WARSAW UNIVERSITY OF TECHNOLOGY Index 351733
FACULTY OF CIVIL ENGINEERING
COMMITTEE FOR CIVIL AND WATER ENGINEERING
POLISH ACADEMY OF SCIENCES ISSN 1230-2945

DOI: 10.24425/ace.2022.140632

ARCHIVES OF CIVIL ENGINEERING

Vol. LXVIII ISSUE 2 2022

pp. 105 –123

Research paper

Fire resistance of FRP strengthening concrete beams at elevated temperature using ABAQUS

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Abstract: Mechanical properties of FRP such as strength and stiffness as well as the bonding interface between FRP and concrete will be badly deteriorated when exposed to high temperature. Furthermore, the effect of thickness of insulation with different type of concrete strength has not yet been studied elsewhere in numerical studies. Therefore, this study is to assess the thermal-structural behaviour of insulated FRP strengthened RC beam exposed to elevated temperature using ABAQUS. The proposed numerical model of 200 ×300 mm RC beam subjected to 2 hours standard fire curve (ISO 834) had been validated with the analytical solution. The validated numerical model then is used in parametric study to investigate the behaviour of fire damaged normal strength concrete (40 MPa) and high strength concrete (60 MPa) of RC beam strengthened with CFRP using various fire insulation thickness of 12.5 mm, 25 mm and 40 mm, respectively. The result of steel characteristic strength reduction factor is compared with analytical using 500°C Isotherm methods. The parametric studies indicated that the fire insulation layer is essential to provide fire protection to the CFRP strengthened RC beams when exposed to elevated temperature. The insulation layer thickness of 25 mm had been found to be the optimum thickness to be used as it is able to meet the criteria of temperature distribution and displacement requirement. In conclusion, the numerical model developed using ABAQUS in this study is to carry

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out assessment on the thermal-structural behaviour of the insulated CFRP-strengthened RC beams at elevated temperature.

Keywords: ABAQUS, concrete, fire insulation, fibre reinforced polymer (FRP), fire resistance, high temperature

1. Introduction

Fibre reinforced polymer (FRP) had been used to strengthen the reinforced concrete (RC) beams due to its high strength to weight ratio (compared to steel plate bonding method), excellent corrosion resistance and light weight. The external bonded FRP sheets are also able to increase the flexural strength and shear capacity of RC beams.

Utilization of the FRP as structural strengthening material for RC beams at elevated temperature had drawn doubts among the structural engineers as they are concerned about FRP performance and thermal properties which may jeopardise its function to strengthen the RC beams. The FRP consists of two (2) main components, namely fibre and polymer matrix or bonding adhesive, which have a very low glass transition temperature (T_g). Generally, the glass transition temperature for common polymers and adhesives is between 60°C and 82°C. When exposed to temperatures beyond the glass transition temperature of the FRP, the polymer matrix from the FRP will undergo changes in mechanical properties, and its stiffness and bond strength severed [1, 2]. Therefore, when the FRP is used to strengthen the deteriorated RC beams, the study on thermal-structural behaviour of the combination of both materials when subjected to high temperature needs to be conducted to assess whether the FRP strengthened RC beams able to withstand its intended loading when exposed to fire [3–5]. One of the common practices to protect FRP sheets used in the strengthening of RC beams is by using an external coating layer of thermal resisting material as an insulation layer for fire protection to the FRP strengthened RC beams [6–8]. However, there are limited studies being carried out to determine the effectiveness on insulation layer provided to FRP strengthened RC beams.

Based on literature reviews, comprehensive method to investigate the thermal-structural behaviour of the FRP strengthening RC beams when exposed to fire is through the full-scale fire test to observe the real structure behaviour and material properties changes during the fire test which had carried out by researchers such as William et al. [9] and Ahmed et al. [10]. Nevertheless, such experimental tests were always very costly, time consuming and prone to human errors. Therefore, one of the alternative approaches is through numerical approach by using finite element modelling [11]. With the advancement in finite element modelling, RC beam model can be developed to observe response of the FRP strengthening RC beams at elevated temperature with different design parameters such as concrete strength, thickness of concrete cover, type of fire exposure and thickness of fire insulation layer [12, 13].

The need of thermal-structural behaviour for the FRP strengthening on RC elements using the numerical approach to evaluate the fire resistance of FRP in RC beams strengthening works at elevated temperature is highly demanding. The numerical study then is carried out to assess the effect of concrete grade and insulation layer thickness on the thermal-structural behaviour of CFRP strengthened RC beams. This paper highlighted the utilization of numerical modelling to assess the thermal-structural behaviour of insulated CFRP strengthened RC beams exposed to elevated temperature.

1.1. Use of Fibre Reinforced Polymer (FRP) in structural rehabilitation

FRP materials are composite materials which are used in construction industry, especially as strengthening materials for RC structures by structural engineers. There are few types of FRP that are generally used and include carbon fibre reinforced polymer (CFRP), aramid fibre reinforced polymer (AFRP) and glass fibre reinforced polymer (GFRP). FRP is a two-component composite material consisted of high strength fibre and polymer matrix. The fibre will provide the strength and stiffness for the FRP while the polymer matrix will protect the fibre from environment and external mechanical damage and transfer and distribute forces to the fibre [14]. Therefore, the polymer matrix plays the most important role in determining the fire-resistance of FRP systems.

Generally, if the temperature rises above the glass-transition temperature (T_g) of polymer matrix, which is typically between 60°C to 82°C, the elastic modulus of polymer matrix will be greatly reduced [15]. When subjected to temperature above the T_g and used without any insulation or fire protection, the stiffness and bond strength of the FRP will severed, leading to rapid changes in mechanical properties, which reduced the force transfer between the fibre and polymer matrix [1]. However, the fibre could practically continue to support the loading until it reached the fibre threshold temperature, which can be as low as 275°C for glass fibre and as high as 1000°C for carbon fibre [16]. Therefore, it is always recommended that a layer of insulation material is applied to the FRP-strengthened RC elements to satisfy the fire resistance requirements [10].

With the advancement in FRP technology, the use of FRP materials in structural application are massive and promising. FRP can be used either as internal reinforcement of RC structure or external of RC member using FRP plates, sheets, or wraps [14]. The exterior use of FRP on RC members generally involved the repair and rehabilitation of existing RC structures. The external application of FRP generally involved the external bonding of RC beams or slabs and wrapping of RC columns [17, 18]. The external bonded FRP will provide additional flexural or shear strength to the RC beams which act as supplement reinforcement to the internal reinforcing bar. In wrapping or confinement of RC columns, FRP to enhance the ductility of the RC columns [19]. These techniques of rehabilitation of RC structure had been found very effective and easy to install.

1.2. Fibre Reinforced Polymer (FRP) subjected to elevated temperature

Currently, there were limited studies to adopt the numerical modelling to evaluate the fire resistance and thermal-structural behaviour of FRP-strengthened RC members as well as to address the design of fire protection system for FRP-strengthened RC members at elevated temperature. Bisby et al. [20] and Chowdhury et al. [21] carried out numerical modelling to evaluate the FRP-confined circular and rectangular RC columns, respectively, using the heat transfer analysis models. The models had been developed by themselves using computer programme such as FORTRAN [21]. Hawileh et al. [22], Ahmed et al. [10] and Dai et al. [23] developed the thermal-structural model in developing the finite element (FE) numerical models investigate the behaviour of FRP-strengthened RC beams exposed to fire condition.

On the other hand, over the last 20 years, a number of experimental fire test studies have been conducted to investigate the thermal-structural behaviour of FRP-strengthened RC members subjected to fire loads [24–26]. One of the full scale fire tests was carried out by Blontrock et al. [27]. They conducted a study on ten (10) RC beams strengthened by CFRP subjected to different insulation system and exposed to ISO 834 standard fire. The fire test results indicated that the U-shape fire insulation system was more effective to prolong the fire exposure before degradation of bond strength and postponed the temperature increase of internal steel bar.

Available literature study showed that it is difficult to achieve certain fire resistance rating for un-insulated CFRP-strengthened beams as compared to the same beams with fire insulation layer, which led to better fire resistance. Although some numerical studies had identified the need of the insulation layer in protecting FRP strengthened RC beam, the effect of thickness of insulation with different type of concrete strength has not yet been studied elsewhere.

2. Finite element modelling

Numerical modeling of the heated RC beam begins with development of finite element model using ABAQUS for FRP strengthened RC beams protected by insulation layer and subjected to standard fire test and results will be validated against analytical solution [23, 28]. Then, parametric analysis of different grade of concretes and various insulation layer thickness on the thermal-structural behaviour of FRP strengthened RC beam when subjected to elevated temperature is conducted. Temperature of steel rebar and steel characteristic strength reduction factor of the FRP strengthened RC beam using analytical solution and numerical analysis. The computer modeling development are divided into three stages:

i. Stage 1: Development & Validation of Numerical Model
   There are two (2) types of structure fire analysis conducted in this paper, namely the heat transfer analysis and thermo-mechanical analysis. Temperature-time curve adopted is based on the ISO 834 for 2 hours as fire loading used in the finite element modelling. The fire load is applied at the soffit of beam with the use of fixed concrete cover adopted from the experimental setup for fire test [27]. The type of FRP used in this study is Carbon Fibre Reinforced Polymer (CFRP) with the all material temperature-dependent properties used in accordance to the study by Dai et al. [23] and recommended in BS EN 1992-1-2. The uninsulated beam is used to validate the numerical model. The analysis results from the numerical model such as

reinforcement temperature – fire exposure time (heat transfer analysis) and the beam deflection – fire exposure time (thermo-mechanical analysis) then are compared and verified against the analytical solution to validate the numerical model that had been developed.

ii. Stage 2: Parametric Study on Thermal-Structural Behaviour of FRP Strengthened RC Beam

In the parametric study, there are two (2) types of concrete grade and four (4) thickness of insulation layer to be used. The concrete grade of C40/50 and C60/75 with cylinder compressive strength of 40 MPa and 60 MPa, respectively, are used to represent the concrete grade that is normally used in the industry as normal strength concrete and high strength concrete. Meanwhile, the insulation layer thickness used in the parametric study are 0 mm, 12.5 mm, 25 mm and 40 mm. Thus, there are total of eight (8) sets of combination data being generated from the numerical model.

iii. Stage 3: Investigation on Temperature of Steel Rebar and Steel Characteristic Strength Reduction Factor

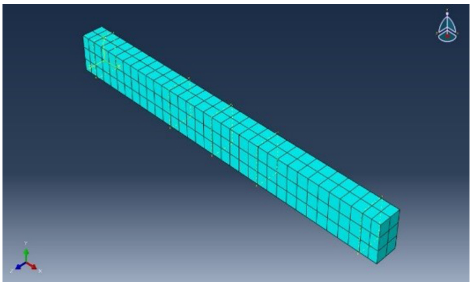
The analytical calculation is performed to determine the steel rebar temperature using the 500°C Isotherm Method. The calculated and numerical models predicted steel reinforcement temperature, then are used to determine the steel characteristic strength reduction factor (K_s) using equations from BS EN 1992-1-2. The comparison of calculated and numerical predicted result for steel rebar temperature and steel characteristic strength reduction factor (K_s) are analysed and discussed.

3. Results and discussion

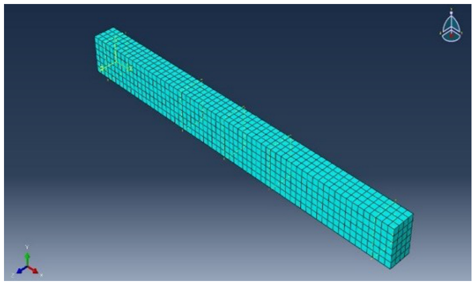
3.1. Validation of numerical model

3.1.1. Mesh convergence study

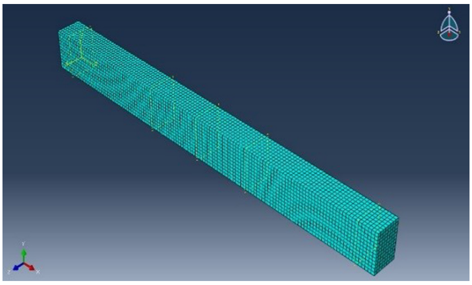
In Finite Element Analysis (FEA), the accuracy of the analysis results is determined by the appropriate size of mesh or element used in the modelling. It is very important to determine level of refinement required and use the suitable finite element size or mesh density to get accurate and adequate analysis simulation. Generally, there is no definite value for the optimum mesh size that can be used in finite element models as it depends on the shape of model, type of analysis, meshing techniques and other factors. This paper examined what is the suitable size of mesh (how small the elements) needed to ensure the results of the FEA are not affected by changing the size of mesh. If two (2) subsequent mesh refinements do not change the result substantially, then the results can be assumed to have converged. To determine the more suitable and optimum size of mesh to be used for the FEA model, a mesh convergence study had been carried out where selected mesh sizes range from 10 mm to 100 mm were used to determine the accuracy of FEA displacement results with the analytical calculation of displacement value as shown in Fig. 1 and Table 1. The computation time required for each of the mesh size is also being recorded.



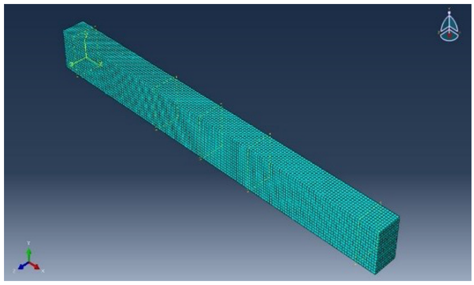
(a) Mesh size 100 mm



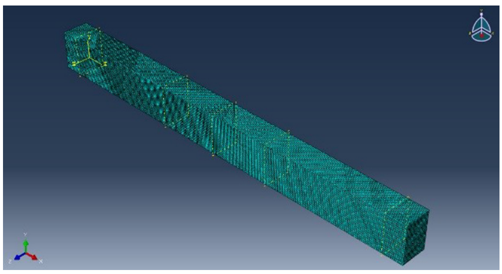
(b) Mesh size 50 mm



(c) Mesh size 25 mm



(d) Mesh size 20 mm



(e) Mesh size 10 mm

Fig. 1. FEA Model with difference mesh size

As shown from the results of the mesh convergence study in Table 1, the more suitable and optimum mesh size to be used for the FEA model is 25mm due to the following justifications:

1. For mesh size of 100 mm and 50 mm, the differences between FEA results of displacement with the analytical calculation are the smallest (between 2 to 5%). However, from Fig. 1, it is clearly shown that the mesh of 100 mm and 50 mm are coarse mesh and found not suitable to use in the FEA model.
2. Meanwhile, the model generated with mesh size of 20 mm and 10 mm are found to be very fine and take longer analysis computation time. The difference in pre-

dicted displacement values (from FEA result) are slightly higher with the analytical calculation when compared to model with mesh size of 25 mm.

3. For the model with mesh size of 25 mm, the mesh being found as fine mesh with total 12,096 elements. The estimated difference in displacement is about 7.6% compared with the analytical calculation and takes lesser computation time when compared to model with mesh size of 20 mm and 10 mm. There is also no significant difference in displacement when compared to mesh size of 20 mm and 25 mm. Due to these factors, the further mesh refinement from 25 mm had not changed the result substantially, therefore, result generated with mesh size of 25 mm has converged.

Table 1. Result Comparison for Mesh Convergence Study

Mesh (Global Size) (mm)	No. of Element	Result			Time Taken (s)
		Displacement $U_{2,max}$ (mm)	Analytical Calculation of Displacement (mm)	% Difference	
100	502	2.251	2.2076	1.97	49
50	2 248	2.095	2.2076	–5.10	48
25	12 096	2.040	2.2076	–7.59	64
20	25 722	2.036	2.2076	–7.77	85
10	193 044	2.032	2.2076	–7.95	425

3.1.2. Verification of thermal analysis

In thermal analysis, the numerical model is verified using results from the heat transfer analysis in ABAQUS with the analytical solution using 500°C Isotherm Method. The typical temperature contour in the beam cross section from ABAQUS is shown in Fig. 2.

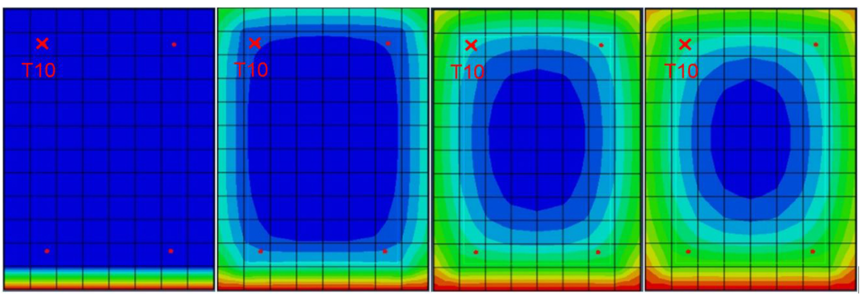


Fig. 2. Temperature contour at mid-span of beam cross section
(from left to right: 10 s, 1080 s, 3800 s and 7200 s)

The steel reinforcement (top left rebar of T10) temperature from the analytical solution and by the finite element (FE) prediction of numerical model then are plotted and compared as shown in the Fig. 3. From the figure, temperatures in the steel reinforcement measured from FE are in good agreement with the analytical calculated temperature for steel reinforcement. The numerical model generally predicted slightly lower temperature up to 90 minutes of fire exposure period. Meanwhile, the FE temperature prediction made by numerical model exhibited higher temperature in the steel reinforcement compared to the analytical calculated temperature of steel reinforcement after 90 minutes exposure to elevated temperature. The differences might be due to coefficient used in the 500°C Isotherm Method equations not describe real behaviour of the fire and material of the beam. However, the numerical prediction seems able to generate satisfactory heat transfer as per the analytical solution.

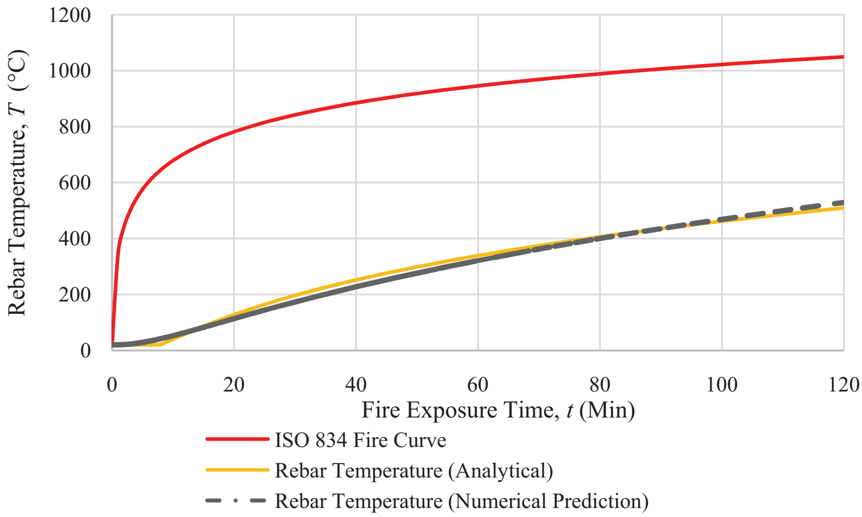


Fig. 3. Graph comparison calculated and FE predicted temperature distribution on steel reinforcement in beam

The steel reinforcement temperature data from Fig. 2 was then analysed and relative comparison of analytical solution with numerical model prediction is made and summarised as shown in Table 2. The result of relative comparison indicated that the average difference in analytical solution and prediction results is about 7.25% with standard deviation of 5.18%. The different of steel reinforcement temperature (in %) between analytical calculation

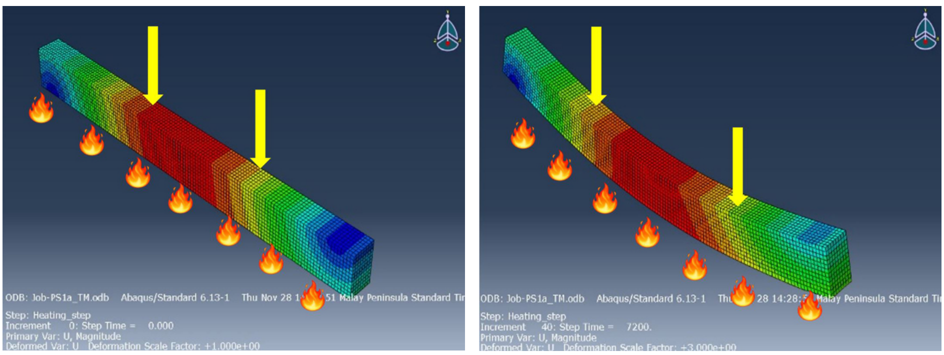
Table 2. Relative comparison of Analytical Solution with Numerical Model Prediction on Steel Reinforcement temperature

Analysis Parameter	Difference of Temperature (Prediction with Analytical), %		
	Average	Standard Deviation	Range
Steel Reinforcement T10	7.25	5.18	0.02–18.50

and numerical prediction from Fig. 2 is between 0.2% to 18.5%. Therefore, it can be overall assumed that the proposed numerical model is generally able to predict the steel reinforcement temperature with a good accuracy that match closely with the analytical solution.

3.1.3. Thermo-mechanical analysis of RC beam

For thermo-mechanical analysis, there are two (2) steps involved in the ABAQUS analysis. The first step involved applying the two (2) point loads to the numerical models of uninsulated RC beam at ambient temperature. In the second step, the time-temperature results obtained from the heat transfer analysis was imported to simulate the action of fire while the loading remained constant until end of fire exposure for 120 minutes or until failure of the beam. Fig. 4 showed the predicted deformed shape for the proposed numerical model of uninsulated RC beam which is before loading and after loading and exposure to elevated temperature for 120 minutes.



(a) (b)

Fig. 4. Deformed shape for the numerical model using ABAQUS: (a) Before loading; (b) After loading and exposure to elevated temperature

In the thermo-mechanical analysis, the verification of numerical model was performed by comparing the mid-span deflection of beam over the fire exposure time with the analytical solution as shown in Fig. 5. From Fig. 5, the comparison between the predictions by numerical model with the calculated mid-span deflection results indicated that proposed numerical model exhibited less accuracy with the analytical solution but still somewhat reasonable with the calculated mid-span deflection. It is believed that this might due to the difference in concrete temperature in numerical prediction with the analytical solution using 500°C Isotherm Method which lead to the differences in determining the concrete elastic modulus reduction used to calculate the mid-span deflection. Furthermore, the concrete plasticity input parameter used in the ABAQUS (concrete damaged plasticity) might also exhibit differences when compared to the deflection calculation being used in analytical solution.

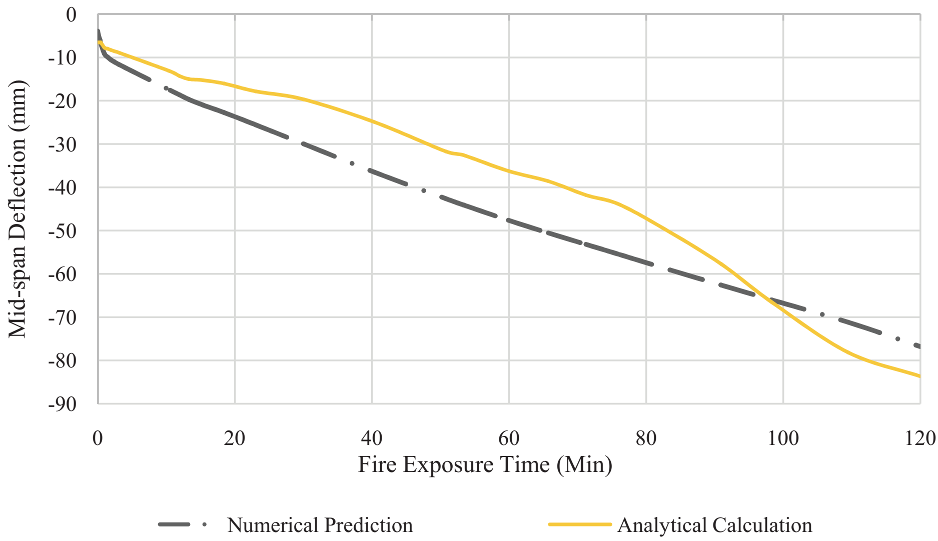


Fig. 5. Graph comparison of mid-span deflection for numerical model with analytical solution

The mid-span deflection for the uninsulated RC beam in Fig. 5 is then analysed and relative comparison of analytical solution with numerical model prediction is made and summarised as shown in Table 3. The result of relative comparison indicated that the average difference in calculated displacement with the predicted displacement is about 20.22% with standard deviation of 8.74%. Relative comparison had exhibited lowest difference of mid-span deflection between numerical prediction and analytical calculation for the uninsulated RC beam of 0.79% which showed very high accuracy. However, the largest difference in calculated and prediction mid-span deflection is about 34.37%, which is quite high but still acceptable due to the assumption used in the calculation of the concrete temperature and the effect of concrete plasticity from the numerical model.

Table 3. Relative comparison of Analytical Solution with Numerical Model Prediction on Steel Reinforcement temperature

Analysis Parameter	Difference of Temperature (Prediction with Analytical), %		
	Average	Standard Deviation	Range
Steel Reinforcement T10	7.25	5.18	0.02–18.50

Overall, it can be assumed that the proposed numerical model is generally able to predict the thermo-mechanical analysis with a satisfactory accuracy

3.2. Parametric analysis of varies thickness of insulation layer

The validated numerical model for RC beam then was used to undertake the parametric analysis to assess the effect of different grade of concrete with varies of insulation layer thickness on the thermal-structural behaviour of the CFPR strengthened RC beam when exposed to elevated temperature. The combination of input parameters used in parametric analysis are as shown in Table 4. The insulation thickness used in the study are range from 12.5 mm to 40 mm with the grade concrete of normal strength concrete (40 MPa) and high strength concrete (60 MPa).

Table 4. Parameters used in Parametric Analysis

No.	Beam Reference	Insulation Layer Thickness (mm)	Concrete Grade (Cylinder strength) MPa
1	PS1	0	40
2	PS2	0	60
3	PS3	12.5	40
4	PS4	12.5	60
5	PS5	25	40
6	PS6	25	60
7	PS7	40	40
8	PS8	40	60

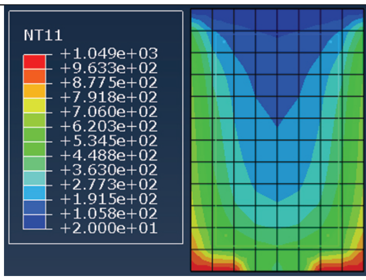
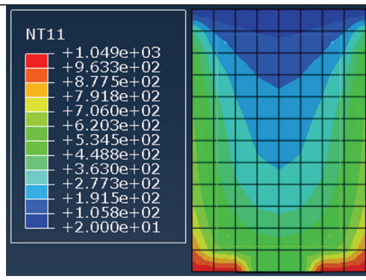
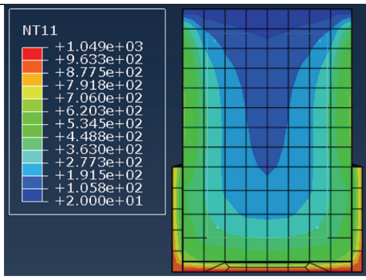
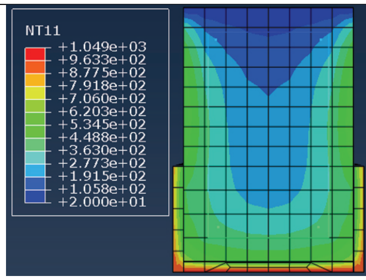
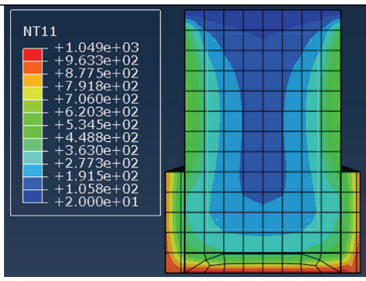
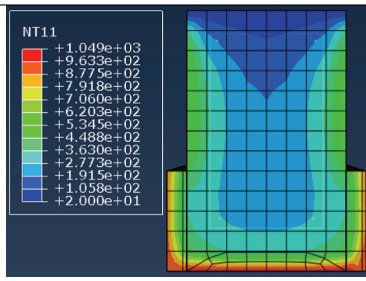
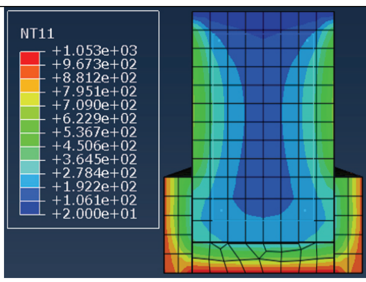
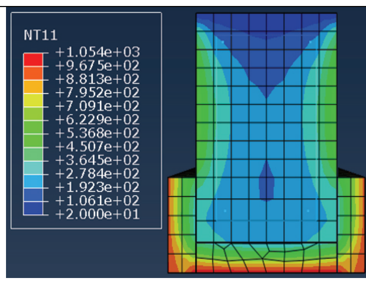
3.2.1. Effect of concrete strength

The temperature distribution (temperature contour) at the mid-span in beam cross sections after subjected to 2 hours heating for grade concrete C40/50 (normal strength concrete) and grade concrete C60/75 (high strength concrete) are shown in Table 5.

From Table 5, it is shown that the use of higher grade of concrete (60 MPa) generates higher and faster heat transfer as compared to lower grade of concrete (40 MPa) with the same insulation scheme for the CFRP strengthened RC beams. This might be due to the higher thermal conductivity in the high strength concrete as well as use of lower water/cement ratio and different binder in the high strength concrete.

The grade of concrete also influences the temperature distribution in the steel reinforcement of the insulated CFRP strengthened RC beams. From the graph shown in Fig. 6, the higher grade of concrete (60 MPa) exhibited higher thermal conductivity and transferred the heat to the steel reinforcement in a quicker manner. This is shown in Fig. 6 where the concrete grade 60 MPa had consistently exhibited higher temperature distribution in the steel reinforcement for all the insulation thickness (0 mm, 12.5 mm, 25 mm and 4 0 mm) when compared to concrete grade 40 MPa.

Table 5. Temperature Distribution at Mid-span of Beam Cross Section subjected to two 2 hours ISO 834 Fire (a) PS1, (b) PS2, (c) PS3, (d) PS4, (e) PS5, (f) PS6, (g) PS7 and (h) PS8

Insulation Thickness	Grade Concrete 40 MPa	Grade Concrete 60 MPa
0 mm	 (a)	 (b)
12.5 mm	 (c)	 (d)
25 mm	 (e)	 (f)
40 mm	 (g)	 (h)

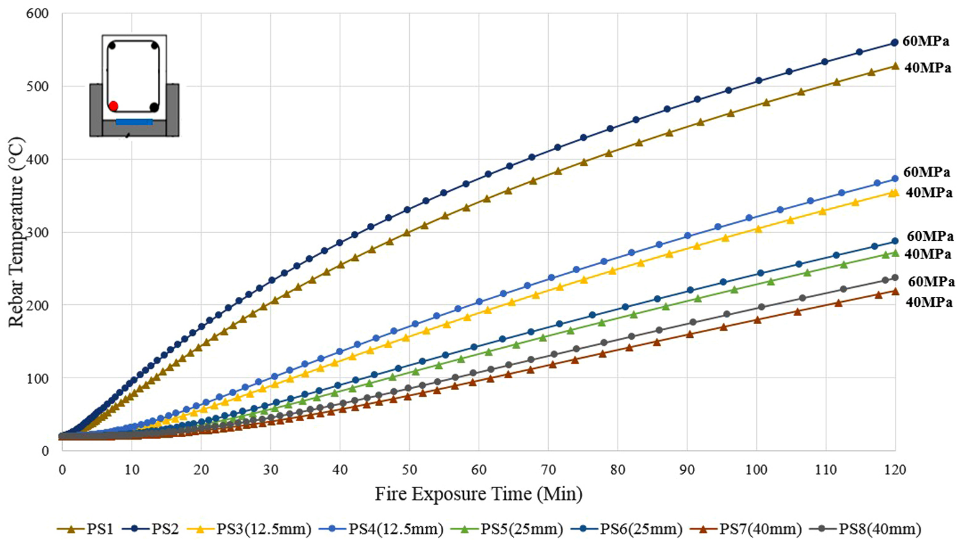


Fig. 6. Graph of Temperature Distribution in Steel Reinforcement

3.2.2. Effect of fire insulation thickness

The temperature distribution at the steel reinforcement for different fire insulation thickness is shown in Fig. 7a and Fig. 7b. It is indicated that whether using normal strength concrete (40 MPa) or high strength concrete (60 MPa), the thicker the fire insulation layer thickness, the greater the ability to resist higher temperature on steel reinforcement (lower

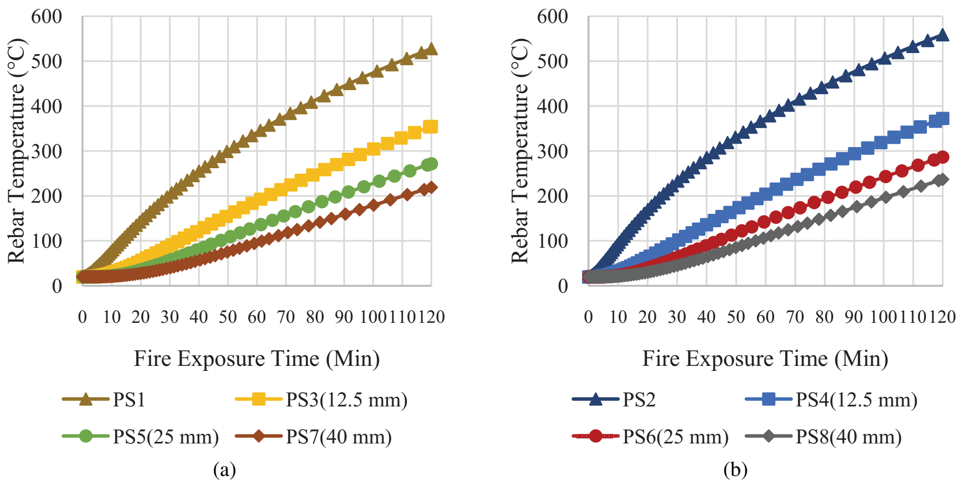


Fig. 7. Graph of Temperature Distribution in Steel Reinforcement with Different Fire Insulation Layer Thickness (a) Concrete Grade 40 MPa (b) Concrete Grade 60 MPa

rebar temperature), which indirectly can be assumed to provide better fire resistance for the structural elements such as the CFRP strengthened RC beams.

In addition, according to temperature distribution curve of steel reinforcement as shown in Fig. 8, when comparing the same insulation scheme provided to the CFRP strengthened RC beams (same insulation layer thickness), the higher grade of concrete exhibited higher heat transfer in the steel reinforcement (and also in the concrete) when compared to lower grade of concrete. This indicated that the use of higher grade of concrete will be needed to provide thicker insulation layer as compared to normal strength concrete that produce lower heat transfer capability to achieve same fire resistance.

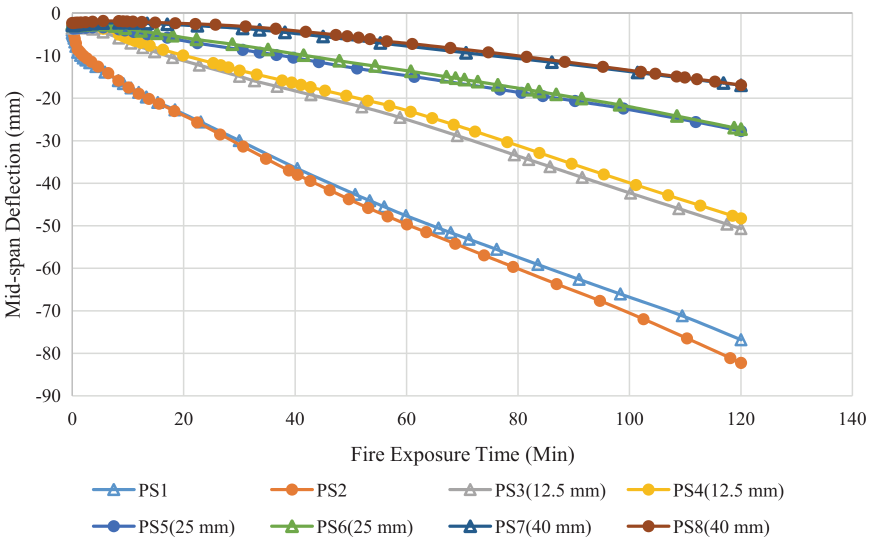


Fig. 8. Comparison of Mid-span Beam Deflection with Different Insulation Scheme

Besides that, when comparing the mid-span deflection of CFRP strengthened RC beams using different thickness of fire insulation layer, the beam displacement curve shown in Fig. 8 indicated that the use of thicker insulation layer capable to reduce the beam mid-span deflection for a particular time of fire exposure. The curve of beam displacement produced the same trend of curve either for normal strength concrete or high strength concrete.

However, the effect on deflection using the same insulation layer thickness yield different results when compared with non-insulated CFRP strengthened RC beams that use normal strength and high strength concrete. For uninsulated CFRP strengthened RC beam (PS1 and PS2), the deflection of beam PS2 generally is higher than beam PS1. This is due to the use of high strength concrete in beam PS2, which without insulation protection to its concrete & CFRP layer, might gain higher temperature for the same fire exposure duration than the normal strength concrete used in beam PS1. Besides that, beam PS2 also might exhibit cracking and spalling after exceeding the critical temperature for high strength concrete (around 400°C), which lead to the higher displacement for uninsulated CFRP strengthened high strength reinforced concrete beam than the beam using normal strength concrete.

On the other hand, for the insulated CFRP strengthened RC beam (PS3 to PS8) as shown in Fig. 7, the deflection of normal strength concrete beams (PS3, PS5 and PS7) are higher than the high strength concrete beam (PS2, PS4 and PS8) for each insulation scheme. This might be because the deflection of those insulated beams is governed by the strength of concrete and the insulation layer had contributed positively in protecting the high strength concrete beams, thereby reducing the effect of high temperature on their loss of strength when exposed to elevated temperature.

3.2.3. Deflection analysis

From the deflection curve as shown in Fig. 8, the deflection or displacement data of the uninsulated and insulated CFRP strengthened RC beams are further being analysed. As illustrated in Table 6, the magnitude of deflection using same insulation layer thickness yield different displacement rate when compared with the use of normal strength concrete (40 MPa) and high strength concrete (60 MPa).

From Table 6, the uninsulated CFRP strengthened RC beams (PS1 and PS2) yield the largest average displacement differences (33.18%) when compared to other insulated CFRP strengthened RC beams. As the insulation layer thickness used increased from 12.5 mm to 40 mm in beam PS3 to beam PS8, the average differences in displacement had been decreased from 18.33% to 4.74%, which indicated that the grade of concrete play lesser role (less significant) in reducing the beam displacement when using larger thickness of insulation layer. From this analysis, the use of 40 mm thick insulation layer only exhibited 4.74% difference in displacement when compared to normal strength and high strength concrete.

Table 6. Displacement Analysis using Different Insulation Scheme and Grade of Concrete

Beam Ref. Differences In Displacement	Insulation Layer Thickness							
	0 mm		12.5 mm		25 mm		40 mm	
	PS1	PS2	PS3	PS4	PS5	PS6	PS7	PS8
Average (mm)	0.61		0.60		0.44		0.33	
Standard Deviation (mm)	0.79		0.57		0.30		0.20	
Average (%)	33.18		18.33		9.94		4.74	

On the other hand, the use of 12.5 mm and 25 mm thick of insulation layer generated the average difference in displacement of 18.33 mm and 9.94% for those beams using normal strength and high strength concrete, respectively. It showed that the increase of insulation layer thickness from 12.5 mm to 25 mm was able to reduce the difference in displacement by about 8.39% when compared to the beam using normal strength and high strength concrete. Thus, from these 3 insulation schemes used in this parametric analysis, the insulation layer thickness of 25 mm seems to be the optimum thickness to use in the insulated CFRP strengthened RC beam as it able to meet the deflection criteria, regardless of whether normal strength or high strength concrete was used

3.3. Steel Characteristic Strength Reduction Factor

3.3.1. Steel reinforcement temperature

The analytical calculation for the steel reinforcement temperature is based on the 500°C Isotherm Method which used the Wickstrom formula for the normal weight concrete. The calculated steel reinforcement temperature then is compared with the rebar temperature prediction from the numerical model of beam reference PS1 (uninsulated beam) as illustrated in Table 7. The cross section of the beam PS1 is shown in Fig. 9.

Table 7. Comparison of Steel Reinforcement Temperature by Analytical Calculation, θ_s and Finite Element (FE) Prediction, θ_s'

Fire Exposure t (hour)	Rebar 1 Temp (Analytical) θ_{s1}	Rebar 1 Temp (FE) $\theta_{s1'}$	Rebar 1 Temp $(\theta_{s1'}/\theta_{s1})$	Rebar 2 (Analytical) θ_{s2}	Rebar 2 (FE) $\theta_{s2'}$	Rebar 2 Temp $(\theta_{s2'}/\theta_{s2})$
0.5	333.51	251.03	0.75	196.69	173.191	0.88
1.0	610.31	416.76	0.68	338.91	327.136	0.97
1.5	799.10	523.72	0.66	435.36	433.847	1.00
2.0	944.32	614.63	0.65	509.37	528.85	1.04
Average			0.69	Average		0.97

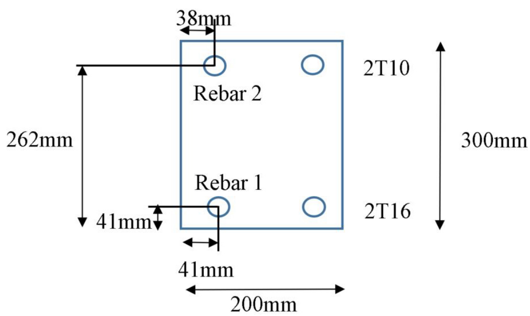


Fig. 9. Cross section and dimension of beam PS1

According to Table 7, the numerical model capable to predict the rebar 2 temperature with good accuracy as compared to analytical calculation using 500°C Isotherm Method (average ratio predicted over calculated rebar temperature of 0.97). However, the prediction of Rebar 1 temperature exhibited low accuracy with the average ratio predicted rebar temperature over calculated rebar temperature of about 0.69. This might be due to the different assumptions of heat transfer in the numerical model (heating at soffit of beam) as compared to the heat transfer in 500°C Isotherm Method, in which heating started at the bottom left corner of the beam.

3.3.2. Comparison of coefficient of steel strength reduction factor

The formula used in the coefficient of steel strength reduction factor is according to the clause 4.2.4.3 of BS EN 1992-1-2. The steel strength reduction factors then are calculated using the steel reinforcement temperature from the analytical calculation and temperature prediction Finite Element (FE) model as shown in Table 5.

The ratio of steel strength reduction factor in FE predicted over analytical calculation as shown in Table 8 is identical with the result shown in Table 7 as Rebar 2 is highly accurate when comparing both methods. However, Rebar 1 yield very low accuracy in determination of the coefficient of steel strength reduction factor. This is mainly because the calculation of steel strength reduction factor is based on the steel reinforcement temperature. The rebar 2 temperature is in good accuracy between the analytical calculation and numerical prediction, thus the computed coefficient of steel strength reduction factor is more accurate.

Table 8. Comparison of Steel Strength Reduction Factor by Analytical Calculation and Numerical Prediction

Time t (hour)	Rebar 1 (Analytical) k_{s1}	Rebar 1 (FE) $k_{s1'}$	Rebar 1 $(k_{s1'}/k_{s1})$	Rebar 2 (Analytical) k_{s2}	Rebar 2 (FE) $k_{s2'}$	Rebar 2 (k_{s2}'/k_{s2})
0.5	0.7665	0.8490	1.11	0.9033	0.9268	1.0260
1.0	0.3108	0.6782	2.18	0.7611	0.7729	1.0155
1.5	0.0802	0.5143	6.41	0.6540	0.6560	1.0030
2.0	0.0511	0.3006	5.88	0.5480	0.5022	0.9165
Average			3.90	Average		0.9902

4. Conclusions

In this paper, a numerical model for the insulated CFRP strengthened RC beams had been developed using finite element modelling software, ABAQUS. The numerical model has been used to undertake the parametric analysis to predict the steel reinforcement temperature and beam deflection for the insulated CFRP strengthened RC beams when subjected to elevated temperature by using different grade of concrete with various thickness of insulation layer. The finding from analysis as follows:

1. Average difference in calculated and prediction results of the steel reinforcement temperature between numerical and experimental result is 7.25%, the model is able to predict the temperature in the steel reinforcement with a good accuracy which matched closely with the calculated temperature.
2. The use of higher grade of concrete has been found consistently to exhibit higher temperature distribution in steel reinforcement as well as concrete element for the insulation layer thickness of 12.5 mm, 25 mm and 40 mm when compared with the lower grade of concrete.

3. From parametric study, the insulation layer thickness of 25 mm can be considered to be the optimum thickness to use in the insulated CFRP strengthened RC beams, in which it is able to meet the criteria in terms of temperature distribution and deflection requirement regardless of the strength concrete.

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Received: 28.07.2021, Revised: 25.01.2022