ANALYTICAL STUDIES ON THE FIRE RESISTANCE OF THE RC BEAMS EXPOSED TO PARAMETRIC TIME-TEMPERATURE CURVE

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Abstract: An accidental fire is an extreme event that may alter the performance and behavior of structural elements. The Reinforced Concrete (RC) elements have considerably better fire resistance than structural steel elements. Owing to this, reinforced concrete elements are widely used in the construction industry. It is evident from previous literature that the actual fire resistance capacity of the reinforced concrete beams varies with the change in fire scenarios. However, the present Indian standard provides prescriptive tables to improve the fire resistance of RC structural elements. Therefore, a detailed study needs to be carried out to determine the fire resistance of reinforced concrete beams specified in the Indian Standard code by adopting the fire dynamics. In the present work, finite element analysis of RC beams is conducted using Finite Element Software ABAQUS. The RC beams were subjected to parametric time-temperature curves and standard time-temperature curves to determine the equivalent fire severity of the Eurocode design fires (Parametric Time-Temperature Curve). The considered concrete and steel properties for the simulation are validated by the previous literature works. Further, the limiting fire load of the RC beams subjected to parametric design fires is determined. Further, the concept of Duration of Heating Phase is applied to compare with the standard time-temperature fire and the deflections of the fire-exposed RC elements are considered for detailed discussion and conclusion.

***Index Terms* – Beam, Fire Resistance, Temperature Curve , Structural Analysis ,Concrete, Steel**

I. INTRODUCTION

The accidental fire in buildings is a severe environmental condition where structural elements will be subjected to spontaneous increment in temperature. Fire in RC structures causes geometric (thermal expansion) and material effects (loss in stiffness and strength) in reinforced concrete elements. The exposure of structural elements to fire causes deterioration in structural elements' material properties and may lead to the total collapse of RC structure [1]. The RC structural members should have minimum fire resistance usually known as standard fire resistance or a performance-based design needs to be adopted to overcome the catastrophic failure of the RC structure. A series of numerical and experimental analysis on RC beams has been done by various researchers in the past. During the analysis, the reinforced concrete beams were subjected to a parametric time-temperature curve and standard fire curve to determine the effect of permeability, concrete strength, support conditions, fire scenario, exposure face, and load level. Dwaikat. M. B, et. al (2009) [2], observed that the fire resistance of reinforced concrete beams is observed to be higher with the normal strength concrete and axially restrained conditions. Also, the change in fire exposure period, fire exposure surfaces, and type of load acting on the element significantly affects the ultimate capacity of the reinforced concrete beams. [3]. However, the residual capacity of RC beams determined by applying the 500-isotherm method (EN-2) [4] shows, that the influence of clear cover is maximum to the residual load-bearing capacity of fire-exposed RC beams than the grade of concrete [5]. To explore the outcome of significant parameters on the residual response of RC beams exposed to fire, a nonlinear finite element analysis is used [6]. It is inferred that the decrease in residual capacity of RC beams subjected to fire almost doubles when the load level increases from 30% to 60% of the room temperature capacity of the RC beams. To assess the performance of reinforced concrete beams subjected to natural or parametric time-temperature curves, a performance indicator called Duration of Heating Phase (DHP) was developed by T. Gerney and J.-M.Franssen (2015) [7]. The DHP represents the burning period in the parametric time-temperature curve. It is observed that the RC beams have a DHP value lower than the fire resistance of the beams [7]. However, an attempt is yet to be made to determine the actual fire resistance, limiting fire load capacity, and DHP of the RC beams considered as per IS 456:2000 and EN-2 standards by exposing parametric time-temperature curves. A limiting fire load capacity is the amount of combustible material that can be stored inside a compartment or under the beam.

The present building standards IS456:2000 [8] and EN-2 [4] suggest guidelines in the form of prescriptive conditions such as clear cover, width and depth of the member, and protective coatings to increase the fire resistance of reinforced concrete (RC) structural elements. It has to be noted that, the provided prescriptive conditions are limited for exposure of elements to a standard time-temperature curve. This means, that the prescriptive conditions neither consider the advanced fire behavior nor the performance of elements during the exposure to fire. Because of this, the behavior of structural elements, either part of the building or global behavior of the building is difficult to estimate. Hence, there is a need for the application of performance-based design to predict the actual behavior of the reinforced concrete elements by incorporating the fire dynamics. In the current study, an effort has been made to determine the influence of fire load on the fire resistance of RC beams. To conduct performance-based design, Finite Element Analysis (FEA) provides cost-effective and time-saving solutions compared to experimental-based studies. The FEA enables the simulation of fire behavior and determines the performance of structural elements. A sequentially coupled thermo-structural analysis is performed in ABAQUS [9]. The EN 1992-1-2(2004) [4] suggested mechanical and thermal properties of steel and concrete at elevated temperatures. This data has been adopted to perform the sequentially coupled thermo-structural analysis.

II. METHODOLOGY

A. Sequentially coupled thermo-structural analysis

A sequentially coupled thermal stress analysis can be performed in Abaqus standard if the stress/displacement solution depends on a temperature field but there is no inverse dependency. In order to execute a sequentially coupled thermal stress analysis, the pure heat transfer problem must first be solved before reading the temperature solution into a stress analysis as a predefined field. The heat transfer analysis model and the thermal stress analysis model can use meshes that are different in Abaqus.

B. Heat transfer analysis

A heat transfer analysis simulates the temperature distribution in a RC Beam exposed in the fire. The three modes of heat transfer in the fire are conduction, convection, and radiation. The process in which thermal energy is a transfer within the RC Beam is called conduction, and the process in which thermal energy is transfer between the RC Beam and hot gas is called convection. Furthermore, the process in which thermal energy is exchanged between two surfaces is called radiation. The heat transfer from the fire into the RC Beam surface by a combination of radiation and convection is treated as a boundary condition, and heat transfer within the structure by conduction is governed by heat conduction equation. The output of the thermal analysis is the temperature distribution in RC Beam which will be used as temperature boundary conditions for structural analysis.

C. Structural analysis

A structural model is needed in conjunction with the heat transfer model to trace structural response during fire exposure. A structural analysis simulates the response of RC Beam when mechanical loading and fire are present in the RC Beam. The main aim of structural analysis is to find the distribution of displacement and stress under the static or dynamics loading and boundary conditions. Due to the dynamic nature of fire temperature curve and nonlinearity, a nonlinear dynamic structural analysis must be considered for structural fire analysis. There are three primary sources of nonlinearity: geometric nonlinearity, material nonlinearity, and contact nonlinearity. Geometric nonlinearity occurs when there are large displacements in the structure, whereas material nonlinearity occurs due to plasticity beyond the linear-elastic material behaviour. Similarly, contact nonlinearity occurs when boundary conditions in the FE model change during the analysis.

D. ABAQUS FE analysis procedure

The finite element solution process consists of a step-by-step procedure summarized as follows regarding thermal and structural analysis procedures.

Thermal analysis:

1. Create geometry of concrete and rebar in part module.
2. Thermal properties of concrete and steel are given as input in the property module.
3. Assemble the steel rebar, concrete and supports in assembly module.
4. Select heat transfer analysis in step module.
5. Apply the thermal boundary conditions in interaction module.
6. Assign the element type and mesh seed in mesh module.
7. Create job in job module
8. Obtain the temperature results in visualization module

Structural analysis

1. Copy the heat transfer analysis model and following changes are made in model
2. Mechanical properties of concrete and steel are given as input in the property module.
3. Select static general in step module.
4. Apply mechanical load and boundary condition in load module
5. Apply result obtained from heat transfer analysis into predefined field in load module.
6. Change the element type of heat transfer to structural analysis element type.
7. Create job and submit for analysis in job module
8. Obtain the desired result such as temperature and deflection in visualization module.

III. DISCRETIZATION OF RC BEAM MODEL

The commercially available finite element package ABAQUS is considered to conduct the sequentially coupled thermo-structural simulation. In the ABAQUS standard, the first heat transfer case is executed and then the reading from the temperature solution is incorporated for mechanical analysis as a predetermined field. The element required to complete both analysis cases are detailed as under. Thermal analysis: An (DC3D8) eight-noded linear brick element is used to model concrete and two noded truss element or link element (DC1D2) is considered for reinforcement. To allow heat transfer from concrete to steel elements, a tie constraint is taken into consideration. The thermal boundary condition is assigned to the RC beam model. In the analysis, the convective heat transfer coefficient for fire-exposed concrete surfaces is 25 W/m2oC and the coefficient for the unexposed concrete exterior surface is 9 W/m2oC is considered as per Eurocode-2 recommendations. Further, the emissivity factor of 0.7 was taken into account for exterior concrete surfaces [10]. Structural analysis: In the RC beam model after completion of thermal analysis the thermal element property is changed to (C3D8R) eight noded continuum elements for concrete and the reinforcing steel to two noded (T3D2) link element. The interlinkage between reinforcing steel and concrete is completed through the embedded region. Further, a simply supported condition is assigned in the boundary condition to perform structural analysis.

A. Material properties

To complete the intended thermo-mechanical analysis, non-linear material properties of steel and concrete are considered at each analysis phase. The mechanical analysis was performed by considering the damage plasticity model of concrete developed by Lee and Fenves [12] based on work by Lubliner et al [11]. Concrete and reinforcing steel's temperature-dependent thermal properties, namely specific heat, thermal expansion, and thermal conductivity, and in the two-phase study, the mechanical properties of concrete and reinforcing steel are assigned by considering the Eurocode 2 and 3 recommendations [13,14,15]. Density for steel is 7850 kg/m3, Poisson ratio 0.3 and which are independent of temperature. Steel yield strength of 420 N/mm2 and 500 N/mm^2 are considered for validation and parametric study respectively. Density for normal weight concrete is 2300 kg/m3, poison ratio 0.15 which are independent of temperature. Concrete compressive strength of 55 N/mm^2 and 25 N/mm^2 are considered for validation and parametric study respectively. The equation for obtaining thermal material data for steel and concrete are discussed in appendices.

B. Thermal properties of concrete and steel

Concrete and reinforcing steel's temperature-dependent thermal properties, namely specific heat, thermal conductivity and thermal expansion are considered as per EN 1992-1-2,2004). The equations are detailed in appendix 2. Graphical representation of these properties are shown in following figures.

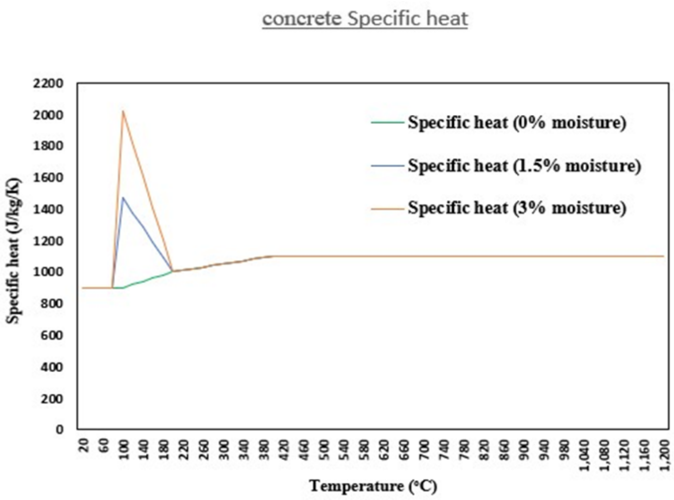

Fig 2.1 Specific heat of concrete

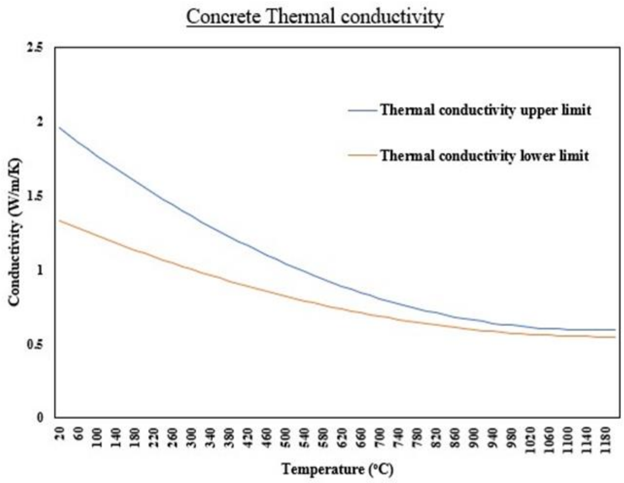

Fig 2.2 Thermal conductivity of concrete

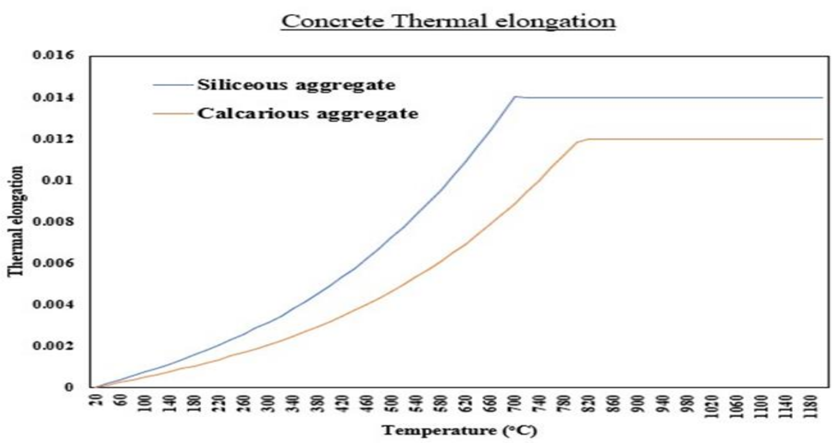

Fig 2.3 Thermal elongation of concrete

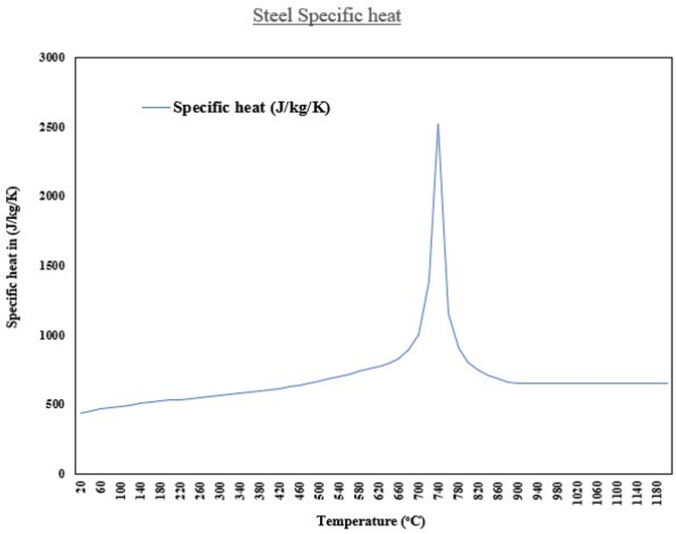

Fig 2.4 Specific heat of steel

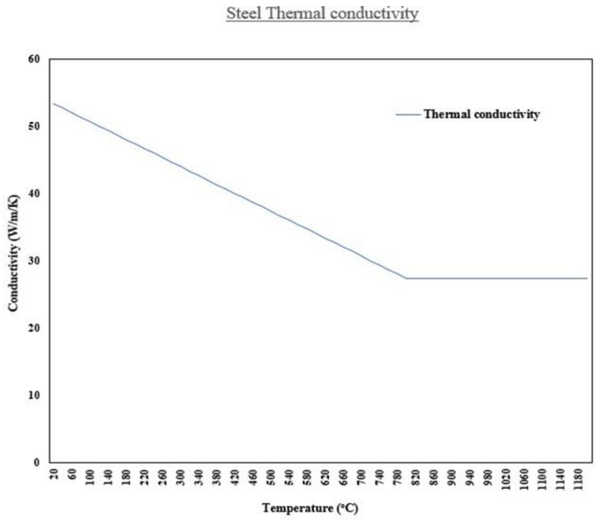

Fig 2.5 Thermal conductivity of steel

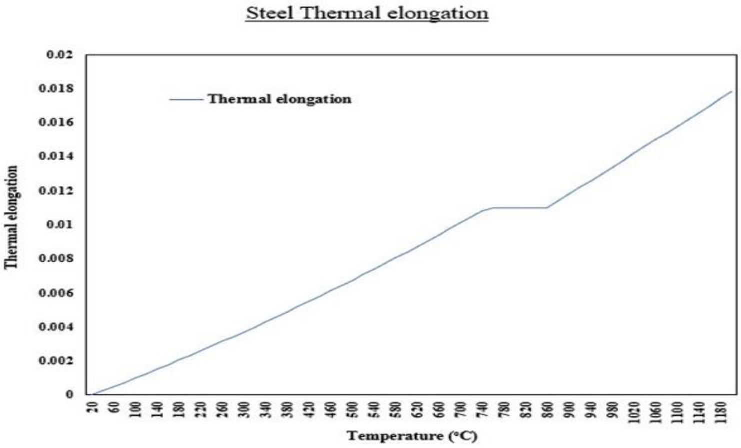

Fig 2.6 Thermal elongation of steel

C. Mechanical properties of concrete and steel

Fig 2.7 shows the stress strain relationships of concrete under compression at elevated temperatures. The equation for stress strain relationships of concrete under compression at elevated temperatures are given in appendix 2.The concrete damaged plasticity CDP model in ABAQUS was used to define the inelastic behaviour of concrete. Damage of concrete is associated with the two main failure mechanisms, namely tensile cracking and compressive crushing, and evaluation of the yield surface is controlled by the equivalent plastic strains in tension and compression, respectively (ABAQUS, 2013). There are five parameters which need to define the damaged plasticity model and they are given in appendix 2. The classical metal plasticity model available in ABAQUS was used to model steel materials.

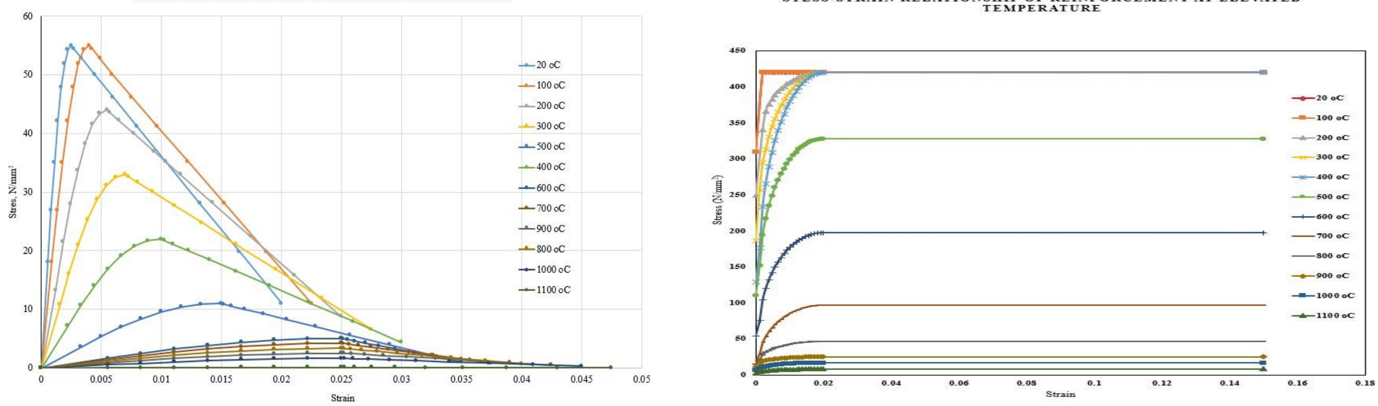

Fig 2.7 Stress-strain relationship of concrete at elevated temperature

D. Fire Scenarios

The term "standard fire" refers to the time-temperature curve used in standard fire resistance testing. The standard time temperature curve and test methods are given in standards, such as ISO 834, BS 476 and ASTM-E119.In the present study ASTM-E119 standard time temperature curve is used.

Parametric time-temperature curve are generated according to EN 1991-1-2 (2002) as shown in appendix 3. Parametric fire is more realistic time–temperature curve and behaves like natural firere. Parametric fires can be produced for any combination of fuel load, ventilation openings and wall lining materials. In the present study the compartment dimension of 9 X 6m, Ventilation Factor 0.046, fire load density of 300 to 6400 MJ/m2, and compartment lining concrete are considered to generate the parametric time-temperature curves. Fig 2.9 illustrates the graphical representations of ASTM-E119 standard and parametric time temperatures curves.

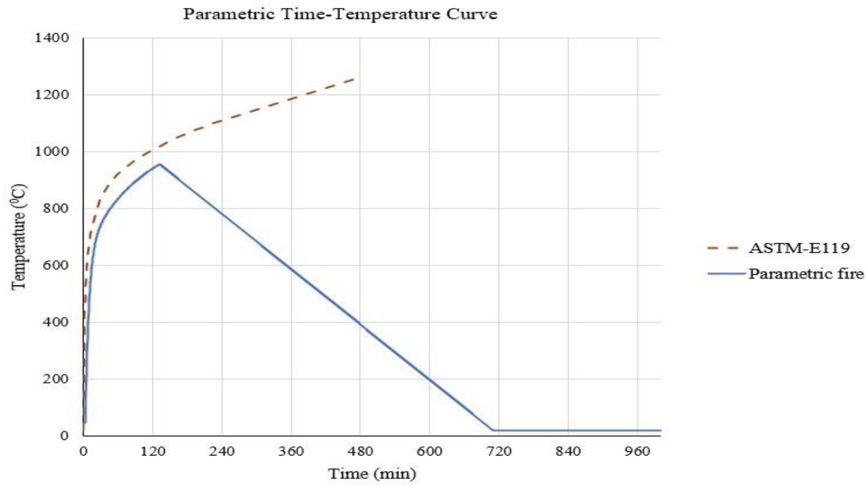

Fig 2.9 Time-temperature curves

E. Deflection Failure criteria

The RC beams considered in the present undergo predominantly larger deflection owing to the increment in the exposure temperature of standard fire or parametric fire. Hence, a limiting condition is required to define the failure criteria of the considered RC beam. The deflection limit state adopted for failure is considered as per CEN (2012) [16] is as follows

a. The RC beam's maximum deflection surpasses $L^2/(400d)$ (mm) at any fire exposure time or

b. The rate of deflection exceeds $L^2/(9000d)$ (mm/min).

Where,

c. L = Span length of the beam (mm)

d. d = Effective beam depth (mm)

IV. Numerical Method

The adopted material properties and modeling technique is validated by considering the experiment conducted by Dwaikat and Kodur (2016) [2]. The RC beam B1 and B2 experimental details are considered to compare the results of the validated model. The RC beam B1 and B2 has a length of 3960 mm with a width of 254 mm, an overall depth of 406mm and is subjected to ASTM-E119 standard fire and short design fire respectively [17] to a length of 2440 mm as shown in Fig 3.1 and Fig 3.2. A simply supported condition was considered for Beam B1 during the experiment. B2 beam was axially restrained during fire exposure. Three 19mm tensile reinforcements were placed at the bottom of the beam, and at the top, two 13mm compression reinforcements were installed. Along the length of the beam, the 6 mm diameter shear reinforcement was positioned 150 mm apart. The grade of longitudinal reinforcement was 420 MPa and transverse reinforcement 280MPa, respectively. The concrete has a 55 Mpa compressive strength.

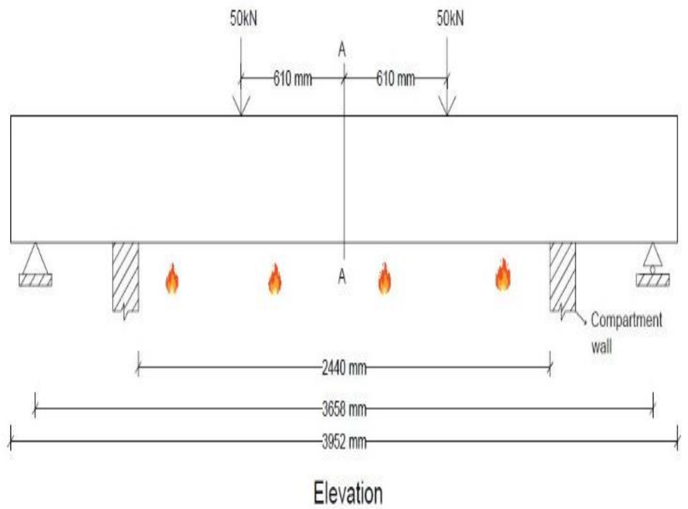

Fig 3.1 Elevation of beam selected for validation

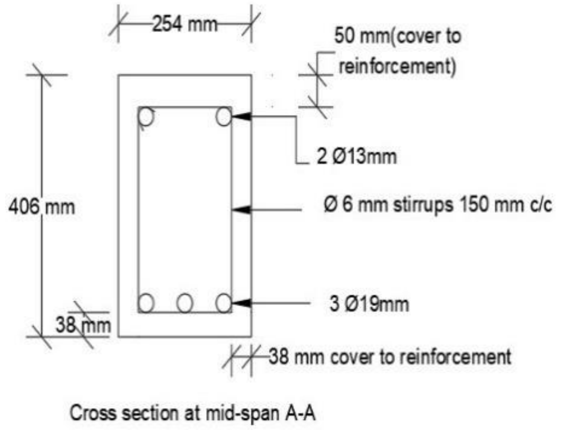

Fig 3.2 Cross-section of beam

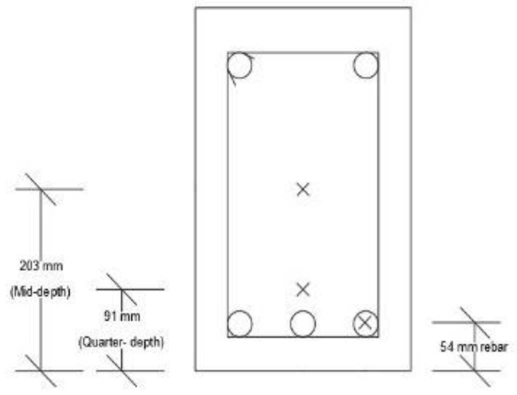

Fig 3.3 Location of thermo-couples

The RC beams were loaded with two 50kN forces at a 610mm distance from the center of the beam. The loading of the beam started 30 minutes before the fire exposure and was maintained constant until the reinforced concrete beam fails is witnessed [6,18]. As illustrated in Fig 3.3, thermocouples were positioned at three positions (i.e., quarter-depth, mid-depth, and bottom rebar) to detect the temperature rise within the beam.

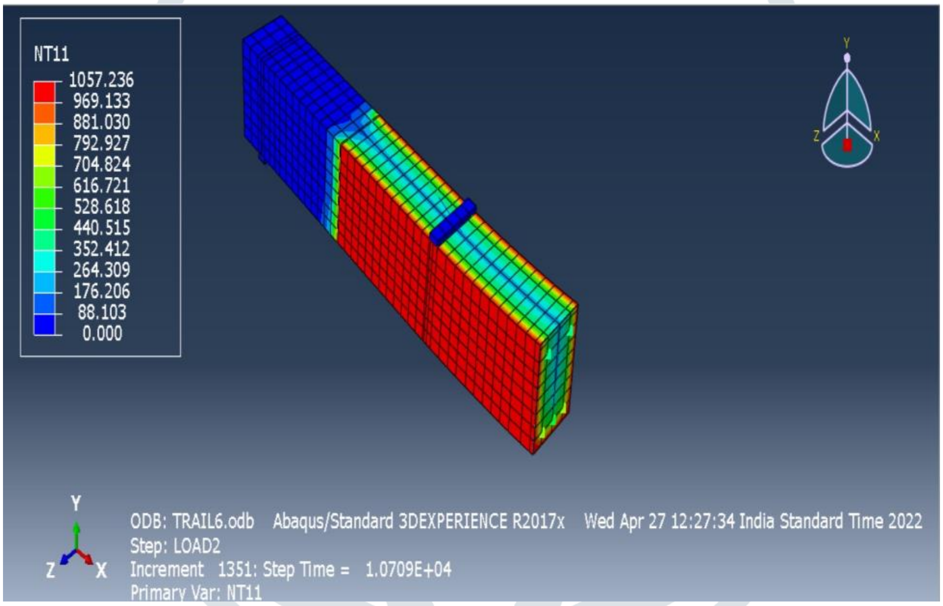

Fig 3.4 Cross-sectional temperatures for beam B1

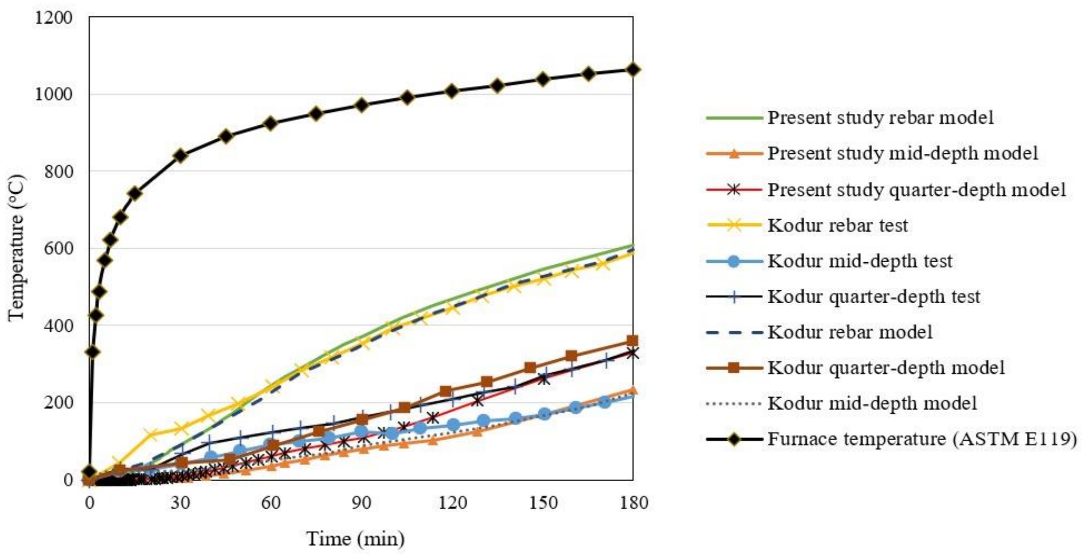

Fig 3.5 Comparison of cross-sectional temperatures for beam B1

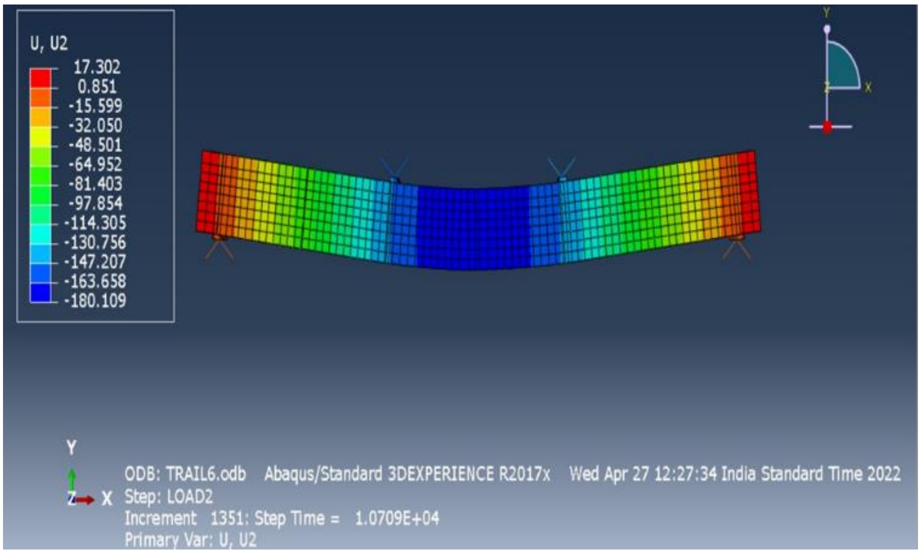

Fig 3.6 Mid-span deflection for beam B1

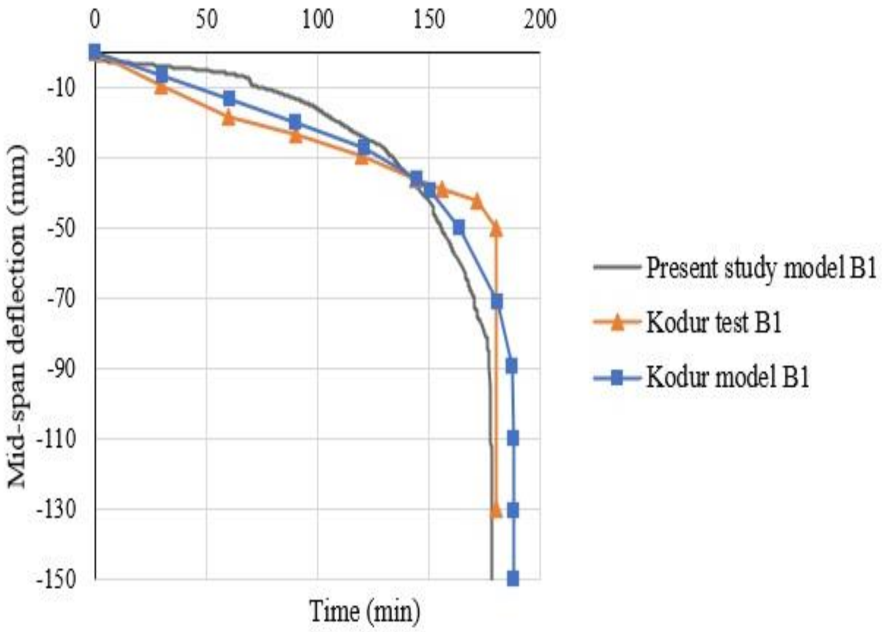

Fig 3.7 Comparison of mid-span deflection for beam B1

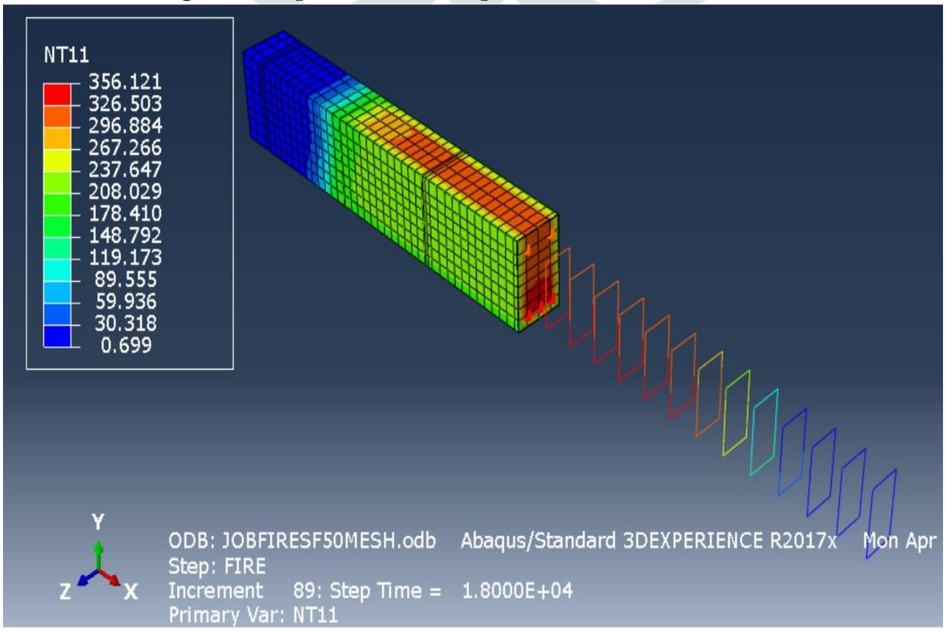

Fig 3.8 Cross-sectional temperatures for beam B2

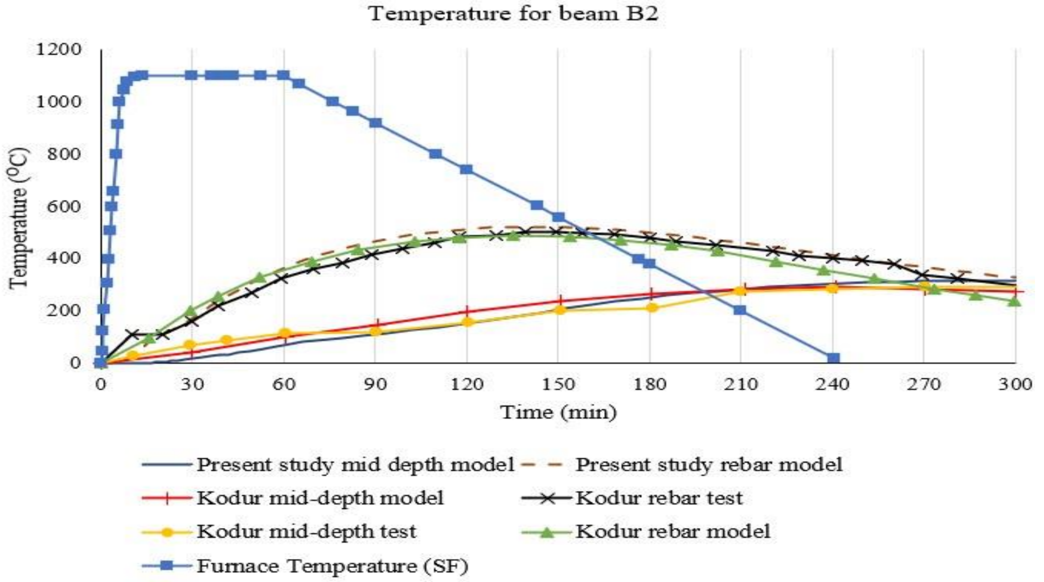

Fig 3.9 Comparison of cross-sectional temperatures for beam B2

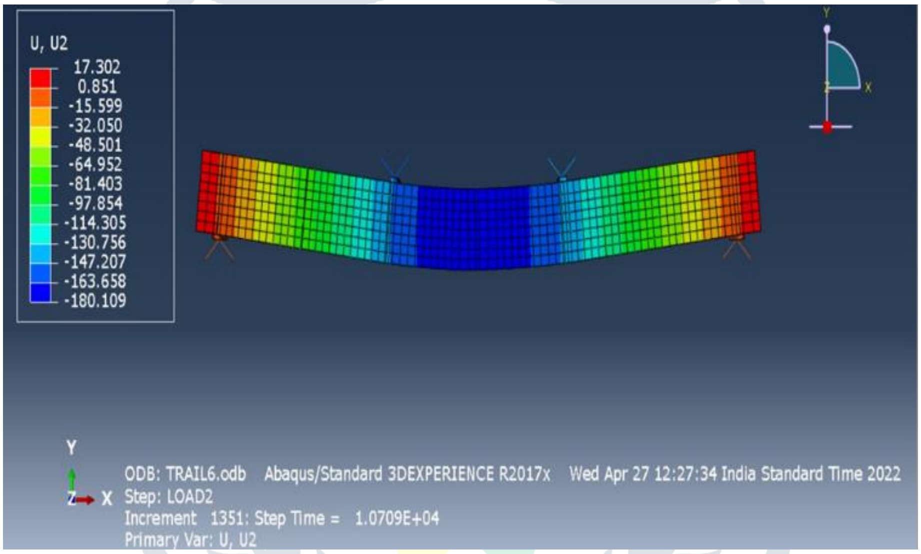

Fig 3.10 Mid-span deflection for beam B2

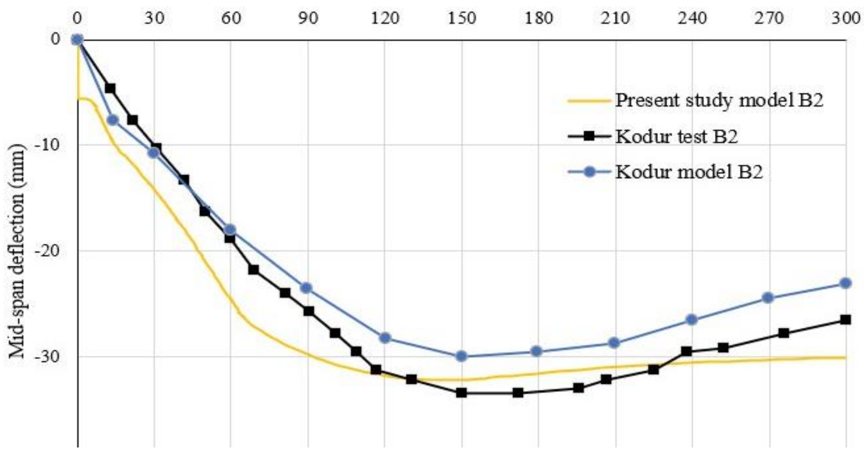

Fig 3.11 Comparison of mid-span deflection for beam B2

V. Results and Discussions

The RC beams considered in the present study were exposed to standard and parametric time-temperature curves to determine the actual fire resistance capacity of the by changing the load ratio on the beam. Also, the limiting fire load and the time at the failure of the RC beams are determined for all the exposure scenarios. A limiting fire load is the amount of combustible material that can be stored under the beam or inside the compartment of the present study. Further, the Duration of the Heating Phase (DHP) is determined by the parametric time-temperature curve obtained during the parametric fire curve of the limiting fire load scenario. The duration of the heating phase is the total time of burning or total time to reach the highest temperature present in the parametric time-temperature curves.

A. ABAQUS FE analysis results

i. RC Beam B1 exposed to ASTM E119 standard and parametric time-temperature curve for Load ratio 40%

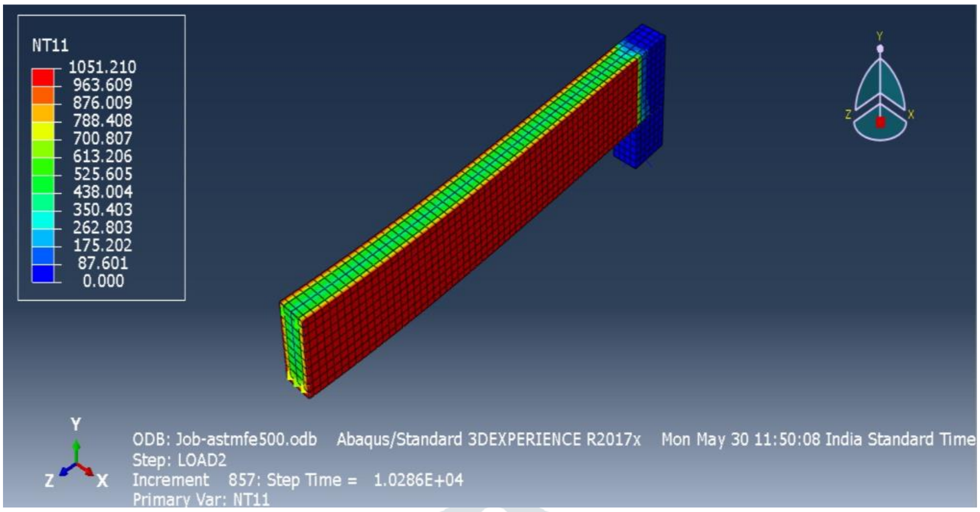

Fig 5.1 Nodal temperature of beam B1 exposed to ASTM-E119 curve

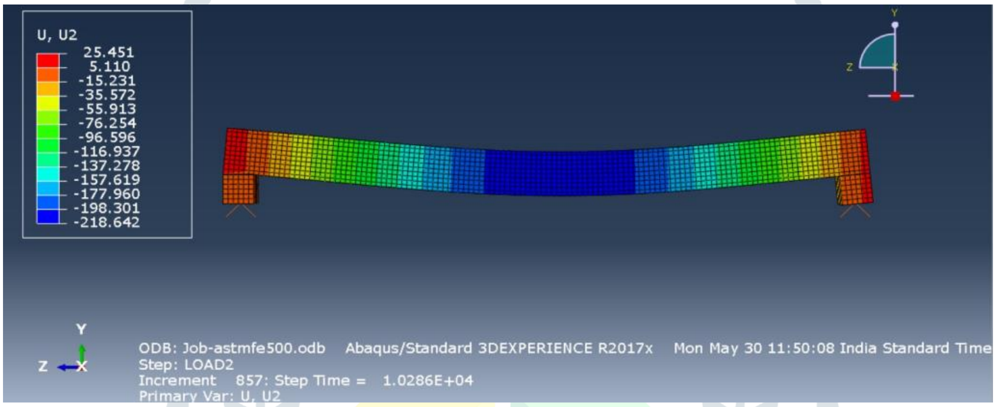

Fig 5.2 Mid-span deflection for beam B1 exposed to ASTM-E119 curve

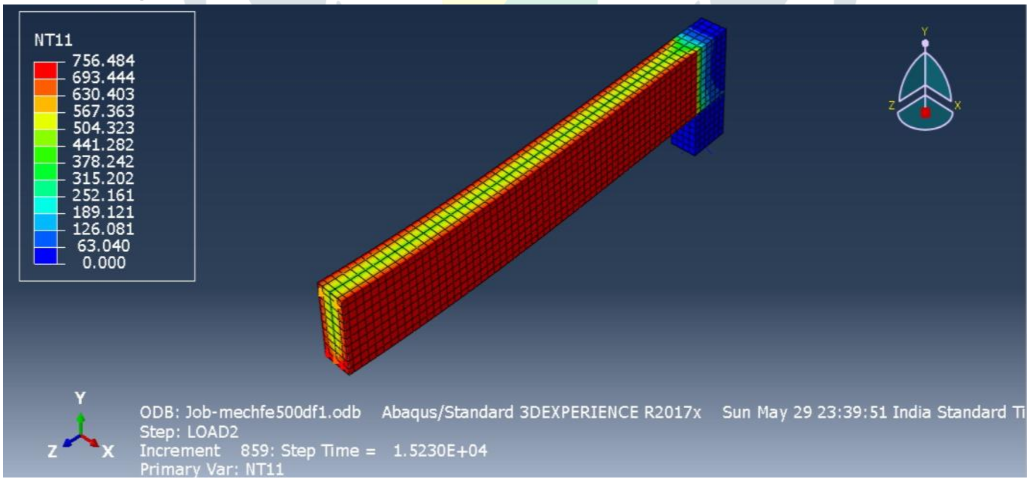

Fig 5.3 Nodal temperature of beam B1 exposed to parametric curve

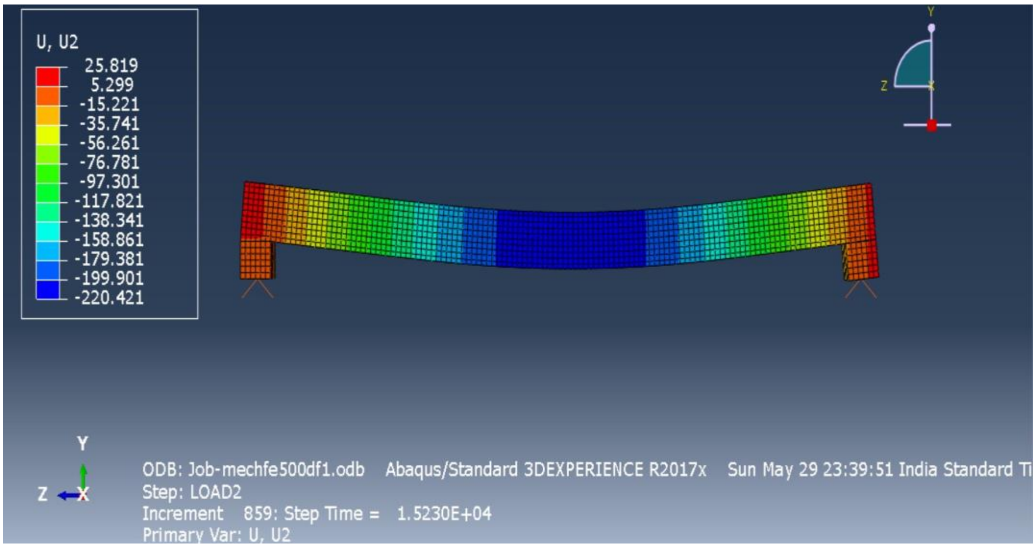

Fig 5.4 Mid-span deflection for beam B1 exposed to parametric curve

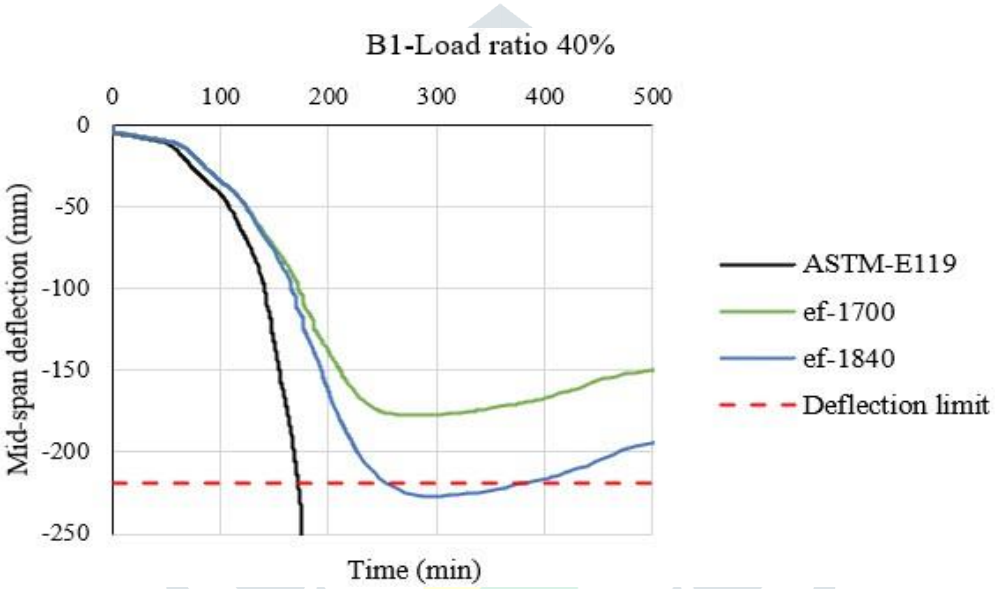

Fig 5.5 Mid-span deflections for beam B1 for load ratio 40%

B1-Load ratio 40%

- The beam B1 failed during 2.8 hrs with mid span deflection of 218.5 mm for ASTM E119 fire exposure.
- The beam B1 failed during 4.2 hrs with mid span deflection of 218.5 mm for design fire ef-1840 with heating phase of 2.1 hrs

The limiting fire load for this beam is 1840 MJ/m^2

ii. RC Beam B1 exposed to ASTM E119 standard and parametric time-temperature curve for Load ratio 60%

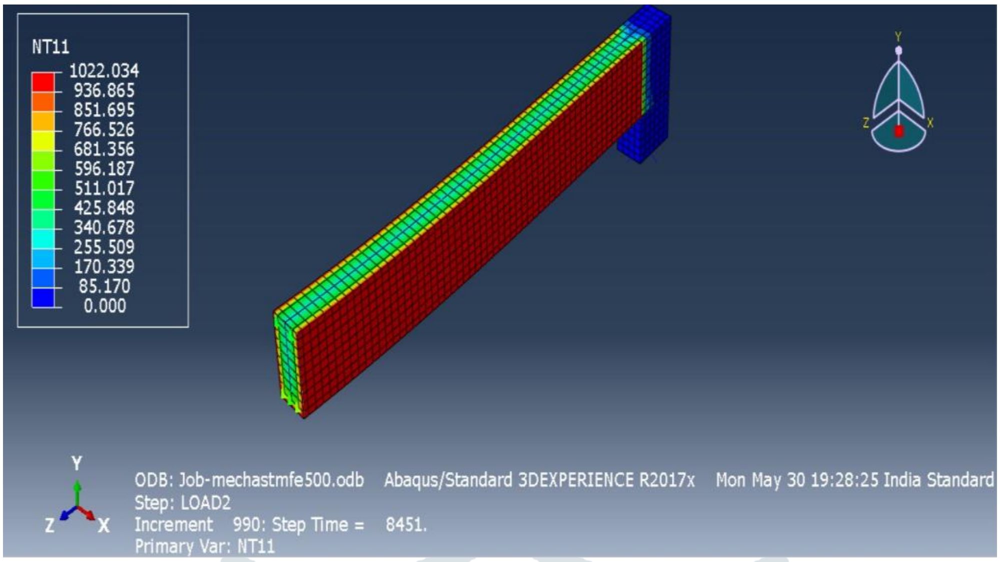

Fig 5.6 Nodal temperature of beam B1 B1 exposed to ASTM-E119 curve

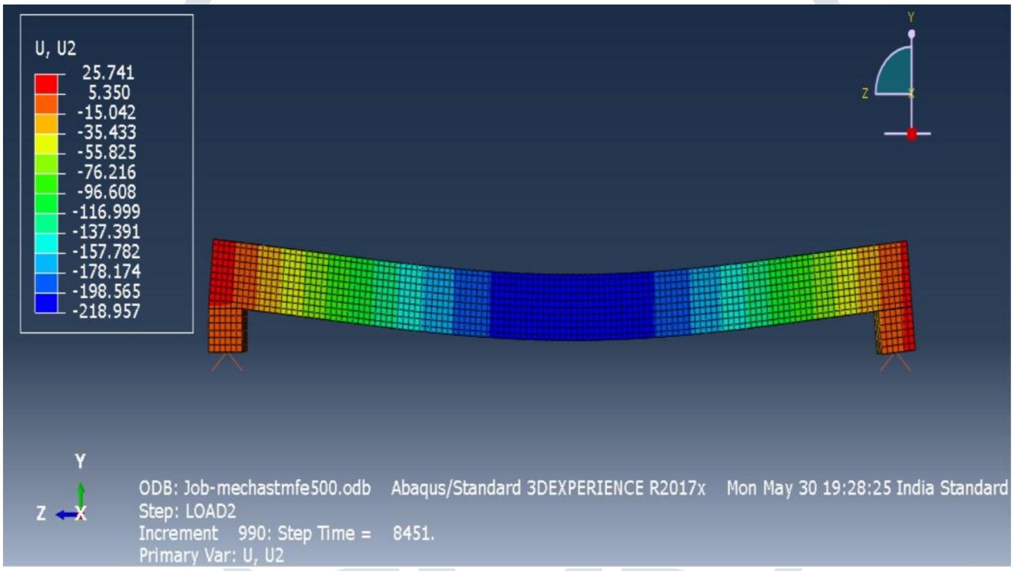

Fig 5.7 Mid-span deflection for beam B1 exposed to ASTM-E119 curve

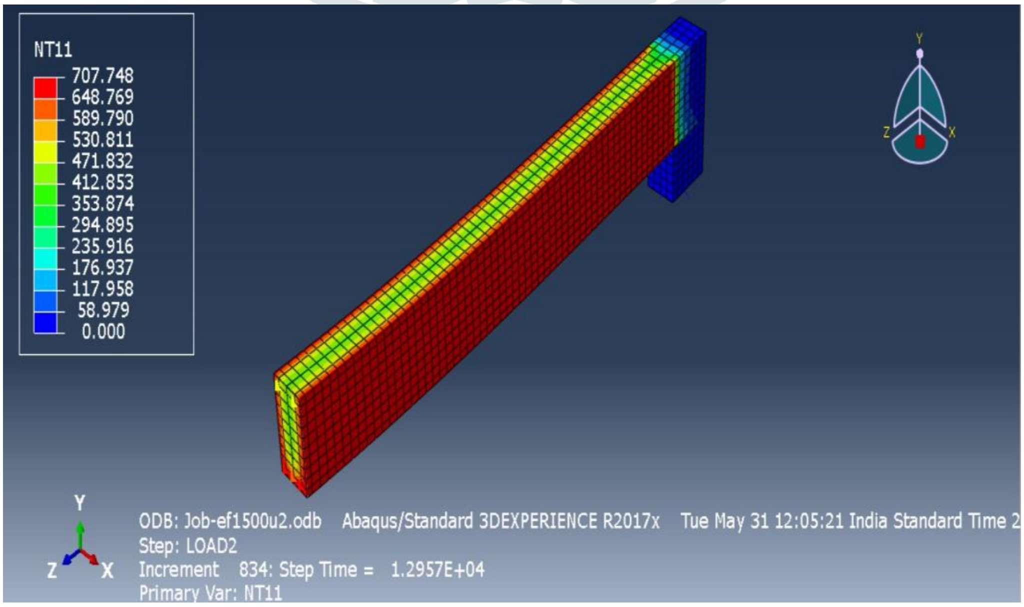

Fig 5.8 Nodal temperature of beam B1 exposed to parametric curve

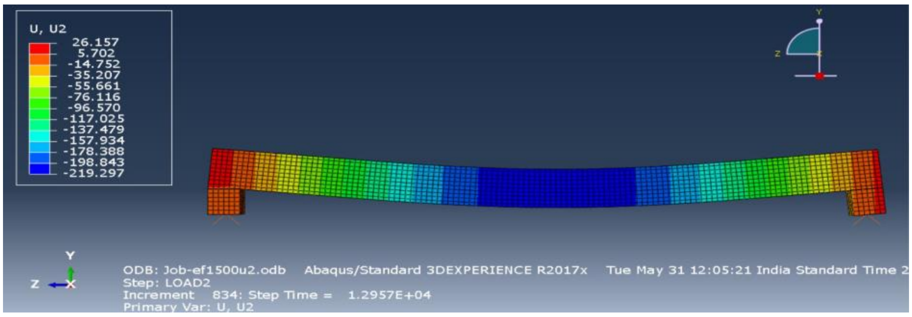

Fig 5.9 Mid-span deflection for beam B1 exposed to parametric curve

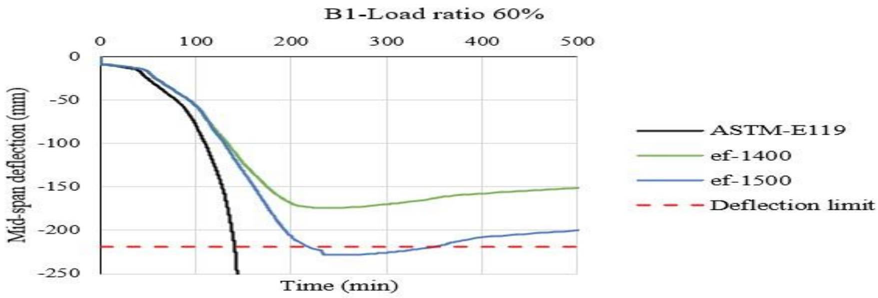

Fig 5.10 Mid-span deflections for beam B1 for load ratio 60%

B1-Load ratio 60%

TABLE I. The beam B1 failed during 2.3 hrs with mid span deflection of 218.5mm for ASTM E119 fire.

TABLE II. The beam B1 failed during 3.6 hrs with mid span deflection of 218.5mm for design fire ef-1500 with heating phase of 1.7 hrs.

TABLE III. The limiting fire load for this beam is 1500 MJ/m^2

TABLE IV. **Table 5.1 Results from the parametric study**

Code	Beam Number	Beam dimensions (mm)			Code F.R (hrs)	Actual Fire Resistance (hrs)				DHP (hrs)		Fire load (MJ/m^2)		Failure criteria	
						ASTM - E119		Parametric fire		Parametric fire					
		B	D	CC		LR 40 %	LR 60 %	LR 40 %	LR 60 %	LR 40 %	LR 60 %	LR 40 %	LR 60%	mm	mm/min
IS 456	B1	200	450	20	1.50	2.8	2.3	4.2	3.6	2.1	1.7	1840	1500	218.40	9.70
	B2	200	450	40	2.00	3.6	2.6	5.3	4.8	2.4	1.8	2100	1600	229.00	10.20
	B3	240	550	60	3.00	5.1	4.4	7.7	6.7	3.2	2.5	2800	2200	190.67	8.47
	B4	280	550	70	4.00	6.0	4.9	7.9	6.9	4.2	3.2	3600	2800	194.80	8.65
EN-2	B5	200	450	45	1.50	3.8	2.7	5.2	4.2	2.7	1.9	2300	1700	232.55	10.33
	B6	240	550	60	2.00	5.1	4.4	7.7	6.7	3.2	2.5	2800	2200	190.67	8.47
	B7	300	550	70	3.00	6.4	5.2	8.1	7.6	4.5	3.5	3900	3000	194.80	8.65
	B8	350	550	80	4.00	8.3	6.8	10.6	8.9	7.5	5.0	6400	4300	199.11	8.84
NOTE: B- Width, D – Depth, CC- Clear Cover, LR – Load Ratio.															

iii. Fire resistance of the RC beams subjected to ASTM E-119 time-temperature curve:

The actual fire resistance of the RC beams subjected to the standard curve determined for 40% and 60% load ratio is presented in Table 5.1 The RC beams B1 to B8 considered in the present study are observed to have higher fire resistance capacity for all the load ratios than the fire resistance specified by IS 456:2000 and EN-2. The ratio of actual fire resistance to the code-specified fire resistance for all exposure conditions is represented in Fig 5.43 Further, the actual fire resistance capacity of the RC beams considered as per IS 456:2000 is less than the RC beams considered as per EN2 specifications. The difference between the actual fire resistance of reinforced concrete beams is owing to the increased beam width and clear cover of the RC beams. Furthermore, the actual fire resistance capacity of the RC beams reduces with an increase in load ratio and beam B4 is observed to have only a 23% higher fire resistance capacity at 60% load ratio than the code-specified fire resistance.

The actual fire resistance of the RC beams subjected to the standard curve determined for 60% and 80% load ratio is presented in Table 5.1 The RC beams B1 to B8 considered in the present study are observed to have higher fire resistance capacity for all the load ratios than the fire resistance specified by IS456:2000 and EN2 except B2 for load ratio 80%. The ratio of actual fire resistance to the code-specified fire resistance for all exposure conditions is represented in Fig 5.44 Further, the actual fire resistance capacity of the RC beams considered as per IS456:2000 is less than the beam's considered as per EN2 specifications and the difference in the actual fire resistance of reinforced concrete beams is owing to the increased beam width and clear cover of the RC beams. Furthermore, the actual fire resistance capacity of the RC beams reduces with an increase in load ratio and beam B4 is observed to have only a 2.43 % higher fire resistance capacity at 80% load ratio than the code-specified fire resistance.

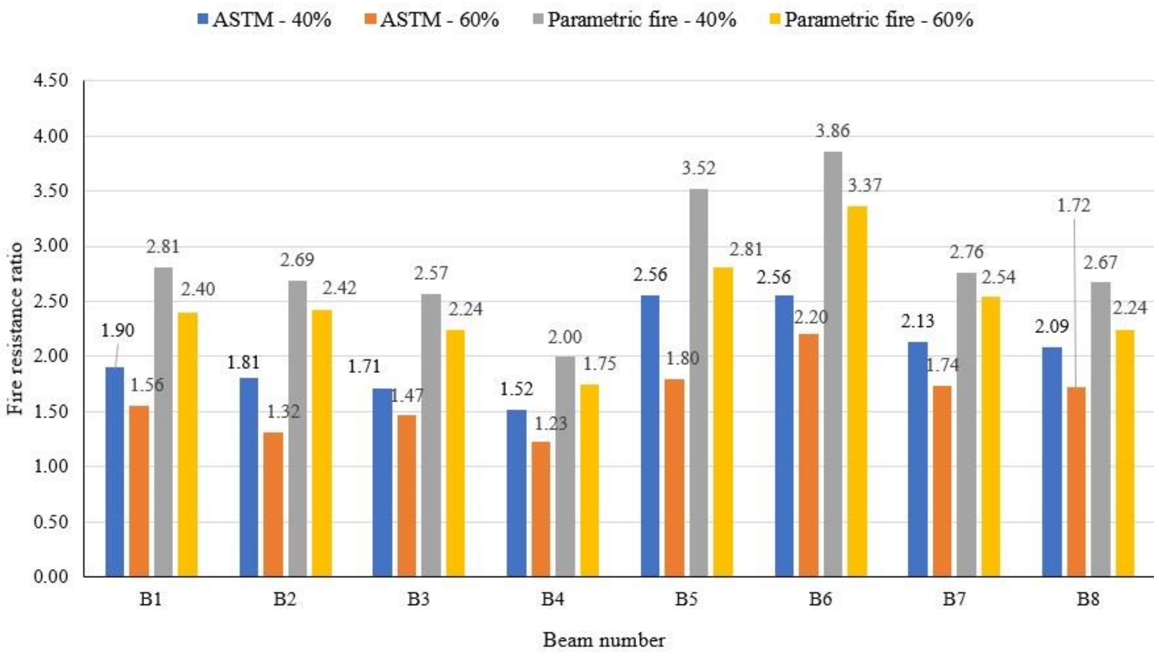

Fig. 5.43 The ratio of actual and code-specified fire resistance of RC beams

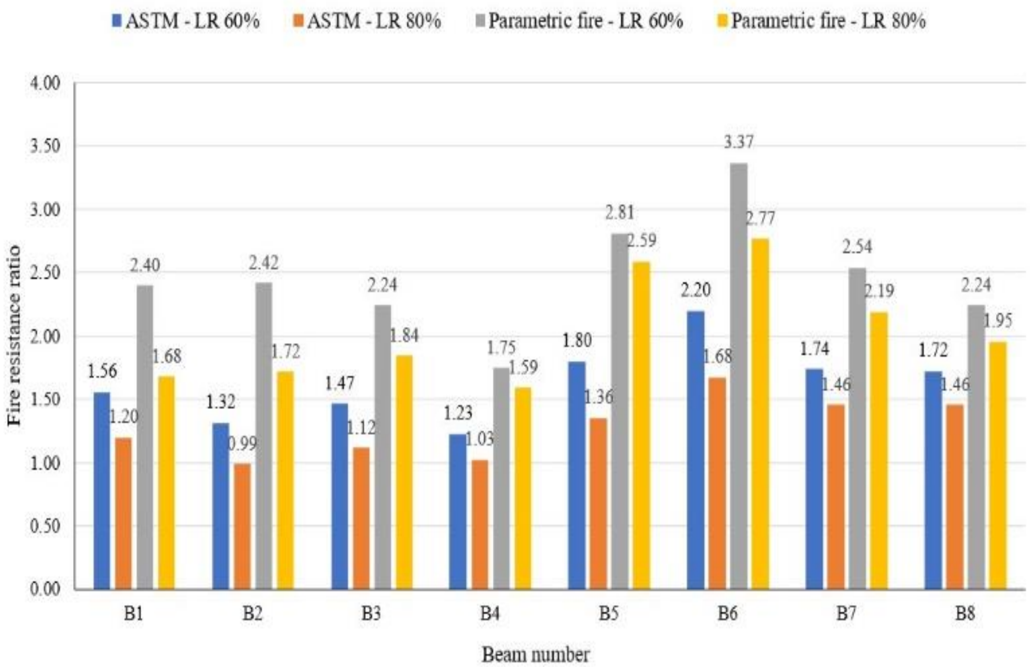

Fig 5.44 The ratio of actual and code-specified fire resistance of RC beams

iv. Performance of RC beams subjected to parametric time-temperature curves:

The performance of RC beams B1 to B8 exposed to parametric time-temperature is determined and fire resistance of RC beams to parametric fire conditions is tabulated in Table 5.1. It is observed from Table 5.1 that, the actual fire resistance of RC beams exposed to the parametric time-temperature curve is higher than the actual fire resistance capacity of the RC beams subjected to the standard time-temperature curve for both 40% and 60% load ratios. It is noted that the fire resistance capacity of reinforced concrete beams B5 and B6 exposed to parametric time-temperature fire at 40% load ratio is 3.52 and 3.86 times higher than EN-2 specified fire resistance. It may be noted that, the RC beams subjected to parametric time-temperature curve is failed during the cooling phase of the curve unlike in the standard fire exposure. Hence, the duration of the heating phase (DHP) of each beam is determined and presented in Table 5.1. It is observed that the DHP of each beam is close to the standard fire resistance specified in IS456:2000. A ratio of DHP to Standard Fire Resistance (SFR) is presented in Fig 5.45 to understand the reduction in DHP from the SFR. The DHP of RC beams B2, B3, and B4 is 7%, 14%, and 18% lower than the standard fire resistance given by IS 456:2000. This indicates that the IS 456:2000-considered beams B2 to B4 will fail for an exposure length shorter than the fire resistance stipulated by design standards. However, it may be noted that the time of failure is larger than the standard fire resistance capacity of the (RC) reinforced concrete beams. Further, the DHP of all the beams is reduced with the increase in load ratio to 60%. The limiting fire load obtained by exposing each beam to various parametric time-temperature curves is presented in Table 5.1.The limiting fire load value for the considered beam is increasing with the increase in the standard fire resistance of the beam. The limiting fire load capacity of the beams B1 to B4 considered as per IS 456 has a lower limiting fire load capacity than the beams considered as per EN-2. Further, the percentage reduction in limiting fire load for B1 to B8 beams with an increase in load ratio to 60% is shown in Fig 5.46 The Beam B8 is noticed to have 32.81% lower fire load capacity during the 60% load ratio scenario.

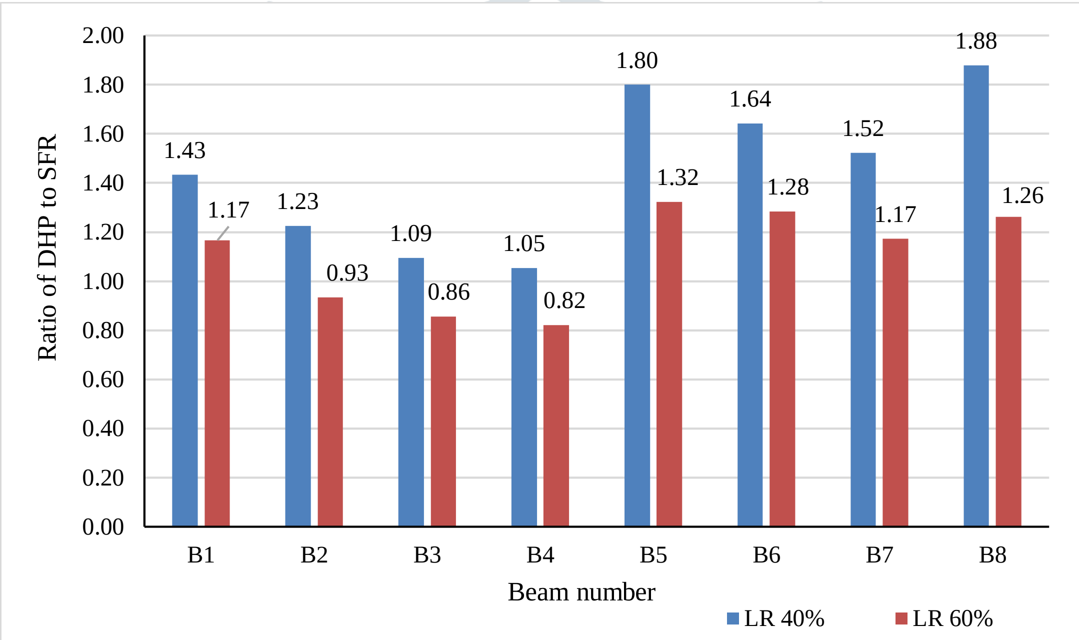

Fig 5.45 The ratio of Duration of Heating Phase (DHP) to Standard Fire Resistance (SFR)

Fig 5.46 Percentage reduction in fire load at 60% load ratios

The performance of RC beams B1 to B8 exposed to parametric time-temperature is determined and fire resistance for parametric fire conditions is tabulated in Table 5.1. Observations from Table 5.1 indicate that the RC beams actual fire resistance during the exposure to the parametric time-temperature curve is higher than the actual fire resistance capacity of the RC beams subjected to the standard time-temperature curve for 60 % and 80% load ratios. It is noted that the fire resistance capacity of (RC) reinforced concrete beams B5 and B6 at a 60% load ratio is 2.81 and 3.37 times higher than EN2 specified fire resistance after exposure to parametric fire respectively. It may be noted that the beams subjected to parametric time-temperature fire failed in the cooling phase of the curve unlike in the standard fire exposure. Hence, the duration of the heating phase (DHP) of each beam is determined and presented in Table 5.1. It is observed that the DHP of each beam B5, B6, B7 and B8 is very close to the standard fire resistance specified in

IS456:2000 for 80% load ratio. A ratio of DHP to Standard Fire Resistance (SFR) is presented in Fig 5.47 to understand the reduction in DHP from the SFR. The DHP of RC beams B2, B3, and B4 is 18 %, 34 %, and 36 % lower than the standard fire resistance given by IS456:2000 for 80% load ratio. This indicates that the IS456:2000-considered beams B2 to B4 will fail for an exposure length shorter than the code's stipulated standard fire resistance. However, it may be noted that the time of failure is larger than the standard fire resistance capacity of the (RC) reinforced concrete beams. Further, the DHP of all the beams is reduced with the increase in load ratio to 80%. The limiting fire load obtained by exposing each beam to various parametric time-temperature curves is presented in Table 5.1. The limiting fire load value for the considered beam is increasing with the increase in the standard fire resistance of the beam. The limiting fire load capacity of the beams B1 to B4 considered as per IS456 has a lower limiting fire load capacity than the beams considered as per EN2. Further, the percentage reduction in limiting fire load for B1 to B8 beams with an increase in load ratio to 80% is shown in Fig 5.48. The Beam B8 is noticed to have 18.60 % lower fire load capacity during the 80% load ratio scenario.

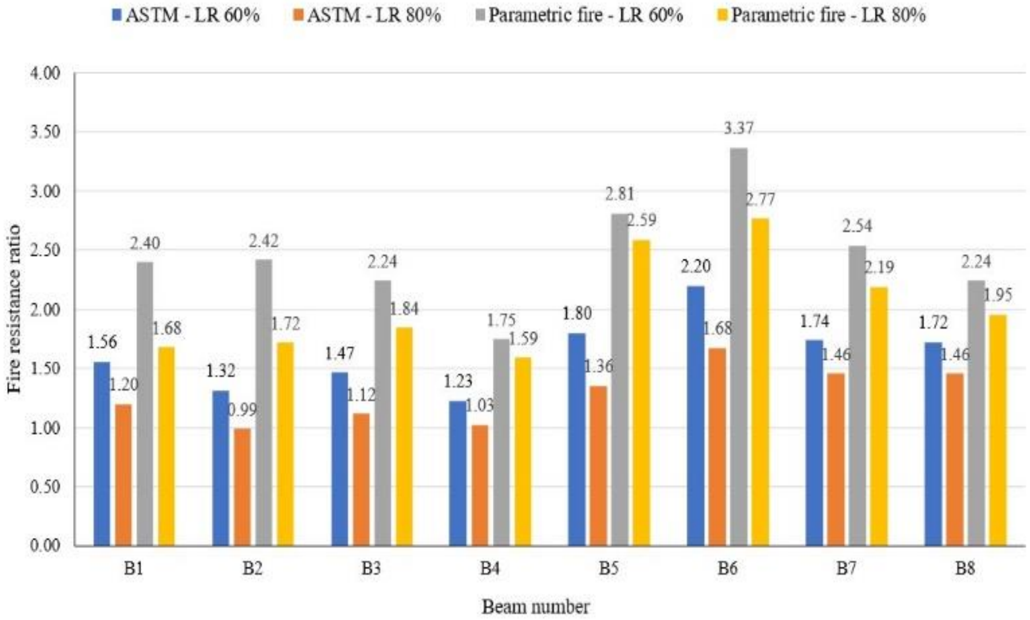

Fig 5.47 The ratio of Duration of Heating Phase (DHP) to Standard Fire Resistance (SFR)

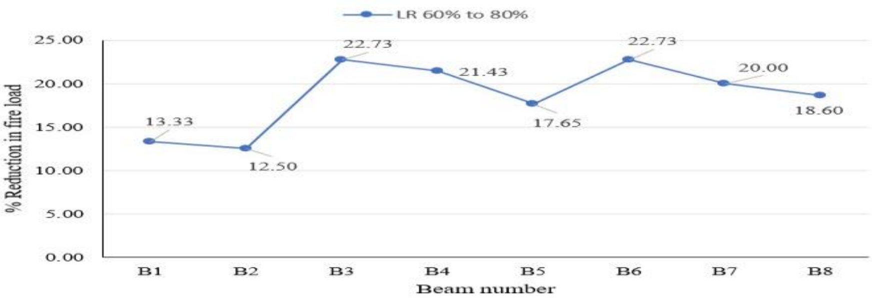

Fig 5.48 Percentage reduction in fire load at 80% load ratios

v. Limiting fire load on RC beams

Observations from Table 5.1 shows the limiting fire load for B1 to B8 RC beams with 40%,60% and 80% load ratios. The limiting fire load and the time at the failure of the RC beams are determined for all the exposure scenarios. A limiting fire load is the amount of combustible material that can be stored under the beam or inside the compartment of the present study. In beam B1 limiting fire load is 1840 MJ/m2,1500 MJ/m2,1300 MJ/m2 for load ratio 40% , 60% and 80%.Similarly for beam B8 limiting fire load is 6400 MJ/m2,4300 MJ/m2 and 3500 MJ/m2 for load ratio 40%, 60% and 80%.Therfore limiting fire load decreases as the load ratio increases. Limiting fire load increases with increase of width, depth and clear cover.

VI. CONCLUSIONS

The actual fire resistance of reinforced concrete beams B1 to B8 is more than the standard fire resistance specified by IS456:2000 and EN2. The actual fire resistance of RC beams considered as per IS456:2000 is less compared to the beams considered as per EN-2 in all load ratios. This is owing to the increased beam width and clear cover of the beams. The RC beams B5 and B6 at a 40% load ratio are 3.52 and 3.86 times higher than the standard fire resistance specified in EN2.The duration of the heating phase (DHP) of each reinforced concrete beam is close to the standard fire resistance mentioned in IS456:2000 and EN-2 standards. The DHP of RC beams B2, B3, and B4 is consistently reduced to 7%, 14%, and 18% than the standard fire resistance given by IS456:2000. This means the beams B2 to B4 beams considered as per IS456:2000 will fail for an exposure duration less than the code-specified standard fire resistance. It is evident from the present study that, the reinforced concrete beam's fire resistance may reduce with the increase in the fire load and load ratios. The fire resistance of the reinforced concrete beams subjected to parametric time-temperature curves needs to be checked both during the heating phase along with the cooling phase of the curve to obtain the criticality of the scenario.

In the present validation program, the RC beam was modeled using the procedure described in the preceding sections, and the results of the finite element analysis and the results of the experiment were compared. Fig 3.4 and Fig 3.6 shows the cross-sectional temperature and mid-span deflection obtained during the FE analysis in ABAQUS software. The validation of Beam B1 is done by comparing the cross-sectional temperature Fig 3.5 and mid-span deflection Fig 3.7 obtained from the present study with the experimental results. The concrete and reinforcing steel in beam B1 subjected to ASTM E119 standard fire time-temperature curve caused temperatures to rise gradually until failure, can be observed. It is evident that mid-span deflections rise during the initial phases of fire exposure as a result of the deterioration of the strength and stiffness characteristics of concrete and the reinforcing steel with an increase in temperature. Since temperatures in beam B1 rise gradually for the duration of the fire exposure, failure happens after 180 minutes due to considerable deterioration in the characteristics of the reinforcing steel and concrete. In numerical simulations, the beam failed in a flexural mode, which is similar to the failure mode observed in experiments. In the instance of temperature change over time, Fig 3.5 displays nearly coinciding experimental and modeling data at three particular locations (i.e., quarter depth, mid-depth, and bottom rebar). In general, the suggested model's mid-span deflection matches the experimental deflection and is observed to be on the conservative side during the fire exposure Fig 3.7.

Fig 3.8 and Fig 3.10 shows the cross-sectional temperature and mid-span deflection obtained during the FE analysis in ABAQUS software. The validation of Beam B2 is done by comparing the cross-sectional temperature Fig 3.9 and mid-span deflection Fig 3.11 obtained from FE analysis with that of experimental results. The concrete and reinforcing steel in beam B2 subjected to a short design curve caused temperatures to increase is slightly higher than in the case of beam B1 during the heating phase of the fire. This is due to the rapid increase in fire temperature during the initial stages of fire exposure. Cross-sectional temperatures in the beam begin to decrease after reaching a maximum value as a result of the decay phase present in this design fire exposure. For beam B2, however, the temperatures encountered in the concrete and reinforcing steel of the beam are significantly lower due to the cooling phase, and recovery in mid-span deflection is noted during the fire's decay phase. Fig 3.11 shows that the mid-span deflections of the model and the deflection during the fire test experiment agree quite well. In the instance of temperature change over time, the figure displays nearly coinciding experimental and modeling data at three particular locations (i.e., quarter depth, mid-depth, and bottom rebar) Fig 3.9. In general, the suggested model's mid-span deflection matches the measured deflection response pattern and is observed to be on the conservative side during the fire exposure Fig 3.11. The minor differences in cross-sectional temperature and mid-span deflection in the model can be due to minor differences in idealization considered in the analysis, such as the stress-strain relationship of concrete and steel.

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