Effect of Spalling Depth on the Fire Behaviour Simulation of High-Volume Fly Ash Nano-Silica Reinforced Concrete Slab at Various Fire Curves

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Abstract

Concrete spalling is characterised by a repetitive breakaway of concrete elements exposed to elevated temperatures. Due to its dense microstructure, high-strength concrete tends to spall and lose its strength. This paper presents the thermal behaviour simulation of a high-strength high-volume fly ash nanosilica (HSHVFANS) reinforced concrete (RC) slab exposed to various fire curves (HC, RABT-ZTV, and RWS) using finite element (FE) analysis. Two FE models, namely No Spalling (NS) and Maximum Spalling (MS) were validated with the fire test of the HSHVFANS RC slab exposed to ISO 834 fire curve for 120 min. The validated FE models were used to predict the effect of spalling depth at different fire curves. The temperature profile shows that the NS FE model remains inside the experimental results, while the MS FE model showed an increase in the temperature ranging from 680 °C up to 840 °C. The reinforcement temperature for the NS FE model remained below 300 °C until 120 min, while the MS FE model was higher than 300 °C at 72 min (ISO 834), 55 min (HC), 48 min (RABT-ZTV), and 46 min (RWS). It has been found that the fire-resistant period of the HSHVFANS RC slab was significantly affected at maximum spalling depth.

Keywords: high-strength high-volume fly ash nanosilica concrete, finite element simulation, spalling depth, fire curves, temperature distribution

1 INTRODUCTION

Concrete spalling is a natural impair that occurs in high strength concrete (HSC), characterised by a sudden and repetitive breaking of surface layers exposed to high and rapid exothermal activity like fire (Dwaikat & Kodur, 2010; Lottman et al., 2017; So, 2016). Due to its dense microstructure, HSC tends to spall in fire, and the strength loss is more obvious after higher temperature. The fire-induced concrete spalling is due to a build-up of pore pressure, thermal stresses and combined high pore pressure and thermal stress mechanisms (Liu et al., 2018). In the event of a fire, the temperature, especially in a tunnel, rises exceptionally quickly. Spalling has a damaging effect on fire-exposed concrete, destroying the entire cross-section of the tunnel (Hertz, 2003). The anchorage capacity of exposed reinforcement will be affected whilst reducing the load-bearing capacity of a structure substantially.

The use of HSC is essential to create a resilient infrastructure and sustainable construction industry. The
HSC offers significant economic, architectural, and structural advantages over normal-strength concrete (NSC) (Afroughsabet & Ozbakkaloglu, 2015; Aitcin & Flatt, 2015; Kodur & Phan, 2007; So, 2016). HSC has been used in the construction of buildings, offshore structures, tunnels, and bridge elements. High strength concrete with fly ash and nano silica (HSCFANS) has been proven to be a fire resistance material at high temperatures up to 700 °C (Ibrahim, 2013). Furnace test on the 150 mm HSCFANS concrete cube at 700 °C shows no spalling on the concrete surface. The use of fly ash (FA), a by-product of thermal electric plants as a partial replacement of Portland cement, is highly recommended due to its beneficial effects on concrete behaviour and low cost. The use of FA also will reduce the massive production of Portland cement resulted in significant growth in environmental pollution. The study on the effects of using FA on concrete’s thermal properties shows that the high quantities of FA slightly decreased the thermal expansion of concrete and provided higher relative compressive strength.

Figure 1 shows the standard time-temperature fire curves developed to simulate various types of fire with different heating rates, peak temperatures, and heating period (EFNARC, 2006; Ingason, 2008; Maraveas & Vrakas, 2014). The Rijkswaterstaat (RWS), hydrocarbon (HC), modified hydrocarbon (HCM), and RABT-ZTV fire curves were developed based on medium-scale fire tests conducted and associated with tunnel fires involving petrochemical sources including fuel, oil, and petrol tankers. In tunnel lining design, it is important to determine the critical temperature of concrete and steel reinforcement during the fire to ensure that the structure has sufficient resistance to high temperature. The critical temperature at which the strength of concrete and the functional capacity begins to deteriorate was determined to be between 430 °C and 660 °C (Fletcher et al., 2007), 300 °C and 600 °C (Majorana et al., 2010), and the bearing capacity of steel reinforcement begin to decrease at 300°C (Majorana et al., 2010; Yan et al., 2012).

The finite element (FE) simulation can be used to predict the behaviour of HSC exposed to various fire curves. The software will facilitate the calculation of complex structures whilst saving time (Dwaikat & Kodur, 2010). A combination of physicochemical parameters such as concrete strength, concrete density, moisture content, heating rate and fire profiles contribute to concrete spalling. In a study by Bo (2011) and Deeny et al. (2008), a 2D heat transfer analysis with spalling effect is performed on the beam and slab cross-section (Deeny et al., 2008; Shunan Bo, 2011). The onset of spalling is triggered when the bottom surface temperature reached 375 °C to 425 °C. Concrete spalling is modelled by removing all the elements making up the bottom concrete cover. The analysis is continued, and the temperature distribution for the reduced cross-section is calculated. However, in a study by Maraveas & Vrakas (2014) and Materazzi & Breccolotti (2004), a 2D heat transfer analysis is performed without spalling effect on the beam’s cross-section slab (Maraveas & Vrakas, 2014; Materazzi & Breccolotti, 2004). The influence of the fire curves on the developed temperatures in an unprotected reinforced concrete (RC) section is examined, and the analytical evaluation of the fire endurance on the concrete is refined. From the studies, it is observed that spalling has threatened the stability of the reinforcement structure. The FE simulation result is compared with the guideline and previous studies to determine the conformity and compliance of the concrete at high temperature. World Road Association (PIARC) and International Tunnelling Association (ITA) have recommended the design criteria to be used in the tunnel according to the type of structure and traffic as shown in Table 1 (Eskesen et al., 2004; Ingason et al., 2015). This guideline distinguishes according to the type of traffic according to the possible fire load and the consequences of a structural failure due to a fire. Table 2 shows the previous experimental studies on concrete exposed to high temperature. The constraints considered in the tests are the type of fire curve, type of material used, the loading factor, and the compressive strength. Based on the studies, it is observed that the result can be divided into no spalling and spalling concrete.

This paper presents thermal behaviour simulation of HSCFANS RC slab exposed to various fire curves using FE analysis. For validation, this paper compares the simulation and test results from the recently conducted fire test of the HSCFANS RC slab exposed to ISO 834 fire curve for 120 min. Afterwards, the validated FE models were used to predict the effect of spalling depth at different fire curves such as HC, RABT-ZTV and RWS.

2 RESEARCH METHOD
2.1 Experimental Test
Details of the fire tests are provided in Radzi (2016) and Mussa et al. (2021) (Mussa et al., 2021; Radzi, 2016). The concrete mix design was followed ACI 211.4R-93 (ACI Committee 211, 1998) for compressive strength, fcu of 60 MPa with a water/binder ratio of 0.29. The binder was composed of 52.5 % FA, 45.0 % cement and 2.5 % nano-silica. A superplasticizer (1 % of the total quantity of binder) was added to improve the workability. The polypropylene (PP) fibre added was 1 % of the volume of concrete. The slump test gives the value of slump between 25 mm and 50 mm (true slump). The materials were
Portland cement, FA class F, nano-silica (type Cembinder 8), superplasticizer (brand Grace Darex Super 20). The cement fulfilled the requirements of ASTM C150/C150M-12 (ASTM C150/C150M, 2019) and the maximum size of gravel equivalent to 10 mm. The proportion of concrete mixture was listed in Table 3. The compressive strength of the concrete at 28 days and 90 days was 62.7 MPa and 74.4 MPa, respectively.

A medium-scale slab concrete specimen (1850 mm × 1700 mm × 200 mm) was fabricated following the European Federation of National Associations Representing Concrete (EFNARC, 2006). The reinforcement bar with a diameter of 12 mm and spacing of 100 mm × 200 mm was arranged based on the actual design of pre-cast concrete used in the Brenner Base Tunnel, Italy-Austria (Eskesen et al., 2004). Four K thermocouples (∅3.0 mm) were placed at 30 mm, 60 mm, 90 mm and 200 mm from the bottom surface of the concrete slab. The thermocouples were installed during casting to ensure high coupling efficiency, avoiding any air space and providing complete physical contact between the thermocouple and the concrete. The fire test was conducted using a medium-scale furnace (1.5 m × 1.5 m × 1.5 m) with a maximum temperature of 1057 °C based on the ISO 834 fire curve (ISO 834-1, 1999). The concrete slab was placed horizontally on one side on the opening of

### Table 1 Fire resistant period for tunnel (Eskesen et al., 2004; Ingason et al., 2015)

<table>
<thead>
<tr>
<th>Traffic Type</th>
<th>Main Structure</th>
<th>Secondary Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cars / vans</td>
<td>ISO 60 min</td>
<td>ISO 60 min</td>
</tr>
<tr>
<td>Trucks / tankers</td>
<td>RWS/ HC 120 min</td>
<td>RWS/ HC 120 min</td>
</tr>
<tr>
<td></td>
<td>RWS/ HC 120 min</td>
<td>RWS/ HC 120 min</td>
</tr>
<tr>
<td></td>
<td>RWS/ HC 120 min</td>
<td>RWS/ HC 120 min</td>
</tr>
</tbody>
</table>

### Table 2 Previous experimental study on concrete exposed to high temperature

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Fire Curves</th>
<th>Materials</th>
<th>Loadings</th>
<th>Compressive Strength (MPa)</th>
<th>Maximum Temperature (°C)</th>
<th>Depth of Spalling (mm)</th>
<th>Researchers</th>
</tr>
</thead>
<tbody>
<tr>
<td>No spalling concrete</td>
<td>ISO 834 Not reported</td>
<td>With load</td>
<td>64.5</td>
<td>1057</td>
<td>None</td>
<td>(Xu et al., 2015)</td>
<td></td>
</tr>
<tr>
<td>Spalling concrete</td>
<td>ISO 834 OPC, PP fiber With load</td>
<td>35</td>
<td>1057</td>
<td>Average 20 mm</td>
<td>(Chung et al., 2013)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RWS OPC</td>
<td>Without load</td>
<td>50</td>
<td>1100</td>
<td>Average 20 mm</td>
<td>(Caner &amp; Bönco, 2009)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISO 834 OPC, FA</td>
<td>With load</td>
<td>55</td>
<td>1057</td>
<td>30 mm to 110 mm Average 20 mm</td>
<td>(Choi &amp; Shin, 2011)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISO 834 OPC</td>
<td>With load</td>
<td>35</td>
<td>1057</td>
<td>Average 20 mm</td>
<td>(Chung et al., 2013)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISO 834 OPC, FA, GGBS</td>
<td>With load</td>
<td>60.5</td>
<td>1057</td>
<td>26 mm to 51 mm</td>
<td>(Yan et al., 2012)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RWS OPC, PP fiber</td>
<td>Without load</td>
<td>40.5</td>
<td>1350</td>
<td>36 mm</td>
<td>(Yoshitake et al., 2005)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 3 Concrete mixing proportions (kg/m³)

<table>
<thead>
<tr>
<th>Cement</th>
<th>FA</th>
<th>Nanosilica</th>
<th>Water</th>
<th>Gravel</th>
<th>Sand</th>
<th>PP Fiber</th>
<th>Superplasticizer</th>
</tr>
</thead>
</table>
| Portland cement, FA class F, nano-silica (type Cembinder 8), superplasticizer (brand Grace Darex Super 20). The cement fulfilled the requirements of ASTM C150/C150M-12 (ASTM C150/C150M, 2019) and the maximum size of gravel equivalent to 10 mm. The proportion of concrete mixture was listed in Table 3. The compressive strength of the concrete at 28 days and 90 days was 62.7 MPa and 74.4 MPa, respectively.

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the furnace at the top. A heating duration of 120 min was used in the test to simulate one possible fire scenario in tunnels based on the guideline of EFNARC (2006) (EFNARC, 2006). During the fire test, thermal distribution in HSHVFANS concrete was recorded via data logger software.

2.2 FE Simulation

A 2D heat transfer analysis on the cross-section of the RC slab was performed using ABAQUS/CAE structural analysis modelling tool version 6.14. The fundamental assumption in this simulation is that the spalling will occur. Thus, it is assumed that the material and geometric conditions which trigger spalling are satisfied. The temperature in the RC slab will rise evenly from the heated surface, and the load temperature of the fire curves will follow the selected fire amplitude. Several factors such as material properties, depth of spalling, period of spalling, and type of fire curves were accounted for in the FE simulation.

The case study for this FE simulation is based on the experimental test where the spalling of the HSHVFANS RC slab begins when the temperature at the bottom surface reach the range of 680 °C to 780 °C. The spalling measured at the thermocouple location was 5 mm, and the maximum spalling measured was 23 mm. In this FE simulation, the spalling was modelled by removing the elements making up the bottom concrete cover. The analysis on the reduced cross-section was continued, and the temperature distribution until 120 min was obtained. The FE model has used the same thickness of 200 mm while the width was reduced to 300 mm considering the position of two steel reinforcement, a similar method by Bo (2011) and Deeny et al. (2008) (Deeny et al., 2008; Shunan Bo, 2011). The reinforcement was located at 60 mm from the bottom surface of the slab. The reduction in width does not affect the result because heat exposure in the bottom slab is uniform at every location. The boundary conditions of the RC slab are adiabatic on the perpendicular sides (left and right sides of the model), and the initial temperature is 28 °C. Convection and radiation are the boundary conditions on the top and bottom surfaces.

The FE models were divided into five parts, as shown in Figure 2. The five parts consist of Concrete 1 with a thickness of 30 mm, Concrete 2 with a thickness of 30 mm, Concrete 3 with a thickness of 30 mm, Concrete 4 with a thickness of 110 mm and the 10 mm diameter reinforcement. Each connection is set as the position of the thermocouple at 30 mm, 60 mm, 90 mm and 200 mm. For the control analysis, the time period and increment are set based on the time and temperature of fire curves. The total time is 7200 s with 120 min of heating. The amplitude of the temperature fire curves is established for the purpose of loading. The type of interaction selected is ‘constraints’, and the bond selected is ‘tie’ to allows the flow of temperature between the two parts. The size of the meshing selected was 15 mm. The material properties used in the FE simulation shown in Table 4. The material properties from experimental tests were the density of concrete and reinforcement, Young’s modulus of concrete and RC, and the thermal conductivity of concrete. The material properties assumed from other studies are the specific heat of concrete and reinforcement and the thermal conductivity of reinforcement. Bentz et al. (2011) used the specific heat of concrete due to the 45% similarity of the composition of HSHVFANS concrete (Bentz et al., 2011). The thermal conductivity and specific heat of the reinforcememt by Naus (2005) were used due to the similarity in size, type and properties to this current study (Naus, 2005).

![Figure 2 FE model: (a) part of model, and (b) meshing and loading](image-url)
reinforcement to remain below the critical temperature of 300 °C was analysed based on the guidelines from PIARC and ITA (Eskesen et al., 2004; Ingason et al., 2015).

3 RESULTS AND DISCUSSIONS

3.1 Experimental Behaviour

The experimental result (Mussa et al., 2021; Radzi, 2016) is shown in Figure 3. The temperature distribution shows that, at 30 mm depth, the temperature was increased from 28 °C to 174 °C for 33 min, and the final temperature at 120 min is 523 °C. At 60 mm depth, the temperature showed constant behaviour from 35 min to 65 min and at temperature ranged of 90 °C to 120 °C before increased to the final temperature at 283 °C. At 90 mm depth, the temperature showed a similar trend with a constant temperature of 120 °C during the 30 min to 70 min. After 73 min, the temperature started to increase slightly until the final temperature reached 211°C. The constant behaviour during the temperature ranged from 90 °C to 120 °C, indicating evaporation of the Calcium Silicate Hydrate (CSH) gel's free water and chemically bonded water. The spalling depth ranged from 5 mm to 10 mm was considered minor, and spalling depth more than 10 mm, was quite damaging with a total spalling coverage area of 34.3 %. A maximum depth of 23 mm spalling covered an insignificant small area. At the thermocouple location, the spalling measured was 5 mm in depth.

3.2 Simulated Behaviour

Figure 4 shows the temperature distribution comparison between simulation results to experimental results. It was observed that the temperature distribution result for the experimental test is higher than the simulation at the onset of the test due to the process of evaporation of free water and chemically bonded water of the CSH gel, similar to the observation reported by Kodur & Phan (2007), Yan et al. (2012), Naus (2005), Hou et al. (2013), and Yan et al. (2013) which is not taken into account in the modelling (Hou et al., 2013; Kodur & Phan, 2007; Naus, 2005; Yan et al., 2012, 2013). At 30 mm depth, the temperature distributions for both methods are similar from 35 min to 109 min. The final temperature difference between 110 min to 120 min was recorded as 32 °C. At 60 mm depth, the temperature distribution of simulation increased after 60 min and remained higher until 120 min with a final temperature difference of 28 °C. Similar observations were observed at a depth of 90 mm when the temperature distribution of the experimental test was higher than the simulation between time 10 min to 70 min. The temperature distribution of simulation increased after 60 min and remained higher than experimental until 120 min with a final temperature difference of 28 °C. Similar observations were observed at a depth of 90 mm when the temperature distribution of the experimental test was higher than the simulation between time 10 min to 70 min. The temperature distribution of simulation increased after 71 min and remained high until 120 min with a final temperature difference of 25 °C. There was a slight difference in temperature distribution at a depth of 200 mm between both methods, with the final temperature difference recorded at 34 °C.
evaporation of free water and chemically bonded water of the CSH of the concrete slab that occurred between 25 min to 71 min. This process was the limitation of the FE simulation. However, the temperature distributions for both methods in the last 50 min of heating showed a constant difference between 25 °C to 34 °C. The percentage differences at the maximum temperature of the ISO 834 fire curve (at 1057 °C) ranged between 2.37 % to 3.22 % at all depths. With this slight difference, the FE model could predict the effects of different spalling depths of the same ISO 834 loads and other tunnel fire curves (HC, RABT-ZTV, and RWS) loads on the behaviour of the HSHVFANS RC slab under those fire loads.

3.3 Simulation of Temperature Distribution Under Various Tunnel Fire Curves Loads

Table 6 shows the simulated temperature-time graphs and temperature profiles of the NS FE model under other tunnel fire curve loads. For the ISO 834 fire curve, the final temperature recorded at 30 mm, 60 mm, 90 mm, and 200 mm depths for 120 min were 559 °C, 236 °C, 92 °C and 29 °C, respectively. Consequently, the final temperature recorded at a depth of 30 mm, 60 mm, 90 mm, and 200 mm at 120 min were 623 °C, 275 °C, 107 °C and 29 °C, respectively under the HC fire curve loading. For the RABT-ZTV fire curve, the final temperature recorded at 30 mm, 60 mm, 90 mm, and 200 mm depths for 120 min were 545 °C, 282 °C, 114 °C and 29 °C, respectively. The temperature of reinforcement at 60 mm depth for the ISO 834, HC, and RABT-ZTV was below the critical temperature of 300 °C until 120 min. However, exposure to the higher temperature of the RWS fire curve has shown a different result. The temperature of reinforcement at 60 mm depth was below 300 °C for 110 min. The temperature exceeded the critical temperature for 120 min with a difference of 25 °C. The final temperature recorded at 30 mm, 60 mm, 90 mm, and 200 mm depths for 120 min were 729 °C, 325 °C, 123 °C and 29 °C, respectively.

Figure 5 to Figure 7 show the comparison of FE simulation results with other scholars. To determine the consistency of the FE simulation results, a 25 % maximum difference between the laboratories result is considered. Figure 5 shows the comparison of temperature distribution at a depth of 30 mm. For ISO 834 fire curve, the temperature distribution of FE simulation was found to be low compared to Xu et al. (2015), Chung et al. (2013), and Choi & Shin (2011) (Choi & Shin, 2011; Chung et al., 2013; Xu et al., 2015). However, at 96 min to 120 min, the temperature distribution of FE simulation is higher than Choi & Shin (2011) (Choi & Shin, 2011). Figure 6 and Figure 7 show the comparison of temperature distribution at a depth of 60 mm and 90 mm. The temperature distribution of FE simulation for all types of fire curve (ISO 834, HC, RABT-ZTV, and RWS) was lower than Xu et al. (2015), Chung et al. (2013), Choi & Shin (2011), Dorgarten (2004), and Yoshitake et al. (2005) (Choi & Shin, 2011; Chung et al., 2013; Dorgarten, 2004; Xu et al., 2015; Yoshitake et al., 2005). The difference in experimental results to the FE simulation results was higher than 25 % because of
different depth of concrete spalling, lower strength concrete used, and shorter period of experimental test. In addition, the effect of the process of evaporation of free water and chemically bonded water of the CSH of the concrete slab was not considered in the FE simulation.

3.4 Effect of Maximum Spalling Depth on Simulation Under Various Tunnel Fire Curves

Table 7 shows the temperature-time graph and temperature profile of the MS FE model (23 mm spalling) exposed to the tunnel fire load. For the ISO 834 fire curve, the spalling occurs between 9 min to 18 min. The final temperature recorded at a depth of 30 mm, 60 mm, 90 mm, and 200 mm at 120 min is 948 °C, 456 °C, 181 °C and 30 °C, respectively. It was observed that the temperature of the reinforcement at a depth of 60 mm is below 300 °C until 72 min. After that, the temperature exceeded the critical temperature until 120 min with a difference of 156 °C. For the HC fire curve, the spalling...
occurs between 4 min to 5 min. The final temperature recorded at a depth of 30 mm, 60 mm, 90 mm, and 200 mm at 120 min is 1004 °C, 522 °C, 218 °C and 30 °C, respectively. It was observed that the temperature of the reinforcement at a depth of 60 mm is below 300 °C until 55 min. After that, the temperature exceeded the critical temperature until 120 min with a difference of 222 °C.

For the RABT-ZTV fire curve, the spalling occurs at 3 min. The final temperatures recorded at a depth of 30 mm, 60 mm, 90 mm, and 200 mm at 120 min are 1123 °C, 615 °C, 258 °C and 31 °C, respectively. It was observed that the temperature of the reinforcement at a depth of 60 mm is below 300 °C until 46 min. After that, the temperature exceeded the critical temperature until 120 min with a difference of 315 °C.

Table 8 and Figure 4 show the comparison of temperature distribution from the experimental and simulation results (5 mm concrete spalling and 23 mm
maximum spalling). Overall, the temperature distribution for simulation with 23 mm spalling is larger than the experimental test and simulation with 5 mm spalling at a depth of 30 mm and 60 mm from the heated surface. This behaviour occurred because the concrete without enough concrete cover is directly exposed to high temperature. However, the temperature distribution difference at 90 mm to 200 mm from the heated surface is smaller because of the distance factor.

### 3.5 Fire Resistant Period

Table 9 shows the FE simulation on the temperature of reinforcement at 60 mm depth and critical temperature of 300 °C. For the NS FE model, the temperature of reinforcement that had reached the critical temperature of 300 °C was found to simulate the RWS fire curve. For the 23 mm spalling FE model, the temperature of reinforcement had reached the critical temperature of 300 °C in the simulation of all various fire curve loadings. The
fastest time to reach the critical temperature (at 46 min) is in the RWS fire curve load simulation.

The guidelines by PIARC and ITA for the fire resistance period of the ISO 834 fire curve is at least 60 min for the main and the secondary structure of the car and van usage (Table 1). For the truck and tanker trucks, the requirements for the fire resistance period of the HC and RWS fire curve is at least 120 min. As for the ISO 834 fire curve, the NS FE model and 23 mm spalling FE model exceeded the fire-resistant period of the critical temperature 300 °C for 120 min and 72 min, respectively. This result complies with the fire resistance period requirement of the fire curve ISO 834. For the HC and RWS fire curve, the FE simulation results show that the NS FE model complies with the fire resistance period requirement for 120 min. However, the 23 mm spalling FE model resisted the fire for a duration of 46 to 55 min, lesser than the stated period in the guideline.

4 CONCLUSIONS

In this paper, a comprehensive 2D FE simulation was conducted to determine the effect of spalling in HSHVFANS RC slab exposed to various tunnel fire curves. Based on the presented test results, the following conclusions can be drawn:

a. In the experimental test, minor spalling with a small insignificant area of maximum depth of 23 mm and total coverage area of 34.3 % was observed on the heated surface of the concrete slab during the time of 10 min to 30 min and temperature range of 680 °C to 840 °C. The experimental result can be classified into two categories as material-related factors and heating characteristics.

b. The temperature distribution of the FE simulation result varies slightly with the experimental due to not considering the evaporation of free water and chemically bonded water of the CSH process of the concrete slab in the modelling that occurred between 25 min to 71 min.

c. The difference of temperature distribution difference for experiment and simulation exposed to ISO 834 fire curve is between 25 °C to 34 °C (2.37 % to 3.22 %) from the maximum temperature 1057 °C.

d. The temperature distributions of FE simulation for all types of fire curve (ISO 834, HC, RABT-ZTV and RWS) were found to be low than other scholars due to the difference of concrete spalling depth, lower strength of concrete, a shorter period of experimental test, and the effect of chemical reaction of the concrete that was limited in the simulation.

e. For the NS FE model, the reinforcement temperature had reached the critical temperature of 300 °C for the simulation with the RWS fire curve. For the 23 mm spalling FE model, that the temperature of reinforcement reached the critical temperature of 300 °C for all simulations.

f. For the ISO 834 fire curve, the fire-resistant period for the NS FE model and 23 mm spalling FE model is 120 min and 72 min, respectively. For the HC and RWS fire curve, the fire-resistant period for the NS FE model was 120 min, while the 23 mm spalling FE model resisted the fire for a duration of 46 min to 55 min.

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