



# Numerical Analysis of One Way – Prestressed Concrete Slab Subjected to Fire

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**Abstract.** Over years, the world has experienced many disasters due to fire resulting in death and loss of property. The construction industry started using modern construction ideas due to the need for speedy and economical construction. This paper focuses on the effect of high temperatures generated during a fire on prestressed concrete slab. It is very important to identify the changes in temperature in pre-stressed concrete under fire to study fire resistance and deformation performance in high rise buildings. It is essential to identify stresses induced by temperature distribution for adequate structural design. The aim of the research is to show a linear transient assessment of a pre-stressed concrete slab through its cross sections. The reaction of concrete to fire is dependent on its mechanical, temperature and deformation characteristics. The traditional method of assessing fire resistance through fire testing is costly, time-consuming, and restricted to study parameters. Numerical modelling is an option to fire testing to analyze the resistance of pre-stressed concrete slabs to fire. Numerical methodology enables the effective and cost-effective integration of multiple parameters. Stress distribution along the length of prestressed slab when subjected to fire for various times is investigated and studied. It was found that stresses in the pre-stressing forces in the tendons reduced with an increase in temperature. Loss in stresses was about 28.2%, 45.2% and 49.8% for external fire curve, ISO 834 fire curve and hydrocarbon fire curve respectively. Also the deflection pattern of the prestressed concrete slab at various time intervals of fire exposure were investigated.

**Keywords:** Fire curves · Pre-stressed slab · Thermal response

## 1 Introduction

Production of prestressed concrete structures started many years ago but it became popular only recently. Pre-stressed concrete slabs have more advantages than normal reinforced concrete slabs in aspects such as construction speed, readily achievable clear

spans, cost effectiveness, slender and light weight structures etc. Hence it is used in almost all constructions. Presently, the test data on pre-stress concrete behaviour at high temperature was very limited. Pre-stressed concrete was more fire sensitive than reinforced concrete due to the usage of cold worked pre-stress bars. Fire-resistance rating for pre-stressed concrete was specified by the codes since fire safety of the structure is a major factor in buildings. Conventional fire test was the technique used for assessing the fire resistance of pre-stressed concrete slab. The thickness of the slab and its cover influence fire resistance. In comparison with concrete, steel shows less fire resistance properties. So, prestressing tendons were the governing factors in slab fire resistance. Kodur and Shakya (2017) investigated hollow core pre-stressed concrete slabs under fire and structural loads. Parameters taken for study included aggregate type, support conditions, fire and service load levels. The results showed that most of the slabs underwent flexure failure. It was revealed by the occurrence of flexural cracks on the slabs bottom face. Slabs withstood fire for two hours under service load conditions. Also providing axial restraint support condition increased fire resistance capacity of the slabs by about 30 min. Nearly, 10% greater fire resistance was provided by the slabs made with carbonate aggregates than those with siliceous aggregates. With the provision of adequate cover and reinforcement over the supports in continuous systems, pre-stress members resist fire up to 4 h.

Hou et al. (2139) studied the continuous pre-stressed slabs response under fire by varying the degree of pre-stressing, thickness of the concrete cover and load. More cracks occurred near the intermediate support on the unexposed face due to increased secondary moments occurred from the fire. These cracks widened up with increased exposure time. The slabs also experienced greater deflection when the adopted concrete cover thickness was lower. Decrease in fire resistance and increase in spall area of slabs was observed with higher degree of pre-stressing. The degree of pre-stressing had only minor influence on the response of slabs compared to cover thickness. Bond strength between rebar and concrete decreased with increased fire. Therefore, a better reinforcement layout with an extended length of rebar was advised. Gang Zhang et al. (2004) assessed the performance of girders under fire loads by varying the degree of pre-stressing produced through hydrocarbon fuels. Four stages for the progression of mid-span deflection were established. Lower ductility was seen in higher degree of pre-stress. Hou et al. (2015) conducted experimental study to assess the factors governing the behaviour of pre-stressed beams under fire. It was found that using grout for ducts increased the moisture levels thereby reduced increasing temperature values in the pre-stressing strands. Beams with continuous supports showed better fire performance than those with simply support due to the continuity effect which was needed for redistribution of secondary moments resulting from fire. Pre-stressed beams with a greater concrete cover having a low magnitude of service load acting on it revealed better fire resistance. Zheng et al. (2010) conducted experiments using continuous and simply supported pre-stress slabs under fire loads. Extent of spalling was determined by concrete strength. Spalling was more serious when the compressive stress on the exposed concrete surface and water content in concrete at ambient temperature was too high. Cooke et al. (2001) studied the influence of imposed load, type and thickness of concrete on concrete slabs under fire loads. Use of light weight concrete reduced deflection by 30% because of

low thermal expansion and conductivity coefficients. The effect of imposed load on deflection was not dominant when exposed to fire of a greater duration. Kodur and Shakya (2017) numerically studied the factors governing shear failure for hollow core slabs under fire. From the results of parametric study, slab failure shifted from shear to flexure under greater fire exposure and with decreased slab thickness. Bamonte et al. (2018) investigated the behaviour of pre-stressed members under natural fires. Results showed that failure was delayed for massive sections due to the time taken to reach the maximum temperature being too long. Torić et al. (2012) investigated the hollow core pre-stressed slabs under fire loads. The study revealed slab behaviour depended on tendon temperature.

When validated, finite element modeling can be used to depict the overall behaviour of concrete and steel exposed to fire. To investigate the characteristics of concrete when subjected to fire, various finite element models were generated by Hosser et al. (1994), Bailey et al. (2006), Izzuddin (1997), Song et al. (2000), Elghazouli and Izzuddin (2001) and, Gillie (2001). To date, very few numerical models and studies are available for investigating the pre-stressed slabs behaviour under fire exposure. Some progress was offered by Lee and Bailey (2006) and Ellobody and Bailey (2007). The purpose of this study is to provide wide knowledge about the pre-stressed concrete slabs behaviour under fire exposure scenarios through numerical investigation.

## 2 Finite Element Modelling

### 2.1 Description

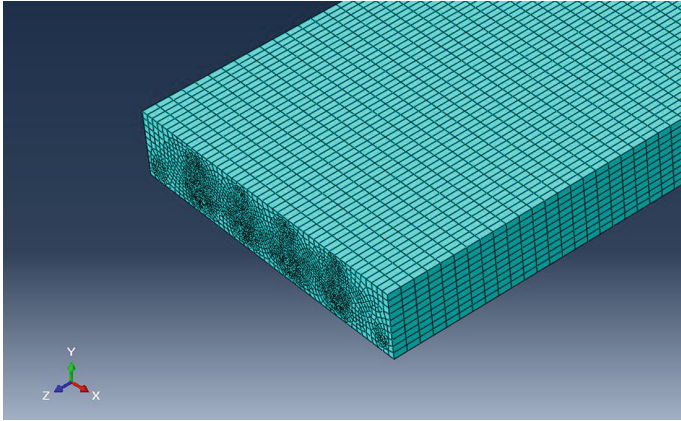
Using finite element software, a numerical model was produced to portray the fire reaction of a pre-stressed concrete slab. This model helped to acquire the material and geometric linearity and also the thermal-dependent heat and mechanical characteristics of the pre-stressing tendons and concrete. The analysis of fire resistance of prestressed concrete slab was carried out at incremental time steps. This project discussed many features of the model including details of discretization, and variation of properties at high temperatures.

### 2.2 Details of Discretization

The dimensions of the slab used were 7.96 m in length, 1.2 m wide and 250 mm deep. Two models for two different stages of analysis were developed. The model used for thermal analysis in the Abaqus was a DC3D8: An 8-node linear heat transfer brick. This element transfer heat from the bottom to the top surface of the slab. In other words, it is a uniaxial element which transfer heat through the nodes which have a unit degree of freedom. It can produce both steady-state as well as transient analysis.

In the mechanical analysis, the element used for modelling the pre-stressing tendons and concrete was a “C3D8R- an 8-node linear brick, reduced integration, and hourglass control element”. These elements are used to calculate large stress deformations and deflections in the pre-stressing concrete slab.

As the analysis was conducted in two stages, heat distribution along the depth of the slab was calculated in the thermal analysis. Heat distribution was used as thermal load



**Fig. 1.** Typical layout of prestressed concrete slab and idealization of FE analysis.

with pre-stressing forces and the other loads such as live and dead loads in the next stage of the mechanical analysis.

To discretize the slab into small elements, “MESH” module was used in the Abaqus. A suitable mesh size was obtained by performing mesh convergence study. For this study, results of temperature distribution for various mesh sizes was generated and compared. The comparison results showed that beyond an increase of 10 layers along the depth of the slab, there was not much significant variation in percentage of results as in Fig. 2. Therefore, 15 layers along the slab depth was taken.

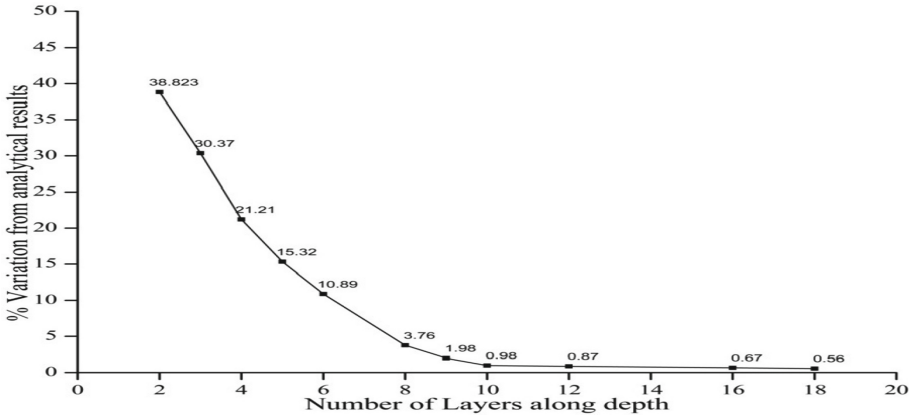
### 2.3 Material Properties at High Temperature

Fire exposure degrades the concrete and pre-stressed tendons properties. Variations in the high temperature material properties were studied to evaluate its response of fire. Based on the relations specified in the Euro code, all thermal and mechanical properties were defined with respect to variation in temperature. The values of these properties are represented graphically.

Heat distribution in the prestressed concrete slab took place in three modes. The first mode is the transfer of heat within the solid element by conduction. This property in abaqus was assigned to the model by means of Thermal conductance in a contact interaction. The next modes of heat transfer were through convection and radiation. These properties were assigned to the model by a Surface Film interaction property where the corresponding film coefficient and emissivity values were provided. During the mechanical analysis, the bond between the pre-stressing tendons and concrete was given by the interaction property in the ABAQUS element library (Surface to surface contact).

### 2.4 Loading and Boundary Conditions

The pre-stressed concrete slab was subjected to both the mechanical and thermal loads. Initially the slab experienced loads such as live load, dead load and pre-stressing forces



**Fig. 2.** Percentage variation v/s number of layers along the depth for Mesh Convergence study.

thereby undergoing certain deflections. Then the slab was exposed to thermal loads till failure. Thermal loads applied were based on Nominal time-temperature curves i.e. standard time-temperature fire curve, external fire curve and hydrocarbon curve, given in the Euro code 1 (2002). Structures fire resistance is tested using this curves. Figure 3 shows the derived temperature curves that can be used as thermal loads in this analysis. These curves were obtained by using the equations as given in Euro code 1 (2002) and ISO (1975).

Dead and live loads were taken as a percentage of the total capacity of the slab. Floor was the assembly for the slabs; hence, the presence of adjacent slabs stimulated the slab's edge conditions. For this, the longitudinal edges of the slab were assumed to be free for vertical deflection and restrained horizontally. The modelled slab could simulate this effect. For the convenience of the analysis, the entire slab was bisected along the longitudinal and latitudinal axis and a quarter portion of the slab was used for analysis. The quarter portion of the slab also generated results similar to that of the full slab.

## 2.5 Failure Criteria and Numerical Convergence

Based on ASTM E119 which is similar to the ISO 834 fire, a pre-stressed concrete slab was said to have thermal failure when the following happened:

- When the pre-stressing strand exceeds the temperature of 427 °C (800 °F), which is considered as the critical temperature of the pre-stressing steel. Also, at this temperature the pre-stressing strand loses 50% of its strength.
- When the temperature of the unexposed surface of the slab reaches 181 °C (356 °F) at any single point or reaches an average temperature of 139 °C (282 °F) at nine points on the unexposed surface.

Strength failure happens when moment carrying capacity of the slab drops than the moment generated by the loads.

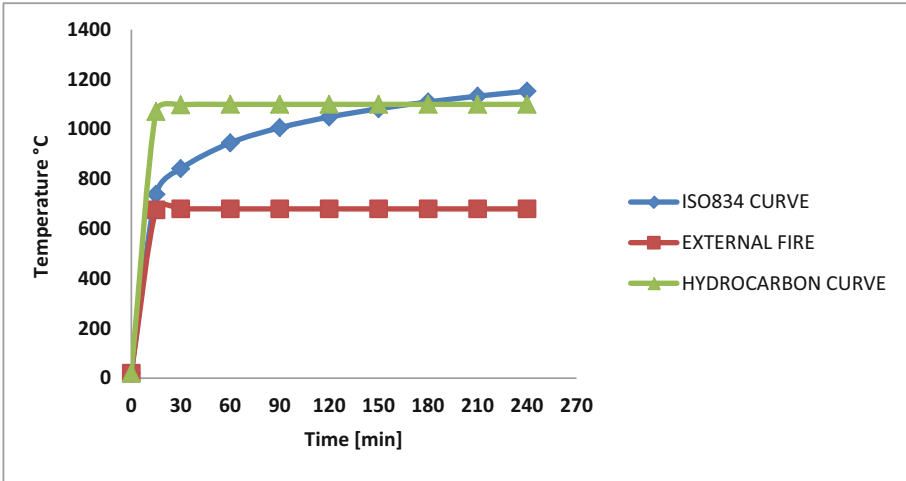


Fig. 3. Nominal time-temperature curves.

Deflection and its rate of deflection helps in obtaining the behaviour of a prestressed concrete slab subjected to fire. Due to the deterioration of stiffness members and temperature induced creep, both deflection and the rate of deflection values observed were expected to be higher at elevated temperatures. According to BS 476, pre-stressed slabs fail when the deflection rate becomes greater than  $\frac{L^2}{9000d}$ , or when the slabs maximum deflection becomes more than  $\frac{L}{20}$  during any fire exposure time. (Where  $L$  and  $d$  were span and effective depth of the slab.)

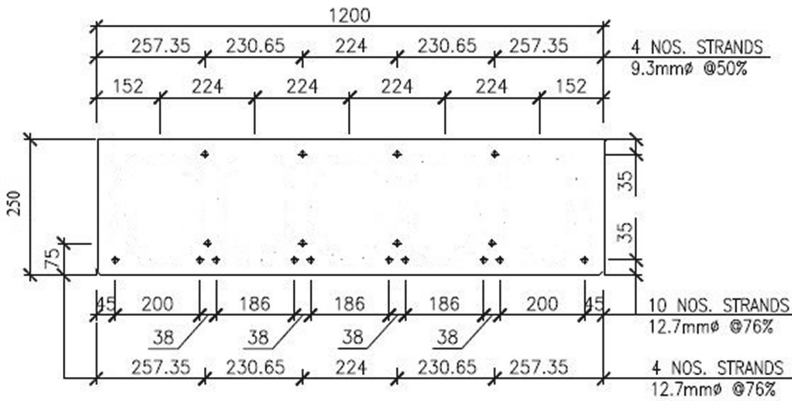


Fig. 4. Cross-section of the prestressed concrete slab.

## 2.6 Characteristics of Prestressed Concrete Slab

The slab chosen for the model has length 7.96 m, width 1.2 m, and depth 250 mm as shown in the Fig. 4. Concrete compressive strength of 40 MPa was used. The prestressing strands were of 9.3 mm and 12.7 mm diameter having a yield stress of 1770 MPa and 1860 MPa respectively. The strands were provided at three different levels from the bottom of the slab. The cover thickness for 10 no. 12.7 mm diameter strands was 35 mm and for 4 no. of 12.7 mm diameter strands was 45 mm from the bottom end. Cover thickness for 4 nos. of 9.3 mm diameter strands was 35 mm from the top end.

## 2.7 Model Validation

Non-linear heat transfer analysis on the prestressed concrete slab was performed to validate the finite element model, loading procedures and boundary conditions. Initially, the model was exposed to a ISO 834 curve as shown in the Fig. 5. Heat distribution curves from the FE model were plotted are Shown in Fig. 6. and compared with the temperature profiles as given in the Euro code -1-2. It was observed that the prestressed slab model after the finite thermal analysis followed the same temperature distribution history as in Euro code -1-2.

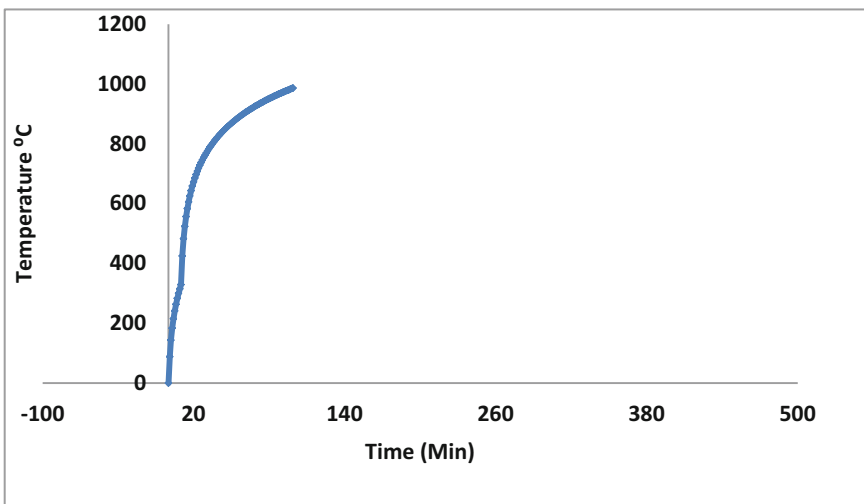


Fig. 5. ISO fire curve.

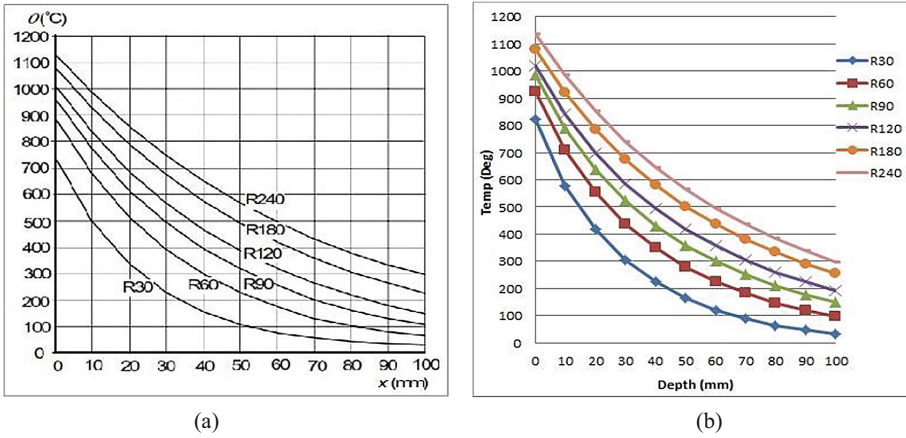


Fig. 6. Temperature profiles for slab (a) EN1992-1-2 (b) ABAQUS model.

The pre-stressed concrete slab was designed based on an analysis of an RCC slab. In ABAQUS, the RCC slab was modelled as shown in Fig. 7 and heated from below by an ISO-834 fire for 90 min. The slab withstood fire only up to 90 min. The analysis was conducted in two phases- structural and heat analysis.

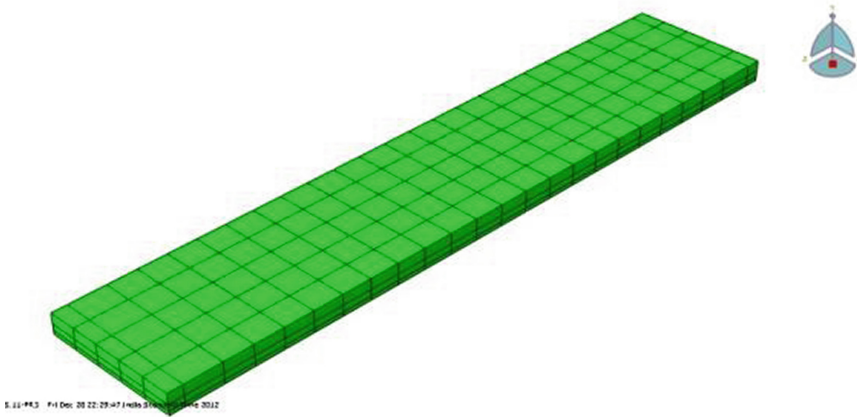
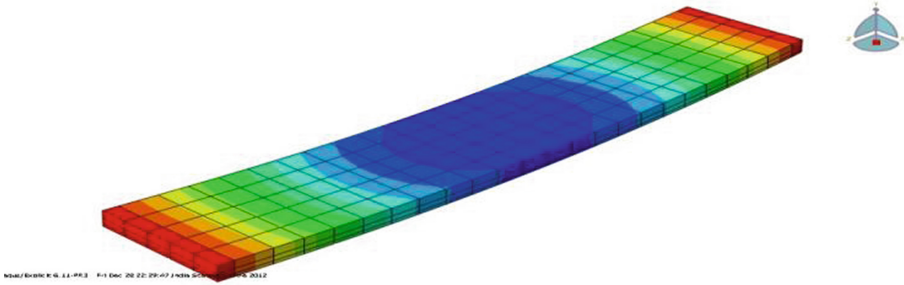


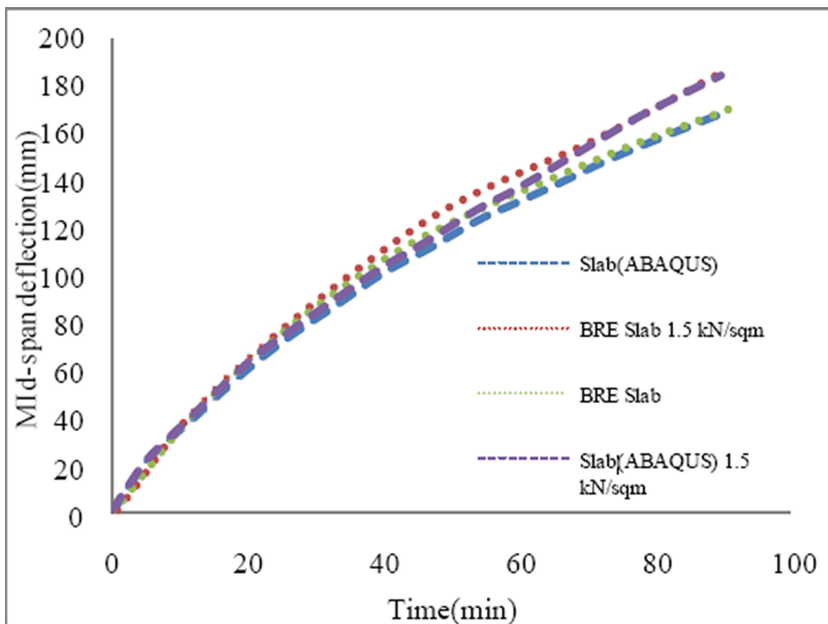
Fig. 7. Model of Slab developed in the ABAQUS.

Figure 8 shows the slabs displacement contour under the ISO 834 fire curve loading. On comparing these displacement values with the experimental results it was found that results from the numerical analysis were in agreement. Also, temperature profiles at various slab depths was compared as shown in Fig. 6 revealing a variation pattern in temperature with the depth for all R values was the same. The present model slightly over predicted the temperature values at the 0 mm and 100 mm depth of slab for the R30 and R60. Therefore, validation of the structure was completed.





**Fig. 8.** Displaying the displacement contour after temperature loading of RCC slab model in ABAQUS.



**Fig. 9.** Comparison made between ABAQUS model slab and BRE slab of 150 mm depth, Mid-span deflection versus time graph.

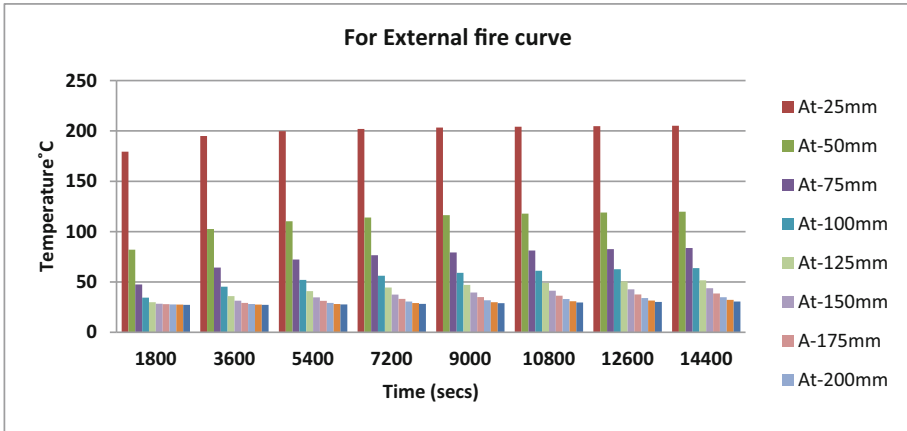
### 3 Results and Discussions

#### 3.1 Thermal Response

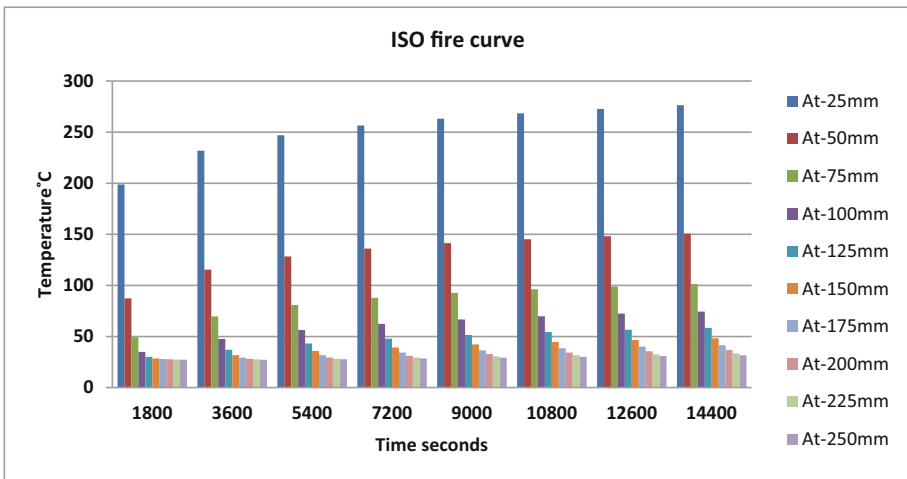
Pre-stressed concrete slab was selected to understand the response to fire in a building. A non-linear thermal analysis was done with complete temperature at the slab bottom. The slab was designed as a 3D element and exposed to the ISO-834 fire curve. ABAQUS/CAE 6.11 was the software used.

The slab was subjected to three different fire curves and the distribution along the depth of the slab was studied. Thermal properties for both steel and concrete were given

as input based on the Euro code as mentioned above. From the investigation, it was observed that temperature distribution through the slab depth under each considered time followed a non-linear trend.

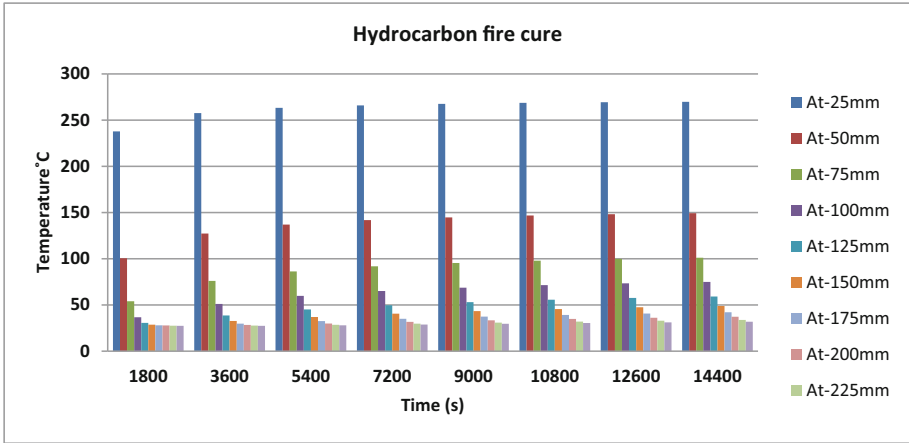


**Fig. 10.** Temperature change along the depth of the prestressed concrete slab when subjected to External fire curve.



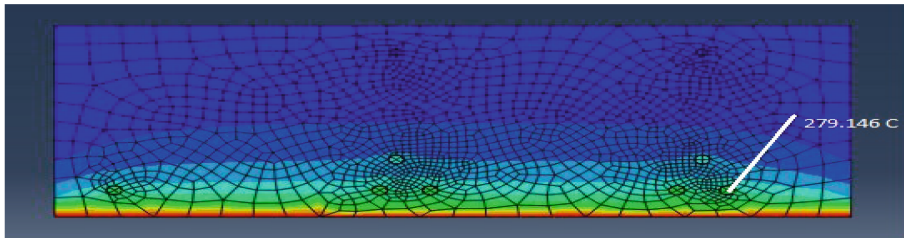
**Fig. 11.** Temperature change along the depth of the pre-stressed concrete slab when subjected to ISO834 curve.

Figures 10, 11 and 12 shows the time dependent variations in temperature along the depth of the slab under external, ISO and hydrocarbon fire curves, respectively. It was found that only smaller depths of slabs were exposed to greater temperature. The variation in temperature at different depths was significant only up to a depth of 125 mm. For rest of the slab’s depth, temperature did not vary much. Slabs under the external fire curve experienced less temperature than in the other two fire curves.

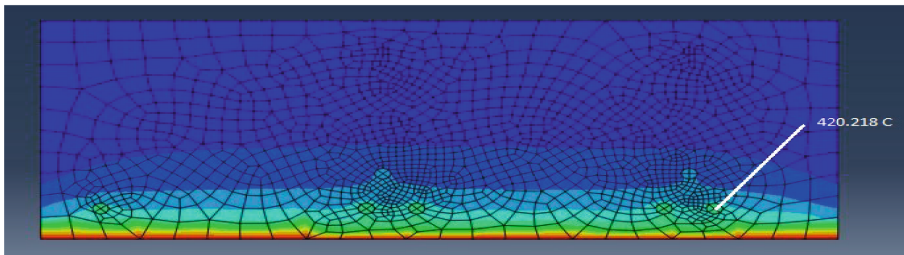


**Fig. 12.** Temperature change along the depth of the pre-stressed concrete slab when subjected to Hydrocarbon curve.

Figure 13 shows the temperature variation contour of pre-stressed slabs through their depth. The bottom surface of the slab reached a higher temperature. Tendons in the slab experienced a temperature of about 279° and 470 °C for 30 and 60 min duration of fire load.



(a)



(b)

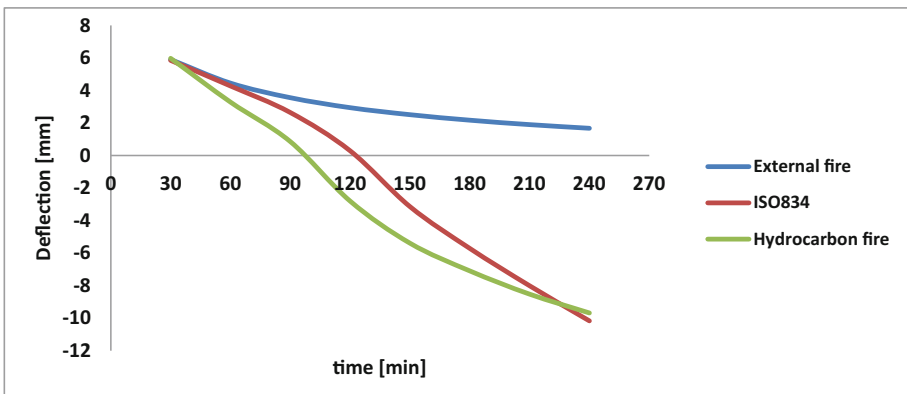
**Fig. 13.** Represents the temperature distribution at (a) 30 min (b) 60 min.

### 3.2 Structural Response

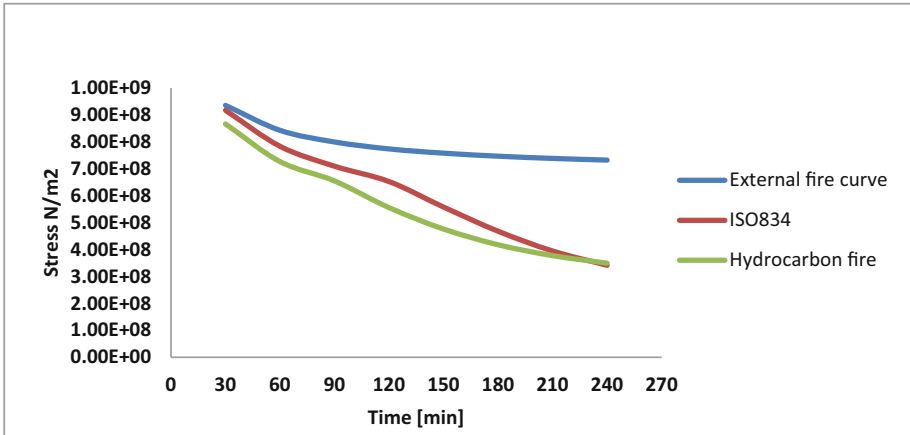
Under normal conditions, the pre-stressed concrete slab showed an initial upward deflection (camber) due to the pre-stressing forces. Deflection in the initial stage was 18.3 mm. Deflection was reduced to 6.58 mm when subjected to dead and live loads. On exposing the bottom surface of slab to fire, deflection began to increase due to an increase in thermal expansion of concrete with exposure time. Thermal gradients and the corresponding thermal strains were evolved along the slab depth when exposed to fire at the bottom surface alone.

After 15 min of exposure to fire, the thermal strains stabilized due to the increase of temperature through the inner layers. With a further exposure to fire for 30 min, deflection continued to increase slowly as there was loss of material stiffness and strength. It was observed that up to 100 min, the slab underwent an increase in deflection gradually after which there was a sudden increase in deflection revealing the start of structural instability. Similarly, the pre-stressed concrete slab was subjected to other two fire curves too. The change in deflection and stress were observed and plotted for varying exposure times at 30 min interval up to 240 min.

Deflection obtained at the midpoint of the slab for the steel tendon was less when exposed to external fire and maximum for the ISO 834 and Hydrocarbon curves at an exposure time of 240 min. This was because for an increase in temperature, material stiffness decreased along with the strength of the slab. With the decrease in material stiffness, young’s modulus also decreased. As stiffness of the material is inversely proportional to deflection, stiffness of the material will be minimum at the point of maximum deflection. Figure 14 shown shows the relation between deflection at the midpoint of the tendon and the exposure fire duration. Under the hydrocarbon fire curve, deflection rate was faster than under the other two curves. At 230 min, midpoint deflection of the tendon under the hydro carbon and ISO 834 curves was the same. After this deflection trend under ISO curve continued to remain the same. But the deflection-rate of the tendon was reduced for the hydrocarbon curve.



**Fig. 14.** Deflection of steel tendon at the midpoint of the slab when exposed to various fire curve for different time interval.



**Fig. 15.** Stress distribution of steel tendon at the midpoint of the slab when exposed to various fire curve for different time interval.

As the temperature increased with increasing exposure time, stress in the steel tendons tend to reduce. Average reduction of stress at the midpoint of the steel tendon for varying time interval of exposure to fire was 28.2%, 45.2% and 49.8% for the external, ISO 834 and hydrocarbon fire curve respectively. Figure 15 represents the stress behaviour at the midpoint of the steel tendon with respect to different time intervals after exposure to different fire curves.

As half part of the slab length was considered for the convenience of analysis, deflection in the steel tendons up to the centre of the min slab can be depicted. Deflection along the length of the slab for a 30 min time interval for 30–240 min of exposure to three different fire curves is shown in the graph. As shown in Fig. 16, deflection is around 6 mm in the upward direction initially, due to the application of pre-stressing forces and load. With the increase in temperature, deflection continues to increase downward. Stress distribution in the tendon along the slab length was determined similarly. The stresses reduced initially due to a sudden increase in temperature and then remain constant. Figures 17, 18, 19, 20, 21 and 22 show the stress and deflection distribution through the length of the slab for varying time intervals for the three different fire curves.

### 3.3 Deflection History Under Different Fire Curves

As discussed earlier, deflection in the tendon increased with the duration of exposure to fire. Figure 17 shows the variation in deflection of the tendon under different exposure durations for the ISO fire curve. It was seen that variation in deflection of the tendon for different duration of fire exposure was less for a length of 0 to 0.5 m. For the remaining length there was a huge variation in deflection. Up to an exposure duration of 120 min, deflection remained positive which meant that the effect of pre-stressing was nullified only after an exposure duration of 120 min in case of the ISO fire curve. Tendon undergone a maximum deflection of 10 mm for fire exposure duration of 210 min.

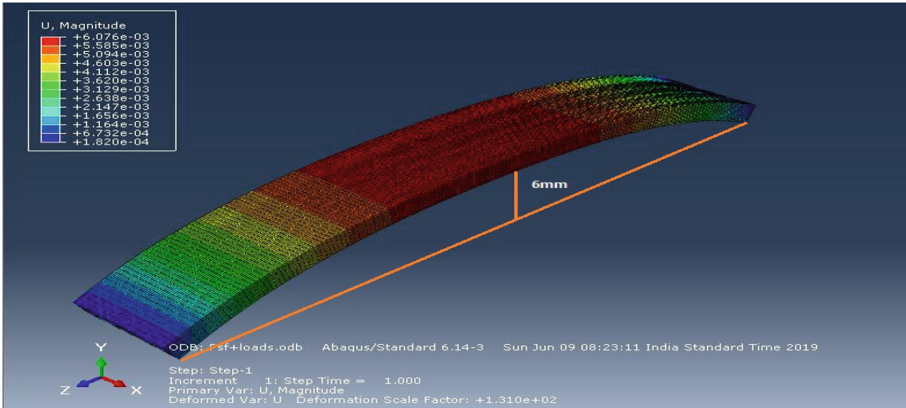


Fig. 16. Deflected profile of the pre-stressed concrete slab before fire.

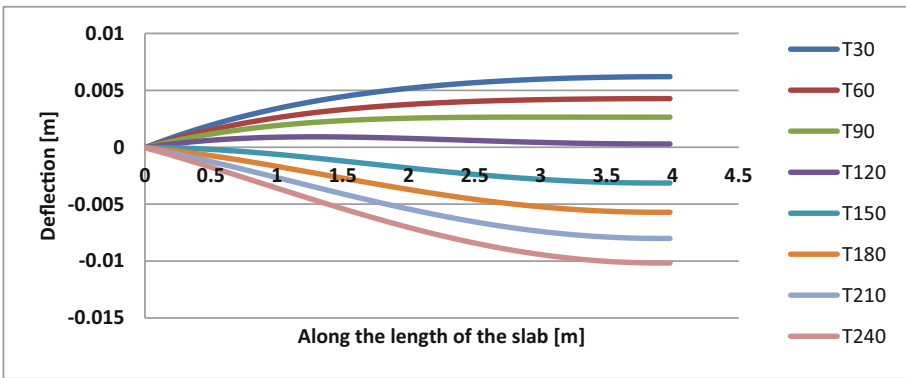


Fig. 17. Deflection of the pre-stressing steel tendon along the length (mid-span) of the slab for different time of exposure to ISO fire curve.

Figure 18 reveals that deflection in the tendon was more or less similar to the ISO and hydrocarbon fires. A major difference observed under these two curves was the deflection of tendon for 120 min exposure retained a positive value for earlier one. But in later case it was under a negative value indicating tendons underwent excess deflection.

From the Fig. 19 it was seen that deflection trend of tendon along the length under external fire was parabolic and this remained the same at different time intervals. Change in deflection of tendons were more till 2 m length. From 2 m to 4 m length, deflection of tendon remained almost constant for different exposure time under consideration. On comparing the Figs. 17, 18 and 19 the deflection experienced by the tendon was very lower in case of external fire curve for all durations. The deflection of tendons increased up to 1.75 mm for 210 min of exposure.

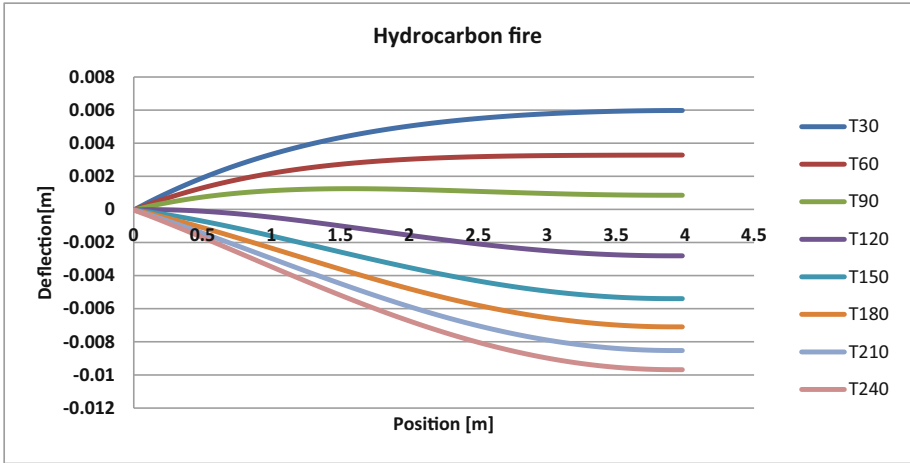


Fig. 18. Deflection of the pre-stressing steel tendon along the length (mid-span) of the slab for different time of exposure to Hydrocarbon fire curve.

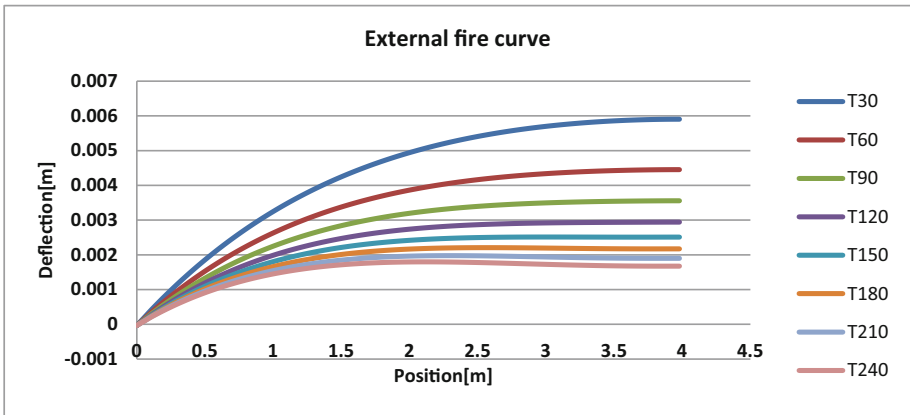
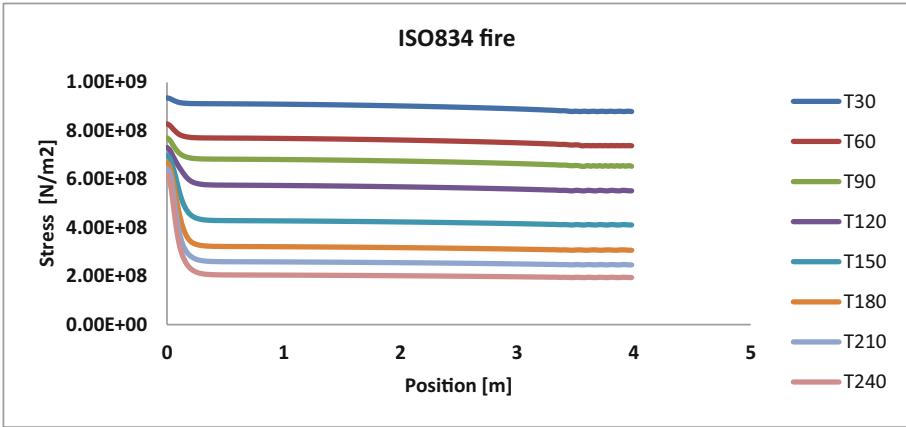


Fig. 19. Deflection of the pre-stressing steel tendon along the length (mid-span) of the slab for different time of exposure to External fire curve.

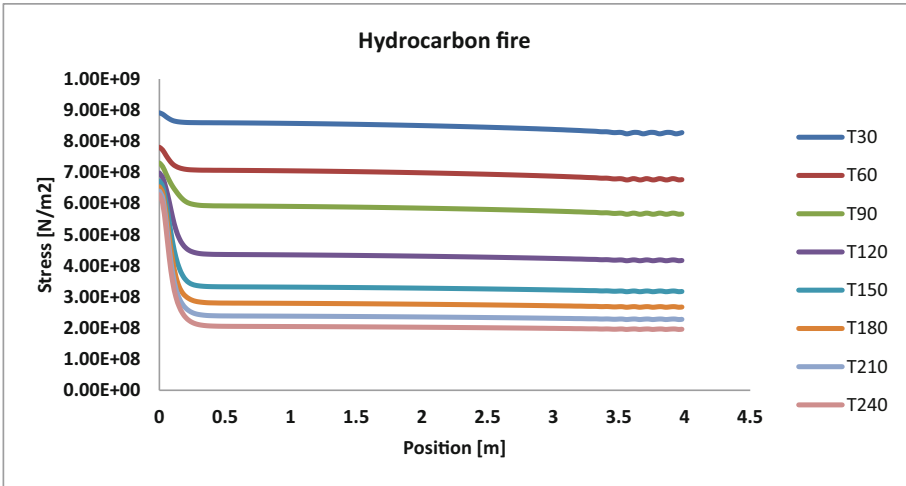
### 3.4 Stress History Under Different Fire Curves

Figure 20 shows stress in the tendon along its length under different exposure durations for the ISO fire curve. With greater exposure duration, stress in the tendon kept decreasing along its length. Sudden decrease in stress occurred for a tendon length of 0 to 0.2 m. For the remaining tendon length, stress value continued to be constant as attained at 0.2 m under each exposure duration. The slope of fall in stress value was found to be same for the exposure durations ranging from 150 to 240 min. The rate at which the stress value was reduced decreased after an exposure duration of 150 min.

Stress reduction in tendons in the hydrocarbon fire for various exposure duration is shown in Fig. 21. The reduction pattern was similar to that in the ISO fire curve. In



**Fig. 20.** Stress of the pre-stressing steel tendon along the length (mid-span) of the slab for different time of exposure to ISO fire curve.

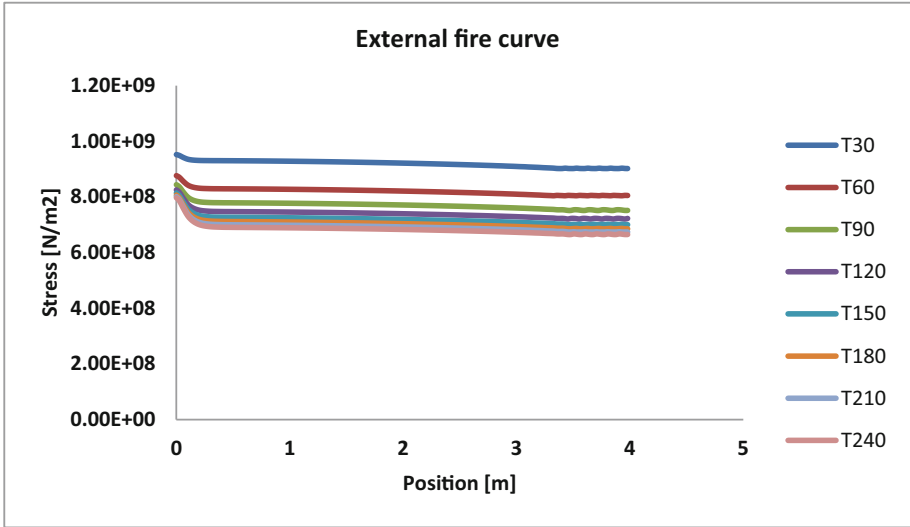


**Fig. 21.** Stress of the pre-stressing steel tendon along the length (mid-span) of the slab for different time of exposure to Hydrocarbon fire curve.

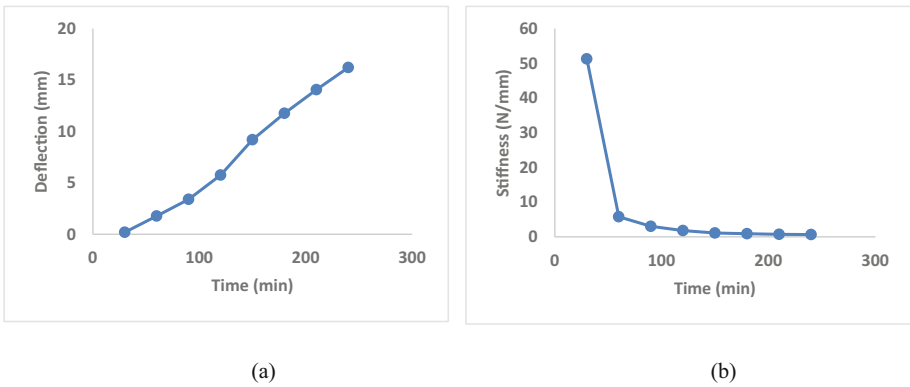
this case, the slope of stress reduction up to 0.2 m tendon length was similar for the exposure durations ranging from 120 min to 240 min. The magnitude of stress reduction decreased when the duration exceeded 150 min.

Figure 22 shows that stress reduction in the external fire curve was very less compared to the other two curves. Stress values were reduced to a maximum of  $7 \times 10^8$  N/m<sup>2</sup>. In this case exposure duration not had a great effect. Reduction in stress values were more or less similar for exposure durations ranging from 120 min to 240 min. Stress reduction was significant only between 30 and 60 min exposure.





**Fig. 22.** Stress of the pre-stressing steel tendon along the length (mid-span) of the slab for different time of exposure to External fire curve.



**Fig. 23.** (a) Stiffness vs exposure time curve for slab under ISO fire curve (b) Deflection vs exposure time curve for slab under ISO fire curve.

From Fig. 23 a steep decrease in slab stiffness occurred till an exposure time of 60 min indicating that the slab underwent more damage within 60 min. As stiffness and deflection were inversely proportional to each other, deflection of the slab started to increase but at a reduced rate. After 60 min, the degradation in stiffness become slower, thereby indicating that damage was occurring at slower rate. But the deflection rate started to speed up. At 240 min the slab's stiffness was almost degraded resulting in maximum deflection.

## 4 Conclusion

Stress distribution of the pre-stressing steel along the slab length under fire exposure was determined for 30 min time intervals from 30 to 240 min. Thus, losses in stress with an increase in temperature were found. Mid-span deflection for varying temperatures was also found. From the investigation, the following points were deduced:

- Stresses kept decreasing at a steady rate with an increase in the duration of exposure of the slab to fire due to a decrease in the stiffness of material with an increase in temperature.
- Stress losses due to an increase in temperature and exposure time were 28.2%, 45.2% and 49.8% for the external fire curve, ISO 834 fire curve and the hydrocarbon fire curve respectively.
- Elasticity modulus value also reduced with a rise in temperature and thus the structure lost its elastic nature and entered the plastic strain region.
- Mid-span deflection increased with an increase in temperature due to increased exposure time.

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