STEEL REINFORCEMENT IN CONCRETE STRUCTURES AND FIRE IMPACT

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Abstract: Experiments and analytical studies on the behaviour of RC beam and column members at high temperatures are commonly used. However, very little focus is placed on examining the behaviour of structures with SFRC, such as exposed SFRC beams, columns, portal structure, etc. This research work examines the effects of fire on steel reinforcement in reinforced concrete structures at high temperatures. It was validated using the commercially available programmes ANSYS and SAFIR and was done in three dimensions (3D) nonlinear transient thermo-mechanical FE analysis.
Key Words: SFRC, Fire, Steel, Concrete, ANSYS, SAFIR.

1 INTRODUCTION

For the majority of buildings and structures, fire continues to pose a substantial risk. Given how frequently concrete is used in construction, it is important to thoroughly comprehend how fire affects reinforced concrete structures. Since the dawn of time, fire has both brought comfort and disaster to the human race. A destructive force like fire results in thousands of fatalities and billion-dollar property losses. Disasters involving fire can happen above, on, or below the ground. They occasionally happen under the most unforeseen or unforeseeable conditions. Since the first attempts to establish fire safety, there has been a significant advancement in our understanding of structural fire protection.. After a major fire in London in 1666, fire protection efforts were started, but research on structural fire protection didn't start until the second half of the 18th century. Further developments in material science gave rise to the idea of gypsum-based fireproof constructions. Later, the London Building Act (1894) and the Metropolitan Borough Act (1844) refined the building control laws. After the creation of the British Fire Protection Committee towards the end of the 19th century, structural fire resistance research adopted a scientific methodology (BFPC). The first British Standard (BS 476:1932) pertaining to fire was released in 1932 and it set forth the criteria for determining fire resistance. The recent collapse of the World Trade Center's twin towers in New York, USA, caused by a terrorist attack and subsequent fire has rekindled interest in building designs that are fire resistant. In structural design, fire resistance for reinforced concrete (RC) structures and components is typically addressed indirectly. Most design processes presume adequate fire resistance assuming a certain criteria are maintained, most notably the gap between the reinforcing bars and the concrete surface. Building codes outline requirements for structures made in a way that they perform adequately in the case of a fire. These rules address issues including avoiding the spread of the fire to nearby premises, preventing premature collapse, and providing for the evacuation of residents from a burning property. However, structural integrity is the last line of defence when fire suppression systems or other methods of limiting a fire are ineffective. As a result, when constructing structures, the risk of fire must be taken into account. Though it is often believed that concrete has good fire resistance, reinforced concrete columns' behaviour under high temperatures is primarily influenced by the concrete's strength, changes in material properties, and explosive spalling. High temperatures, however, deteriorate the strength of the concrete by explosive spalling, which compromises the structural integrity of the concrete buildingNumerous academics have recently examined the fire behaviour of reinforced concrete columns in their studies, which also included experimental data. Because it is non-combustible and has a high thermal mass, concrete exhibits excellent intrinsic behaviour when exposed to fire. This considerably slows the propagation of heat through concrete components. In reality, only the outer layer of concrete, which has a thickness of around 3 to 5 cm, is harmed in the majority of typical fires (Denol, 2007). Therefore, many burned-out concrete structures can be easily rebuilt and reused. The Windsor Tower in Madrid serves as a prime illustration of how concrete buildings should behave when on fire (Denol, 2007).The fire occurred on 14 February 2005, during which the building was fortunately unoccupied. Despite that the fire spread over numerous floors and lasted for 26 hours, the building remained standing, as can be seen in Figure 1.1, only the part that collapsed were the steel perimeter columns above the 20th floor, which supported the floors.

FE ANALYSIS PROCEDURE FOR THERMAL AND MECHANICAL LOADS

It has long been understood that the fire loading must be taken into account when designing structures, but security-related concerns are making this need much more urgent. As a result of the structure's intended use, environmental factors, hydration heat, or exposure to fire, reinforced concrete structures are frequently subjected to thermal loads. The study of advanced analysis and design of reinforced concrete structures subject to thermal loads has thus attracted increasing attention. Currently, transient thermal studies are not frequently used in the thermal (fire) design of reinforced concrete structures. Rather, code provisions are typically based on detailing and cover requirements, which is based on the empirical data and provide an acceptable fire-rating in terms of the length of time that the structure must sustain its mechanical loads in the presence of fire without collapsing (e.g. three hours).

ANALYSIS PROCEDURE

To understand the response of structures to thermal loads, various stages of analyses must be considered separately. The general procedure for transient thermal-stress analysis of a RC structure in ANSYS R15.0 consists of:

i. Building a two-dimensional or three-dimensional model of the structure. The model incorporates the geometry (concrete and steel bars as reinforcement), appropriate material properties, and boundary conditions.
ii. Applying the thermal loads to the desired surface of the structure resulting from the furnace transient fire (in the form of transient temperatures versus time curves) using 24
iii. "Standard Time-Temperature" curve applied according to the ISO 834 or ASTM-E119 (shown in Figure 4.8). The remaining surfaces of the structure are considered exposed to ambient temperature.

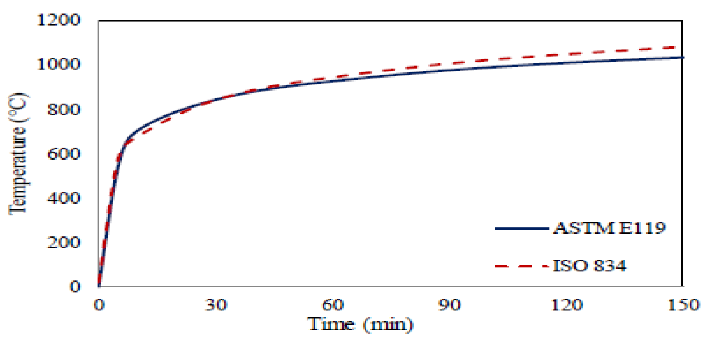

Figure 2.1: Standard Fire Curve as per ISO 834 and ASTM E119.

iv. Applying a mechanical load on suitable surface (uniformly distributed load on top face for beam and axial load for column) to simulate the dead and live gravity service loads during fire exposure. Also, apply the thermal loads in the form of nodal temperature at several time points obtained in step 2, on the structural finite element model and compute the deflection and strains in the structure.
v. Evaluate the total deflection as well as the thermal, mechanical, and total strain for different points within the mid-span beam cross section and at the beam column joint during the time of exposure to the fire from the structural finite element model. The deformation field in the model at the first load step (due to the applied gravity load) is used to verify the correct behaviour of the model and correct modeling of boundary and loading conditions.

VALIDATION PROBLEMS

AIM AND OBJECTIVES

The main aim of the present study is to provide a better understanding of the impact of fire on steel reinforcement in reinforced concrete structures this was achieved by carrying out the investigations with following objectives:

- Modelling a cross section and analysing the effect of fire on the cross section.
- Using SAFIR & ANSYS software to analyses the impact of fire on the steel reinforcement.
- Validating it with the experimental data available in the literature.

A number of FE models are available in literature studying the behaviour of RC structural elements such as beams and columns under mechanical loading with or without fire. Therefore, as a part of validation process a problem was carried out by:

TRANSIENT HEAT TRANSFER ANALYSIS

A two dimension transient analysis was carried out of unprotected steel under elevated temperature.

PROBLEM SPECIFICATION

The experimental results used for validating the model in SAFIR are based on the investigation carried out by Newman (1990). Using this numerical example given below as done in the design of steel structures book that to calculate the temperature rise for an ISMB 400 heated on four sides after 15min to ISO 834 fire.

Taking the mean of values 14 and 16min, the steel has reached a temperature of 671.5^{o}c after 15 min.

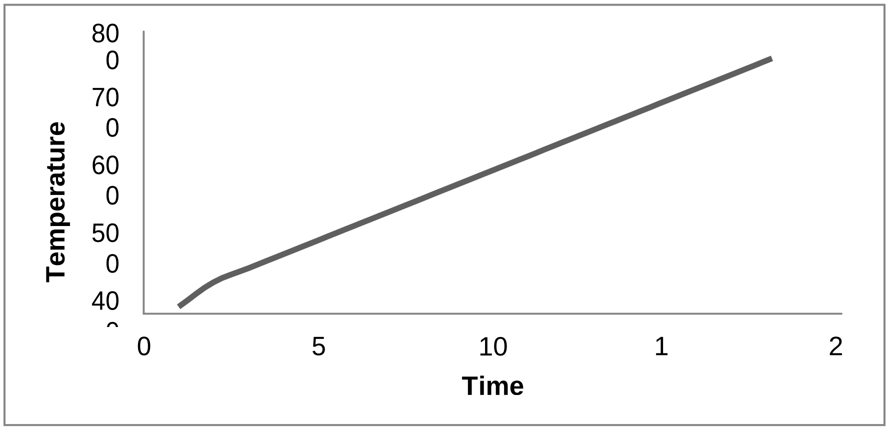

Figure 4.1: Newman (1990) Temperature-time curve for steel reinforcement (ISMB400).

MODELING USING SAFIR

Similarly this calculation was done using SAFIR software in other to calculate the amount of temperature exacted on the steel reinforcement as done by Newman (1990) structures. The steps are as follows:

STEPS OF CALCULATING TEMPERATURE USING SAFIR SOFTWARE

First Step: Using WIZARD2007 to obtain the input file for SAFIR.

i. Click on from desktop wizard2007
ii. A dialogue box opened select user, input the height of section as (D) 400, input , input thickness of the web as 8.9mm also input the Root fillet if any and click on concrete slab if you want to add concrete slab to your problem. And click on next -

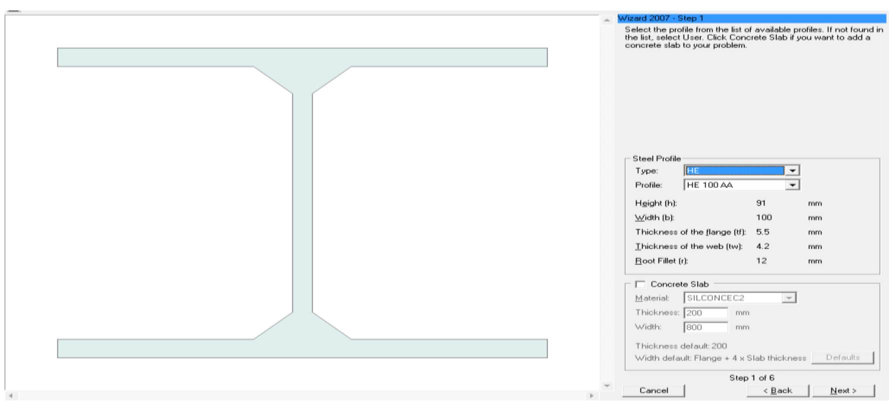

Figure 4.2: Wizard 2007 input data dialog box.

iii. Select the protection material and the protection thickness. If any, but in this case protection layer select (None) and click on next -

iv. Select the number of elements to describe the steel profile, that is the meshing and click on next -

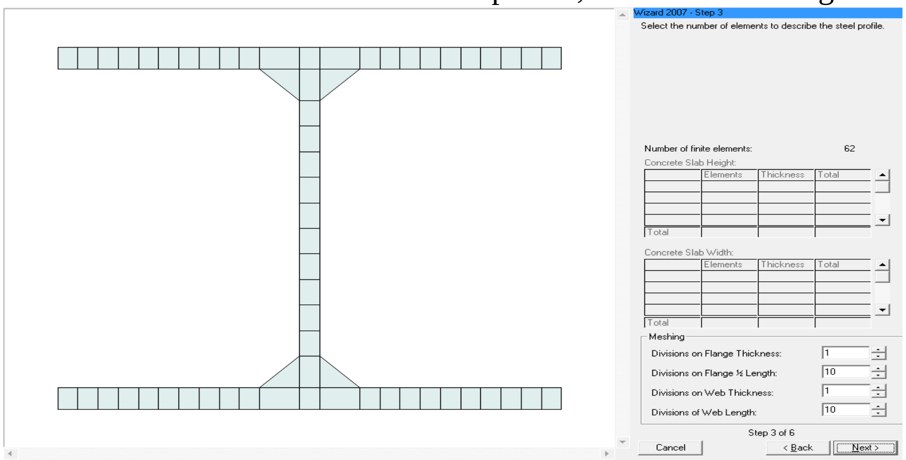

Figure 4.3 mesh configuration.

v. Select the fire curve as FISO and the rotation angle if any, and so also the exposed faces of the steel profile, Face1,face2,face3,fac4 and press on next -

vi. Select the number of integration points on elements and global coordinates of the node line and the centre of torsion. Precision as -4,number of integration points as 3 , time step as 12sec, end time 2400, time print 60sec, and press on next -

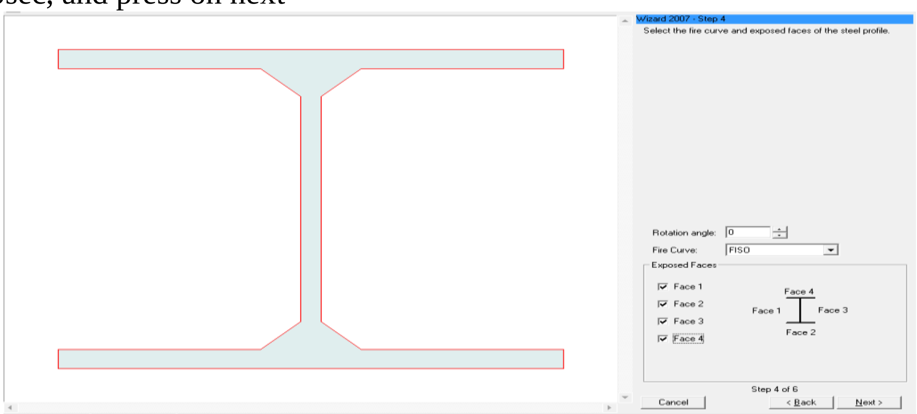

Figure 4.4: Wizard 2007 faces in contact with fire dialog box

vii. The wizard has finished collecting the information needed for the SAFIR input file, and press Finish to save values in the data file as **abba.**

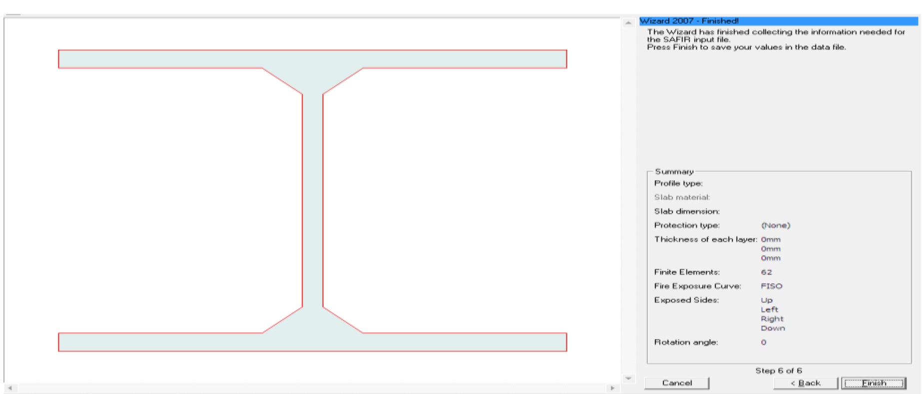

Figure 4.5: Wizard 2007 input data final dialog box.

Second Step: Using DIAMOND software obtain the temperatures as well as the charts

1. Click on DIAMOND 2011 from the desktop
2. Click on file menu and select open from there select the output file created from SAFIR as abba.
3. The file will opened from there click on result and select temperature, the result for temperature will be displayed, also by clicking on result by selecting chart the chart is going to be plotted.

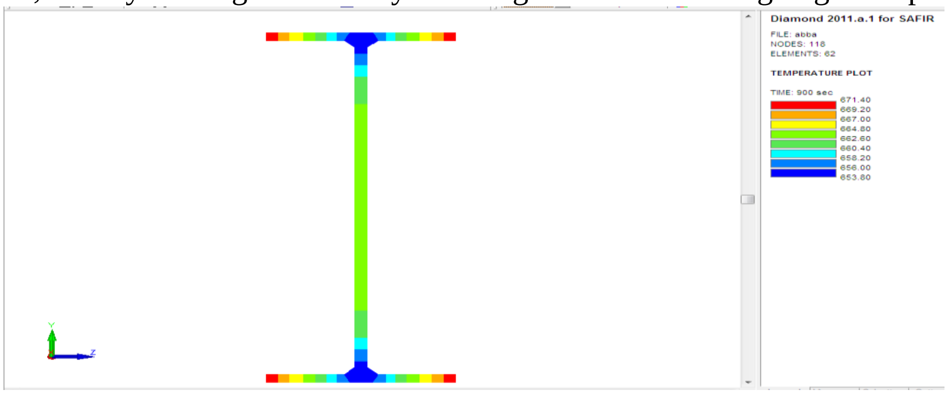

Figure 4.7 Diamond output chat file.

The image above shows the screen snap from the DIAMOND environment, representing the temperatures at the time of 900sec (15minutes). Similarly the chart for that is as shown below.

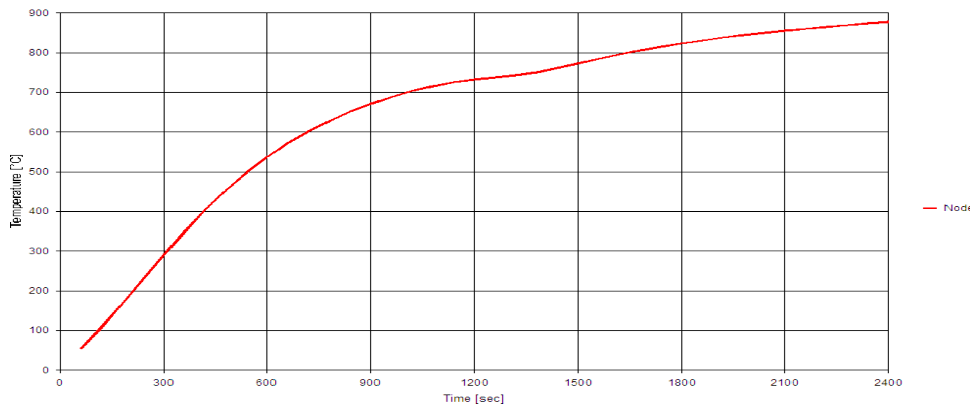

Figure 4.8: Temperature-time curve for steel reinforcement Model (SAFIR v2011a3)

It can be seen that the value of temperature obtained as the time of 15min (900sec) from Fig.4.1 the manual calculation (Newman 1990) the steel has reached a temperature of 671.5^{o}C after 15 min. while from Fig. 4.8 the DIAMOND Output file the steel reached 671.4^{o}C at 900sec (15min). Almost about 0.5^{o}c is the difference between the two. Thus, it is clear to say that the predicted and experimental values are in good agreement with each other.

HEAT TRANSFER ANALYSIS

Three Dimensional (3D) Transient heat transfer analysis of a concrete beam reinforced with glass fiber reinforced polymer (GFRP) rebar is carried out.

4.2.1 PROBLEM SPECIFICATION

Here, ANSYS 15.0 is used to perform a 3D heat transfer analysis on an RC beam reinforced with GFRP rebars. Based on Pratik's investigation, the experimental results used to validate the model in ANSYS (2013). Abbasi and Hogg also developed a FE model for the same (2006). Using a conventional fire curve in accordance with ISO 834, the beam is simulatively exposed to fire at its bottom and on its sides. The beam is 4400 mm long and has a cross section that measures 350 by 400 mm. It is reinforced with 12.7 mm diameter 7 GFRP rebars, two layers of which are at the bottom and two layers of which are at the top, and it is encircled by 9 mm stirrups that are spaced 160 mm apart, as illustrated in Figure 4.9. The cover to the reinforcement is 75 mm at the bottom and 50 mm on the sides and at the top. The thermal properties of the material useful for the analysis are provided in Table 4.2 below. The strength of concrete used is 42 MPa i.e. a normal strength concrete.

Table 4.2: Thermal Properties of the Materials for Validation Problem

Material	l conductivity (k) Wm/K	Density (ρ) kg/m^3	Specific heat (C) J/kgK
Concrete	2.7 X 10^{-3}	2.32 X 10^{-6}	722.8
Rebars	4.0 X 10^{-3}	1.60 x 10^{-6}	1310

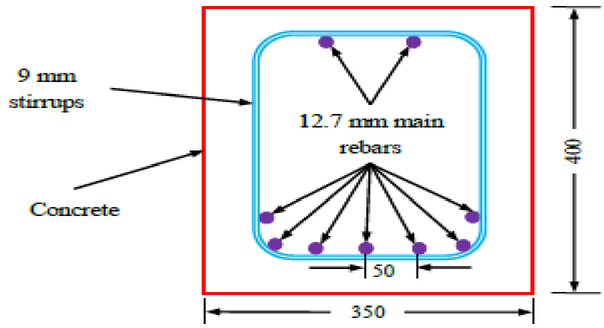

Figure 4.9: Arrangement of GFRP Rebars in Concrete Beam (All Dimensions are in mm)

5.0 MODELLING, RESULTS AND DISCUSSION

Utilizing the ANSYS 15.0 programme, a 3D reinforced concrete beam model was created. SOLID 65 element was used to model the concrete beam and analyse heat transmission. At nodes, there is just one degree of freedom, which is temperature. Modeling of the GFRP bars is done using the LINK180 element. At each node, this element also only has one degree of freedom, temperature. No mechanical boundary conditions are applied to the model because no mechanical load is being supplied. The software was able to achieve the full bonding that is believed to exist between the rebars and the concrete. As a predetermined initial condition, the complete beam model was exposed to a room temperature of 200C. Then, the vertical sides and soffit of the beam are exposed to fire in the form of an ISO 834 standard time temperature curve. For this, temperature boundary conditions that fluctuate with the amplitude of the standard time temperature curve during the transient heat transfer are defined at the sides and soffit of the beam. Figures 5.8 and 5.9 below demonstrate the temperature fluctuations at various time intervals discovered by this research using ANSYS at the centre and side of the beam.

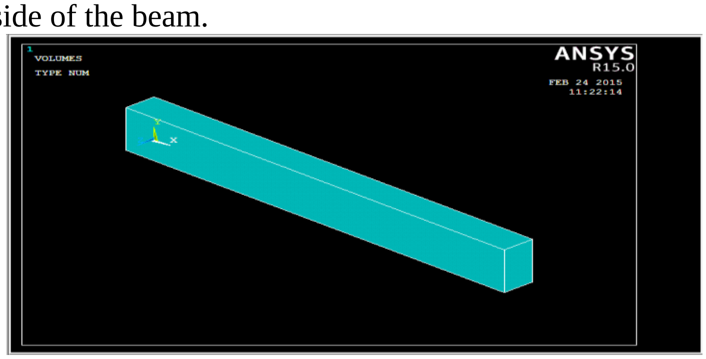

Figure 5.1: Concrete beam layer FEM model.

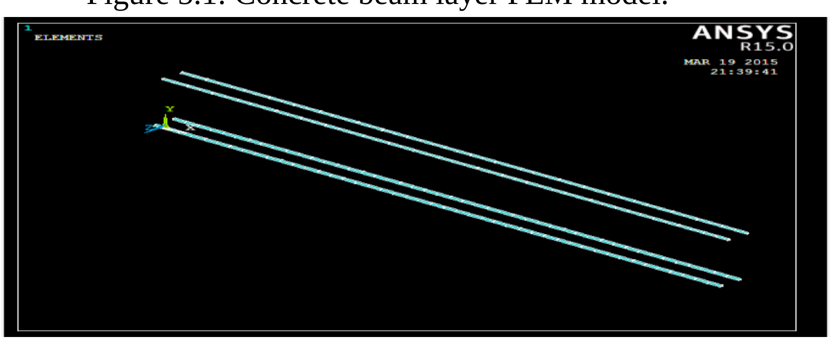

Figure 5.2: GFRP layer FEM model.

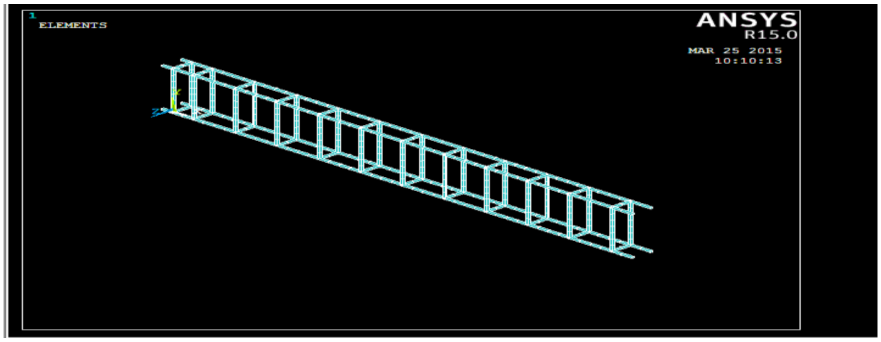

Figure 5.3: GFRP and stirrups layer FEM model.

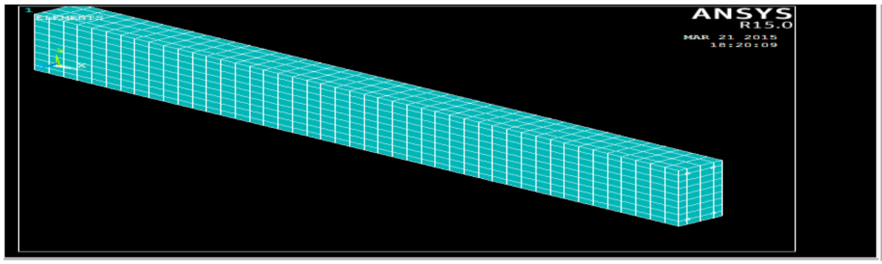

Figure 5.4: Mesh configuration of Beam with Rebars for problem.

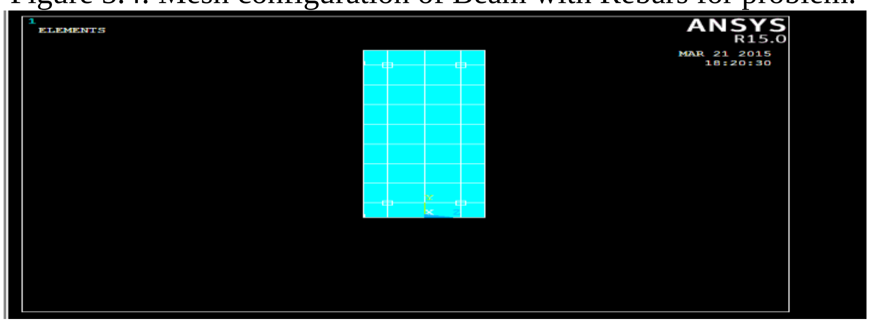

Figure 5.5 Side view of the RC beam.

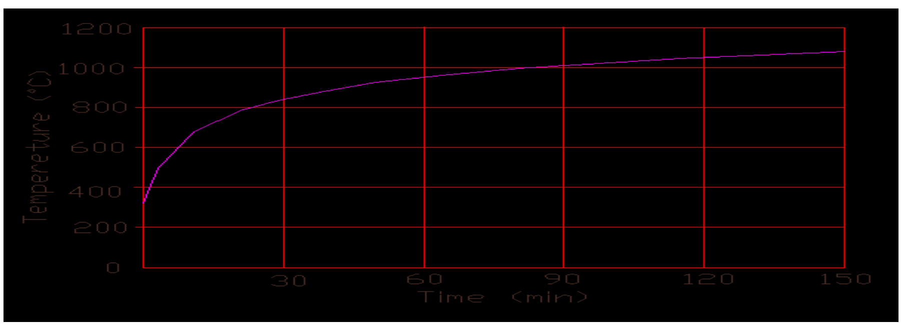

Figure 5.6 ISO 834 fire curve.

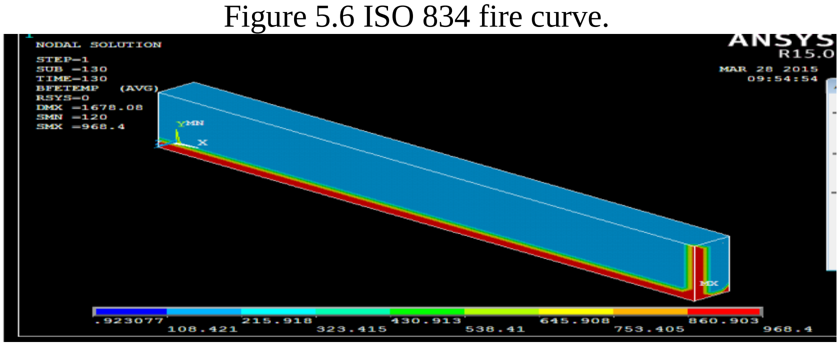

Figure 5.7: Nodal Solution for the FEM model at 130min.

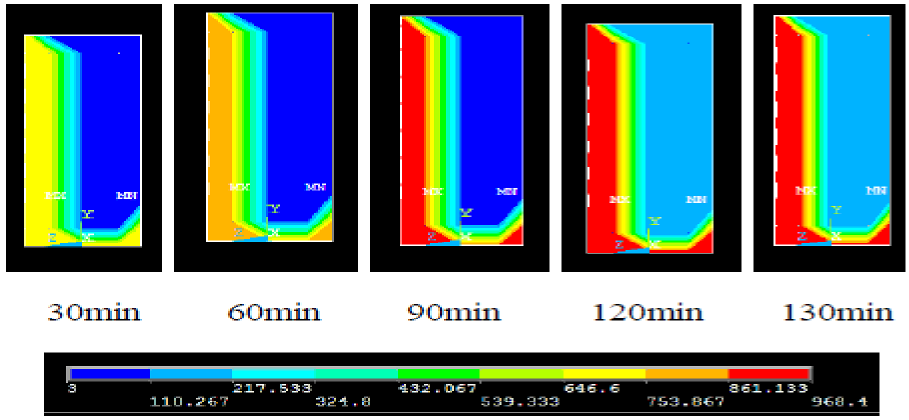

Figure 5.8: Side view Temperature Variation in Beam Obtained in Present Study.

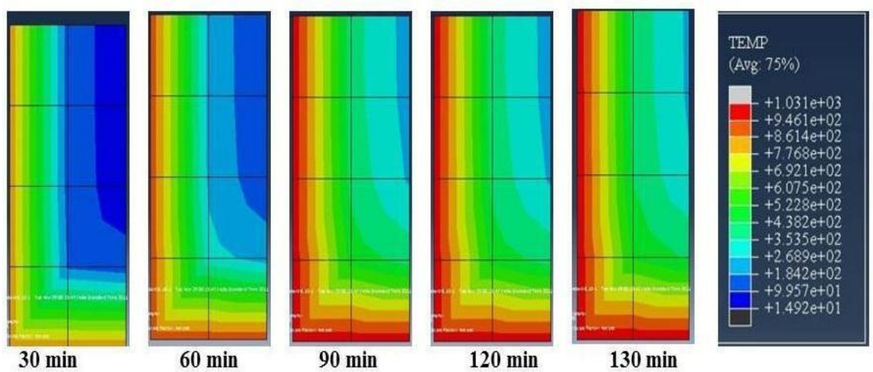

Figure 5.9: Temperature Variation in Beam (Pratik, 2013).

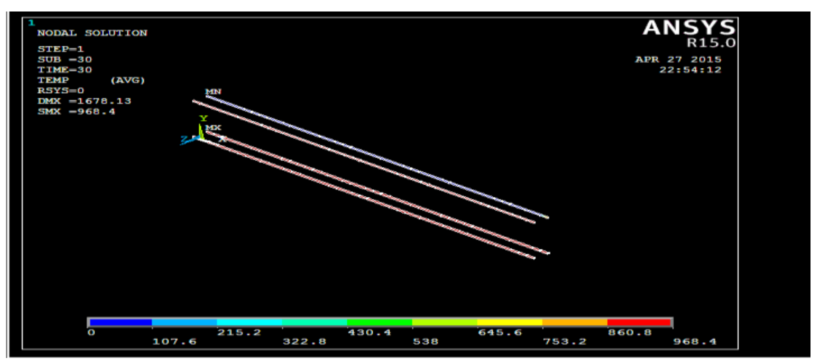

Figure 5.10: Temperature Variation in GFRP Obtained in this study

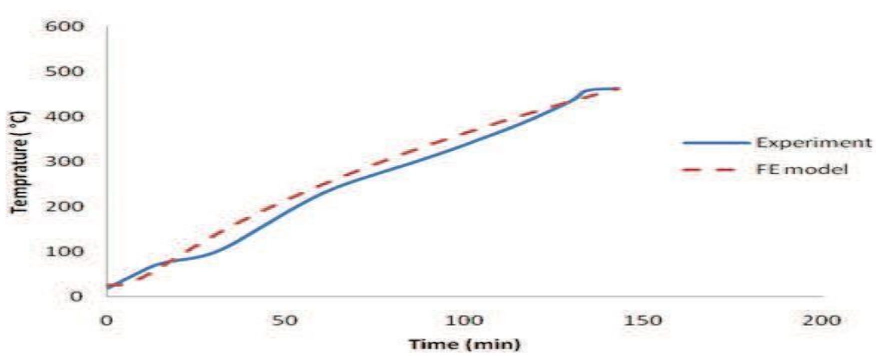

Figure 5.11: Temperature Variation in GFRP Obtained at the end of this simulation.

According to the ANSYS model, as shown in Figure 5.11 above, the temperature in the GFRP bars hit 4300C at a time that was deemed to be the start of their failure at around 90 minutes, which is 3 minutes earlier than the experimental measurement. It is evident that the experimental and projected values agree well with one another. The fact that GFRP rebars lack temperature-dependent material properties may explain why the FE model overestimates the outcome when compared to experimental data. In order to accurately estimate the temperature distribution in concrete beams, a developed FE model can be employed.

CONCLUSION AND RECOMMENDAATION

CONCLUSION

As can be seen in the preceding sections, a nonlinear 3D FE model was created in this study and validated against the experimental programme run by Abbasi and Hogg (Abbasi & Hogg, 2006). At all stages of fire exposure, good agreement was found between the average temperature observed experimentally and the anticipated FE simulation. The results of the fire tests and FE simulations revealed that concrete beams reinforced with GFRP bars can achieve a fire endurance of about 130 minutes, despite the UK Building Regulations' (Building Regulations, 2000) recommendation that the minimum periods of the fire resistance for the majority of structural elements be of 90 minutes. So it appears that using GFRP bars as concrete reinforcement satisfies the fire design specifications. Upon the validation of the measured data, the FE modeling could provide full field of results, in terms of 3D temperature distribution. One may draw the conclusion that the created FE model is an excellent tool for helping designers and researchers to numerically forecast the temperature distribution of RC beams reinforced with GFRP bars. Therefore, especially in design-oriented parametric research, the validated model could be utilized as a reliable tool in place of actual testing.

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