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Thermal Simulation and Performance Prediction of High Strength Concrete (HSC) Columns Subjected to Fire

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الملخص:

يهدف هذا البحث إلى دراسة التأثير الذي يحدثه ارتفاع في درجة الحرارة نتيجة لوجود حريق على اداء عمود خرساني مسلح في مبنى. و تجدر الاشارة هنا الي ان هذا البحث يخص الاعمدة الخرسانية علية المتانه والتي تتأثر سلبا بالحرائق اكثر من الاعمدة العادية المتانة. في البداية تم عرض ومناقشة تأثير درجة حرارة الحريق على الانهيار الكامل للعمود الخرساني وتم تحديد الحد الأدنى لانهيار وتفتت الخرسانة عالية المتانة وتمت كذلك الاشارة الى الحد المستخدم من بعض المواصفات القياسية للخرسانة عادية المتانة وتم التطرق الي الاساليب المستخدمة والمؤثرة في رفع هذه الدرجة و كذلك تم التطرق الى طريقة حساب فترة تعرض العمود للحريق والتي يعتبر عندها العمود الخرساني في حالة انهيار تام. وحيث ان التركيز في هذه الدراسة على تقييم المتانة المتبقية للعمود الخرساني فانه لم يتم الاستفاضة في تقدير حالة الانهيار التام من عدمها. بعد ذلك تم إيجاد توزيع درجات الحرارة خلال العمود الخرساني وذلك بحل المعادلة العامة للتوصيل الحراري ثلاثية الأبعاد الغير مستقر مع الزمن حلاً عددياً ومن معرفة توزيع درجات الحرارة. 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. تم وضع مخطط رياضي مبسط يمكن بواسطته معرفة تأثير ارتفاع درجات الحرارة وزمن الحريق خلال العمود على إجهاد الضغط للخرسانة وقد أظهرت هذا الدراسة أن لزيادة درجات الحرارة الناتجة عن الحريق تأثير سلبي على إجهاد الضغط للخرسانة على وجه العموم خصوصاً عند درجات الحرارة أعلى من 100^0C. وفي النهاية تم عرض كيفية استخدام نتائج البحث في اسلوب التصميم – ضد الحرائق لعمود خرساني معرض لإجهاد ضغط محوري فقط كمثال، حيث انه يمكن استخدام هذه النتائج في تصميم الاعمدة الخرساني المعرضة لأحمال مختلفة.

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Abstract

The aim of this work is to study the effects of high temperature generated from fire on the high strength concrete (HSC) columns performance. Studies in the literature indicate that high strength concrete (HSC) columns are more adversely affected by fires than normal strength concrete (NSC) columns. In this study, first the sapling failure mode of HSC columns is reviewed and the spalling temperature and methods to improve it is discussed. Then the fire resistance criteria based on time is highlighted. The previous deals with total failure. However, as the main focus of this work is to determine the HSC columns strength degradation. Accordingly , the fire behavior of HSC columns is numerically investigated. A basic heat transfer model for predicting the temperature distribution through the concrete column is presented. The governing partial differential equation is approximated into a set of ordinary differential equations (ODE's) using the finite difference method. The boundary and the 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. Then, by using a correlation from the existing literature, an estimation of the reduction in the concrete's compression strength based on temperature and time is developed. The results show that the model predictions of temperatures distributions within the concrete column are in good agreement with the experimental data. Furthermore the increase of temperature within the column due to fire will cause a consid-erable reduction in column's concrete compression strength. Finally, a simplified approach for fire design of axially laded HSC columns based on Rankine formula is presented.

Keywords: High Strength Concrete, Fire Design, Modeling Fire Behavior, Model Validation, RC columns, Spalling

1.0. Introduction

Fire is one of the most severe conditions which may be encountered by a reinforced concrete (RC) building during its service life. Therefore, the fire performance of RC members is an important issue that needs to be considered in the design of RC buildings. Columns are the primary structural elements that transfer the loads of a building vertically to the foundation .When RC columns are exposed to fire, the material properties of concrete and the reinforcing steel change as a result of the temperature increases. The decreases in yield strength and modulus of elasticity reduce the overall strength of the column. Once the column strength decreases lower than the applied load, the column will fail either by crushing or by flexural buckling.

In recent years, building construction industry has shown significant interest in the use of high strength concrete (HSC) instead of normal strength concrete NSC concrete. This is due to the enhancement in the reinforced concrete structural performance, such as high strength and durability, that it can provide compared to traditional normal strength concrete (*NSC*). Recent editions of building codes such as the ACI code in USA and the CSA code in Canada contain detailed specifications on the structural design of HSC structural members; however, they do not include any guidelines for their fire performance design [18]. According to several studies in the literature [1–6,10–12] , NSC columns exhibit good performance under fire situations compared to HSC columns. This is due to the deference in material properties especially the low porosity (permeability) of HSC. 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. This study is aimed to develop a fundamen-tally based model to predict the transient behavior of the temperature distribution through a *HSC* columns. A 3D general conduction equation is solved and the temperature distribution within the concrete columns is predicted. The results of the temperature distribution within the column are then used for estimating the reduction of the concrete compression strength of the column. In the literature design charts which estimate concrete strength reduction due to fire are based on fire temperature only. In this study a design chart based on fire temperature and fire duration is proposed for estimating concrete strength reduction due to fire. Finally, a simplified HSC column fire design approach is presented which can effectively utilize the thermal simulation finding.

2.0. Behavior of HSC Exposed to Fire

Concrete columns are generally classified into three main types namely, the normal strength concrete (*NSC*), the high strength concrete (*HSC*), and the ultra-high strength concrete. The compressive strength of normal strength concrete (*NSC*) used to be around 20 to 50MPa. In recent years, concrete with a compressive strength in the range of 50 to 120MPa has become widely available and is referred to as high-strength concrete (*HSC*). When compressive strength exceeds 120MPa, it is often referred to as ultrahigh performance concrete (*UHP*) [3]. The building contribution of the *UHP* concrete is still limited to some critical applications and will not be considered any further in this work. 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. This is discussed in more details in the next section.

3.0. Spalling Failure and Design Consideration

Spalling of concrete under fire conditions is one of the major concerns in HSC and should be accounted for in designing HSC columns exposed to fire [10]. Fire induced spalling is attributed to dense micro-structure and lower permeability of HSC that prevents dissipation of pore pressure generated from water vapor in HSC members when exposed to high temperatures. When this pore pressure build-up it exceeds the tensile strength of concrete, and pieces of concrete break-off from the surface of concrete structural member [9]. With increasing temperature, tensile strength of concrete decreases and thus the risk of spalling increase. The faster degradation of compressive strength with temperature, combined with occurrence of spalling, leads to lower fire resistance in HSC members. Data from various studies show that spalling in HSC is affected by concrete strength, concrete density, load intensity and type, moisture content, tie configuration, fire intensity, aggregate type, addition of fibers and specimen dimensions [8].

For accurate modeling to predict spalling, pressure-temperature relationship is required. However, such data is not well documented in the literature at present. Hence in design codes and standards a simplified approach is used in order to minimize the risk of spalling[11]. Based on detailed experimental studies on HSC columns, it was found that spalling occurs when temperatures in concrete reach above 350^{o}**C**[12]. This is shown in Fig.1a. However, the Eurocode states a value of 500^{o}C for the NSC spalling limit [11]. It can be seen from Fig. 1b that spalling is likely to occurs in the column surface zone when the temperature reaches 350 ^{o}C, and after sufficient time duration the spalling zone spreads toward the center. Data from the experimental studies also showed that, while spalling occurs throughout the cross-section in the case of columns with straight ties, spalling occurs only outside the reinforcement core when the ties are bent in to the concrete core as shown in Fig. 1b[1].

Further, the addition of fibers to concrete helps in minimizing the extent of spalling in HSC members. The presence of polypropylene or steel fibers in concrete influence the extent of spalling [7,12-14]. The polypropylene fibers melt at relatively low temperatures (about 167-170°C) and create randomly oriented micro and macro channels inside concrete.

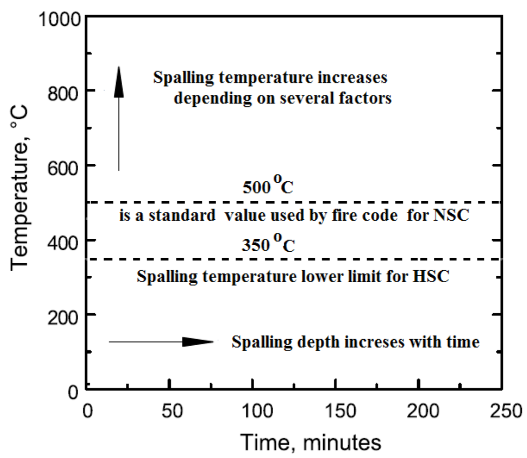

Fig. 1 (a) Spalling temperature limits and progression.

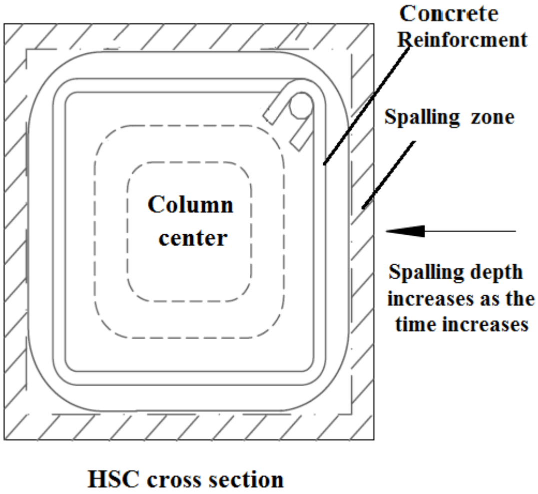

Fig. 1 (b) spalling zone and spalling depth increase direction.

These channels facilitate dissipation of high vapor pressure generated in concrete members. The addition of steel fibers overcomes spalling through enhancing tensile strength of concrete [9,12-13]. Alternatively, hybrid fibers comprising of both polypropylene and steel fibers can be added to HSC to mitigate fire induced spalling in HSC members [14]. There have been numerous studies on fire performance of HSC columns with polypropylene fibers, but there are only limited studies on fire performance of HSC columns with steel and hybrid fiber reinforcement. Further there is lack of sufficient information on comparative fire performance of HSC columns made with different fibers. In this regard, to illustrate comparative fire performance of HSC columns with different fiber combinations Wasim Khaliq et al.[7] carried out fire resistance tests on HSC columns with different fiber reinforced concrete mixes. As shown in Fig. 2, in the HSC column without any fibers, severe spalling occurred which led to loss of concrete cross-section, while the HSC-P column with polypropylene fibers experienced some level of surface scaling and resulted in minor loss of concrete cross-section. No fire induced spalling was observed in HSC-S with steel fibers and HSC-H columns with hybrid steel and polypropylene fibers. This can be attributed to increased tensile strength facilitated by the presence of steel fibers in HSC-S and HSC-H and also increased permeability achieved through melting of polypropylene fibers in the case of HSC-H column [7 and 15].

In summary, it can be concluded that the present plain HSC column spalling temperature is around 350 ^{o}C, which is at the lower side. The extent or spalling zone would be determined according to the time duration of the fire and the HSC column composition. Clearly this shows that spalling improvement of the HSC columns is demanding and it is a significant active area for further research. This subject will not be covered further in this article and the reader can refer to the more detailed studies in this issue [10-15].

4.0. Fire Resistance Rating

Fire resistances defined as the property of a building assembly to withstand fire, or give protection from it according to the criteria defined by ASTM Methods E119[11].

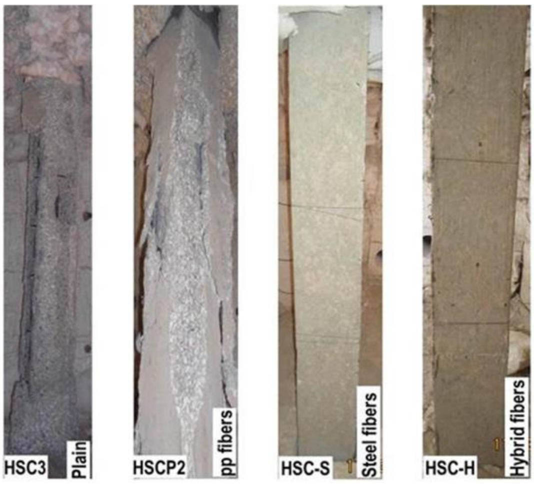

Fig. 2. The effect of polypropylene fibers , steel fibers and hybrid fibers on spalling of HSC columns [7].

Fire rating or fire resistance rating is a time required, usually expressed in hours, for an element in a building to maintain its particular fire-resistant properties. Model codes establish the required fire ratings for various building elements. In the definition of fire resistance there are two issues. The first issue is the ability of a building assembly to maintain its structural integrity and stability despite exposure to fire. Secondly, for some assemblies such as walls and floor-ceiling assemblies, fire resistance also involves serving as a barrier to fire spread. Wade C. A. et al. (1997) [36] carried out an extensive research work on reinforced concrete columns fire resistance rating. From the results of their research an empirical model for fire resistance rating (in minutes) was developed as given the equation below:

$$R = \frac{K \times {\sigma'_c}^{a} \times B^{b} \times D^{c}}{C^{d} \times Le^{e}} \qquad (1)$$

$$\text{for } C \geq \; 0.4 \; \times Nu_0 \qquad (2)$$

Where: a, b, c, d & e are linear regression constants

R: fire resistance period of the column (min)

k: a constant dependent on the cover and amount of steel

σ_c': the 28-day compressive strength of the concrete (MPa)

B: least dimension of the column cross-section area (mm)

D: the greatest dimension of the column cross-section area (mm)

C: the design axial load for fire conditions (kN)

L_e: effective length [mm]

Nu_0: initial ultimate strength in compression (kN)

They compared their model prediction against the Australian fire resistance standard requirement AS 3600 [37] and the results are as shown in Table 1. They concluded that "the existing requirements for fire resistance of reinforced concrete columns are overly conservative". This finding is agreed by other researchers as well. This approach is also criticized by other researchers as well from various issues such that this approach dose not account for spalling, and other factors [20].

This shows that this criteria should be considered as an active area of further research particularly for the HSC columns. Therefore due to this reason and also due to the fact that this work focus on the evaluation of the remaining performance of the HSC column, it is not considered any further in this work. It is mentioned here to provide a complete picture on the failure criteria adopted in this filed.

Table 1. Comparison of eq. (1) and AS3600 standard fire resistance [37]

Column Size (mm/mm)	Fire Resistance from Eq. 1 (min)	Fire Resistance from AS 3600 (min)
150 x 600	68	30
150 x 1200	84	30
200 x 200	64	60
200 x 400	78	60
200 x 800	96	60
200 x 1600	119	60
300 x 300	103	90
300 x 600	127	90
300 x 1200	157	90
500 x 500	191	120
500 x 1000	235	120

5.0. Thermal Analysis and Modeling

In structural fire performance testing, a building element, such as a column, is placed in a furnace and subjected to a controlled fire (i.e time-temperature curve) while being loaded to prescribed force. Two time-temperature curves are used universally: ASTM E119 and ISO 834 (Buchanan, 2002). Even though these are two different time-temperature curves from two different standards, both are very similar as shown in Fig 3. In this study both of the ASTM E119 and ISO 834 fire curves can be used in the heat transfer modeling and analysis.

The model proposed in this study 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.5*). A high strength concrete column (*400×400×3810*) mm subjected to high temperature is modeled as shown in Fig 4 . This situation simulates a (*HSC*) column subjected to high temperature due to fire.

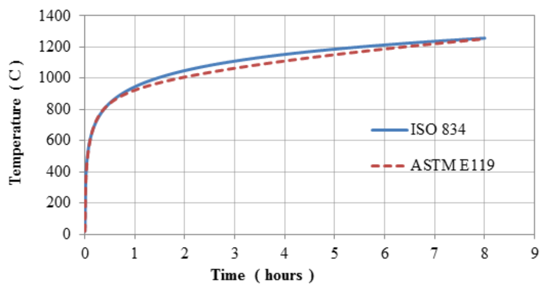

Fig. 3. Standard temperature- time ISO 834 and ASTM E119 curves [16-17]

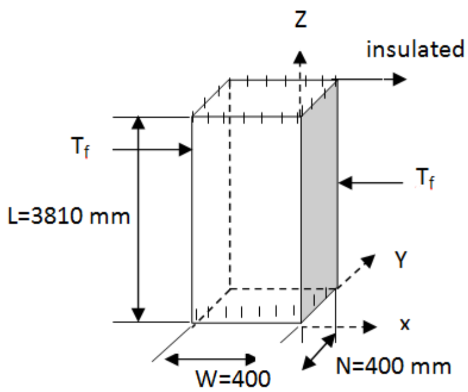

Fig. 4. Schematic description of the model

5.1. Governing Equation and Key Assumptions

The basic equation used in this study is the general transient conduction equation with constant thermal properties [22-28].

$$\frac{\partial}{\partial x}\left(k\frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(k\frac{\partial T}{\partial y}\right)+$$
$$\frac{\partial}{\partial z}\left(k\frac{\partial T}{\partial z}\right)+q=\rho c\frac{\partial T}{\partial \tau} \qquad (3)$$

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

$$\frac{\partial^2 T}{\partial x^2}+\frac{\partial^2 T}{\partial y^2}+\frac{\partial^2 T}{\partial z^2}=\frac{1}{\alpha}\frac{\partial T}{\partial \tau} \quad (4)$$

As mentioned above, the heating curve of a standard fire is known as the standard fire curve [16-20]. In this particular work, the ASTM E119 curve is correlated as a function of time τ (Eq. 5) and used as boundary condition in both *x* and *z* directions (*severe exposure condition*) [20].

$$T_f = T_i + 750[1-e^{-3.79533\sqrt{\tau}}] + 170.41\sqrt{\tau} \quad (5)$$

where T_f is temperature in Celsius, τ is time in hours, and T_i is ambient temperature in Celsius.

5.2. Boundary and Initial Conditions

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

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

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

5.3. 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 determined.

6.0. Results and Discussions

The results of this study are presented in this section. They are laid out in three main subsections. In the first, 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[5]. 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.

6.1. General Behavior of the Model

Figs. 5 and 6 show the effect of one of the most important key parameters in the thermal simulation, namely the thermal diffusivity of the column material (α). Fig.5 shows how the simulation is significantly affected by the value of α. For instance, three values of α were used ($\alpha = 1.45x10^{-4}$, $\alpha=2x10^{-5}$ *and* $\alpha = 1.49\times10^{-6}\ m^2s^{-1}$). As can be observed, the lower the value of (α), the lower the temperature reading. Also, as shown in Fig.6, when the value of α was kept constant at $\alpha = 1.49\ x\ 10^{-5}\ m^2/s$, there is a good agreement between the model prediction and the actual experimental data. However, when the value of α is assumed to be a function of temperature (given by a correlation), there is some deviation from the experimental data as shown in the lower curve of Fig.6. 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 more refined correlation which is more realistically and efficiently describing this property (α).

6.2. Model Validation

This subsection is primarily concerned with the model validation through comparison of the model prediction to the experimental data obtained from reference [5]. In Fig.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.

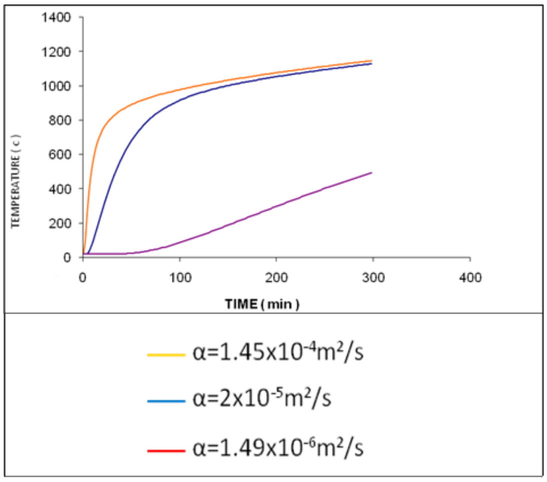

Fig. 5. Temperature distribution with time at different values of thermal diffusivity (α) (middle node)

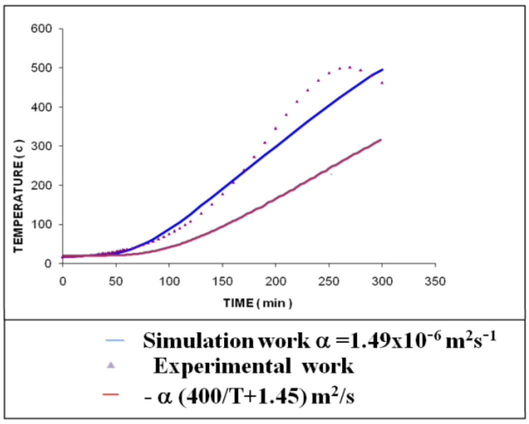

Fig. 6. Temperature distribution with time for simulation with $\alpha = 1.49 \times 10^{-5}\ m^2/s$, experimental, and correlation of thermal diffusivity(α) (middle node)

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 refined correlation using the value of α as a function of temperature. This is part of ongoing research work in the department at the present time.

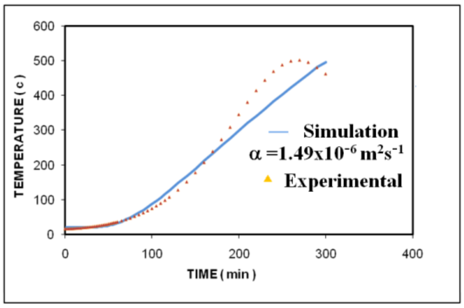

Fig. 7. Model validation with the

experimental data (middle node)

6.3. The Effect of High Temperature on the Concrete Compression Strength

As was already stated, the main objective of this research was to study the effect of the high fire temperature on the performance of the high strength concrete columns (*HSC*).

In this context, an equation developed by Eurocode 2 [11] and used by previous research[21]is adopted here to evaluate the effect of elevated temperature on the strength of concrete columns. When concrete is exposed to elevated temp-eratures for prolonged periods of time, it begins to lose its strength. This loss of strength is characterized as the ratio between the column concrete strength at the specified elevated temperature to the strength of the column concrete at room temperature. This ratio is termed as the concrete reduction factor (K_c). Abbasi et al (2005) has already used the relation adopted by Eurocode 2 in the 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[11]:

$$\frac{\sigma'_{cT}}{\sigma'_{c}} = K_c \qquad (6)$$

Where σ'_{cT}= concrete strength at the specified elevated temperature.

σ'_{c} = concrete strength at a normal (room) temperature.

K_c= reduction factor of the strength of the concrete.

Equation (6) should be used along with the following equations depending on the value of temperature (T, in ^{0}C).

$K_c = 1$ for $T \leq 100$ (7)

$K_c = (1.067 – 0.00067\ T)$

for $100 \leq T \leq 400$ (8)

$K_c = (1.44 – 0.0016\ T)$

for $400 \leq T \leq 900$ (9)

$K_c = 0$ for $T \geq 900$ (10)

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, ($100 \leq T \leq 400$) K_c is given by a specific straight-line equation (Eq. 8). In the third range of temperature ($400 \leq T \leq 900$), K_c is given by Eq. (9). Finally, when the temperature is above 900 ^{0}C, the value of K_c is given by Eq. (10) in which the ratio between them approaches zero (complete failure).

The above-mentioned ranges have been introduced to the developed thermal model and the results of this analysis are presented in Fig.8 below. These correlations depict the negative effect of elevated temperature on the strength of concrete and hence the durability of the concrete structure, in general. For instance, if the column is subjected to a fire for 200 mins, in accordance to the prediction of this model, the temperature in the middle node will reach approximately 300 ^{0}C. Therefore, based on Fig.8, the value of K_c will be in the neighborhood of 0.6. In other words, it loses about 40% 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. Using the previous data with other correlation which is developed by Abasi et al. (2005) [21] leads to a K_c factor of 0.329 which is considered in the lower side of concrete degradation. Clearly, there is a large difference between the two K_c results. The same procedure is required for the reinforcing bar reduction K_σ which is part of the ongoing research at the department in the present time. However, much work remains to be done in this field analy-tically and experimentally in order to get more insight and more reliable correlations for linking the fire with physical properties of the reinforced concrete structures, 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 strength of the concrete structure elements of the building.

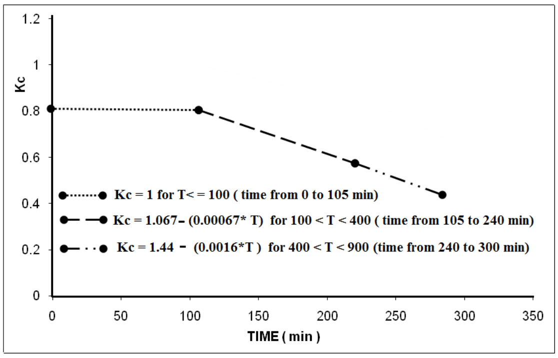

Fig. 8. Reduction factor K_c versus time.

7.0. Simplified HSC Column Fire Design (Approach Utilization)

General speaking, the analysis and design of reinforced concrete columns subjected to a fire may be performed by many methods with different levels of complexity and accuracy. On the simplest practical level, descriptive methods in the form of tabulated data may be applied, but only within the ranges specified by appropriate codes [11]. On the other hand, in recent years there has been observed significant progress in working out more and more sophisticated methods for the fire design of reinforced concrete members, taking advantage of modern computational tools and advanced material modeling [29].

In this work, the emphasis was placed onto simplified methods for determining fire load capacity of reinforced concrete columns subjected axial (normal) force, which can be located between the two aforementioned extreme approaches. Despite advanced thermal and mechanical models for reinforced concrete columns, there is still a strong need for developing and improving simplified design methods for everyday practices that are usually limited to more typical engineering solutions. Such methods should allow an engineer to exert control over calculation procedures carried out for fire design situations and may also constitute initial designs for complex, non-typical structures [29]. In this regard , the following subsection will present how we can utilize the previously obtained fire design chart (Fig.8) which is basically the main outcome of this research work in the simplified fire design approach of HSC columns.

7.1. Rankine Formula

The premise is that HSC columns in fire can fail under two modes: crushing for stocky columns and buckling for slender columns. For, the mostly encountered, HSC columns in the intermediate range, these two modes will interact with each other, causing a reduction in the load capacity of real columns [30]. The Rankine approach assumes a linear interactive relationship between the two failure modes. The method has been applied to steel columns and frames, and also composite columns [31-32]. For the particular application of reinforced concrete RC columns four case studies including a total of 76 RC columns were used to verify this approach [30]. For all the four case studies, the Rankine predictions give consistent predictions with coefficient of variations around 25%, which are reasonably good for RC columns under fire conditions. Furthermore, for most of the columns, the Rankine predictions are on the conservative side due to its interactive nature [30-31].The Rankine formula for HSC columns under fire conditions takes the following form:

$$\frac{1}{P_R(t)} = \frac{1}{u_{pr}P_p(t)} + \frac{1}{P_e(t)} \qquad (11)$$

with P_R predicted failure load by the Rankine formula;

u_{pr} reduction factor of the plastic squashing load;

P_p plastic squashing (direct compression)load;

$u_{pr}P_p$ short column capacity;

P_e elastic buckling load;

t fire exposure time; t = 0 for ambient conditions.

The theoretical basis of the above formula has been discussed by Tang et al [31].Clearly, the Rankine formula provides a linear interaction relationship between the plastic squashing load P_p and the elastic buckling load factor P_e. The actual behavior of a column is dependent on its slenderness ratio [33]:

$$\wedge = \sqrt{P_p}/P_e \qquad (12)$$

The term $\wedge$ provides a simple and direct indication of the column slenderness. In Equation (11), the plastic collapse load *Pp(t)* can be determined by:

$$P_p(t) = \beta_c(t)\sigma_c' A_c + \beta_{yr}(t)\sigma_s' A_{sr} (13)$$

with σ_c' concrete strength at normal (room) temperature;

σ_s' yield strength of steel reinforcement at normal (room) temperature

A_c area of concrete;

A_{sr} area of steel reinforcement.

The terms $\beta_c(t)$ and $\beta_{yr}(t)$ are the respective strength reduction factors accounting for the deterioration of concrete and steel reinforcement under fire conditions and given by:

$$\beta_c(t) = \frac{\int \sigma_{cT}' dA_c}{\sigma_c' A_c} \qquad (14)$$

$$\beta_{yr}(t) = \frac{\sum \sigma_{sT}' A_{sr}}{\sigma_s' A_{sr}} \qquad (15)$$

With σ_{cT}' :concrete strength at the specified elevated temperature (as above)

σ_{sT}' : yield strength of steel reinforcement at the specified elevated temperature conditions

Similarly, the elastic buckling load can be determined by:

$$P_e(t) = \frac{\pi^2[\beta_{EC}(t).0.2E_c(0)I_c + \beta_{Esr}(t).E_{sr}(0)I_{sr}]}{L_e^2} \quad .... \quad (16)$$

with *Ec* elastic modulus for concrete; (0) at ambient temperature and (t) at elevated temperature.

Ic second moment of area of concrete;

Esr elastic modulus of steel reinforcement;

Isr second moment of area of steel reinforcement;

Le column effective length taking account of different support conditions

The terms $\beta_{EC}(t)$and $\beta_{Esr}(t)$ are the respective stability reduction factors accounting for the deterioration of concrete and steel reinforcement under fire conditions.

$$\beta_{EC}(t) = \frac{\int E_c(t)dI_c}{E(0)I_c} \qquad (17)$$

$$\beta_{yr}(t) = \frac{\sum E_{sr}(t)I_{sr}}{E_{yr}(0)I_{sr}} \qquad (18)$$

The material reduction factors $\beta_c(t)$ and $\beta_{yr}(t)$, $\beta_{EC}(t)$and $\beta_{Esr}(t)$ can be determined either experimentally or by finite element analysis. Dotreppe et al. [35] performed thermal analysis for RC columns under ISO 834 fire using a finite element program named SAFIR [35], which is developed at the University of Liège and developed an empirical colorations to these factors. However, these factors might be replaced by the reduction factors obtained by this fundamental study performed here and shown in Fig.8 for K_c . They could be considered as equivalent to K_c and K_σ mentioned in the previously subsections for the strength and K_{EC} and $K_{E\sigma}$ for the young's modulus which should be determined for the concrete and the steel reinforcing bars respectively. This proposed solution need further analytical and experimental verifications. If it is found working well, this will make it possible for professional design engineers to use it at least for un-critical application. This is in fact the main objective of the ongoing research project at the mechanical engineering department which showed a promising preliminary results. Upon completion of this work, it is expected that this work would provide a simple and fairly reliable method(s) for fire design of HSC columns and could be extended for other loading cases and other concrete structures.

8.0. 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 and at certain temperature it may fail by spalling.

ii). Based on analysis results of the proposed model, as column temperature increase its compression strength reduces.

iii). The proposed model predictions of temperatures distributions within the concrete column are in good agreement with the experimental data taken from literature.(see Fig 7)

iv). In the literature, design charts which estimate concrete strength reduction due to fire are based on fire temperature only. In this study a design chart based on fire temperature and fire duration is proposed for estimating concrete strength reduction due to fire. The chart based on a model which specify three ranges for the strength reduction factor K_c. In the first range when column temperature less or equal to 100 ^{0}C , the value of K_c is equal to one. In the second range when column temperature $100 \le T \le 400$ the value of K_c is given by Eq. (8) .In the third range when column temperature $400 \le T \le 900$ the value of K_c is given by Eq. (9). Finally, When the column temperature exceeds 900 ^{0}C then the value of K_c is equal to zero. As an example according to the proposed model , if the column is subjected to a fire for 200 mins, the temperature in the middle node will reach approximately 300 ^{0}C. Therefore, based on chart given Fig.8, the value of K_c will be in the neighborhood of 0.6. In other words, the column loses about 40% of its original strength.

v). A simplified procedure to utilize this work finding for designing axially loaded HSC columns subjected to fire is presented. Even though sophisticated simulation are available for fire resistance simulations, the subject is still under refinements, and also simplified procedures are still helpful for unsever cases. The simulation programs may need sufficient time for familiarization and expertise, which could not be guaranteed by many junior design engineers.

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