FINITE ELEMENT ANALYSIS OF THE BEHAVIOR OF RC BEAMS DURING FIRES

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ABSTRACT

A detailed 3-Dimensional time domain transient thermal stress finite element analysis was carried out to study the performance of reinforced concrete beams exposed to fire attack. A FE model that represented a reinforced concrete beam continuous over one support was developed. The beam was subjected to ASTM E119 standard fire exposure to the bottom and side surfaces, in the form of transient temperatures versus time, while maintaining constant transverse loading on top surface. Material nonlinearity was taken into account because of the changes in material properties experienced in fire. The more complicated aspects of structural behavior in fire conditions, such as thermal dilation, cracking or crushing of concrete, and yielding of steel were modeled. The validation of the applicability of the FE model was illustrated by comparing the finite element solution with the results of experimental testing carried out for similar RC beams within the same boundary conditions. The FE analysis showed that a reinforced concrete flexural member properly designed to fail in flexure at ambient conditions, would experience the same mode of failure when exposed to realistic fire exposures.

Keywords: Thermo-mechanical response, thermal dependent behavior, FE analysis, RC beams.

INTRODUCTION

Predicting the thermo-mechanical response of a reinforced concrete structure subjected to fire, is a highly nonlinear problem. In addition to geometric nonlinearity, when large thermal deformations at elevated temperatures are associated with mechanical deformations, material nonlinearity is reflected in the temperature dependent thermal and mechanical properties of steel reinforcement and concrete.

The extent of strength and stiffness degradation in fire exposed concrete members depends on a number of factors, including type of exposure, properties of concrete and steel reinforcement, load level and boundary conditions (Kodur and Agrawal, 2015).

Fire safety for reinforced concrete structures is one of the primary considerations in building applications. The provision of fire resistance is usually treated indirectly in structural design by applying certain conditions such as minimum concrete cover, minimum section dimensions. According to Ellingwood and Lin (1991), fire ratings of structural components in the United States are measured by tests conducted in accordance with ASTM E119 (2002).

Zhaohui *et al.* (1999) stated that in terms of reinforced concrete construction, design against thermal loading is still based on simplistic methods, which have been developed from standard fire tests that do not necessarily represent real building behavior. This makes it very difficult, if not impossible, to determine the level of safety achieved in real concrete structures, or whether an appropriate level of safety could be achieved more efficiently.

The need to design durable concrete structures, leads ever more to sophisticated modeling of deterioration phenomena. At the structural scale, elevated temperatures induce restrained thermal dilation due to kinematic restrained. At the material scale, thermal load induces strong micro-structural changes that alter the concrete behavior (Nechnech *et al.* 2015). There has been an increasing recognition of the benefits of employing performance based fire design, in comparison with prescriptive approaches, which are based on unrealistic idealizations (Elgazouli *et al.* 2009). According to Kowalski (2010), concrete mechanical properties are significantly influenced by high temperature, mainly above 400 to 500 ^{o}C. Knaack *et al.* 2011 indicated that there is a need for predictive, performance based structural fire design standards as an alternative to the current perspective to the design methodology. Indeed such need lead, even at earlier stages, to extensive studies on the mechanical properties of the materials used in construction, considering various variables, such as the different constituents used in concrete, concrete and steel reinforcement of different grades, the boundary conditions and loading, including thermal loading. In this regard, Chen *et al.* 2016 carried out an experimental program to investigate the mechanical properties of both high strength steel and mild steel at elevated temperatures, the elastic moduli and yield strengths were obtained at different strain levels, and the ultimate strength and thermal elongation were evaluated at different temperatures. Their work also involved presenting tables for temperature dependent material models of high strength steels.

Elghazouli *et al.* 2009 presented experimental evaluation of the mechanical properties of steel reinforcement at elevated temperatures. They also proposed temperature dependent material models for steel reinforcement. Knaack *et al.* 2011 proposed temperature dependent material models for concrete.

The availability of temperature dependent material models, for the different constituents used in reinforced concrete, paved the way towards authentic fire behavior modelling of structures by means of computer aided design, as the use of specialized computer codes can significantly increase the profitability of the project works and increase their efficiency. The weakness in case of modelling is that the degradation of material properties is simplified (Kudryashov *et al.* 2012).

At early stages, a number of researchers worked on developing structural modelling approaches such as Lie and Celikkod (1991), who developed a model for the high temperature analysis of circular reinforced concrete columns. Huang and Platten (1997) developed planar modelling software for reinforced concrete members in fire.

The simulation of heat exposed RC structures in fire requires the solution of an hygro-thermo-mechanical problem. If spalling is disregarded, only the thermo-mechanical problem can be dealt with (Dahmani and Kouane, 2012).

The basis for thermal analysis, assuming no heat generation, is a heat balance equation obtained from the principal of conservation of energy (Bamonte and Monte, 2015).

$$k\frac{\partial^2 T}{\partial x^2}+k\frac{\partial^2 T}{\partial y^2}+q=\rho.c.\frac{\partial T}{\partial t} \tag{1}$$

Where:
c: specific heat coefficient.
k: thermal conductivity coefficient.
q: rate of heat generated internally per unit volume.
ρ: density.
T: temperature.
t: time.

For heat transfer analysis of an RC beam exposed to fire, internal heat generation is inactive, thus q can be neglected.

The governing equation becomes

$$k\frac{\partial^2 T}{\partial x^2}+k\frac{\partial^2 T}{\partial y^2}=\rho.c.\frac{\partial T}{\partial t} \tag{2}$$

Assuming the temperature distribution in the structure as independent of the structural behavior, allows performing thermal analysis and structural analysis in two consecutive steps (Feist and Aschaberb, 2015).

The objective of this work is first to find acceptable results for thermal and mechanical response of reinforced concrete beams using finite element modeling, then to study the performance of RC beams at elevated temperatures. The work involves investigating whether RC beams that are properly designed to fail in flexure at ambient temperatures would experience the same mode of failure during fires. Calculation involves two analyses, each belonging to a different physical field. Coupling is made by applying results from thermal analysis, as load to structural analysis.

ANALYTICAL METHODOLOGY

The numerical model development and transient thermo-structural analysis involved the following:

1. Constructing a 3-D finite element model of the RC beam. The model involves the geometry of concrete and rebars detailing, appropriate material models, meshing, and boundary conditions.
2. Applying the standard ASTM E119 (1991) fire exposure to the bottom and side surfaces of the beam in the form of transient temperatures versus time. The top surface, being exposed to ambient temperature. Carry out transient thermal analysis to obtain the temperature distribution in beam as a function of time.
3. Investigating the temperature distribution within the cross section. Obtain the predicted temperatures and the associated time at several locations within the beam cross section.
4. Performing structural stress analysis to obtain deflection, stress and strain results. The temperature distribution along the beam from the aforementioned thermal analysis run is applied as nodal temperature, while sustained transverse loading is applied at the beam top surface.
5. Evaluating the deflection, total strains comprising mechanical and thermal strains and the associated stresses within beam cross sections, due to the applied transient thermal loading. Furthermore tracing crack initiation, crack progression up to failure, and ultimately identifying the mode of failure.

FINITE ELEMENT MODEL

Geometry

The FE model of the RC beam has the same geometry, configuration and dimension as the specimens tested by Ellingwood and Lin (1991), (Figure-1). The RC beam has a 6.1 m span and 1.8 m cantilever. The rectangular section width is 240 mm, and depth is 560 mm. There are two layers of bottom reinforcement, each involving two 20 mm diameter rebars. In addition, there are two layers of top reinforcement each containing two 25 mm rebars. Rebar cover is 40 mm. Bar diameters and lengths are given in Table-1. Stirrups diameter is 10 mm. The values of P, P_o shown in Figure-1, are 44.5 kN and 115kN, respectively.

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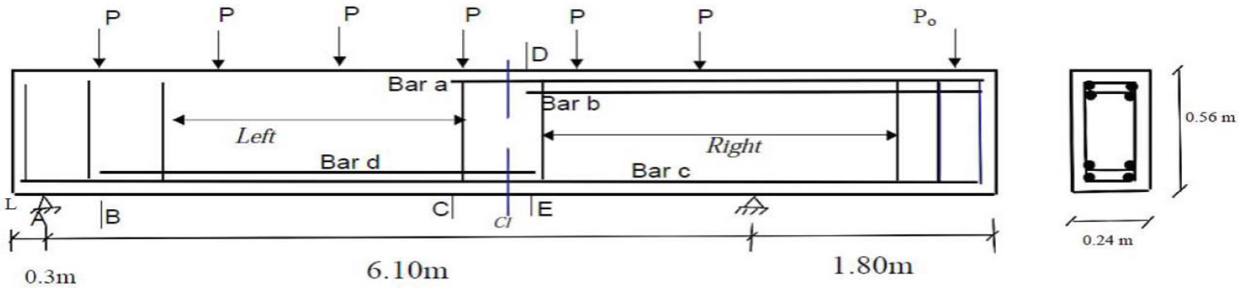


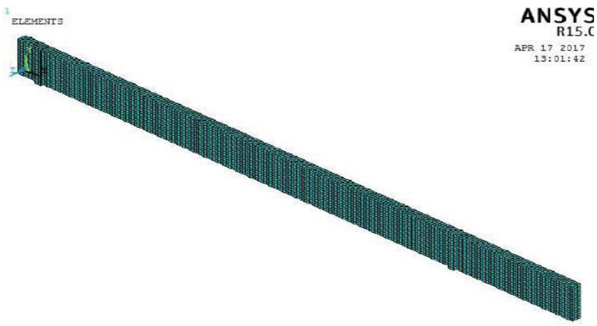
Figure-1. Geometry and reinforcement bars detailing of the RC beam.

Table-1. Details of reinforcement.

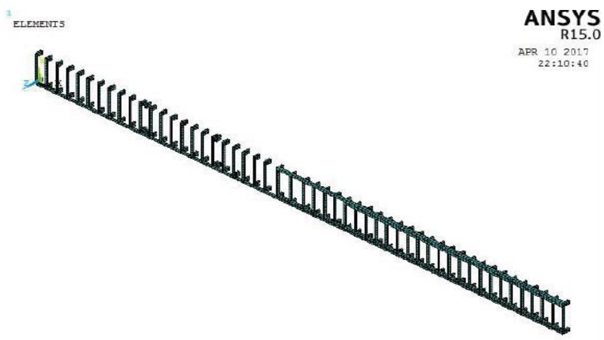
Bar Size (mm)				Bar End Location from L (m)					Stirrup Spacing (mm)	
a	b	c	d	A	B	C	D	E	Left	Right
25	25	20	20	0.04	0.64	3.76	4.74	4.88	200	160

Finite elements

Due to symmetry about a vertical plane at b/2 along the length of the beam, in geometry, loading, boundary conditions and material, it was decided to build a half beam model in order to save the computational time, (Figure-2).



(a) Isometric view of the half beam FE model.



(b) Isometric view of the Rebars and Stirrups for half beam FE model.

Figure-2. FE model of RC beam.

The FE model was built in Ansys15 (2013). Concrete was represented by the solid70 element for heat conduction in thermal analysis. It is a three dimensional eight noded tetrahedral element having thermal degrees of freedom. The distributions of thermal elastic stress components were then calculated by switching the solid70 thermal element to solid65 structural element, when performing structural analysis. The solid65 element models the nonlinear response of reinforced concrete.

The behavior of the concrete material was based on a constitutive model for the triaxial behavior of concrete after Williams and Warnke (1996). It is a nonmetal plasticity model with isotropic hardening and non-associated plastic flow. The criterion for failure of concrete due to a multi axial stress is expressed in the form:

$$\frac{F}{f_c} - S \geq 0 \tag{4}$$

Where:

F: a function of principal stress state

S: failure surface

fc: uniaxial concrete strength.

If equation 4 is satisfied, the material will crack or crush. Solid65 is capable of plastic deformation, cracking in three orthogonal directions at each integration point. The cracking is modelled through an adjustment of the material properties that is done by changing the element stiffness matrices.

Element Link33 was used to model the steel reinforcement. It is a uniaxial element with the ability to conduct heat between its nodes, and in structural analysis was replaced by element link180, which has a single degree of freedom at each nodal point, as shown in Table-2. A bilinear plasticity model that utilizes Von Mises yield surface with associated plastic flow and isotropic hardening, available in Ansys (2013), was adopted for the constitutive modeling of reinforcing steel.

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Table-2. Thermal and structural elements.

	Switchable elements concrete		Switchable elements Steel reinforcement	
Element	Thermal	Structural	Thermal	Structural
Type	Solid 70	Solid65	Link33	Link180
Number of nodes	8	8	2	2
Number of DOF per node	1	3	2	3

Material models

Table-3 illustrates concrete and steel reinforcement material properties at room temperature. The temperature dependent compressive stress strain curves of concrete are illustrated in Figure-3. There exists a linear elastic stage, followed by a nonlinear increase in stress representing development of microcracks in the concrete. Figure-3 illustrates the degradation of concrete stiffness and strength as the concrete temperature increases. The stress-strain relationship for concrete in tension is represented by a linear relationship, which is elastic up to failure. The temperature dependent stress strain relationship of steel reinforcement is presented in Table-4.

Table-3. Material properties for concrete and steel at ambient temperature (20 ^{o}C).

Material	Concrete	Steel reinforcement
Compressive strength	28 MPa	
Tensile strength	3.28 MPa	420 MPa
Elastic modulus	24.870 GPa	200 GPa
Poisson's ratio	0.2	0.3
Density	2400 Kg/m^3	7800 Kg/m^3
Thermal conductivity (k)	1.2W/m°c	60W/m°c
Specific heat capacity (c)	1000J/kg°k	500J/kg°k
Thermal expansion coefficient α	1.2x10-5 /°c	1.08x10-5 /°c

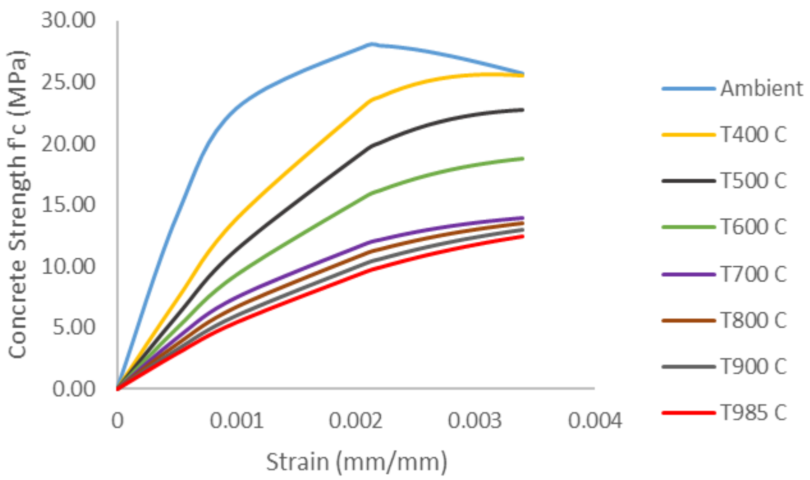

Figure-3. Temperature dependent concrete material model (Knaack, 2011).

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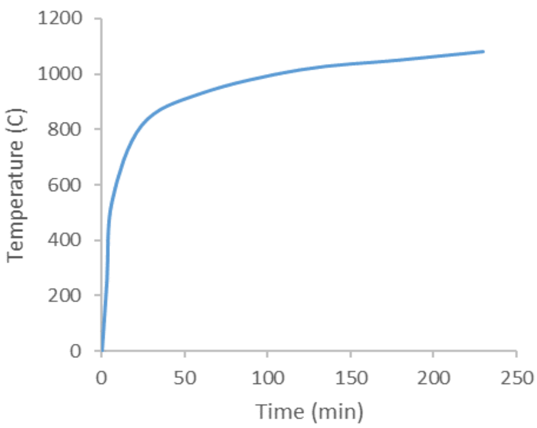
Table-4. Reduction factors at elevated temperatures for stress-strain profiles at elevated temperatures q relative to the value of f_y or E_a at 20°C (Elgazouli *et al.* 2009).

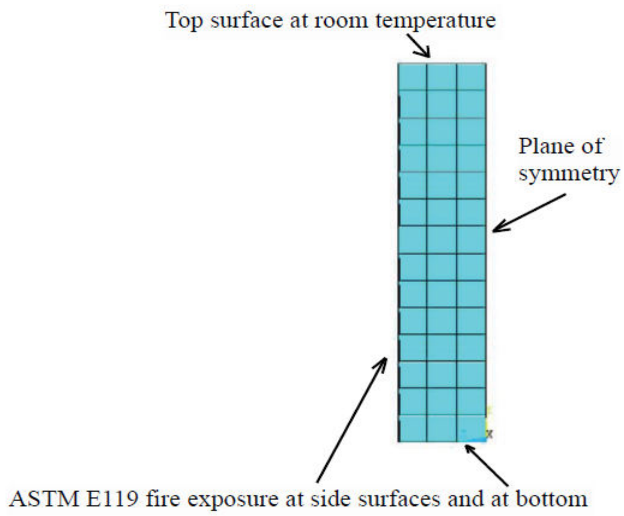
Steel temperature (t)	Reduction factor for effective yield strength	Reduction factor for proportional limit	Reduction factor for the slope of the linear elastic range
°C	$K_{y,\theta}$ ($f_{y,\theta}/f_y$)	$K_{p,\theta}$ ($f_{p,\theta}/f_y$)	$K_{E,\theta}$ ($E_{a,\theta}/E_a$)
20	1	1	1
100	1	1	1
200	1	0.807	0.900
300	1	0.613	0.800
400	1	0.420	0.700
500	0.780	0.360	0.600
600	0.470	0.180	0.310
700	0.230	0.075	0.130
800	0.110	0.050	0.090
900	0.060	0038	0.068
1000	0.040	0.025	0.045
1100	0.020	0.013	0.023
1200	0.000	0.000	0.000

Thermal and structural boundary conditions

The bottom and side surfaces of the beam are subjected to thermal load by convection with a film coefficient of 50 W/m2/c according to ASTM E119 fire exposure as shown in Figure-4. The top surface being exposed to ambient temperature. Ansys performs the numerical calculation for temperature, utilizing Galerkin finite element technique that is capable of performing heat exchange calculations (Moaveni, 2003). The thermal conductivity of the structure is taken into account. The RC beam is pin supported at the left end and roller supported at the right support, (Figure-5).



(a)ASTM E119 fire exposure curve.



(b)Surfaces subjected to fire exposure for half beam FE model.

Figure-4. Thermal boundary conditions.

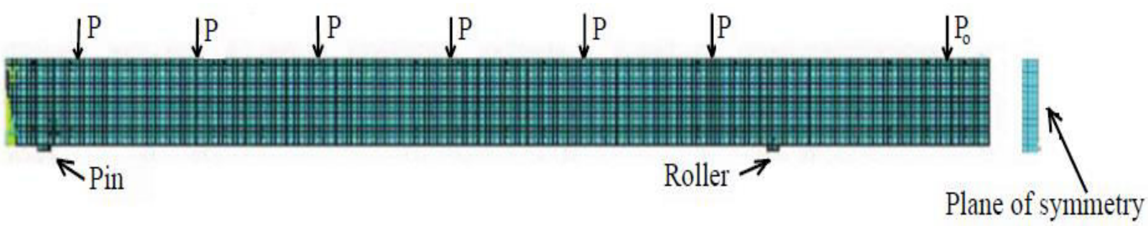


Figure-5. Model of transverse loads on FE the RC beam.

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RESULTS AND DISCUSSION

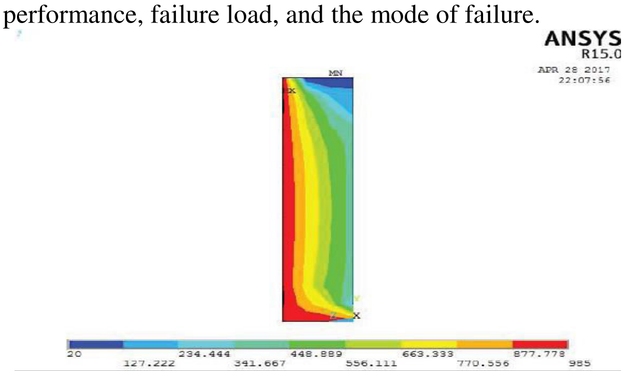
Model validation

The proposed finite element method was validated by comparing the model solution with the experimental results of Ellingwood and Lin (1991); the tested specimens have similar input parameters, geometry, detailing and boundary conditions. The specimens were tested to simulate the end span of a continuous beam, as continuity over supports permits redistribution of moments to take place during fires.

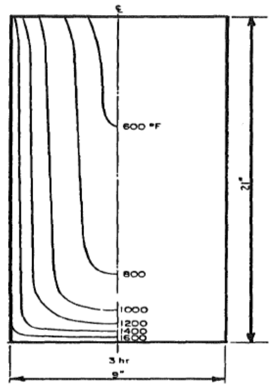
Figure-6a shows the temperature distribution along the cross section of the FE model after 210 minutes of fire exposure. The bottom and side surfaces temperatures of RC beam section are 985 Celsius, the top surface is at ambient temperature. At the vertical mid plane, which is the plane of symmetry for the FE model, the temperature varies from 20 Celsius at the top surface until 985 Celsius at the bottom surface. Figure 6-b shows the temperature distribution in degrees Fahrenheit, obtained by Ellingwood and Lin (1991) just before failure of the tested specimens.

Figure-7a illustrates the predicted deflected shape for the Finite element model. Figure-7b compares the predicted maximum deflections at the positive moment region in the FE model with the deflections, obtained by experimental testing carried out by Ellingwood and Lin (1991), along the whole period of fire exposure. The predicted deflections compare well with those obtained in tested specimens. In the FE model convergence couldn't be attained after 210 minutes of fire exposure, associated with 130 mm deflection, indicating that failure took place at a maximum deflection of 130 mm. Flexural failure occurred in the tested specimens after 220 minutes, associated with a maximum deflection of 152 mm.

The FE model analysis showed that a flexural failure took place at the maximum positive moment region after 210 minutes of fire exposure. It can be concluded that the FE solution compares reasonably well with the results obtained by Ellingwood (1991), in terms of the temperature distribution across the cross section, performance, failure load, and the mode of failure.

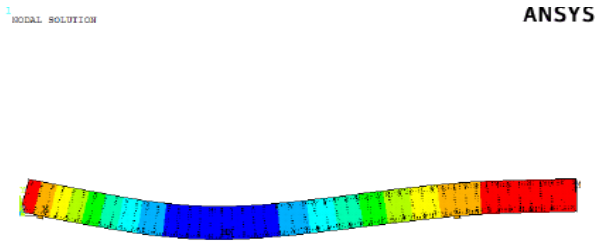


(a)Temperature distribution for half a beam FE model.

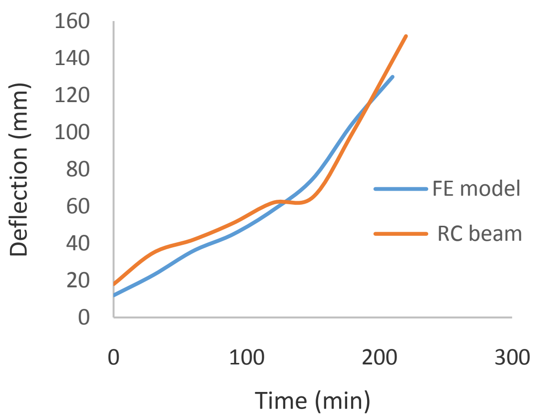


(b)Temp. distribution (°F) for the specimen (Ellingwood, 1991).

Figure-6. Temperature distribution at the end of fire exposure time.



(a)Deflected shape for FE model.



(b)Max span deflection for FE model and specimen during the exposure time.

Figure-7. Beam deflection.

FE analysis Results

The performance of the FE model of the RC beam, when subjected to ASTM E119 (2002) fire exposure, was investigated. The thermal load was applied at the bottom and side surfaces of the beam with the top surface kept at ambient temperature, while the FE model was under sustained transverse load. Figure-7a shows the predicted deflected shape of the FE model just before failure. Figure-7(b) simulates maximum span deflection for FE model of the RC beam from the initiation of thermal load until failure. It can be noted that the rate of deflection was nearly steady up to 140 minutes, subsequently the rate of deflection increased. At time 210 min of fire exposure time, solution convergence couldn't be attained, indicating that the RC beam had failed.

Figure-8 shows the increase in the average temperature in the two layers of the bottom bars and the top bars along with the fire exposure time.

Shear cracks that resulted from nonlinear thermal and mechanical strain gradients, formed above the right support. The shear cracks then extended diagonally upward and ultimately stabilized as illustrated in Figures-9(a,b). Flexural cracks formed at the positive moment region. After 140 minutes, the flexural cracks increased and extended upward.

As the nodal temperature increased with time, severe flexural cracks were observed at the positive moment region, associated with minor diagonal cracks near the supports.

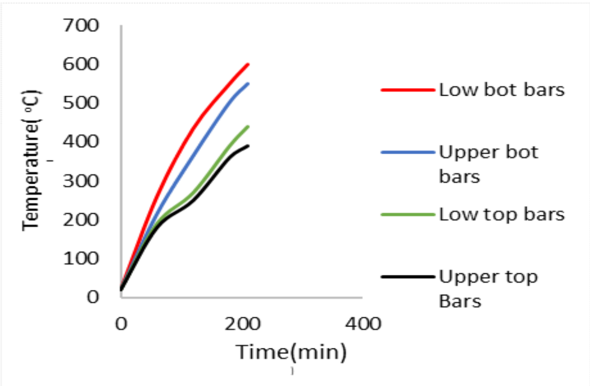
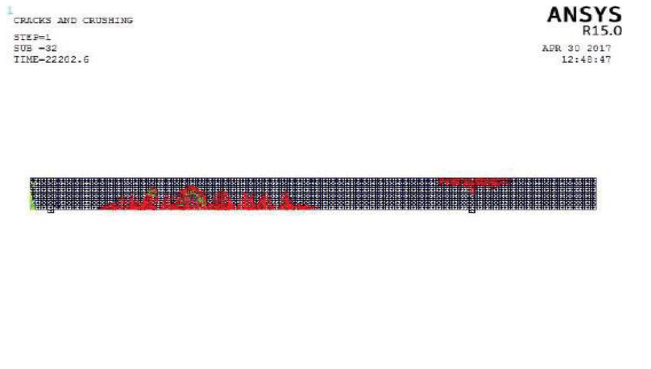
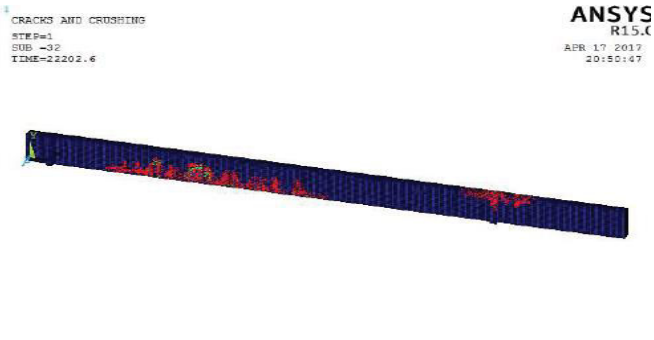


Figure-8. Temperature distribution at Rebars in the FE model.

Ultimately, the beam failed in flexure when a plastic hinge formed at the positive moment region after 210 minutes. Such mode of failure occurred because the RC beam was designed as tension controlled under-reinforced flexural member, in accordance with Building Code Requirements for Structural Concrete ACI (2014). Theoretically, flexural failure is expected to be triggered at the tension zone, when the factored applied moment exceeds the nominal strength developed by the under-reinforced section. Moreover, the temperature dependent material models for concrete and steel reinforcement clearly show that steel reinforcement is more sensitive, in terms of strength and stiffness degradation, than concrete to variations in temperatures developed during fires. Thus, the temperature history in the reinforcement is a vital factor affecting the performance of RC beam.



(a) Side view



(b)Isometric view

Figure-9. Cracks just before failure.

CONCLUSIONS

After validating the FE model, FE analysis was carried out to investigate the performance of the RC beam, exposed to ASTM E119 fire, while under sustained transverse load. The conclusions are as follows:

- Concrete compressive strength and stiffness are less sensitive to variations in temperatures typically developed during fire, than steel reinforcement.
- The significant factor affecting the behavior of properly designed concrete flexural members is the temperature history in the reinforcement.
- The high temperature gradient at elevated temperatures between the beam bottom surface, side surfaces and beam top resulted in high values of tensile and shear stresses in concrete. Such stresses provoked cracking at the negative moment and positive moment regions at early stages.
- Diagonal cracks formed and propagated towards the beam top at the -ve moment region close to the right support. Nonetheless, the amount of cracks was limited.
- Vertical flexural cracks formed at the positive moment region. The flexural cracks increased and extended towards the top as the nodal temperatures increased with time. Diagonal shear cracks appeared in regions closer to the supports. Finally, a plastic hinge formed at the positive moment region after 210 minutes and the beam failed in flexure.

- An RC beam properly designed to fail in flexure at ambient conditions will experience the same mode of failure at realistic fire exposures.

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