FIRE RESISTANCE OF SIMPLY SUPPORTED REINFORCED CONCRETE BEAMS WITH TENSION LAP SPLICES

FEUERWIDERSTAND VON STAHLBETONBALKEN MIT ÜBERGREIFUNGSSTÖßEN

Hitesh Lakhani[1], Jan Hofmann[1], Andreas Grübel[2]

[1] *Institute of Construction Materials, University of Stuttgart*

[2] *Faculty 2: Civil and Environmental Engineering, University of Stuttgart*

SUMMARY

The extensive numerical and experimental studies, on Reinforced Concrete (RC) beams, available in literature are limited to beams with continuous tension reinforcement. Hence, to investigate and understand the behaviour of RC beams with tension lap splices (designed as per EN1992-1-1) under fire, the presented numerical study was conducted. As in case of lap splices the bond between the reinforcement and concrete might play an important role, temperature depended bond strength degradation has also been considered. The geometry (dimensions and cover to reinforcement) of the beam, selected for the numerical study, qualifies for a fire rating of 90 minutes (for beam without splice) as per EN1992-1-2. The numerical results indicated a slightly increased fire resistance for RC beams with splices as compared to beams without splices.

ZUSAMMENFASSUNG

Die in der Literatur verfügbaren numerischen und experimentellen Studien zu Stahlbetonbalken beschränken sich auf Balken ohne Übergreifungsstöße. Um das Verhalten von Stahlbetonbalken mit Übergreifungsstößen (nach EN 1992-1-1) unter Feuer zu untersuchen und zu verstehen, wurde die vorgestellte numerische Studie durchgeführt. Da bei Übergreifungsstößen der Verbund zwischen der Bewehrung und dem Beton eine wichtige Rolle spielen könnte, wurde auch die temperaturabhängige Verschlechterung der Verbundfestigkeit berücksichtigt. Die für die numerische Studie ausgewählte Geometrie des Balkens qualifiziert sich für eine Feuerwiderstandsdauer von 90 Minuten (für Balken ohne Übergreifungsstöße) gemäß EN1992-1-2. Die numerischen Ergebnisse zeigten einen erhöhten

Feuerwiderstand für Stahlbetonbalken mit Übergreifungsstößen im Vergleich zu Balken ohne Übergreifungsstöße.

1. INTRODUCTION

Behaviour of Reinforced Concrete (RC) beams under fire have been extensively studied, experimentally and numerically, over the last decades by various researchers [1–5]. The current design requirements in various design standards [6, 7], for minimum cover to reinforcement & minimum member dimensions to achieve the required fire resistance, are based on studies mostly on simply supported beams with continuous tension reinforcement. The applicability of these design requirements to RC beams with splices needs to be ascertained. Moreover, since the bond behaviour between rebar and concrete changes during fire [8], this factor might also affect the behaviour of RC beams with tension splices exposed to fire. Although the effect of bond between reinforcement (ribbed bars) & concrete on the fire resistance (failure time under limit state of load carrying capacity) was found to be limited by Gao et al. (2013) [9] and Kodur & Agrawal (2017) [10]. But once again these studies were limited to beams without tension splices.

The paper presents the first set of results to answer the questions raised above. The study also provides a direct comparison between the fire resistance of RC beam without and with splice designed as per EN1992-1-1 [11].

2. NUMERICAL MODEL

The 3D sequentially coupled thermo-mechanical model used for the numerical investigation is briefly described in the following sections. The numerical model has previously been used for simulating the behaviour of RC columns during fire exposure [12–14].

2.1 TRANSIENT HEAT TRANSFER ANALYSIS

In the first step transient heat transfer analysis is conducted i.e., for a given thermal boundary conditions at time *t*, temperature distribution over a solid volume Ω is calculated. In each point of continuum, which is defined by the Cartesian coordinates (x, y, z), the conservation of energy, defined by Eq. (1), is fulfilled.

$$\lambda \Delta T(x,y,z,t) - c\rho \frac{\partial T}{\partial t}(x,y,z,t) = 0 \tag{1}$$

Where, T = temperature, λ = conductivity, c = heat capacity, ρ = density and Δ = Laplace-Operator. The surface boundary condition that must be satisfied is given by Eq. (2):

$$\lambda \frac{\partial T}{\partial n} = \alpha (T_M - T) \tag{2}$$

Where, n = normal to the boundary surface Γ, α = equivalent heat transfer coefficient (given by Eq. (3)) and T_M = temperature of the media (hot gases/air) in which surface Γ of the solid Ω is exposed. The above-mentioned differential equations are solved using finite element method with

$$\alpha = h_c + \varepsilon\sigma\left[(T_M)_t^2 + (T)_{t-1}^2\right] \times \left[(T_M)_t + (T)_{t-1}\right] \tag{3}$$

The equivalent heat transfer coefficient given by Eq. (3) is used to account for the convective and radiative heat transfer from the hot gases to the member surface, where h_c is the convective heat transfer coefficient (25 W/m^2 K), ε is the surface emissivity (0.8) and σ is Stefan-Boltzmann constant (5.67×10^{-8} W/m^2 K^4).

The thermal properties of concrete are taken as the lower bound thermal conductivity and specific heat for dry concrete, as per EN1992-1-2 [6]. The selection of these thermal properties is based on the sensitivity/validation studies conducted by Lakhani et al. (2013) [15]. The thermal properties for steel were taken as per EN1993-1-2 [16].

2.2 MATERIAL CONSTITUTIVE LAWS

The total strain tensor for concrete, given by Eq. (4), has three components: mechanical strain, free thermal strain and load induced thermal strain. The mechanical strain component is further composed of elastic, plastic, and damage part. Concrete was modelled using isothermal microplane model proposed by Ožbolt et al. (2001, 2005) [17,18].

$$\varepsilon_{ij} = \varepsilon_{ij}^{m}(T,\sigma) + \varepsilon_{ij}^{ft}(T) + \varepsilon_{ij}^{lits}(T,\sigma) \tag{4}$$

Where, ε_{ij}^{m} = mechanical strain tensor; ε_{ij}^{ft} = free thermal strain tensor and ε_{ij}^{lits} = load-induced thermal strain tensor.

The temperature dependencies of various mechanical properties of concrete are shown in Fig 1.

Steel is modelled using classical plasticity model (von Mises). Fig. 2 shows the variation of Young's modulus and yield stress with temperature, used in the model.

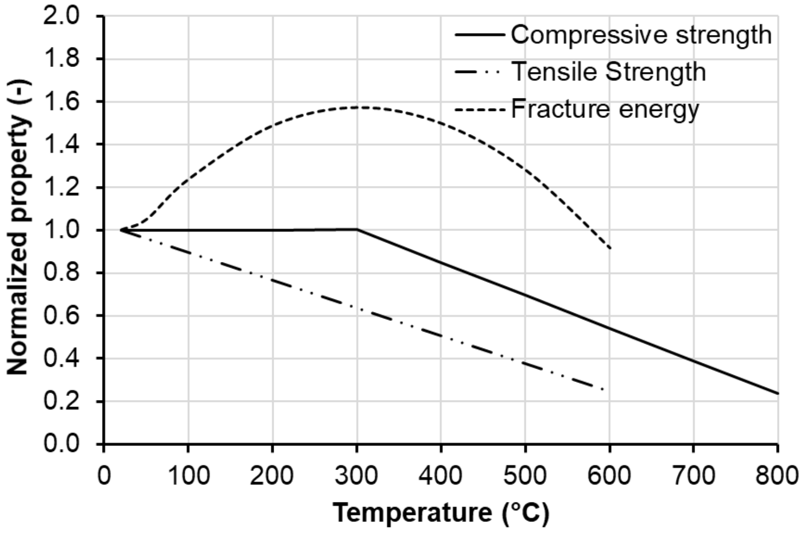


Fig. 1: Degradation of mechanical properties of concrete with temperature

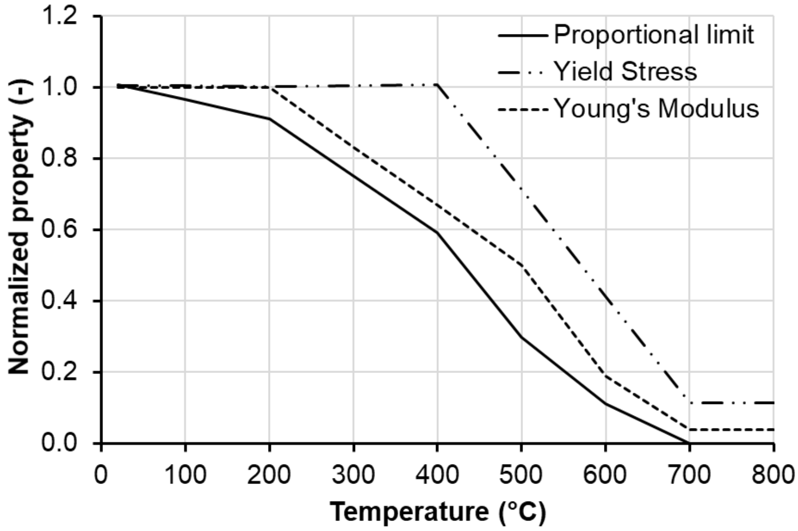


Fig. 2: Degradation of mechanical properties of reinforcing steel with temperature

2.3 BOND BEHAVIOUR AT ELEVATED TEMPERATURE

Lakhani and Hofmann (2018) [8] did an extensive literature review of existing experimental studied on bond between ribbed bars and concrete at elevated temperature. They concluded from the limited experimental data available at elevated temperature that the degradation of the pull-out capacity with temperature is similar to that of the compressive strength of concrete. Since a 3D model is used for the present study the, the splitting would be automatically accounted. Fig. 3 shows the used bond degradation with temperature.

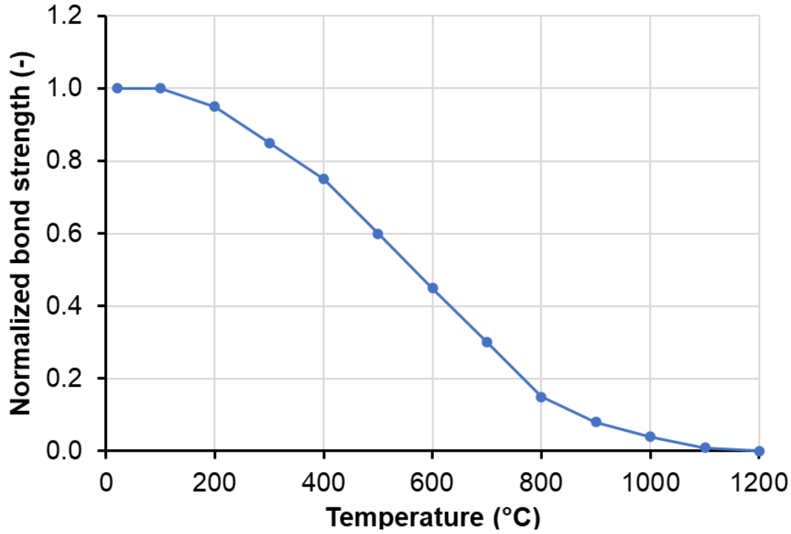


Fig. 3: Bond variation with temperature

3. BEAM DETAILS AND FE DISCRETIZATION

The beam had a cross-section of 300 mm × 380 mm, with 2 × 20 mm diameter rebars in tension zone. The beam had a total length of 3900 mm and a span of 3825 mm between the supports. The rebar had an axis distance (from surface to centroidal axis) of 50 mm from the bottom and side face of the beam. The beam was loaded under 4-point bending, with the tension lap splice in the constant moment zone (as shown in Fig. 4). The shear reinforcement of the beam consisted of 8mm diameter stirrups provided at a spacing of 150mm. The beam was exposed to standard fire as per ISO 834 [19] from 3 sides, the top face of the beam was assumed to be insulated. The concrete had a compressive strength of 48 MPa. The Young's modulus, tensile strength & fracture energy of concrete at ambient temperature were taken as 33,000 N/mm^2; 3.40 N/mm^2 & 0.08 N/mm, respectively. The reinforcing steel had a yield strength of 470 MPa and an ultimate strength of 555 MPa.

Only half of the beam along the width was modelled using symmetry boundary conditions. The concrete was discretized using 4-noded tetrahedron elements and the reinforcement with 8-noded solid elements. Linear steel plates were modelled at the supports and the loading points to avoid numerical divergence. The bond between concrete and rebar was modelled using unit length bond elements (shown in Fig. 5), whose shear behaviour is defined by temperature dependent bond-slip law (discussed in Section 2.3). The bond strength at ambient temperature was taken as 12 N/mm^2. Fig. 6, shows the bond slip curve at 20°C, 300°C and 600°C.

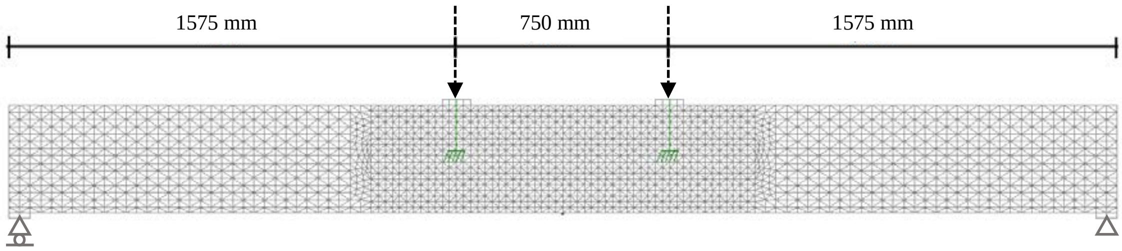


Fig. 4: Discretization of the beam and loading position

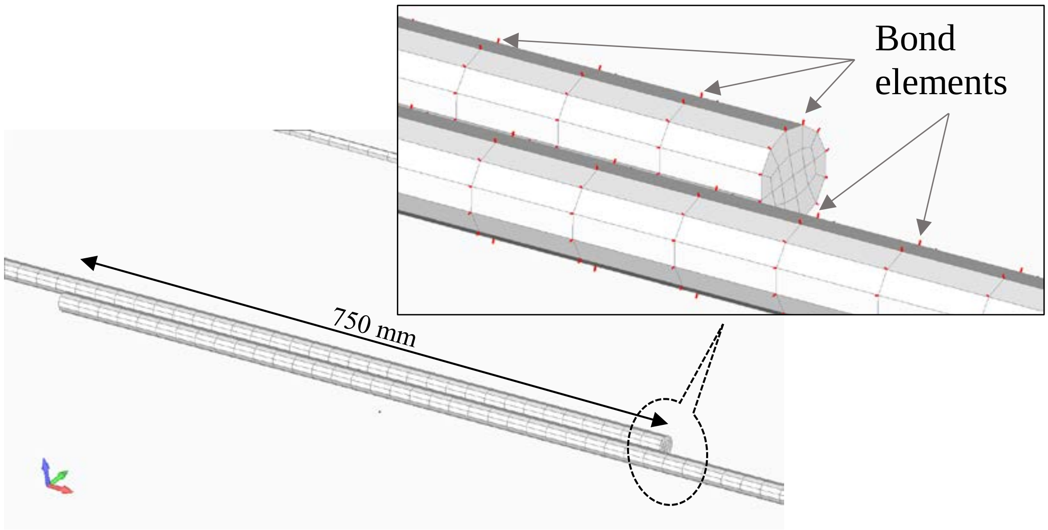


Fig. 5: Discretization of reinforcement and the bond elements

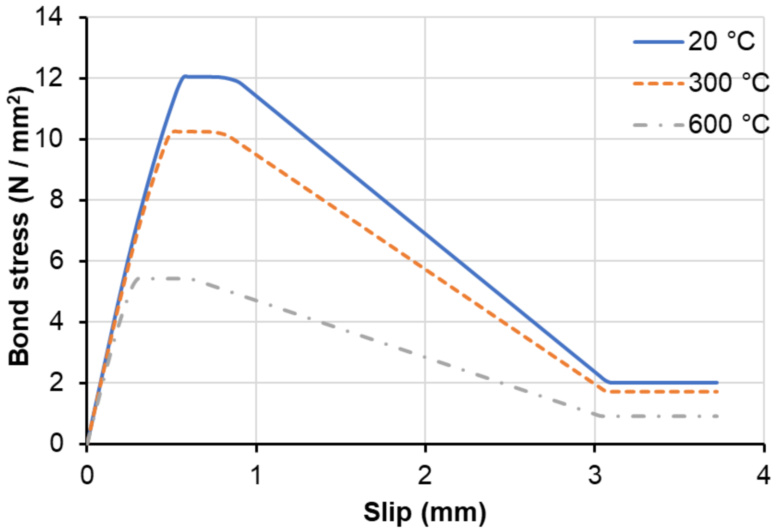


Fig. 6: Bond slip curve at various temperature

4. RESULTS

Before analysing the behaviour of the beams under fire, the reference cases without fire were analysed for both the beams. The failure mode for both the beams was same i.e., yielding of tension reinforcement. Moreover, due to the same loading positions for both the beams, the hinges were also formed at same location i.e., below the point of loading. Since at the yielding sections the amount of reinforcement is same the beams had same failure load. The load-displacement diagrams for the reference beams are shown in Fig. 7. The response of beam with lap splice is stiffer than the beam without lap splice, due to increased tension reinforcement in the constant moment zone (lapping doubled the amount of reinforcement). Fig. 8 (a) & (b) shows the deformations of the beams at maximum load. Lesser flexural cracks developed for the beam with splices as compared to beam without splice where cracks are uniformly distributed between the loading points.

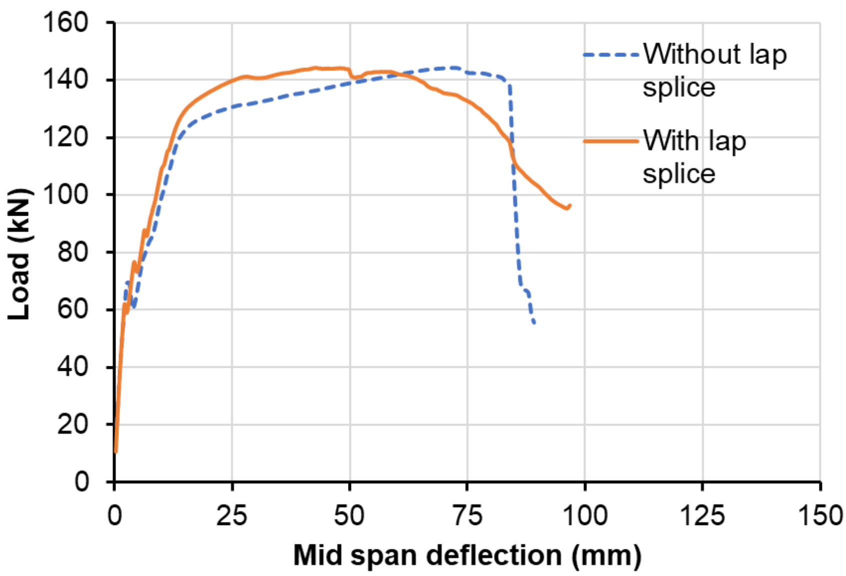
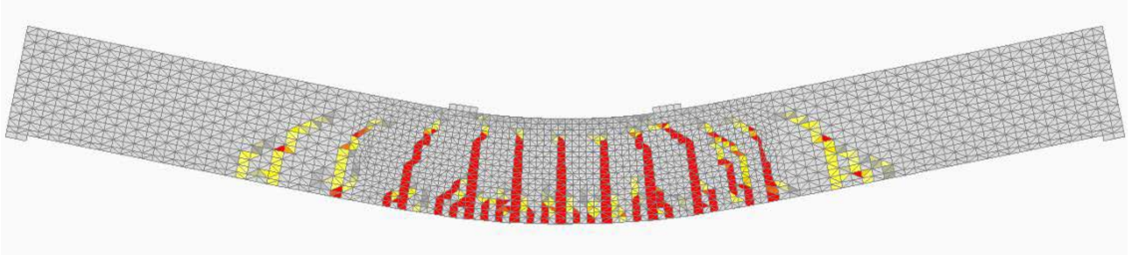
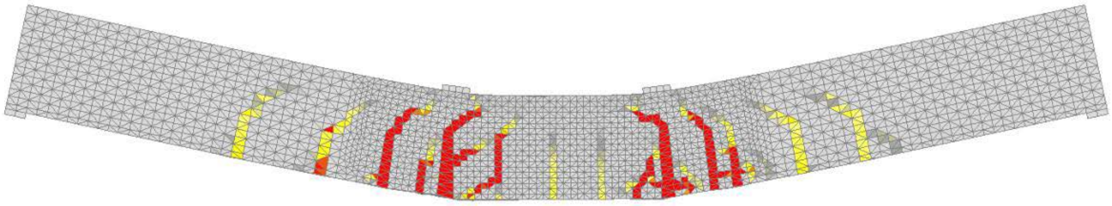


Fig. 7: Load displacement characteristics of the beams



(a) Beam without lap splice



(b) Beam lap splice

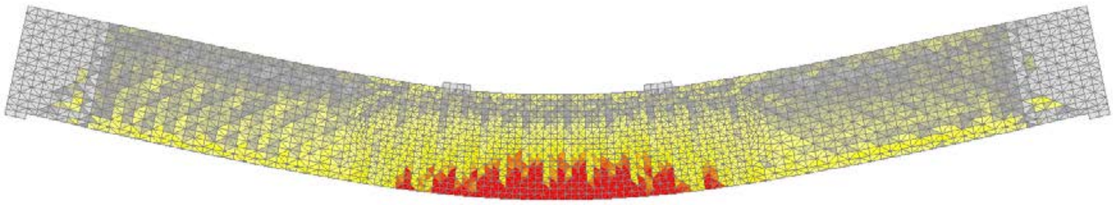
Fig. 8: Reduction in load carrying capacity with exposure time

To compare the fire resistance of the beams without and with tension lap splice. Each beam was loaded with three different load levels and the numerically obtained failure time corresponding to these load levels was noted. The load levels correspond to 96% (LL1), 68% (LL2) and 42% (LL3) of the load carrying capacity at ambient temperature. Hence, total 6 simulations with fire exposure were conducted and in all the cases the beams (with/without lap splice) failed due to yielding of tension reinforcement.

Fig. 9 (a) & (b) shows the deformed profile of beam without lap splice with load level LL2 at failure. Because of the uniformly distributed damage on the exposed surface due to high temperature, no distinct cracks were visible. But on the longitudinal section, large number of uniformly distributed and interconnected flexural & shear cracks are visible. The crack profile for the beam with lap splice (refer Fig. 10) and same load level looks similar to those for beam without tension splice but with lesser number of cracks.

The comparison between the fire resistance of the investigated RC beam with and without lap splice is shown in Fig. 11. The failure time was taken as the minimum of the last converged time step or time at which the limiting deflection (110 mm for the beam investigated) was reached. The beams with tension splice had a slightly longer (≈ 15 minutes) fire resistance.

The observed higher fire resistance is due to the lower temperature of the spliced rebar, which was a result of the increased side cover due to splicing. To further understand this point, one must understand that in beams the rebars are normally spliced in horizontal plane to keep the same effective depths (d). This means one of the spliced rebar is moved away from the exposed face thus leads to increased time to reach the same temperature.



(a) Fire exposed face of beam without lap splice at failure (LL2)

(b) Section along the length of the beam without lap splice at failure (LL2)

Fig. 9: Deformed profile with cracks for beam without lap splices (LL2)

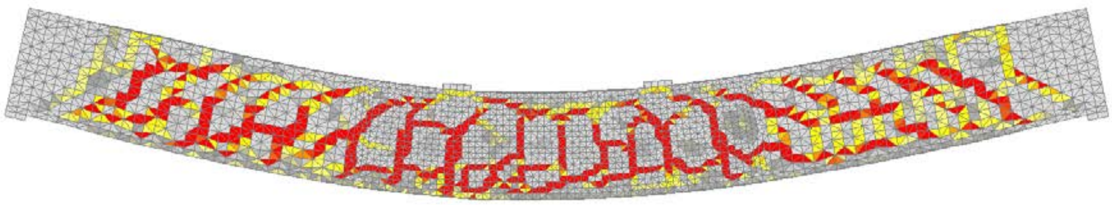


Fig. 10: Deformed profile with cracks for beam with lap splices (LL2)

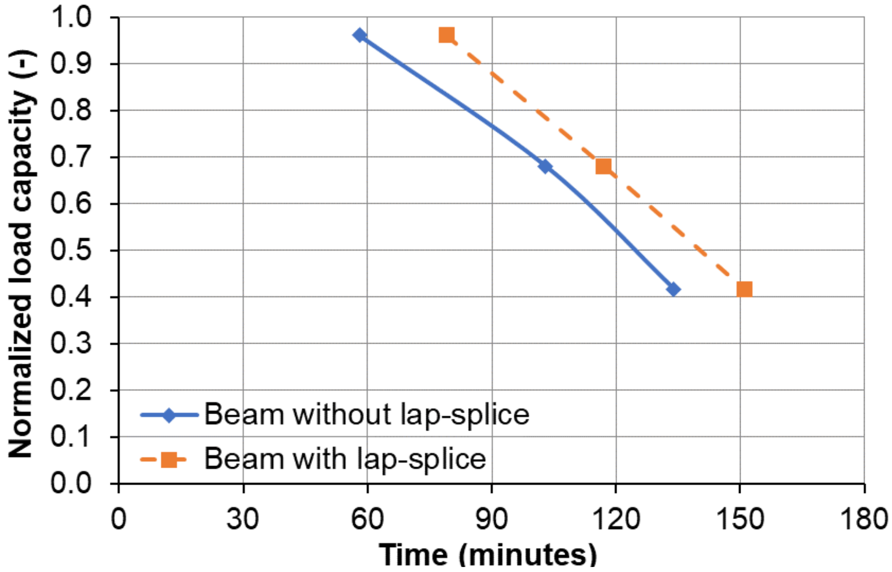


Fig. 11: Reduction in load carry capacity with exposure time

5. CONCLUDING REMARKS

Based on the first set of numerical results, the following conclusions can be drawn:

1. The failure mode during fire for RC beam with splice was same as that for beam without splice, i.e., yielding of tension reinforcement. No failure mode associated with bond was observed.
2. The failure time for RC beam with splice was slightly higher than the beam without splice. This is due to slightly lower temperature of the tension reinforcement which was a result of increased side cover to the spliced rebar.

Although the first set of results presented in this paper, indicate that the available design guidelines are also applicable to beams with tension lap splices. But it should be acknowledged that the clear cover to tension reinforcement was 40mm for the case investigated by the authors. Hence, further investigations are needed to understand the effect of various parameters like cover thickness, rebar diameters, concrete strengths etc.

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