POST FIRE RESIDUAL CAPACITY OF REINFORCED CONCRETE BEAM

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Abstract

Incidents of fire are in increasing trend in developing countries. Exposure to fire may lead to a decrease in load carrying capacity and stiffness of a reinforced concrete (RC) structure. The degree of damage is related to the fire intensity in terms of temperature and duration of fire. Assessment of the residual capacity of a fire damaged RC structure is difficult since there is no direct method to identify the extent of damage in the structural member. Sustained high temperature on a RC beam surface leads to progressive heating of the inner layers of concrete. Consequently, reinforcement bars get exposed to excessive high temperature which reduces the strength of rebars. In this article, an attempt was made to determine the post fire residual capacity of a RC beam subjected to fire using finite element (FE) model. For this, FE models of 4 (four) RC beams with different clear cover thickness and depth subjected to fire were developed. After model validation these were used to determine the residual capacity of RC beams exposed to fire for a duration of 30 min to 4 h. Results show that exposure to fire reduce the residual capcity and stiffness of a RC beam significanty. The nature of failur of a RC beam may become brittle after exposure to fire for longer period. Effect of increased clear cover on the residual capacity of the RC beam was found insignificent. Comparing the post fire residual capacity of RC beams obtained from FE analysis with the residual capacity calculated by simplified method showed that simplified method may overestimate the residual capacity the RC beam exposed in fire for long duration.

Keywords

RC Beam; post fire; residual capacity

1. Introduction

Over the last few years, the fire hazard is a common scenario in developing countries due to lake of proper planning. In these regions, reinforced concrete (RC) is the main construction material due to its availability and low-price. Low thermal conductivity coupled with high heat capacity give RC structure good fire resisting property[1, 2]. However, RC members loss their strength and stiffness under fire exposure condition comparing to that of ambient conditions [3]. This degradation of strength and stiffness of RC members under fire exposure condition depends on the temperature and duration of the fire exposure and material properties of concrete and reinforcing steel at elevated temperature.

The assessment of the post fire residual capacity of the structure is critical for the rehabilitation of a fire affected structure. Previous studies showed that there are different approaches to evaluate the residual capacity of RC structures. Some of the individuals used laboratory environment to examine the residual capacity [1, 4–7]. They noticed that duration of fire and rebar peak temperature had significant effect on the residual capacity of RC members. Some other researchers [8–10] used simplified cross-sectional or finite element analysis method to determine the residual capacity of RC beams. Hsu and Lin [8] divided critical section into number of strips and calculate the temperature of each strip, then used strength-temperature relation of concrete and steel reinforcement to evaluate the residual capacity which was very cumbersome. In the other hand, Kodur et al. [10] used residual strength of rebar at the peak temperature and design equations of ambient temperature [11] to computed the post fire residual capacity of RC beams. In this method, reduction of concrete strength due to the exposure to fire was ignored. Ožbolt et al. [12] proposed detailed transient 3D thermo structural FE analysis to evaluate post fire capacity of RC beams.

The aim of this study was to examine the post fire residual capacity of the RC beam using FE model. For this purpose, FE model of RC beams with different span lengths and cross sections were developed to determine their residual capacities when exposed to fire for 30 min to 240 min. In addition, the clear cover of the RC beams was varied to observe the effect of clear cover on the residual capacity. Finally, the residual capacity determined by FE analysis was compared with the computed residual capacity using simplified method.

2. Finite Element Model

The post fire capacity of RC beam was determined by using FE Software package ABAQUS [13]. The full analysis was conducted in two steps: thermal analysis and structural analysis. At first, concrete beam with steel reinforcement was modeled where reinforcements were tied with concrete for proper transfer of heat and force. In thermal analysis, ASTM E119 [14] fire curve was applied as a fire temperature on the bottom and sides of the RC beam. Top of the beam and top 150 mm of both sides of the RC beam were considered unexposed to fire due to the presence of slab. The convective heat transfer coefficients were taken as 25 and 9 W/(m^2.K) for fire exposed and unexposed surfaces of RC beam, respectively in accordance with Eurocode 2 [3]. While considering radiative heat transfer, the emissivity constant of 0.8 was considered for fire exposed concrete surface [3]. In the FE thermal analysis, eight node continuum element DC3D8 and two node link element DC1D2 were used for concrete and steel reinforcement, respectively [13].

In the second step, temperature of different elements from thermal analysis was assigned as the initial condition for the structural analysis from FE model using predefined field. In the FE model for structural analysis, the mesh distribution remained same as the thermal model. However, DC3D8 and DC1D2 elements were replaced with C3D8R element for concrete and T3D2 element for reinforcement, respectively [13]. To achieve sufficiently accurate result within optimal computation time, a tolerance limit of 0.02 was used in Newton-Raphson solution technique and line search control was activated for rapid convergence [2, 15-19].

3. Material Property for FEA

In this study, the compressive strength of concrete ($f_c^{'}$) at room temperature was consider 35 MPa and the tensile strength was taken as 10% of $f_c^{'}$ [20]. The yield strength of the steel reinforcement f_y at room temperature was 420 MPa. Previous studies reported that the compressive and tensile strength of concrete decrease at elevated temperature due to fire exposure and these strengths further decrease after a cool down period, commonly known as residual strength [2, 21]. Fig. 1a shows the variation of compressive strength of concrete at elevated temperature according to Eurocode 2 [3] and residual strength of concrete according to [2, 21]. The residual strength of concrete thus found was used in the structural analysis model to determine the residual capacity of RC beam after fire exposure. Fig. 1b shows that at 600°C the tensile strength capacity of concrete becomes zero as per Eurocode 2 and an increase of this temperature makes convergency problem in FE model. To solve this problem, Dwaikat and Kodur [22] modified the tensile strength capacity of concrete above 600℃ temperature. The modified tensile strength relationship of concrete with temperature was used in this study.

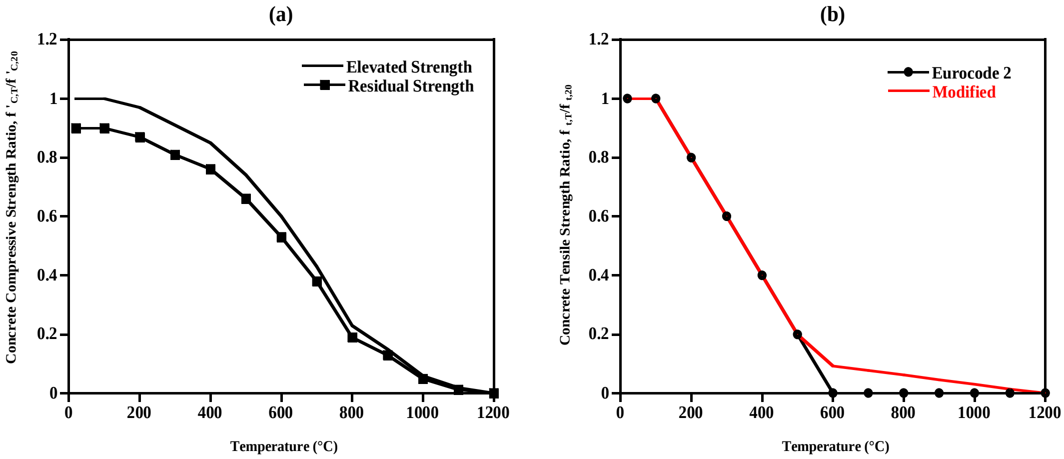

Fig. 1: Variation of Concrete (a) Compressive and (b) Tensile Strength Ratio with temperature.

The stress strain relationship of concrete generally divides into three parts linear elastic, hardening and softening. The first part is linear elastic up to 30 to 50% of compressive strength of concrete [2, 20, 23]. So, in this study, linear elastic part was assumed 33% of f_c'. The hardening portion of this curve tends to parabolic [3, 20, 21, 23] and softening part may be linearly descending [3, 24, 25] or parabolic [24, 26]. For both hardening and softening parts, parabolic stress strain relationship was considered. Moreover, the dilation angle of concrete was taken as 35° [27, 28]. The yield strength of reinforcing steel at elevated temperature and their residual yield strength after cooldown is presented in Fig. 2 [2]. The temperature dependent properties, e.g. conductivity, specific heat and density of concrete and steel were used according to Eurocode 2 for this analysis [3].

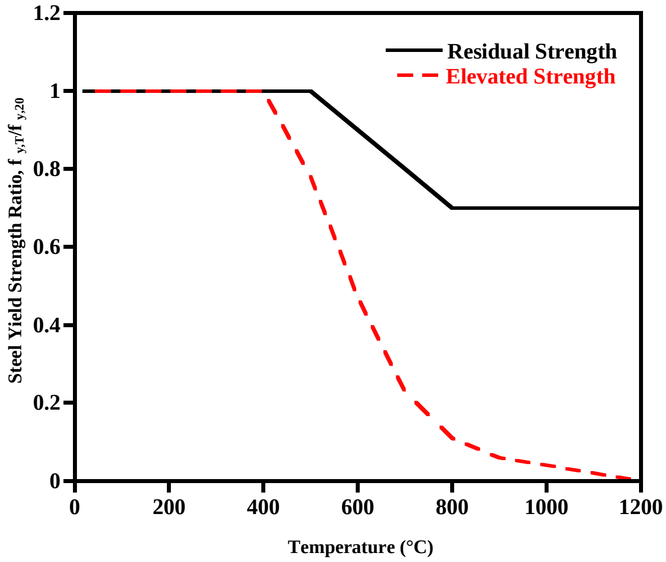

Fig. 2: Variation of Yield Strength of steel with temperature

4. Validation of the FE Model

The FE model was validated against the test performed by Dwaikat and Kodur [1]. They tested a 3960 mm long simply supported beam with 406×254 mm rectangular cross section. Middle 2440 mm of the beam was exposed to ASTM E119 fire [14]. The beam had 3ϕ19 mm tension bars at the bottom and 2ϕ13 mm compression bars at the top. As shear reinforcement, ϕ6mm was provided with a spacing of 150 mm over the beam length. The compressive strength on test day and 28-day tensile strength were measured as 58.2±3.1 MPa and 3.7±0.5 MPa, respectively. The yield strength of longitudinal bars and transverse bars were 420 MPa and 280 MPa, respectively.

Authors applied 50 KN load at 610 mm apart from center of the beam for 30 min before the beam was exposed to fire. In the laboratory, structural fire test chamber was designed specially so that load could be maintained constantly throughout the duration of the fire. Following the procedure discussed in the previous section, the beam was modeled and the results obtained from FE analysis were compared with test results. Fig. 3(a) shows the near overlapping experimental and modeling results in case of temperature variation with respect to time at three specific points (i.e. at mid depth, at quarter depth and at bottom rebar) of the beam mid-section. Both experimental and modeling results show that the mid span deflection increased with time until the beam failed after 180 min of fire exposure due to diminished strength and stiffness (Fig. 3b).

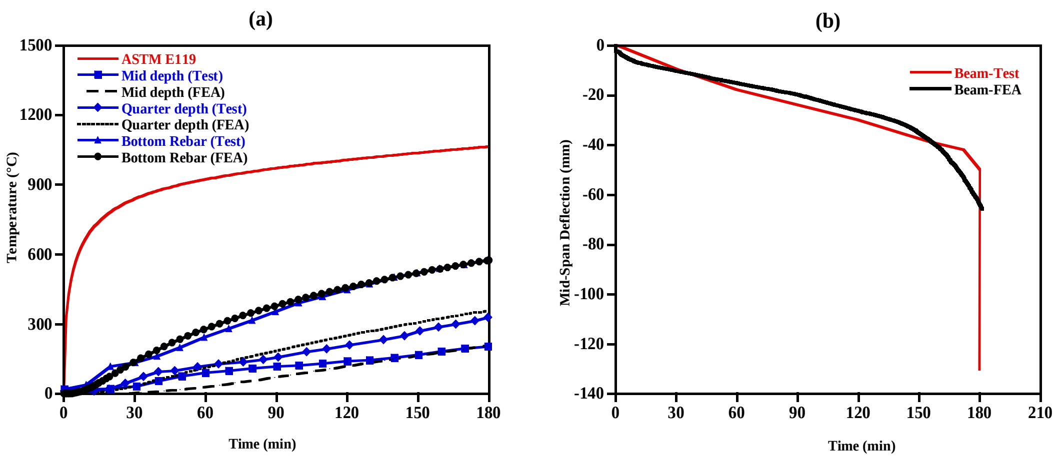

Fig. 3: Comparison of the RC beam tested by Dawaikat [1]: (a) test and FE computed temperature at various location (b) test and FE computed mid-span deflection of simply supported beam.

5. Parametric Study

The validated FE approach was used to determine the residual capacity of 4(four) RC beams after exposure to fire. The length (L) and support to support distance (L_s) of B1 and B2 beams were 6000 mm and 5600 mm, respectively. The cross section and reinforcement of beam B1 and B2 were same as shown in Fig. 4, albeit clear cover of B1 and B2 were 38 mm and 50 mm, respectively. For B3 and B4 beams pair, L and L_s were 4500 mm and 4200 mm, respectively. The cross section and reinforcement of B3 and B4 were same, albeit clear cover of B3 and B4 were 38 mm and 50 mm, respectively.

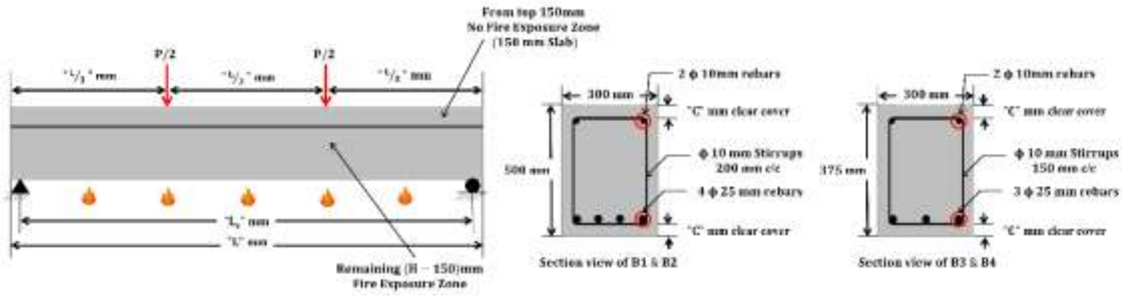

Fig. 4: Beam sectional profile detail of typical RC beams selected for parametric study

The RC beams were designed as per ACI Code 318-08 [11] for 4 kip/ft uniformly distributed load considering f_c' as 35 MPa and f_y as 420 MPa as discussed in Section 3. Fire was applied on three sides of the beam as depicted in Fig. 4. The capacity of all four RC beams were determined in ambient conditions and after 30, 60, 120, 180 and 240 min exposure to standard ASTM E119 fire [14]. A total of 24 models were thus analyzed for different fire scenarios.

6. Results and Discussions

The post fire residual capacity and stiffness of all 4 beams (i.e., B1, B2, B3 and B4) decrease with longer fire exposure time. These contribute to the higher deflections at ultimate loads for longer period (i.e., 240 min) and shown in Fig 5 and 6 for beams B1 and B2, and B3 and B4, respectively. Beam B1 and B2 had the same cross section and span length with different clear cover. However, up to 120 minutes of fire exposure, the residual capacity of B1 and B2 did not differ significantly. After that, the residual capacities of B2 were slightly higher than B1. Both B1 and B2 beams showed ductile post failure behavior up to 120 minutes of fire exposure. After 180 min and 240 min of exposure, failure of both B1 and B2 beams were brittle in nature as failure occurred due to the crushing of concrete. From Fig. 6, similar behavior was found for B3 and B4 beams. For B3 and B4 beams, brittle failure was observed after 240 minutes of exposure.

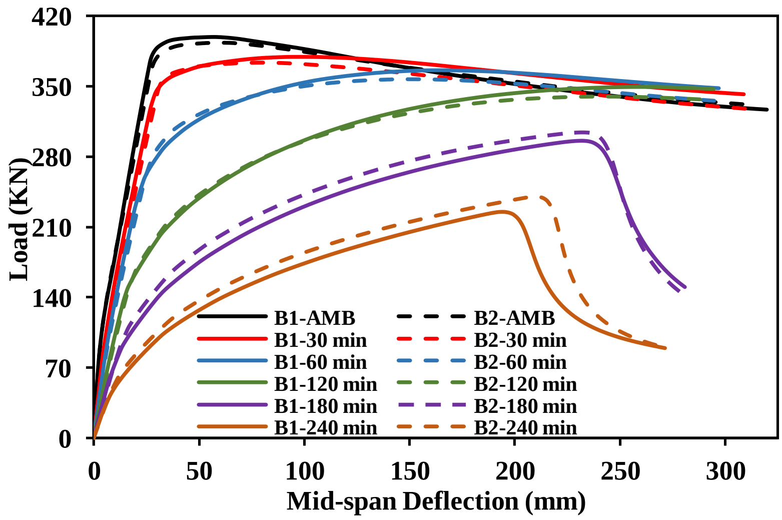

Fig. 5: Post fire load-displacement behavior of B1 and B2 after different fire exposure time

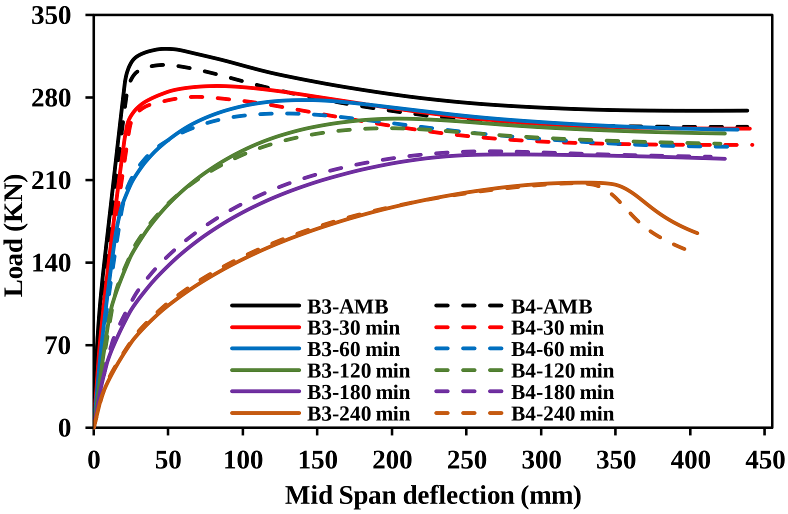

Fig. 6: Post fire load-displacement behavior of B3 and B4 after different fire exposure time

In Fig. 7, the ratio of post fire residual moment capacity to the ambient moment capacity of all the four RC beams were shown for different exposure times. The residual moment capacity of B1 and B2 beam pair decreases slower than B3 and B4 beam pair up to 120 min of fire exposure which was reversed after 120 min of fire exposure. In addition, it is observed that up to 120 min of fire exposure, clear cover of the beam does not play any important role in the residual moment capacity. But for exposure longer than 120 min, the residual moment capacity of the beams with 50 mm clear cover (i.e., B2 and B4 beams) were 3 to 5% higher than the residual capacity of the beams with 38 mm clear cover (i.e., B1 and B3 beams).

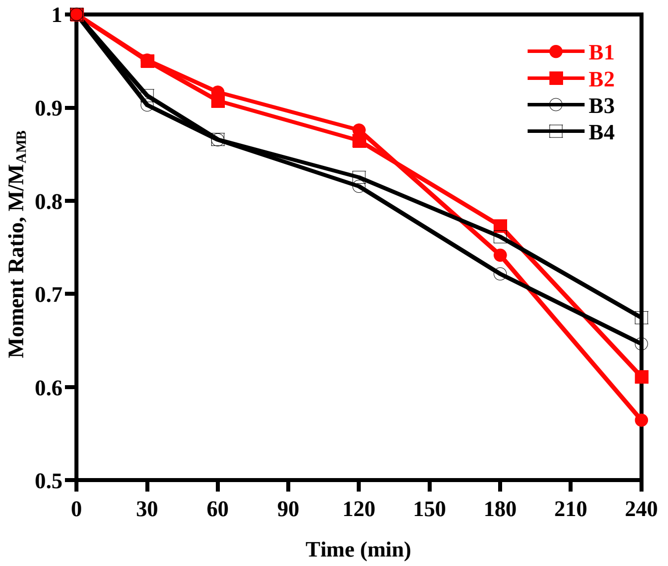

Fig. 7: Variation of the ratio of post fire residual moment capacity to the ambient moment capacity with fire exposure time

The post fire residual capacity of the studied RC beams with their peak temperature at bottom reinforcement after different fire exposure times are presented in Table 1. The residual capacities determined by FE analysis were also compared with the residual moment capacity computed using simplified method proposed by Kodur et al. [10]. In the simplified method, the post fire residual moment capacity depends on the residual strength of steel reinforcement. As shown in Fig. 2, residual yield stress of reinforcement is equal to the yield stress at ambient temperature up to 500°C. This explains the fact that when peak rebar temperature is less than 500°C, RC beam’s residual capacity remain same as ambient condition. From Table 1, it is observed that for B3 and B4 beams, simplified method conservatively predicts the post fire residual capacity. However, for B1 and B2 beams, simplified method seems nonconservative when beams exposed in fire for longer period. In FE analysis, both residual strength of concrete and steel were considered for determining the residual capacity. On the other hand, in simplified method, only residual strength of steel was used assuming that failure would be initiated by the yielding of reinforcement. If a beam fails due to the crushing of concrete, then the simplified method overestimates the residual capacity of that beam. For example, when B1 and B2 beams were exposed for 180 min and 240 min in fire, they showed brittle failure due to crushing of concrete. This contributes to overestimation of residual moment, i.e., 303.4 kN against 295.7 kN for B1 beam under 180 min exposure time.

Table 1. The post fire residual moment capacity of RC beam for different exposure time

Specimen ID	Rebar Temperature (℃)	Residual Capacity (kN)	
		FEA	Simplified Approach [10]
B1-AMB	20	398.8	363.0
B1-30 min	163	379.3	363.0
B1-60 min	336	365.6	363.0
B1-120 min	551	349.3	346.5
B1-180 min	681	295.7	303.4
B1-240 min	740	225.2	283.4
B2-AMB	20	393.1	352.1
B2-30 min	128	373.6	352.1
B2-60 min	281	356.7	352.1
B2-120 min	486	339.9	352.1
B2-180 min	621	303.9	313.9
B2-240 min	713	240.3	284.0
B3-AMB	20	321.2	274.2
B3-30 min	188	290.0	274.2
B3-60 min	370	277.9	274.2
B3-120 min	589	261.9	252.5
B4-120 min	520	253.7	258.2
B3-180 min	713	231.8	221.5
B3-240 min	764	207.7	208.4
B4-AMB	20	307.5	262.8
B4-30 min	147	280.7	262.8
B4-60 min	309	266.3	262.8
B4-180 min	654	234.2	226.8
B4-240 min	732	207.5	207.9

7. Conclusion

In this present study, a parametric study was performed to compute the post fire residual capacity of RC beam using FE analysis. Before parametric studying, FE model was verified against experimental results. A total of 24 models were analyzed for two different span length and four beam sections with 38 mm and 50 mm clear covers. From the analysis, it was observed that with the increase of exposure time, the post fire residual capacity decreased. After 120 min of exposure, the residual moment capacity became 80% of its ambient capacity. After that point, residual capacity reduced more quickly. After 240 min of exposure, the residual capacity became around 55% of its ambient capacity. It was observed that, after longer period of fire exposure, failure of RC beam may become brittle in nature. In this study, it was found that clear cover of RC beams has no significant effect on the post residual capacity of the RC beams. The FE model results were also compared with the calculated residual capacity using simplified method. Comparison showed that, simplified method can be nonconservative when the RC beam failed due to the crushing of concrete.

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