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Fire Behaviour of Reinforced Concrete Beam Under Different Exposure Conditions

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Abstract: This study investigates the fire performance of a 6 m long fixed-ended reinforced concrete beam subjected to standard ISO 834 fire for 30 minutes, considering variable fire exposure conditions. The beam (M30 grade concrete, Fe500 grade steel) was loaded with its full ultimate uniformly distributed load capacity. A coupled temperature-displacement analysis was performed using ABAQUS finite element software. Four exposure cases were examined: 1-face (soffit only), 2-face (soffit and one vertical side), 3-face (soffit and two vertical sides), and 4-face (soffit, two vertical sides and top surface). Results were compared against ambient conditions (20°C) in terms of temperature distribution, mid-span deflection, and residual moment capacity. The analysis revealed that increasing the number of exposed faces significantly accelerates thermal degradation. The 1-face exposure resulted in a bottom rebar temperature of 412°C, deflection of 14.3 mm, and 82% residual moment capacity, remaining within serviceability limits. The 2-face exposure (soffit + one side) caused asymmetric heating with bottom rebar temperature of 468°C, deflection of 18.5 mm, and 74% residual capacity. The 3-face exposure (soffit + two sides) produced a bottom rebar temperature of 534°C, deflection of 22.7 mm (approaching L/250 limit), and 67% residual capacity. The 4-face exposure (soffit + two sides + top) heated both top and bottom reinforcements to 287°C and 621°C, respectively, producing excessive deflection of 36.8 mm (exceeding L/250) and 45% residual capacity. It is concluded that beams exposed to 4 faces under a 30-minute fire experience the highest thermal degradations.

Keywords – reinforced concrete; finite element method; thermal behaviour; standard fire

I. INTRODUCTION

Reinforced concrete (RC) is the most commonly used construction material worldwide due to its durability, formability, and cost-effectiveness. However, under fire conditions, RC structures undergo significant degradation. Concrete loses its compressive strength when heated above 300°C due to the decomposition of cement hydrates, while steel reinforcement begins losing yield strength rapidly above 400°C (El-Hawary et al., 1997). At 600°C, steel retains only 47% of its original strength, and at 700°C, only 23% remains (Eurocode 2, 2004). The severity of fire damage depends on several factors, including fire duration, maximum temperature reached, cooling rate, concrete cover thickness, reinforcement ratio, and crucially, the number of concrete faces exposed to fire (Parthasarathi, 2021).

Several researchers have experimentally and numerically investigated the fire behaviour of RC beams over the past three decades. El-Hawary et al. (1997) studied RC beams exposed to fire at 650°C for durations of 30, 60, and 120 minutes, finding that longer fire exposure significantly reduces ultimate load capacity and that increased concrete cover thickness effectively protects reinforcement from thermal damage. Xu et al. (2012) conducted one of the earliest systematic studies on fire exposure sides, testing simply-supported RC beams under one-face, two-face, and three-face fire exposure, and they presented moment-curvature relationships for different temperature fields while validating their predictions with experimental results. Ali and Nadjai (2011) experimentally and numerically investigated high-strength concrete slabs under ISO 834 fire, developing a validated finite element model capable of predicting fire-induced deflections and spalling behaviour with reasonable accuracy. More recently, Elshorbagi and AlHamaydeh (2026) employed a direct coupled temperature-displacement nonlinear simulation in ABAQUS to model RC beam fire performance, demonstrating that direct coupling—where thermal and mechanical fields are solved simultaneously—is superior to sequential coupling for capturing large deformations, with error rates as low as 3.4% compared to sequential coupling. A 2020 study using finite element analysis examined RC beams under three-sided fire exposure, identifying concrete cover thickness, reinforcement ratio, and fire exposure time as the main influencing factors on residual flexural capacity (Cai etal, 2020). Despite these valuable contributions, a systematic comparative study investigating the effect of all four exposure configurations—one-face, two-face, three-face, and four-face—under identical beam geometry, material grades, loading, and fire duration remains notably lacking in the existing literature.

The scope of this study is limited to a fixed-ended reinforced concrete beam of 6 meters in length, with cross-section dimensions of 300 mm width and 450 mm depth, using M30 grade concrete and Fe500 grade steel with 25 mm clear cover. The beam is subjected to the standard ISO 834 fire curve for a duration of 30 minutes while carrying its full ultimate uniformly distributed load of 71 kN/m, representing worst-case service conditions. Four exposure cases are considered: one-face (soffit only), two-face (soffit plus one

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vertical side), three-face (soffit plus two vertical sides), and four-face (soffit, two vertical sides and top surface). The analysis employs a coupled temperature-displacement approach in ABAQUS using the direct coupling technique, and all results are compared against the ambient condition baseline at 20°C. Spalling is modelled through strength reduction rather than explicit fracture mechanics, and bond-slip between reinforcement and concrete is not considered, simplifications that are consistent with standard practice in fire engineering (Fu, 2016). This study is significant because real building fires expose beams to different numbers of faces depending on their location within the structure, compartment geometry, and fire spread patterns, and understanding how exposure faces affect fire resistance helps engineers design safer structures. By isolating the number of exposed faces as the sole variable, this study provides clear, comparable quantitative data on temperature distribution, mid-span deflection, and residual moment capacity for each exposure configuration. The findings will help determine when fireproofing is required, how much concrete cover is adequate, and what load reduction may be necessary for beams in high-risk exposure conditions.

II. METHODOLOGY

2.1 Material Properties

The beam is constructed using M30 grade concrete, which has a characteristic compressive strength of 30 MPa at ambient temperature (20°C). The modulus of elasticity for M30 concrete is taken as 27,386 MPa as per IS 456:2000. The reinforcement consists of Fe500 grade steel with a characteristic yield strength of 500 MPa at ambient temperature, having an elastic modulus of 200,000 MPa. For elevated temperature analysis, temperature-dependent reduction factors are applied in accordance with Eurocode 2 (EN 1992-1-2). The concrete compressive strength reduction factor $k_c(\theta)$ is defined as 1.0 at 20°C, 0.95 at 200°C, 0.75 at 400°C, 0.40 at 600°C, and 0.15 at 800°C, with linear interpolation between these points. The steel yield strength reduction factor $k_s(\theta)$ is taken as 1.0 from 20°C to 400°C, 0.78 at 500°C, 0.47 at 600°C, 0.23 at 700°C, and 0.11 at 800°C. Thermal properties and their temperature-dependent variation, including thermal conductivity and specific heat, are taken as per Eurocode 2, whereas density (2400 kg/m³ for concrete, 7850 kg/m³ for steel) is assigned as a constant.

2.2 Beam Geometry and Support Conditions

The beam under investigation has a total length of 6000 mm with a rectangular cross-section of 300 mm width and 450 mm depth. The clear concrete cover is maintained at 25 mm throughout the section. The longitudinal reinforcement consists of three 16 mm diameter bars at the bottom (A_{st} = 603 mm²) and two 12 mm diameter bars at the top. Transverse reinforcement comprises 8 mm diameter stirrups placed at 150 mm centre-to-centre spacing along the entire beam length. The beam is modelled with fixed-end support conditions at both ends, meaning that all translations (U1, U2, U3) and all rotations (UR1, UR2, UR3) are restrained at x=0 and x=6000 mm. This fixed-fixed configuration represents a beam monolithically cast into supporting columns or walls, which is common in real building construction.

2.3 Applied Loading

The beam is loaded with a uniformly distributed load (UDL) representing its full ultimate capacity under ambient conditions. The ultimate moment capacity M_u for the given section under ambient temperature is calculated as 213.9 kNm using the limit state method of IS 456:2000, assuming a limiting depth of neutral axis corresponding to $0.138 f_{ck} b d^2$. For a fixed-ended beam, the negative moment at supports equals $wL^2/12$ and the positive moment at mid-span equals $wL^2/24$. The critical moment is at the supports, and solving it gives the ultimate UDL as 71.3 kN/m. This fully factored load of 71 kN/m is applied uniformly along the entire beam length and remains constant throughout the fire exposure. This represents a worst-case scenario where the beam is operating at its maximum design capacity before fire initiates.

2.4 Thermal Boundary Conditions and Analysis Cases

The fire exposure is applied according to the standard ISO 834 curve, which defines furnace temperature as a function of time: $T(t) = 20 + 345 \log_{10}(8t + 1)$, where t is time in minutes. For the present study, the fire duration is fixed at 30 minutes, at which point the furnace temperature reaches approximately 841°C. The convective heat transfer coefficient at exposed surfaces is taken as 25 W/m²K, and the surface emissivity is 0.7 for concrete. All surfaces designated as exposed in each case are subject to the ISO 834 thermal boundary condition, while all other surfaces are assumed to be thermally insulated (adiabatic boundary) with zero heat flux. Four distinct exposure cases are defined. In Case 1 (one-face exposure), only the bottom soffit surface of the beam is exposed to fire, Case 2 (two-face exposure), the bottom soffit and one vertical side surface are exposed, Case 3 (three-face exposure), the bottom soffit, both vertical sides, Case 4 (four-face exposure), bottom, both sides, top is exposed to fire, representing a standalone beam or a beam within a fully developed compartment fire where flames surround all sides. For the reference ambient condition, no thermal load is applied, and all materials retain their ambient properties.

2.5 Numerical Implementation in ABAQUS

The analysis is performed using ABAQUS finite element software employing a coupled temperature-displacement analysis procedure with direct coupling. This approach solves the thermal and mechanical equilibrium equations simultaneously at each increment, which is essential for capturing the interactive effects where mechanical deformation alters thermal contact conditions and thermal

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strains induce mechanical stresses. The beam is discretised using 8-node thermally coupled brick elements (C3D8T), which have both temperature and displacement degrees of freedom at each node. A structured mesh is generated with an element size of 25 mm, resulting in a total of approximately 12,000 elements. The reinforcement bars are embedded within the concrete mesh using the ABAQUS embedded element constraint, assuming perfect bond between steel and concrete (no slip). The analysis is performed as a transient procedure over a total time of 1800 seconds (30 minutes). The initial time increment is set to 0.1 seconds, with a maximum increment of 10 seconds, allowing stable convergence while maintaining solution accuracy. Newton-Raphson iteration with automatic time stepping is employed to resolve nonlinearities arising from temperature-dependent material properties and geometric nonlinearity (large deflection effects). The ambient initial temperature of the entire model is set to 20°C. The coupled temperature-displacement analysis provides as output the temporal evolution of temperature at every node, the displacement field, and the stress-strain state throughout the beam, from which mid-span deflection and residual moment capacity are extracted for comparison across the five analysis cases.

2.6 Determination of Residual Moment Capacity

After completing the 30-minute fire exposure analysis for each case, the residual moment capacity is determined using the approach, wherein the temperatures of the bottom steel, top steel, and concrete compression zone are extracted at the end of the fire step, reduction factors are applied to material strengths using Eurocode 2, and the flexural capacity is recalculated using the limit state method of IS 456:2000 with reduced strengths.

III. RESULTS AND DISCUSSIONS

3.1 RESULTS

3.1.1 Temperature Distribution

The temperature distribution across the beam cross-section at mid-span after 30 minutes of fire exposure varies significantly with the number of exposed faces. For the ambient condition, the entire beam remains uniformly at 20°C.

- For Case 1 (one-face exposure, soffit only), heat penetrates upwards from the bottom surface only, creating a steep thermal gradient. The bottom concrete surface reaches approximately 720°C, the bottom reinforcement (25 mm from the bottom) reaches 412°C, and the top reinforcement records only 42°C.

- For Case 2 (two-face exposure, soffit and one vertical side), asymmetric heating is observed. The bottom reinforcement reaches 468°C, the reinforcement on the exposed vertical side reaches 460°C, while the unexposed side reinforcement remains at 85°C. The top reinforcement records 58°C.

- For Case 3 (three-face exposure, soffit and both vertical sides), the bottom reinforcement temperature rises to 534°C. Both side reinforcements are significantly heated to approximately 520°C each, while the top reinforcement remains relatively cool at 58°C due to the absence of direct flame exposure on the top surface.

- For Case 4 (four-face exposure, soffit, both sides, and top), the entire cross-section experiences substantial heating. The bottom reinforcement reaches 621°C, the top reinforcement heats to 287°C, and the side reinforcements reach approximately 600°C and 350°C on exposed and interior portions, respectively. The concrete core at mid-depth reaches approximately 350°C.

3.1.2 Mid-Span Deflection

The mid-span deflection measured at the end of 30 minutes of fire exposure shows a clear progression with increasing exposed faces. Under ambient conditions with the full design UDL of 71 kN/m, the fixed-ended beam exhibits an elastic deflection of 8.2 mm at mid-span.

- For Case 1 (one-face exposure), the deflection increases to 14.3 mm after 30 minutes. This remains well within the serviceability limit of L/250 (24 mm for a 6000 mm span) specified by IS 456:2000.

- For Case 2 (two-face exposure with one side), the deflection reaches 18.5 mm. The asymmetric heating induces vertical deflection along with a slight lateral deflection of 2.1 mm and twist of 0.8 degrees, though the vertical component remains the dominant deformation mode.

- For Case 3 (three-face exposure, both sides), the mid-span deflection reaches 22.7 mm, approaching the L/250 limit of 24 mm.

- For Case 4 (four-face exposure), the deflection becomes excessive at 36.8 mm, well exceeding the L/250 limit.

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3.1.3 Residual Moment Capacity

The residual moment capacity after 30 minutes of fire exposure, expressed as a percentage of the ambient ultimate moment capacity (213.9 kNm), follows a progressive degradation pattern.

- For Case 1 (one-face exposure), 82% of the ambient capacity is retained. This corresponds to a residual moment capacity of approximately 175 kNm.
- For Case 2 (two-face exposure with one side), 74% of the ambient capacity is retained, corresponding to approximately 158 kNm.
- For Case 3 (three-face exposure, both sides), the residual capacity drops to 67% of the ambient value, corresponding to approximately 143 kNm.
- For Case 4 (four-face exposure), the residual capacity is severely reduced to 45% of the ambient value, corresponding to approximately 96 kNm.

3.2 DISCUSSIONS

3.2.1 Mechanism of Temperature Development with Increasing Exposed Faces

The results show a clear and progressive increase in reinforcement temperatures as the number of exposed faces increases. Bottom rebar temperature rises from 412°C in Case 1 to 468°C in Case 2, 534°C in Case 3, and 621°C in Case 4. This trend occurs because each additional exposed face provides a new path for heat to enter the beam. When only the soffit is exposed (Case 1), heat travels upward through 450 mm of concrete. Concrete has low thermal diffusivity, approximately 0.5×10^{-6} m²/s, meaning it resists heat flow effectively (Malik et al., 2010). The top reinforcement at 42°C remains nearly at ambient temperature because the concrete above the neutral axis acts as a thermal insulator. This insulating property is the fundamental reason why one-face exposure causes minimal damage (Bailey, 2002). When one vertical side is also exposed (Case 2), heat enters from two perpendicular directions. The distance from the vertical side face to the other is only 300 mm, which is less than the 450 mm distance from the soffit to the top. Therefore, the side reinforcement heats rapidly to 460°C. More importantly, the convergence of heat fronts from the bottom and side raises the bottom reinforcement temperature to 468°C higher than in Case 1. This "thermal convergence" effect has been documented by Wickström (2016), who showed that heat flux at interior points increases when multiple surfaces are heated because the thermal gradients from each direction add vectorially. When the second vertical side is exposed (Case 3), heat now enters from three directions: bottom, left side, and right side. The bottom reinforcement, located at the intersection of all three heat fronts, experiences the most significant temperature rise to 534°C. The 66°C increase from Case 2 to Case 3 is larger than the 56°C increase from Case 1 to Case 2. This occurs because the two side heat fronts converge symmetrically, creating a "hot zone" in the lower portion of the beam (Dwaikat and Kodur, 2009). When the top surface is also exposed (Case 4), heat enters from all four directions. The top reinforcement now receives direct flame exposure, reaching 287°C. More critically, the temperature gradient through the beam depth flattens because the top is no longer a cool boundary. Heat now flows toward the bottom reinforcement from above as well as from below, raising its temperature to 621°C. This represents an 87°C increase from Case 3, the largest single increment among all cases. The mechanism is "thermal reversal"—instead of heat flowing only upward, it now converges on the bottom reinforcement from both above and below (Liew, 2004).

4.2 Mechanism of Deflection Progression and Threshold Behaviour

The deflection results show a non-linear progression: 14.3 mm (Case 1), 18.5 mm (Case 2), 22.7 mm (Case 3), and 36.8 mm (Case 4). The most striking feature is the large jump between Case 3 and Case 4, where deflection increases by 14.1 mm compared to increments of 4.2 mm and 4.2 mm between previous cases. This deflection behaviour is governed by the temperature-dependent reduction in steel yield strength. According to Eurocode 2 (EN 1992-1-2), steel retains 100% of its yield strength up to 400°C. Between 400°C and 500°C, the reduction is minimal (about 5-10%). However, between 500°C and 600°C, yield strength drops rapidly from 78% to 47% of the ambient value. Above 600°C, strength retention falls below 44% (Eurocode 2, 2004). In Case 1 (bottom rebar 412°C), steel strength loss is negligible, so deflection (14.3 mm) remains primarily due to reduced concrete modulus and thermal curvature. In Case 2 (468°C), steel strength loss is still minimal (approximately 5%), so deflection (18.5 mm) increases moderately. In Case 3 (534°C), the steel has lost approximately 28% of its yield strength ($k_s = 0.72$), causing the beam to approach its limit (22.7 mm). In Case 4 (621°C), the steel has lost 56% of its yield strength ($k_s = 0.44$), causing a dramatic deflection increase to 36.8 mm. This threshold behaviour around 500-550°C has been experimentally confirmed by El-Hawary et al. (1997), who observed rapid deflection acceleration when reinforcement temperatures exceeded 500°C. The sharp deflection increase at 22 minutes in Case 4 corresponds to the time when the bottom rebar temperature crosses 550°C. At this temperature, the steel undergoes microstructural transformation from ferrite to austenite, causing a sudden loss of load-bearing capacity (Wang et al., 2020). Additionally, the concrete compression zone by this time has been heated above 300°C, causing dehydration of calcium silicate hydrate (C-S-H) gel and

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irreversible strength loss (Arioz, 2007). The combined effect of steel weakening and concrete softening produces the observed deflection surge. The asymmetric deflection in Case 2—vertical deflection of 18.5 mm accompanied by lateral deflection of 2.1 mm and twist of 0.8 degrees—occurs because the heated side expands more than the unexposed side. This differential expansion creates a torsional moment about the beam's longitudinal axis. EI-Hawary (1996) documented this phenomenon in beams under localised fire exposure, noting that torsional effects become significant when the temperature difference between opposite sides exceeds 200°C. In Case 2, the difference between the exposed side (approximately 460°C) and the unexposed side (85°C) is 375°C, well above this threshold.

4.3 Mechanism of Residual Capacity Reduction

The residual moment capacity decreases progressively: 82% (Case 1), 74% (Case 2), 67% (Case 3), and 45% (Case 4). Three distinct mechanisms contribute to this reduction: tension steel weakening, compression zone softening, and lever arm reduction.

- **Tension Steel Weakening:** This is the primary mechanism in all cases but dominates in Cases 1-3. At 412°C (Case 1), steel retains a major portion of its yield strength, so capacity loss is only 18%. At 468°C (Case 2), steel retains approximately 95% strength, but capacity loss is 26%, indicating other mechanisms are active. At 534°C (Case 3), steel retains 72% strength, and capacity loss is 33%, closely matching the steel loss. At 621°C (Case 4), steel retains 44% strength, but capacity loss is 55%, far exceeding the steel loss alone. This indicates that additional mechanisms become dominant in four-face exposure (Gao et al., 2009).

- **Compression Zone Softening:** This mechanism becomes significant in Case 4. When the top surface is exposed, the compression zone concrete is heated to temperatures exceeding 300°C. At 300°C, concrete retains approximately 85% of its compressive strength; at 350°C, approximately 75% (Arioz, 2007). The reduction occurs because the C-S-H gel—the primary binding agent in concrete—begins to dehydrate and decompose above 200°C, with accelerated decomposition between 300°C and 500°C. This decomposition is irreversible and results in permanent strength loss (Malik et al., 2010). In Case 4, the upper 100 mm of the beam (approximately half the compression zone depth) reaches temperatures between 300°C and 400°C, reducing its average compressive strength to approximately 80% of ambient. This 20% reduction in concrete strength directly reduces the compression force that can be developed, requiring a deeper neutral axis to maintain force equilibrium (Liew, 2004).

- **Lever Arm Reduction:** The lever arm is the distance between the centroid of tension steel and the centroid of the compression force. In an under-reinforced beam at ambient temperature, the neutral axis depth is approximately 0.2d to 0.3d, giving a lever arm of approximately 0.9d. When the compression zone concrete softens due to heating, the neutral axis must move deeper into the section to develop the same compression force with weaker concrete. In Case 4, this effect is pronounced, thus reducing the lever arm to the greatest extent (Kodur and Dwaikat, 2008). This lever arm reduction reduces the moment capacity of the beam.

IV. Conclusions

Based on the coupled temperature-displacement analysis of a 6 m fixed-ended RC beam (M30 concrete, Fe500 steel) under ISO 834 fire for 30 minutes with variable exposed faces, the following conclusions are drawn:

- Temperature increases progressively with more exposed faces. Bottom rebar temperature rises from 412°C (1-face) to 468°C (2-face), 534°C (3-face), and 621°C (4-face). Four-face exposure also heats the top rebar to 287°C.
- Deflection exceeds serviceability limit only for 4-face exposure. Deflections are 14.3 mm (1-face), 18.5 mm (2-face), 22.7 mm (3-face), and 36.8 mm (4-face).
- Residual moment capacity decreases systematically**.** Capacity retention is 82% (1-face), 74% (2-face), 67% (3-face), and 45% (4-face).
- Two-face exposure (soffit + one side) causes asymmetric heating, inducing lateral deflection (2.1 mm) and twist (0.8 degrees) in addition to vertical deflection.
- Four-face exposure is most critical, activating three simultaneous degradation mechanisms: tension steel weakening (56% loss), compression zone softening (20% loss), and lever arm reduction.
- The number of fire-exposed faces is a critical parameter that must be considered in fire-safe design of reinforced concrete beams.

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