The 8th European Congress on Computational Methods in Applied Sciences and Engineering
ECCOMAS Congress 2022
5 – 9 June 2022, Oslo, Norway

FE ANALYSIS OF FRCM STRENGTHENED RC COLUMNS EXPOSED TO FIRE

REEM TALO*, MUHAMMAD KYAURE*, SALEM KHALAF* AHMED EL REFAI†, AND FARID ABED*

* American University of Sharjah (AUS)
Sharjah, UAE
e-mail: fabed@aus.edu, www.aus.edu

† Laval University
Quebec, Canada
email: ahmed.elrefai@gci.ulaval.ca

Key words: FE, Reinforced Concrete, Column, FRCM, Fire

Abstract. Reinforced concrete (RC) structures deteriorate over time and therefore, need to be strengthened. Despite the fact that RC structurtes have a decent fire rating, the performance of the strengthening system under fire exposure needs to be evaluated. One of the main restrictions associated with Fibre Reinforced Polymer (FRP) systems is their poor resistance to high temperatures, which originates from the combustible polymer matrix. Therefore, Fibre Reinforced Cementitious Matrix (FRCM) systems have been introduced due to their improved performance during fire exposure. The potential of Poly-paraphenylene-ben-zobisoxazole (PBO) FRCM to strengthen circular RC columns is evaluated using a nonlinear finite element analysis (FEA). The commericial software ABAQUS is used to develop 3D FE models to investigate the axial performance of the strengthening system. The modeling approach is performed by first conducting a thermal analysis to generate the nodal tempertaures. The second step of the model includes a displacement-controlled loading condition with imported nodal temperatures from the first model. The temperature dependent material properties are incorporated in both models. The modeling approach is validated against published literature and a parametric study is conducted on PBO FRCM strengthened and unstrengthened columns heated at durations of 1, 2, and 4 hours following the ASTM standard fire temperature-time curve. Results indicated a decrease in the axial capacity and stiffness of the strengthened and unstrengthened columns upon heating. Moreover, an increase in the columns' ductility due to the increase in temperature was observed. A decrease in ultimate strength of up to 78 and 68% was observed for the unstrengthened and strengthened columns respectively.

1 INTRODUCTION

Concrete is the most commonly used construction material due to its many advantages including relatively high compressive strength and durability. However, reinforced concrete

(RC) structures have been proven to deteriorate at some point in their lifetime. FRP (Fibre Reinforced Polymers) are one of the most common composite systems used in strengthening RC structures, which usually consist of unidirectional fibres embedded in a polymer adhesive matrix. These composite systems are highly efficient due to many reasons including corrosion resistance, large deformation capacity, high strength-to-weight ratio, speed of installation, etc. Although FRP composites are generally considered to be effective and efficient, they have several drawbacks including poor compatibility of the organic adhesives with the concrete substrate. Moreover, the organic adhesives used in FRP have poor resistance to elevated temperatures, which is a serious concern in fire events. The organic adhesives lose their mechanical properties under high temperatures and release toxic fumes when temperature levels exceed their glass transition temperature. To overcome the drawbacks linked to the organic adhesives used in FRP systems, FRCM (Fibre Reinforced Cementitious Matrix), also known as TRM (Textile Reinforced Matrix) strengthening systems were introduced. FRCM consist of open mesh textile fibres that are embedded into inorganic cementitious matrices as an alternative to the organic adhesives used in FRP systems [1]–[5].

An experimental study by Ding et. al [6] investigated the fire performance of fibre-reinforced inorganic polymer (FRiP) compared to fibre-reinforced polymer (FRP) for beam strengthening and the behavior of both composite systems was examined before and after fire exposure. Results showed that FRiP composites can significantly contribute to the strength enhancement of the beams. Moreover, they displayed considerably improved resistance to fire compared to FRP. After exposure to fire, the FRiP composites could retain around 47% of their strengthening efficiency in ambient temperature conditions.

Chowdhury et. al [7] conducted full-scale fire tests on FRP-strengthened RC columns with and without supplemental fire protection. The results indicated that although FRPs are sensitive to high temperatures, it is possible to achieve satisfactory fire endurance ratings for FRP strengthened columns by providing suitable fire protection. However, the insulation system in the performed experiments was not able to maintain the temperature of the FRP system below its glass transition temperature.

The current study aims at investigating the structural performance and residual capacity of PBO FRCM-strengthened and unstrengthened columns exposed to different heating durations following the ASTM standard fire temperature-time curve [8]. ABAQUS FEM software is utilized to perform the simulations and the obtained results are presented and discussed[9].

2 MATERIALS

2.1 Therimal Properties

To determine the extent of heat transfer to the structural member, the temperature rise in different regions, heat distribution, and thermal properties should be accounted for in the finite element model. In the heat transfer analysis stage, only the thermal properties are considered for concrete, steel, and PBO FRCM materials. Specific heat and thermal conductivity are input for each material and vary with temperature [10]. The thermal properties of concrete and steel are determined according to relations provided by Eurocode 1992-1-2 and Eurocode 1993-1-2 [11], [12]. Specific heat and thermal conductivity of the FRCM cementitious material are input according to published literature [13], [14].

2.2 Structural Properties

In fire resistance design, the primary properties that should be considered for concrete and steel are the modulus of elasticity and the stress-strain response in compression and tension. In the current study, concrete and steel are assumed to have initial compressive and yield strength values ($f'c$ and fy) of 30 MPa and 500 MPa respectively. The variation of the modulus of elasticity, the compressive strength, and the tensile strength are input according to relations provided by Eurocode 1992-1-2 for both concrete and steel reinforcement [11]. Stress-strain relations of concrete in both compression and tension are generally used to express the mechanical response of concrete. They are typically utilized in mathematical models to investigate the fire resistance of structural members. The behavior of concrete in compression is generally characterized by an initial linear response proportional to the elastic modulus, followed by a nonlinear response until its compressive strength is reached, and finally a descending branch. The compressive strength of concrete decreases with the increase in temperature, unlike its ductility, which increases with elevated temperature levels. This, in turn, leads to a decreased slope of the stress-strain curve indicating compromised mechanical properties.

The elevated temperature has a significant impact on the stress-strain behavior of concrete, particularly above 500°C. As the temperature increases, the strain corresponding to the peak stress can reach up to four times the strain at room temperature[10]. Figure 1 displays the compressive stress-strain curves adopted for concrete in the modelled specimens generated based on the Eurocode relations [11]. Under uniaxial tension, the stress-strain response of concrete can be characterized by a linear elastic behavior, followed by softening, and finally sudden brittle failure. When the temperature exceeds 300°C, the thermal damage in the form of microcracks is more pronounced, causing the tensile strength to decrease at a rapid rate [10].

On the other hand, the structural properties of the PBO composite are obtained from the published literature [15]. Upon heating, FRCM materials undergo decreases in both their ultimate strength and stiffness, which is also accounted for in the finite element model [16].

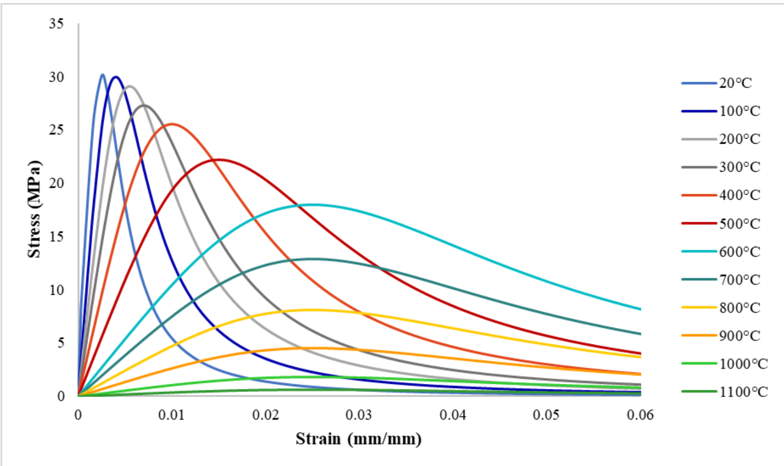


Figure 1: Concrete compressive stress strain curves [11].

3 PARAMETRIC PROGRAM

A total of 8 RC columns are modelled in this study including strengthened and unstrengthened columns subjected to different heating durations following the ASTM standard fire temperature-time curve [8]. All columns have a cross sectional diameter of 170 mm and a length of 1000 mm. The reinforcement detailing in all specimens comprises of 6#10 longitudinal bars and #6 @ 100 mm transverse ties. The parametric program of the current study is represented in Table 1.

Table 1: Parametric Program

Specimen Identification	Number of FRCM Layers	Heating Duration (h)
C-0h	—	—
C-1h	—	1
C-2h	—	2
C-4h	—	4
1L-0h	1	—
1L-1h	1	1
1L-2h	1	2
1L-4h	1	4

4 FINITE ELEMENT MODELLING APPROACH

4.1 Heat Transfer Analysis

For all heated columns, an initial heat transfer analysis model is conducted to obtain the element nodal temperatures. The thermal response of the columns is evaluated using a transient heat transfer analysis in which heat flows from the surface of the column and propagates to its inner parts. Transmission of heat through the column surface is simulated through convection and radiation. The convective heat transfer coefficient (h_c) is taken as $25\left(\frac{W}{m^2}\right)K$. The absolute zero temperature (T_z) is input as -273.15°C and the Stephan Boltzmann constant (σ) is taken as $5.67 \times 10^{-8}\left(\frac{W}{m^2}\right)K^4$. Moreover, the emissivity of the exposed column surface (ε_m) is input as 0.8. All specimens are heated following the ASTM standard fire temperature-time curve. The concrete, FRCM, and steel reinforcement are assigned as DC3D8: 8-node linear heat transfer brick, DS4: 4-node heat transfer quadrilateral shell, and DC1D2: 2-node heat transfer link elements respectively [17].

4.2 Structural Analysis

To evaluate the mechanical response of the columns subjected to elevated temperatures, a second finite model with two structural steps is developed. The nodal temperatures from the initial heat transfer model are imported as a predefined field to the first structural step. A second displacement-controlled loading step is added afterwards to load the columns until failure. The concrete, FRCM, and steel reinforcement are assigned as C3D8R: 8-node linear brick with reduced integration, S4R: 4-node shell with reduced integration, and T3D2: 2-node linear 3-D truss elements respectively[15], [18].

5 MODEL VERIFICATION

An experimental study by Cerniauskas et. al [19] is selected to verify the proposed modelling approach. A series of tests are conducted on TRM and FRP confined concrete cylinders which are heated to different temperature levels up to 400 °C and directly loaded in axial compression until failure. Selected control and Carbon TRM confined cylinders (1 and 3 layers) in the chosen study are modelled for verification. Figure 2 represents the load displacement plots of the unstrengthened control cylinders tested at ambient temperature (20°C), 200°C, and 400°C. A decrease in stiffness can be observed with the increase in temperature; moreover, a slight decrease in strength occurs as the temperature increases to 400°C. As mentioned previously, significant decrease in the concrete strength is observed particularly above 500°C, which could be the reason for the insignificant decrease in capacity between the cylinder tested at ambient temperature and that tested with a surface temperature of 400°C.

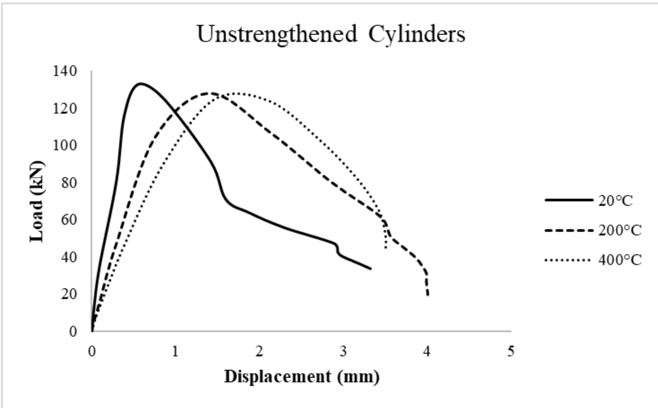
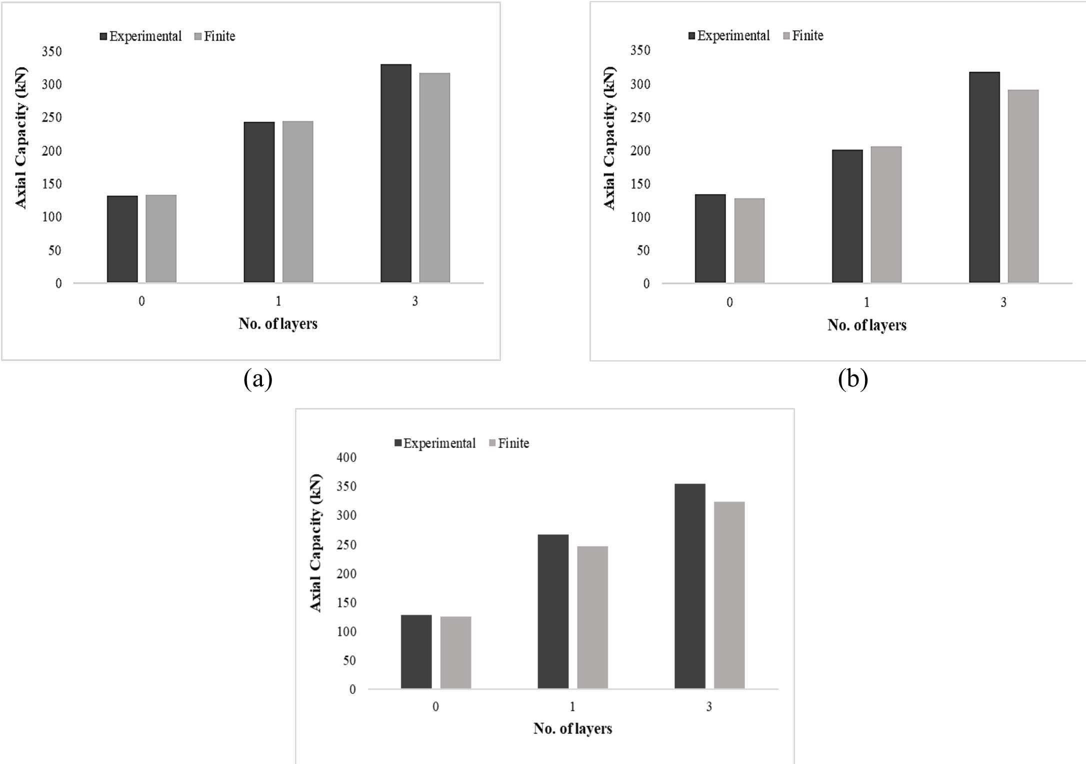


Figure 2: Unstrengthened cylinder load-displacement curves.

Figure 3 displays the ultimate capacity of the unstrengthened and strengthened cylinders at 20°, 200°, and 400°C obtained from the performed experiments and finite element models. It can be observed that cylinders strengthened with 1 and 3 TRM layers exhibit minor decrease in capacity corresponding to 16 and 8%, respectively, when the surface temperature reached 200°C. However, for the strengthened cylinders that reached a surface temperature of about 400°C, both the experimental and finite element results display minor increase in capacity. Since concrete exhibits a considerable decrease in its compressive strength at relatively high temperature levels (above 500°C) and is a highly durable material with good fire resistance, it is expected to observe such fluctuations in ultimate capacity at lower temperature ranges [10]. A maximum percentage difference of 9% is observed between the finite element results and the results obtained from the experimental study [19].



(a) (b) (c)

Figure 3: Experimental and finite element axial capacities at (a) 20°C (b) 200°C (c) 400°C.

6 RESULTS AND DISCUSIÓN

6.1 Nodal Temperatures

The temperature-time curves for selected nodes located on the column's outer surface are obtained for the unstrengthened columns heated for durations of 1, 2, and 4 hours and the plots are shown in Figure 4.

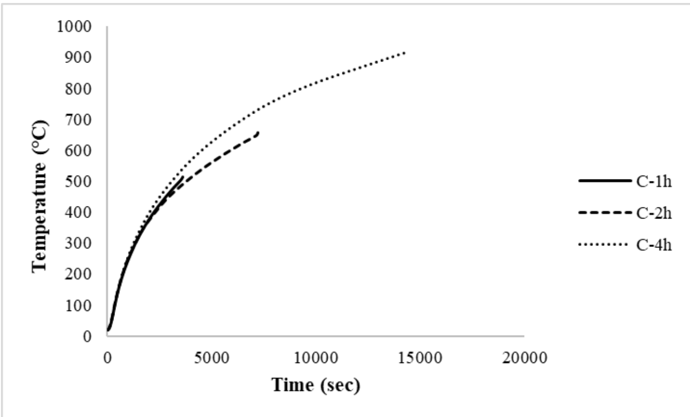
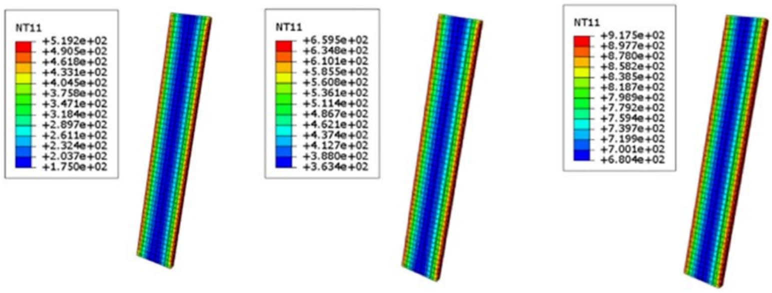


Figure 4: Nodal temperature-time curves.

Following the ASTM standard fire temperature-time curve, the final outer surface temperatures reached for C-1h, C-2h, and C-4h are 514 °, 657 °, and 917 °C, respectively. The nodal temperature contours for each of the unstrengthened columns are presented in Figure 5.



(a) (b) (c)

Figure 5: Unstrengthened column nodal temperature contours for (a) C-1h (b) C-2h (c) C-4h.

6.2 Load-Displacement Curves

For the control, unstrengthened columns, a general trend is observed as temperature levels increase where the axial capacity decreases along with a decrease in stiffness. Increase in ductility is also observed with longer heating durations. When columns are subjected to elevated temperatures, they tend to display a more ductile behavior and experience higher axial strains at the ultimate stress combined with reduction in stiffness, leading to increased deformations. Concrete experiences numerous physical, chemical, and mechanical changes when exposed to high temperatures including the loss of moisture resulting in losses in both strength and stiffness [10]. The load displacement curves of the control columns are shown in Figure 6. After heating the column for a period of 1 hour, it loses about 15% of its axial capacity. However, when heated for a period of 2 hours, it loses approximately 32% of its ultimate capacity. After being heated for a period of 4 hours, the column is considered to have lost most of its structural integrity with a 78% decrease in capacity.

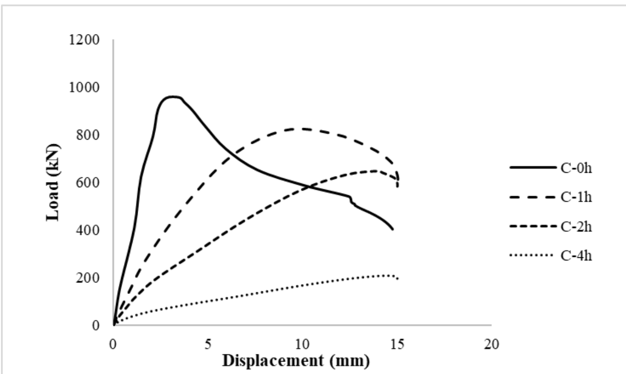


Figure 6: Load-displacement curves for unstrengthened columns.

For the strengthened columns, a shell sensitivity analysis is performed for a column under room temperature conditions. Figure 7 represents the resulting load-displacement curves for selected shell thicknesses ranging between 0.15 mm and 0.3 mm. Minor differences in capacity between the shell thicknesses are observed, with a percentage difference of around 4% between the 0.15 mm and 0.3 mm thicknesses. For the current experimental program, a shell thickness of 0.15 mm is selected.

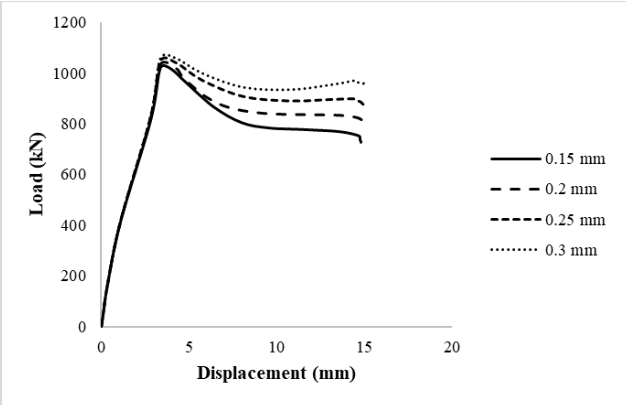


Figure 7: Shell sensitivity analysis.

The plots for the strengthened columns represented in Figure 8 indicate a significant decrease in stiffness between the column at room temperature and that heated for 1 hour. However, the decrease in stiffness between the 1-hour and 2-hour heated columns is minor and not of much significance. Moreover, specimens 1L-1h, 1L-2h, and 1L-4h exhibit 10%, 31%, and 68% decrease in capacity compared to their control counterpart at room temperature conditions. In addition to the degradation of mechanical properties that occurs in both concrete and steel at elevated temperatures, FRCM materials are also expected to exhibit decrease in ultimate tensile strength and elastic modulus upon heating [16].

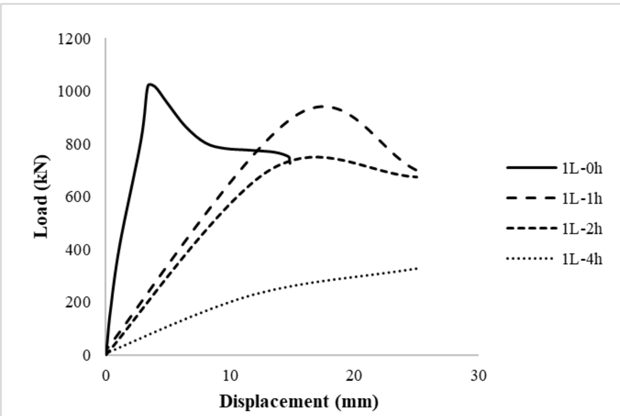


Figure 8: Load-displacement curves for strengthened columns.

Table 2 provides the axial capacities obtained for all specimens. Generally, the ultimate strengths for the strengthened specimens are significantly high compared to those for the unstrengthened columns. The nature of the bond between the FRCM cement layer and the concrete substrate as well as its superior performance under high temperature exposure compared to inorganic matrices provide FRCM with potentially improved fire performance compared to FRP composite laminates in strengthening and retrofitting applications [1].

Table 2: Column axial capacities

Specimen	Axial Capacity (kN)
C-0h	957
C-1h	817
C-2h	647
C-4h	206
1L-0h	1020
1L-1h	914
1L-2h	704
1L-4h	329

7 CONCLUSIONS

FRCM strengthening systems are expected to display improved fire performance compared to FRP systems due to the nature of the binding matrix in terms of fire resistance and bond to the concrete substrate. In this study, a finite element model consisting of a heating and loading stages was developed to predict the residual axial capacity of PBO FRCM strengthened RC columns exposed to fire. The following conclusions can be drawn from the results obtained:

- The proposed finite element approach was successful in predicting the axial capacity of unstrengthened and TRM strengthened concrete cylinders.
- The unstrengthened RC columns displayed a decrease in axial capacity along with a decrease in stiffness upon heating. On the other hand, an increase in ductility of the columns due to increase in temperature was observed.
- Shell sensitivity analysis of strengthened columns conducted at room temperature revealed negligible differences in in ultimate capacity and similar load-displacement curve behavior.
- PBO FRCM strengthened columns displayed similar behavior compared to the unstrengthened columns upon heating, experiencing decrease in strength and stiffness along with increase in ductility with longer heating durations. The overall axial capacities of the heated strengthened columns were higher than the corresponding control columns heated for the same durations.

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