Advances in Computational Design, *Vol. 1, No. 1 (2016) 105-117*
DOI: http://dx.doi.org/10.12989/acd.2016.1.1.105

Prediction of response of reinforced concrete frames exposed to fire

Aneesha Balaji*, Muhamed Luquman K.[a], Praveen Nagarajan[b] and T.M. Madhavan Pillai[c]

Department of Civil Engineering, National Institute of Technology, Calicut, India

(Received August 16, 2015, Revised December 14, 2015, Accepted December 23, 2015)

Abstract. The objective of this work is to study the restraining effect in fire resistance of framed structures and to evaluate the global response of reinforced concrete frames when exposed to fire based on advanced finite element method. To study the response a single portal frame is analyzed. The effect of floor slab on this frame is studied by modeling a beam-column-slab assembly. The evolution of temperature distribution, internal stresses and deformations of the frame subjected to ISO 834 standard fire curve for both the frames are studied. The thermal and structural responses are evaluated and a comparison of results of individual members and entire structure is done. From the study it can be seen that restraining forces has significant influence on both stresses and deflection and overall response of the structure when compared to individual structural member. Among the various structural elements, columns are the critical members in fire and failure of column causes the failure of entire structure. The fire rating of various structural elements of the frame is determined by various failure criteria and is compared with IS456 2000 tabulated fire rating.

Keywords: reinforced concrete; fire safety; fire resistance; thermal analysis; thermo-structural analysis

1. Introduction

Traditionally, fire safety assessment of reinforced concrete (RC) has been based on a prescriptive single element analysis, neglecting statical redundancies and restraints to thermal expansions. Even if a structure is labeled as safe when exposed to fire within the scope of these prescriptive rules, structural engineers are not able to assess the real level of fire safety as the real global structural response is unknown. An RC structure subjected to thermal loads will develop stresses as a result of restrained thermal expansion. These stresses play a major role in causing structural distress (Vecchio 1987). The behavior of statically indeterminate structures depends not only on the behavior of structural members (at elevated temperature) but also to a great extent on the variation and distribution of internal forces within the structure (Guo and Shi 2011). The

*Corresponding author, Research Scholar, E-mail: aneeshabalaji@gmail.com
[a]M Tech student, E-mail: muhamedluquman@gmail.com
[b]Associate Professor, E-mail: praveen@nitc.ac.in
[c]Professor, E-mail: mpillai@nitc.ac.in

http://www.techno-press.com/journals/acd&subpage=7 ISSN: 2383-8477 (Print), 2466-0523 (Online)

common method of design of structures for fire is based on the study of the behavior of single elements in standard furnace test. However the real behavior of actual structure can be predicted only from the response of assembled structures due to restrained thermal expansion effects and resulting geometrically nonlinear responses. These responses cannot be captured from the study of individual members (Jowse *et al*. 2005). Therefore this paper deals with the study of global response of three dimensional (3D) RC frames exposed to fire to understand the real behavior of structures in the event of fire.

2. Methods of assessment of fire resistance

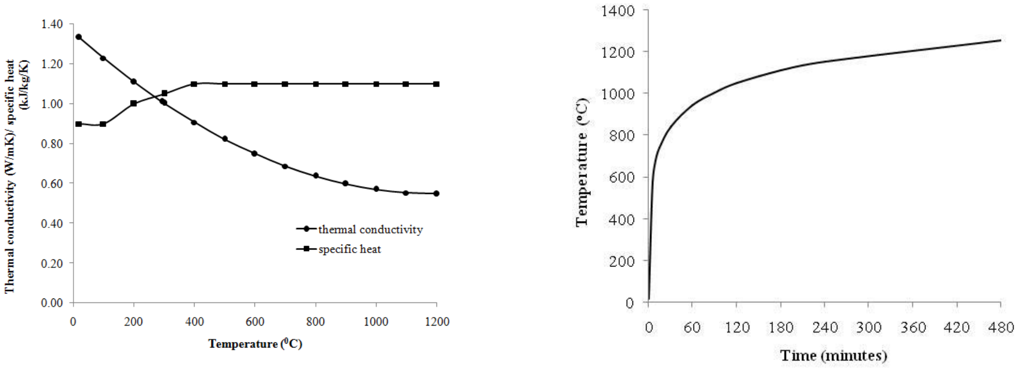
Fire safety is the ability of structures to withstand fire load and fire resistance is defined as the time for which the structures will be able to perform its function without any failure when exposed to fire. There are several methods for assessing the fire resistance of RC structures ranging different levels of complexity and accuracy. The main procedures available are (Eurocode2 2004)

1. Standard fire tests
2. Tabulated data
3. Simplified calculation methods
4. Full-scale fire tests
5. Advanced calculation methods

Standard fire test constitutes one of the most expensive assessment procedures which are mainly applied to single structural elements. The results of these tests are very limited because it is very difficult to properly reproduce in a furnace the real structural restraint, continuity and to simulate the realistic load level. Tabulated data are directly attached to a prescriptive approach of fire resistance assessment. Basically, these methods specify minimum geometric size of cross-section and concrete cover that ensure fire resistance to standard fire up to some predetermined time of exposure and most of the codes and standards prescribe these data (Eurocode2 2004, BS476: Part 20 1987, IS456 2000). Even if these data are very easy to apply especially in regular building design, it has a severe obstacle to innovative engineering and architectural solutions. The Simplified calculation methods given in Eurocode2 can be used for the analysis of single structural elements or of part of a structure. It has presented so far a significant accuracy for current structural elements such as beams, columns and slabs but is not efficient in finding global response of structures. Full-scale fire tests are rare in practice as they are highly time consuming, complex and very expensive (Pyl *et al.* 2012). Therefore the next option is advanced calculation methods basically conducted by numerical analysis which is a time-dependent thermal and mechanical analysis performed to assess the fire behavior of the whole structure, parts of the structure or simple structural elements (James and Milke 1999, Valipour and Foster 2010).

3. Numerical analysis

The numerical analysis can be done using finite element method, finite difference method, boundary element method etc. Most of the numerical analysis rely on a finite element approach which evaluates the nonlinear response of structures. So far, limited works were done for non-linear global analysis of 3D building frames when subjected to fire (Ma and Liew 2004). In this study finite element method is used and analysis is conducted using finite element software



(a) Thermal properties of concrete (b) ISO 834 fire curve

Fig. 1 Material properties and fire load for thermal analysis

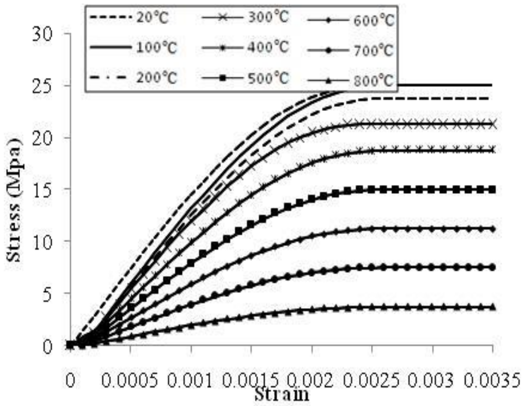
ANSYS® Release14. The analysis consists of two parts; thermal analysis and thermo-structural analysis. The first step is to calculate the transient temperature field inside the cross-sections exposed to fire and second step is to determine the mechanical response of the frame due to the effect of thermal and mechanical load (Capua and Mari 2007). SOLID 70 and SOLID 65 elements are used to model concrete in thermal and structural analysis respectively which is available in ANSYS element library. The reinforcement is modeled using LINK 33 (thermal analysis) and LINK 180 (structural analysis) elements (ANSYS 2010).

3.1 Thermal analysis

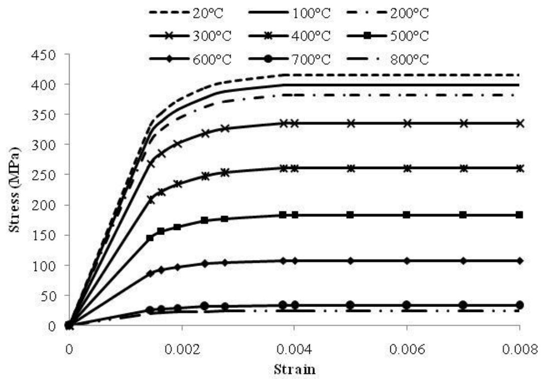
Thermal analysis is conducted to determine the heat transfer inside the member and to find the extreme temperatures in the reinforcement. All structural members exposed to fire get heated up, but the rate of temperature rise in members is different (Al-Jabri *et al.* 2008). It depends on material properties mainly thermal conductivity and specific heat of the material. These material properties are given as input data for the analysis and are taken from Eurocode2 shown Fig. 1(a) (Eurocode2 2004). The frames were analyzed by exposing to standard time-temperature curve specified as ISO 834 standard fire and is given in Fig. 1(b) (ISO 834 1999). Heat transfer from fire to element is by convection on area with a convection film coefficient of 25 W/m^{2}K. A uniform temperature of 20℃ is taken as reference temperature. A transient thermal analysis is performed and temperature at each node is evaluated for every time step.

3.2 Thermo-structural analysis

For thermo-structural analysis, "Coupled Physics Analysis" method available in ANSYS is used in which the first step is to perform thermal analysis. After conducting thermal analysis, the next step is to carry out structural analysis wherein the results from the thermal analysis (temperature at all nodes) are also entered along with structural loads. The input data are modulus of elasticity, stress-strain relationship, density and co-efficient of thermal expansion for steel and concrete. The present study is carried out using M25 grade concrete and Fe415 grade steel. The stress-strain curve for both at various temperatures used in present study is given in Figs. 2(a)-(b).



(a) M20 concrete



(b) Fe415 steel

Fig. 2 Stress-strain curves of materials at various temperatures

Table 1 Specimen details of Beam modeled for validation study

Beam 1	Properties
Description	Tested by Dotreppe and Franssen
Cross Section	200×600 mm
Length	6.5 m
Reinforcement	2ϕ12 mm top bars 3ϕ22 mm bottom bars
Applied load	65 kN
Concrete cover	40 mm
Support conditions	Simply supported
Aggregate type	Siliceous

For M20 concrete and Fe415 steel, stress-strain curve based on IS 456:2000 is used. Again, these curves are re-drawn for various temperatures by using the reduction factor for parameters of stress-strain relations for siliceous aggregate concrete and cold worked steel given in Eurocode2. The density of concrete is taken as 2300 kg/m^3 and that of steel as 7850 kg/m^3. The co-efficient of thermal expansion for steel (α_s) and concrete (α_c) are given in Eqs. (1)-(2).

$$\alpha_s=(0.004T+12)10^{-6}/^{o}C \tag{1}$$

$$\alpha_c=(0.008T+6)10^{-6}/°C \tag{2}$$

The assumptions considered during this structural analysis are spalling does not occurs and no bond failure between steel and concrete happens. The perfect bond between concrete and steel is achieved by limiting the tolerance ratio.

3.3 Validation of numerical model

Before performing the finite element analysis, it is essential to find out the dependency of the software. For this purpose, a validation study is carried out with the experimental work done on

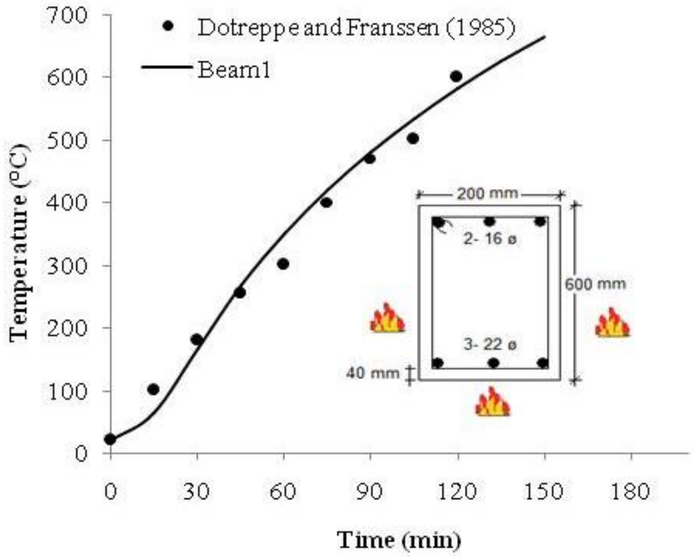


Fig. 3 Comparison of corner reinforcement temperature obtained from numerical analysis with test data (Dotreppe and Franssen 1985)

simply supported beam by Dotreppe and Franssen (1985). The geometric details and various properties are shown in the Table 1. A comparison between the results obtained from the numerical study and experimental work is done and is shown in Fig. 3.

A comparison of reinforcement bar temperature for Beam1 indicates good agreement between ANSYS and those measured in the fire test. The close agreement between the experimental results and the predictions from numerical analysis demonstrates the validity and accuracy of the numerical model.

4. Analysis of RC frames subjected to fire

In present study, to find the restraining effect and the global response of structures when subjected to fire various framed structures are analyzed and compared. The first one is a simple beam column assembly (portal frame) and in second case the structure is modeled as a beam column slab (B-C-S) assembly. To study the global response a three bay three storeyed frame is also analyzed. The frames are subjected to an interior fire which is modeled using ISO 834 standard fire curve (ISO 834 1999). The beams and columns in all frames are of same size and the main reinforcement of size 16 mm is used throughout. The column height is 3 m and beam length is 5 m. The beams and columns in both the frames are identical and the cross-section details are shown in Figs. 4(a)-(c). The structural load is taken as 36 kN/m which is inclusive of both dead load and live load.

4.1 Portal frame

Thermal and thermo-structural behavior of single portal frame was observed for different times of fire exposure. The reinforcement temperature of individual structural elements, i.e., beam and column are plotted. Maximum displacements at middle of beam, horizontal drift at top of column and stresses developed are also noted down.

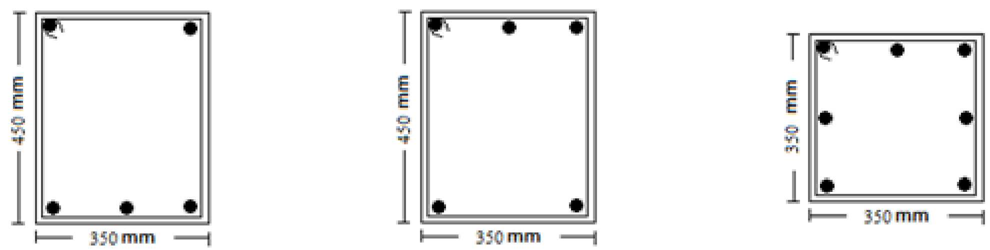
4.2 Beam column slab assembly

The real behavior of structures is more accurately modeled using B-C-S assembly in which the effect of restraints on all structural members can be considered. A thermo-structural analysis is carried out for the assembly. The slab is of thickness 150 mm and the main reinforcement of 12 mm diameters at a spacing of 120 mm centre to centre. As the frame is mounted with RC slab, the behavior will be different from that of a single portal frame.

For Solid 65 element, the mesh was set up such that square or rectangular elements were created. The reinforcement model was meshed using line elements so that the nodes of the line elements come exactly over the node of the solid elements which are later merged so that both reinforcement elements and the concrete elements share the same nodes. The finite element model for both the frames is shown in Fig. 5.

4.3 Three-Bay multi storey frame analysis

To understand exact global behavior of RC framed structure exposed to fire, a full scale analysis of complete frame is required. This structural model is an extension of the frame analysed



(a) beam at mid-span (b) beam at support (c) column

Fig. 4 Cross-section and reinforcement details of members of RC frames exposed to fire

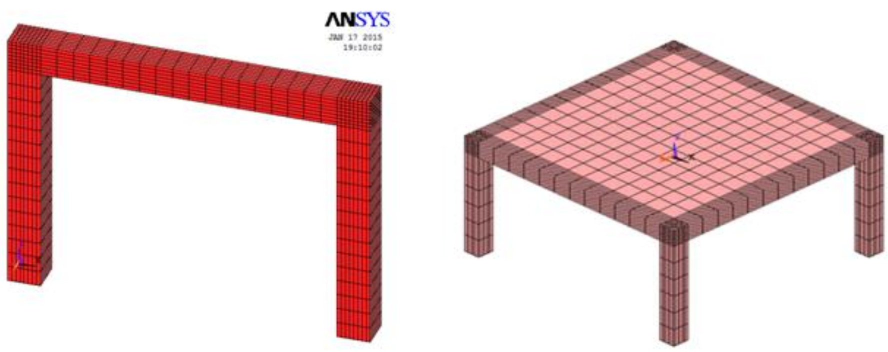
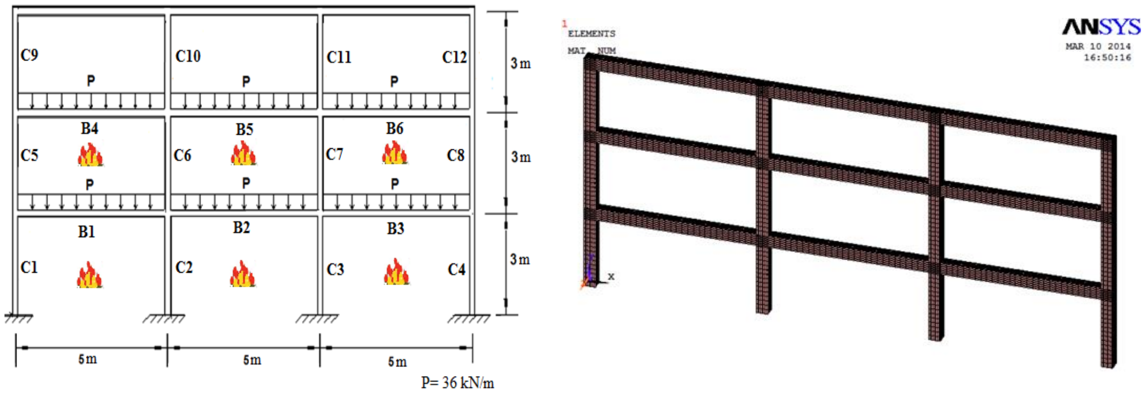


Fig. 5 Finite element model of RC frames exposed to fire



(a) Geometry (b) Finite element model

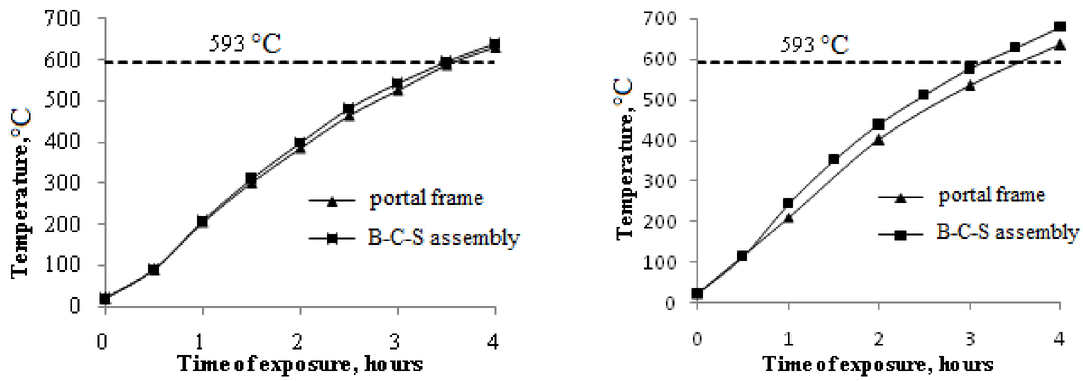
Fig. 6 Multi storey RC frame exposed to fire

in section 4.1. Fig. 6(a) illustrates the modified frame's model under analysis. The first modification is related to the number of bays, augmented from one to three. This modification is aimed to evaluate the impact of a multiple beams in the interior column structural behavior and to understand behavior of interior and exterior bays under fire. The second modification is added to the structural system by increasing number of floors to three. This is for understanding behavior of adjacent storeys and the effect of propagation of fires between storeys when one storey is exposed to fire. The geometric properties of beams and columns are same as that of portal frame discussed in section 4.1. Finite element model of frame is shown in Fig. 6(b). In this analysis third floor is assumed to never subject to fire in all scenario, so that parts of structure will induce a restraining effect in frame which is under fire.

4.4 Results and discussions

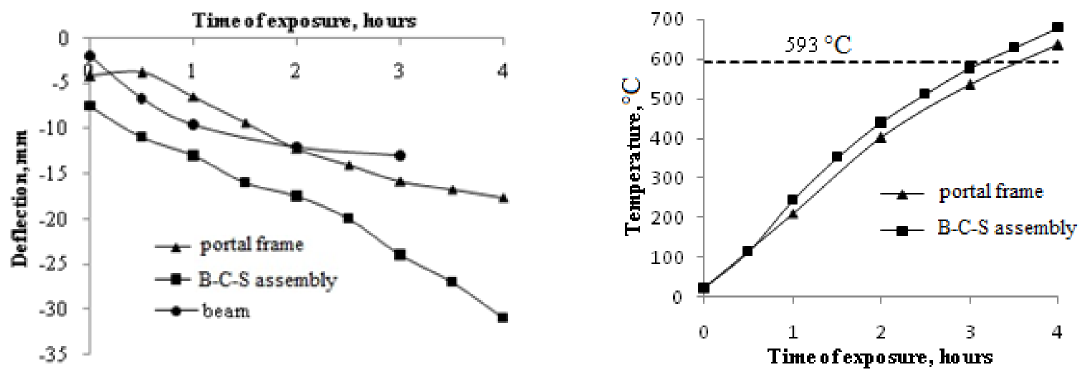
The thermal analysis results establish temperature distribution inside the members of the frame. When the frame is heated under constant load, the inner and outer sides of the beam or column are at high and low temperatures respectively and non-uniform temperature fields are formed on their sections. The corner bar temperature of members of the frames are obtained from thermal analysis are plotted for portal frame and B-C-S assembly and is shown in Figs. 7(a)-(b). As the reinforcement temperature depends only on the fire load and exposed surface area of the structure, there is not much variation in results for both cases. A careful examination of temperature around the reinforcement yields local disturbance in the concrete, which is explained by the fact that steel presents a greater thermal conductivity than that of concrete.

The thermo-structural analysis helps to understand the structural behavior of frames and the effect of floor slab on the frame. The deflection of individual beams and which is part of portal frame and B-C-S assembly are compared in Fig. 8(a). The horizontal drift of column in both frames is plotted in Fig. 8(b). This drift is imposed by the beam's axial expansion and can be seen that the drift of B-C-S assembly is more than twice that of portal frames after four hour fire exposure. This may be due to the combined axial expansion of beam and slab which influences the horizontal drift of column. When the frames are loaded to predetermined values at room temperature (design load, P_u=36 kN/m), the flexural and axial deformations are small. During the process of heating under constant load, the deflection at the mid-span of the beam and the horizontal drift of the column increases with temperature increase. This increase in deflection may be attributed to the stiffness reduction induced by the concrete and reinforcement steel degradation at elevated temperatures.



(a) Beam (b) Column

Fig. 7 Temperature variation in corner reinforcement with respect to time

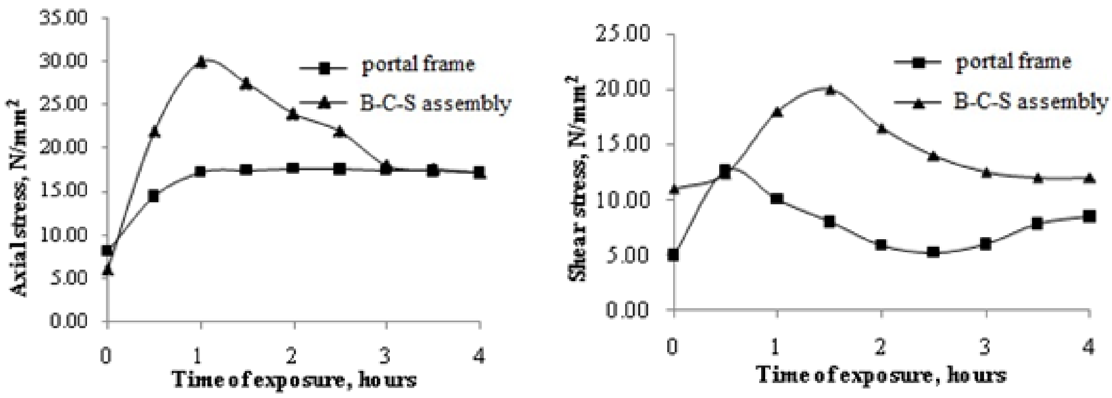


(a) Vertical displacement at middle of Beam (b) Horizontal drift of Column

Fig. 8 Post fire deflection vs fire exposure time for various structures

The axial stresses cannot be evaluated if the analysis is performed for individual members. The axial stresses are plotted for both frames and there is much variation in stress for both cases as shown in Fig. 9(a). The axial stress reaches a maximum intensity after one hour in both cases and after that due to degradation of strength of both steel and concrete the stress values are reduced. As the frame is heated continuously, the internal stresses increase constantly. After one hour of fire exposure, the axial stress starts to decrease despite of increase in temperature. This may be due to the degradation of material, which reduces the axial stiffness, which also causes the increase in deflection. The shear stresses of beam in both frames are also plotted in Fig. 9(b) and these values are also higher for B-C-S assembly.

The evolution of bending moments during the fire exposure in the key points of the portal frame is plotted in Fig. 10. As a first observation to the results obtained, it is very clear that the frame's bending moment field changes its form since the beginning of the fire, which is impossible to understand relying only in sectional simplified methods of structural fire safety assessment. When the fire process begins, due to the flexural restraint imposed by the columns, the thermal gradient within the beam's cross-section originates a constant hogging bending moment along the beam. Due to that reason sagging moment at middle decreases by time and after one hour it changes to hogging. Due to the same reason moment at end of beam (hogging) increases up to approximately two times and it come down to a constant value which is 1.5 times higher than initial value. The evolution of bending moment in the bottom of the fire exposed column shows an increase until approximately 60 minutes of fire duration, and then it starts to decrease at a constant rate, at 2.5 hour exposure bending moment change its signal. After the inversion, the bending moment keeps increasing until the end of the analysis.



(a) Axial stress plot at beam-column joint (b) Shear stress plot at beam end

Fig. 9 Stress vs fire exposure time for various structures

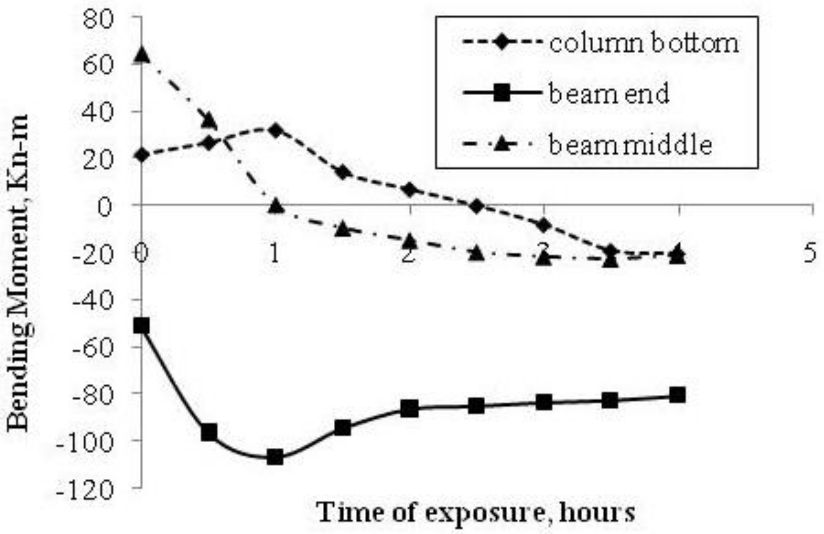


Fig. 10 Evolution of bending moments in portal frame subjected to fire

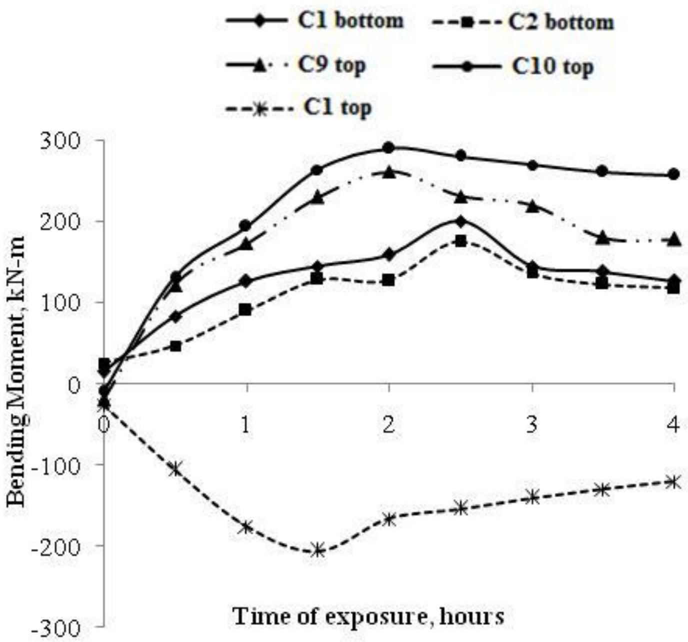
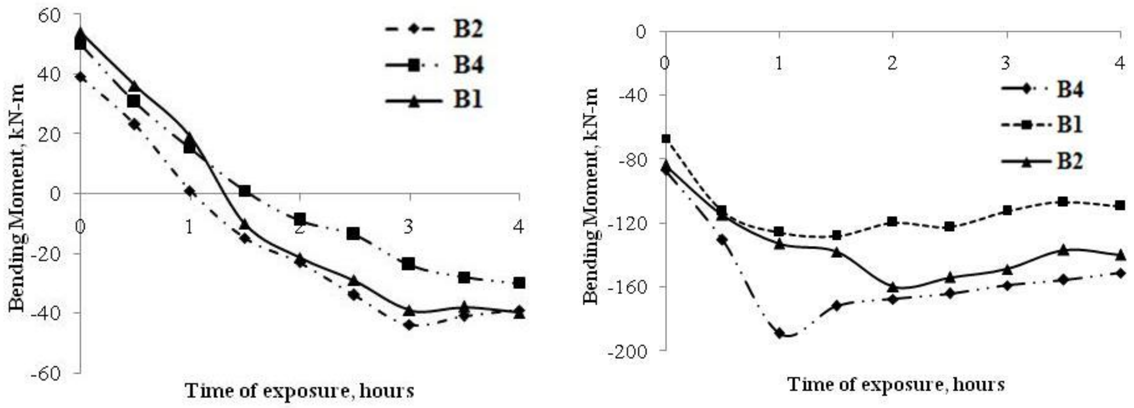


Fig. 11 Evolution of bending moment in columns of multi storeyed frame

As it was already observed in the single-bay frame, the bending moment field has a considerable change during the course of the fire. Fig. 11 presents the evolution of bending moments in key points of the multi storeyed frame's columns. Maximum bending moment occurs at top of unheated columns C9 and C10. The bending moment at these points reaches a peak value approximately after 120 minutes since the beginning of the heating process (290 kN-m). It is observed that this is the instant when the maximum axial stress in the first floor's beams occurs. After that the bending moment in this column decreases until the structural failure is achieved. This happens because of material degradation of concrete and moment redistribution due to change in stiffness. It can be visualized that both first and second floor are imposing a large drift to the ends of the column, meaning that the bending moment diagram should suffers a shift in one direction. It can also be noted that even if the third storey is not subjected to fire, the maximum bending moment occurs in columns in that storey which is different from the frames when subjected to structural loads.

Fig. 12(a) illustrates the evolution of bending moments in mid-span of the beams and Fig. 12(b) shows the evolution of bending moment at end of beams. From these two graphs it is clear that moment at end of beam and mid-span of beam shows a variation in the same direction, which means that the beam's bending moment diagram shift upwards due to the flexural restraint to the rotation induced by the thermal gradient along the beam's cross-section. As a result of this, an inversion of the bending moment sign is recorded after 60-90 minutes of fire exposure at mid-span. In case of end moment the bending moment diagram keeps its shift upwards until 60 minutes of fire duration, after that due to material deterioration it starts redistributing to the mid-span. In middle of beam, bending moment diagram keeps its shift upward until 3 hour and undergoes irregular slight variations until failure.



(a) at mid-span (b) at end

Fig. 12 Evolution of bending moment in beams

The main phenomena governing the structural response have been identified, such as the evolution of bending moments at the top of columns. The maximum bending moment developed in unheated column shows the significance of global analysis. In column 9 and 10 bending moment increases upto 300 kN-m and this significant change may lead to failure of the structure. Bending moment at middle of all beams changes from sagging to hogging within one hour.

5. Failure criteria

The model generates various critical parameters such as temperatures, stresses, strains, internal forces and deformations at various fire exposure times. These output parameters are used to check against predefined thermal, strength and deflection failure criteria which are given below:

1. The temperature in the longitudinal steel reinforcement bars (tension reinforcement) exceeds the critical temperature which is 593℃ for reinforcing steel (Kodur and Dwaikat 2008)
2. The member is unable to resist the specified applied service load
3. The average temperature of the unexposed face of the specimen shall not increase above the initial temperature by more than 140℃
4. The maximum deflection of the member exceeds $l/20$ at any fire exposure time, where l is span length,
5. The rate of deflection exceeds the limit given by the following expression: $l^2/9000d$, mm/min; where l=span length of the beam (mm), d=effective depth of the beam (mm)

Failure criteria 3 is applicable only for slab (IS3809 1979), and failure criteria 4 and 5 are taken from British Standard BS476: Part 20 specifications and are valid for flexural members. The codal provisions for fire ratings are available only for structural members. The failure of structure occurs with the failure of individual members. To identify the critical member in fire, the fire rating of individual members of frame obtained relying on the advanced calculation model is compared against the fire rating provided in IS456 2000. Fire rating of an assembly is governed by the element which has the least fire rating. Therefore, the fire ratings of individual members of the frames are determined based on thermal and deflection criteria. The fire ratings according to different failure criteria are tabulated in Table 2 and minimum fire rating is considered as

Table 2 Fire rating of members of framed structure

	Fire rating (minutes)			
	Thermal criteria (A)	Deflection criteria (B)	Design fire rating (Minimum of A & B)	IS 456:2000
Beam	200	>240	200	>240
Column	185	>241	185	180
Slab	210	>242	210	180

design fire rating. From the results it is clear that fire rating of the frame is governed by thermal failure of column as it is the minimum value. Therefore the fire rating of the frame is taken as 185 min and it is converging with the IS code recommended fire rating.

6. Conclusions

The response of structural system when subjected to fire is studied using the analysis of portal frame and effect of floor slab is checked using B-C-S assembly. The thermal analysis helps to determine the reinforcement temperature of corner bars and the fire rating can be determined based on thermal criteria. As the reinforcement temperature depends only on the fire load and exposed surface area of the structure, there is not much variation in results for both the frames. But there are much variation in deflection, axial and shear stresses of frames with floor slab and without floor slab. When a fire occurs locally in a building, the structural member at elevated temperature is constrained by the surrounding members at elevated temperature and considerable additional internal forces are caused. It can be noted from the analysis that the maximum bending moments are developed in columns of storey which is not exposed to fire. Therefore from the present study it may be concluded that to find the realistic behavior and global response of actual structure, the detailed modeling and analysis of structures are found relevant. It may also be visualized that among the structural members, columns are having the minimum fire rating and it is comparable with IS code recommendations. Hence concluded that columns are the critical members in fire and failure of column causes the failure of entire structure.

References

ANSYS Release 14.0 (2010), Documentation for ANSYS.

BS 476 (1987), *Fire tests on building materials and structures -Part 20: Method for determination of the fire resistance of elements of construction (general principles)*, British Standards Institution.

Di Capua, D. and Mari, A.R. (2007), "Nonlinear analysis of reinforced concrete cross-sections exposed to fire", *Fire Saf. J.*, **42**(2), 139-149.

Eurocode 2 (2004), *Design of concrete structures- Part 1-2: General rules- Structural Fire Design (EN 1992-1-2),* Commission of European Committee for Standardization, Brussels.

Guo, Z. and Shi, X. (2011), *Experiment and Calculation of Reinforced Concrete at Elevated Temperatures*, Tsinghua University Press, Published by Elsevier Inc.

ISO 834-1(1999), *Fire resistance tests-elements of building construction*, *Part1: General requirement*, International Organization for Standardization, Geneva.

IS 3809 (1979), Fire resistance test of structures, Bureau of Indian Standards, New Delhi.

IS 456 (2000), *Plain and reinforced concrete- Code of Practice,* Fourth revision, Bureau of Indian Standards, New Delhi.
James, A. and Milke, P.E. (1999), "Analytical methods to evaluate fire resistance of structural members", *J. Struct. Eng*., **125**(10), 1179-1187.
Jowse, A., Torero, J.L., Usmani, A., Lane, B. and Lamont, S. (2005), "Determination of fire induced collapse mechanisms of multi-storey steel framed structures", *Fourth European Conference on Steel and Composite Structures*, Maastricht, Netherlands.
Al-Jabri, K.S., Davison, J.B. and Burgess, I.W. (2008), "Performance of beam-to-column joints in fire-a review", *Fire Saf. J.*, **43**(1), 50-62.
Kodur, V.K.R. and Dwaikat, M. (2008), "Flexural response of reinforced concrete beams exposed to fire", *Struct. Concrete*, **9**(1), 45-54.
Pyl, L., Schueremans, L., Dierckx, W. and Georgieva, I. (2012), "Fire safety analysis of a 3D frame structure based on a full-scale fire test", *Thin-Wall. Struct.*, **61**, 204-212.
Ma, K.Y. and Richard Liew, J.Y. (2004), "Nonlinear plastic hinge analysis of three-dimensional steel frames in fire", *J. Struct. Eng*., **130**(7), 981-990.
Valipour, H.R. and Foster, S.J. (2010), "Finite element modelling of reinforced concrete framed structures including catenary action", *Comput. Struct.*, **88**(9), 529-538.
Vecchio, F.J. (1987), "Nonlinear analysis of reinforced concrete frames subjected to thermal and mechanical loads", *ACI Struct. J.*, **84**(S51), 492-501.

TK