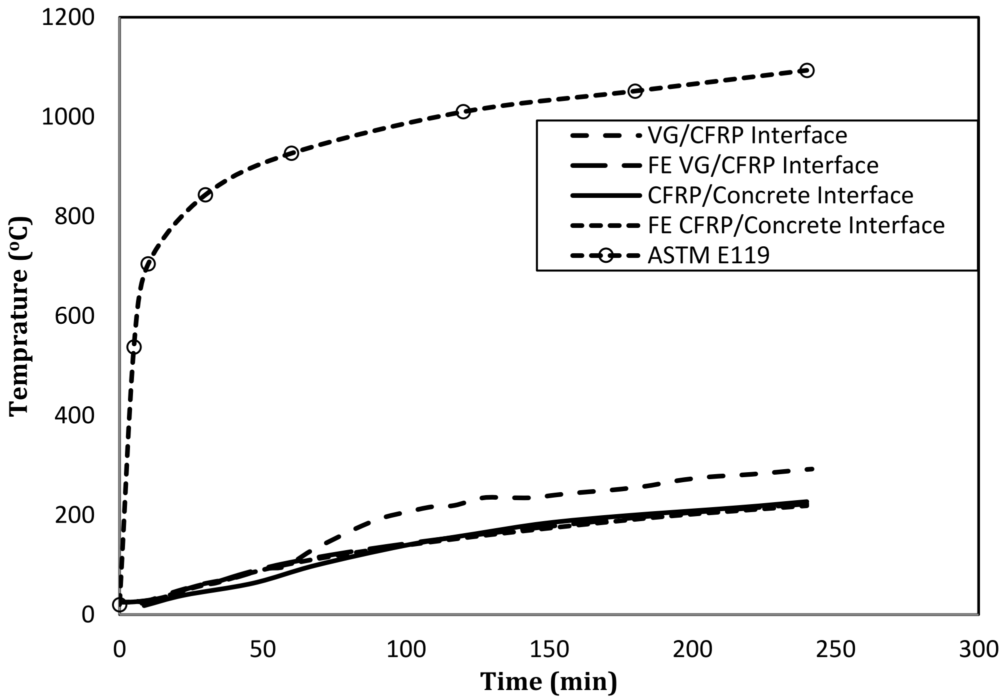


(a) Beam 1



(b) Beam 2

Figure 7. Comparison of the predicted FE and measured temperatures for Beams 1 and 2

In order for a tested specimen to pass the ASTM E119 testing procedure [19], the following three major criteria must be met during a fire test:

1. The structural member is able to withstand its applied loads.
2. The temperature of the reinforcing steel does not exceed 593 °C.
3. The average temperature increase at the unexposed face must be less than 140 º C and no individual point temperature on the unexposed side reach 180 °C above room temperature.

It was experimentally and numerically demonstrated that the tested RC beams (Beam 1 and Beam 2) were able to withstand the applied loading. In addition, Figure 8 shows that the temperature of the steel bars was kept below the threshold determined by the ASTM E119 standard (593 °C), thus passing the fire endurance criteria related to the temperature of the tension reinforcement. In addition, it can be seen that the temperature at the steel bars increased in proportion to time up to the end of the test. The steel temperature after 2 and 4 hours of heating was 116, 240 °C and 100 and 160 °C for Beam 1 and Beam 2, respectively. This implies that the temperature at the compression fibers of the RC tested beam was kept well below 140 ºC with no individual nodal temperature on the unexposed side reaching 180 °C above room temperature. Hence, the ASTM E119 failure criterion of the average temperature increase at the unexposed face was less than 140 ºC. Thus, the strengthened beam specimen with an insulation thickness of 25 mm and 38 mm were able to maintain its structural integrity during the entire fire exposure.

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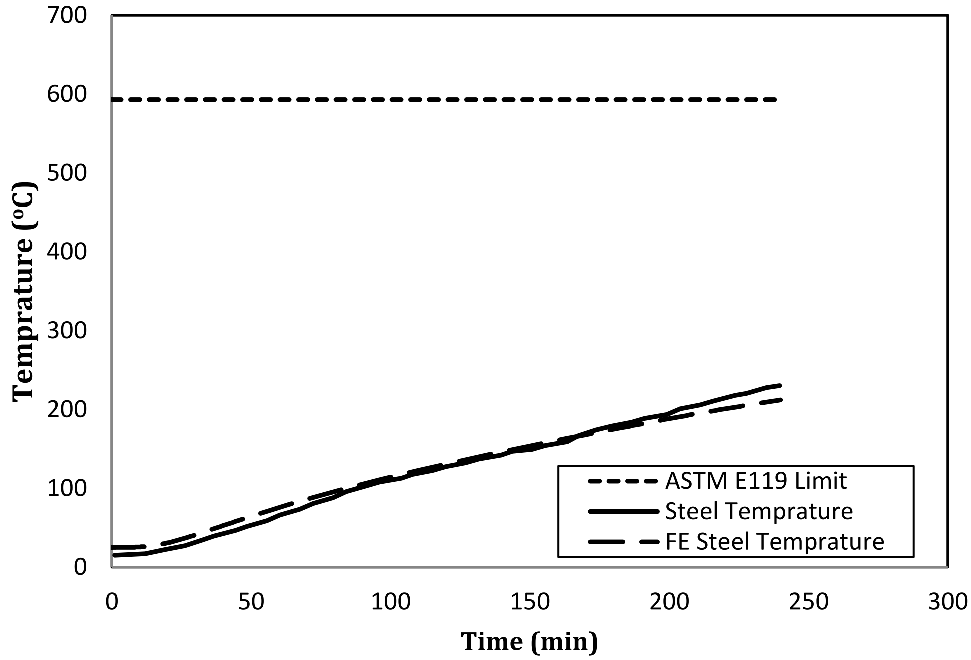
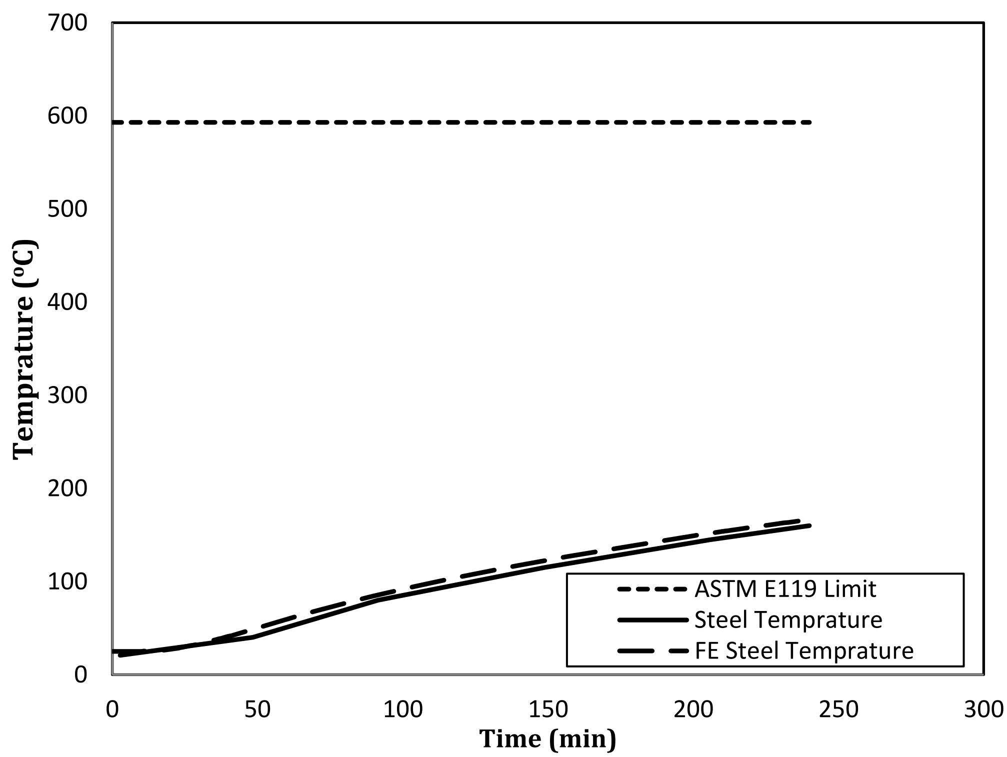


(a) Beam 1



(b) Beam 2

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Figure 8. Comparison of the FE and measured temperatures at the steel bars for the tested Beams

Structural Validation:

In order to further validate the performance of the developed FE model, the predicted and measured mid-span deflection response is compared during fire exposure. The mid-span deflection response of the measured and predicted results is shown in Fig. 9 for Beam 1 (Fig. 9a) and Beam 2 (Fig. 9b), respectively. A sustained applied load of 34kN/m as calculated by Williams et al. [3] was applied and kept constant during the entire fire test. It should be noted that the applied load corresponds to 56% and 48% of the theoretical ultimate capacity of the RC beam and actual ultimate capacity of the strengthened beam, respectively. It is clear that it was due to an accidental loss of hydraulic pressure in the loading rig during the fire test that caused a sudden drop in the mid-span deflection of the tested beam. It should be noted that the measured mid-span deflection was significantly reduced as shown in Fig. 9 during the test due to sudden loss of the hydraulic jacks [3] and was then increased to a level way less than its previous maximum value when the load was partially regained. Afterwards, the mid-span deflection steadily increased as shown in Fig. 9, but did not retain its initial level up to the end of the test which resulted in relatively lower measured deflections. On the other hand, the predicted FE model results indicatethat the mid-span deflection would gradually increases during the fire exposure time due to heating the bottom surface of the beam which would result in additional downward deflection. It could be concluded that for such type of tests which require a well-controlled environment at high temperatures and advanced testing setups, FE analysis is advantageous and yields a better prediction on the performance of the tested beam specimens at all stages of fire exposure.

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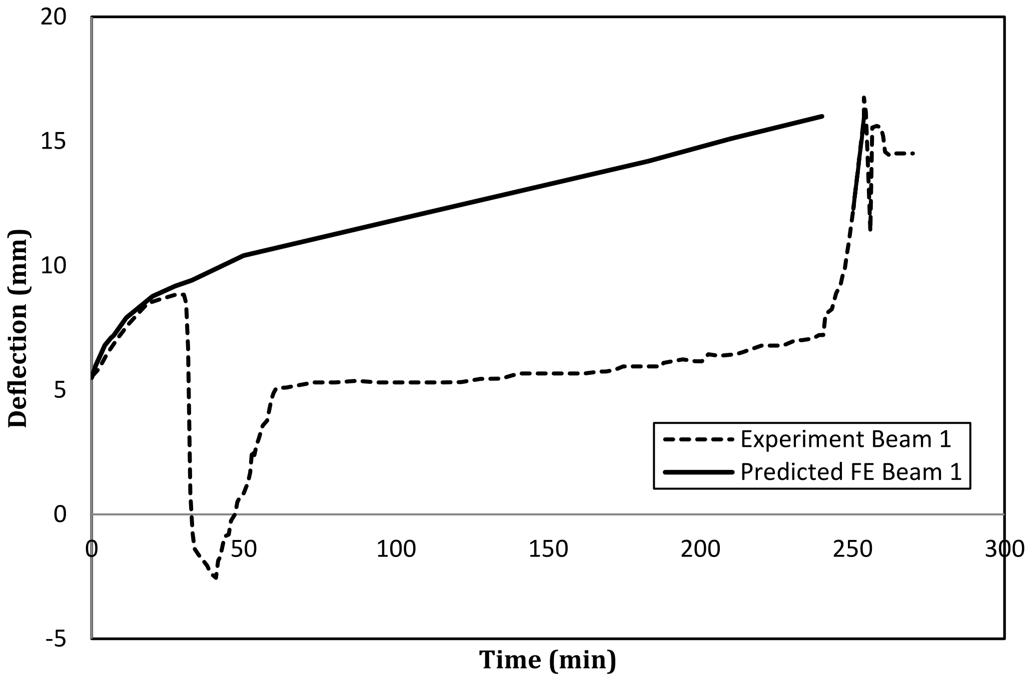

(a) Beam 1

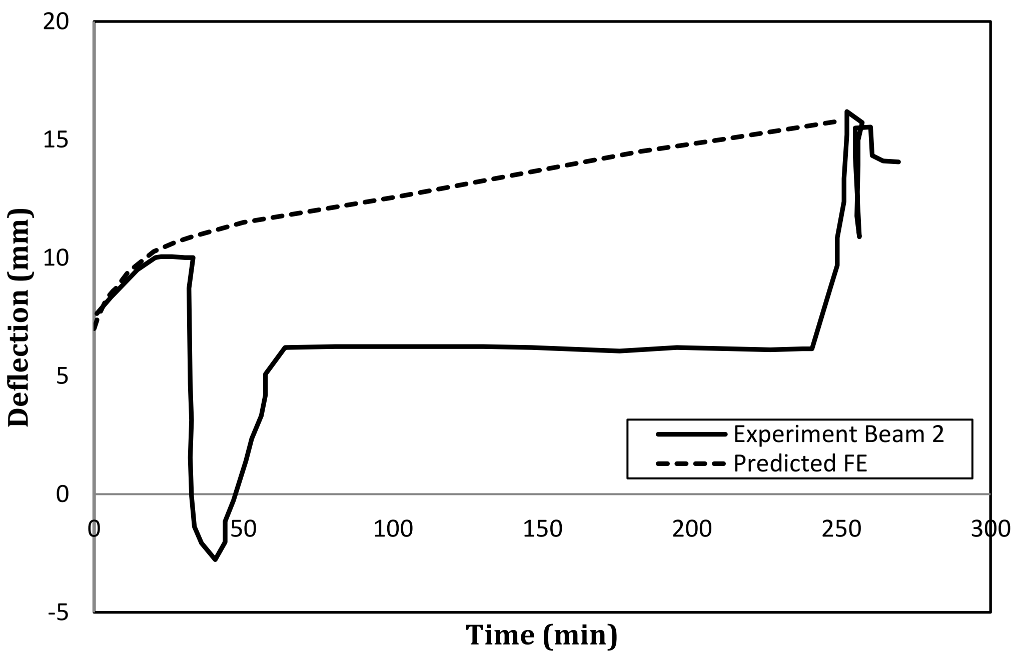

Figure 9. Comparison of the FE predicted and measured mid-span deflection

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Time to Failure

Failure is assumed to occur when the axial stress in the CFRP plate reaches its tensile strength that degrades with increasing temperature. This failure criteria to predict the time to failure of the tested beams was developed earlier by Hawileh et. al. [20]. Similar approach was undertaken here. Another failure criteria is taken to predict the fire endurance time and defined as the time when the CFRP temperature reaches its critical temperature. The critical temperature is taken as the temperature at which the CFRP material reaches approximately 50% of its original strength. According to Bisby [4], the critical temperature of the CFRP material is in the range of 200°C to 300 ºC. The FE simulation predicts that both beams will fail when the temperature of the CFRP/Concrete interface reaches approximately 220 ºC. Hence, the predicted time to failure of Beam 1 and Beam 2 is 133 and 210 minutes, respectively.

It was proven that although the temperature at the CFRP plate reached comparatively higher temperatures in the range of 300-400 °C, the strengthening system was able to maintain its applied loads and achieved a fire endurance of 4 hours in which the beams did not fail under the applied sustained loading. Thus, it could be concluded that the use of the glass transition temperature T_g of (65-82 °C) according to the ACI 440 guidelines [8] as a failing criteria is very conservative. This criterion requires further experimental and numerical investigation and currently does not seem to represent the actual behavior of FRP strengthening systems.

Parametric Study:

Upon validating the developed FE models with the experimental results, a parametric study based on the validated FE model of the first tested beam, Beam 1 is performed to further investigate the effect of several parameters associated with the behavior of CFRP strengthened RC members under elevated temperatures. The variables of the parametric study include the effect of applying different fire curves and scenarios, different applied live load combinations, as well as the effect of using different insulation schemes, types and thicknesses. The following subsections will discuss each part of the parametric study individually.

Effect of Different Fire Curves

The use of the ASTM E119 and/or ISO834 fire curves were exclusively used in most of the experimental programs conducted so far. This section aims to investigate the effect of using different fire curves on the performance of the strengthened RC beam validated previously. The effect of the ASTM E1529, RWS, Hydrocarbon Modified (HCM), Compartment fire 1 and 2 curves were investigated herein. It should be noted that the ASTM E1529, RWS, Hydrocarbon Modified (HCM) are considered to be standard fire curves used for different purposes [30] including petroleum and chemical industries and tunnels. The compartment fire curves represent fire most likely to occur in buildings. Figure 10 shows the different fire curves used in this section. it is clear from Fig. 10 that the compartment fire curves [31] were chosen to reach higher temperatures than the standard ASTM E119 fire curve in a shorter duration followed by a decaying period. It should be noted that the insulation type and thickness remained as those in Beam 1.

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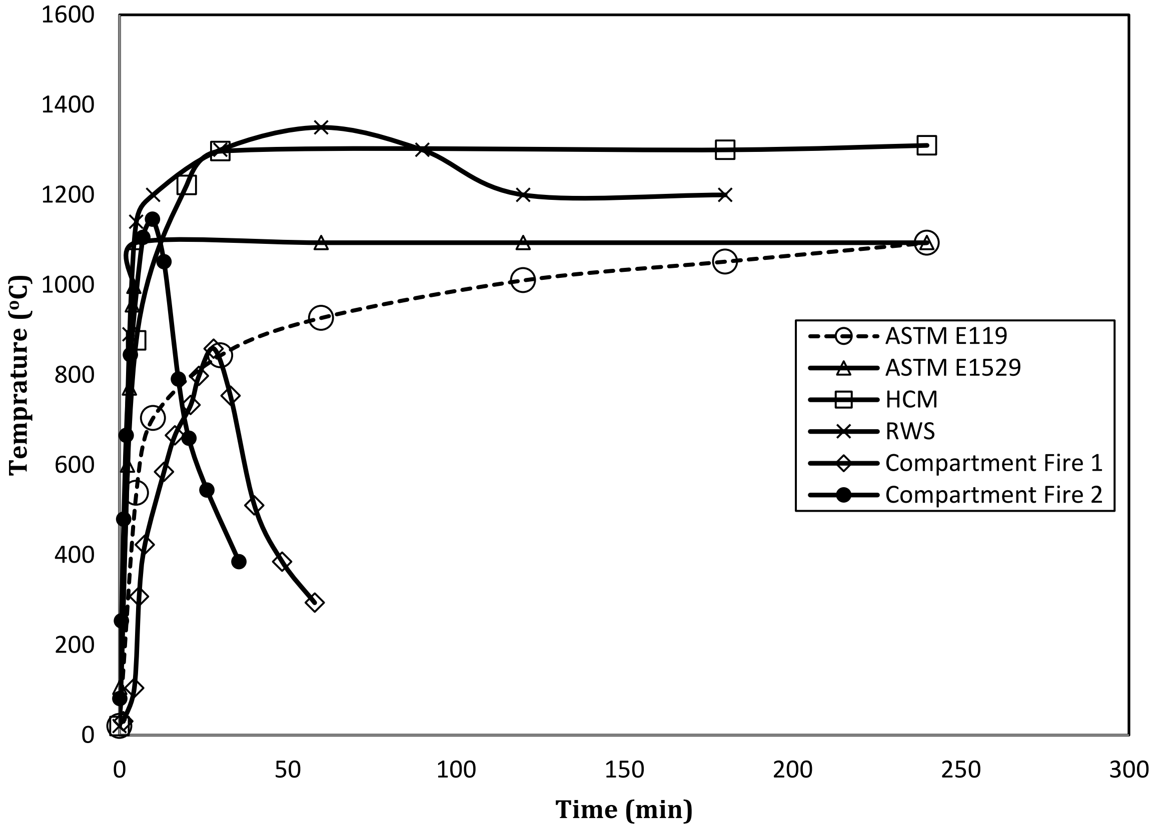

Figure 10. Different fire curves used in the parametric study

Thermal Response

The temperatures at the VG/CFRP and CFRP/concrete interfaces for the investigated fire curves are shown in Figs. 11 and 12, respectively. It can be seen that although the RWS fire curve is the most severe fire curve compared to the other fire curves, the temperature at the interfaces of the model exposed to the HCM fire curve was the highest among the different fire curves. This is due to the fact that the RWS only lasts for 180 minutes, while the HCM lasts for 240 minutes. In addition, the RWS curve tends to decrease its temperature from 1350 °C at 60 min to 1200 °C at 180 min. Because of the short durations of the compartment fire curves, the temperatures at the interfaces seemed not to increase beyond 110 °C. This implies that the duration of fire and its

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severity are the key parameters that should be considered in the fire performance analysis and design of such structures.

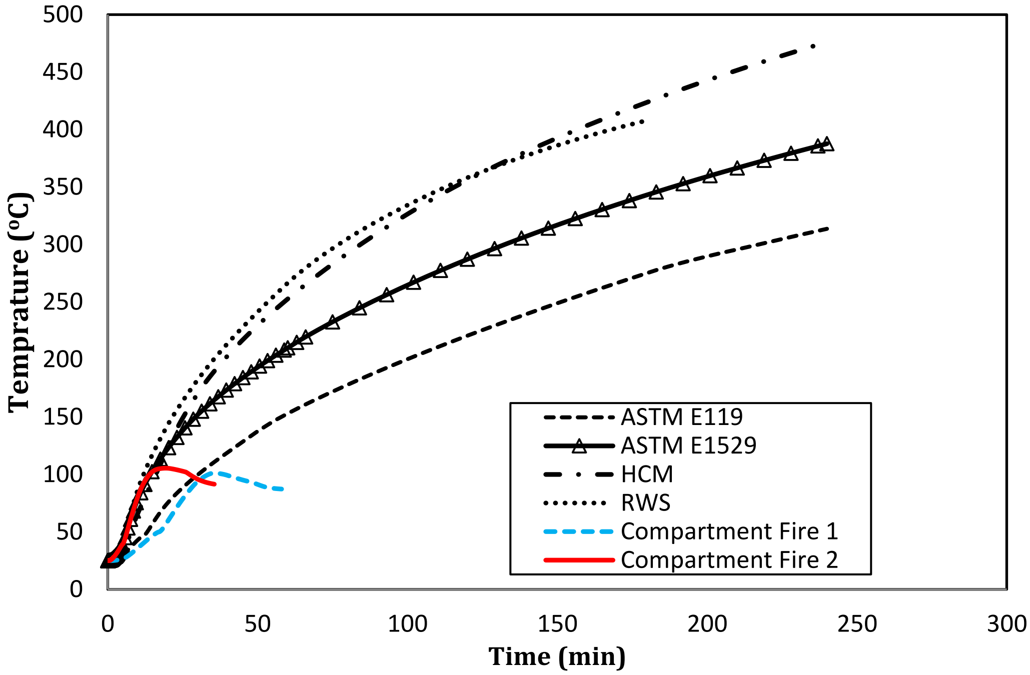


Figure 11. Temperature at the VG/CFRP interface for different fire curves

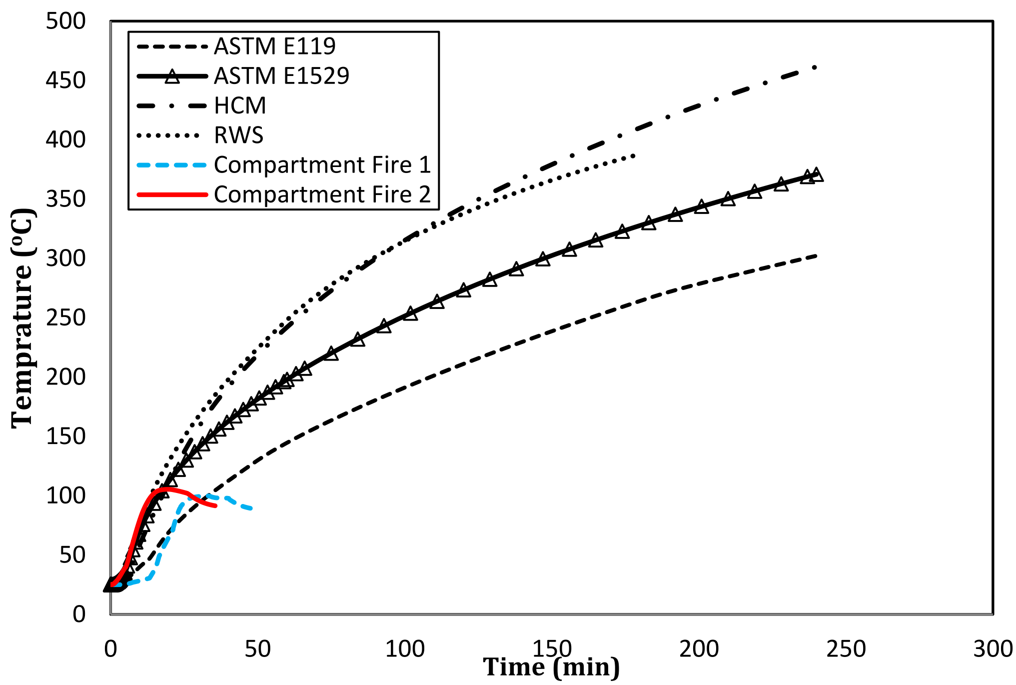


Figure 12. Temperature at the CFRP/Concrete interface for different fire curves

Structural Response

The structural response of the RC beam when exposed to the different fire curves was predicted by comparing the mid-span deflection with that of the ASTM E119 fire curve. As mentioned previously, the severe temperature exposure of both RWS and HCM led to a large increase in the mid-span deflection of the RC beam. The mid-span deflection of the specimen when exposed to the RWS and HCM was greater than the rest of the other fire exposures, Fig. 13.

On the other hand, both compartment fire curves achieved the lowest mid-span deflections during the fire exposure, although they have the same peak temperature for the first 30 minutes as of the ASTM E119 (Compartment Fire 1) or relatively higher during the whole course of exposure (Compartment Fire 2). This was mainly due to their short duration and presence of the decay period during the second phase of exposure. Figure 13 shows the mid-span deflection of the different models associated with the different fire curves used in this study. In addition, Table 2 shows the predicted time to failure of the strengthened beams when exposed to the different investigated fire curves.

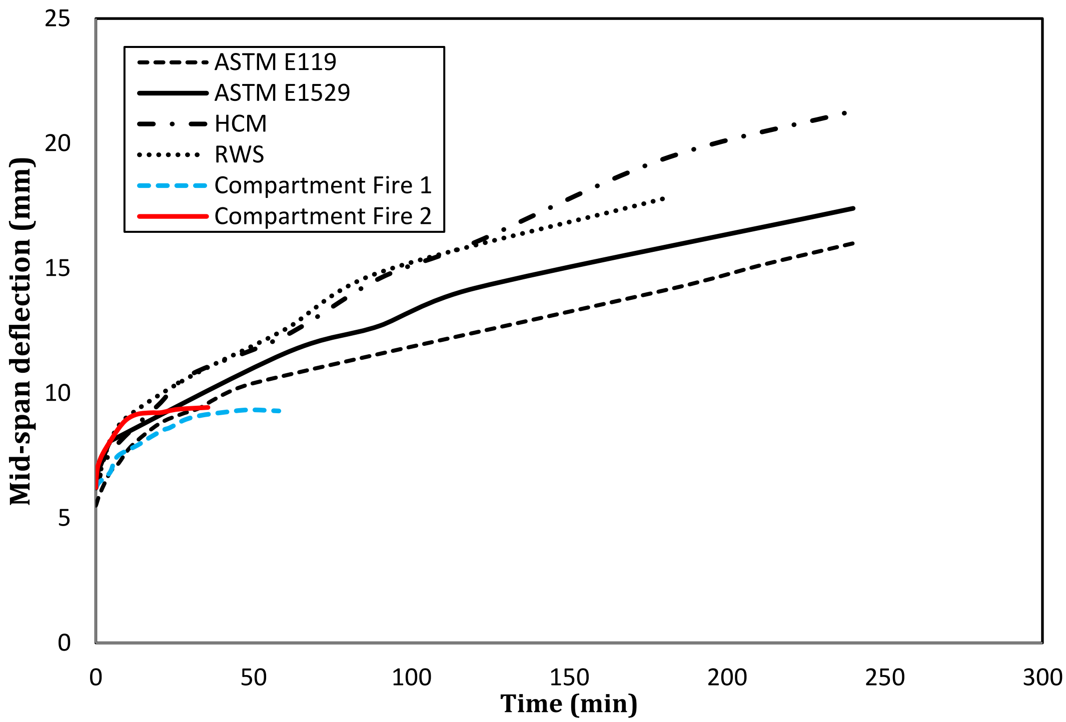


Figure 13. FE predicted mid-span deflection when exposed to different fire curves

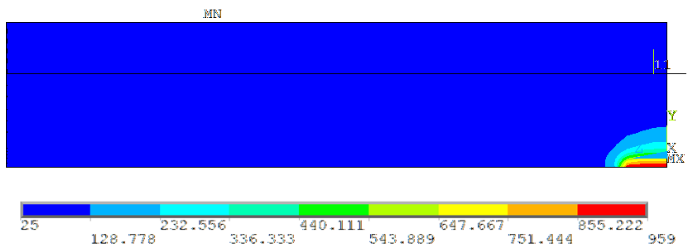
Table 2 Fire performance of the RC beams subjected to different fire curves

	E119	***E1529***	***RWS***	***HCM***	***Comp. Fire 1***	***Comp. Fire 2***
Time to failure (min)	133	98	85	86	No Failure	No Failure

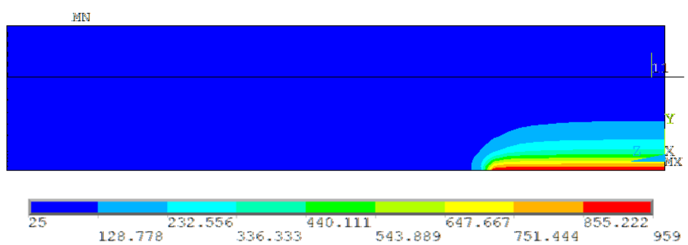
Effect of Different Fire Scenarios

The effect of using different fire scenarios on the behavior of RC beam is studied in this section. The different fire scenarios were chosen as if the fire was localized on certain percentages of the span length of the T-beam as shown in Fig. 14. The span length percentages were taken as

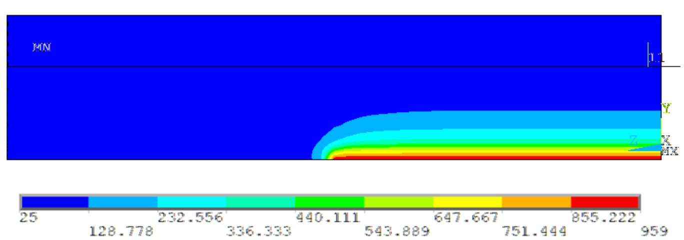
5, 25, 50 and 90% of the beam's span. Such cases might occur in RC beams spanning between two rooms or apartments. Once fire initiated in a room, the soffit of the beam will be exposed to elevated temperatures while the rest of soffit; spanning to the adjacent room with ambient temperature, will still experience ambient temperatures. The corresponding FE models associated with these runs are designated as FE 5%, FE 25%, FE 50%, and FE 90%, respectively. Furthermore, this section of the parametric study used the ASTM E119 as the main temperature-time fire curve.



(a) FE 5%



(b) FE 25%



(c) FE 50%

Figure 14. Temperature variations of different fire scenarios

Thermal Response

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Figure 15 shows the temperature at the VG/CFRP interface for the different fire scenarios used herein. It can be seen that the temperature at the interface of specimens FE 50 % and FE 90% are almost identical. On the other hand, the temperature at the same interface seems to reduce with smaller exposed percentages as shown in FE 5% and FE 25%.

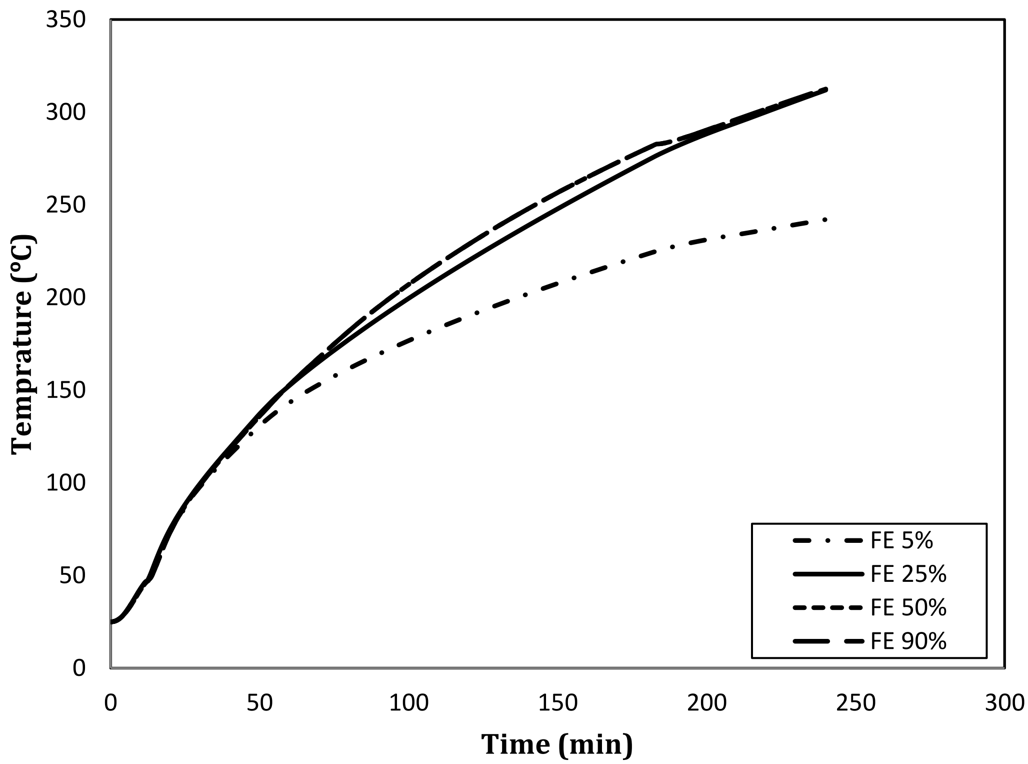

Figure 15. Temperature taken at the mid span at the VG/CFRP interface with various fire scenarios

Structural Response

Figure 16 shows the predicted mid-span deflection with time of the investigated models exposed to the different fire scenarios. It can be seen from Fig. 16 that the mid-span deflection increases as the exposed area to fire increases. This is due to the localization of the temperature effect when a small portion of the beam's span is exposed to fire, which will barely affect the entire member.

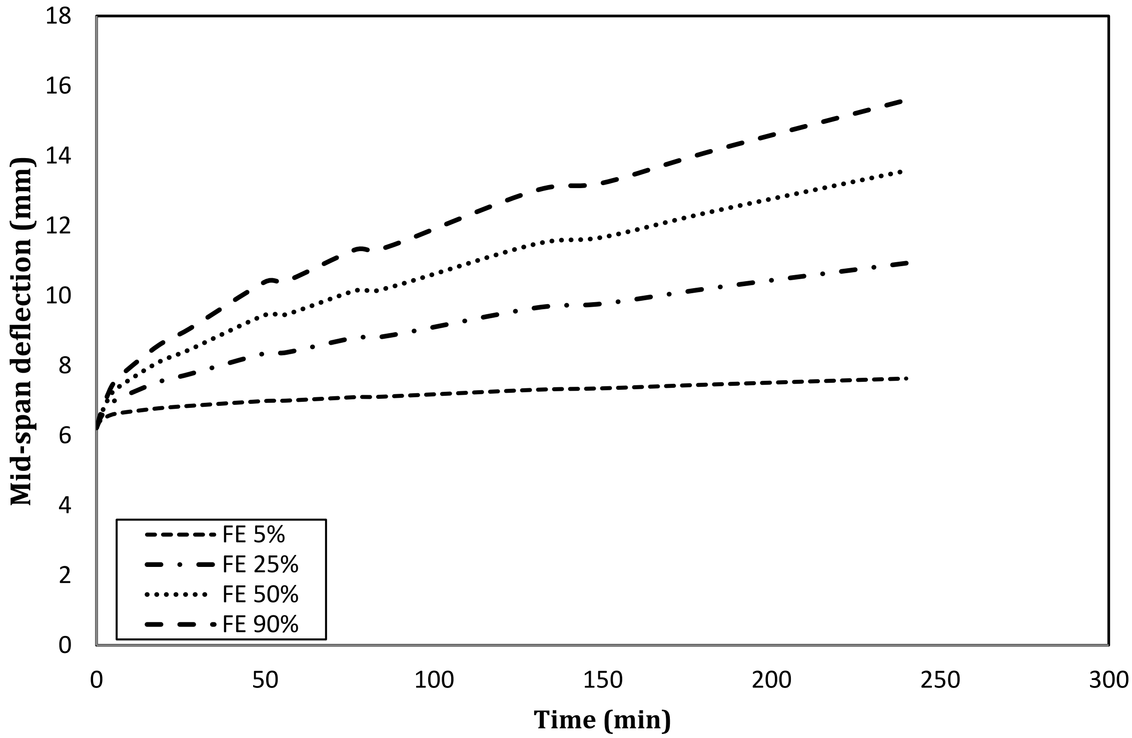

Figure 16. Comparison of the mid-span deflection with time of the models exposed to different fire scenarios

Effect of Different Applied Live Load Levels

In order to investigate the performance of the tested beam under different applied live load levels, five different cases are investigated in this section. The applied loads were based on the service loads carried by the tested RC beam. The live loads were varied to represent the presence of only the self-weight of the beam (0 kPa), 25% of the live load (27.8 kPa), 50% of the live load (55.7 kPa), 60% of the live load (66.8 kPa) and 75% of the live load (83.6 kPa). The FE models associated with each case are designated as FE 00-LL, FE 25-LL, FE50-LL FE 60-LL and FE 75-LL, respectively. It should be noted that the ASTM E119 was used herein as the applied fire curve.

Thermal Response

Since the FE thermal simulation does not acquire any structural inputs in terms of mechanical material properties or applied sustained loading, the thermal response of the modeled beams did not change from that of the validated model. Please refer to Figs. 7 and 8 to refer to the thermal response of the FE models.

Structural Response

Figure 17 shows the mid-span deflection versus time for the investigated case studies. It is clear from Fig. 17 that the mid-span deflection of the FE 00-LL model started very close to zero, unlike the rest of the models which experienced larger initial deflections with the increase in the applied live load level. Furthermore, the shape of the mid-span deflection of specimen FE 00-LL is different than the rest, in which it behaved by a linear increase rather than a parabolic increase.

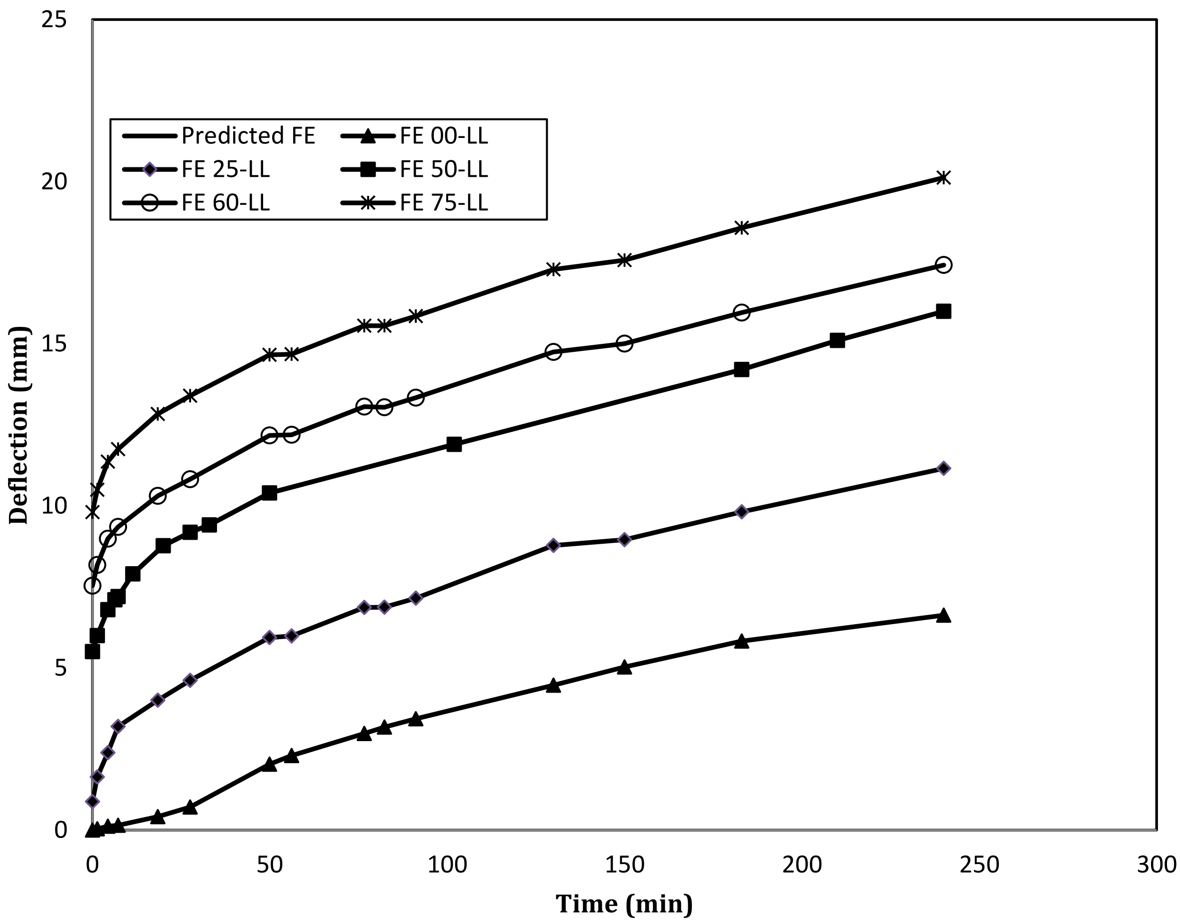

Figure 17. Mid-span deflection of the models with different applied live loads

The failure of the different specimens was based on the strength approach discussed earlier. Table 3 shows the predicted time at failure for each of the investigated models. It could be concluded that the higher the applied live load level on a structural member in the event of fire, the shorter is the fire endurance, the beam would achieve.

Table 3 Fire performance of the RC beams subjected to different fire curves

	FE 00-LL	*FE 25-LL*	*FE 50-LL*	*FE 60-LL*	*FE 75-LL*
Time to failure (min)	>240	240	133	87	33

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Effect of Different Insulation Schemes, Types and Thicknesses

The use of insulation materials is important for maintaining the structural integrity of CFRP strengthened RC beams. This section investigates the different parameters involved in the insulation systems, including schemes, types and thicknesses e. The investigated case studies are composed of four FE models designated as: FE-Soffit to investigate the effect of insulating only the beam's soffit, FE-PROMAGLAF to investigate the effect of using a different available insulation material type, and FE-35mm and FE-50mm to investigate the effect of using different insulation thicknesses. The investigated models were compared against the validated model "Beam 1" that was protected with a VG insulation material having a thickness of 25 mm wrapped in a U-Shape scheme and tested under ASTM E119 fire curve.

Thermal Response

The thermal response at the CFRP/concrete interfaceof the developed FE models is shown in Fig. 18. It is clearly shown that the temperature at the CFRP/concrete interface dropped with the use of thicker insulations. In a similar manner, the temperature at CFRP/concrete interface of the FE model that was protected using the PROMAGLAF insulation material performed better than VG insulation for the same insulation thickness and scheme. This is due to the better thermal properties of the PROMAGLAF insulation material provided in Table 4.

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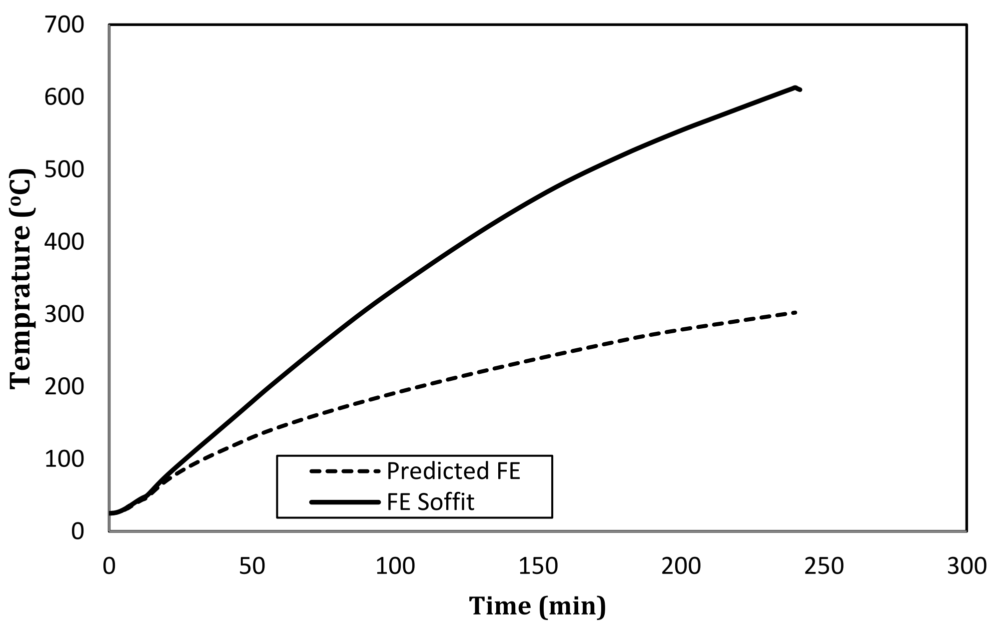
Table 4 PROMAGLAF Insulation material properties [32]

Property	***PROMAGLAF***
Density (Kg/m³)	100
Thermal Conductivity (W/m.K)	@ 200°C => 0.06 @ 400°C => 0.10 @ 600°C => 0.14 @1000 => 0.20
Specific heat (kJ/Kg.K)	1.13

It could be also concluded that the used of U-shape wrapped insulation would provide a longer fire endurance compared to protecting the beam's soffit only. This is because the U-Shaped insulation will protect both vertical sides and soffit of the beam, unlike the insulation applied to the soffit of the beam only.



(a) Different insulation schemes

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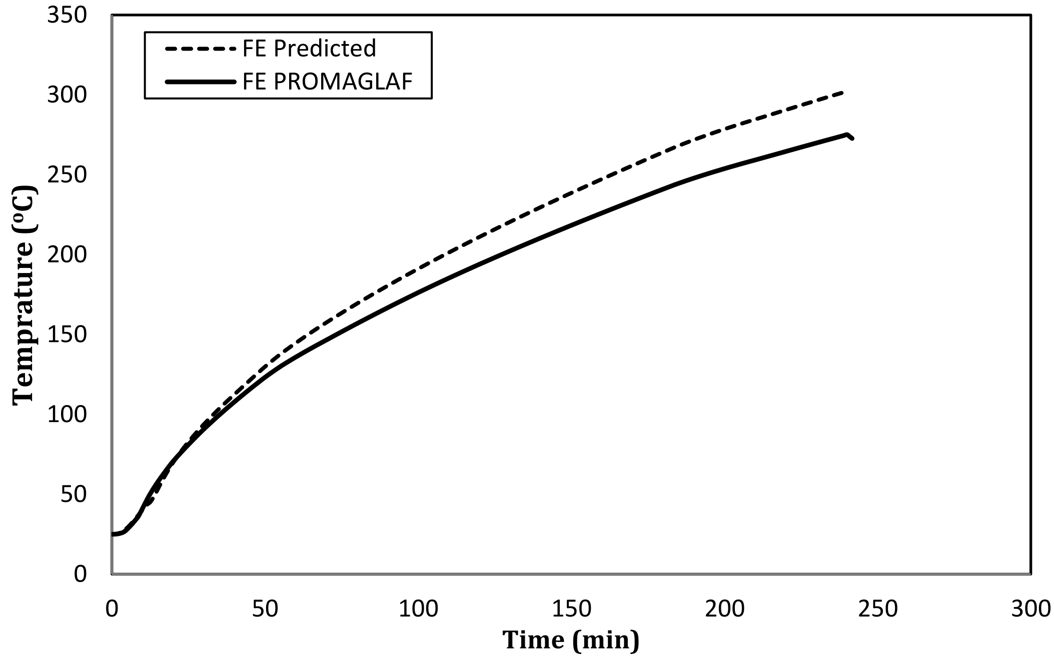
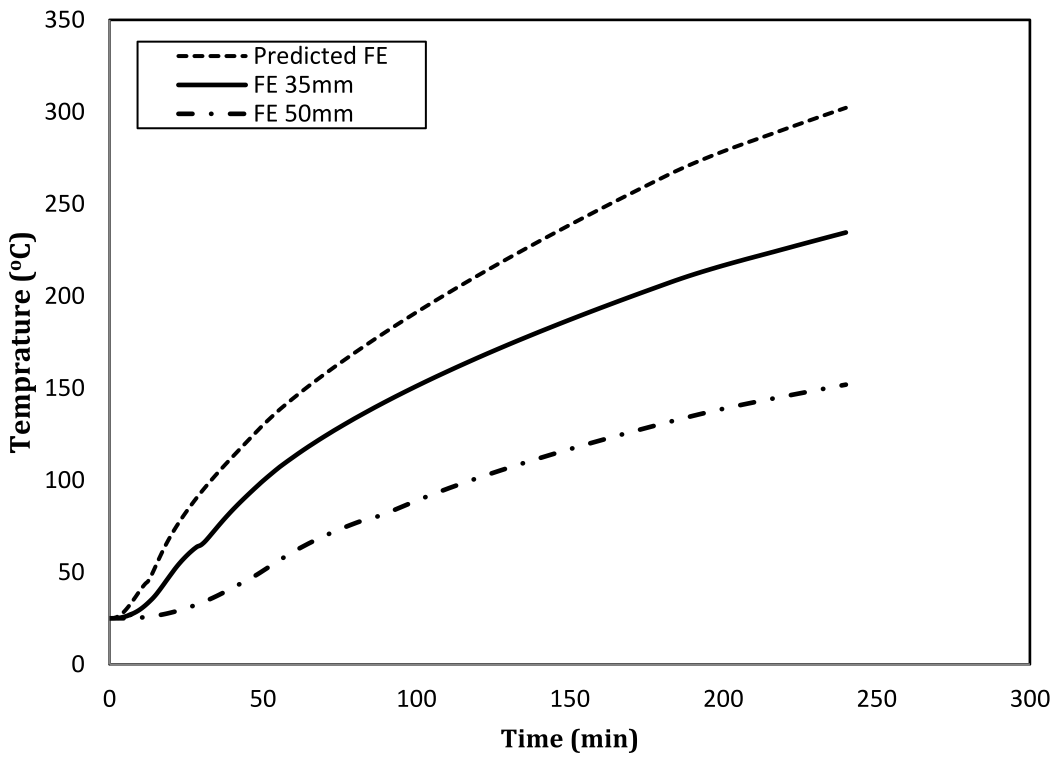


(b) Different insulation types



(c) Different insulation thickness

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Figure 18. Thermal responses at the CFRP/concrete interface of the different insulation properties

Structural Response

As expected, the structural response of the FE models insulated with thicker 35 and 50 mm insulation is better than that of Beam 1. Furthermore, the response of the model that used PROMAGLAF insulation is also better than the validation model. The only case where the performance dropped was shown in the FE –Soffit model. This is due to the absence of the insulation at the web's sides that led the temperature to rapidly increase across the member which led to an earlier failure of the strengthened beam. Figure 19 shows the mid-span deflection of different cases studied herein.

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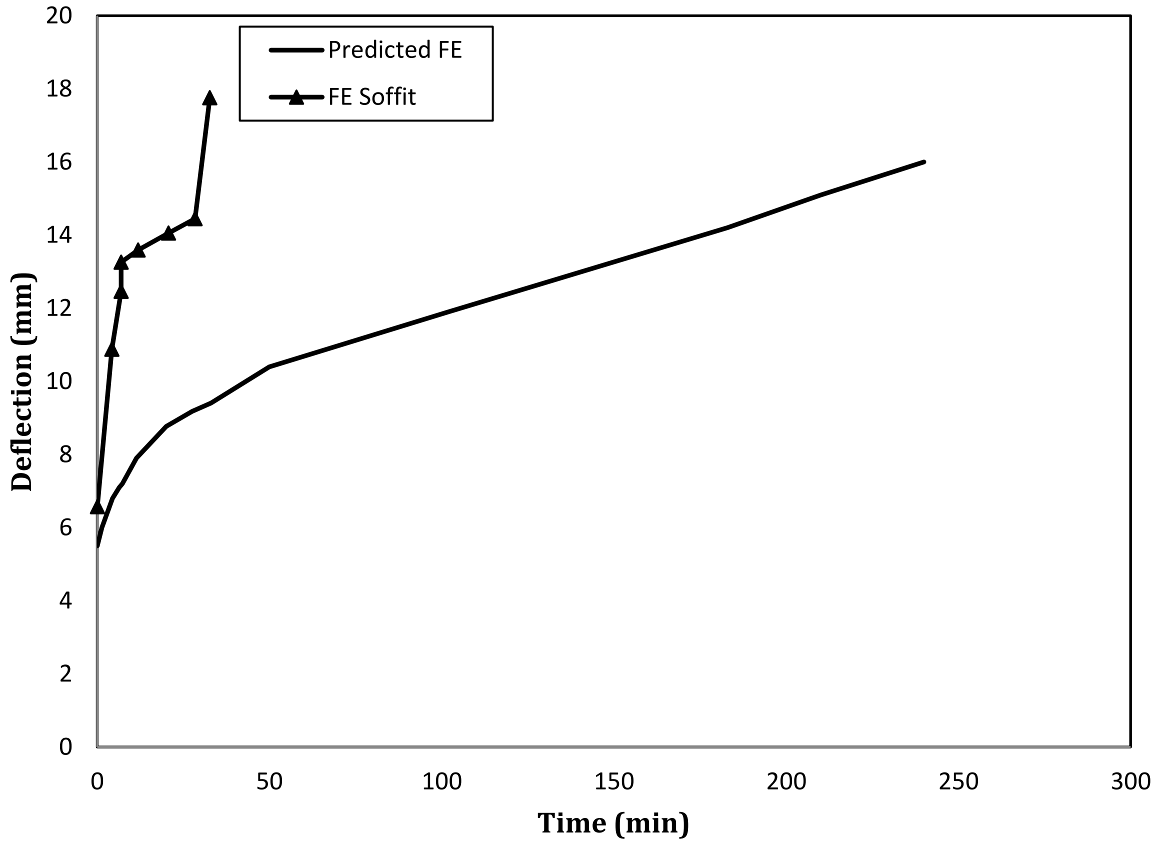

(a) Different insulation schemes

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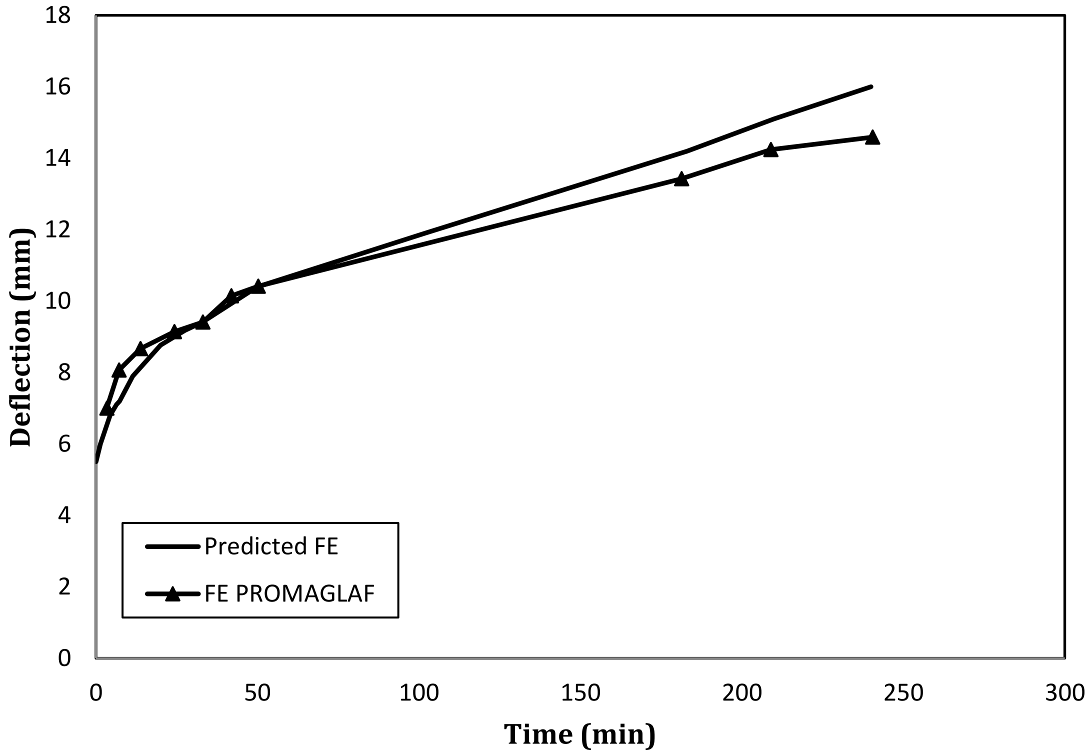
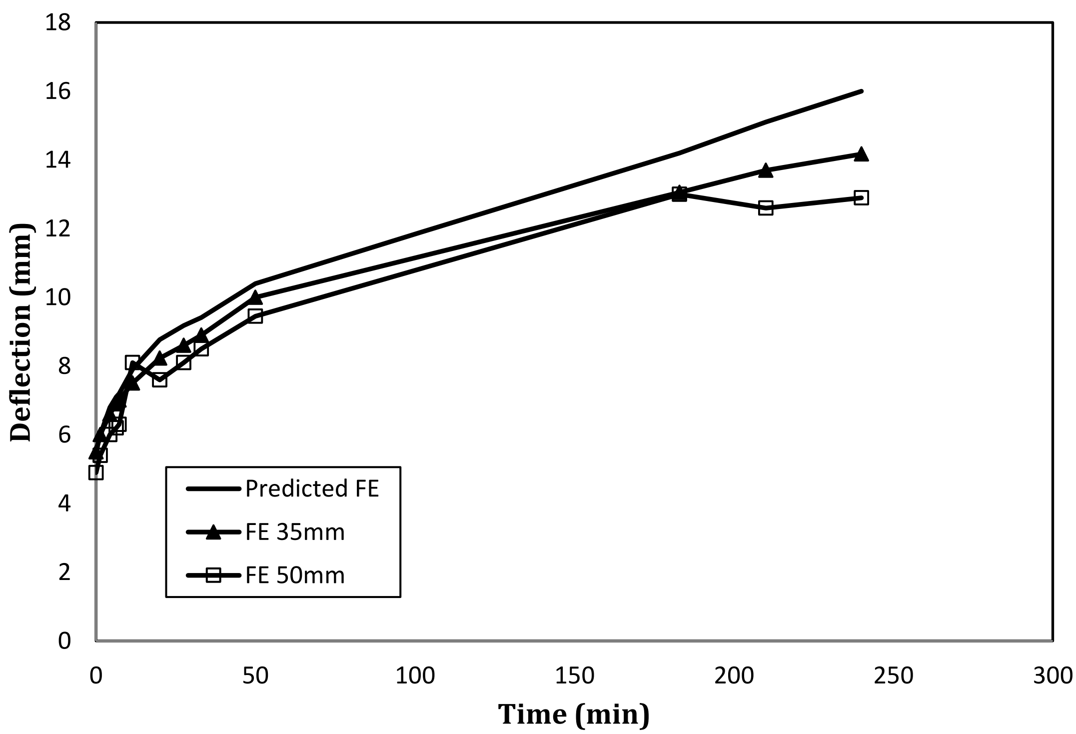


(b) Different insulation types



(c) Different insulation thickness

Figure 19. Structural responses of the different insulation parameters

The predicted time to failure for models FE-50mm, FE-35mm and FEPROMAGLAF was 144, 140 and 137 min, respectively. On the other hand, FE-Soffit model failed very pre-maturely at approximately 18 min.

SUMMARY AND CONCLUSIONS:

A detailed Finite Element model is developed in this numerical study to investigate the performance of strengthened RC beams with CFRP plates under fire loadings. The developed FE models were validated against an experimental program conducted by other researchers. The validation process was based on both thermal and structural results including comparison of temperature distribution at key points across the beam's cross section, mid-span deflection response, and time to failure.

A parametric study was then conducted to investigate the effect of different fire curves and scenarios, different sustained loads levels and different insulating materials, schemes and thickness.

The following conclusions were drawn from the results of this numerical investigation:

1. Good correlation was obtained between the experimental and predicted results (both thermal and structural) at all stages of the fire loading up to failure of the tested specimens.

2. The developed models would be used as an alternative to the time consuming and expensive fire testing, especially in design oriented parametric studies.

3. The FE modeling can be a great tool to aid designers and researchers in the process of investigating the different aspects of the structural fire engineering. In addition, FE analysis is simulated in a well-controlled environment and provide

full field of results, in terms of temperature distribution, mechanical stresses, and deformations.

4. The temperature variation with time is highly dependent on the applied fire curve.

5. The type of fire scenario plays a critical role on the fire performance of concrete beams and should be considered in both analysis and design.

6. Severe fire curves such as HCM and RWS produce larger mid-span deflections and damage to the FRP system compared to both standard (ASTM E119) building and compartment fire curves presented herein.

7. The mid-span deflection increases as the applied local fire exposure area increases.

8. The use of U-Wrap insulation enhances the fire endurance of the insulated strengthened beams. On the other hand, the use of insulating materials attached to the soffit of the beam would compromise the strengthening system.

9. The mid-span deflection would decrease in the event of fire with the increase in the insulation thickness.

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NOTATION SECTION

E_{xo} is the modulus of elasticity in the "x" direction (MPa)

E_{yo} is the modulus of elasticity in the "y" direction (MPa)

E_{zo} is the modulus of elasticity in the "z" direction (MPa)

K_o is the thermal conductivity *(W/mm.K)*

C_o is the specific heat (J/kg.K)

μ_{xo} is the Poisson's ratio in the "x" direction

$\mu_{y,z}$ is the Poisson's ratio in the "y and z" directions

α_{xo} is the coefficient of thermal expansion (1/˚C)

ρ_o is the density(Kg/mm^3)

T_o is the initial room temperature take as 25 °C

t is the time (hours)

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ILLUSTRATIONS

Figure 1. Different views of the FE quarter model

Figure 2. Variation of normalized stiffness with temperature [3]

Figure 3. Variation of concrete compressive strength with temperature [22]

Figure 4. Variation of steel reinforcement strength with temperature [22]

Figure 5. Variation of normalized density with temperature [4]

Figure 6. Variation of normalized thermal conductivity and specific heat with temperature [4]

Figure 7. Comparison of the predicted FE and measured temperatures for Beams 1 and 2

Figure 8. Comparison of the FE and measured temperatures at the steel bars for the tested Beams

Figure 9. Comparison of the FE predicted and measured mid-span deflection

Figure 10. Different fire curves used in the parametric study

Figure 11. Temperature at the VG/CFRP interface for different fire curves

Figure 12. Temperature at the CFRP/Concrete interface for different fire curves

Figure 13. FE predicted mid-span deflection when exposed to different fire curves

Figure 14. Temperature variations of different fire scenarios

Figure 15. Temperature taken at the mid span at the VG/CFRP interface with various fire scenarios

Figure 16. Comparison of the mid-span deflection with time of the models exposed to different fire scenarios

Figure 17. Mid-span deflection of the models with different applied live loads

Figure 18. Thermal responses at the CFRP/concrete interface of the different insulation properties

Figure 19. Structural responses of the different insulation parameters