Behavior of one-way reinforced concrete slabs subjected to fire

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Finite difference;
Heat transfer;
Slabs

Abstract  A finite difference analysis was performed to investigate the behavior of one-way reinforced concrete slabs exposed to fire. The objective of the study was to investigate the fire resistance and the fire risk after extinguishing the fire. Firstly, the fire resistance was obtained using the ISO834 standard fire without cooling phase. Secondly, the ISO834 parametric fire with cooling phase was applied to study the effect of cooling time. Accordingly, the critical time for cooling was identified and the corresponding failure time was calculated. Moreover, the maximum risk time which is the time between the fire extinguishing and the collapse of slab was obtained. Sixteen one-way reinforced concrete slabs were considered to study the effect of important parameters namely: the concrete cover thickness; the plaster; and the live load ratio. Equations for heat transfer through the slab thickness were used in the fire resistance calculations. Studying the cooling time revealed that the slabs are still prone to collapse although they were cooled before their fire resistance. Moreover, increasing the concrete cover thickness and the presence of plaster led to an increase in the maximum risk time. However, the variation in the live load ratio has almost no effect on such time.

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1. Introduction

Structural fire performance engineering is a recent philosophy of design that has developed recently in structural engineering. Fire safety design can be achieved by active and passive fire protection systems. Active systems are generally self-activated once the fire is triggered. Such systems include fire detectors, smoke control systems, and sprinklers. However, passive systems are built into the structures such as building codes limitations, fire doors and windows and fire protection materials that prevent or delay the temperature rise in structural elements [1]. Many different fire exposures are used to study the reinforced concrete structure such as standard fire (without cooling phase), parametric and natural fires (with cooling phase) [1–4]. The parametric and natural fires usually represent actual fires better than the standard fire.

The behavior of reinforced concrete slabs under fire loading has been studied by researchers for many decades [5–9]. It is well known that when the temperature increases the slab fire resistance decreases. This is because when concrete is exposed to...
heat, chemical and physical reactions occur such as loss of moisture, dehydration of cement paste and decomposition of the aggregate. Such changes lead to high pore pressures caused by the water evaporation, internal microcracks and damages appear in concrete [10]. Also, the increase in the temperature leads to a decrease in the yield strength of the steel reinforcement. Concrete spalling under high temperatures is a major factor of reducing its fire resistance [11,12]. The spalling is caused by the build-up of pore pressure during heating. High strength concrete is believed to be more susceptible to this pressure build-up because of its low permeability compared to the normal strength concrete. Thus, high strength concrete is known to have less fire resistance than normal strength concrete [13,14]. The behavior of concrete slabs under fire is very sensitive to the stiffness and ends restrain condition. The fire resistance of one-way restrained slabs is generally higher than those for unrestrained slabs because compressive restraint in the surrounding structure decreased the slabs thermal expansion [15–17]. It is well known that the bottom concrete cover has significant influence on the fire resistance of the flexural member, but the lateral concrete cover has a less beneficial effect on the member fire resistance compared to the bottom concrete cover [18]. Codes of practice state that the temperature rise leads to strength degradation in both concrete and steel reinforcement based on the aggregate type and the grade of the steel [19,20]; However, such codes believed that the steel reinforcement temperature played the important role in strength degradation. The Eurocode 2-2004 [19] gives profiles for temperature distributions through the slab thickness in the case of slabs or through the cross section in the case of beams and columns based on fire resistance class for the load-bearing criterion for 30, 60, . . . minutes in standard fire exposure. It gives simple calculation methods for calculating the mechanical behavior namely 500 °C isotherm and zone methods. The ACI committee 216 [20] gives fire resistance for the slab based on the relative slab bending capacity which is the ratio of the moment due to applied load to the moment capacity of the section where the cover thickness is based on the aggregate type. The ECP 203-2007 [21] gives the fire resistance for different structural elements, (slabs, beams, and columns) according to their dimensions and the concrete cover thickness.

Most of research work found in the literature studied the behavior of reinforced concrete slabs and their fire resistance during exposure to fire. However little research work dealt with the influence of cooling time on the fire resistance of concrete slabs. Such studies considered only the time of cooling start on the fire resistance [5,7,10]. However, up to the knowledge of the authors there is no research work studied the risk of cooling time before the fire resistance. There is a critical time between the time of cooling start and the fire resistance. This critical time leads to failure of the concrete slab if cooled before its fire resistance. This is because the fire starts to decrease (cooled) while the concrete slab core temperature still increasing [22].

This paper presents a finite difference approach [23–25] for tracing the fire response of RC slabs under the standard and parametric ISO834 fire. Several parameters were considered such as concrete cover thickness, presence of plaster at exposed surface, and live load ratio. The scope of this study covers the behavior of simply-supported one way reinforced concrete slabs under fire. The model is verified against experimental and numerical data by comparing the predicted temperatures to the measured ones from Lie and Leir [23]. The model is capable of predicting the fire resistance and the influence of cooling before the fire resistance (time).

2. The model

The Finite difference method is considered in the current research to study the behavior of reinforced concrete simply supported one-way slabs under fire loading. The model considers the heat transfer through the slab thickness and both concrete and steel reinforcement strength degradation due to exposure to fire. The ISO834 standard fire without and with cooling phase is considered. The model considers three parameters namely: concrete cover, live load ratio, and the plaster thickness at the exposed surface to identify fire resistance. Also, the effect of cooling time before the fire resistance on the possibility of such slabs to collapse is studied and the corresponding failure time could be estimated. Moreover, the maximum risk time is determined. To perform such model the following assumptions are considered:

- Plane sections before deformation remain plane after deformation (linear strain).
- The concrete tensile strength is neglected.
- The concrete slab is under static load.
- The effect of spalling, expansion and shrinkage are neglected.
- Slab edge restraint is neglected.

2.1. Heat transfer through concrete slab

There are generally three modes of heat transfer namely conduction, convection and radiation. The surface of the element exposed to fire is subjected to heat transfer by conduction, convection and radiation. For concrete members, the convection is usually ignored when calculating the exposed surface temperature because convection is responsible for less than 10% of the heat transfer at the exposed surface of the concrete members [8]. On the other hand, convection is usually accounted for when calculating the unexposed surface temperature. The internal heat transfer through concrete members is typically calculated by conduction only [9]. To study the performance of reinforced concrete slab under fire, the distribution of temperature inside the slab has to be known. The assessment of the slab behavior under fire should start by applying the standard fire temperature on the exposed surface, after that prediction of the temperature through the slab is obtained. This prediction is performed using a heat transfer analysis. The heat transfer analysis was performed using finite difference method. In such analysis, the temperature distribution mainly depends on the thermal properties of materials such as thermal conductivity, emissivity, specific heat and convection heat transfer coefficient.

2.1.1. Fire temperature

The fire temperature was calculated assuming that the slab was exposed to uniform fire from below (tension side). The fire temperature followed the ISO834 standard fire without cooling phase (phase 1) and the ISO834 parametric fire with cooling phase (phase 2). The time–temperature relationship for phase 1 and phase 2 could be described by the following expressions given by [11] and as shown in Fig. 1:
Phase 1: temperature increasing stage only as:

\[ T_j = T_a + 345 \log_{10}(8t + 1) \]  

(1)

Phase 2: follows Eq. (1) up to the cooling point and after that follows the decreasing temperature stage as:

\[ T_j = T_h - 10.417(t - t_h) \quad (t_h \leq 30) \]  

(2)

\[ T_j = T_h - 4.1673(3 - t_h/60)(t - t_h) \quad (30 < t_h < 120) \]  

(3)

\[ T_j = T_h - 4.167(t - t_h) \quad (t_h \geq 120) \]  

(4)

where \( t \) is the fire exposure time (min); \( t_h \) the time of maximum temperature or cooling phase time (min); \( T_h \) the maximum temperature (°C); \( T_a \) the initial temperature (°C) and \( T_j \) is the fire temperature (°C).

2.1.2. Calculation of temperature distribution through concrete slab

The calculation of temperature distribution through the concrete slab thickness is calculated based on one dimensional heat transfer as given by Lie and Leir [23]. In such calculation, the cross section of slab is divided into layers. The thickness of each layer is \( x \) and the number of layers into which the slab is divided is \( M \). Each layer is represented by a point \( T_m \) see Fig. 2. The temperature in each layer is assumed to be uniform and equal to that of the representative point. The bottom of the slab is exposed to the fire and the top of the slab can cool in ambient temperature. To calculate the temperature history, a heat transfer equation is written for each layer for the time \( j \Delta t \) where \( j = 0, 1, 2 \) and \( \Delta t \) is the appropriate time increment.

Using these equations, the temperature of each layer can be successively evaluated for any time \( t = (j + 1) \Delta t \) if the temperature at time \( j \Delta t \) is known and moisture effect is neglected. To ensure that the error existing in the solution at any instant will not be amplified in subsequent calculations, stability criterion must be satisfied. For a given value of \( \Delta x \), a limit is set for the maximum value of \( \Delta t \). For fire-exposed slabs made of concrete the criterion is given as

\[ \Delta t \leq \frac{\rho C_p(x) \Delta x^2}{2(k(x) + \Delta x h_{max})} \]  

(5)

where \( \Delta t \) is the suitable time increment (min); \( \rho C_p(x) \) the minimum volumetric specific heat (specific heat × density); \( \Delta x \) the Layer thickness (m); \( k(x) \) the maximum thermal conductivity of concrete; and \( h_{max} \) is the maximum value of the coefficient of heat transfer at the fire-exposed surface and is given by:

\[ h_{max} = 4\sigma e(T_{max} + 273)^3 \]  

(6)

where \( T_{max} \) is the maximum fire temperature; \( \varepsilon \) the emissivity of the concrete surface = 0.9; and \( \sigma \) the Stefan–Boltzmann radiation constant = 5.669 × 10⁻⁸ W/m² K⁴.

2.1.2.1. Exposed surface temperature. For calculating the exposed surface temperature at any time step, heat transfer by radiation and conduction was commonly considered. The increase in the exposed surface temperature is calculated from the sum of the temperature rise by radiation and the temperature dropped by conduction. The increase in the exposed surface temperature was calculated by the following equation:

\[ T_{1}^{i+1} = T_{1}^{i} + \left( \frac{\Delta t}{\rho C_p(T_{1}) \Delta x} \right) \left\{ 2[\sigma e(T_f + 273)^4 - (T_{1}^{i} + 273)^4] \right\} 

- \left[ T_{1}^{i} - T_{2}^{i} (k_{1} + k_{2}) \right] \} \]  

(7)

where \( T_{1} \) is the temperature at the exposed surface, \( T_f \) the fire temperature, \( \varepsilon \) the emissivity of the fire = 1.0, \( \rho C_p(T_{1}) \) the volumetric specific heat of the exposed surface, and \( k_{1}, k_{2} \) is the thermal conductivity of the exposed surface and above layer of concrete respectively.

2.1.2.2. Interior slab temperature. Heat was transferred through the slab by conduction between layers only. The increase in layer temperature is calculated from the sum of the temperature rise by conduction from previous layer and the temperature drop by conduction from the next layer. The increase in the interior layer temperature was calculated by the following equation:

\[ T_{m}^{i+1} = T_{m}^{i} + \left( \frac{\Delta t}{\rho C_p(T_{m}) \Delta x} \right) \left[ (T_{m-1}^{i} - T_{m}^{i})(k_{m-1} + k_{m}) \right. 

- \left. (T_{m}^{i} - T_{m+1}^{i})(k_{m} + k_{m+1}) \right] \} \]  

(8)

2.1.2.3. Unexposed surface temperature. For calculating the unexposed surface temperature at any time steps, heat transfer by radiation, convection and conduction was considered. The increase in the unexposed surface temperature is calculated from the sum of the temperature rise by conduction and the temperature drop by air radiation and convection. The increase in the unexposed surface temperature was calculated by the following equation:
\[ T_{f}^{M+1} = T_{f}^{M} + \left( \frac{\Delta t}{pC_{p}k_{M}} \right) \left\{ \left[ T_{M+1}^{M} - T_{M}^{M} \right] - k_{M} \left\{ \left[ T_{M+1}^{M} - T_{M}^{M} \right] - T_{M+1}^{M} - T_{M}^{M} \right\} \right\} \]

\[ -2\left( \alpha T_{a}^{2} + \left( T_{a} + 273 \right)^{4} + \left( \gamma \left( T_{f}^{M} - T_{a} \right) \right) \right) \]  

where \( T_{a} \) is the ambient temperature and equal to 20 °C and \( \gamma \) is the convection heat transfer coefficient from horizontal slab surface to air. It generally equals to 2.49 Wm\(^{-2}\)K\(^{-1}\).

2.1.3. Steel temperature

Steel reinforcement was not specifically considered in the thermal analysis because it does not significantly influence the temperature distribution [16]. Moreover, measurements at various locations during fire tests showed that the differences in the bar and sections are small [26]. Thus, the steel reinforcement temperature was considered equal to the concrete temperature at the location of the steel reinforcement bars [26].

2.2. Verification of heat transfer model

The numerical and experimental data given by Lie and Leir [23] were used to verify the accuracy of the heat transfer model. The slab thickness was 100 mm and the slab temperature distributions were considered as given from Eqs. (5)–(9). The aggregate used was of siliceous type and the slab was exposed to ASTM E119 fire.

The ASTM E119 fire used is given by the following expressions:

\[ T_{f} = T_{a} + 0.555 \left[ 1044 \tan(h(0.00023413t)) + 498.2 \tan(h(0.00027044t)) + 1286 \tan(h(0.00024757t)) \right] \]  

For \( t < 7200 \) s

\[ T_{f} = 927 + 0.011574t \]  

For \( t \geq 7200 \) s

The high fire emissivity value was \( (\varepsilon = 1) \). The concrete slab thermal properties were considered as given by Lie and Leir data [23]. The slab was divided into 4 layers. Using the stability criterion given by Eq. (5), it was found that the maximum time increment was 30 s to check the slab fire resistance up to three hours. The results were compared with numerical and experimental data obtained from Lie and Leir [23] as shown in Fig. 3. It was found that the numerical and experimental temperature distributions given by Lie and Leir [23] match well with the proposed model.

2.3. Material properties

2.3.1. Thermal properties

The steel thermal properties were neglected, however, the steel reinforcement temperature was considered equal to the concrete temperature around the steel. Also, the concrete slab was assumed to be made of siliceous aggregate. The siliceous aggregate thermal properties were obtained from Eurocode2-2004 [19]. Such thermal properties are:

- The concrete density was 2400 kg/m\(^3\).
- The thermal conductivity was calculated using the following equations:

\[ k = 1.36 - 0.136 \frac{T}{100} + 0.0057 \left( \frac{T}{100} \right)^{2} \text{ W/mK for } 20 ^\circ C \leq T \leq 1200 ^\circ C \]  

- The specific heat may be determined from the following:

\[ C_{p} = 900 \text{ J/kg K for } 20 ^\circ C \leq T \leq 100 ^\circ C \]  

\[ C_{p} = 900 + \frac{T}{100} \text{ J/kg K for } 100 ^\circ C < T \leq 200 ^\circ C \]  

\[ C_{p} = 1000 + \frac{T - 200}{2} \text{ J/kg K for } 200 ^\circ C < T \leq 400 ^\circ C \]  

\[ C_{p} = 1100 \text{ J/kg K for } 400 ^\circ C < T \leq 1200 ^\circ C \]  

- The emissivity of the surface was 0.9.

2.3.2. Mechanical properties

The concrete slab was assumed to have a characteristic compressive strength of 25 N/m\(^2\) and the reinforcing steel was assumed to have yield stress of 360 N/m\(^2\). The variation of concrete strength at elevated temperature was accounted for by considering a reduction factor for siliceous aggregate concrete \( k_{c} \), which was given by Eurocode2-2004 [19] and defined as follows:

\[ k_{c} = 1 \text{ for } 20 ^\circ C \leq T \leq 100 ^\circ C \]  

\[ k_{c} = 0.95 - 0.05 \frac{T - 200}{100} \text{ for } 100 ^\circ C \leq T \leq 200 ^\circ C \]  

\[ k_{c} = 0.75 - 0.2 \frac{T - 400}{200} \text{ for } 200 ^\circ C \leq T \leq 400 ^\circ C \]  

\[ k_{c} = 0.15 - 0.6 \frac{T - 800}{400} \text{ for } 400 ^\circ C \leq T \leq 800 ^\circ C \]  

\[ k_{c} = 0 \text{ for } 800 ^\circ C \leq T \]
\[ k_c = 0.08 - 0.07 \frac{T - 900}{100} \quad 800 \text{ °C} \leq T \leq 900 \text{ °C} \]  
(21)

\[ k_c = 0.04 - 0.04 \frac{T - 1000}{100} \quad 900 \text{ °C} \leq T \leq 1000 \text{ °C} \]  
(22)

\[ k_c = 0.01 - 0.03 \frac{T - 1100}{100} \quad 1000 \text{ °C} \leq T \leq 1100 \text{ °C} \]  
(23)

\[ k_c = 0.1 \frac{1200 - T}{100} \quad 1100 \text{ °C} \leq T \leq 1200 \text{ °C} \]  
(24)

Also, Eurocode2-2004 [19] proposed a steel strength reduction factor with the temperature increase. The variation of the reduction factor for the tensile reinforcement with time can be obtained as shown in Fig. 5. Once the reinforcement temperature increases rapidly as the fire temperature increases because the steel is located at the lower part of slab section. The steel strength decreases with the increase in the temperature which lead to a decrease in the compression force. The compression block was divided into layers to study the variation in the stress block height with temperature. The concrete and steel strength reductions were calculated using the layer temperature shown in Fig. 4.

The height of the stress block is calculated using the equilibrium equation between the compression force given by concrete and the tensile force given by steel reinforcement. The equation of equilibrium can be written as:

\[ C_1 + C_2 + C_3 + \ldots + C_n + \ldots + C_N = T_s \]  
(31)

where

\[ C_n = \frac{0.67f_{yc}k_{cn}0.5\Delta X}{1.5} \]  
(32)

\[ T_s = \frac{A_s f_y}{1.15} \]  
(33)

The moment capacity of the slab section is then calculated as:

\[ M_{\text{welded}} = T_s(d - a/2) \]  
(34)

where \( k_{cn} \) is the concrete strength reduction factor at \( n \)th layer, \( f_{yc} \) the compressive strength, \( A_s \) the area of steel, \( k_s \) the reinforcement steel strength reduction factor, \( f_y \) the steel yield strength; \( T_s \) the tensile force, \( a \) the total height of compression stress block, and \( d \) is the part of last layer contribute to the compression stress block.

For a given slab provided with flexural reinforcement and subjected to ISO834 Standard fire using heat transfer equations and both of Eqs. (31)-(33), the moment capacity degradation with time can be obtained as shown in Fig. 5. Once the

**Figure 4** Heat transfer model and stress distribution.

<table>
<thead>
<tr>
<th>Ethan</th>
<th>0.5aX</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>compression force</th>
</tr>
</thead>
<tbody>
<tr>
<td>T_m</td>
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<td></td>
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<tr>
<td>T_1</td>
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<td>T_2</td>
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<td>T_3</td>
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<tr>
<td>T_s</td>
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<td></td>
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<td></td>
<td>tension force</td>
</tr>
</tbody>
</table>

**Figure 4** Heat transfer model and stress distribution.
moment capacity of the slab subjected to fire degrades to a value equal to the applied moment, the slab fails and the time corresponding to this moment capacity is the fire resistance.

3. Slab geometry, loads, design and parametric study

The study was conducted on sixteen simply supported one-way reinforced concrete slabs of span 3.0 m and thickness of 120 mm. The live load considered was 10 kN/m² and 6 kN/m² representing full live load (LL) and 60% of live load (0.6 LL), respectively. Four concrete cover thicknesses were considered namely; 15, 20, 25, 30 mm. According to the ECP 203-2007 design code and based on $f_{cu} = 25 \text{ N/mm}^2$, $f_y = 360 \text{ N/mm}^2$, and under full live load, the required areas of steel reinforcement were 850, 911.8, 972 and 1059.9 mm² respectively. Also, the presence of 20 mm plaster thickness at the exposed surface was considered. Fig. 6 and Table 1 show the combinations of the studied parameters. Based on the stability criterion given by Eq. (5), it was found that dividing the slab thickness into a number of layers greater than 16 would be practically acceptable [22]. The suitable number of layers in the current parametric study was 22 layers which simplifies the calculation of temperature at the location of the steel reinforcement. The heat transfer through the concrete slab without plaster was calculated based on 120 mm slab thickness and the layer thickness was 5.5 mm. It is to be noted that the thermal properties of plaster was considered similar to those of concrete material. Thus, the heat transfer through concrete slab with plaster was calculated based on 140 mm overall thickness and the layer thickness was 6.4 mm. Fig. 6 shows the slab layers for both cases. Moreover, the stability criterion given by Eq. (5) was used to find the suitable time increment. It was found that the suitable time increment is 5 s to check the fire resistance for eight hours for all concrete slab cases.

4. Fire resistance of slab (phase 1)

Figs. 7 and 8 show the temperature distribution for exposed and unexposed surfaces of concrete slabs as well as steel temperature for different concrete cover thickness in the case of slabs without plastering and with plastering respectively. It is clear from the figures that the temperature on the exposed surface increased rapidly during the initial stages however temperature on the unexposed surface rises after the first hour. Figs. 9 and 10 show the relationships between moment capacity degradation and time in the case of slabs without plastering and with plastering respectively. Fig. 11 shows the variation of fire resistance with different concrete cover thickness. Fig. 12 shows the effect of the presence of plastering on temperature distribution of concrete slab surfaces. Figs. 13 and 14 show the effect of the presence of plastering on the moment capacity degradation for the cases of 30 mm and 15 mm concrete cover respectively. Fig. 15 shows the effect of the live load ratio on the moment capacity degradation for the cases of 30 mm and 15 mm concrete cover. Table 2 gives the calculated fire resistance for the sixteen studied slabs.

4.1. Effect of concrete cover thickness

It was found that the concrete cover thickness mainly affect the temperature of the steel reinforcement. The steel reinforcement in a slab having 30 mm concrete cover was away from exposed surface than that in other slabs having less concrete cover thickness. Therefore, as the concrete cover thickness increased the steel reinforcement temperature decreased. Such concrete cover protected the steel reinforcement from rising temperature as shown in Figs. 7 and 8. Fig. 11 shows the variation
in the fire resistance for slabs having different concrete cover thicknesses. It is clear from the table and the figure that generally as the concrete cover thickness increased the fire resistance increased. Furthermore, examining Table 2 along with Fig. 11, the following can be observed: (i) for slabs without plaster in the case of live load ratio 0.6, increasing the concrete cover thickness from 15 to 30 mm resulted in an increase in the fire resistance from 65.37 to 143.75 min which represents an increase of 120%. However for similar concrete slabs but subjected to full live load, increasing the concrete cover thickness from 15 to 30 mm resulted in an increase in the fire resistance from 51 to 113.25 min which represents an increase of 122%; (ii) for slabs having 20 mm plaster and in the case of live load ratio 0.6 it was observed that increasing the concrete cover thickness from 15 to 30 mm resulted in an increase in the fire resistance from 171.62 to 289.75 min which represents an increase of 69%. However for similar concrete slabs but subjected to full live load increasing the concrete cover thickness from 15 to 30 mm resulted in an increase in the fire resistance from 136 to 227.25 min which represents an increase of 67%. It can be concluded herein that the concrete cover thickness is one of the most important parameters that affects the fire resistance. Increasing the concrete cover thickness led to almost linear increase in the fire resistance. However, such increase in the fire resistance for slabs without plaster was greater than that in the case of slabs having 20 mm plaster. Moreover, the percentage of increase in the fire resistance as a result of increasing the concrete cover thickness is almost not influenced by the variation in the live load ratio.

### 4.2. Effect of plaster

The effect of the presence of plaster on the temperature distribution was studied by considering the case of no plaster and 20 mm thickness of plaster. It was observed from the temperature distribution through concrete slab that the exposed surface, unexposed surface and steel reinforcement temperature in case of 20 mm plaster thickness were lower than those of the slabs without plaster. This is because the plaster played a role similar to that of the concrete cover in protecting the steel reinforcement. Such effect was also sounder at the unexposed surface but with less significant difference, as shown in Figs. 7, 8 and 12. The presence of plaster increased the fire resistance of the concrete slab. Fig. 13 shows that the fire resistance of reinforced concrete slab for case of 0.6 live load, with concrete cover 30 mm and without plaster (slab S4) was 143.75 min,

### Table 1: Study parameters.

<table>
<thead>
<tr>
<th>Slab</th>
<th>Plaster (mm)</th>
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<th>Cover (mm)</th>
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<tbody>
<tr>
<td>S1</td>
<td>None</td>
<td>0.6L.L</td>
<td>15</td>
</tr>
<tr>
<td>S2</td>
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<td>20</td>
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<tr>
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<td>None</td>
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<tr>
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<tr>
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Figure 7 Temperature distribution for concrete slab surfaces and steel without plaster.

Figure 8 Temperature distribution for concrete slab surfaces and steel (with plaster).

Figure 9 Moment capacity degradation for different concrete cover (without plaster).
however, for similar concrete slab but with 20 mm plaster (slab $S_{12}$) the fire resistance was 289.70 min. Fig. 14 shows that the fire resistance of the reinforced concrete slab for case of 0.6 live load, with concrete cover 15 mm and without plaster (slab $S_1$) was 65.37 min However, for similar concrete slab but with 20 mm plaster (slab $S_0$) the fire resistance was 171.62 min.
Table 2 also shows the effect of plaster on the fire resistance of concrete slabs for different concrete cover values and live load ratios. For slabs subjected to 0.6 live load it was observed that the presence of plaster increased the fire resistance by 106 min which represents an increase of 162% in the case of 15 mm cover and by 146 min which represents an increase of 102% in the case of 30 mm cover. However, for slabs subjected to full live load, it was observed that the presence of plaster increased the fire resistance by 85 min which represents an increase of 167% in the case of 15 mm cover and by 114 min which represents a decrease of 27% in the case of 30 mm cover. Therefore, the effect of plaster is more pronounced for slabs with thin concrete cover than slabs with thick concrete cover. The reason for that is attributed to the ratio of plaster thickness to the concrete cover thickness which varied from 133% in the case of 15 mm cover to 67% in the case of 30 mm cover.

4.3. Effect of live load ratio

Two live load ratios were considered in this study namely; 0.6 and 1.0 representing 60% and 100% of the LL. Fig. 15 and Table 2 show the effect of live load ratio on the fire resistance of concrete slabs for different concrete cover values and plaster cases. It is clear from the table that the fire resistance of slabs having 30 mm concrete cover and without plaster was 143.75 and 113.2 min under live load ratios of 0.6 and 1.0 respectively. However, the fire resistance of slabs with 15 mm concrete cover and without plaster was 65.37 and 51 min under live load ratios of 0.6 and 1.0 respectively. For slabs without plaster, it was observed that increasing the live load ratio from 0.6 to 1.0 decreased the fire resistance by 14 min which represents a decrease of 28% in the case of 15 mm cover and by 31 min which represents a decrease of 28% in the case of 30 mm cover. However, for slabs having 20 mm plaster, it was observed that increasing the live load ratio from 0.6 to 1.0 decreased the fire resistance by 36 min which represents a decrease of 27% in the case of 15 mm cover and by 62 min which represents a decrease of 27% in the case of 30 mm cover. Thus, increasing the live load ratio lead to a decrease in the fire resistance of the concrete slabs. It can be concluded that the live load ratio have a significant effect on the fire resistance of slabs. As the live load ratio increased, the fire resistance decreased and such decrease is almost constant regardless of the cover thickness value and the presence of plaster.

5. Study of cooling time (phase 2)

The study of cooling phase aims to investigate the effect of starting cooling before the fire resistance with main objective to detect the critical time of cooling and the maximum risk time. Such critical time is the minimum time at which if the slab is cooled at that time or after and even such time is still less than the fire resistance of slab, there is still a possibility for such slab to fail. The maximum risk time is the time span between the critical time of cooling and the corresponding expected failure time. The importance of finding the maximum risk time is that such time is important for staying observant for possible impending failure after the start of the cooling phase. To achieve that, the ISO834 parametric fire with cooling phase was used (phase 2). To detect the critical time, three different starting times for cooling before fire resistance were used within a process of trial and error. To illustrate such process, slab S1 is considered as an example. The fire resistance of slab S1 is 65.37 min. Three arbitrary times of 2, 4 and 6 min before the fire resistance were considered. It means that cooling (decaying) starts at 63.37, 61.37 and 59.37 min. Using heat transfer equations the distributions of the temperature for

<table>
<thead>
<tr>
<th>Slab</th>
<th>Plaster (mm)</th>
<th>Live load ratio</th>
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<th>Fire resistance (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>No</td>
<td>0.6L.L</td>
<td>15</td>
<td>65.37</td>
</tr>
<tr>
<td>S2</td>
<td>No</td>
<td>0.6L.L</td>
<td>20</td>
<td>87.75</td>
</tr>
<tr>
<td>S3</td>
<td>No</td>
<td>0.6L.L</td>
<td>25</td>
<td>114.25</td>
</tr>
<tr>
<td>S4</td>
<td>No</td>
<td>0.6L.L</td>
<td>30</td>
<td>143.75</td>
</tr>
<tr>
<td>S5</td>
<td>No</td>
<td>L.L</td>
<td>15</td>
<td>51.00</td>
</tr>
<tr>
<td>S6</td>
<td>No</td>
<td>L.L</td>
<td>20</td>
<td>68.62</td>
</tr>
<tr>
<td>S7</td>
<td>No</td>
<td>L.L</td>
<td>25</td>
<td>89.62</td>
</tr>
<tr>
<td>S8</td>
<td>No</td>
<td>L.L</td>
<td>30</td>
<td>113.25</td>
</tr>
<tr>
<td>S9</td>
<td>20</td>
<td>0.6L.L</td>
<td>15</td>
<td>171.62</td>
</tr>
<tr>
<td>S10</td>
<td>20</td>
<td>0.6L.L</td>
<td>20</td>
<td>206.75</td>
</tr>
<tr>
<td>S11</td>
<td>20</td>
<td>0.6L.L</td>
<td>25</td>
<td>244.50</td>
</tr>
<tr>
<td>S12</td>
<td>20</td>
<td>0.6L.L</td>
<td>30</td>
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</tr>
<tr>
<td>S13</td>
<td>20</td>
<td>L.L</td>
<td>15</td>
<td>136.00</td>
</tr>
<tr>
<td>S14</td>
<td>20</td>
<td>L.L</td>
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<tr>
<td>S15</td>
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<tr>
<td>S16</td>
<td>20</td>
<td>L.L</td>
<td>30</td>
<td>227.25</td>
</tr>
</tbody>
</table>
concrete surfaces and steel were obtained at those times. Fig. 16 shows such distributions including mid-depth position of slab at 63.37 min. It is clear from the figure that, at a particular time; the temperature of the exposed surface became higher than the fire temperature. Also, it could be noted that as the fire temperature continued to drop, the mid-depth temperature became higher than the exposed surface temperature. Also, it is noted that the start of temperature drop, due to fire cooling phase, at any concrete depth was lagging behind the start of the fire cooling phase. It means that the maximum temperature inside the concrete slab did not take place at the same time of the maximum temperature of the fire. Using thermal analysis the strength degradation in the steel and concrete can be obtained. When the reinforced concrete slab exposed to fire with decay phase, the steel strength decreased with the increase in the temperature this lead to a decrease in the tension force and consequently the compression force. Therefore, the stress block height began to decrease with the increase in temperature but at a particular time increment in cooling phase stage the stress block height returned to increase. This lead to a decrease in the flexural strength of the slab section up to a certain time and then the flexural strength of slab started to increase again.

5.1. Critical time for cooling and maximum risk time

The following are the steps to find the critical time for cooling and the maximum risk time:

5.1.1. First step

The moment capacity degradation under fire using decay phase starting at 63.3, 61.3 and 59.3 min was obtained as given in Fig. 17. It is shown from the figure that the applied moment corresponding to 0.6 LL is 11.68 kN m. It is clear from the figure that the concrete slab collapsed under fire with decay phase starting at 63.3 and 61.3 min, but the concrete slab did not collapse under fire with decay phase at 59.3 min. Also, the moment capacity of the section showed more degradation for a certain time after cooling time before ascending. This is attributed to the increase in the temperature inside the section just after cooling time. The minimum moment capacities of the section corresponding to time 61.3 and 59.3 min are 11.4 and 11.82 kN m respectively as given from Fig. 17.

5.1.2. Second step

Using linear interpolation along with the applied moment corresponding to 0.6 LL which is 11.68 kN m, the critical time for cooling is obtained from the intersection which is 59.99 min as given in Fig. 18. Now such time is called the critical time for cooling the slab S1. Also, the critical net time is then defined as the difference between the critical time for cooling and fire resistance of the slab, i.e., the critical net time is 5.31 min. If the slab is cooled before such time it will not be collapsed however, if it is cooled after that time it will collapse.

5.1.3. Third step

Now if the slab is cooled at the critical time 59.99 min the slab is predicted to collapse and the corresponding failure time can be calculated. The failure time corresponding to starting cooling of 63.3 and 61.3 min is 65.7 and 67.1 min as obtained from Fig. 17. Using linear extrapolation as shown in Fig. 19 the failure time corresponding to the critical time of cooling is obtained as 68.01 min. The maximum risk time is then calculated as the time span between the critical time for cooling and the expected failure time which is 8.02 min.

5.2. Effect of studied parameters on the maximum risk time

The maximum risk time is the maximum time span between the critical time of cooling and the collapse of slab. The concrete slabs are still prone to collapse even when, they were extinguished before their design fire resistance. For example; the fire resistance for slab S1 was 65.3 min, however, it was still subjected to failure if it was extinguished 5.3 min before its fire resistance (critical decay phase starting at 59.99 min), and the expected failure at 68.01 min. Table 3 gives the fire resistance, the critical time for cooling, critical net time, failure time corresponding to critical cooling time and the maximum risk time.

5.2.1. Effect of concrete cover thickness on the maximum risk time

Table 3 shows the variation of maximum risk time corresponding to different concrete cover thickness for all cases. It is clear from the table that, in general, as the concrete cover thickness increased the maximum risk time increased. Also, examining

Figure 16  Temperature distributions through concrete slab S1 at 63.3 min decay phase time.
Table 3 the following can be detected: (i) for slabs without plaster and subjected to live load ratio of 0.6, it was observed that increasing the concrete cover thickness from 15 to 30 mm increased the maximum risk time from 8.02 to 26.86 min which represents an increase of 235%. However, for similar concrete slabs but subjected to full live load, increasing the concrete cover thickness from 15 to 30 mm increased the maximum risk time from 8.02 to 24.90 min which represents an increase of 210%; (ii) for slabs had 20 mm plaster and subjected to live load ratio of 0.6, it was observed that increasing the concrete cover thickness from 15 to 30 mm increased the maximum risk time from 32.05 to 45.2 min which represents an increase of 41%. However for similar concrete slabs but subjected to full live load, increasing the concrete cover thickness from 15 to 30 mm increased the maximum risk time from 31.8 to 48.72 min which represents an increase of 53%. It can be concluded herein that the concrete cover thickness had significant effect on the maximum risk time. Increasing the concrete cover thickness led to almost linear increase in the maximum risk time. However, such increase as a percentage of increase in the maximum risk time for slabs with no plaster was greater than that for slabs with 20 mm plaster.

5.2.2. Effect of plaster on the maximum risk time

Table 3 shows the effect of the presence of the plaster on the maximum risk time of slabs. It is clear from the table that the presence of plaster increased the maximum risk time of the concrete slabs. The maximum risk time of reinforced concrete slab for the case of 0.6 live load, with concrete cover 30 mm and without plaster as given by slab S4 was 26.86 min, however, for similar concrete slab but, with 20 mm plaster as given by slab S12, the maximum risk time was 45.21 min. Table 3 shows also that the maximum risk time of the reinforced concrete slab for the case of 0.6 live load, with concrete cover 15 mm and without plaster as given by slab S1, was 8.02 min, however, for similar concrete slab but, with 20 mm plaster as given by slab S9, the maximum risk time was 32.05 min. Moreover, Table 3 shows the effect of plaster on the variation of maximum risk time of the concrete slabs for different concrete cover values and live load ratios. For slabs subjected to 0.6 live load, it was observed that the presence of the plaster increased the maximum risk time by 24.03 min which represents an increase of 300%, in the case of 15 mm cover and by 18.35 min which represents an increase of 68%, in the case of 30 mm cover. However, for slabs subjected to full live load, it was observed that the presence of plaster increased the maximum risk time by 23.78 min which represents an increase of 296%, in the case of 15 mm cover and by 23.82 min which represents an increase of 95%, in the case of 30 mm cover. It can be concluded that the presence of plaster...
increased the maximum risk time; however, the effect of the plaster decreased with increasing the concrete cover thickness.

### 5.2.3. Effect of live load ratio on the maximum risk time

Table 3 shows the effect of live load ratio on the maximum risk time of concrete slabs for different concrete cover values and plaster cases. It is clear from the table that the maximum risk time of slabs having 30 mm concrete cover and without plaster was 26.86 and 24.9 min as given by S4 and S9 under live load ratios of 0.6 and 1.0 respectively. However, the maximum risk time of slabs having 15 mm concrete cover and without plaster was 8.20 and 8.02 min as given by S1 and S5 under live load ratios of 0.6 and 1.0 respectively. The presence of plaster increased the fire resistance. Increasing the concrete cover thickness from 15 to 30 mm resulted in an increase in the fire resistance with 120% in the case of no plaster and 69% in the case of plaster with thickness 20 mm as observed in the case of 0.6 live load. Moreover, the percentage of increase in the fire resistance with the increase in the concrete cover thickness is almost not affected by the variation in the live load ratio.

### 6. Conclusions

The behavior of one-way reinforced concrete slabs exposed to fire was investigated using a numerical finite difference analysis. Firstly, the fire resistance of slabs was obtained using the ISO834 standard fire without cooling phase. Secondly, the ISO834 parametric fire with cooling phase was applied to study the effect of cooling time. The critical time for cooling was identified and the corresponding expected failure time was calculated. The maximum risk time was obtained. Sixteen simply supported one-way reinforced concrete slab models were considered to study three different parameters namely: the concrete cover thickness; the presence of plaster at the exposed surface; and the live load ratio. Heat transfer equations through the concrete slab thickness were considered for the fire resistance calculations. From the current study, the following conclusions could be drawn:

- The steel reinforcement temperature played an important role in the strength degradation of the slabs. As the steel reinforcement temperature increased, the yield strength of the steel reinforcement decreased and accordingly the capacity of the section decreased.
- The concrete cover thickness had a significant effect on the fire resistance. Increasing the concrete cover thickness led to almost a linear increase in the fire resistance. However, such increase in the fire resistance for slabs with no plaster was greater than that for slabs with 20 mm plaster. Increasing the concrete cover thickness from 15 to 30 mm resulted in an increase in the fire resistance with 120% in the case of no plaster and 69% in the case of plaster with thickness 20 mm as observed in the case of 0.6 live load. Moreover, the percentage of increase in the fire resistance with the increase in the concrete cover thickness is almost not affected by the variation in the live load ratio.
- The presence of plaster significantly increased the fire resistance of slabs. This is because the plaster played a role similar to that of the concrete cover in protecting the steel reinforcement. However, the effect of the presence of the plaster is more pronounced for slabs with thin concrete cover compared to slabs with thick concrete cover. The presence of plaster increased the fire resistance by 162% in the case of 15 mm cover and by 102% in the case of 30 mm cover as observed in the case of 0.6 live load.
- The live load ratio has a significant effect on the fire resistance of slabs. As the live load ratio increased, the fire resistance decreased and such decrease is almost constant in all cases with about 28%.
- Slabs are still prone to collapse even if they were cooled before their fire resistance. As the fire temperature continued to drop, the mid depth temperature became higher than the exposed surface temperature. The start of temperature drop, due to fire cooling phase, at any concrete depth was lagging behind the start of the fire cooling phase. Accordingly, the maximum temperature inside the concrete slab did not take place at the same time of the maximum temperature of the fire.
• The concrete cover thickness had significant effect on the maximum risk time. Increasing the concrete cover thickness led to almost a linear increase in the maximum risk time. However, such increase in the maximum risk time for slabs with no plaster was greater than that for slabs with 20 mm plaster. Increasing the concrete cover thickness from 15 to 30 mm, in the case of no plaster, increased the maximum risk time from 8.02 to 26.86 min which represents an increase of 235%, however, in the case of plaster increased the maximum risk time from 32.05 to 45.2 min which represents an increase of 41% as observed in the case of 0.6 live load.

• The presence of plaster significantly increased the maximum risk time. However, such increase is higher in the case of small concrete cover than that in the case of large concrete cover. The plaster increased the maximum risk time by 300%, in the case of 15 mm cover and by 68% in the case of 30 mm cover as observed in the case of 0.6 live load.

• The variation in the live load ratio has almost no effect on the maximum risk time values. The variation in the live load ratio from 0.6 to 1.0 gives almost the same maximum risk time.

References


