Structural Engineering and Mechanics, *Vol. 59, No. 5 (2016) 853-866*
DOI: http://dx.doi.org/10.12989/sem.2016.59.5.853

Reliability studies on RC beams exposed to fire based on IS456:2000 design methods

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(Received November 16, 2015, Revised March 29, 2016, Accepted May 5, 2016)

Abstract. This paper examines a methodology for computing the probability of structural failure of reinforced concrete beams subjected to fire. The significant load variables considered are dead load, sustained live load and fire temperature. Resistance is expressed in terms of moment capacity with random variables taken as yield strength of steel, concrete class (or grade of concrete), beam width and depth. The flexural capacity is determined based on the design equations recommended in Indian standard IS456:2000. Simplified method named 500℃ isotherm method detailed in Eurocode 2 is incorporated for fire design. A transient thermal analysis is conducted using finite element software ANSYS® Release15. Reliability is evaluated from the initial state to 4h of fire exposure based on the first order reliability method (FORM). A procedure is coded in MATLAB for finding the reliability index. This procedure is validated with available literature. The effect of various parameters like effective cover, yield strength of steel, grade of concrete, distribution of reinforcement bars and aggregate type on reliability indices are studied. Parameters like effective cover of concrete, yield strength of steel has a significant effect on reliability of beams. Different failure modes like limit state of flexure and limit state of shear are checked.

Keywords: reinforced concrete; fire exposure; reliability index; failure modes; IS456:2000

1. Introduction

Fire remains one of the serious potential risks to most buildings and structures. The extensive use of concrete as a structural material has led to the need to fully understand the effect of fire on concrete. Fire safety in buildings has been evolved as one of the prime objectives in concrete design after incidents such as the World Trade Centre attack on September 11, 2001 (U.S.A) and the terrorist attack in the Taj Hotel on November 26, 2011 (INDIA). This has created awareness and renewed interest in fire resistant design of buildings worldwide. Most of the buildings in India are constructed using concrete and hence this study is concentrated in that area. The fire rating of structural elements are determined either by conducting a standard fire test such as those specified in ASTME119 (ASTM 2001), ISO 834 (ISO834 1989) or IS 3809 (IS3809 1978) or by

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http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

calculations based on simplified methods or from the tabulated ratings specified in various codes (Eurocode 2 2004, IS456 2000, ACI216R 2007). Most of the codes rely on tabulated data for fire resistance prediction. The tabulated data specify only the minimum cover and size of cross-section required for a particular fire rating. The Indian Standard code for plain and reinforced concrete (IS456 2000) provides only these tabulated data, which are the conditions deemed to be satisfied for fire resistant design. These data are insufficient to determine the fire rating properly. Eurocode 2 suggests two simplified methods for the fire resistant design of concrete structural elements: a) 500℃ isotherm method and b) Zone method. The numerical analysis based on the finite element (FE) model is another alternative for determining the fire response of structures. The popularity of numerical techniques has facilitated to simulate the fire conditions which have reduced the number of expensive fire tests. These techniques made fire analysis easier and well accepted.

The fire rating determined using above deterministic methods is very prescriptive. Fire exposure of structures comes under the category of extreme loads and the behavior of structures in fire is mostly unpredictable. The performance of the structures when exposed to fire is assessed by its safety, serviceability and economy. The absolute safety of the structures is impossible due to the unpredictability of fire exposure, structural loads and material strength of the structural members. In conventional deterministic fire analysis and design methods, it is assumed that these parameters are not subjected to statistical variations. Fire ratings provided in various codes and standards may not be adequate in this circumstance. Therefore, the probability of failure or the prediction of reliability of structures when exposed to fire is much relevant in this context.

Reliability is defined as the ability of a structure or structural element to satisfy certain design requirements under given operating conditions during its life time. Each of these design requirements may be termed as the limit state. Violation of these limit states is treated as failure state of structures (Mislav Stepinac 2012, Ranganathan 1990). For the past few years, limited studies are done on the probabilistic approach to fire resistant analysis of reinforced concrete (RC) structures. Simplified equations of heat transfer like Wickstrom equations which assume one dimensional heat flow can be used to predict the temperature propagation and reduced cross-section when exposed to fire (Wickstrom 1986, Eamon and Jenson 2012, Eamon and Jenson 2013a, b). The temperature propagation inside the structure depends on various material properties such as thermal conductivity, specific heat, moisture content etc., (Eurocode2 2004, Kodur 2008). In the present work, a parametric study was done to find the effect of two types of aggregates namely siliceous and carbonate aggregate. The thermal conductivity of these aggregates differs much and it has significant influence on temperature propagation inside the cross-section of beam. The variation in thermal conductivity values can be directly incorporated in the FE model. Moreover, the thermal contours of cross-section vary with fire loads and exposure conditions. The temperature contours developed based on these properties cause rise in temperature inside the structural element and reduces the load carrying capacity of members with time due to the deterioration of materials at high temperature. This causes reduction in the reliability of RC beams with time **(**Wang *et al*. 2010, Le and Xue 2014). To account all these variations in the thermal model, a numerical analysis is needed. The present study focuses on the reliability analysis of structural elements, based on results of thermal analysis done on the FE model.

The essential components of a reliability analysis procedure include analytical modeling, formulation of functional requirements, identification of random variables and verification (Petterson 1994). Level II reliability analysis is used in this study and a programme is coded in MATLAB. The design equations specified in IS456:2000 are considered as design requirements. The equations are modified using 500℃ isotherm method suggested in Eurocode (Eurocode 2

2004).

2. Development of limit state functions

The major step in any reliability analysis is the formulation of failure functions based on various failure criteria. Then the random variables involved are identified and its statistical data are collected (Lu *et al.* 1994). Probabilistic calculations were done by First Order Reliability Methods (FORM) where a reliability index is estimated based on limit state functions. The reliability index (β) is related to the probability of failure (p_f), as $p_f = \Phi(-\beta)$, where Φ is the cumulative distribution factor. The reliability analysis was carried out by the First Order Reliability Method (FORM) based on limit state functions formulated. The Hasofer-Lind (H-L) approach was used for the solution. H-L approach requires that all random variables are normally distributed. Hence, the non-normal variables are transformed to equivalent normal distributions at the design point using Rackwitz and Fiessler algorithm (Madson 1986). In the present work, two limit states are taken into account; limit state of collapse against flexure and limit state of collapse against shear. Failure functions are formulated based on Indian standard codes and the literatures available (Eurocode 2 2004, IS456 2000, IS1308 1979).

2.1 Limit state of flexure

The limit state of flexure is considered in terms of the moment of resistance and bending moment. (Ranganathan 1990, Eamon and Jenson 2012). The moment of resistance of a beam without considering the factor of safety and load factor is given by

$$M_R = f_y A_{st} d \left[1 - \frac{0.42 f_y A_{st}}{0.54 f_c b d} \right] \tag{1}$$

The bending moment due to external loading is given by

$$M_S = \frac{(w_l + w_d)}{8} l^2 \tag{2}$$

Failure function for limit state of collapse with respect to flexure is given by

$$G_f = f_y A_{st} d \left[1 - \frac{0.42 f_y A_{st}}{0.54 f_c b d} \right] - \frac{(w_l + w_d)}{8} l^2 \tag{3}$$

where f_y is the yield strength of steel, f_c is the mean strength of concrete, b is the breadth of beam, d is the depth of beam, l is the span of the beam, w_l is the live load, w_d is the dead load and A_{st} is the area of tension steel. Random variables identified are f_y, f_c, b, d, w_l and w_d. Statistical data of each random variables are taken from the literature (Eamon and Jenson 2013a, b).

2.2 Limit state of shear

Resistance against shear for beam is given by

$$V_R = \tau_c b d + V_{us} \tag{4}$$

Table 1 Statistical distribution of random variables of beam (Eamon and Jenson 2013a, Ranganathan 1990)

Variables		Distribution type	Parameters	Units
limit state of flexure	limit state of shear			
f_y	f_y	Normal	μ=468.9, COV=0.05	MPa
f_c	-	Normal	μ=30.28, COV=0.145	MPa
w_l	w_l	Extreme largest, Type-1	μ=10, COV=0.3	kN/m
w_d	w_d	Normal	μ=5+self-weight, COV=0.05	kN/m
b	b	Normal	μ=250, COV=0.04	mm
d	d	Normal	μ=450, COV=0.04	mm

where τ_c is shear strength of concrete and is calculated as per IS456:2000, Table 23. V_{us} is the shear resistance given by shear reinforcement, calculated as per IS456:2000.

$$V_{us}=\frac{f_y A_{sv} d}{s_v} \tag{5}$$

s_v is the spacing of stirrups, A_{sv} is the area of shear reinforcement.
Shear force due to external load is calculated as

$$V_s=\frac{(w_l+w_d)\,l}{2} \tag{6}$$

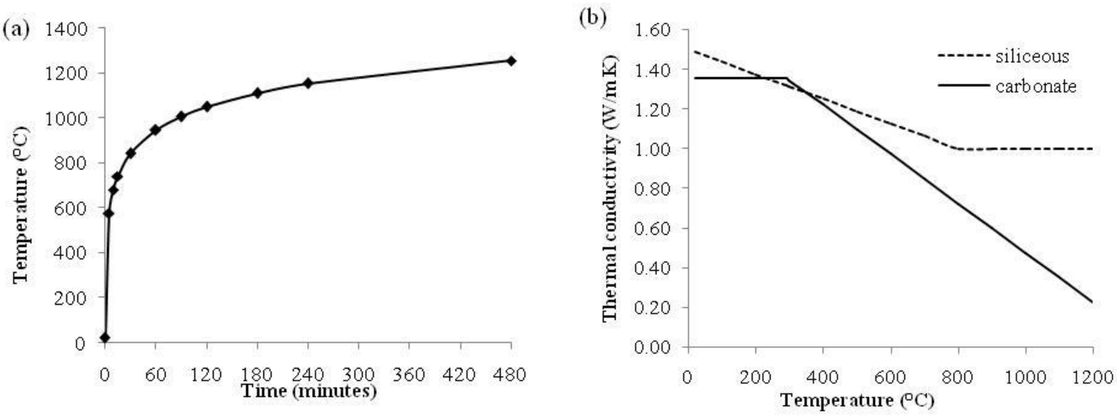
Therefore failure function is

$$\mathrm{G_S}=\tau_c bd+V_{us}-\frac{(w_l+w_d)\,l}{2} \tag{7}$$

Statistical data of each random variables are taken from the literature (Eamon and Jenson 2013a, b) in which the distribution of the strength variables was used as a normal distribution. The distribution of live load (w_l) is considered as Extreme largest, Type-1 (Melchers 2002) and the COV is taken from the literature (Ranganathan 1990).

3. Structural reliability analysis of RC beam

In the present study, it is assumed that the beam is subjected to fire from the bottom and sides only. The capacity of the beam decreases due to reduced strength of steel and concrete, although the former becomes more significant as exposure time increases. To describe the temperature propagation and to determine the strength loss, a thermal analysis has been performed using FE software ANSYS® Release15. The standard time-temperature curve available in ISO 834 is used as fire load to obtain thermal profiles and is shown in Fig. 1(a). Material properties like thermal conductivity and specific heat are defined for the heat transfer analysis and are taken from Eurocode 2 as shown in Fig. 1(b). The approximate design values of specific heat are 1000 J/kgK for siliceous and calcareous aggregate concrete. The density of concrete is taken to have a constant value of 2300 kg/m^3. The lower limit of thermal conductivity and specific heat for dry concrete



(a) Fire load ISO 834 fire curve (b) Thermal conductivity of concrete

Fig. 1 Input parameters for ANSYS thermal analysis

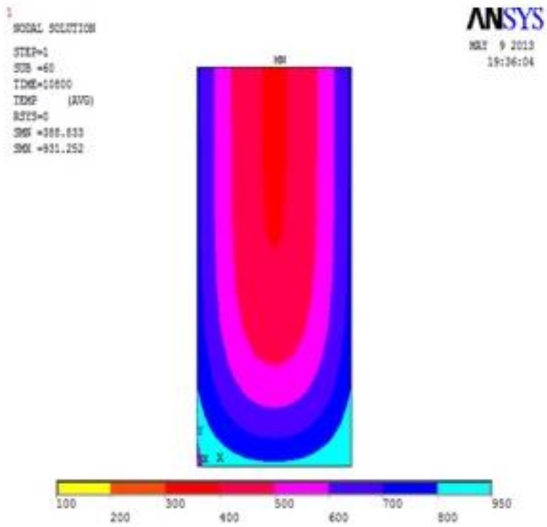


Fig. 2 ANSYS- thermal analysis contour plot

with moisture content 0% specified in Eurocode 2 is adopted in this work (Eurocode 2 2004). A nonlinear thermal analysis is done for RC beams exposed to fire. Solid70 element is used to model concrete and Link33 element to model steel reinforcement. The thermal analysis is performed for different time of exposure varying from 0-4 hours. The FE model used in this work has been able to match the internal temperatures of a column exposed to high temperatures (Balaji *et al*. 2016) and different studies were conducted using the model (Balaji *et al*. 2015).

In thermal analysis SOLID70 element is used for modeling concrete which has a 3-D thermal conduction capability and has eight nodes with a single degree of freedom (i.e., temperature) at each node. The temperature propagation in the cross-section of the member is predicted with the help of thermal contours generated from FE analysis for different intervals of time. A thermal contour obtained as output of typical cross-section of the analyzed beam is shown in Fig. 2. From

Table 2 Reduction factors from E&J’s model and ANSYS-thermal analysis

Time of exposure(hours)	Reduction factor	
	E&J's model	ANSYS-thermal analysis
0	1	1
0.5	1	1
0.75	0.81	1
1	0.62	1
1.5	0.36	1
2	0.17	0.95
2.5	0.15	0.78
3	0	0.59
3.5	0	0.43
4	0	0.27

the profiles, the temperature at each node can be located. The strength reduction of steel and reduced cross-section of concrete is calculated with respect to these temperatures. The moment capacity of RC beams is checked using the 500℃ isotherm according to Eurocode 2. According to this method, concrete having temperature above 500℃ is discarded and a reduced cross-section is used to determine the capacity. The thickness of the damaged concrete is established based on the average depth of 500℃ isotherm in the compression zone of the cross-section. The damaged concrete with a temperature in excess of 500℃ is discarded in computing load carrying capacity of the member, while the residual area of cross-section is assumed to retain full strength. After obtaining the reinforcement temperature, the reduction factor is computed. In the reliability analysis, no uncertainties were considered when calculating the effects of temperature in the cross-section. That is, the random variables b, d, f_y, and f_c were taken as deterministic in the FEA when temperature was calculated.

Table 2 gives the comparison in the reduction factor as per E&J’s model and ANSYS thermal analysis. It is observed from the table that there is significant difference in the values of the reduction factor as per E&J’s model and ANSYS thermal analysis. The E&J’s model was calibrated to match the time-to-failure of actual beam fire tests, whereas the model presented in this paper was not validated in this manner. However, the model used in this paper could match the displacements and internal temperatures of a column.

Reliability analysis is carried out in following steps.

1. Thermal analysis is carried out, subjecting the structural elements to exposure of fire.
2. For a given time of exposure, the temperature distribution inside the cross-section is obtained. The reduction of cross-section and reduced yield strength of reinforcement are calculated with the help of this temperature distribution which is given as input data for programme.
3. The reduced resistance of the structural elements and the revised limit state function is obtained. Thus, the limit state functions become time-dependent.
4. The iterations in the study are performed in the time limit and once the convergence reached, the new set of values is given as input to FE model.
5. Using the time-dependent limit state functions, reliability index is calculated.

Table 3 Random variables considered (Eamon and Jenson 2013a)

*Random variables	Mean Value	Units	COV
f_y	474	MPa	0.05
d	562	mm	0.04
$f_{c'}$	34	MPa	0.145
b	308	mm	0.04
Dead load, D	65.6	kN/m	0.05
Live load, L	15	kN/m	0.65
Fire temperature, T	1056.7	℃	0.45
Thermal diffusivity, α	0.417×10^{-6}	m^2/s	0.06

* All distributions are normal, except L which is gamma distribution

In order to check the accuracy of H-L method (Level 2) with Monte Carlo simulation (Level 3), a validation study is performed for reliability analysis of a beam exposed to fire as reported in available literature (Eamon and Jenson 2013a). The beam considered is a rectangular section with b=305 mm, d=610 mm, f_{ck}=28 MPa with siliceous aggregate, 4-9 # tension steel bars, and 38 mm cover. The beam is simply supported with span 4.5 m and uniformly loaded with a load ratio 0.5. The random variables are given in Table 3. The strength reduction factor for steel is obtained from E&J's model (Table 2). The failure function is given by Eamon and Jenson (2013a) is

$$G_f = f_y A_{st}(d-d') + (A_{st} - A_{sc}) f_y \left[d - \frac{a}{2} \right] + M_r - \frac{(w_l + w_d)}{8} l^2 \tag{8}$$

where depth of compressive stress block

$$a = \frac{A_{st} f_y}{0.85 f_c^{'} b} \tag{9}$$

A_{sc} is the area of compression steel; M_r is the resistance caused by axial and/or rotational thermally induced restraints on the section; f_c' is the concrete compressive strength; d and d' are the depths of the tension and compression steel, respectively.

The obtained reliability degradation curve shown in Fig. 3 is similar to that reported in literature (Eamon and Jenson 2013a). It is to be noted that the reliability degradation as per Eamon is obtained by Monte Carlo simulation. For the comparison study, the strength reduction factor used by E&J model is used here. The validated results show that the developed MATLAB code using FORM can be used for various parametric studies. Further studies are carried out using FORM and strength reduction factor is obtained by ANSYS thermal analysis.

4. Parametric study

From various literature, it is observed that the input parameters that affect the capacity and cause probability of failure of RC beams when exposed to fire are effective cover, strength of concrete and steel, reinforcement distribution and aggregate type (Kodur 2008, Eamon and Jenson 2012, Shi and Tan 2004). The beam is initially designed separately for a given set of parameters as

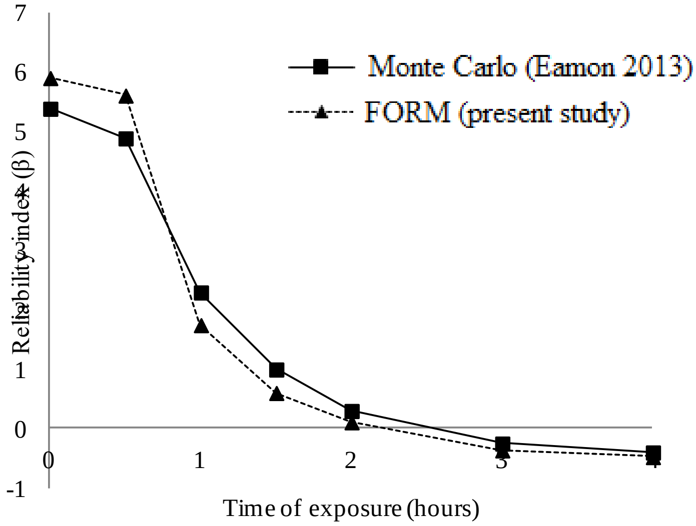


Fig. 3 Reliability index degradation curve for validation

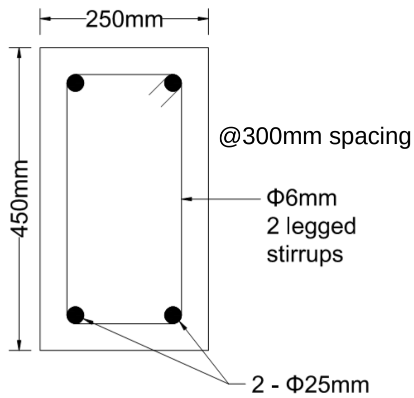


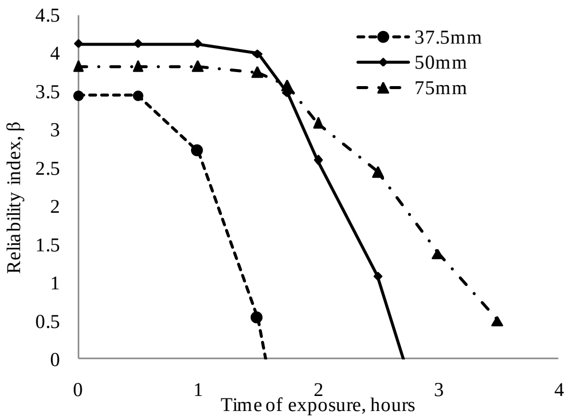
Fig. 4 Cross section of beam

per IS456:2000. Then the reliability analysis is performed by varying time of exposure from zero to 4 hours.

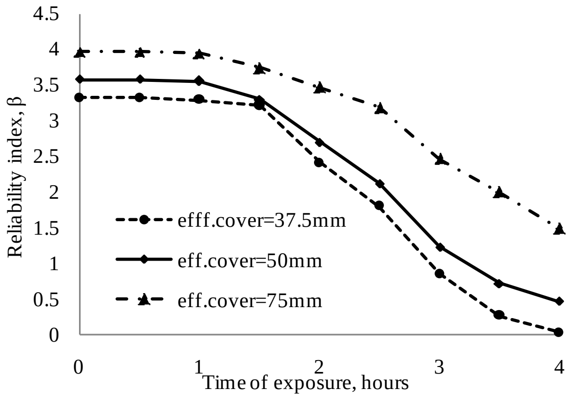
A reliability study is done on the RC beam for various parameters for different failure limit states. The cross-section of the base beam considered in the study is shown in Fig. 4. The other parameters used are: f_{ck}=25 MPa, f_y=415 MPa, effective span, l=6 m, dead load, w_d=7.81 kN/m (including self weight), live load, w_l=10 kN/m, effective cover=37.5 mm. All the parametric studies are done using live load taken from Indian code (IS875-Part2: 1987). The statistical distribution of random variables and coefficient of variation (COV) under limit state of flexure and shear is tabulated in Table 1.

4.1 Influence of effective concrete cover

Beams are designed for different effective cover of 37.5 mm, 50 mm and 75 mm and analysis is



(a) limit state of flexure



(b) limit state of shear

Fig. 5 Effect of cover to reinforcement bar for beam

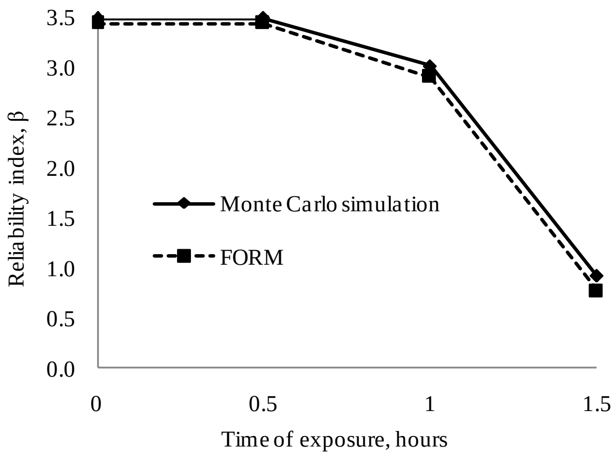
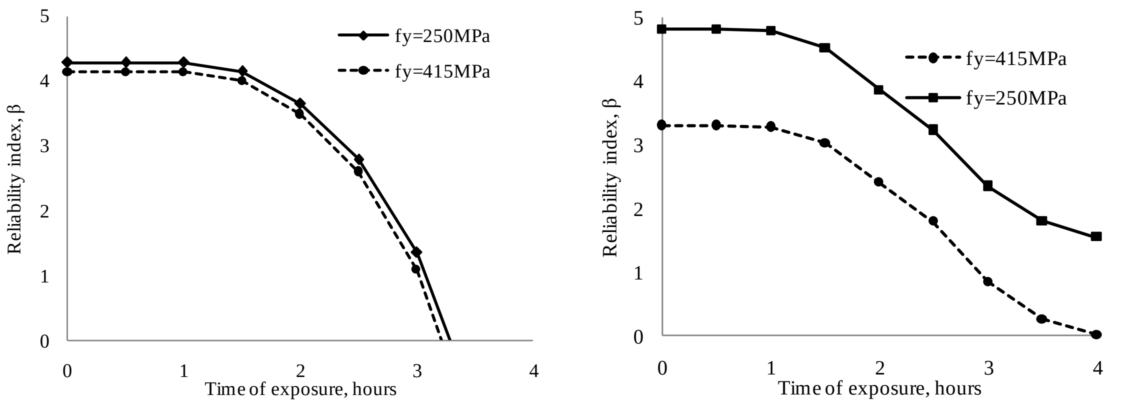


Fig. 6 Comparison of Reliability index degradation curve using different methods

carried out. Fig. 5 gives the variation of reliability index β with time of exposure. From the figure it is noted that the trend in the variation is significantly different from which is observed when E&J's model is used (Fig. 3). This is because, the E&J model used Wickstrom equations to determine the reduced cross-section of concrete while in present study it is based on thermal analysis using FE model. Also, failure with respect to flexure is more critical, compared to shear, in that the rate of decrease in β is higher for flexure. The β remains almost constant initially and then reduces at faster rate. As expected, β increases as the cover increases.

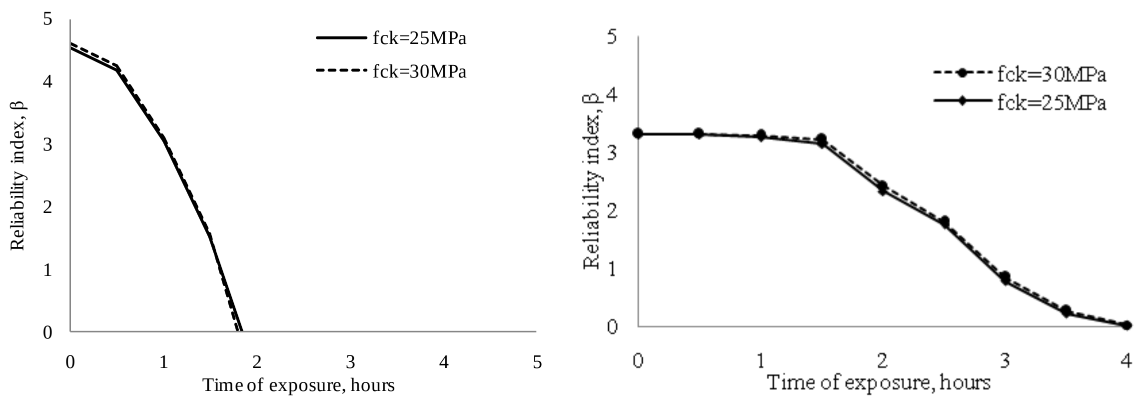
Contrary to the expectations, β values are higher for 50 mm cover. This is due to higher percentage of reinforcement provided than it is really required as per the design due to the limited availability of various diameter reinforcement bars. Nevertheless, concrete cover does affect reliability. Also its effect is more in case of flexure than in shear.

As an additional check, the reliability index (β) values are evaluated using Level 3 method (Monte Carlo simulation) for RC beam with effective cover 37.5 mm and the curves are compared in Fig. 6. By comparing the results from both the analysis, it can be noted that the obtained results



(a) limit state of flexure (b) limit state of shear

Fig. 7 Effect of yield strength of steel for beam



(a) Limit state of flexure (b) Limit state of shear

Fig. 8 Effect of grade of concrete for beam

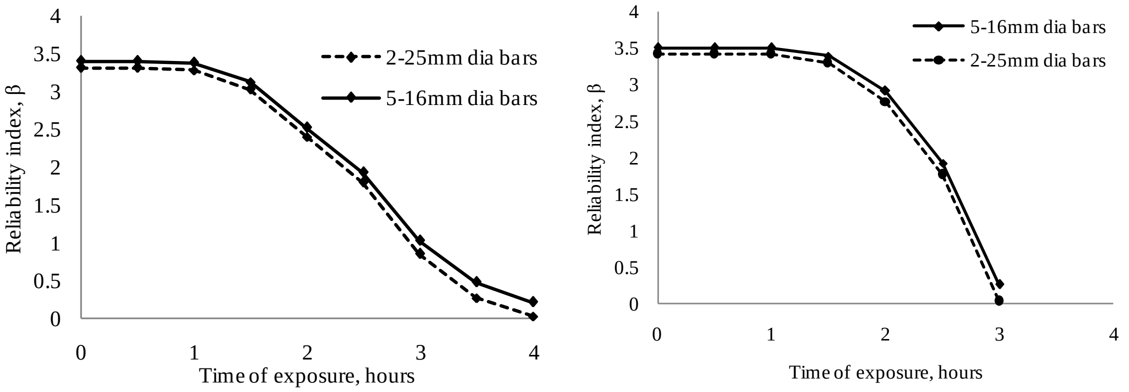
are almost equal. Hence the further studies are done with FORM (Hasofer and Lind's method).

4.2 Influence of yield strength of steel

The beam is designed using two different steel grades namely Fe415 and Fe250. The variation in β with time of exposure is plotted in Fig. 7. The reliability indices values are almost similar for both the steel grades in case of flexure, but are showing a large difference in case of shear. It is attributed to increase in percentage of reinforcement as per design for Fe250 steel and subsequent increase in τ_c value.

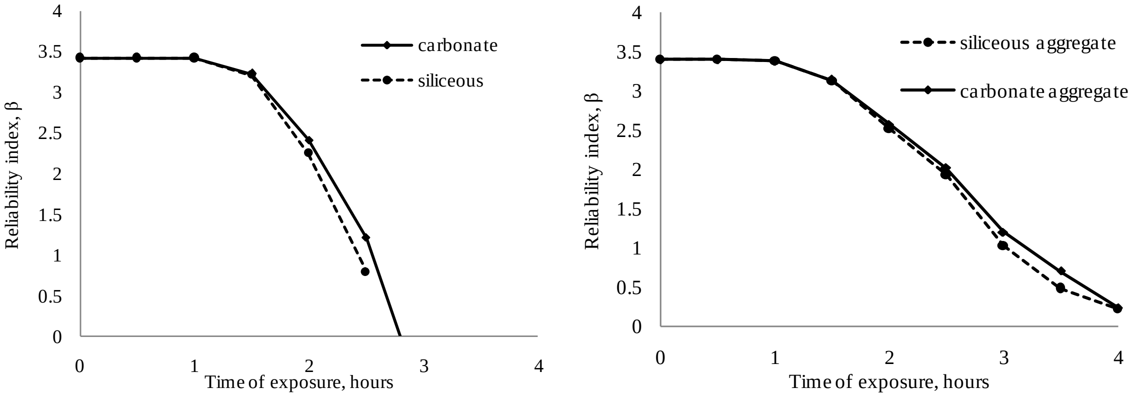
4.3 Influence of grade of concrete

The beam is designed using two different concrete classes: characteristic compressive strength (tested 150 mm cube size) of 25 MPa and 30 MPa. The variation in β with time of exposure is plotted in Fig. 8.



(a) Limit state of flexure (b) Limit state of shear

Fig. 9 Effect of number of reinforcement bars for beam



(a) Limit state of flexure (b) Limit state of shear

Fig. 10 Effect of type of aggregate for beam

The design of beam as per IS456:2000 for under-reinforced (tension controlled) section, concrete compressive strength makes little difference in reliability as shown in Fig. 8.

4.4 Influence of distribution of reinforcement bars

Fig. 9(a) and (b) presents the results considering the effect of distribution of reinforcement bars of 2-25 mm ϕ bars and 5-16 mm ϕ bars, while keeping the reinforcement ratio constant. Here, as the number of reinforcement bars increases keeping total reinforcement area same, the reliability index value increases marginally. Beam with reinforcement of smaller diameter causes improvement in moment capacity than that with reinforcement of larger diameter. This will lead to an increase in reliability index.

4.5 Influence of type of aggregate

In concrete construction, the aggregate used may be siliceous or carbonate, based on which the

Table 4 Comparison of reliability in terms of shear and flexure (β values)

Time of exposure (hours)	Reliability index β	
	Shear	Flexure
0	3.31	3.51
0.5	3.31	3.51
1	3.28	3.51
1.5	3.03	3.39
2	2.41	2.91
2.5	1.79	1.92
3	0.85	0.03

failure probability may be changing. The effect of siliceous and carbonate aggregate types is shown in Fig. 10(a) and (b). The siliceous aggregate has high thermal conductivity compared to carbonate aggregate resulting in a high rate of temperature rise inside the concrete and in turn in reinforcement. The effect of aggregate type in β value is not much significant during the initial time of fire exposure and is modest for fire exposure greater than 2 hours. During initial time of exposure the rate of temperature increase is similar in both cases. As time passes the rate of increase in temperature is more for siliceous aggregate which causes a decrease in reliability indices.

5. Comparison of reliability degradation with respect to flexure and shear

The reliability with respect to flexure and shear are compared in this section. For the same set of parameters, beam with effective cover of 37.5 mm, the results are compared and tabulated in Table 4.

It is observed that, the reliability with respect to shear is less than that of flexure in the initial stages of fire (up to about 2.5 hour) and after that the condition reverses. This implies that at the initial stages of fire, for the beam considered, limit state of shear is more critical and after about 2.5 hour limit state of flexure become critical.

7. Conclusions

The probability of failure increases gradually initially and then at a faster rate as time of exposure increases. This implies that for a particular type of structure, Indian standard IS456:2000 satisfies its objective of having a uniform reliability for a given failure mode with a given set of variables, for initial time of exposures. The reliability index values as per IS code are highly fluctuating according to Ranganathan (1990). Hence from Figs. 5-10, it can be noted that the initial (cold-strength) reliability levels are slightly varying based on different failure modes depending on the load and resistance models while exposed to dead load and live load. The reliability indices are rapidly decreased as a function of time during fire exposure. The curve is gradual in the case of shear mode of failure, while it is abrupt in flexure mode of failure.

From the parametric studies conducted, it can be concluded that the reliability indices are most sensitive to effective concrete cover (flexure and shear failure modes). Concrete cover is recognized as an important parameter for fire endurance of RC beams subjected to fire and is notable in reliability indices. Reliability values increase by an average of 25% for 25 mm increase in cover. Effect of yield strength of steel is more in case of shear failure mode than flexural failure mode and grade of concrete has little effect in reliability. Distribution of reinforcement bars has a minor effect on reliability in both failure modes. Concrete manufactured using carbonate aggregate is having more reliability by 10% compared to siliceous aggregate.

From the present study, it is possible to fix the time of exposure such that the reliability index falls below a user defined target value depending upon the importance of structure, such that within that time the building can be evacuated.

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