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Fire performance of reinforced concrete frames using sectional analysis

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ABSTRACT

Global behavior of RC structures during fire events can be predicted using complex nonlinear thermal-structural numerical simulations. However, such simulations are computationally expensive, which limit their use by design engineers. A practical approach to track the performance of RC frames during fire exposure is proposed and validated in this paper. A previously developed simple heat transfer technique is used to calculate an average 1D temperature distribution for heated RC sections. Consequently, the flexural and axial stiffnesses as well as the unrestrained thermal deformations are evaluated using sectional analysis. Based on rational assumptions, simplified expressions are also driven to evaluate those values. The proposed method can be easily applied using available commercial linear structural analysis software to predict the fire performance of RC framed structures. Additional experimental and analytical work is required to validate the proposed method in non-standard fire scenarios.

1. Introduction

Fire initiates when combustible materials ignite. Then, it spreads horizontally and vertically depending on the compartment boundaries [1]. A temperature gradient is generated through exposed RC elements. These elevated temperatures cause the element's stiffness to degrade and produce thermal deformations [2]. Structural fire safety of RC structures is currently evaluated based on the fire ratings of single elements, i.e. columns, beams, walls, and slabs [3]. However, the overall behavior of the structure during a fire should be assessed to ensure the safety of the occupants and the fire fighters during evacuation.

Fire testing is the most reliable approach to assess the fire endurance of a structure but its use for concrete frames is very limited [3] because of its cost. Finite Element (FE) tools are very powerful and capable of analyzing RC structures during fire events [5]. Drawbacks of using the FE method including: the need for a comprehensive computer program, the difficulty to comprehend its results and to identify potential modeling errors, and the long running time make it impractical for design engineers. To the best of the author's knowledge, simplified methods to analyze RC frames during fire exposure do not exist [6–8].

This paper provides engineers with a practical approach to predict the fire response of statically determinate and indeterminate RC frames. The proposed approach extends the work done by El-Fitiany and Youssef [9,10] that proposed converting the two-dimensional (2D) temperature distribution to an average one-dimensional (1D) temperature distribution to predict the flexural behavior of heated sections at different axial load levels (λ) [11]. This paper provides the derivation of closed form formulations for concrete stiffness, flexural and axial, and the steps needed to apply these formulations to analyze RC frames during fire exposure. The proposed method eliminates the need to divide a fire exposed section into finite elements or layers to conduct heat transfer and nonlinear stress analyses. The proposed method is validated for standard fires by comparing its predictions with experimental and analytical results by others.

Fig. 1a shows a 300 mm square concrete cross-section exposed to fire from three sides. Fig. 1a shows the elevated temperature contours within the heated cross-section after 1 h of ASTM-E119 fire exposure. The low thermal conductivity of concrete results in a steep temperature distribution near the heated faces and almost a constant temperature at the core of the heated section [10]. Fig. 1b shows the average temperature (T_{avg}) across the section width (b). As shown in Fig. 1b, T_{avg} is variable within distance z from the bottom face and constant within $h - z$ distance, where h is the cross-section height. Thus, the concrete mechanical properties become variable, i.e. non-linear, near the bottom heated face

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Nomenclature

A_1	factor used in calculating internal concrete force, equals to $e^{y_1 z_2}$
A_2	factor used in calculating internal concrete force, equals to $e^{y_2 z_2}$
b	column width in x direction
C_c	internal compression force in concrete
$C_{co(v)}$	concrete compression force at $\varepsilon_{cT} \leqslant (\varepsilon_{oT} + \varepsilon_{tr})$ for variable T_{avg} distribution
$C_{co(v)} \cdot y$	concrete moment about x axis at $\varepsilon_{cT} \leqslant (\varepsilon_{oT} + \varepsilon_{tr})$ for variable T_{avg} distribution
$C_{cu(v)}$	concrete compression forces at $\varepsilon_{cT} > (\varepsilon_{oT} + \varepsilon_{tr})$ for variable T_{avg} distribution
$C_{cu(v)} \cdot y$	concrete moment about x axis at $\varepsilon_{cT} > (\varepsilon_{oT} + \varepsilon_{tr})$ for variable T_{avg} distribution
$C_{co(c)}$	concrete compression force corresponding to $\varepsilon_{cT} \leqslant (\varepsilon_{oT} + \varepsilon_{tr})$ for constant T_{avg}
$C_{co(c)} \cdot y$	concrete moment about x axis at $\varepsilon_{cT} \leqslant (\varepsilon_{oT} + \varepsilon_{tr})$ for constant T_{avg}
$C_{cu(c)}$	concrete compression force corresponding to $\varepsilon_{cT} > (\varepsilon_{oT} + \varepsilon_{tr})$ for constant T_{avg}
$C_{cu(c)} \cdot y$	concrete moment about x axis at $\varepsilon_{cT} > (\varepsilon_{oT} + \varepsilon_{tr})$ for constant T_{avg}
f'_c	compressive strength for concrete at ambient temperature
f_y	yield strength of steel bars at ambient temperature
f'_{cT}	reduced compressive strength at elevated temperatures
f_{cT}	compression stress in heated concrete
f_{yT}	reduced yield strength of reinforcing bars at elevated temperatures
f_{sT}	compression or tension stress in heated steel bars
$(f_{cT})_{avg}$	average concrete compressive stresses
h	cross-section height
M_{app}	flexural moment
n_w	ratio between the surface temperature and the fire temperature
n_x and n_y	ratios between the internal and surface temperatures due to heating in the x and y directions, respectively
P_{app}	axial load
t	fire duration
t^*	equivalent fire duration assuming ISO 834 standard fire
T	temperature in degree Celsius [1 °F = 1.8 °C + 32]
T_{xy}	temperature rise at any point located at (x, y)
T_{avg}	algebraic average distribution along the section height
T_{avg1}	average temperature for regions affected by heating from either left or right
T_{avg2}	average temperature for regions not affected by heating from left or right
T_{avg3}	average temperature due to heating from the left and right sides simultaneously
T_f	fire temperature
$T_{f(\mathrm{ISO})}$	ISO 834 standard fire temperature at a modified fire duration t^*
x, y	horizontal and vertical coordinates for any point within the column/beam section, origin located at bottom left of the section
y_1, y_2	boundaries of internal concrete compression force measured in y direction
z	boundary of fire affected regions
z_1, z_2	constants of average temperature fitting equation, Eq. (1)
z_3, z_4	constants defining the linear variation of ε_{cT} in y direction, Eq. (14)
$\overline{z_3}, \overline{z_4}$	constants defining the linear variation of $\overline{\varepsilon_{th}}$ in y direction, Eq. (11)
ε	total concrete strain at elevated temperatures
ε_{th}	unrestrained thermal strain of concrete
ε_{tr}	transient creep strain in concrete
ε_c	instantaneous stress-related strain
ε_{cT}	equivalent mechanical strain in concrete during fire exposure
$\overline{\varepsilon_{th}}$	equivalent linear thermal strain
ε_i	unrestrained center thermal axial strain
ε_{st}	self induced thermal strains
ε_{sT}	equivalent mechanical strain in steel during fire exposure
ε_{oT}	value of ε_c at peak stress f'_{cT}
ε_{uT}	ultimate compressive strain of concrete
$\Delta\varepsilon$	difference between ε_{uT} and $(\varepsilon_{oT} + \varepsilon_{tr})$ equals to 0.02
ψ_i	unrestrained thermal curvature
λ	axial or flexural load level
Γ	compartment time factor

and constant at the inner concrete core. This concept is the basis of the proposed method to predict the fire performance of concrete frames during fire exposure.

2. Proposed method

El-Fitiany and Youssef [7,10] presented a sectional analysis technique to predict the behavior of heated RC cross-sections while accounting for temperature gradient, strain nonlinearity, and material degradation. Fig. 2a and b shows the T_{avg} distribution for a RC section subjected to fire from three faces. The free thermal strain (ε_{th}) can be predicted using T_{avg}, Fig. 2c. To retain the section linearity, a self-induced strain (ε_{st}) is generated to convert the nonlinear strain to an equivalent linear strain ($\overline{\varepsilon_{th}}$) as shown in Fig. 2d and e. $\overline{\varepsilon_{th}}$ is defined by the value of the center axial strain (ε_i) and the curvature (ψ_i). The values of ε_i and ψ_i represent the unrestrained thermal deformation, i.e. elongation and curvature, of the heated section. Alternatively, $\overline{\varepsilon_{th}}$ can be defined using the factors $\overline{z_3}$ and $\overline{z_4}$ for the mathematical expressions presented in this paper. ε_{st} is considered as a residual strain and is added to the instantaneous stress-related strain (ε_c). The transient creep strain (ε_{tr}) is implicitly included in the concrete stress-strain relationship as explained later in this paper. As shown in Fig. 3, the total strain (ε) is then calculated by adding ($\varepsilon_c + \varepsilon_{tr} + \varepsilon_{st}$) to the equivalent linear strain ($\overline{\varepsilon_{th}}$). The effective flexural stiffness (EI_{eff}) and axial stiffness (EA_{eff}) can be calculated using the applied axial force (P_{app}) and flexural moment (M_{app}) and the corresponding total strain (ε).

The results of the sectional analysis are used to predict the global behavior of RC frames during fire exposure as explained later in this paper. Fig. 4 summarizes the main steps of the proposed method. For a given fire duration, the structural performance of RC frames can be predicted by:

(1) determining an equivalent one-dimensional average temperature distribution for the sections of the fire exposed beams and columns,
(2) identifying the needed constitutive thermal and mechanical models for the heated elements,
(3) predicting the self-induced strain and unrestrained thermal deformations for the heated elements,
(4) evaluating the effective, i.e. secant, flexural and axial stiffnesses of the heated elements based on the applied axial forces (P_{app}) and flexural moments (M_{app}),