

STRESS-BLOCK PARAMETERS FOR REINFORCED CONCRETE BEAMS DURING FIRE EVENTS

S.F. El-Fitiany and M.A. Youssef *

Department of Civil and Environmental Engineering, The University of Western Ontario,
London, ON, Canada N6A 5B9

Biography:

ACI student member **S.F. El-Fitiany** is a Ph.D. candidate at The University of Western Ontario, Canada. Mr. El-Fitiany obtained his M.E.Sc. from The University of Western Ontario and his B.Sc. from Alexandria University, Egypt. His research interests include behavior of structures during and after exposure to fire and development of performance based fire design approaches.

ACI member **M.A. Youssef** is an associate professor of Civil Engineering at The University of Western Ontario. Dr. Youssef received his Ph.D. from McMaster University, Hamilton, Ontario. His research interests include: modeling of reinforced concrete structures, fire resilience of concrete structures, use of innovative materials (Fiber Reinforced Polymers, FRP, and Shape Memory Alloys, SMA) to upgrade the capacity of existing structures, using SMA to minimize structural damage during earthquakes, and seismic capacity of existing structures.

* Corresponding authors: E-mail: youssef@uwo.ca, Phone: 519-661-2111 Ext. 88661

ABSTRACT

Fire safety is a critical criterion for designing reinforced concrete (RC) structures. As new design codes are moving towards performance-based design, analytical tools are needed to help engineers satisfy code criteria. These tools are also needed to assess the fire performance of critical structures. As full scale experiments and finite element simulations are usually expensive and time consuming options for designers to achieve specific fire performance, a simplified sectional analysis methodology that tracks the axial and flexural behavior of RC square sections subjected to elevated temperatures from their four sides was previously developed and validated by the authors. In the first part of this paper, the proposed methodology is extended to cover rectangular beams subjected to standard ASTM-E119 fire from three sides. An extensive parametric study is then conducted to study the distribution of the concrete compressive stresses at different ASTM-E119 fire durations. Based on the parametric study, simple equations expressing the equivalent stress-block parameters at elevated temperatures are presented. These equations can be utilized by designers to accurately estimate the flexure capacity of simply supported and continuous beams exposed to fire temperatures.

Keywords: Concrete; Elevated temperatures; Sectional analysis; Fire resistance, Stress-block parameters.

INTRODUCTION

Concrete as well as steel reinforcing bars experience significant deterioration when subjected to elevated temperatures¹. This deterioration is accompanied by the generation of thermal and transient strains which adds to the complexity of estimating the flexural capacity of a reinforced concrete (RC) section at elevated temperatures. Currently, concrete structures are designed for

fire safety using prescribed methods that are based on computational modeling and experimental investigations. These methods usually specify the minimum cross-section dimensions and clear cover to achieve specific fire ratings. As new codes are moving towards performance-based design and conducting experimental tests to satisfy different fire scenarios would be an expensive solution, numerous design tools are needed by design engineers². One of these tools would facilitate the estimation of the flexural behavior of a RC beam at elevated temperatures. These tools are also needed to estimate the fire safety of critical structures.

A simplified method to track the axial and/or the flexural behavior of square column sections subjected to fire at their four sides was previously introduced by El-Fitiany and Youssef³. This paper starts by extending the proposed method to be applicable to RC beams exposed to fire from three sides. The overall behavior of RC beams during fire exposure is tracked by constructing the moment-curvature relationships at different fire durations. The unrestrained simply supported beam tested by Lin et al.⁴, Fig. 1a, is taken as an illustrative example for the proposed methodology. The tested beam has a normal strength concrete with carbonate aggregate and subjected to ASTM-E119 standard fire.

Civil engineers are familiar with the use of concrete stress-block parameters to calculate the ultimate capacity of RC members at ambient temperatures. These parameters convert the parabolic distribution of concrete compression stresses to an idealized rectangular stress-block. Evaluation of these parameters at elevated temperatures allows designers to easily estimate the flexural capacity of RC beams during fire exposure. The second part of this paper presents a parametric study to evaluate the compressive stresses distribution for different rectangular concrete cross-sections. The effect of different parameters including section dimensions, reinforcement ratio, concrete strength, fire duration, and aggregate type is evaluated. Equations to predict the stress block parameters at elevated temperatures are developed. The use of these

equations to predict ultimate moment of concrete beam in fire conditions is explained and their estimates is compared to other standard methods.

RESEARCH SIGNIFICANCE

The proposed simplified method extends the use of sectional analysis to be applicable at elevated temperatures. Designers are familiar with this method at ambient temperature, which will allow them to use it in their fire calculations. The ultimate/nominal flexural capacity of RC beams can be evaluated at ambient temperature using equivalent stress-block parameters. The second part of this paper presents an extensive parametric study on the non-linear distribution of compression stresses for a number of rectangular cross-sections at different ASTM-E119 fire durations up to 2.5 hr. It ends by providing simplified equations for designers to allow them to estimate the stress-block parameters at elevated temperatures.

SECTIONAL ANALYSIS AT AMBIENT TEMPERATURE

At ambient temperature, RC sections can be analyzed using the well-known sectional analysis approach⁵. For cases of single curvature, i.e. bending about horizontal axis, the concrete section is divided into horizontal discrete layers. Utilizing the uniaxial stress-strain relationship for each layer and taking into account equilibrium and kinematics, the mechanical behavior of the section is analyzed. To simplify the analysis, two variables can be assumed; incremental centroidal axial strain, $\Delta\epsilon_c$, and incremental curvature, $\Delta\psi$. Assuming a linear strain distribution, the incremental moment and axial force are obtained using Eq. (1).

$$\begin{pmatrix} \Delta M \\ \Delta P \end{pmatrix} = \begin{pmatrix} \sum_{i=1}^n E_i \times A_i \times y_i^2 & -\sum_{i=1}^n E_i \times A_i \times y_i \\ -\sum_{i=1}^n E_i \times A_i \times y_i & \sum_{i=1}^n E_i \times A_i \end{pmatrix} \times \begin{pmatrix} \Delta \psi \\ \Delta \varepsilon_c \end{pmatrix} \quad (1)$$

Where E_i is the modulus of elasticity of layer i , A_i is the area of layer i , y_i is the distance between the center of area of layer i and center of area of the cross-section.

For a given axial load, the moment-curvature behavior is obtained in two stages. In the first stage, the axial strain is increased incrementally while curvature is kept equal to zero until reaching the required axial load. In the second stage, the axial load is kept constant and the applied curvature is increased. The corresponding change in the axial strain and the moment are calculated using Eq. (1). This process is repeated until reaching the required curvature value.

SECTIONAL ANALYSIS AT ELEVATED TEMPERATURES

To apply sectional analysis at elevated temperatures, a number of modifications were proposed and validated by El-Fitiany and Youssef³. These modifications account for the two dimensional temperature gradient within the concrete cross section, which affects its homogeneity and increase the nonlinearity of the mechanical strain distribution. The following sections generalize the previously developed method to be applicable to rectangular sections exposed to fire temperature at a number of or all of their sides. Beam B1, shown in Fig. 1, is used to illustrate the concepts. The beam is exposed to ASTM-E119 fire at three of its sides for duration of an hour.

Concrete and steel constitutive models

The constitutive models proposed by Youssef and Moftah⁶ are adopted and their application to beam B1 is presented in the following sub-sections.

Concrete compressive strength

Hertz model⁷, Eq. (2), is used to predict the reduced concrete compressive strength (f'_{cT}) at elevated temperatures. If concrete is loaded prior to fire, f'_{cT} should be increased by 25%⁷.

$$f'_{cT} = R \times f'_c \quad (2.a)$$

$$R = \frac{1}{1 + \frac{T}{15,000} + \left(\frac{T}{800}\right)^2 + \left(\frac{T}{570}\right)^8 + \left(\frac{T}{100,000}\right)^{64}} \quad , \text{ for concrete with siliceous aggregate } (2.b)$$

$$R = \frac{1}{1 + \frac{T}{100,000} + \left(\frac{T}{1080}\right)^2 + \left(\frac{T}{690}\right)^8 + \left(\frac{T}{1000}\right)^{64}} \quad , \text{ for concrete with carbonate aggregate } (2.c)$$

Where R is a reduction factor, T is the temperature in degree Celsius [$1^\circ\text{F} = 1.8^\circ\text{C} + 32$], and f'_c is the concrete compressive strength at ambient temperature.

Fire induced strains

Total concrete strain at elevated temperatures (ε_{tot}) is composed of three terms⁶: instantaneous stress related strain (ε_{fT}), unrestrained thermal strain (ε_{th}), and transient creep strain (ε_{tr}). The value of ε_{fT} at the peak stress (ε_{oT}) defines the stress-strain relationship during the heating stage and can be predicted using the model proposed by Terro⁸, Eq. (3). Flexural RC elements have different stress values within concrete compression zone which implies different preloading levels λ_L for each layer.

$$\varepsilon_{oT} = (50\lambda_L^2 - 15\lambda_L + 1) \varepsilon_{o1} + 20(\lambda_L - 5\lambda_L^2) \varepsilon_{o2} + 5(10\lambda_L^2 - \lambda_L) \times 0.002 \quad (3)$$

where $\varepsilon_{o1} = 2.05 \times 10^{-3} + 3.08 \times 10^{-6} T + 6.17 \times 10^{-9} T^2 + 6.58 \times 10^{-12} T^3$
 $\varepsilon_{o2} = 2.03 \times 10^{-3} + 1.27 \times 10^{-6} T + 2.17 \times 10^{-9} T^2 + 1.64 \times 10^{-12} T^3$

ε_{th} is the free thermal strain resulting from fire temperature and can be predicted using the Eurocode model⁶, Eq. (4).

$$\varepsilon_{th} = -1.8 \times 10^{-4} + 9 \times 10^{-6} (T - 20) + 2.3 \times 10^{-11} (T - 20)^3 \leq 14 \times 10^{-3} \quad , \text{ for concrete with siliceous aggregate} \quad (4.a)$$

$$\varepsilon_{th} = -1.2 \times 10^{-4} + 6 \times 10^{-6} (T - 20) + 1.4 \times 10^{-11} (T - 20)^3 \leq 12 \times 10^{-3} \quad , \text{ for concrete with carbonate aggregate} \quad (4.b)$$

ε_{tr} is induced during the first heating cycle of loaded concrete and is considered the largest component of the total strain. Its value can be estimated using Terro's model⁸.

$$\varepsilon_{tr} = \varepsilon_{0.3} \times \left(0.032 + 3.226 \frac{f_c}{f'_c} \right) \frac{V_a}{0.65} \quad (5)$$

Where

V_a is the volume fraction of aggregates

$\varepsilon_{0.3}$ is the transient creep strain for initial axial stress of $0.3 f'_c$, and is given by Eq. (6)

$$\varepsilon_{0.3} = 43.87 \times 10^{-6} - 2.73 \times 10^{-8} T - 6.35 \times 10^{-8} T^2 + 2.19 \times 10^{-10} T^3 - 2.77 \times 10^{-13} T^4 \quad (6)$$

Reinforcing steel tensile stress-strain relationship

Lie's model¹ is used to predict the reduced yield strength of reinforcing bars f_{yT} , Eq. (7).

$$f_{yT} = \left(1 + \frac{T}{900 \times \ln(T/1750)} \right) \times f_y \quad 0 < T \leq 600 \text{ } ^\circ\text{C} \quad [32 < T \leq 1112 \text{ } ^\circ\text{F}] \quad (7.a)$$

$$f_{yT} = \left(\frac{340 - 0.34 \times T}{T - 240} \right) \times f_y \quad 600 < T \leq 1000 \text{ } ^\circ\text{C} \quad [32 < T \leq 1112 \text{ } ^\circ\text{F}] \quad (7.b)$$

Lie¹ has also proposed another model representing a general stress-strain ($f_{sT} - \varepsilon_{sT}$) relationship of reinforcing bars at elevated temperatures, Eq. (8). The effect of creep of steel bars is found to

have a minor effect on the behavior of RC sections during fire exposure⁹ and, thus is not included in this study.

$$f_{sT} = \frac{f(T,0.001)}{0.001} \times \varepsilon_{sT} \quad \varepsilon_{sT} \leq \varepsilon_p \quad (8.a)$$

$$f_{sT} = \frac{f(T,0.001)}{0.001} \times \varepsilon_p + f(T, [\varepsilon_{sT} - \varepsilon_p + 0.001]) - f(T,0.001) \quad \varepsilon_{sT} > \varepsilon_p \quad (8.b)$$

$$\varepsilon_p = 4 \times 10^{-6} f_y \quad (8.c)$$

$$f(T,0.001) = (50 - 0.04T) \times [1 - e^{(-30+0.03T)\sqrt{0.001}}] \times 6.9 \quad (8.d)$$

Concrete compressive stress-strain relationship

The model proposed by Youssef and Moftah⁶ is adopted in this study. The model includes simplified representation of transient creep strains. The relationship between the compressive stress, f_{cT} , and the corresponding compressive strain, ε_{cT} , is given by Eq. (9).

$$f_{cT} = K_{hT} \times f'_{cT} \left[2 \times \left(\frac{\varepsilon_{cT}}{\varepsilon_{oT} + \varepsilon_{tr}} \right) - \left(\frac{\varepsilon_{cT}}{\varepsilon_{oT} + \varepsilon_{tr}} \right)^2 \right] \quad \varepsilon_{cT} \leq \varepsilon_{oT} + \varepsilon_{tr} \quad (9.a)$$

$$f_{cT} = K_{hT} \times f'_{cT} [1 - Z(\varepsilon_{cT} - \varepsilon_{oT} - \varepsilon_{tr})] \quad \geq 0.2 \times f'_{cT} \quad \varepsilon_{oT} \geq \varepsilon_{oT} \varepsilon_{tr} \quad (9.b)$$

where,

$$K_{hT} \text{ (confinment factor)} = 1 + \frac{\rho_s \times f_{yT}}{f'_{cT}} \quad (9.c)$$

$$\rho_s = \frac{\text{volume of transverse reinforcement}}{\text{volume of concrete core measured to their perimeter}}$$

f_{yT} is the reduced yield strength for the stirrups at elevated temperature

Z is the slope of the descending branch of the concrete stress-strain relationship and is given by Eq. (9.d)

$$Z = \frac{0.5}{\frac{3 + 0.29 f'_c}{145 f'_c - 1000} \times \frac{\varepsilon_{oT}}{\varepsilon_o} - \varepsilon_{oT}} \quad (9.d)$$

The ultimate compressive strain at failure can be assumed 0.0035 in ambient conditions according to the Canadian standards CSA A23.3-04¹⁰. Due to the limited literature on the failure compressive strain at elevated temperature, this value is increased by the transient strain ε_{tr} as proposed by El-Fitiany and Youssef³.

$$\varepsilon_{cuT} = \varepsilon_{cu} + \varepsilon_{tr} \quad (10)$$

Heat transfer model

Several methods were developed to predict the temperature distribution in a concrete section during fire exposure¹. The Finite Difference Method (FDM) is chosen in this research because of its ability to account for irregular shapes, its accuracy, and the ease of implementation in any programming code. A detailed description of the FDM, in the form of prescribed equations, is given by Lie et al.¹.

For beam B1, a 45 degree heat transfer mesh of 5.4 mm by 5.4 mm [0.21 in] square elements is generated as shown in Fig. 2. Based on the size of the elements, the total fire duration ($\tau_f = 1$ hour), is divided into time steps $\Delta \tau_f$ of 4.2 seconds to accurately predict the temperature of the elements. Concrete initial moisture content is assumed to be zero due to its negligible effect on the temperature predictions¹. A heat analysis based on the FDM is then conducted and the temperatures for steel bars were found to range from 302 °C [576 °F] to 513 °C [955 °F]. Fig. 3 shows a comparison between the average measured temperatures of bottom steel bars by Lin et al.⁴ and the FDM predictions at different fire durations. To allow using sectional analysis, the 45 degree mesh elements are converted to horizontal square mesh elements^{1,3}. The temperature at the

center of each square element, Fig. 2b, is taken as the average temperature of the adjacent 45 degree mesh elements.

Average layer temperature

The methodology proposed by El-Fitiany and Youssef³ is adopted. The square mesh elements are grouped into horizontal layers to simplify the use of sectional analysis. Therefore, an equivalent temperature T_i has to be assigned for each layer to allow estimating the concrete compressive strength, its modulus of elasticity, transient creep, and thermal strains. To accurately predict the behavior using sectional analysis, El-Fitiany and Youssef³ suggested the use of two different T_i 's, one for estimating stresses and the other for strain values. It is clear from Eq. (1) that the tangential modulus of elasticity is the most important factor, thus the first average layer temperature is estimated such that it produces the average modulus of elasticity for the square elements within the layer. At elevated temperatures, initial modulus of elasticity of loaded concrete is proportional to its reduced compressive strength⁶. Therefore, the first average temperature distribution for each layer is based on the average strength of the square mesh elements, Fig. 2d, composing this layer. The second average temperature distribution is used to estimate the thermal and transient creep strains. Eqs. (4) and (5) show that these variables are proportional to the fire temperature. Thus, the second temperature is equal to the algebraic average of the square mesh elements composing the layer. Fig. 4 shows the two distributions for the analyzed beam section after one hour of ASTM-E119 standard fire exposure. The temperature of steel bars can be assumed to be the same as the temperature of the square mesh element within which they are located¹.

Thermal and effective strain calculation

Fig. 5 shows the expected linear distribution of total strains (ε_{tot}) resulting from an applied moment (M). This linear shape is based on the fact that plane sections remain plane after loading, which is still valid at elevated temperatures^{11,12}. El-Fitiany and Youssef³ used the same principle but their total strain was having a constant value representing the case for a section exposed to fire from four sides. The distribution of ε_{th} for a rectangular cross-section subjected to fire temperature from three sides is shown in Fig. 5.

For unrestrained concrete sections, the effective strain (ε_{eT}) can be calculated by subtracting concrete and steel thermal strains from the total strain. The nonlinear distribution of thermal strains results in a nonlinear effective (mechanical) strain distribution, Fig. 5. As Eq. (1) is only applicable for linear strain distributions, the following sub-sections propose a methodology to conduct sectional analysis for the non-linear varying effective strain distribution.

Isolation of thermal strain component for beam sections

For square column sections exposed to fire temperature from four sides, details for thermal strain equilibrium are given by El-Fitiany and Youssef³. For the case of a rectangular cross-section exposed to fire from three sides, the thermal strain is expected to be asymmetric as shown in Fig. 5. The equivalent shape is defined by the mid-height axial strain ($\overline{\varepsilon_{th}}$) and the curvature ($\overline{\psi_{th}}$), Figs. 5 and 6. The values of $\overline{\varepsilon_{th}}$ and $\overline{\psi_{th}}$ are evaluated such that the axial forces and bending moments in concrete and steel layers resulting from the difference between the actual thermal strain distribution and $\overline{\varepsilon_{th}}$ are in self equilibrium. An iterative procedure is used to calculate the values of $\overline{\varepsilon_{th}}$ and $\overline{\psi_{th}}$ such that the forces, shown in Fig. 6.b, are producing zero axial force and zero moment. Concrete tensile strength is neglected in the analysis.

Fig. 6.a shows the nonlinear thermal strain distribution for the studied beam B1 after 1 hr standard ASTM-E119 fire exposure. The presented thermal strain distribution is converted to a linear distribution by considering section equilibrium. The equivalent uniform strain reflects the actual deformation of the concrete section under zero external loads and moments. Differences between the non-linear and equivalent uniform strains represent concrete and steel internal stresses that are in equilibrium.

Thermally induced stresses

The conversion from the actual nonlinear strain distribution to the equivalent linear distribution induces self-equilibrating internal strains ($\varepsilon_{\sigma th}$), Fig. 6.b. These internal strains are part of mechanical strains required to retain the geometric linearity of B1's cross-section. To account for these strains, they are included as initial strains for each concrete and/or steel layer.

ANALYSIS STEPS OF RC BEAMS UNDER FIRE LOADING

Sectional analysis can be divided into three main steps;

1. The heat transfer model is applied and the heat gradient through the cross section is predicted. The average temperatures for each layer are then obtained.
2. The equivalent uniform thermal strain $\overline{\varepsilon_{th}}$ and the curvature $\overline{\psi_{th}}$ are then calculated by equilibrating the forces in the concrete and steel layers resulting from the actual thermal strain distribution. The difference between the actual and uniform strain distributions represents the induced strains $\varepsilon_{\sigma th}$ in concrete and steel layers to satisfy the section geometry. These strains are considered as initial strains in the following step.
3. Sectional analysis is conducted to construct the moment-curvature diagrams.

Validation of the sectional analysis methodology

Moment-curvature curves represent the flexural behavior of a RC section under specified axial load level ($\lambda = f_c / f'_c$). Fig. 7 shows the effect of 1 hr standard ASTM-E119 fire exposure on the studied unrestrained section of beam B1 (λ equals to zero). As shown in Fig. 7, elevated temperatures increase the ductility and reduce the capacity of RC sections during fire exposure. The initial point of the moment-curvature diagram after 1 hr fire exposure defines the equilibrium curvature $\overline{\psi_{th}}$. This curvature value represents the initial rotation of B1 due to the non-linear thermal distribution combined with the material weakening. This initial curvature will occur regardless of the external effect of the applied moments.

The moment-curvature diagrams were constructed for beam B1 at different fire durations. The vertical deflection w^* at each time step was estimated by applying the moment-area method to the estimated curvature distribution along the beam length. The obtained results are plotted in Fig. 8. An excellent matching is found between the sectional analysis, the FEM conducted by Kodur and Dwaikat², and the experimental results (up to 80 minutes). Failure criteria proposed by BS 476 and adopted by Kodur and Dwaikat² are used. These criteria are setting limits for the maximum allowable deflection, Eq. (11), and maximum rate of deflection, Eq. (12).

$$\Delta_{\max} = \frac{L}{20} = 305 \text{ mm} \quad [12 \text{ in}] \quad (11)$$

$$\left(\frac{\partial \Delta}{\partial t} \right)_{\max} = \frac{L^2}{9000d} = 13 \text{ mm/minute} \quad [0.51 \text{ in/minute}] \quad (12)$$

Where L is the span between the supports (mm) and d is the effective depth of the beam (mm) [1 in = 25.4 mm]

As shown in Fig. 8, the proposed sectional analysis results in about 17 minutes difference in predicting failure compared with the reported fire test resistance.

STRESS-BLOCK PARAMETERS AT AMBIENT TEMPERATURE

Reinforced concrete beams are currently designed for flexure at ambient conditions by assuming a linear strain distribution and converting the nonlinear stress distribution to an equivalent stress-block. This conversion is done using the stress-block parameters α_1 and β_1 . The recommended values for the stress-block parameters at ambient temperature in the Canadian code CSA A23.3-04¹⁰ depend on f'_c to account for high strength concrete and are given by:

$$\alpha_1 = 0.85 - 0.0015 \times f'_c \quad (13.a)$$

$$\beta_1 = 0.97 - 0.0025 \times f'_c \quad (13.b)$$

The nominal moment at failure M_n can therefore be calculated from section equilibrium as follows

$$C = \alpha_1 \times f'_c \times \beta_1 \times c \times b \quad (14.a)$$

$$T_s = f_y \times A_s \quad (14.b)$$

$$C = T_s \quad (\text{calculate } c) \quad (14.c)$$

$$M_n = (C \text{ or } T_s) \times \left(d - \frac{\beta_1 \times c}{2}\right) \quad (14.d)$$

Where C is the compressive force in the concrete, c is the depth of neutral axis, b is the width of the compression zone, T_s is the tension force in steel assuming yielding or reinforcement, f_y is the yield stress of steel bars, A_s is the area of tensile steel bars, and d is the effective depth of the section.

STRESS-BLOCK PARAMETERS AT ELEVATED TEMPERATURES

The effect of fire temperature on the ambient stress-block parameters α_1 and β_1 is evaluated in this section through an extensive parametric study. A total of 28 rectangular cross-sections are

analyzed to study the stress distribution at different fire durations. Table 1 shows details of the beams that are subjected to sagging bending moments (positive moments). On the other hand, Table 2 shows details of beams that are subjected to hogging bending moments (negative moments). The studied parameters are; fire duration (τ_f), geometry of the sections (b and h), reinforcement ratio (ρ) and configuration, concrete compressive strength (f'_c), and aggregate type. The effect of compression reinforcing bars is neglected for simplicity.

Fig. 9 shows details for the studied cross-sections. All the beams are subjected to the standard ASTM-E119 fire from three faces, i.e. the two sides and the bottom surface of each beam. A sectional analysis was conducted for each beam at different fire durations starting from 0.0 hr and up to 2.5 hr with a time interval of 15 min. For each time step, the stress-block parameters at elevated temperature α_{1T} and β_{1T} are evaluated by predicting the total strain distribution (ϵ_{tot}) at failure using the proposed sectional analysis methodology. The mechanical strain (ϵ_{cT}) is then isolated and the stress distribution is obtained based on it. The following sub-sections explain these steps for beams B4, sagging moment, and B18, hogging moment.

Behavior of beams subjected to sagging moments during fire

A heat transfer analysis was conducted for beam B4. The height of the beam was slightly increased to 705 mm [27.8 in] to retain the aspect ratio of the 45 degree heat transfer mesh. Fig. 10 shows the average temperature distributions along B4 cross-section height.

The studied beam B4 was analyzed using the proposed sectional analysis methodology. A constant spacing for stirrups of 600 mm [23.6 in] was assumed. Fig. 10 shows the predicted total linear strain distribution at failure. The mechanical strain ϵ_{cT} is isolated from the total strain ϵ_{tot} by subtracting the thermal strain ϵ_{th} . Knowing the constitutive model for the stress-strain

relationship of concrete at elevated temperatures, i.e. Eq. (7), the stress distribution is plotted as explained earlier in this paper.

The equivalent rectangular stress-block is obtained by considering the equilibrium in forces and moments between the equivalent block and the actual stress distribution. The stress-block parameters α_{1T} and β_{1T} of beam B4 after 1 hr fire exposure are found to be 0.87 and 0.80, respectively.

Fig. 10 indicates that the nonlinear effect of the bottom surface heating is limited to the bottom zone of the beam cross-section where the tensile reinforcement is located while temperature distribution is uniform in the concrete compression zone. This uniform temperature has consequently resulted in a linear mechanical strain distribution because of the constant thermal strain ε_{th} at the top of the concrete section. The uniform temperature has also resulted in a constant concrete stress-strain relationship for concrete at different locations in the compression zone. This explains why the stress distributions at elevated temperatures for beams subjected to sagging moments have similar shape to the ambient temperature. The uniform average temperatures in the compression zone are found to be functions of the section width and aggregate type. Fig. 11 allows predicting the average temperature (based on the average strength) in the concrete compression zone for both siliceous and carbonate aggregate. Fig. 12 shows the algebraic average temperature in the concrete compression zone for different aggregate types.

Stress-block parameters for beams subjected to sagging moments

Tables 3 and 4 show the results of the parametric study for the beams described in Table 1. The effect of reinforcement ratio and section height on α_{1T} and β_{1T} is negligible as shown Fig. 13. The effect of the concrete compressive strength f'_c , distribution of reinforcing bars, and aggregate

type on the shape of the equivalent stress-block is negligible. On the other hand, the effect of section width b is found to have the greatest influence on the studied stress-block parameters α_{1T} and β_{1T} . This conclusion is reasonable since the effect of the fire temperature is usually limited to the outer concrete layers exposed to the flames while the beam core remains mostly undamaged. A statistical study was conducted on the results presented in Tables 3 and 4 to propose a simplified expression for α_{1T} and β_{1T} . The effects of fire duration t , concrete compressive strength f'_c , and section width b are accounted for in a multiple regression analysis using the Ordinary Least Square method (OLS). Based on the results of the parametric study, Equations (15.a) and (15.b) are proposed and plotted in Fig. 13. A good matching is found for all the results up to 2.50 hr fire duration.

$$\alpha_{1T} = \alpha_1 - 1.533 \times 10^{-2} + 24.397 \times 10^{-3} \tau_f + 15.758 \times 10^{-4} f'_c - 10.089 \times 10^{-5} b \quad (15.a)$$

$$\begin{aligned} \beta_{1T} = \beta_1 - 2.907 \times 10^{-2} + 20.734 \times 10^{-3} \tau_f^2 - 94.794 \times 10^{-3} \tau_f - 75.057 \times 10^{-5} f'_c \\ + 15.413 \times 10^{-5} b \end{aligned} \quad (15.b)$$

Flexural capacity for beams subjected to sagging moments

This section presents a methodology that can be used by designers to predict the reduced nominal flexural resistance M_{nT} for beams subjected to positive moments under ASTM-E119 standard fire exposure. This procedure is illustrated considering a 350×750 mm [13.8×29.5 in] rectangular beam (B30), Fig. 14, cast with a 35 MPa [5076 psi] siliceous aggregate concrete. The beam has reinforcement ratio ρ (grade 400) of 1.5%. The sectional analysis methodology predicted a reduced resistance moment after 1.5 hr ASTM-E119 fire exposure of 541 kN.m [3.99×10^{11} ft.lb]. The designer can predict the reduced resistance using the proposed equation, Eq. (15), using the steps explained in Appendix I. Eq. (15) estimates a nominal failure moment of 536 kN.m

[3.95×10^{11} ft.lb] with a difference of 1% from the sectional analysis. The ASCE simplified method¹ estimates 496 kN.m [3.66×10^{11} ft.lb] (difference is 8% from the sectional analysis methodology). Applying the 500 °C [932 °F] isotherm method as described in the ENV 1992-1-2¹³ requires constructing the elevated temperatures contour map within the beam cross section, Fig. 15. The next step is to neglect the concrete where the temperature is above 500 °C [932 °F] in calculating the reduced flexural strength. As shown in Fig. 15, the reduced cross section is 289×750 mm [11.4×29.5 in]. This reduced cross section results in 487 kN.m [3.59×10^{11} ft.lb] nominal failure moment, Appendix I. The difference between the 500 °C [932 °F] isotherm method and the sectional analysis methodology is 10%.

Calculation of stress-block parameters for beams subjected to Hogging moments

The same procedure illustrated for beams subjected to sagging moments is repeated here. A heat transfer analysis followed by a sectional analysis is conducted for beam B18 as an illustrative example. The mechanical strain ε_{ct} is isolated from the total strain ε_{tot} by subtracting the thermal strain ε_{th} , Fig. 16. Knowing the constitutive model for the stress-strain relationship of concrete at elevated temperatures, the stress distribution is predicted at different fire durations as shown in Fig. 16. The effect of fire on the concrete compression zone becomes more pronounced in case of negative moments. This effect is characterized by the significant reduction in the concrete compressive strength and the shape of the compression stress distribution. Fig. 10 shows a steep variation in the average temperature in the concrete compression zone located close to the heat source. This variation results in different stress-strain relationships for adjacent concrete layers. Similar stress distribution was recommended by Tan and Yao¹⁴. The stress-distribution after 1.0 hr fire exposure has two peak points. Tan and Yao have suggested to estimate α_{1T}

and β_{1T} by replacing f'_c with f'_{cT} in Eq. (13). Their modification does not have a rational basis as f'_c in Eq. (13) is accounting for the compressive strength enhancement for high strength concrete. It should be noted that concrete failure has to occur at the top compression layers where the maximum mechanical strain occurs. The top concrete layers sustain higher transient strains ε_{tr} because of the higher elevated temperatures they experience which subsequently increase the failure strain ε_{cuT} predicted by Eq. (10). The equivalent stress-block can be obtained by considering the equilibrium in forces and moments between the equivalent block and the actual stress distribution at different fire durations.

Stress-block parameters for beams subjected to hogging moments

Tables 5 and 6 show the results of the parametric study for the beams presented in Table 2. The force equilibrium factor α_{1T} is found to be better described in terms of the fire duration τ_f only, Fig. 17. The moment equilibrium factor β_{1T} is described in terms of the fire duration τ_f , reinforcement ratio ρ , aggregate type, concrete compressive strength f'_c , and section width b . Fig. 18 is presented as a sample for the obtained results. For simplicity and practicality, the reduction in the concrete compressive strength R was accounted for in the parameters α_{1T} and β_{1T} for beams subjected to negative moments.

A statistical study is conducted on the results presented in Tables 5 and 6 to propose a simplified expression for α_{1T} and β_{1T} for beams subjected to hogging moments. Eq. (16.a) and (16.b) are proposed based a multiple regression analysis using the Ordinary Least Square (OLS) method plotted in Figs 17 and 18. A good matching can be found for all the results up to 2.50 hr fire duration.

$$\alpha_{1T} = \alpha_1 - 2.735 \times 10^{-2} - 1.497 \times 10^{-1} \tau_f + 7.579 \times 10^{-2} F_{agg}. \quad (16.a)$$

$$\begin{aligned} \beta_{1T} = \beta_1 - 1.965 \times 10^{-1} - 4.054 \times 10^{-2} \left(\frac{\tau_f}{\rho} \right)^2 + 2.448 \times 10^{-1} \left(\frac{\tau_f}{\rho} \right) - 3.456 \times 10^{-2} F_{agg}. \\ + 3.687 \times 10^{-3} f'_c + 2.342 \times 10^{-4} b \end{aligned} \quad (16.b)$$

where F_{agg} is a factor to account for the aggregate type; 0.0 for siliceous concrete and 1.0 for carbonate concrete.

Flexural capacity for beams subjected to hogging moments

This section presents a methodology that can be used by designers to predict the flexural resistance M_{nT} for beam B30 in case of negative bending. The sectional analysis methodology predicted moment of resistance, after 1.5 hr ASTM-E119 fire exposure, is 627 kN.m [4.62×10^{11} ft.lb]. On the other hand, structural engineers can predict a reduced flexural resistance of 593 kN.m [4.37×10^{11} ft.lb] using Eq. (16), Appendix II. The ASCE manual¹ recommends using a reasonable average value for the compressive strength, which is hard to achieve by engineers. The 500 °C [932 °F] isotherm method recommended by the ENV EC2, 1992¹³ results in 571 kN.m [4.21×10^{11} ft.lb] failure moment with 9% difference than the sectional analysis.

SUMMARY AND CONCLUSIONS

The sectional analysis methodology, introduced by the authors in a previous publication, is extended in the first part of this paper to cover RC beams subjected to fire from three sides. The proposed methodology is found to be a simple yet accurate method to track the behavior of rectangular RC beams at elevated temperatures. In a similar fashion to ambient temperature analysis of RC beams, temperature-dependent stress-block parameters are developed in the

second part of this paper to convert the non-linear compression stresses distribution to a linear and constant stress distribution.

A parametric study aiming at investigating the effect of ASTM-E119 fire temperature on the stress distribution is conducted by applying the proposed methodology on a number of unrestrained rectangular beams. The studied parameters are; fire duration (τ_f), geometry of the sections (b and h), reinforcement ratio (ρ) and configuration, concrete compressive strength (f'_c), and aggregate type. The studied cross-sections were subjected to a standard ASTM-E119 fire durations up to 2.5 hour. For each time step, the total strain and stress distributions were predicted at failure. The actual distributions of compression stresses at different fire durations were approximated to equivalent stress-blocks. The equivalent stress-block parameters were evaluated for the studied sagging and hogging moment cases by applying a multiple regression analysis on the parametric study results. Failure of beams subjected to sagging moments was found to occur at top compression layers similar to ambient temperature. On the other hand, failure of beams subjected to hogging moments occurs at a location within the compression block where the mechanical strain in concrete reaches the failure strain at this location. Simplified expressions for the proposed stress-block parameters were derived and validated in this paper for nominal failure moment prediction. The use of the proposed parameters is relatively easy and practical to be implemented in design codes. Prediction of nominal flexural strength of fire-damaged concrete beams can also be useful to check the capacity design of beam-column assemblage in seismic design.

ACKNOWLEDGMENTS

This research was funded by the Natural Sciences and Engineering Research Council of Canada (NSERC).

REFERENCES

1. Lie, T.T., ed., "Structural Fire Protection," ASCE Manuals and Reports on Engineering Practice, no. 78, New York, NY, 1992, 241 pp.
2. Kodur, V.K.R., and Dwaikat, M., "Performance-based fire safety design of reinforced concrete beams," Journal of Fire Protection Engineering, vol. 17, no. 4, 2007, pp. 293-320.
3. El-Fitiany, S., Youssef, M.A. 2009, "Assessing the flexural and axial behaviour of reinforced concrete members at elevated temperatures using sectional analysis", Fire Safety Journal, vol. 44, no. 5, pp. 691-703.
4. Lin, T.D., Gustaferro, A.H., and Abrams, M.S., "Fire Endurance of Continuous Reinforced Concrete Beam", Portland Cement Association, Bulletin RD072.01B, Skokie, 1981.
5. Youssef, M.A., and Rahman, M., "Simplified seismic modelling of reinforced concrete flexural members," Magazine of Concrete Research, vol. 59, no. 9, 2007, pp. 639-649.
6. Youssef, M.A. and Moftah, M., "General stress-strain relationship for concrete at elevated temperatures," Engineering Structures, vol. 29, no. 10, 2007, pp. 2618-2634.
7. Hertz, K.D., "Concrete Strength for Fire Safety Design," Magazine of Concrete Research, vol. 57, no. 8, 2005, pp. 445-453.
8. Terro, M.J., "Numerical modeling of the behavior of concrete structures in fire," ACI Structural Journal, vol. 95, no. 2, 1998, pp. 183-193.

9. Bratina, S., Saje, and M., Planinc, I., "The effects of different strain contributions on the response of RC beams in fire," *Engineering Structures*, vol. 29, no. 3, 2007, pp. 418-430.
10. Cement Association of Canada, "Concrete design handbook, CAN/CSA A23.3-04," 3rd Ed., Ottawa, 2006.
11. Collins, M.P., Mitchell, D. 1987, "Prestressed Concrete Basics," Canadian Prestress Concrete Institute, Ottawa, ON, Canada.
12. Tassios, T.P., Chronopoulos, M.P., "Structural response of RC elements under fire," *The Structural Engineer*, vol. 69, no. 15, 1991, pp. 277-281.
13. Eurocode 2, "Design of Concrete Structures," ENV EC2, 1992.
14. Tan, K.H., and Yao, Y., "Fire resistance of four-face heated reinforced concrete columns," *Journal of Structural Engineering*, vol. 129, no. 9, 2003, pp. 1220-1229.
15. Anderberg, Y, and Thelandersson, S., "Stress and deformation characteristics of concrete at high temperatures: 2 experimental investigation and material behaviour model", Bulletin 54, Lund institute of Technology, Sweden, 1976.
16. Youssef, M.A., El-Fitiany, S.F., and Elfeki, M.A., "Flexural Behavior of Protected Concrete Slabs After Fire Exposure," *Designing Concrete Structures for Fire Safety*, ACI SP-255-4, 2008, pp. 47-74.

Table 1–Parametric study cases (sagging moment)

Beam #	f_y (MPa) [psi]	f'_c (MPa) [psi]	b (mm) [in]	h (mm) [in]	ρ (% A_g)	Studied variables	Notes
B2	400 [58015]	30 [4351]		502.5	1.0	h, ρ	carbonate aggregate
B3				[19.8]	2.0	h, ρ	
B4			300 [11.8]	705.0	1.0	h, ρ, b , agg. type	
B5				[27.8]	2.0	h, ρ, b , agg. type	
B6				907.5	1.0	h, ρ	
B7				[35.7]	2.0	h, ρ	
B8		40 [5802]	400 [15.7]	700.0	1.0	f'_c, b	
B9				[27.6]	2.0	f'_c, b	
B10			300 [11.8]	705.0	1.0	aggregate type	
B11				[27.8]	2.0	aggregate type	
B12				907.5	1.0		
B13				[35.7]	2.0		
B14			300 [11.8]	705.0	1.0	RFT configuration	2 layers RFT
B15				[27.8]	1.0	RFT configuration	
B16			400 [15.7]	700.0	1.0	f'_c	
B17				[27.6]	2.0	f'_c	

Table 2–Parametric study cases (hogging moment)

Beam #	f_y (MPa)	f'_c (MPa)	b (mm)	h (mm)	ρ (% A_g)	Studied variables	notes
B18	400 [58015]			705.0	1.0	h, b, ρ , agg. type	carbonate aggregate
B19				[27.8]	2.0	h, b, ρ , agg. type	
B20			300 [11.8]	907.5	1.0	h, b, ρ	
B21				[35.7]	2.0	h, b, ρ	
B22		30 [4351]		705.0	1.0	aggregate type	
B23				[27.8]	2.0	aggregate type	
B24				700.0	1.0	b, f'_c	
B25				[27.6]	2.0	b, f'_c	
B26		40 [5802]	400 [15.7]	900.0	1.0	b	
B27				[35.4]	2.0	b	
B28				700.0	1.0	f'_c	
B29				[27.6]	2.0	f'_c	

Table 3–Parametric study results (sagging moment)

τ_f (hr)	B2		B3		B4		B5		B6		B7		B8		B9	
	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}
0.00	0.846	0.881	0.847	0.880	0.842	0.881	0.845	0.879	0.836	0.886	0.829	0.891	0.834	0.888	0.837	0.885
0.25	0.856	0.852	0.855	0.852	0.847	0.856	0.850	0.860	0.846	0.857	0.844	0.858	0.839	0.869	0.842	0.865
0.50	0.860	0.828	0.863	0.825	0.860	0.830	0.850	0.830	0.854	0.831	0.851	0.833	0.843	0.846	0.847	0.844
0.75	0.861	0.817	0.866	0.813	0.860	0.816	0.861	0.815	0.857	0.818	0.858	0.817	0.850	0.832	0.851	0.831
1.00	0.870	0.803	0.874	0.800	0.870	0.802	0.872	0.800	0.870	0.802	0.873	0.799	0.850	0.826	0.858	0.820
1.25	0.879	0.796	0.882	0.790	0.879	0.793	0.880	0.790	0.877	0.792	0.878	0.791	0.857	0.817	0.862	0.811
1.50	0.880	0.789	0.886	0.782	0.882	0.786	0.885	0.782	0.883	0.784	0.884	0.782	0.862	0.809	0.868	0.803
1.75	0.885	0.782	0.887	0.778	0.885	0.779	0.887	0.777	0.886	0.778	0.888	0.776	0.867	0.803	0.873	0.796
2.00	0.891	0.778	0.891	0.772	0.887	0.775	0.890	0.772	0.888	0.774	0.890	0.771	0.870	0.799	0.878	0.792
2.25	0.887	0.775	0.891	0.769	0.890	0.772	0.892	0.768	0.889	0.771	0.892	0.768	0.876	0.793	0.880	0.787
2.50	0.887	0.772	0.892	0.766	0.890	0.768	0.892	0.765	0.892	0.767	-----	-----	0.882	0.791	-----	-----

Table 4–Parametric study results (sagging moment) - Cont'd

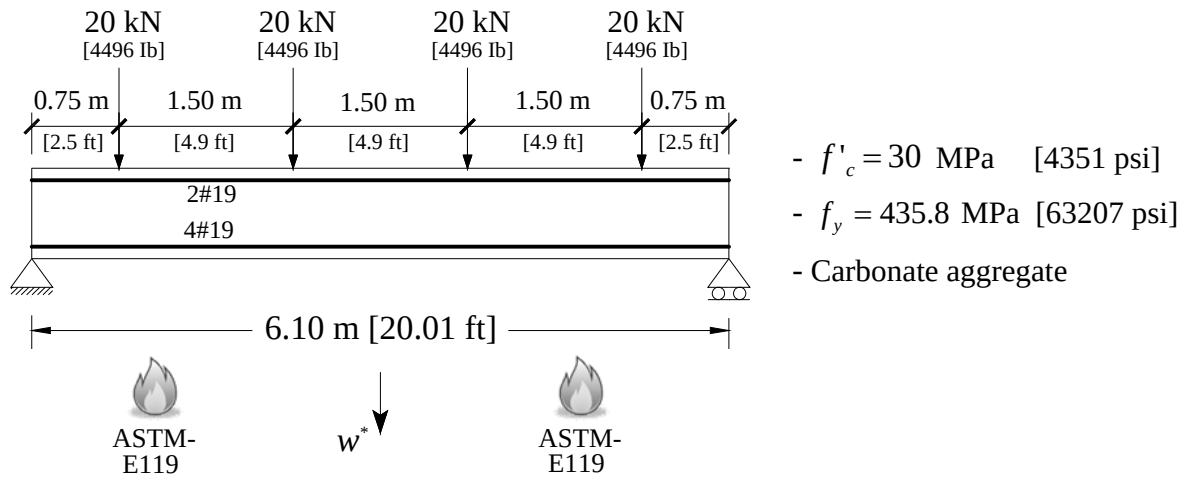
τ_f (hr)	B10		B11		B12		B13		B14		B15		B16		B17	
	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}
0.00	0.844	0.881	0.845	0.879	0.802	0.902	0.806	0.899	0.810	0.898	0.811	0.897	0.801	0.906	0.802	0.902
0.25	0.846	0.860	0.845	0.861	0.819	0.869	0.817	0.871	0.824	0.868	0.823	0.868	0.811	0.882	0.809	0.882
0.50	0.854	0.834	0.858	0.831	0.831	0.841	0.837	0.837	0.833	0.842	0.834	0.840	0.822	0.859	0.826	0.853
0.75	0.858	0.822	0.859	0.820	0.839	0.826	0.840	0.825	0.838	0.828	0.839	0.826	0.824	0.845	0.830	0.840
1.00	0.869	0.806	0.871	0.804	0.854	0.810	0.854	0.810	0.854	0.813	0.854	0.810	0.826	0.837	0.833	0.832
1.25	0.877	0.796	0.879	0.793	0.863	0.800	0.869	0.794	0.862	0.801	0.866	0.797	0.836	0.830	0.841	0.823
1.50	0.883	0.787	0.885	0.785	0.870	0.791	0.875	0.786	0.869	0.792	0.873	0.790	0.839	0.822	0.852	0.811
1.75	0.888	0.781	0.889	0.779	0.876	0.783	0.881	0.779	0.874	0.786	0.878	0.782	0.845	0.817	0.856	0.806
2.00	0.889	0.777	0.892	0.773	0.879	0.779	0.883	0.775	0.881	0.782	0.881	0.778	0.855	0.811	0.862	0.799
2.25	0.892	0.772	0.893	0.769	0.883	0.775	0.886	0.771	0.881	0.777	0.886	0.773	0.859	0.803	0.866	0.795
2.50	0.895	0.769	0.895	0.765	0.885	0.771	0.888	0.767	0.881	0.775	0.887	0.768	0.856	0.804	0.868	0.792

Table 5–Parametric study results (hogging moment)

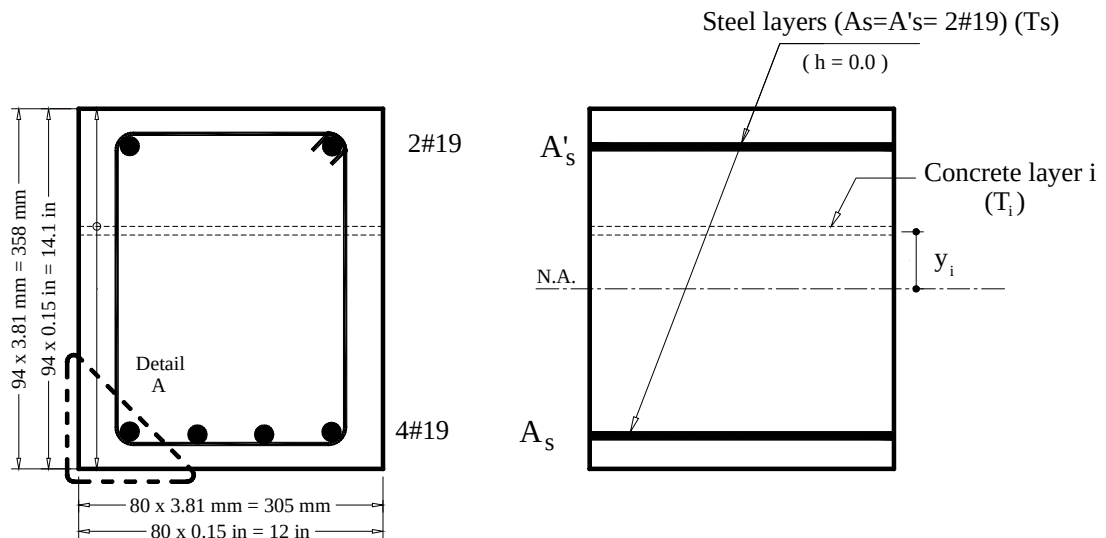
τ_f (hr)	B18		B19		B20		B21		B22		B23	
	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}
0.00	0.843	0.845	0.835	0.829	0.842	0.845	0.834	0.837	0.833	0.832	0.797	0.801
0.25	0.805	0.820	0.813	0.817	0.838	0.839	0.798	0.818	0.800	0.815	-----	0.773
0.50	0.696	0.755	0.722	0.765	-----	0.788	-----	0.752	0.685	0.736	-----	-----
0.75	0.631	0.705	0.659	0.719	0.743	0.758	0.638	0.691	0.663	0.694	-----	-----
1.00	0.557	0.660	0.598	0.691	0.707	0.761	0.572	0.662	0.613	0.653	0.518	0.605
1.25	0.491	0.604	0.536	0.640	0.668	0.739	0.515	0.627	0.559	0.615		0.575
1.50	0.439	0.553	0.483	0.593	0.631	0.717	0.464	0.585	0.510	0.573	0.410	0.530
1.75	0.396	0.508	0.437	0.548	0.596	0.693	0.422	0.543	0.468	0.532	0.371	0.492
2.00	0.359	0.468	0.399	0.507	0.564	0.667	0.387	0.505	0.431	0.495	-----	0.452
2.25	0.327	0.432	0.366	0.471	0.534	0.641	0.357	0.473	0.399	0.462	0.310	0.423
2.50	0.298	0.400	0.336	0.438	0.507	0.616	0.330	0.442	0.371	0.433	0.285	-----

Table 6–Parametric study results (hogging moment) - Cont'd

τ_f (hr)	B24		B25		B26		B27		B28		B29	
	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}	α_{IT}	β_{IT}
0.00	0.881	0.879	0.886	0.891	0.881	0.879	0.888	0.885	0.887	0.887	0.906	0.902
0.25	0.927	0.902	0.915	0.898	0.912	0.892	0.933	0.908	0.925	0.909	-----	0.930
0.50	0.995	0.933	0.968	0.919	-----	0.918	-----	0.944	0.994	0.953	-----	-----
0.75	1.047	0.964	1.012	0.944	0.989	0.937	1.055	0.982	1.021	0.982	-----	-----
1.00	1.101	0.993	1.057	0.961	1.022	0.945	1.110	1.005	1.063	1.013	1.167	1.050
1.25	1.140	1.021	1.091	0.984	1.057	0.964	1.157	1.033	1.106	1.043	-----	1.081
1.50	1.173	1.045	1.123	1.002	1.089	0.985	1.191	1.063	1.142	1.074	1.250	1.115
1.75	1.200	1.068	1.150	1.022	1.114	1.007	1.223	1.087	1.170	1.098	1.276	1.140
2.00	1.221	1.089	1.173	1.042	1.130	1.028	1.246	1.110	1.196	1.121	-----	1.166
2.25	1.240	1.108	1.193	1.059	1.140	1.047	1.268	1.131	1.218	1.143	1.318	1.187
2.50	1.255	1.126	1.209	1.075	1.150	1.060	1.287	1.152	1.239	1.163	1.333	-----



(a) Beam B1 elevation



(b) Cross-section and fiber model of beam B1

Fig. 1-Beam B1 loading and configuration

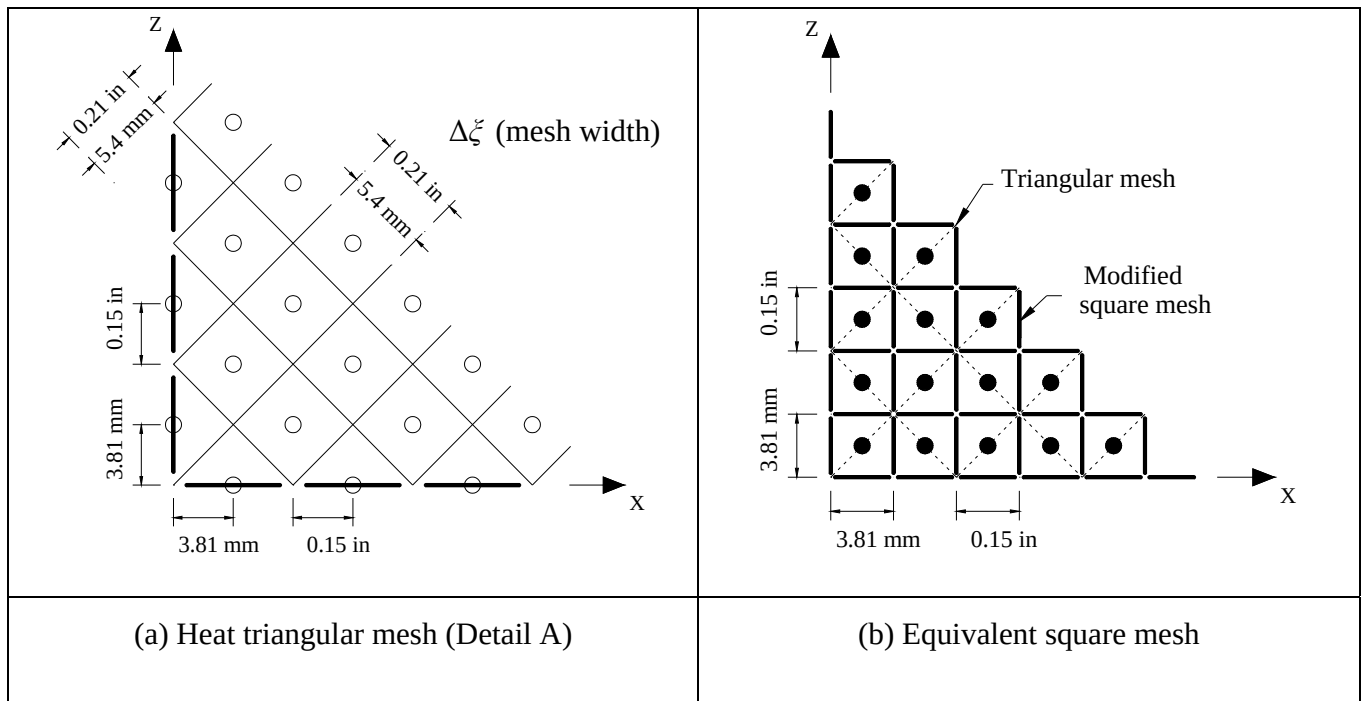


Fig. 2-Heat Transfer mesh of beam B1

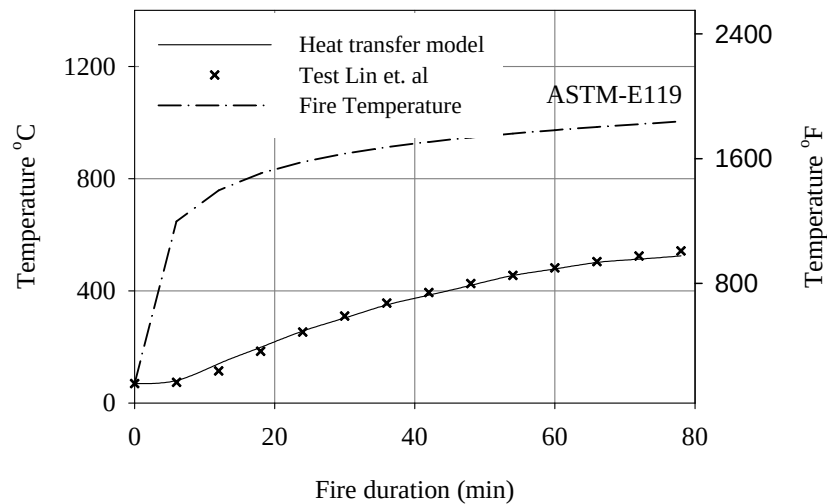


Fig. 3. Validation of the heat transfer model (average temperature of bottom reinforcing bars)

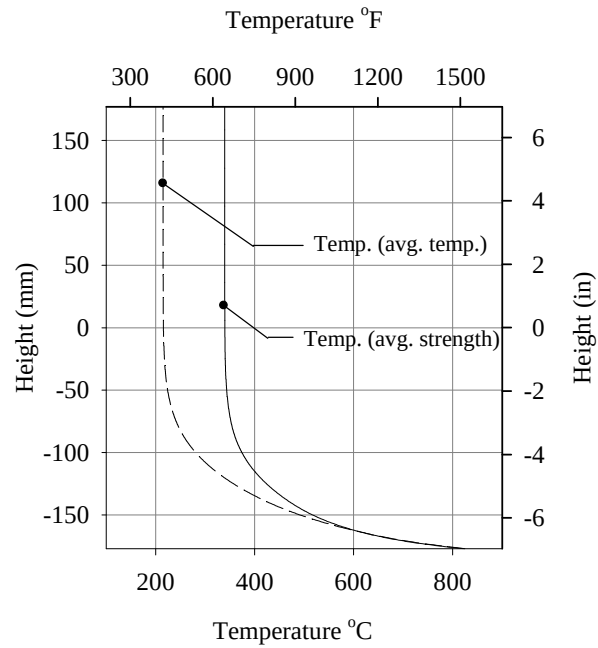


Fig. 4-Average temperature distributions for B1 after 1hr ASTM-E119 fire exposure

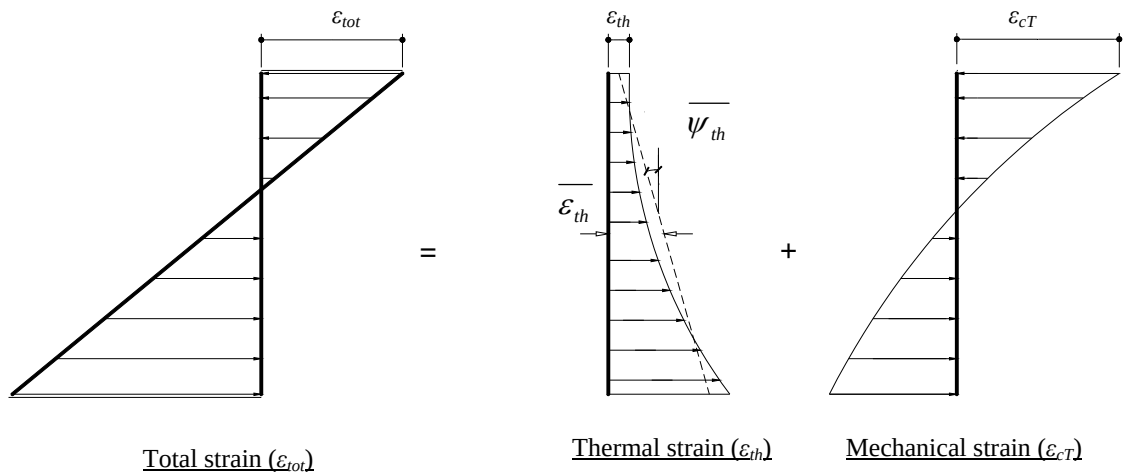


Fig. 5-Components of total strain at elevated temperatures

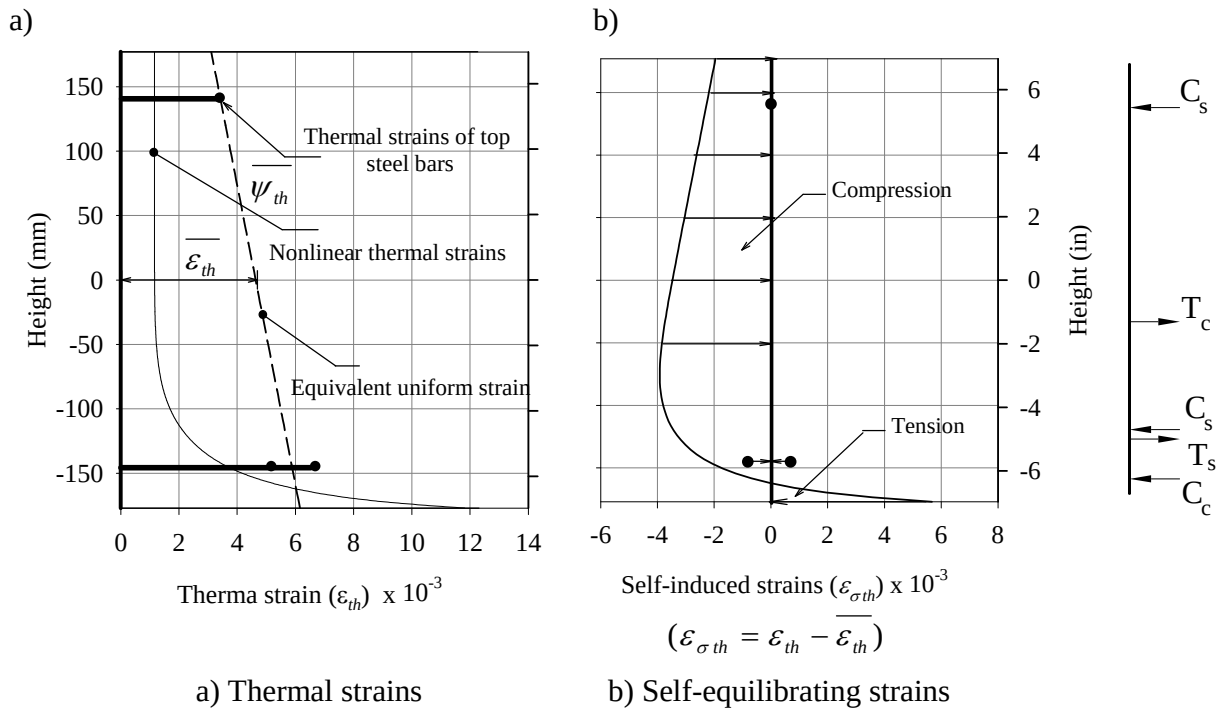


Fig. 6-Strain distribution and self-equilibrating forces along B1 height

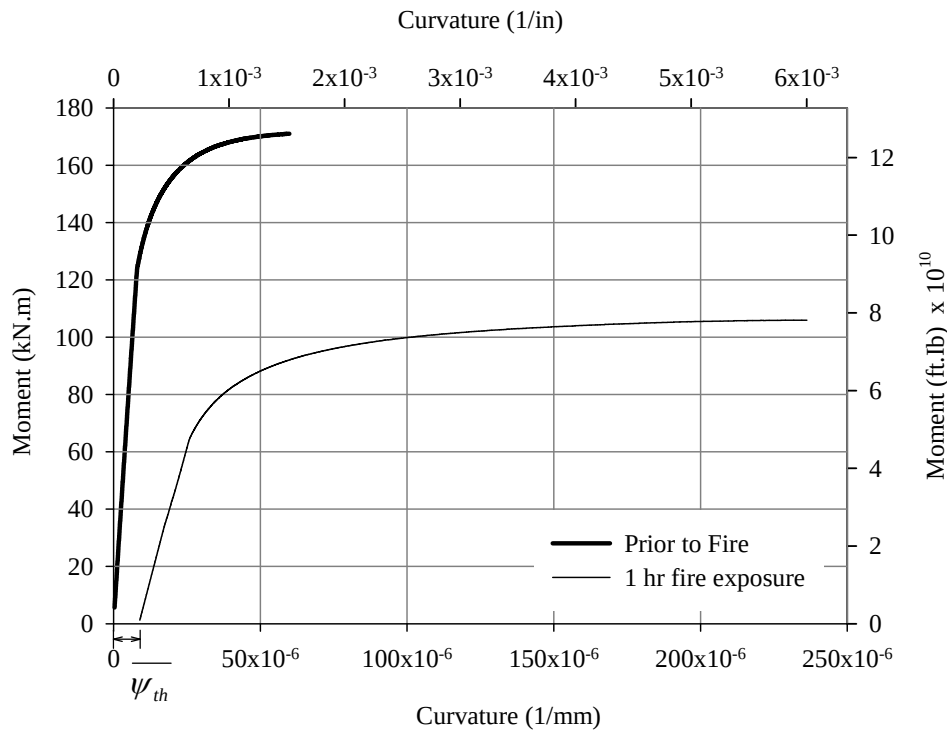


Fig. 7-Effect of fire temperature on the moment-curvature relationship for B1

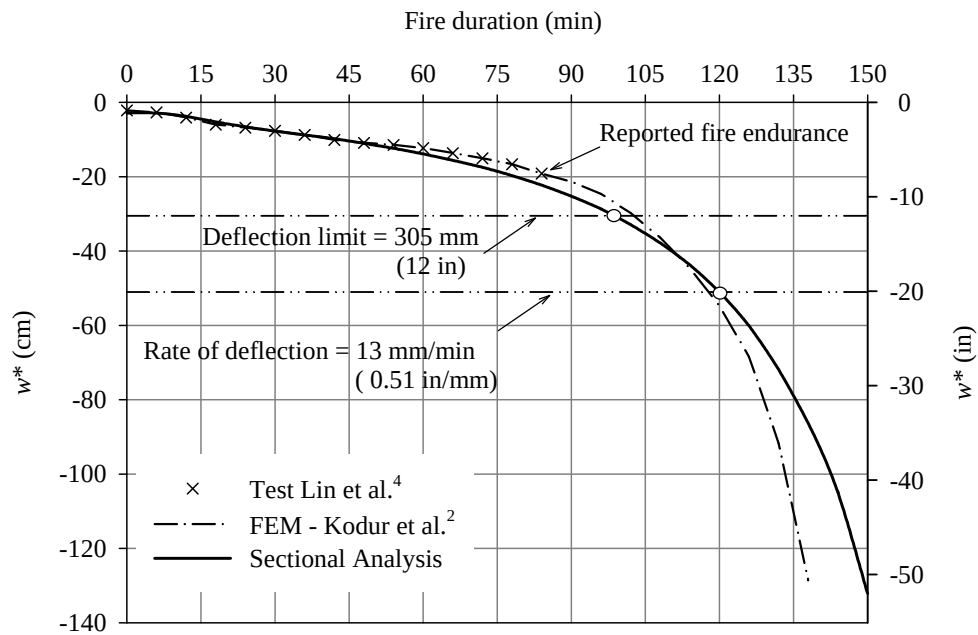


Fig. 8-Validation of sectional analysis methodology by prediction the deflections of B1 during fire exposure

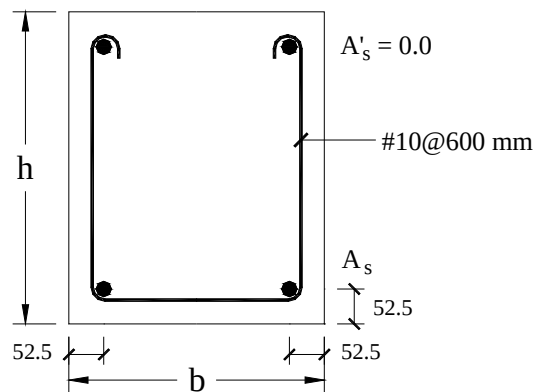


Fig. 9-Details of the studied RC beam “B4” in the parametric study
Dimensions in mm [1 in = 25.4 mm]

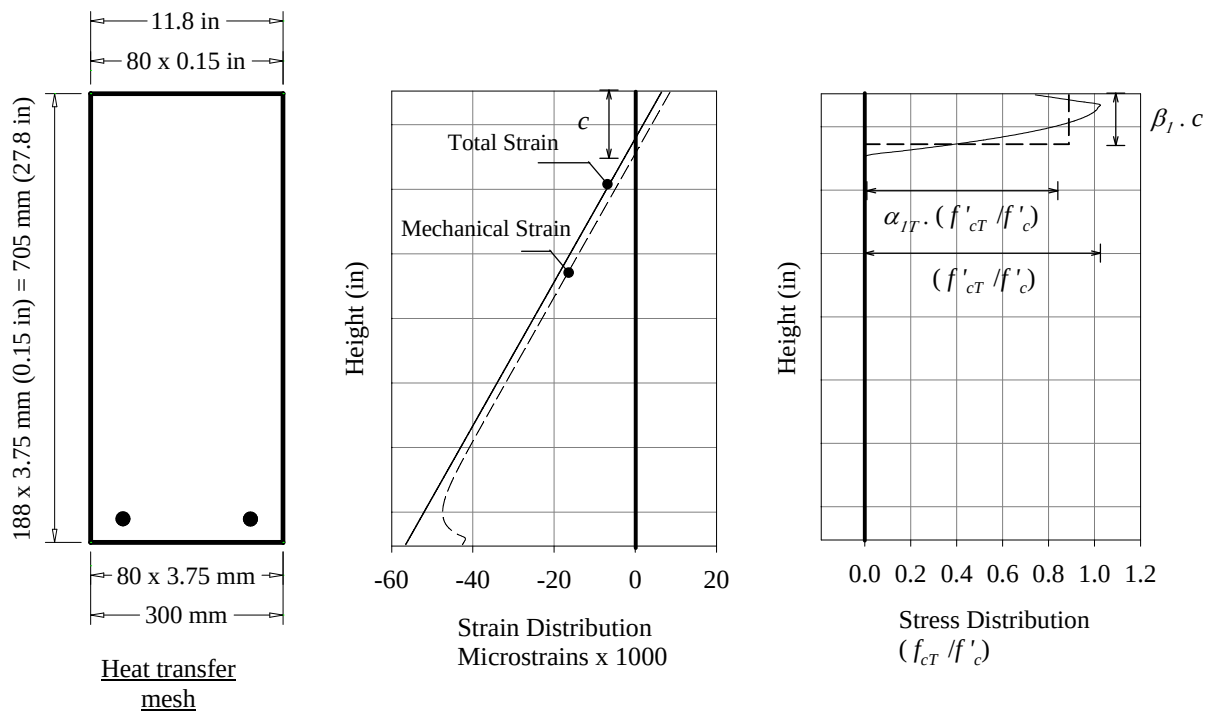


Fig. 10-Strain and stress distributions of B4 at 1.0 hr fire exposure

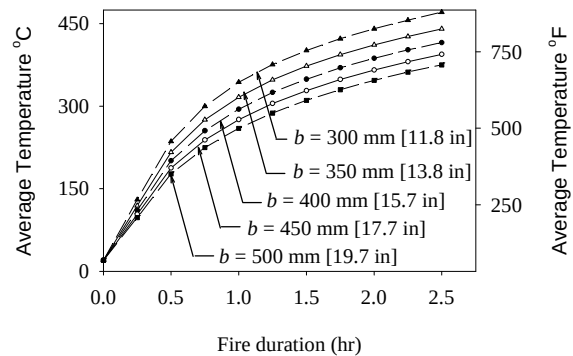
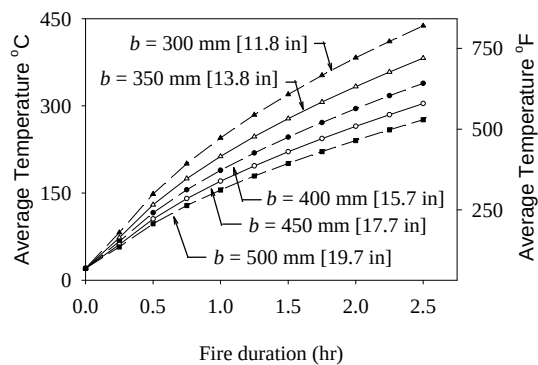
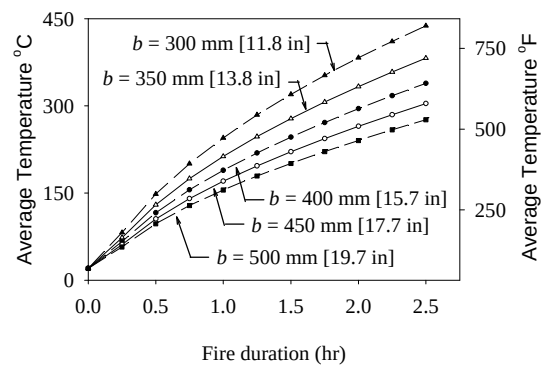


Fig. 11-Effect of ASTM-E119 fire duration and section width on the average strength temperature

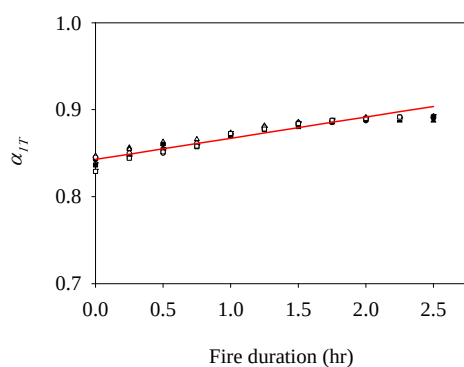


a) Siliceous aggregate

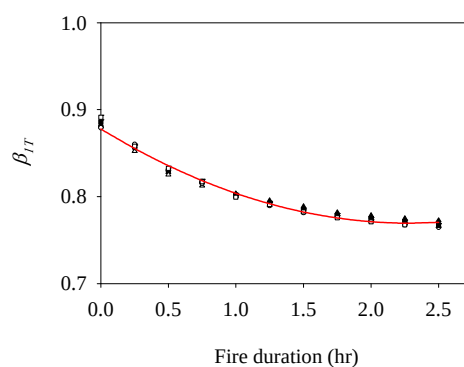


b) Carbonate aggregate

Fig. 12-Effect of ASTM-E119 fire duration and section width on the average temperature



a) Variation of α_{1T}



b) Variation of β_{1T}

B2 (300 x 503, r = 1%)
B3 (300 x 503, r = 2%)
B4 (300 x 705, r = 1%)
B5 (300 x 705, r = 2%)
B6 (300 x 908, r = 1%)
B7 (300 x 908, r = 2%)
Eq. (15)

Fig. 13-Effect of fire duration, RFT ratio, and section height on α_{1T} and β_{1T}

Dimensions in mm [1 in = 25.4 mm]

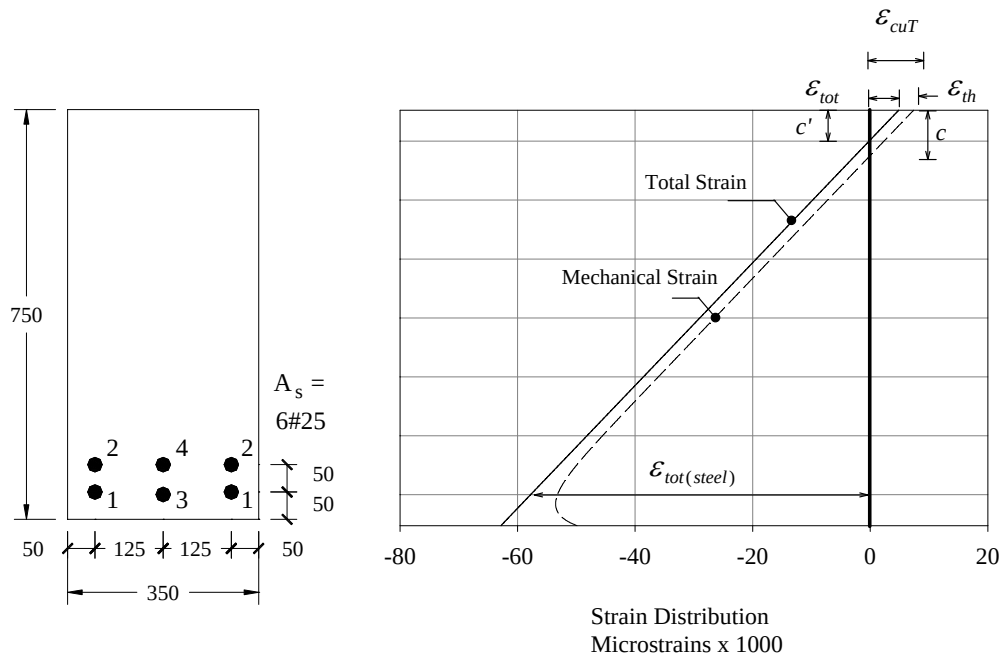


Fig. 14- Example beam (B30) for calculating the nominal failure moment M_{nT} using the proposed α_{1T} and β_{1T} - Dimensions in mm [1 in = 25.4 mm]

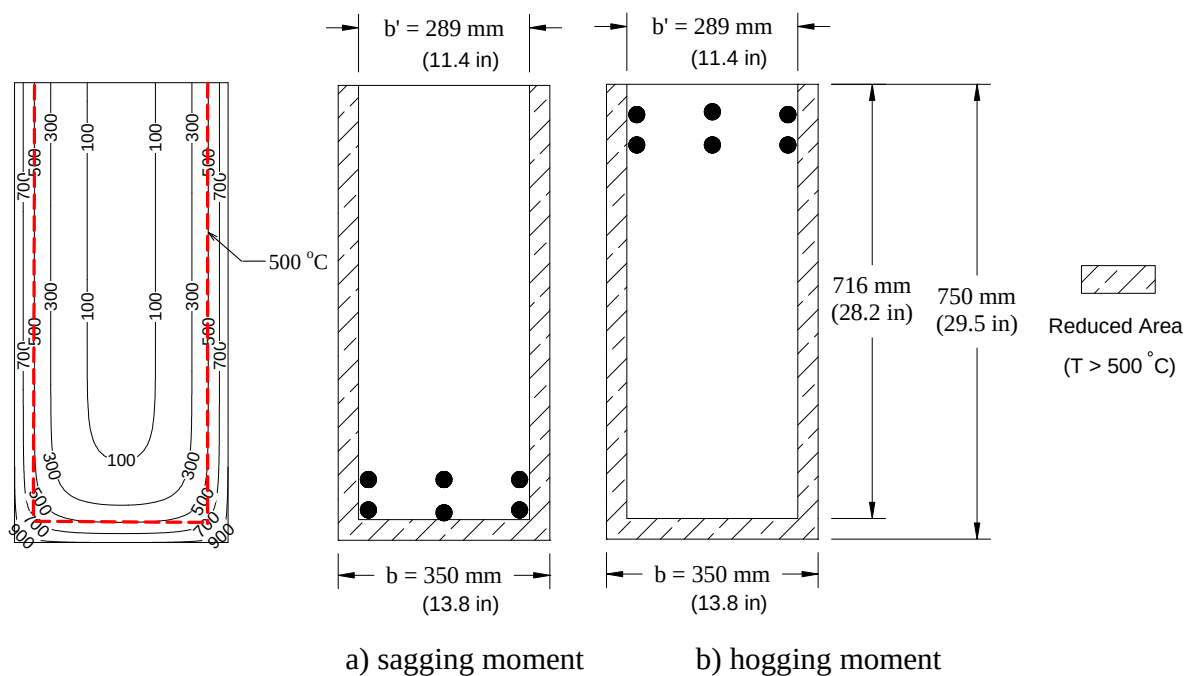


**Fig. 15- Temperature contour map in °C of B30
[1 °F = 1.8 °C + 32]**

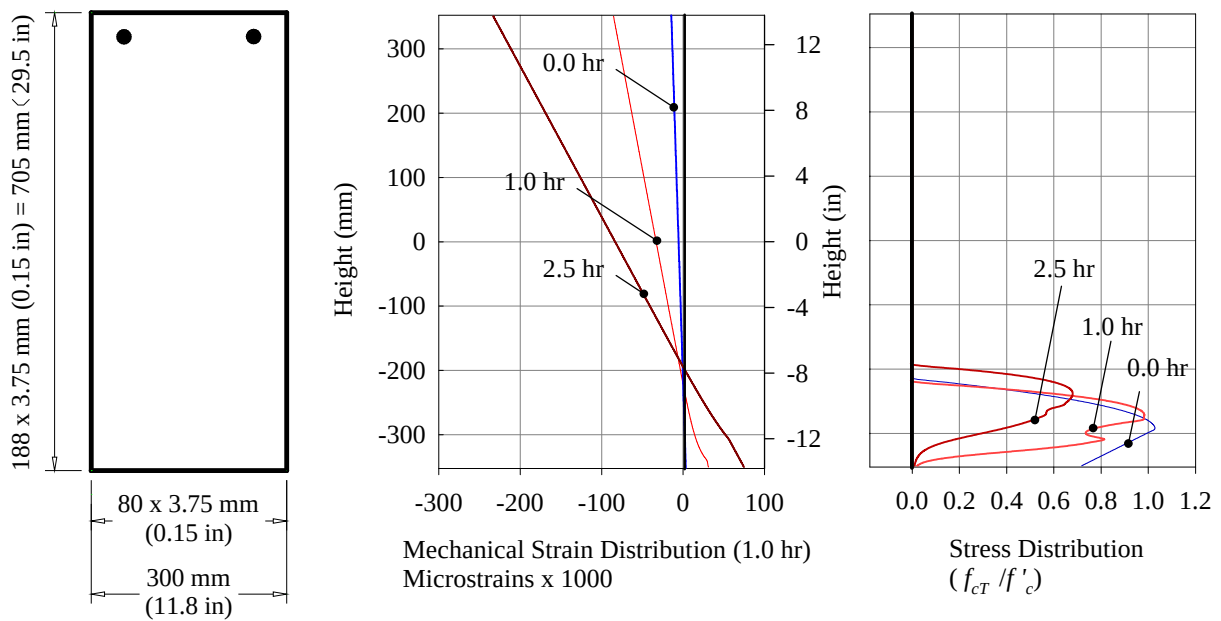


Fig. 16-Mechanical strain and stress distributions of B18 at different fire durations

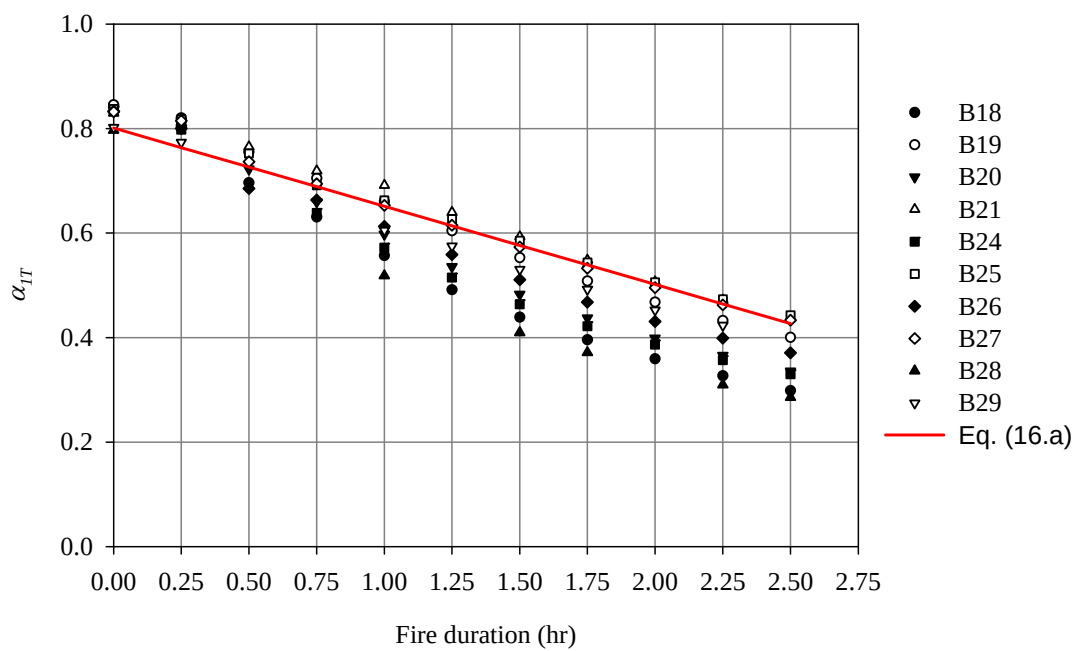


Fig. 17- Effect of fire duration and other studied factors on α_{1T}

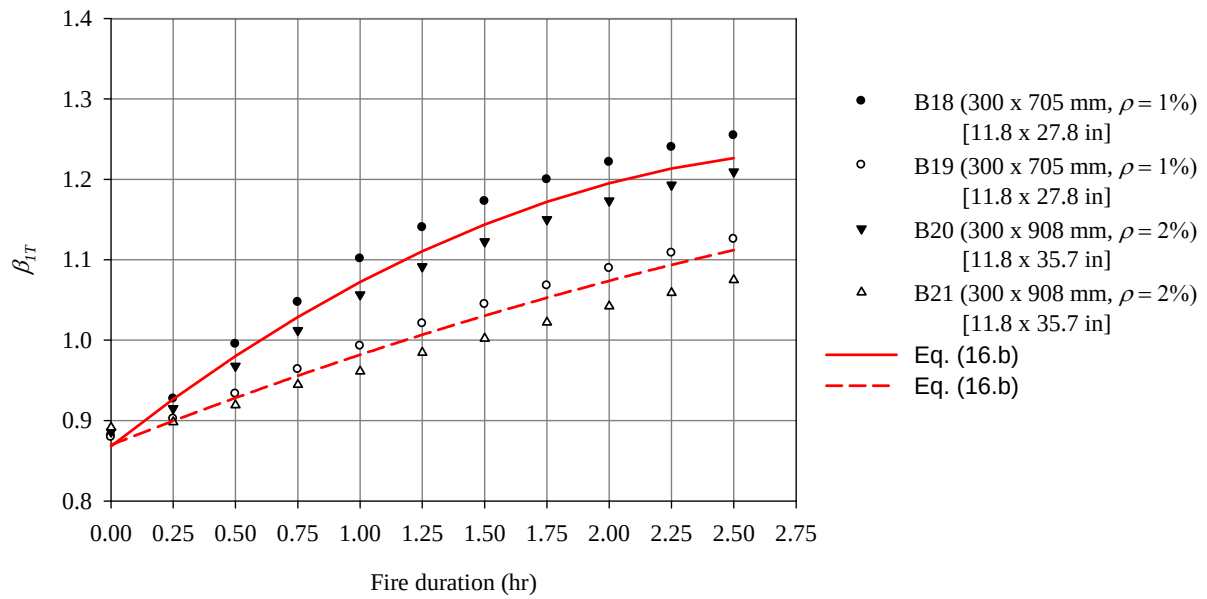


Fig. 18- Effect of fire duration, RFT ratio, and section height on β_{IT}

Appendix I

Simplified calculation of the M_{nT} for beams subjected to sagging moments

1. The reduced concrete compressive strength can be calculated using **Fig. 15** and **Eq. (2.b)**

for $b = 350$ mm [13.8 in] and $t = 1.50$ hr $\rightarrow T_{av. str} = 373$ °C [703 °F] [Fig. 15]

$$R = 1.0 \quad [\text{Eq. (2.b)}]$$

$$f'_{cT} = 1.25 \times (1.0 \times f'_c) \leq f'_c \quad [25\% \text{ increase because beam is loaded before fire}]$$

$$\therefore f'_{cT} = f'_c = 35 \text{ MPa} [5076 \text{ psi}]$$

$$f'_{cT} = K_{hT} \times f'_{cT} = 1.0 \times 35 \text{ MPa} \quad [\text{neglect confinement}]$$

2. The compression force in the concrete is calculated as follows

$$C = \alpha_{1T} \times f'_{cT} \times \beta_{1T} \times c \times b \quad [\text{Eq. (15.a) and (15.b)}]$$

$$\alpha_1 = 0.85 - 0.0015 \times 35 = 0.80$$

$$\therefore \alpha_{1T} = 0.80 - 1.533 \times 10^{-2} + 24.397 \times 10^{-3} \times 1.50 + 15.758 \times 10^{-4} \times 35 - 10.089 \times 10^{-5} \times 350 \\ = 0.84$$

$$\beta_1 = 0.97 - 0.0025 \times 35 = 0.88$$

$$\therefore \beta_{1T} = 0.88 - 2.907 \times 10^{-2} + 20.734 \times 10^{-3} \times 1.5^2 - 94.794 \times 10^{-3} \times 1.5 - 75.057 \times 10^{-5} \times 35 \\ + 15.413 \times 10^{-5} \times 350 = 0.78$$

$$C = 0.83 \times 35 \times 0.79 \times c \times 350 = 8,026.2 \times c$$

$$\text{assume } c = 105.9 \text{ mm [4.2 in]} \rightarrow \therefore C = 849991 \text{ N [191.1 kps]}$$

3. The tension force in the steel is calculated as follows

$$\varepsilon_{cuT} = \varepsilon_{cu} + \varepsilon_{tr}$$

$$\text{for } b = 350 \text{ mm [13.8 in] and } t = 1.50 \text{ hr} \rightarrow T_{av. temp} = 278 \text{ °C [532 °F]} \quad [\text{Fig. 16.a}]$$

$$\varepsilon_{tr} = 0.00593 \quad [\text{Eq. (5)}]$$

$$\varepsilon_{cuT} = 0.0035 + 0.00593 = 0.00943 \quad [\text{Eq. (8)}]$$

$$\varepsilon_{tot} = \varepsilon_{cuT} - \varepsilon_{th} = 0.00943 - 0.00167 = 0.00776 \quad [\text{Fig. (22)}]$$

$$c' = \frac{c}{\varepsilon_{cuT}} \times \varepsilon_{tot} = 0.823 \times c$$

$$\varepsilon_{tot(steel)} = \frac{\varepsilon_{tot}}{c'} \times d \quad [\text{Fig. (22)}]$$

$$T_1 = 558^\circ\text{C} \quad [1036^\circ\text{F}], \quad T_2 = 420^\circ\text{C} \quad [788^\circ\text{F}], \quad T_3 = 376^\circ\text{C} \quad [709^\circ\text{F}], \quad \text{and} \quad T_4 = 163^\circ\text{C} \quad [325^\circ\text{F}]$$

$$\varepsilon_{sT} = \varepsilon_{tot} - \varepsilon_{th} \quad [\text{Fig. (22)}]$$

The coefficient of thermal expansion of steel α_s is given by Lie et al. as follows

$$\alpha_s = (0.004T + 12) \times 10^{-6} \quad \text{C}^{-1} \quad T < 1000^\circ\text{C} \quad [1832^\circ\text{F}]$$

$$\alpha_s = 16 \times 10^{-6} \quad \text{C}^{-1} \quad T \geq 1000^\circ\text{C} \quad [1832^\circ\text{F}]$$

Thus,

$$\varepsilon_{sT1} = 0.04661, \quad \varepsilon_{sT2} = 0.04436, \quad \varepsilon_{sT3} = 0.04949, \quad \text{and} \quad \varepsilon_{sT4} = 0.04805$$

$$f_{sT1} = 219 \text{ MPa} \quad [31764 \text{ psi}], \quad f_{sT2} = 282 \text{ MPa} \quad [40901 \text{ psi}], \quad f_{sT3} = 303 \text{ MPa} \quad [43947 \text{ psi}], \quad \text{and}$$

$$f_{sT4} = 397 \text{ MPa} \quad [57581 \text{ psi}]$$

$$T_s = \sum_{i=1}^4 f_{sTi} \times A_{si} = 849991 \text{ N} \quad [191.1 \text{ kps}]$$

$$C = T_s \quad \text{O.K.}$$

$$M_{nT} = \sum_{i=1}^4 f_{sTi} \times A_{si} \times \left(d_i - \frac{\beta_{1T} \times c}{2} \right) = 536 \text{ kN.m} \quad [39.54 \times 10^4 \text{ ft.lb}]$$

Appendix II

Simplified calculation of the M_{nT} for beams subjected to hogging moments

1. The compression force in the concrete is calculated as follows

$$C = \alpha_{1T} \times f'_c \times \beta_{1T} \times c \times b$$

$$\alpha_1 = 0.85 - 0.0015 \times 35 = 0.80$$

$$\alpha_{1T} = \alpha_1 - 2.735 \times 10^{-2} - 1.497 \times 10^{-1} \times 1.50 + 7.579 \times 10^{-2} \times 0.0 = 0.55$$

$$\beta_1 = 0.97 - 0.0025 \times 35 = 0.88$$

$$\begin{aligned} \beta_{1T} = \beta_1 - 1.965 \times 10^{-1} - 4.054 \times 10^{-2} \left(\frac{1.5}{1.5} \right)^2 + 2.448 \times 10^{-1} \left(\frac{1.5}{1.5} \right) - 3.456 \times 10^{-2} \times 0.0 \\ + 3.687 \times 10^{-3} \times 35 + 2.342 \times 10^{-4} \times 350 = 1.10 \end{aligned}$$

assume $c = 132.9$ mm [5.2 in]

$$\therefore C = 984734 \text{ N [221.4 kps]}$$

2. The tension force in the steel is calculated as follows

$$T_1 = 366 \text{ }^\circ\text{C [691 }^\circ\text{F]}, T_2 = 366 \text{ }^\circ\text{C [691 }^\circ\text{F]}, T_3 = 58 \text{ }^\circ\text{C [136 }^\circ\text{F]}, \text{ and } T_4 = 58 \text{ }^\circ\text{C [136 }^\circ\text{F]}$$

$$f_{yT1} = 296 \text{ MPa [42932 psi]}, f_{yT2} = 296 \text{ MPa [42932 psi]}, f_{yT3} = 393 \text{ MPa [57001 psi]}, \text{ and}$$

$$f_{yT4} = 393 \text{ MPa [57001 psi]}$$

$$T_s = \sum_{i=1}^4 f_{sTi} \times A_{si} = 984734 \text{ N [221.4 kps]}$$

$$C = T_s \quad \text{O.K.}$$

$$M_{nT} = \sum_{i=1}^4 f_{sTi} \times A_{si} \times \left(d_i - \frac{\beta_{1T} \times c}{2} \right) = 593 \text{ kN.m} \quad [43.75 \times 10^4 \text{ ft.lb}] \quad (\text{Diff. } \approx 5\%)$$

BOND STRENGTH OF CFRP AND STEEL BARS IN CONCRETE AT ELEVATED TEMPERATURE

Cristián Maluk^{a, b}, Luke Bisby^b, Giovanni Terrasi^c, Mark Green^d

^a Pontificia Universidad Católica de Chile, Escuela de Ingeniería, Departamento de Ingeniería Estructural y Geotecnia

^b BRE Centre for Fire Safety Engineering, University of Edinburgh, UK

^c EMPA Dübendorf, Zurich, Switzerland

^d Civil Engineering, Queen's University, Kingston, Canada

Abstract: Novel concrete elements are emerging utilizing high performance self-consolidating concrete (HPSCC) reinforced with high-strength, lightweight, and non-corroding carbon fiber reinforced polymer (CFRP) prestressed reinforcement. The fire performance of these elements must be understood before they can be used with confidence. In particular, the bond performance of the novel CFRP reinforcement at elevated temperatures requires investigation. This paper examines the bond performance of a specific type of CFRP tendon as compared with steel prestressing wire. The results of transient elevated temperature bond pullout and tensile strength tests on CFRP tendons and steel prestressing wire are presented and discussed, and show that bond failure at elevated temperature is a complex phenomenon which is influenced by a number of interrelated factors, including the type of prestressing, degradation of the concrete, CFRP, and steel, differential thermal expansion, thermal gradients and stresses, release of moisture from the concrete, and loading. It is shown that CFRP tendons are more sensitive to bond strength reductions than to reductions in tensile strength at elevated temperature.

Keywords: Bond strength, composite materials, DMA, elevated temperatures, carbon FRP, prestressing reinforcement, pullout tests, steel wire, TGA.

Cristián Maluk is a PhD candidate in the BRE Centre for Fire Safety Engineering at The University of Edinburgh. He obtained his BSc and MSc in Civil Engineering, with concentrations in Structural Engineering, in 2008 and 2010 respectively, from Pontificia Universidad Católica de Chile, Chile.

Dr. Luke Bisby is a Reader and the Ove Arup Foundation/Royal Academy of Engineering Senior Research Fellow in Structures in Fire within the BRE Centre for Fire Safety Engineering at the University of Edinburgh. He obtained his BEng in Civil Engineering and Applied Mechanics from McGill University, Canada, in 1997, and his MSc and PhD in Structural Engineering, in 1999 and 2003, respectively, from Queen's University, Canada. He is an active member of ACI committees 216 and 440.

Dr. Giovanni Terrasi received his PhD (Civil Engineering) from ETH Zürich in 1997. Between 1998 and 2005 he led the R&D/Engineering Department at SACAC Ltd, Switzerland with a particular interest in high performance concrete reinforced or prestressed with fiber reinforced polymer tendons. Since 2005 he is Head of the Laboratory for Mechanical Systems Engineering at the Swiss Federal Laboratories for Materials Science and Technology. His research focuses on the durability of mechanical components and polymer composites for infrastructure applications.

Dr. Mark Green is a Professor of Civil Engineering at Queen's University, Canada. He obtained his BSc in Applied Mathematics and Structural Engineering from Queen's University in 1987, and his PhD in Structural Engineering from Cambridge University, UK, in 1991. He is an active member of ACI Committees 216 and 440.

INTRODUCTION

The use of fiber reinforced polymers (FRPs) in civil engineering applications has been developing for more than 40 years. FRPs are now widely used as externally bonded FRP sheets and plates to increase

the shear, flexural and/or axial strength of deficient reinforced concrete elements. Another promising and reasonably widely implemented application of FRPs is their use as partial or total replacement of steel reinforcement in new concrete structures.

The use of internal FRP reinforcement for prestressing of concrete structures is motivated predominantly by a desire to prevent electrochemical corrosion of the reinforcement and thus to build more durable structures. Additionally, however, a desire to build more sustainable and durable structures has made careful selection, design, and optimization of both the concrete mixtures and reinforcing materials commonplace. These considerations have promoted the emergence of elements which incorporate high performance self-consolidating concrete (HPSCC) with novel prestressing materials, such as carbon fiber reinforced polymer (CFRP) tendons which are high-strength, creep resistant, lightweight, non-corroding, and magnetically invisible. One example of this is a novel type of HPSCC precast panel with pretensioned carbon fiber reinforced polymer (CFRP) tendons; this is now being implemented in load-bearing panels for building envelopes [1]. Several field projects have been realized in recent years making use of thin-walled structural elements made of high performance concrete prestressed with CFRP tendons [1]. One notable example of this is the use of precast, prestressed 8.8 m (28.87 ft) long beams with L-shaped cross-sections, only 60 mm (2.36 in.) thick in an innovative project in Switzerland [2] (Figure 1).

Designing precast pretensioned concrete elements by combining the advantages of HPSCC and CFRP bars makes it possible to manufacture thin-walled, lightweight, fatigue resistant, and durable HPSCC precast elements, despite the relatively high initial costs of FRP prestressing bars in comparison to prestressing steel.

The performance of these HPSCC precast concrete elements in fire is not, however, well known, and a better understanding of their response to fire must be developed before they can be used with confidence in the wide variety of structural applications for which they might be considered. Numerous

studies and past experience have shown that conventional steel reinforced or prestressed concrete structural elements generally exhibit good performance in fires. This is due primarily to the inherent and beneficial self-insulating characteristics of concrete [3], which protects the internal steel reinforcement from the heat of the fire. For reinforced or prestressed concrete elements, adequate structural fire resistance is typically ensured by prescribing a minimum allowable concrete cover depth to the steel reinforcement/prestressing. In the case of CFRP prestressed HPSCC elements such as those described above, two additional issues become critical and are the focus of the current paper:

1. Because there are essentially no concerns regarding corrosion of the CFRP reinforcement, the concrete cover can be considerably less in CFRP reinforced or prestressed elements than in steel prestressed elements; this allows thin-walled elements without increased risk of corrosion.
2. Reductions of the bond strength between steel reinforcement and concrete are not generally considered to be a governing factor dictating the fire resistance of steel reinforced or prestressed concrete elements. It is widely thought, however, that that degradation of the bond between FRP bars/tendons and concrete at elevated temperature is considerably more critical than loss of the reinforcement's tensile strength [4]. FRP reinforcements' bond strength reductions are widely thought to be the limiting factor for the fire-safety of FRP reinforced/prestressed concrete [4], although the magnitudes of bond strength reductions and their impacts on the load-bearing capacity of heated FRP reinforced/prestressed concrete structures remain largely unknown.

RESEARCH SIGNIFICANCE

The bond between both FRP and steel reinforcing bars (prestressed and non-prestressed) and concrete deteriorates at elevated temperature [5], [6]. The ultimate tensile strength of both steel and FRP bars/tendons is also reduced by exposure to elevated temperatures; these reductions are now well

known for steel reinforcements, as defined by Eurocode 2 [7] for example, but remain largely unknown for FRP reinforcements [8].

National design codes [9], [10], [11], [12] focusing on the design of FRP reinforced concrete structures all assume perfect bond between the FRP reinforcement and the concrete for ambient temperature analysis and design. The bond strength capacity of FRP reinforcing or prestressing bars relies primarily on the strength and stiffness of the polymer resin matrix at the bars' surface (which typically incorporates a sand coating, spiral fibre roving, and/or a ribbed shaped resin). The polymer resin at the surface of the bars softens at temperatures of less than 150 °C (302 °F) for most available FRP reinforcing bars. The glass transition temperature (T_g) of the polymer resin is widely used to define the limiting temperature at which degradation of the tensile strength of composites materials occurs [10], [13], yet the research to support such an approach is relatively scarce. Furthermore, despite the known importance of bond strength reductions for FRP bars in concrete, relatively little attention has been given to defining how the degradation of the polymer resin matrix affects the bond strength capacity at elevated temperatures or in fire; this is the focus of the current paper.

EXPERIMENTAL INVESTIGATION

An experimental program was undertaken, consisting of transient elevated temperature bond pullout and tensile strength tests on CFRP prestressing tendons and deformed steel prestressing wires of comparable strength and stiffness. The following sections provide details of the specific materials and test methods used.

Materials

CFRP prestressing tendons — Round, pultruded, sand-coated CFRP prestressing tendons supplied by SACAC Ltd¹, Switzerland, were used in this study. The carbon fiber volume fraction was 62% and the bars had an epoxy polymer resin matrix. The CFRP bars had a 5.4 mm (0.213 in.) nominal diameter and were fabricated by the pultrusion process [9]. To promote a better bond with concrete, a sand coating was applied to the surface of the bars by broad-casting silica sand into a coating of epoxy resin applied after the primary pultrusion and curing process. The sand coating was approximately 0.3 mm (0.012 in.) thick. The CFRP bar's manufacturer specified nominal tensile strength was 2000 MPa (290 ksi), with an elastic modulus of 150 GPa (21,756 ksi). The tensile stress-strain relationship for the CFRP is linear-elastic to failure, with an ultimate strain of 1.33%. The bars have a nominal mass of 0.123 kg/m (0.0827 lb/ft), which is about half the value of the comparative steel prestressing wire used in the current project. A sample of the CFRP prestressing tendon is shown in Figure 2(a).

Steel prestressing wire — For the purposes of comparison, deformed steel prestressing wire produced specifically for prestressing applications by NEDRI Spanstaal BV was also used in this study. The steel reinforcement was 6 mm (0.236 in.) in nominal diameter with a nominal mass of 0.221 kg/m (0.149 lb/ft). Ribs were applied on the surface of the wire during the cold working manufacturing process. The manufacturer's specified design yield strength (0.2% offset) was 1592 MPa (231ksi) and the ultimate strength was 1770 MPa (257 ksi), with an elastic modulus of 210 GPa (30,458 ksi). The yield strain was 0.76% and the ultimate strain was 5.4%. A sample of the steel wire is shown in Figure 2(b).

Concrete (HPSCC) — The high quality and comparatively high cost of CFRP reinforcements requires a correspondingly high quality concrete mixture to justify use of CFRP prestressing tendons. A high performance, self-consolidating, polypropylene fiber reinforced concrete (HPSCC) with a 28-day

¹ Please note that the specific CFRP and steel prestressing tendon suppliers are noted solely for the purposes of factual accuracy.

minimum cylinder strength of 75 MPa (10.9 ksi) was used in this study. The details of the concrete mixture are proprietary, but it included normal Portland cement, silica fume, fly ash, polypropylene microfibers, super-plasticizer, and a precise grain size distribution of selected 0-8 mm limestone aggregates.

Reinforcements' bond mechanisms

In reinforced concrete elements, the longitudinal tensile forces transferred from the reinforcement to the surrounding concrete are influenced by the reinforcement's surface characteristics and shape, the concrete's tensile and compressive strength, concrete cover, clear spacing between reinforcing bars, casting orientation, load-time history of the structure, and the reinforcement's elastic modulus, Poisson's ratio, and diameter [14].

When deformed steel reinforcement is used, the ribbed surface of the bar interacts with concrete by chemical adhesion, frictional forces, and mechanical anchorage (given by bearing of the ribs against the concrete surrounding the reinforcement). Elevated temperature bond strength reductions for deformed steel reinforcements are most likely to be caused by loss of chemical adhesion and reductions in concrete strength at elevated temperature.

When CFRP reinforcement with a sand-coated epoxy resin surface is used, bond to concrete is also given by chemical adhesion, frictional forces, and a diminished mechanical anchorage, relative to the one in deformed steel, given by the sand coating's silica particles which are chemically and mechanically anchored in the reinforcement's surface resin layer. It is the loss of strength and stiffness of the resin at the surface of the reinforcement at elevated temperature which appears to be critical. For commercially available FRP reinforcing bars, several supplementary methods have been developed to improve bond with concrete under ambient conditions – for example deformation at the bar's surface by wrapping additional resin-saturated fibers in a spiral around the bar. This paper considers only sand-coated CFRP prestressing tendons without additional surface deformations.

It should be noted that, both at ambient temperature and under fire conditions, the bond between steel reinforcements (prestressed and non-prestressed) and concrete is not fully understood and remains a topic of research interest. Nonetheless, the bond between steel and concrete is typically considered to be perfect (even under fire conditions), even if this is clearly not the case in real structures during fire [15].

Pullout test methods and procedures

To study the load transfer and bond between concrete and internal reinforcement, previous researchers [5], [16] have used strain gauges placed at the concrete-reinforcement interface in large or mid-scale reinforced concrete elements, along with other techniques employed to measure stress and relative displacement on both the reinforcement and the concrete. These approaches, while useful for understanding load transfer in real structures, are time-consuming and costly. Bond pullout tests present an economical and simple testing option for comparative evaluation of bond performance of different types of reinforcing bars within concrete, despite the fact that the stress conditions encountered in reinforced concrete elements differ considerably from those encountered in pullout tests [17], [18]. For instance, pullout tests have been used by previous researchers to compare the bond performance of steel and FRP reinforcing bars during (or after) exposure to elevated temperatures [6], [19].

Abbasi and Hogg [19] studied the bond strength of glass FRP reinforcing bars in concrete at temperatures in the range of 25-120 °C (77-248 °F) after long-term exposure to an alkaline environment. Their tests consisted of steady state heat-then-load bond pullout tests on glass FRP bars embedded over a length of 60 mm (2.36 in.) within 100 mm (3.96 in.) cubes of concrete. The specimens were heated in an oven until they reached a pre-determined temperature between 20 °C (68 °F) and 120 °C (248 °F) and were then loaded to failure. Abbasi and Hogg [19] concluded that degradation of bond strength with temperature obeys a similar relationship irrespective of prior

conditioning of the specimens, and that the nature of the polymer resin matrix at the surface of the FRP bars determines the magnitude and rate of degradation bond strength. In their study, average bond pullout strengths at 120 °C (248 °F) were about 57% of ambient temperature values. These data are not particularly helpful, however, since specific information on the deterioration in mechanical properties of the resin with elevated temperature is not given. It should also be noted that steady state (heat-then-load) tests are less representative of the in-service conditions for FRP bars in concrete than transient (load-then-heat) tests, particularly for prestressed FRP reinforcement which is under considerable sustained stress while a structure heats during a fire.

Katz et al. [6] tested five different types of FRP bars using transient (load-then-heat) elevated temperature bond pullout tests under sustained load. These were executed by loading specimens up to a predefined bond stress level and then heating the specimen under constant applied load until failure occurred. The test specimens consisted of FRP bars embedded in 150 mm (5.91 in.) diameter concrete cylinders over a length of 60 mm (2.36 in.). Heating was applied by a heating element wrapped around the cylinders' circumference. Katz et al. [6] observed a severe reduction in the bond strength (in some cases as high as 75%) as the temperature was raised above the glass transition temperature of the bars' polymer resin matrices (which ranged between 60 °C (140 °F) and 124 °C (255 °F)). They also noted that a thick polymeric layer at the surface of the FRP bar negatively affected the bond both at ambient temperature and particularly at elevated temperature. It should be noted that during their initial tests, Katz et al. [6] encountered "premature" failures of the bond, which were attributed to "flux of vapor outward at high pressure which stimulated the slippage of the bar." They artificially prevented these failures in subsequent tests by pre-drying the specimens in ovens at 100 °C (212 °F). This approach is non-representative of the thermal conditions that would be encountered in a real fire, and Katz et al.'s results should be viewed with this in mind given that it appears that moisture plays an important role in influencing bond behaviour at elevated temperature.

The research performed for the current paper was performed in a similar manner as Katz et al.'s testing [6], with the notable exception that the specimens were not artificially pre-dried prior to testing. Testing was performed by loading the specimens (as described below) up to a predefined bond stress and then heating them until failure occurred under sustained load. Heating was accomplished using a silicone rubber heating pad wrapped around the specimens' circumference (Figure 3). A total of 16 specimens were tested, eight with embedded CFRP tendons and eight with embedded steel prestressing wires.

The pullout specimens in the current study, which are shown schematically in Figure 3, consisted of concrete cylinders with diameters of 100 mm (3.96 in.) and lengths of 250 mm (9.84 in.). Each cylinder had a single embedded prestressing bar (CFRP or steel). The casting moulds were designed in such way that the reinforcement was placed vertically and axisymmetrically. Bond breaker was placed over the top (50 mm (1.97 in.)) and bottom (40 mm (1.57 in.)) of the reinforcing bars to allow for a bonded length of 160 mm (6.30 in.). The bond breaker also prevented the development of localized artificial confinement of the bonded length due to compressive load on the concrete at the loaded end, and promoted axisymmetric heat transfer along the bonded length to assure as uniform a bond line temperature as possible. The bonded length was chosen as 160 mm (6.30 in.) because this was determined to be the prestress transfer length for the CFRP tendons at ambient temperature as measured through prior research [20]. Special consideration was given to the procedure used for accurately measuring of the slip of the reinforcement at both the loaded and free ends of the cylinders – a novel digital image correlation analysis technique was used (Figure 3) [21], [22]. The image correlation technique was verified using conventional linear potentiometers (LPs), which were also used to measure slip at both ends. The precision of this measurement technique has been proven to be better than $1/10^{\text{th}}$ of one pixel [23], and the linear pixel density achieved within the digital images collected during the tests was 35 pixels per mm (889 pixels per inch).