A MATHEMATICAL MODEL PREDICTING THE BEHAVIOR OF HIGH STRENGTH CONCRETE COLUMNS SUBJECTED TO FIRE

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الملخص

يهدف هذا البحث إلى دراسة التأثير الذي يحدثه الارتفاع في درجة الحرارة نتيجة لوجود حريق على عمود خرساني مسلح في مبنى، حيث تم إيجاد توزيع درجات الحرارة خلال العمودِ الخرسِاني وذلك بحل المعادلة العامة للتوصيل الحراري ثلاثي الأبعاد الغير مستقر مع الزمن حلا عدديا ومن معرفة توزيع درجات الحرارة تم وضع نموذج رياضي مبسط يمكن بواسطته معرفة تأثير ارتفاع درجات الحرارة خلال العمود على إجهاد الضغط. تم حل المعادلة الحاكمة للتوصيل الحراري ثلاثية الأبعاد بواسطة التحليل العددي (طريقة رنج- كوتا) من الرتبة الخامسة. وتم مقارنة نتائج توقعات النموذج الرياضي لتوزيع درجات الحرارة مع نتائج دراسة عملية وقد بينت نتيجة المقارنة لتوزيع درجات الحرارة المحسوبة من النموذج الرياضي توافق جيد مع القراءات العملية. وقد أظهرت هذا الدراسة أن لزيادة درجات الحرارةِ الناتجة عن الحريق تأثير سلبي على إجهاد الضغط للعمود الخرساني على وجه العموم خصوصا عند درجات الحرارة أعلى من 100 درجة مئوية

ABSTRACT

In this paper, a basic heat transfer model for predicting the temperature distribution through the concrete column is presented. The governing partial differential equation is approximated by using the finite difference method into a set of Ordinary Differential Equations (ODE's). The boundary as well as initial conditions are implemented and the fifth-order Runge-Kutta method is used for integrating the resulting set of ordinary differential equations.

The model predictions for the temperature distributions are validated by using experimental data from literature. The general behaviors of the model as well as the effect of the key model parameters are investigated at length in the review. Finally, by using a correlation from the existing literature, an estimation of the reduction in the column's compression strength is presented.

The results show that the model predictions of the temperatures distributions within the concrete column are in good agreement with the experimental data and the increase of temperature of the High Strength Concrete (HSC) column due to fire shows a considerable reduction in the column compression strength.

KEYWORDS: High strength concrete; Fire; Modeling fire behavior; Model validation; Experimental data; Spalling.

INTRODUCTION

Fire is a global disaster in the sense it disrupts the normal human activities and leaves behind its scars for some time to come. In addition it occurs more often than any other known form of disaster. The accumulated annual loss on account of fire is significant and comparable to the damage caused by other bigger natural disasters.

In recent years, the construction industry has shown significant interest in the use of high strength concrete (*HSC*). This is due to the improvements in structural performance, such as high strength and durability, that it can provide compared to traditional normal strength concrete (*NSC*). *HSC*, which was widely used in applications such as bridges, off-shore structures and infrastructure projects, has been extended to building columns. Often, *HSC* columns form the main load bearing component of a building envelope and hence, the provision of appropriate fire safety measures for these columns is one of the major safety requirements in building design. The basis for this requirement can be attributed to the fact that, when other measures for containing the fire fail, structural integrity is the last line of defense.

Generally, concrete structural members exhibit good performance under normal situations. However, results from a number of studies [1,2,3] have shown that there are well-defined differences between the properties of *HSC* and *NSC* at high temperatures. Further, concern has developed regarding the occurrence of explosive spalling when *HSC* is subjected to rapid heating, as in the case of a fire [4,5,6].

This study is aimed at the development of a fundamentally- based model to predict the transient behavior of the temperature distribution through a *HSC* concrete columns. A 3D general conduction equation is solved and the temperature distribution within the concrete columns is predicted. The resulted temperature distribution within the column is then used for estimating the reduction of the compression strength of the column because of the high temperature.

Behavior of HSC Exposed to Fire

Concrete columns are generally classified into two main types namely, the high strength concrete (*HSC*) and the normal strength concrete (*NSC*). Concrete up to a compressive strength of 55 MPa is referred to as normal strength concrete (*NSC*), while concrete with compressive strength in excess of 55 MPa is classified as high strength concrete (*HSC*). The *HSC* behavior at elevated temperature may be significantly different from that of *NSC* where the behavioral differences between *HSC* and *NSC* are found in two main area: (i) the relative strength loss in the intermediate temperature and (ii) the occurrence of explosive spalling in *HSC* at similar intermediate temperatures. The tendency for explosive spalling of *HSC* mean that *HSC* structural elements may be more susceptible than *NSC* to losing the concrete over that provides thermal protection for the steel reinforcement, as shown in Figure (1) [13].

All information from various studies show that fire performance of *HSC*, in general, and spalling, in particular, is affected by the following factors: (1) original compressive strength (2) moisture content of concrete (3) concrete density (4) model dimensions and shape, and (5) loading conditions.

Data from the studies carried out at specialized laboratories, as well as a number of organizations world-wide [1,2,6], show that fire performance of HSC, in general, and spalling in particular, is complex and is affected by a number of factors. Based on the analysis of model predictions, test data and the visual observations made during and after the fire tests, some of the factors that influence the fire endurance of HSC columns are briefly stated as: (i) Concrete Strength (ii) Concrete Moisture Content (iii) Concrete Density (iv) Fire Intensity (v) Specimen Dimensions (vi) Lateral Reinforcement (vii) Fiber Reinforcement (viii) Load Intensity and Type (ix) Type of Aggregate

Figure 1: Comparison between (NSC) left and (HSC) right [16]

. Figure (2) for example shows the effect of one of those factors namely the effect of the lateral reinforcement configuration on the Spalling behavior in *HSC* Columns after fire resistance test [12]

Figure 2: Comparison of Spalling in HSC Columns after Fire Resistance tests [16]

The modeling used in this paper was based on the ISO 834 curve and its used as a boundary condition for solving the governing equation. The heating curve of such a standard fire curve is known as the ISO 834 curve or the BS476 Part 20 curve as shown in Figure (3).

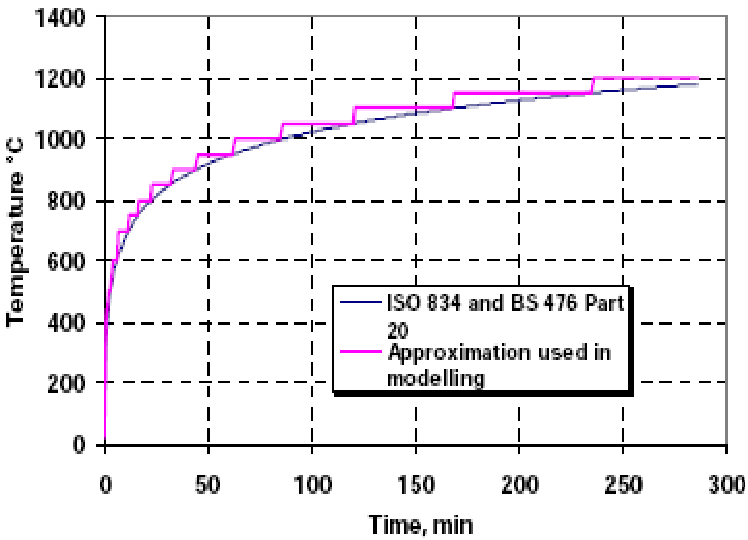

Figure 3: Standard temperature- time, ISO 834 [20]

Development of the Model

As mentioned above, this model is based on the transient conduction equation subjected to the variable thermal boundary condition. There are two exposure conditions which may be used as a thermal boundary condition when we deal with modeling of the concrete column subjected to fire: *(i)* the temperature on the fire exposed surface is same as the temperature of fire in a severe or fierce exposure condition, *(ii)* the mild heating condition is same as the furnace heating where a combination of convection and radiation constitute the heating mechanism. The formulation used in this study assumes a severe exposure condition. In other words, the temperature of the air surrounding the square concrete column is changing with time and referring to it as T_f (*pre-determined by Eq. 3*). As Figure (4) show, a high strength concrete column (*400x400x3810*) mm subjected to high temperature is modeled. This situation simulates a (*HSC*) column subjected to high temperature due to fire.

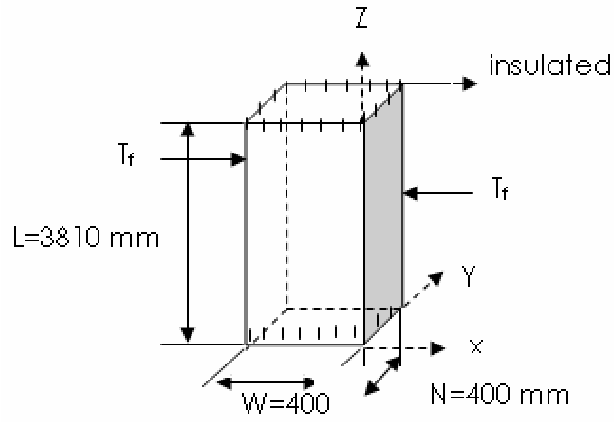

Figure 4: Physical description of the model

Governing Equation and Key Assumptions

The basic equation used in this study is the general transient conduction equation with constant thermal properties (Eq. 1)

$$\frac{\partial}{\partial x}\left(k\frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k\frac{\partial T}{\partial y}\right)+\frac{\partial}{\partial z}\left(k\frac{\partial T}{\partial z}\right)+q=\rho c\frac{\partial T}{\partial \tau} \tag{1}$$

Divide both sides by k and assuming that: *(i)* constant phys-thermal properties *(ii)* no heat generation inside the column we end-up with,

$$\frac{\partial^2 T}{\partial x^2}+\frac{\partial^2 T}{\partial y^2}+\frac{\partial^2 T}{\partial z^2}=\frac{1}{\alpha}\frac{\partial T}{\partial \tau} \tag{2}$$

As mentioned above, the heating curve of a standard fire is known as the ISO 834 fire curve. This curve is correlated as a function of time τ (Eq. 3) and used as boundary condition in both x and z directions (*severe exposure condition*).

$$T_f = 20 + 750\,[1 - e^{-3.79533\sqrt{\tau}}] + 170.41\sqrt{\tau} \tag{3}$$

Boundary and Initial Conditions

We have six boundary conditions at *x, y* and *z* directions and one initial condition at $\tau=0$. These are:

$$T(0,y,z,\tau)=T_f \qquad T(W,y,z,\tau)=T_f \qquad T(x,0,z,\tau)=T_f \qquad T(x,N,z,\tau)=T_f$$

$$\frac{\partial T}{\partial Z}(x,y,0,\tau)=0 \qquad \frac{\partial T}{\partial Z}(x,y,L,\tau)=0 \qquad T(x,y,z,0)=T_i \;=20\,^0C$$

Computation Domaines and Solution Techniques

The first step in the formulation is to subdivide the *x, y and z* directions into equally spaced nodes, Δx, Δy and Δz respectively. For the sake of numerical stability the computational domain is divided into 1000 nodes in each direction. As an approximation, we will let the temperature at each node represent the temperature of a cubic element (*compartment*) with a volume the $(\Delta x \Delta y\ \Delta z)$. In order to implement the boundary conditions, the first and last nodes (*the boundary nodes*) are formulated differently from the rest of the nodes (*interior nodes*). Then the central finite difference expression is used to approximate the terms $\frac{\partial^2 T}{\partial x^2}$, $\frac{\partial^2 T}{\partial y^2}$ and $\frac{\partial^2 T}{\partial z^2}$, and a set of first order Ordinary Differential Equations (*ODE's*) are resulted in each direction. Finally, the resulting set of (*ODE's*) is solved by using the fifth order Runge-Kutta method and the *3-D* transient temperature distributions within the concrete column are founded.

RESULTS AND DISCUSSIONS

The results of this study are presented at length in this section. They are laid out in three main subsections. In the first subsection, the general behaviors of the model as well as the effect of the key model parameters are investigated. In this research, the model predictions for the temperature distribution were validated using data generated from previous experimental study [13]. In that context, this will be presented in the second subsection. Finally, the third subsection is devoted to the effects of the high temperature on the compression strength of the concrete column.

General Behavior of the model

Figures (5) and (6) show the effect of one of the most important key parameters, namely the thermal diffusivity of the column material (α) . As Figure (5) depicts, when the value of α is assumed constant (α= $1.49 \times 10^{-5}\ m^2/s$), there is a good agreement between the model prediction and the actual experimental data.

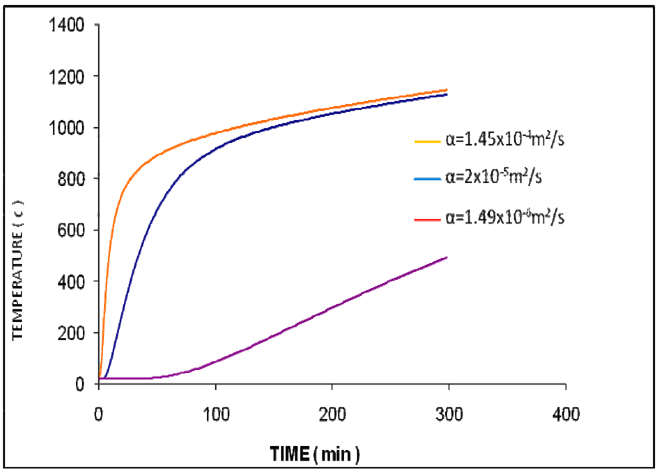

Figure 5: Temperature Distribution with time of nodes and Z_{exp}

However, when the value of α is assumed to be a function of temperature (*which in fact it is*), there is some deviation from the experimental data. This is not due to the fact that α is not a function of temperature, but rather it is largely due in part to inappropriate correlations used in previous studies. Thus, this emphasizes the need for the development of a new correlation highlighting this property of thermal diffusivity.

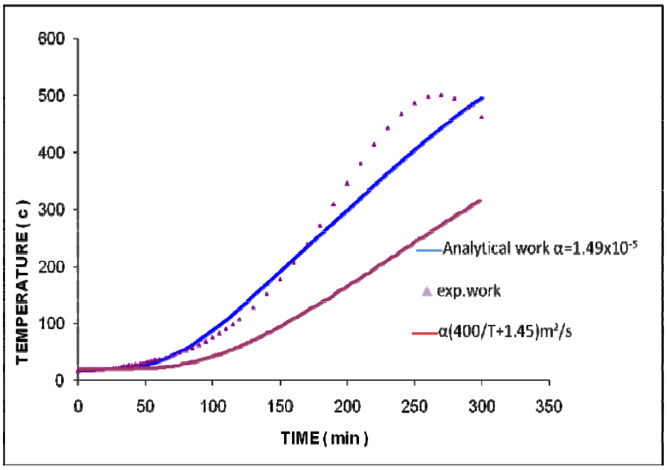

Figure 6: Temperature Distribution with time at different values of thermal diffusivity (α)

In the same logic, Figure (6) depicts the importance of the value of thermal diffusivity in the model prediction. For instance, three values of α were used *(α= $1.45x10^{-4}$, $\alpha=2x10^{-5}$ and $\alpha=1.49x10^{-6}$ m^2s^{-1})*. As can be observed, the lower the value of (α), the lower the temperature reading.

Model Validations

This subsection is primarily concerned with the model validation through comparison of the model prediction to the experimental data. In Figure (7), the temperature as a function of time is depicted in which the values obtained in this research are set into contrast with the experimental data.

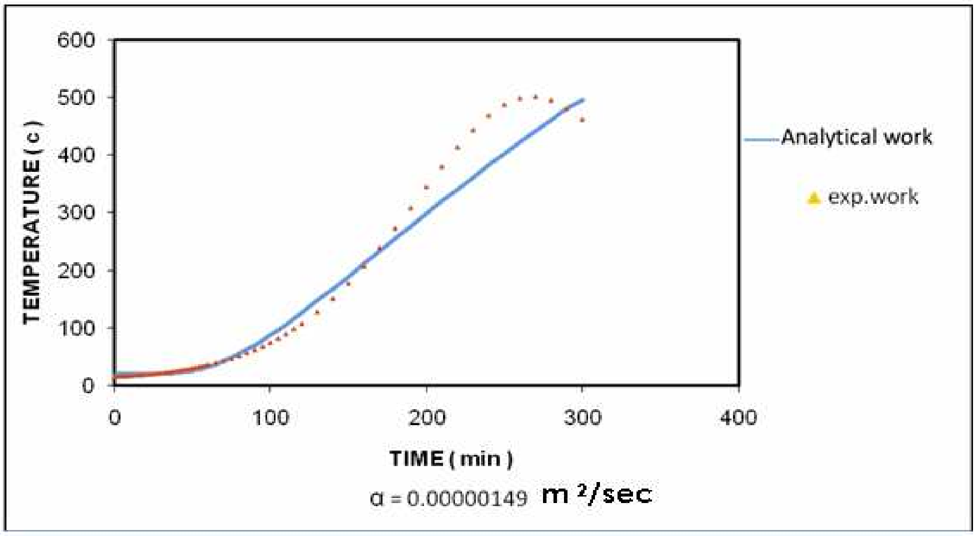

Figure 7: Model validation with the experimental data

It can be seen that there is a good level of agreement and validity between the two results. As already stated above, this agreement in values occurred under the assumption of a constant value of (α). We expect that the validation would be improved if we could obtain a correlation using the value of α as a function of temperature.

The Effect of High Temperature on the Compression Strength

As was already stated, the main objective of this research was to study the effect of high temperature on the compression strength of high strength concrete (*HSC*). In this context, an equation from previous research (Namely Abbasi et. al. [19]) was used to evaluate the effect of elevated temperature on the strength of concrete columns. When concrete is exposed to elevated temperatures for prolonged periods of time, it begins to lose strength. This loss of strength is characterized as the ratio between the column strength at the specified elevated temperature to the strength of the column at room temperature. This ratio is termed as the reduction factor (K_c). Abbasi et al has already derived relations to calculate the reduction factor. Four different correlations to the reduction factor (K_c) were given according to the temperature. The correlations are:

$$\frac{\sigma'_{cT}}{\sigma'_c} = K_c \qquad (4)$$

Where σ'_{cT} = apparent concrete strength at the specified elevated temperature.
σ'_{c} = apparent concrete strength at a normal (room) temperature.
K_c = reduction factor of the strength of the concrete.
Equation (4) should be used along with the following equations depending on the value of temperature (*T,* in ^{0}C).

$$K_c = 1 \qquad \text{for } T \le 100 \qquad (5)$$
$$K_c = (1.067 - 0.00067\,T) \qquad \text{for } 100 \le T \le 400 \qquad (6)$$
$$K_c = (1.44 - 0.0016\,T) \qquad \text{for } 400 \le T \le 900 \qquad (7)$$
$$K_c = 0 \qquad \text{for } 900 \le T \qquad (8)$$

In the first rang, when the temperature is less than or equal to 100 ^{0}C, the value of the reduction factor K_c is equal to a constant value (*unity*). In the second range of temperature, ($100 \le T \le 400$) K_c is given by a specific straight-line equation (Eq. 6). In the third range of temperature ($400 \le T \le 900$), K_c is given by Eq. (7). Finally, when the temperature is above 900 ^{0}C, the value of K_c is given by Eq. (8) in which the ratio between them approaches zero. The above-mentioned ranges are summarized in figure (8) below. These correlations depict the negative effect of elevated temperature on the strength of concrete columns and hence the durability of the concrete structure, in general. For instance, if the column is subjected to a hydrocarbon fire for 200 mins, in accordance to the prediction of this model, the temperature in the middle node will reach approximately 300 ^{0}C. Based on the figure (8), the value of K_c will be in the neighborhood of 0.8. In other words, it loses about 20% of its original strength. This highlights the drastic influence of the elevated temperature and more specifically the effect of fire on the strength of concrete structures. Much work remains to be done in this field analytically and experimentally in order to get more insight and more reliable correlations for linking the fire with physical properties of the concrete element, in general. There should be a comprehensive and thorough fundamentally-based model to predict simultaneously the effect of heat as well as the reduction in the compression strength of the concrete element of the building.

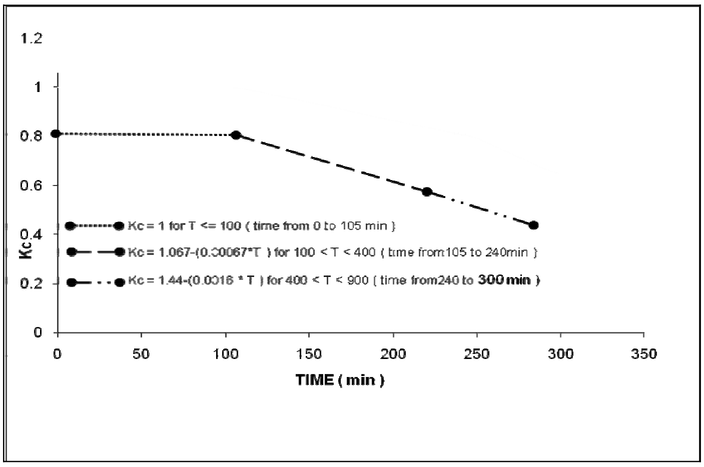

Figure 8: Reduction factor K_c versus time

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the present study, the following conclusions and recommendations can be drawn:

(i) When concrete is exposed to elevated temperatures for prolonged periods of time, it begins to lose its strength.

(ii) Based on the proposed model, the higher the column temperature the more its compression strength is reduced.

(iii) The proposed model, specified three ranges for the reduction factor K_c. In the first range, the value of K_c is equal to one where the column temperature $T \leq 100$. In the second range the value of K_c is given by Eq. (6) and valid when the column temperature $100 \leq T \leq 400$. Finally, in the third range the value of K_c is given by Eq. (7) and valid when the column temperature $400 \leq T \leq 900$. When the column temperature exceeds 900 ^{0}C then the value of K_c is equal to zero.

(iv) It is worth mentioning here that this analysis is based on *severe exposure conditions*. This boundary condition needs to be improved. More comprehensive and fundamentally-based models to account for the simultaneous effect of convection and radiation at the column boundaries are needed in any future work. Much work remains to be done in this field analytically as well as experimentally in order to get more insight and more reliable correlations for linking the fire with physical properties of the concrete element.

(v) It is recommended in any future work, that the effect of fire on the building as a whole rather than in an individual element is investigated. This can be done by using a commercial software that is capable of studying the behavior of the structure subjected to fire from a strength point of view.

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NOMENCLATURE

T	Temperature (^oC)
K_c	Reduction factor
q	Heat generated rate per unit volume
P,N,S,E,W	Positions in a control volume
$\frac{\partial}{\partial t}$	Partial derivatives with respect to time
,z θ r,	Cylindrical coordinate
$\frac{\partial T}{\partial r},\frac{\partial T}{\partial \theta},\frac{\partial T}{\partial z}$	Partial derivatives with respect to r,θ,z
X,Y,Z	Cartesian coordinates
$\frac{\partial}{\partial x},\frac{\partial}{\partial y},\frac{\partial}{\partial z}$	Partial derivatives with respect to x,y,z

Greek symbols

α	Thermal diffusivity ($m^2\ s^{-1}$)
τ	Time
ρ	Density (in kg/ m^3)
θ	Interface temperature
$\overline{\sigma_C}$	Concrete compressive strength at normal temperature (N/m^2)
$\overline{\sigma_{ct}}$	Concrete compressive strength at any temperature .(N/m^2)

Abbreviation

HSC	High Strength Concrete
NSC	Normal Strength Concrete
NRC	National Research Council of Canada
ODE	Ordinary Differential Equations
RH	Relative Humidity
NPD	Norwegian Petroleum Directorate