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*Original Article*

# Validation of Indian standard code provisions for fire resistance of flexural elements

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**Abstract**

Fire resistance provisions in Indian codes are prescriptive in nature and provide only tabulated fire ratings for structural members. Eurocode EN 1992-1-2:2004 suggests simplified methods which include explicit equations for fire resistant design. The aim of this paper is to familiarize the simplified method, i.e., 500°C isotherm method. The procedure is customized for Indian conditions and a parametric study is done to determine the fire rating for flexural elements. Fire ratings recommended in IS 456:2000 is compared with strength criteria by using the 500°C isotherm method. It is also compared by thermal criteria obtained by heat transfer analysis of finite element model. Through these studies, it is shown that for most of the cross-sections, the fire rating obtained from the two methods is higher than that given in IS 456:2000 provisions and the increase in cover has significant effect in increasing fire rating only for lower values of cover to reinforcement.

**Keywords:** 500°C isotherm method, fire rating, transient thermal analysis, tabulated data, structural elements, reinforced concrete

## 1. Introduction

Fire resistance is a measurement of the ability of the structure to resist collapse, fire spread or other failure during exposure to a fire of specified severity or in other words it is the duration a structural member (system) exhibits resistance with respect to temperature transmission, structural integrity, and stability under fire conditions. The fundamental step in designing structures for fire safety is to verify that the fire resistance of the structure or each part of the structure is greater than the severity of the fire to which the structure is exposed. The current prescriptive methods for fire resistance are derived from data obtained from standard fire resistance tests and do not consider the effect of many of the important parameters such as load level, fire scenario, and concrete strength (Kodur and Dwaikat, 2008).

Although, there have been a number of studies conducted on the behavior of reinforced concrete (RC) structures in fire conditions, such studies have largely been research oriented and used specially developed software. Therefore, it is necessary to generalize the analysis using commercial software packages such as ANSYS, ABAQUS, or others. In the present work, a parametric study is done using the 500 °C isotherm method and heat transfer analysis using general finite element software ANSYS. The fire ratings provided in IS 456:2000 are compared with strength criteria by using 500°C isotherm method. It is also compared by thermal criteria using heat transfer analysis of the finite element model.

## 2. Different Methods for Assessment of Fire Resistance

The fire resistance of concrete structural elements can be evaluated using different approaches like tabulated data, standard fire tests, advanced calculation methods and simplified calculation methods (Buchanan, 2001). These methods are briefed in following sections.

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### 2.1 Tabulated data

Tabulated data that are incorporated in most of the codes and standards are available only for structural elements. These data are quite useful in preliminary design stage; but is not applicable for structural systems. The main drawback of this method is that the backgrounds to the data are not very clear. IS code provisions for fire resistance are based on these tabulated data in which there are provisions for minimum cross-section and cover to reinforcement for various structural elements based on support conditions. The code does not mention any further data or other procedures that are available in various foreign codes (Dwaikat and Kodur, 2008).

### 2.2 Standard fire tests

Fire resistance testing is usually done on structural elements such as beams, columns, floors, or walls of specific dimension subjected to a standard fire exposure (like ASTME119, ISO curves) in a specially designed fire test furnace. The failure criterion for this test may be generally based on a simple limit, such as unexposed side temperature or critical limiting temperature in steel (normally taken as 593°C). The fire test is not accurate if scaling is done. This approach provides only minimum data for validation and is too expensive and time consuming.

Full-scale fire tests done on structural systems are more effective than the fire resistance tests on structural elements. The studies like Cardington fire test, conducted by the Building Research Establishment (BRE) in UK confirmed that the fire resistance of complex building (structural system) is significantly higher than that of single elements from which the performance is usually assessed (Kodur *et al.*, 2007). These methods even though it is more accurate and gives real behavior of structure are very expensive and cannot be implemented in regular basis.

### 2.3 Advanced calculation methods

This method is a time-dependent thermal and mechanical analysis based on equations of heat transfer and structural mechanics performed to assess the fire resistance. These are also called numerical methods and are implemented using tools like finite difference method, finite element method, and boundary element method. There are general purpose finite element softwares such as ANSYS, ABAQUS, SAFIR, etc., which are available for analyzing the fire response of structural members and assembly by this numerical approach.

### 2.4 Simplified calculation methods

The above discussed methods cannot be used for routine design calculations performed in a design office. In such cases simple analytical methods are required to predict the capacity of structural elements. So, the next option is the simplified calculation methods. These methods are usually the direct extrapolation to higher temperature of traditional methods that are used in ambient conditions. Different methods are available for each combination of material and element type. Such methods are recommended in FIP-CEB recommendations and in the European Standard EN 1992-1-2:2004 (E) (Eurocode, 2000).

## 3. Introduction to IS Code Provisions

The provisions for fire resistance given in IS 456:2000 and IS 1642:1988 are same and are based on the tabulated data in which there are provisions for minimum cross-section and cover to reinforcement for various structural elements based on support conditions. Table 1 and 2 show the minimum width and cover required for beams and slabs for various fire ratings varying from 30 minutes to four hours.

## 4. Simplified Design Procedures for Fire Resistance Given in EN 1992-1-2:2004 (E)

Eurocode suggest two methods for the fire resistant design of concrete structural elements: a) 500°C isotherm method and b) zone method. The 500°C isotherm method is applicable to both a standard fire exposure and any fire curves, which cause similar effect in the fire exposed members. Concrete section with temperatures lower than

Table 1. Minimum dimensions and nominal cover to meet specified period of fire resistance for RC beam (IS 456:2000).

| Nature of Construction Materials | | Minimum Dimensions (mm), excluding any finish for a fire resistance of | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0.5h | 1 h | 1.5 h | 2 h | 3 h | 4 h |
| 1 Reinforced concrete (simply supported) | Width | 80 | 120 | 150 | 200 | 240 | 280 |
| | Cover | 20 | 30 | 40 | 60 | 70 | 80 |
| 2 Reinforced concrete (continuous) | Width | 80 | 80 | 120 | 150 | 200 | 240 |
| | Cover | 20 | 20 | 35 | 50 | 60 | 70 |

Table 2. Minimum dimensions and nominal cover to meet specified period of fire resistance for RC slab (IS 456:2000).

| Nature of Construction Materials | | Minimum Dimensions (mm), excluding any finish for a fire resistance of | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0.5h | 1 h | 1.5 h | 2 h | 3 h | 4 h |
| 1 Reinforced concrete (simply supported) | Thickness | 75 | 95 | 110 | 125 | 150 | 170 |
| | Cover | 15 | 20 | 25 | 35 | 45 | 55 |
| 2 Reinforced concrete (continuous) | Thickness | 75 | 95 | 110 | 125 | 150 | 170 |
| | Cover | 15 | 20 | 20 | 25 | 35 | 45 |

500°C is assumed to have full strength and those having a temperature with higher value are discarded. The 500°C isotherm for the specified fire exposure can be calculated using standard fire curve specified using Equation 1. A reduced cross-section is obtained by excluding the concrete outside the 500°C isotherm. The zone method is applicable to the standard temperature curve only. In this method, the cross section is divided into a number of parallel zones of equal thickness (rectangular elements). The mean temperature and the corresponding mean compressive strength of each zone is assessed. The reduction of the cross-section is based on a damaged zone of thickness at the fire exposed surfaces. After the determination of reduced cross section, the fire design follows the normal design procedure. Out of these two methods, as 500°C isotherm method is easy to analyze, a parametric study based on cross-section and cover is done using this method for flexural elements. The fire rating is evaluated based on strength criteria. The failure is assumed to occur at the time when the moment of resistance at elevated temperature is less than the design moment for fire. The design is done using IS code recommendations and the temperature profile for the cross-section and reduction factor for steel reinforcement at elevated temperatures are taken from EN 1992-1-2:2004 (E). The basic steps for the design are detailed below.

### 4.1 Fire load modeling

The first step is to determine the fire load; with this the temperature distribution of the structure subjected to fire is established. For this purpose the various fire loads like ASTM E119 fire, ISO 834 fire, hydrocarbon fire etc., are the standard fire scenarios used to develop fire rating in various countries. In present study, ISO 834 fire curve which is also given in IS 3809:1979 is used. The curve is defined according to the equation,

$$T = T_0 + 345 \log_{10}(8t+1) \qquad (1)$$

where $T$ is the applied temperature (°C), $t$ is the time (minutes) and $T_0$ is the ambient temperature (°C).

### 4.2 Temperature profiles for concrete elements

The establishment of temperature depended properties of material are important for understanding the behavior of structures in fire. Hence there is a need to get the temperature profile of structural elements subjected to fire loads for various time of exposure. A common way of providing the temperature data is by using graphical presentation given in design codes or from numerical methods. In the present study, temperature profiles available in EN 1992-1-2:2004 (E) are used.

### 4.3 500°C isotherm method

This method is applicable to both a standard fire exposure and any fire curves, which cause similar effect in the fire exposed members. Concrete section with temperatures lower than 500°C is assumed to have full strength and those with higher value are discarded.

The basic design procedure as per 500°C isotherm method available in EN 1992-1-2:2004 (E) is given below.

1. The 500°C isotherm for the specified fire exposure is calculated using standard fire or parametric fire.
2. A reduced width $b_f$ and effective depth $d_f$ of the cross-section is obtained by excluding the concrete outside the 500°C isotherm. The temperature of the individual reinforcing bars is evaluated from the temperature profiles in Annex A of EN 1992-1-2:2004 (E). Those reinforcing bars which fall outside the reduced cross-section may also be included in the calculation of the ultimate load carrying capacity of the fire exposed cross section. In the present study for lower exposure times, all reinforcement bars fall inside the reduced cross-section. But as the time of exposure increases the cross-section size decreases and hence some reinforcement bars fall outside the reduced cross-section. In those cases, those reinforcements are accounted to calculate ultimate load capacity.
3. The reduced strength of the reinforcement due to the temperature is determined according to Cl. 4.2.4.3 of EN 1992-1-2:2004 (E). The corresponding reduction factor is shown in Table A1 of Appendix I

4. The conventional calculation methods for the determination of the ultimate strength based on limit-state design specified in Indian code is used to find the ultimate load-carrying capacity for reduced cross-section with strength of the reinforcing bars as obtained from step 3 and

5. The ultimate load-carrying capacity is compared with the design capacity or, alternatively, the estimated fire resistance with the required resistance.

The above mentioned method is used to determine the moment capacity and hence the failure time for flexural members. In this work the beams are exposed to fire from bottom and sides while slab is heated only from bottom. Therefore the temperature in slab varies only across the thickness. Hence, heat transfer in slab can be assumed as one dimensional in 500°C isotherm method.

## 5. Numerical Methods

The standard fire resistance test performed in a furnace has been used quite intensively for the evaluation of the fire endurance of structural elements, yet in its present form the test procedure has several shortcomings, like the preparation of the experiment and delays involved, the cost of the test, the size of the element to be tested, the heating and restraint characteristics. Therefore the need for analytical predictions of thermal and structural responses has grown more and more intensively (Dotreppe and Franssen, 1985). In the case of steel structures it is usually accepted that they can be analyzed using simple methods of calculation. This is due to the fact that the temperature of steel does not vary much from one point to another in the same element. This is no longer true when a considerable amount of concrete is present, which is the case for composite and RC structures. Furthermore, the study of the mechanical behavior of concrete at high temperature is complicated. Therefore it is necessary to use more refined models for these types of structures: a step-by-step analysis should be performed taking into account material and geometric non-linearity. To solve these problems it is necessary to use data on thermal and mechanical properties of steel and concrete at high temperature (Kodur *et al.*, 2008). The present work includes a heat transfer analysis using finite element model and the material properties for the analysis are taken from EN 1992-1-2:2004 (E).

### 5.1 Material properties

Concrete generally has good fire resistance properties. The temperature depended material properties are important for establishing an understanding of the fire response of reinforced concrete structures. IS code does not give any provision for material properties of either concrete or steel. These temperature depended properties are specified in codes such as EN 1992-1-2:2004 (E) (Eurocode, 2000) and in ASCE manual (ASCE, 1992). For the analysis presented here, the thermal properties of concrete specified in EN 1992-1-2:2004 (E) are used. The temperature-dependent thermal properties (thermal conductivity, specific heat, and density) of concrete are used in the present work for thermal analysis. The specific heat of concrete for both carbonate and siliceous aggregates and also the lower and upper limit of thermal conductivity of normal strength concrete are given in EN 1992-1-2:2004 (E). The lower limit gives more realistic temperatures for concrete structures than the upper limit, which has been derived from tests for steel / concrete composite structures. The present study is carried out using siliceous aggregate and by using lower limit of thermal conductivity. EN 1992-1-2:2004 (E) gives specific heat values for various moisture content. The specific heat values are taken for concrete with moisture content 1.5%. It can be seen from Figure 1 that these properties of concrete vary significantly with temperature, with large decrease in strength of concrete once the temperature exceeds 500°C. Heat transfer from fire to element is by convection on sides with a convection film coefficient of 25 W/m$^2$K (Kodur *et al.*, 2008).

### 5.2 ANSYS finite element model

A non-linear finite element analysis was done to find the thermal behavior of various flexural elements. A two dimensional (2-D) model is generated in ANSYS to perform the thermal analysis. A non-linear temperature distribution analysis was carried out using ANSYS software. For the thermal analysis, a 4-noded quadrilateral plane stress element named PLANE 55 from ANSYS element library was used for modeling concrete. It can be used as a plane element or as an axisymmetric ring element with a 2-D thermal conduction capability. The element has four nodes with a single degree of freedom i.e., temperature, at each node. The element is applicable to a 2-D, steady-state or transient thermal analysis (ANSYS 2010). Figure 2 shows the geometry of the element which is used for thermal analysis in ANSYS.

The convection or heat flux (but not both) and radiation may be input as surface loads at the element edges. At time zero minute, uniform temperature of 20°C is applied. A transient thermal analysis was performed by dividing it

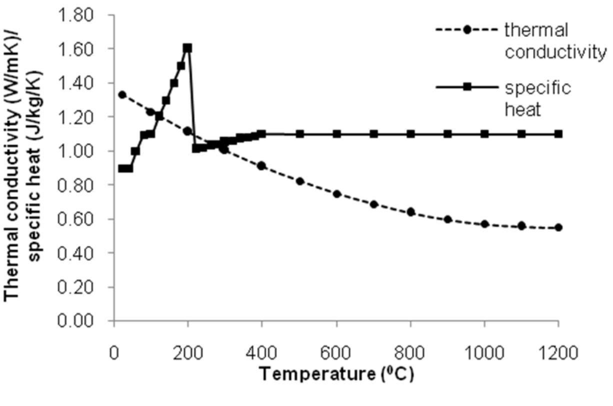


Figure 1. Thermal properties of concrete as a function of temperature.

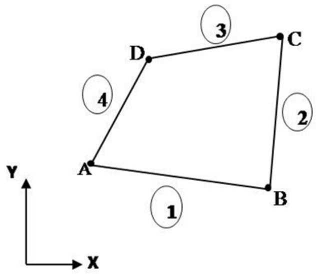


Figure 2. Geometry of PLANE 55 element.

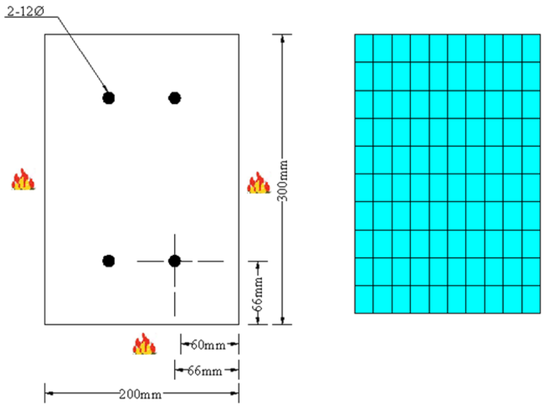
into number of sub steps. Finite element structural elements having the same dimensions as given in the IS 456:2000 fire rating is modeled in ANSYS. Each cross-section is then meshed using “mapped” command. Each element in the meshed cross-section has a size of 10×20 mm for beam and 100×15 mm for slab. The details of cross-section and discretization are shown in Figure 3 and 4. 2-D thermal analysis is done for various cross-sections of structural elements which are used in the parametric study using 500°C isotherm method for various time of exposure starting from 30 minutes to four hours which was considered as maximum fire exposure time in various codes. The details of the fire loading curve used for the model are given in Section 4.1.

### 5.3 Analysis type

A transient thermal analysis was carried out by dividing the fire load into a number of sub steps. For comparison with codal provisions, the standard exposure times specified in IS 456:2000 are used for present analysis. The sub steps are set to indicate load increments used for this analysis. Figure 5 shows a typical temperature contour of beam of cross-section 200×300 mm and for time of exposure 180 min. The variation of temperature for slab of 200 mm thickness for different exposure times is shown in Figure 6.

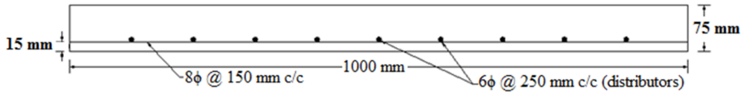
## 6. Comparison of 500°C Isotherm Method and ANSYS Results with IS 456:2000 Provisions

A parametric study is done using 500°C isotherm method and the fire ratings based on strength criteria are compared with thermal analysis results from ANSYS which is based on thermal criteria and tabulated data given in IS 456:2000. All the cross-section of flexural members mentioned in IS 456:2000 are used for comparison. A typical calculation detail of 500°C isotherm method for both slab and beam is given in Appendix I. The span length, amount of reinforcement, dead load and live load are kept constant for all the sections analyzed and is given in Appendix I. The IS code

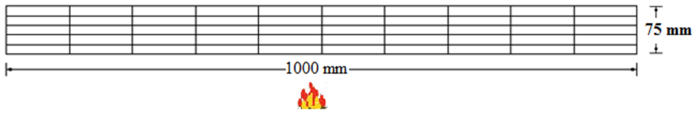


(a) cross-section details (b) discretization and boundary conditions for thermal analysis

Figure 3. Cross-section of RC beam and its discretization for FE analysis.



(a) cross-section details



(b) discretization and boundary conditions for thermal analysis

Figure 4. Cross-section of RC slab and its discretization for FE analysis.

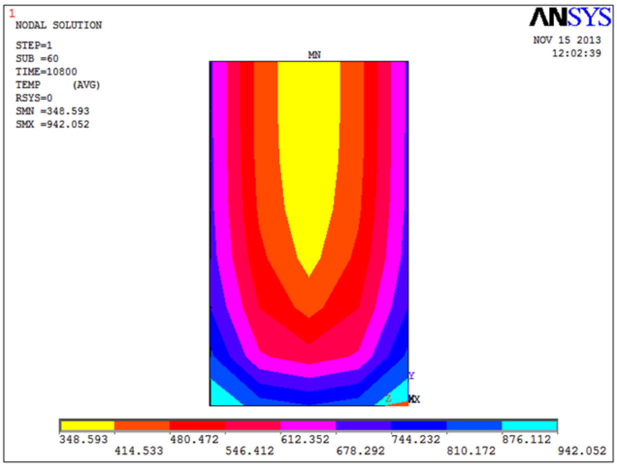


Figure 5. Temperature contour for beam of cross-section size 200× 300 mm for t = 180 min.

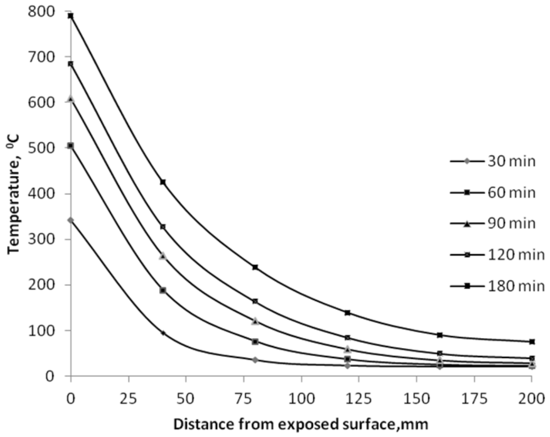


Figure 6. Temperature profile in 200 mm-thick siliceous aggregate slab.

provisions of fire resistance are not specified based on utilization ratio of load. i.e., the ratio of applied load under fire conditions to the nominal capacity of the flexural member at ambient temperature (Kodur and Dwaikat, 2008). Therefore in this study the load ratio is taken as 100%. The parametric study is done for slab and beam by varying only the size of cross-section and clear cover.

The fire ratings from ANSYS results are obtained based on two different thermal failure criteria. The failure criterion for thermal analysis of slab is based on insulation criterion which is defined as for non-exposed surface temperatures, the average heat transmission criterion of temperature rise of 140°C above ambient (IS 3809, 1979). The failure criterion for both beam and slab is defined as the temperature in the longitudinal steel (tension reinforcement) exceeds the critical limiting temperature in steel, normally taken as 593°C (Kodur and Dwaikat, 2008). For slab both the criteria are checked and failure occurs first due to insulation criteria and the fire rating is noted as per this criterion. For 2-D thermal analysis, the reinforcement temperature is assumed as same as the concrete temperature at corresponding position.

The comparison of fire rating for beams and slabs are given in Tables 3 and 4. The fire ratings obtained by the 500°C isotherm method and ANSYS results are compared with the recommended fire rating in IS 456:2000. The studies show that the IS code provisions of fire resistance for beams of smaller cross-section are safe. For beams with cross-section that are usually used in practice gives sufficient fire resistance only when they are provided with a large clear cover. The code does not provide sufficient supporting data for fixing the cross-section and cover for required fire resistance. In case of slabs, the fire resistance obtained from both methods is found to be more than that specified in IS 456: 2000 for smaller thickness. As the thickness increases the ANSYS results are showing a lower fire resistance than that given in IS 456:2000.

## 7. Parametric Study For Simply Supported Beam

A parametric study is done for beam of size 400×800 mm using 500°C isotherm method. The varying parameters are cover to reinforcement and exposure time. A typical beam of span 10 m and cross-section 400×800 mm is considered to find the fire resistance based on strength criteria. The ultimate moment of resistance is calculated as per limit state design recommended in IS 456:2000. The beam is subjected to three

Table 3. Comparison of fire rating for beam from various approaches.

| Size (mm) | Cover (mm) | Fire rating (minutes) | | |
|---|---|---|---|---|
| | | IS 456:2000 | 500°C isotherm method | ANSYS |
| 80×150 | 20 | 30 | 42 | 47 |
| 120×150 | 30 | 60 | 72 | 78 |
| 150 × 300 | 40 | 90 | 114 | 90 |
| 200 × 300 | 60 | 120 | 222 | 178 |
| 240 × 300 | 70 | 180 | 234 | 212 |
| 280 ×300 | 80 | 240 | >240 | 238 |

Table 4. Comparison of fire rating for slab from various approaches.

| Thickness (mm) | Cover (mm) | Fire rating (minutes) | | |
|---|---|---|---|---|
| | | IS 456:2000 | 500°C isotherm method | ANSYS |
| 75 | 15 | 30 | 90 | 50 |
| 95 | 20 | 60 | 117 | 70 |
| 110 | 25 | 90 | 162 | 87 |
| 125 | 35 | 120 | 180 | 105 |
| 150 | 45 | 180 | >240 | 139 |
| 170 | 55 | 240 | >240 | 180 |

side fire exposure. The dead load on the beam is 6.0 kN/m and live load is 10.0 kN/m. The cross-section details are shown in Figure 7. Table 5 shows the variation of bending strength with variation in concrete clear cover. The reinforcement bars are grouped as 1, 2 and 3 as shown in Figure 7. The reinforcement bars having same temperature are grouped together and named as one group and the corresponding reduction factors are calculated using Table 3.2a of EN 1992-1-2:2004 (E) which is given in Table A1. Reinforcement bar temperature and corresponding bending strength are calculated using 500°C isotherm method. It is observed that for all the time of exposures, increase in clear cover thickness has significant effect on the bending strength (fire calculation) at initial stages. However, large increase in cover does not have significant effect on bending strength. That is if the cover is increased from 25 to 30 mm it shows an increase of 25% in

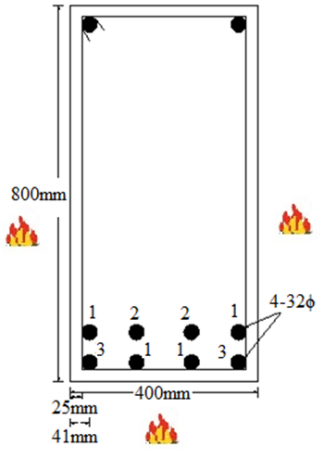


Figure 7. Cross-section details of RC beam of size 400×800 mm.

Table 5. Variation of bending strength with increase in concrete cover thickness for different time of exposure for beam of size 400×800 mm using 500°C isotherm method.

| Time of exposure (hrs) | Clear cover (mm) | Bending strength (kN-m) | |
|---|---|---|---|
| | | Ambient calculation ($M_n$) | Fire Calculation ($M_{nf}$) |
| 1.5 | 25 | 910 | 450 |
| | 30 | 900 | 560 |
| | 40 | 883 | 790 |
| | 50 | 866 | 795 |
| | 60 | 850 | 843 |
| 2 | 25 | 910 | 433 |
| | 30 | 900 | 541 |
| | 40 | 883 | 767 |
| | 50 | 866 | 774 |
| | 60 | 850 | 818 |
| 3 | 25 | 910 | 409 |
| | 30 | 900 | 505 |
| | 40 | 883 | 693 |
| | 50 | 866 | 704 |
| | 60 | 850 | 748 |

bending strength whereas, if the cover is increased from 50 to 60 mm it shows only a 6% increase. Therefore, it may be concluded that increase in cover has significant effect in increasing fire resistance only for initial values. Beyond 40 mm clear cover thickness, the bending strength has no significant variation for different time of exposure.

## 8. Conclusions

The following conclusions are drawn from the present study. The code does not provide any data regarding the failure criteria adopted to obtain the fire ratings, since the cross-section and cover are the only parameters considered. In this study the code provisions are compared with thermal analysis and 500°C isotherm method. The 500°C isotherm method used in the present study is a simplified method which can be used for manual design of structural elements with simple boundary conditions. For beams with cover upto 30 mm and slabs with cover upto 20 mm, fire ratings given in code are less than that of the 500°C isotherm method and finite element results. The fire rating specified in codes are higher than that obtained from finite element results for slabs with cover greater than 20 mm. Hence for slabs with large cover, code provisions are less conservative. The fire ratings obtained from finite element analysis are significantly less than that of the 500°C isotherm method for slabs. It may be due to the reason that slabs are exposed to fire only from bottom surface and the heat transfer is assumed as one dimensional in the 500°C isotherm method for slabs. As a result the strength reduction is less and fire rating will be more. The parametric study done for beam by varying the cover to reinforcement shows that increase in cover has significant effect in increasing the strength for smaller cover (upto 40 mm). But after that there is not much variation in strength even if the cover is increased. Hence, there is a need to modify the code provisions by conducting more rigorous studies.

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## Appendix I

### List of symbols

| | |
|---|---|
| $A_{s1}$ | Area of one reinforcement bar |
| $A_{st}$ | Area of tension reinforcement |
| $d$ | Effective depth |
| $h$ | Overall depth of member |
| $s$ | Spacing of reinforcement |
| $b$ | Width of member |
| $f_{ck}$ | Compressive strength of concrete |
| $f_y$ | Yield strength of reinforcement steel |
| $\rho$ | Concrete density |
| LL | Live load |
| DL | Dead load |
| $w_f$ | Design load for fire |
| $M^*_{fire}$ | Bending moment in fire |
| $t$ | time of exposure |
| $d_f$ | Reduced depth of the section |
| $T_s$ | Temperature of reinforcement steel |
| $k_{\theta,}$ | Reduction factor for yield strength of reinforcement steel |
| $f_{y,T1}$ | Reduced yield strength of reinforcement steel |
| $x_u$ | Depth of neutral axis |
| $b_f$ | Reduced width |
| $c_f$ | Depth of 500°C isotherm |
| $M_n$ | Moment of resistance in ambient calculation |
| $M_{nf}$ | Moment of resistance in fire |

**Design Calculation for Slab**

The cross-section details of the slab used for present study are shown in Figure 4(a). Length of one-way slab, L = 2.0 m (assuming beam spacing as 2m)

Area of one bar, $A_{s1} = 50.24$ mm$^2$

Assuming 1 m width, $A_{st} = \frac{1000 \times A_{s1}}{s} = 335$ mm$^2$ (assuming spacing, $s = 150$ mm)

Overall depth of slab, $h = 75$ mm; Clear cover = 15 mm;

Effective depth, $d = 75 - 15 - \frac{8}{2} = 56$ mm

Concrete compressive strength, $f_{ck} = 25$ MPa; Steel yield stress, $f_y = 415$ MPa

**Dead load Calculation**

Concrete density, $\rho = 25$ kN/m$^3$

Self-weight of slab = 0.075×25×1 = 1.875 kN/m

Finishing = 1 kN/m

Total dead load, DL = 1.875+1.0 = 2.875 kN/m

**Fire Calculations**

Live load, LL = 4 kN/m

Design load (fire), $w_f$ = DL + 0.5 LL

= 2.875 + 0.5 × 4 = 4.875 kN/m

Bending moment, $M^*_{fire} = \frac{w_f l^2}{8}$

$$= \frac{4.875 \times 2^2}{8} = 2.44 \text{ kN/m}$$

Fire duration, $t = 30$ minutes

**500°C Isotherm Method**

Therefore, $d_f = 56$ mm

Steel temperature for reinforcements is taken from the temperature profile provided in appendix A of EN 1992-1-2:2004 (E). The corresponding reduction factor for yield strength of reinforcing bars at elevated temperatures is taken from Table 3.2a: EN 1992-1-2:2004 (E) and is shown in Table A.1.

Bar group (1), $T_s = 350°\text{C}$, $k_{\theta,1} = 0.97$

Design reduced yield strength

$f_{y,T1} = k_{\theta,1} \times f_y = 0.97 \times 415 = 402.55$ N/mm$^2$

**Design Equations for Flexural Elements**

The following section explains the design procedure for flexural members subjected to elevated temperature. The equations are provided for a general cross-section and reinforcement based on Figure A1. These equations are used to evaluate the moment capacity of beams and slabs analyzed in Section 6 and 7.

In the case of concrete, while '$f_{ck}$' is the characteristic cube strength (i.e, 28 day compressive strength of 150 mm cube), the strength of concrete in the actual structure is taken as '$0.67f_{ck}$'.

Partial safety factor of concrete = 1.5;

Maximum design stress of concrete = $\frac{0.67 f_{ck}}{1.5} = 0.45 f_{ck}$

Partial safety factor of steel = 1.15;

Maximum design stress of steel = $\frac{f_y}{1.15} = 0.87 f_y$

Resultant compressive force = $0.36 f_{ck} b x_u$

Depth of the centroid of the stress block from the extreme fibre of the compression zone = $0.42 x_u$

Position of neutral axis for longitudinal equilibrium, equating total compression to total tension, C = T;

i.e., $0.36 f_{ck} x_u b = 0.87 f_y A_{st}$

Depth of neutral axis,

$$xu = \frac{0.87 \times f_y \times A_{st}}{0.36 \times f_{ck} \times b} = \frac{0.87 \times 402.55 \times 335}{0.36 \times 25 \times 1000} = 13.04 \text{ mm}$$

Moment of resistance,

$$M_{nf} = 0.87 A_{St} f_{y,T} (d - 0.42 x_u)$$

$$= 0.87 \times 335 \times 402.55\,(56 - 0.42 \times 13.04) = 5.93 \text{ kNm}$$

$M_{nf} > M^*_{fire}$, so, design is safe.

**Design Calculation for Simply Supported Beam**

The cross-section details of the beam used for present study are shown in Figure 3(a).

Beam span, L = 2.0 m; Area of one bar, $A_{s1} = 113.09$ mm$^2$;

Total steel area, $A_s = 226.18$ mm$^2$; Clear cover = 60 mm;

Effective depth, $d = 300 - 60 - \frac{12}{2} = 234$ mm;

Concrete compressive strength, $f_{ck} = 25$ MPa;

Steel yield stress, $f_y = 415$ MPa

**Dead load Calculation**

Concrete density, $\rho = 25$ kN/m$^3$;

Weight of slab = 0.075×25 +1(finishing) = 2.875 kN/m;

Self-weight of beam = $\rho b h$ = 25×0.3×0.2 = 1.5 kN/m;

Total dead load = 2.875+1.5 = 4.5 kN/m;

Live load = 1 kN/m

**Fire Calculations**

Design load (fire), $w_f$ = Dead Load + 0.5 Live Load

= 4.5 + 0.5 × 1 = 5 kN/m

Bending moment, $M^*_{fire} = \frac{w_f l^2}{8}$

$$= \frac{5 \times 2^2}{8} = 2.5 \text{ kNm}$$

Fire duration, $t = 30$ minutes

**500°C Isotherm Method**

Depth of 500°C isotherm, $c_f = 15$ mm

Reduced width, $b_f = b - 2c_f$

$= 200 - 2\times15 = 170$ mm

Reduced depth, $d_f = 300 - 15 = 285 > d$

Therefore effective depth, $d = 234$ mm

Steel temperature for reinforcements is taken from the temperature profile provided in Appendix A of EN 1992-1-2: 2004 (E). The corresponding reduction factor for yield strength of reinforcing bars at elevated temperatures is taken from Table 3.2a: EN 1992-1-2:2004 (E) and is shown in Table A1.

Bar group (1), $T_s = 100$°C, $k_{\theta,1} = 1.0$

Design reduced yield strength

$f_{y,T1} = k_{\theta,1} \times f_y = 1.0\times415 = 415$ N/mm$^2$

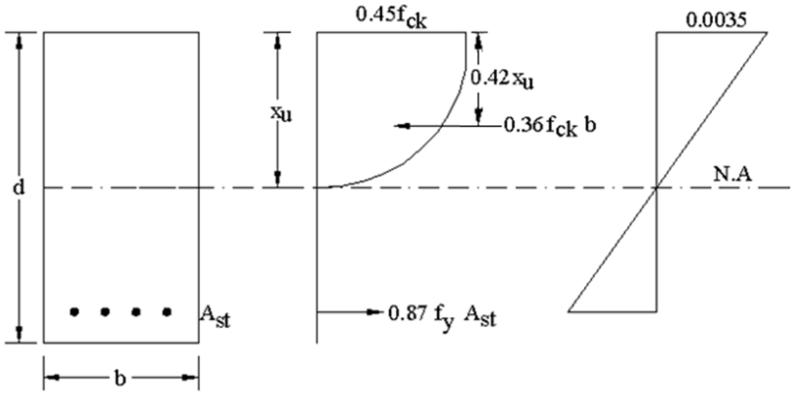
$$x_u = \frac{0.87\times415\times226.18}{0.36\times25\times170} = 53.37\ \text{mm}$$

Moment of resistance, $M_{nf} = 0.87\times226.18\times415(234-0.42\times53.37) = 17.72$ kNm

$M_{nf} > M^*_{\text{fire}}$, so, design is safe.

Table A1. Reduction factor for yield strength of reinforcement steel as per EN 1992-1-2:2004 (E).

| Steel Temperature $\theta$ (°C) | $k_\theta$ | |
|---|---|---|
| | hot rolled | cold worked |
| 20 | 1.00 | 1.00 |
| 100 | 1.00 | 1.00 |
| 200 | 1.00 | 1.00 |
| 300 | 1.00 | 1.00 |
| 400 | 1.00 | 0.94 |
| 500 | 0.78 | 0.67 |
| 600 | 0.47 | 0.40 |
| 700 | 0.30 | 0.12 |
| 800 | 0.11 | 0.11 |
| 900 | 0.06 | 0.08 |
| 1000 | 0.04 | 0.05 |
| 1100 | 0.02 | 0.03 |
| 1200 | 0.00 | 0.00 |



(a) Cross-section details (b) Stress distribution and resultant forces (c) Strain distribution

Figure A1. Cross-section and stress block parameters.