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Post fire bending moment capacity of steel beam under different thermal environments and support conditions

Çelik kirişin farklı termal ortamlar ve destek koşulları altında yangın sonrası eğilme momenti kapasitesi

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Post Fire Bending Moment Capacity of Steel Beam Under Different Thermal Environments and Support Conditions

Highlights

- *Fixed beams showed significant capacity reduction (8.8%–14.7%), while simply supported beams exhibited minimal reduction (0.05%–2.1%) after fire exposure.*
- *Capacity reduction occurs even after full material property recovery, confirming that residual stresses and permanent distortions control post-fire behaviour.*
- *Two-sided fire exposure produced the most critical response, causing up to 9.52% additional reduction compared to one-sided heating, after which capacity reduction stabilizes.*

Graphical Abstract

The steel beam was heated from different sides and post fire bending momnent capacity of the beam was determined.

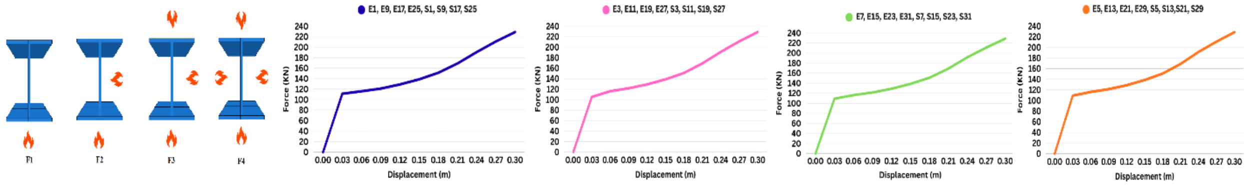

Figure. Beam heating configurations and post fire force displacement curves

Aim

The aim of this study is to determine the post-fire bending moment capacity of the steel beam and the corresponding reduction compared to ambient temperature, and to investigate the effect of fire profiles, cooling durations, support conditions, and exposure sides on the post-fire flexure performance of the beam.

Design & Methodology

A 3D fully coupled temperature–displacement finite element model was developed in Abaqus using C3D8T elements to simulate the complete fire cycle (heating and cooling) followed by a displacement-controlled loading test to determine residual bending capacity. The study systematically investigates 64 parametric cases by varying fire profiles (external and standard), cooling durations (15–60 min), exposure conditions (one to four sides), and support conditions (fixed and simply supported) for an ISWB 150 steel beam.

Originality

This study develops a fully coupled thermal–structural finite element framework to evaluate post-fire bending capacity of steel beams by modelling the complete heating–cooling cycle. It uniquely quantifies the combined influence of support restraint, fire exposure sides, and thermal history on residual capacity, isolating structural effects even under full material strength recovery.

Findings

Fixed beams exhibited a significant reduction in post-fire bending capacity (8.8%–14.7%), while simply supported beams showed only a minor reduction (0.05%–2.1%). Support condition and exposure sides were the most influential parameters, whereas fire profile and cooling duration had negligible effect on residual capacity. Two-sided fire exposure caused the most critical reduction (up to 9.52% more than one-sided), after which the effect stabilizes for three- and four-sided exposure. Capacity reduction occurs due to thermo-mechanical effects (residual stresses and permanent deformation) rather than loss of material strength alone.

Conclusion

Post-fire bending capacity reduction in steel beams is governed mainly by support restraint and fire exposure, with fixed beams showing significantly higher reduction than simply supported beams. The reduction persists even after full material property recovery, indicating that residual stresses and permanent distortions developed during heating control the post-fire structural performance.

Declaration of Ethical Standards

The author(s) of this article declare that the materials and methods used in this study do not require ethical committee permission and/or legal-special permission.

Post Fire Bending Moment Capacity of Steel Beam Under Different Thermal Environments and Support Conditions

Araştırma Makalesi / Research Article

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ABSTRACT

Steel beams are highly susceptible to elevated temperatures during a fire, resulting in a rapid loss of strength and stiffness. However, their post-fire characteristic (after cooling) is equally important as it governs their chances of repair or reusability. This study focuses on assessing this post-fire characteristic in terms of the residual bending moment capacity, using finite element simulations. Different variables are considered to determine their effect on residual moment capacity. These include the heating profile (External Fire and Standard Fire), cooling durations (15, 30, 45, and 60 minutes), exposure conditions (Heating on one, two, three, or all sides of the beam), and support conditions (Fixed and Simple Support). The results show that post-fire bending moment capacity decreases by 8.8% to 14.7 % for the fixed beam, whereas the simply supported beam exhibits only a marginal reduction of about 0.05% to 2.01% when compared to the ambient temperature capacity. Among all variables, the support condition and exposure side had a considerable influence on the post-fire bending capacity, while the heating profiles and cooling durations showed negligible effects. Fixed support was the primary factor reducing capacity, while the two-sided exposure also intensified the response, causing a 9.52% capacity drop compared to one-sided heating.
Keywords: Thermal-structural finite element analysis, post-fire structural performance of steel section, residual bending moment capacity, external and standard fire, support restraint effect.

Çelik Kirişin Farklı Termal Ortamlar ve Destek Koşulları Altında Yangın Sonrası Eğilme Momenti Kapasitesi

ÖZ

Çelik kirişler yangın sırasında yüksek sıcaklıklara karşı oldukça hassastır ve bu da hızlı bir şekilde mukavemet ve sertlik kaybına neden olur. Ancak yangın sonrası özellikleri (soğuduktan sonra) onarım veya yeniden kullanılabilirlik şanslarını belirlediği için aynı derecede önemlidir. Bu çalışma, sonlu elemanlar simülasyonları kullanılarak, yangın sonrası bu özelliğin artık bükülme momenti kapasitesi açısından değerlendirilmesine odaklanmaktadır. Artık moment kapasitesi üzerindeki etkilerini belirlemek için farklı değişkenler dikkate alınır. Bunlar, ısıtma profilini (Harici Yangın ve Standart Yangın), soğutma sürelerini (15, 30, 45 ve 60 dakika), maruz kalma koşullarını (kirişin bir, iki, üç veya her tarafında ısıtma) ve destek koşullarını (Sabit ve Basit Destek) içerir. Sonuçlar, sabit kiriş için yangın sonrası bükülme momenti kapasitesinin %8,8 ila %14,7 oranında azaldığını, oysa basit bir şekilde desteklenen kirişin, ortam sıcaklığı kapasitesiyle karşılaştırıldığında yalnızca yaklaşık %0,05 ila %2,01 oranında marjinal bir azalma sergilediğini göstermektedir. Tüm değişkenler arasında, destek durumu ve maruz kalma tarafı, yangın sonrası bükme kapasitesi üzerinde önemli bir etkiye sahipken, ısıtma profilleri ve soğutma süreleri ihmal edilebilir etkiler gösterdi. Sabit destek, kapasiteyi azaltan birincil faktör olurken, iki taraflı maruz kalma da tepkiyi yoğunlaştırdı ve tek taraflı ısıtmaya kıyasla %9,52'lik bir kapasite düşüşüne neden oldu.
Anahtar Kelimeler: Termal-yapısal sonlu eleman analizi, yangın sonrası çelik kesitlerin yapısal performansı, kalan eğilme momenti kapasitesi, dış kaynaklı ve standart yangın, destek kısıtlama etkisi.

1. INTRODUCTION

Steel is extensively used in modern construction for bridges, hangers, building frames, and other structures due to its high strength, durability, and scrap value [1-2]. Steel structures are generally designed for ambient loading conditions, considering the limit states of strength and serviceability [3-4]. However, when exposed to elevated temperatures, such as during fire, steel undergoes significant changes in mechanical properties. Both strength and stiffness decrease with increased temperature due to microstructural changes, including grain coarsening, phase transitions, and increased atomic diffusion [5-7]. For instance, at approximately 400°C, steel can lose nearly half of its strength and stiffness compared to its ambient temperature condition [8]. Additionally, hardness may

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decrease as the temperature approaches the melting point of the alloy [9]. These reductions are influenced by factors such as heating rate and alloying elements: rapid heating accelerates strength loss, whereas the presence of alloying elements such as molybdenum and boron can mitigate reductions at elevated temperatures [10-11].

1.1. Literature Review

Extensive research has examined the behaviour of steel beams and columns during fire exposure, primarily focusing on how elevated temperatures degrade material properties and structural response. Increasing temperature causes significant reductions in yield strength and stiffness, the disappearance of the yield plateau, and a progressively nonlinear stress-strain relationship [12]. At the structural level, this degradation leads to reduced load-carrying capacity [13]. However, the fire response of steel members is also influenced by section geometry and slenderness, web opening, creep, imperfections, residual stress, load level, restraint conditions, heating duration, peak temperature, exposure configuration and joint behaviour [14-16]. Beyond material degradation, structural restraint plays a critical role in fire performance. Axial and rotational restraint can induce significant internal forces, causing members to behave as beam-columns rather than isolated elements [17]. Non-uniform temperature distributions generate thermal bowing, which strongly affects deformation behaviour. Thermal gradients also modify stress distributions and axial force-bending moment interaction relationships. Higher load ratios further reduce failure temperatures, highlighting the combined influence of mechanical loading and temperature gradients [18].

Considerable research has explored post-fire material properties of steel at the coupon level, focusing on residual yield strength after heating and cooling. Peak temperature is consistently identified as the dominant factor governing strength degradation, with exposure near 1000°C leading to substantial reductions depending on steel type and exposure duration [19]. Cooling method also influences residual properties, as water cooling can produce slightly greater strength loss than air cooling due to steeper thermal gradients and microstructural transformations [20]. Heating and cooling alter phase composition and grain structure, affecting post-fire mechanical performance [21].

Experimental and numerical studies of post-fire steel and composite members indicate that many elements, particularly columns, can retain a substantial portion of their original load-carrying capacity after moderate fire exposure, often around 70-90%, depending on peak temperature, fire duration, cooling conditions, slenderness, and restraint [22-23]. Composite and confined sections, such as CFST (Concrete Filled Steel Tube) columns, generally show improved residual performance due to confinement and composite action [24]. In contrast, beam research is more limited. Available studies suggest that peak temperature governs residual capacity more strongly than heating duration, bending resistance decreases after fire, and the failure mode may shift towards connections, accompanied by increased permanent deformation and residual stresses [25-26].

The cooling phase following fire exposure is also crucial in determining residual mechanical properties. Cooling rate becomes particularly significant when peak temperatures exceed the austenitic transformation range of about 600-700°C, where phase transformations govern post-fire behaviour [27]. Rapid cooling, such as water quenching, may increase residual strength through martensite formation but reduces ductility and increases brittleness [28]. Slower cooling methods, including air cooling, generally result in lower strength but improved ductility and more stable deformation capacity [29].

1.2. Research Gap and Novelty

Most structural fire research focuses on the behaviour of steel members during heating and evaluates fire resistance in terms of failure time, while the residual load-carrying capacity after cooling is rarely quantified. Studies that do address post-fire conditions are largely limited to coupon-level material testing, which cannot capture the effect of restraint, thermal gradient, residual stress, and permanent distortions that control the response of full structural members. The combined influence of fire profile, exposure sides, support restraint, and cooling durations on residual bending capacity of steel beams remains largely unexplored. To address this gap, the present study develops a fully thermal structural finite element framework that models the complete fire cycle (heating and cooling) and evaluates the post-fire bending moment capacity of steel beams. The objectives are to (1) determine the post-fire bending moment capacity of the steel beam and the corresponding reduction compared to ambient temperature, and (2) to investigate the effect of fire profiles, cooling durations, support conditions, and exposure sides on the post-fire flexure performance of the beam. This provides a more realistic basis for post-fire assessment and decision-making regarding repair or reuse of fire-damaged steel members.

2. MATERIAL and METHOD

In this study, the post-fire bending moment capacity of the beam and its dependency on fire profiles, exposure sides, cooling durations, and support conditions are determined. To achieve this, a symmetric I-section beam with a 3 m span is used as a base, configured to ISWB 150 geometry (as per SP: 6 (1) - 1964 (Reaffirmed 2003) [30]):

- Total Depth: 150 mm
- Width of Flange: 100 mm
- Thickness of Flange: 7 mm
- Thickness of Web: 5.4 mm

A three-dimensional model of this section (Figure 1) was created, and a combined temperature-displacement finite element analysis was performed using the finite element software Abaqus. The beam was discretised in

Abaqus using 8 node coupled temperature-displacement brick element C3D8T.

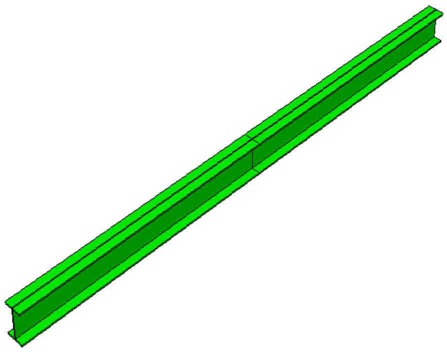


Figure 1. Three-dimensional model of steel I-section

A fully coupled temperature-displacement analysis was performed to simultaneously solve the heat transfer and structural response of the beam. The fire heating and subsequent cooling phases were applied within the same analysis step, allowing thermal expansion, plastic strain accumulation, residual stresses and geometric distortion to develop naturally during the thermal cycle. After the beam cooled back to ambient temperature, an additional displacement-controlled loading step was introduced at mid span to determine the post-fire bending moment capacity of the beam.

2.1. Material Properties

Beam is considered to be made of E-250 grade steel, the properties of which at ambient temperature (20°C) are as follows:

- Yield Strength: 250 MPa
- Modulus of Elasticity: $2*10^5$ MPa

Steel was modelled as a temperature-dependent elastic-plastic material by using the reduction factors for elasticity and yield strength as given by Eurocode 3 [31]. The temperature-dependent variation of thermal and mechanical properties of steel is presented in Table 1.
The density and Poisson's ratio of steel are considered constant as per Eurocode 3, having a magnitude of 7850 Kg/m^3 and 0.3, respectively.
During the cooling phase, the temperature-dependent reduction factors or properties of steel presented in Table 1 were restored to their ambient temperature values. This assumption of complete strength recovery was adopted due to the inherent complexity associated with accurately quantifying the degree of mechanical property restoration in steel under realistic fire scenarios. In actual fire events, the extent of strength recovery is strongly influenced by the cooling regime, including the cooling rate, cooling method (e.g., air-cooling or water quenching), and the duration of elevated temperature exposure.

2.2. Loading

The self-weight of the beam is accounted for as gravity loading, assuming an acceleration due to gravity of 9.81 m/s^2.

In this study, only self weight of the beam is considered a mechanical loading. No additional service or live loads were applied during the thermal analysis. The reason is that the post-fire bending moment capacity was determined through a displacement-controlled test on the cooled beam. This procedure inherently applies external displacement at mid span until yielding, thereby capturing the residual bending capacity of the section. Including extra mechanical loads would have overlapped with the displacement-controlled load step and complicated the interpretation of results. By restricting the analysis to the self-weight only, the study ensures that the observed reduction in bending capacity is solely due to thermal effects, support and exposure conditions, while the displacement-controlled test provides a direct measure of the residual bending strength.

2.3. Limitations of The Present Work

- It has been assumed in the study that the steel regains its full strength after the cooling has entirely taken place. The assumption of full material strength recovery provides an upper bound on the material's residual capacity. This also forms a controlled basis for isolating the influence of thermomechanical history on post-fire structural performance. It thus enables the evaluation of the reduction in the bending capacity attributable solely to residual stress, thermal gradients, and irreversible deformation effects, even when the intrinsic material strength is assumed to have fully recovered. Thus post-fire bending moment capacity values reported in this study should be interpreted as the upper bound of the maximum possible residual values.
- The provision of only self-weight as mechanical loading, neglecting other mechanical loads, may underestimate thermal-induced inelastic deformations, creep effects and residual damage accumulation; hence, this may therefore affect the predicted post-fire capacity. This assumption is a simplified approach adopted for post-fire residual capacity studies. It does not fully represent the real structural fire conditions.
- The adopted heating plus linear cooling histories in the present study resemble a simplified full fire cycle. If interpreted as such, the cooling rate will have some physical linkage to fire/compartment conditions such as ventilation and thermal inertia, as in the case of parametric fires. Since the beam is analysed as an individual member in the present study, and not as a part of a structure, these effects have not been considered herein.
- For real steel beams, especially with slab/ceiling interaction, one-sided bottom exposure may involve shielding effects, which can alter the temperature distribution. This effect has not been considered in the present study.

Table 1*:* Temperature-dependent variation of thermal and mechanical properties of steel (Source: Eurocode 3 [31])

Temperature (°C)	Specific Heat (J/KgK)	Thermal Conductivity (W/mK)	Coefficient of Thermal Expansion (/°C)	Modulus of Elasticity (N/m²)	Yield Strength (N/m²)
20	439.8	53.3	0.00001216	$2*10^{11}$	250000000
100	487.62	50.7	0.0000128	$2*10^{11}$	250000000
200	529.76	47.3	0.0000136	$1.80*10^{11}$	250000000
300	564.74	44	0.0000144	$1.60*10^{11}$	250000000
400	605.88	40.7	0.0000152	$1.40*10^{11}$	250000000
500	666.5	37.4	0.000016	$1.20*10^{11}$	195000000
600	760.21	34	0.0000168	$6.20*10^{11}$	117500000
700	1008.15	30.7	0.0000176	$2.60*10^{11}$	57500000
800	803.26	27.3	0	$1.80*10^{11}$	27500000
900	650	27.3	0.00002	$1.35*10^{11}$	15000000
1000	650	27.3	0.00002	$9.00*10^{11}$	10000000
1100	650	27.3	0.00002	$4.50*10^{11}$	5000000
1200	650	27.3	0.00002	1	1

2.4. Variables

The following variables are considered in this study:

2.4.1. Fire profile

1. **Heating Phase:** Two fire profiles; External Fire and Standard Fire – defined in Eurocode 1 [32] and each having a 30-minute duration (representing the typical time during which firefighting measures are implemented) are considered in this study, as illustrated in Figure 2 (a) and (b).
2. **Cooling Phase:** The peak fire temperature is reduced to ambient temperature by considering a linear cooling phase over four durations: 15, 30, 45, and 60 minutes, as illustrated in Figure 2 (a) and (b).

Since post-fire cooling duration in real structures depends on several factors, such as time of fire-fighting intervention, method of fire suppression (water, foam, or natural decay), ventilation changes, and compartment thermal inertia. The cooling phase in real fires does not follow a single deterministic profile. Instead, a range of decay rates is observed in practice. For this reason, four arbitrary cooling durations (15, 30, 45 and 60 minutes) were adopted to represent a spectrum of plausible cooling severities, from relatively rapid cooling (active firefighting) to slow natural cooling.

2.4.2. Exposure conditions

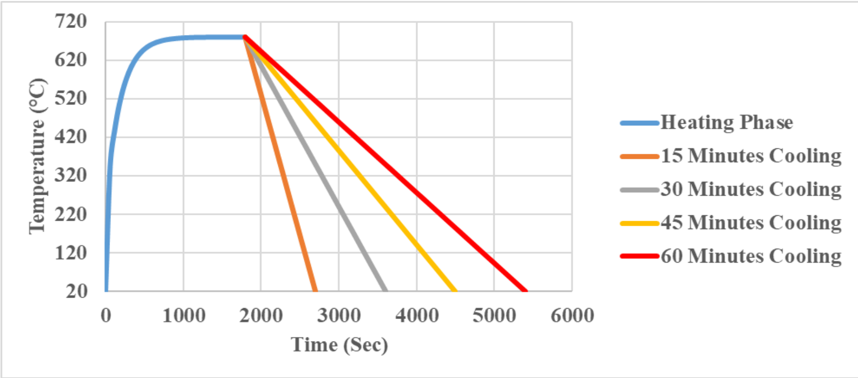
To incorporate different fire-exposure conditions for the beam, four cases were considered (as shown in Figure 3):

- F1: Beam heated from one side
- F2: Beam heated from two sides
- F3: Beam heated from three sides
- F4: Beam heated from four sides

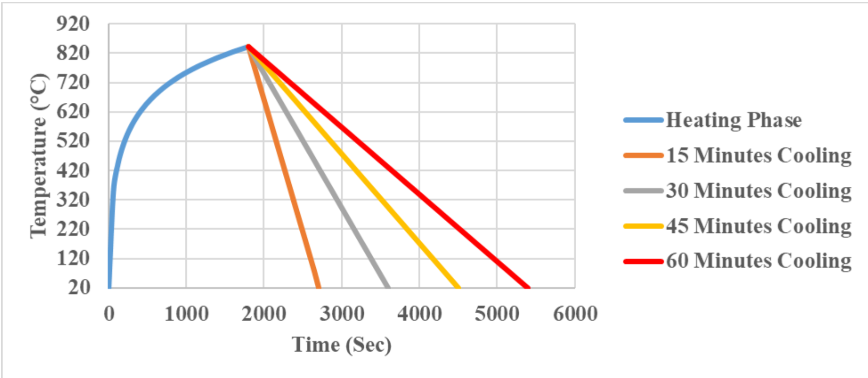
Fire exposure was applied through prescribed temperature-time curves (External and Standard) on the designated exposed beam surfaces (one side, two sides, three and four side heating) along the full length of the beam. In a particular exposure case, the remaining surfaces were treated as unexposed. The heating phase was followed by a cooling phase back to ambient temperature.
The different fire exposure conditions (F1 to F4) were selected to represent the range of realistic fire scenarios that steel beams may encounter in buildings. In practice, the sides exposed to fire depend on factors such as slab continuity, beam position (edge or internal), ceiling systems, and compartment geometry. Since the most critical exposure configuration cannot be known in advance, all exposure cases were considered. This approach enables a more general and physically representative assessment of post-fire structural performance rather than relying on a single assumed heating condition.

2.4.3. Support condition

Based on the support condition, two types of beams were considered: fixed beam and simply supported beam. As, shown in Figure 4, for a fixed beam, all translational and rotational degree of freedom is restrained at both ends of the beam. For the simply supported beam, one end was pinned, and the other was roller supported to allow axial expansion during heating.



(a)



(b)

Figure 2. Thermal profile of fire in **(a)** Case of external fire, and (**b**) Case of standard fire

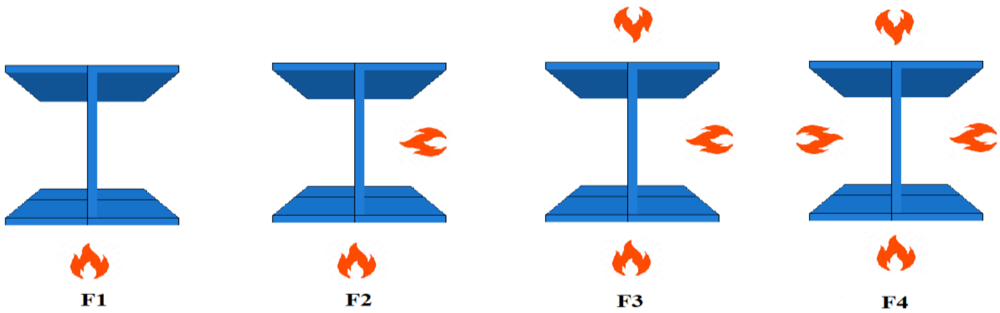


Figure 3. Fire exposure conditions

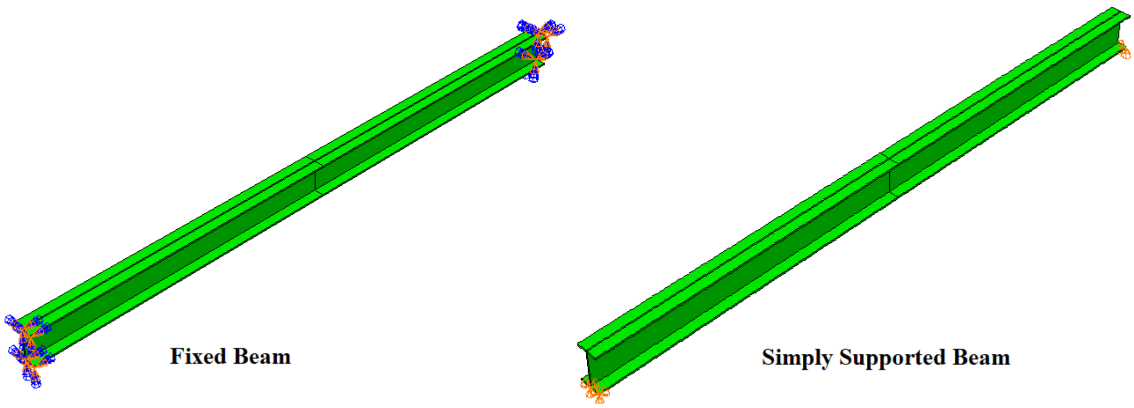


Figure 4. Beam support conditions

2.5. Analysis Cases

Based on the combination of variables considered in this study, 64 analysis cases were formed, as presented in Table 2.

2.6. Determination of Pre and Post-Fire Bending Moment Capacity

A displacement-controlled test is conducted, in which the applied displacement is gradually increased with time. The displacement is imposed at the location of maximum bending moment (i.e., at mid-span). This procedure is used to obtain the force-displacement curves for both the ambient temperature section and the cooled section after fire. From these curves, the force corresponding to the yield point is identified, using which the bending moment capacity of the beam is determined by using the following relations:

1. Fixed Beam with Point Load at Centre
$$M = \frac{F * L}{8}$$
2. Simply Supported Beam with Point Load at Centre
$$M = \frac{F * L}{4}$$

Here,
M: Bending moment on the beam due to external load
F: External force on the beam
L: Length of beam

2.7. Mesh Sensitivity Analysis and Model Validation

A mesh sensitivity analysis was performed on a representative case (Fixed Beam, One Side Exposure, Standard Fire, and 15-minute Cooling Duration) using global mesh sizes of 20 mm, 15 mm and 10 mm. The post-fire bending moment capacity of the beam obtained was 44.76 KN-m, 41.69 KN-m, and 39.65 KN-m, respectively. The difference in post-fire bending capacity between the 20 mm and 15 mm mesh was about 7%, while the difference between the 15 mm and 10 mm mesh was approximately 5%, indicating that the solution was converging with mesh refinement. Therefore, the 15 mm mesh was adopted as a suitable balance between accuracy and computational cost.

The modelling approach employed a fully coupled temperature-displacement formulation with temperature-dependent material properties a specified in Eurocode 3 [31] and fire curves defined in Eurocode 1 [32], both of which are based on extensive experimental calibration. The modelling approach followed in this study follows established thermo-mechanical finite element procedures, commonly used in validated structural fire research. The framework is capable of capturing key fire induced mechanism, including restrained thermal expansion, thermal bowing, stiffness degradation at elevated temperatures, and residual stress development after cooling, as reported in previous studies on steel beams subjected to fire [33-35].

Table 2. Analysis cases based on a combination of different variables

Case Denotation		Cooling Duration (minutes)	Number of Fire Exposure Sides	Support Condition
External Fire	**Standard Fire**			
E1	S1	15	1	Fixed
E2	S2	15	1	Simple Support
E3	S3	15	2	Fixed
E4	S4	15	2	Simple Support
E5	S5	15	3	Fixed
E6	S6	15	3	Simple Support
E7	S7	15	4	Fixed
E8	S8	15	4	Simple Support
E9	S9	30	1	Fixed
E10	S10	30	1	Simple Support
E11	S11	30	2	Fixed
E12	S12	30	2	Simple Support
E13	S13	30	3	Fixed
E14	S14	30	3	Simple Support
E15	S15	30	4	Fixed
E16	S16	30	4	Simple Support
E17	S17	45	1	Fixed
E18	S18	45	1	Simple Support
E19	S19	45	2	Fixed
E20	S20	45	2	Simple Support

Table 2. (Cont). Analysis cases based on a combination of different variables

Case Denotation		Cooling Duration (minutes)	Number of Fire Exposure Sides	Support Condition
External Fire	Standard Fire			
E21	S21	45	3	Fixed
E22	S22	45	3	Simple Support
E23	S23	45	4	Fixed
E24	S24	45	4	Simple Support
E25	S25	60	1	Fixed
E26	S26	60	1	Simple Support
E27	S27	60	2	Fixed
E28	S28	60	2	Simple Support
E29	S29	60	3	Fixed
E30	S30	60	3	Simple Support
E31	S31	60	4	Fixed
E32	S32	60	4	Simple Support

3. RESULTS

By conducting displacement-controlled tests on a steel beam before and after fire exposure and considering various combinations of fire profiles, support and exposure conditions, the post-fire residual bending moment capacity of the beam and its corresponding reduction (compared to ambient conditions) is determined. The following nomenclature will be used in the following sub-sections:

$F_{Pre\text{-}Fire}$: Force corresponding to the yield point, when the beam is tested at ambient temperature

$F_{Post\text{-}Fire}$: Force corresponding to the yield point, when the beam is tested after cooling

$M_{Pre\text{-}Fire}$: Bending moment capacity of the beam at ambient temperature

$M_{Post\text{-}Fire}$: Bending moment capacity of the beam after cooling

R: Percentage reduction in the bending capacity of the beam after fire, compared to the ambient temperature capacity

3.1. Pre-Fire Bending Moment Capacity of Beam

The force-displacement curves for the fixed and simply supported beams, under ambient temperature conditions, are shown in Figure 5. Based on these results, the following observations were made:

1. **Fixed Beam**
   - $F_{Pre\text{-}Fire}$: 122.08 KN
   - $M_{Pre\text{-}Fire}$: 45.78 KN-m
2. **Simply Supported Beam**
   - $F_{Pre\text{-}Fire}$: 51.70 KN
   - $M_{Pre\text{-}Fire}$: 38.78 KN-m

3.2. Post-Fire Bending Moment Capacity of Beam

3.2.1. Fixed beam

The force-displacement curves are shown in Figure 6 (a)-(d), covering various analysis cases. The following ranges of data are obtained (a detailed bifurcation of which is presented in Table 3):

a) **External Fire**
   - $F_{Post\text{-}Fire}$: 104.03 to 111.19 KN
   - $M_{Post\text{-}Fire}$: 39.01 to 41.69 KN-m
   - R: 8.93 to 14.7 %

b) **Standard Fire**
   - $F_{Post\text{-}Fire}$: 104.35 to 111.23 KN
   - $M_{Post\text{-}Fire}$: 39.13 to 41.71 KN-m
   - R: 8.8% to 14.5 %

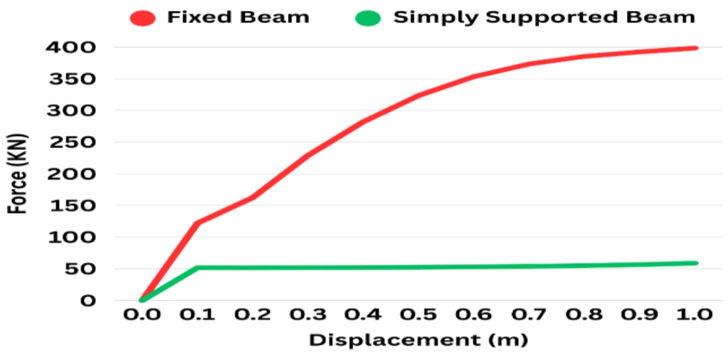


Figure 5. Force-displacement curve for steel beam at ambient temperature

3.2.2. Simply supported beam

The force-displacement curves are shown in Figure 7 (a)-(d), covering various analysis cases. The following ranges of data are obtained (a detailed bifurcation of which is presented in Table 4):

a) External Fire

- $F_{Post\text{-}Fire}$: 50.66 KN to 51.54 KN
- $M_{Post\text{-}Fire}$: 38.0 KN-m to 38.66 KN-m
- R: 0.3 % to 2.01 %

b) Standard Fire

- $F_{Post\text{-}Fire}$: 51.03 KN to 51.68 KN
- $M_{Post\text{-}Fire}$: 38.27 KN-m to 38.76 KN-m
- R: 0.05 % to 1.31 %

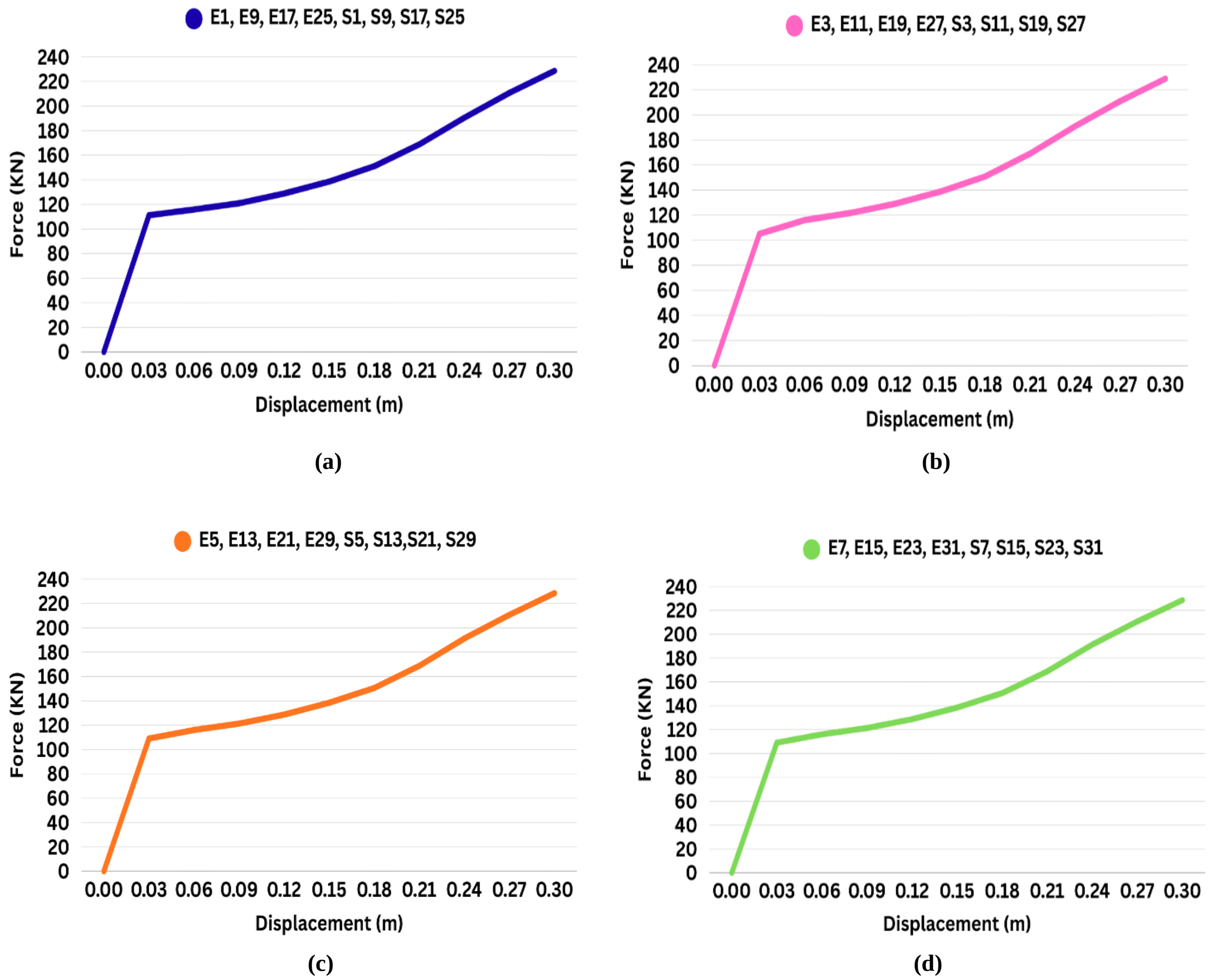


Figure 6 (a)-(d). Force-displacement curves for fixed steel beam after fire under various analysis cases

Table 3. Post-Fire yield force, bending moment capacity and its corresponding reduction for a fixed beam under external and standard fire

External Fire				Standard Fire			
Case	**$F_{Post\text{-}Fire}$ (KN)**	**$M_{Post\text{-}Fire}$ (KN-m)**	**R (%)**	**Case**	**$F_{Post\text{-}Fire}$ (KN)**	**$M_{Post\text{-}Fire}$ (KN-m)**	**R (%)**
E1	111.19	41.69	8.9	**S1**	111.23	41.71	8.8
E3	105.08	39.40	13.9	**S3**	104.46	39.17	14.4
E5	105.1	39.41	13.9	**S5**	104.75	39.28	14.1
E7	104.41	39.15	14.4	**S7**	109.20	40.95	10.5
E9	111.19	41.69	8.93	**S9**	111.23	41.71	8.8

Table 3. (Cont.) Post-Fire yield force, bending moment capacity and its corresponding reduction for a fixed beam under external and standard fire

External Fire				Standard Fire			
Case	**$F_{Post-Fire}$ (KN)**	**$M_{Post-Fire}$ (KN-m)**	**R (%)**	**Case**	**$F_{Post-Fire}$ (KN)**	**$M_{Post-Fire}$ (KN-m)**	**R (%)**
E11	105.03	39.38	13.9	**S11**	104.35	39.13	14.5
E13	105.09	39.41	13.9	**S13**	104.78	39.29	14.1
E15	104.38	39.14	14.5	**S15**	108.86	40.82	10.8
E17	111.19	41.69	8.93	**S17**	111	41.62	9.1
E19	105.03	39.38	13.9	**S19**	108.64	40.74	11.1
E21	105.09	39.41	13.9	**S21**	108.75	40.78	11
E23	104.03	39.01	14.7	**S23**	108.72	40.77	11
E25	111.19	41.69	8.93	**S25**	111.02	41.63	9.6
E27	105.03	39.38	13.9	**S27**	108.62	40.73	11.3
E29	105.10	39.41	13.9	**S29**	109.04	40.89	10.6
E31	104.42	39.15	14.4	**S31**	109.10	40.91	10.6

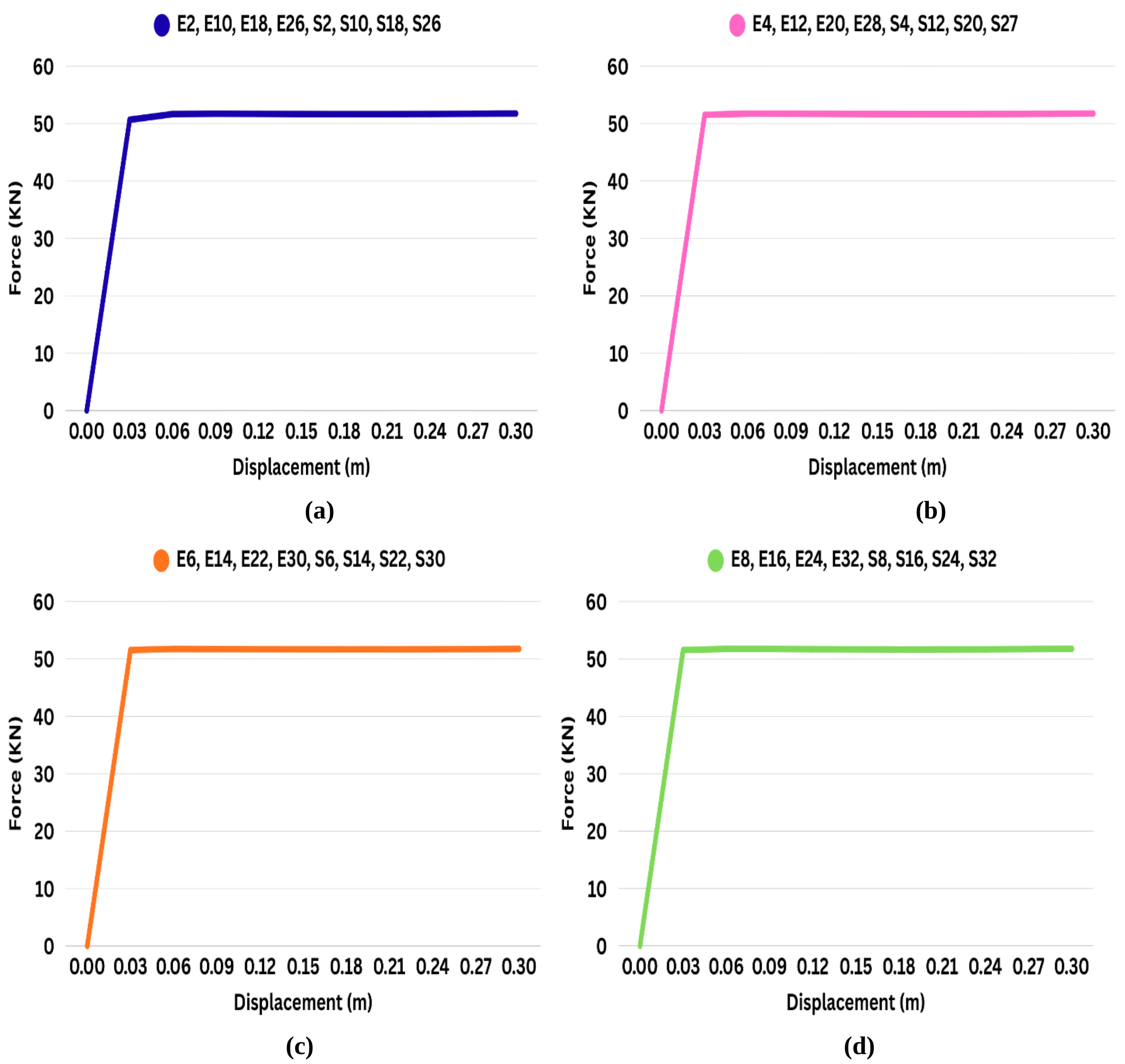


Figure 7 (a)-(d). Force-displacement curves for simply supported steel beam after fire under various analysis cases

Table 4. Post-Fire yield force, bending moment capacity and its corresponding reduction for a simply supported beam under external and standard fire

External Fire				Standard Fire			
Case	**$F_{Post-Fire}$ (KN)**	**$M_{Post-Fire}$ (KN-m)**	**R (%)**	**Case**	**$F_{Post-Fire}$ (KN)**	**$M_{Post-Fire}$ (KN-m)**	**R (%)**
E2	50.66	38	2.01	**S2**	51.55	38.66	0.30
E4	51.52	38.64	0.36	**S4**	51.63	38.72	0.15
E6	51.54	38.66	0.30	**S6**	51.59	38.69	0.23
E8	51.54	38.66	0.30	**S8**	51.55	38.66	0.30
E10	50.66	38	2.01	**S10**	51.53	38.64	0.36
E12	51.52	38.64	0.36	**S12**	51.67	38.75	0.07
E14	51.54	38.66	0.30	**S14**	51.65	38.73	0.12
E16	51.54	38.66	0.30	**S16**	51.58	38.68	0.25
E18	50.66	38	2.01	**S18**	51.68	38.76	0.05
S20	51.52	38.64	0.36	**S20**	51.09	38.31	1.21
E22	51.54	38.66	0.30	**S22**	51.03	38.27	1.31
E24	51.54	38.66	0.30	**S24**	51.45	38.58	0.51
E26	50.66	38	2.01	**S26**	51.62	38.71	0.18
E28	51.52	38.64	0.36	**S28**	51.14	38.35	1.10
E30	51.54	38.66	0.30	**S30**	51.22	38.41	0.95
E32	51.54	38.66	0.30	**S32**	51.59	38.69	0.23

4. DISCUSSIONS

4.1. Model Accuracy

The peak temperature attained by the beam in all analysis cases considered is equal to the peak fire temperature 679.96 °C (External Fire) and 841.8°C (Standard Fire), at 30 minute as per Eurocode 1 [32], while the temperature obtained after cooling is equal to the ambient temperature (20°C), establishing the accuracy of the thermal boundary condition of the model. For instance, the temperature profile obtained for cases E1 and S1 on the fire-exposed face (Figure 8) clearly supports the above observation.

4.2. Post-Fire Bending Moment Capacity of Beam

The post-fire bending behaviour observed in this study is governed by the thermo-mechanical history of the beam during heating. This is demonstrated by the fact that the steel properties were fully restored to their ambient values after cooling, yet the fixed beam still exhibited a reduction in bending capacity ranging from 8.8% to 14.7%, with the maximum reduction in Case E23 (14.7%) and the minimum in Cases S1 and S9 (8.8%). This confirms that the reduction in capacity originates from irreversible stress redistribution and geometric changes that develop while the beam is heated under restraint, also supported by [36]. When a fixed beam is exposed to elevated temperatures, the steel undergoes thermal expansion along its length and through the section depth. Because the support prevents axial expansion, significant compressive axial restraint forces develop [37]. At the same time, non-uniform temperature distribution across the section produces differential thermal strain, leading to thermal curvature (thermal bowing) [38]. Since both the elastic modulus and yield strength of steel reduce at high temperatures, the section becomes more susceptible to inelastic deformation [39].

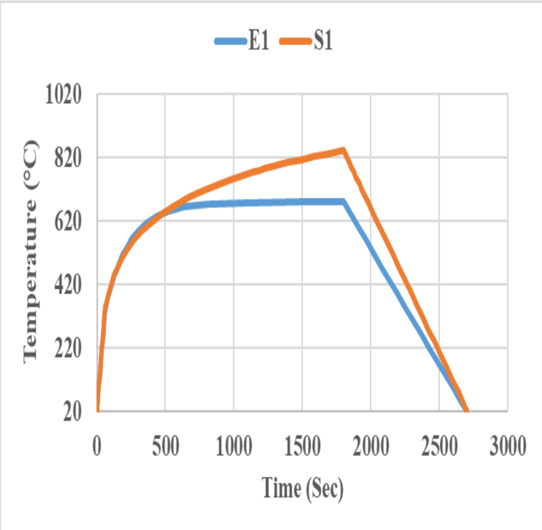
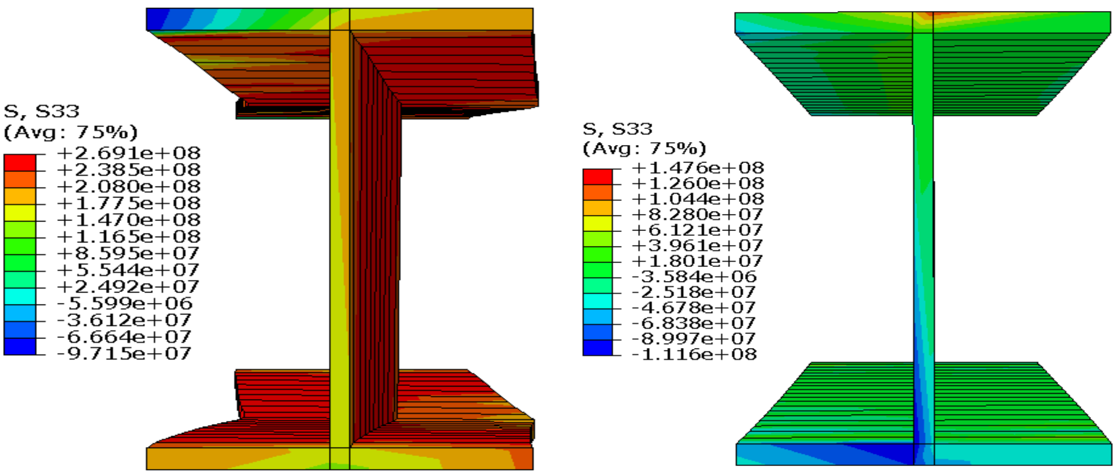


Figure 8. Thermal profile of fire-exposed face in analysis case E1 and S1

During cooling, the steel contracts, but the plastic strain formed during heating is permanent. Elastic strains recover, whereas plastic strains do not, resulting in self-equilibrating residual stress fields within the cross-section [40]. This residual stress occupies part of the section's yield capacity before any external load is applied [41]. Additionally, the reduced stiffness at elevated temperatures allows permanent cross-sectional distortion to develop under restraining forces [42]. After cooling, this distorted shape remains and behaves as an initial imperfection. Under post-fire loading, these imperfections magnify internal stresses through second-order (P-Δ) effects, causing the beam to reach yield earlier and thus reducing its bending moment capacity [43].

This mechanism is clearly illustrated in Figure 9, which shows the residual stress distribution and distorted shapes after cooling for Case E3 (Fixed Beam) and Case E4 (Simply Supported Beam). In Case E3, large residual stress is visible across the section, with a peak value of 269.1 MPa, which is close to the ambient yield strength of the steel. The figure also shows pronounced cross-sectional distortion in the fixed beam. All these indicate that a significant portion of the section capacity is already internally engaged and that geometric imperfections are present before external loading begins. Consequently, Case E3 exhibits 13.9% reduction in post-fire capacity. In contrast, Figure 9 shows that Case E4 develops much lower residual stresses, with a peak of 147.6 MPa, and experiences only minor cross-section distortion. Because the simply supported beam can expand and contract more freely during the thermal cycle, axial restraint forces are small, plastic strain accumulation is limited, and residual stress after cooling is significantly reduced [44]. The geometry of the beam remains close to its original shape, and second-order effects during reloading are minimal. As a result, the post-fire capacity reduction in Case E4 is only 0.36% compared to ambient temperature capacity.

The direct comparison between the stress patterns and deformed shape in Figure 9 therefore provides visual and quantitative confirmation that restrained induced residual stress and geometric distortions are dominant mechanisms controlling post-fire bending performance. Fixed supports thus transform thermal expansion into internal stresses and permanent deformations, while simple supports allow thermal strains to dissipate with minimal damage.



(a) **(b)**

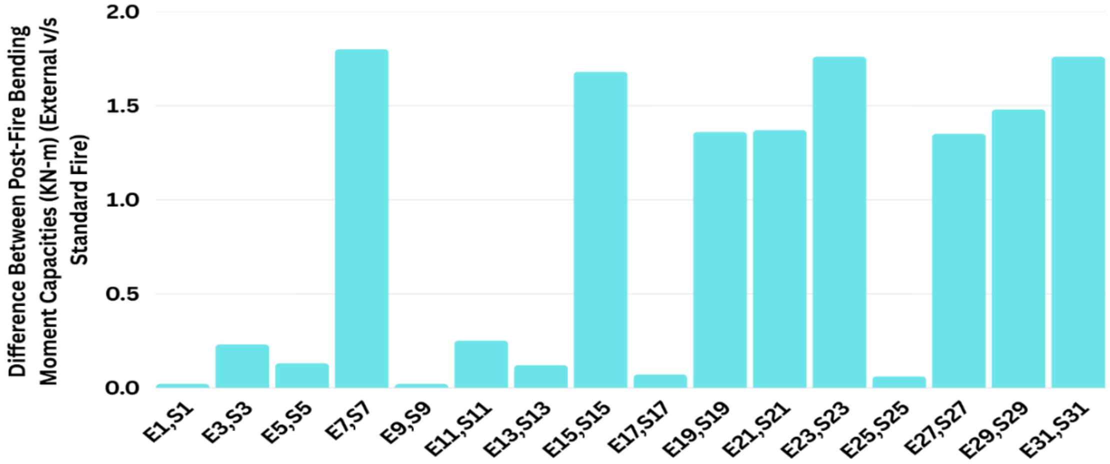
Figure 9. Residual stresses and distorted shape of **(a)** Fixed beam (Case: E3) and **(b)** Simply supported beam (Case: E4) after cooling

4.3. Effect of Fire Profile

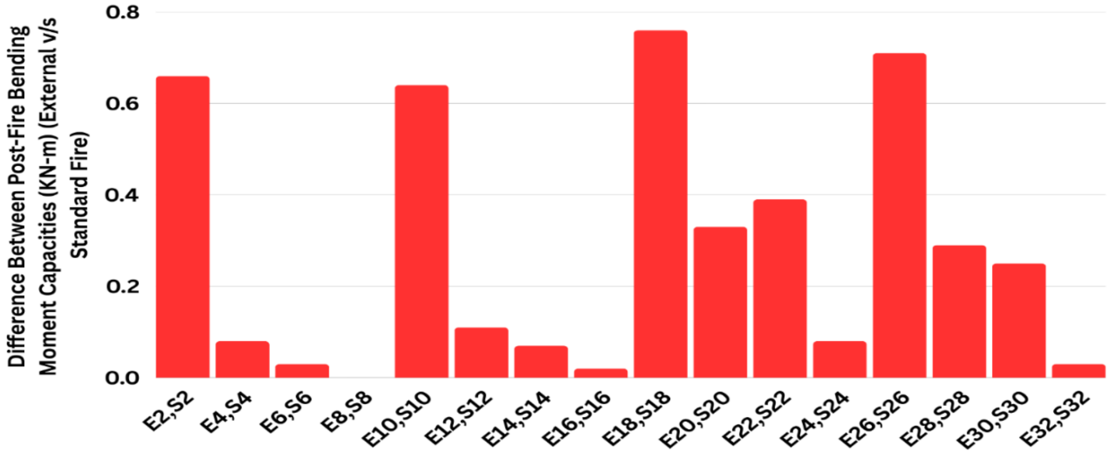
The comparison of post-fire bending moment capacities under the External and Standard Fire scenarios indicates only a marginal difference between the two fire profiles. The maximum variation observed is 1.76 KN-m for the fixed beam (Figure 10 (a)) and 0.71 KN-m for the simply supported beam (Figure 10 (b)). Although the peak gas temperature of the external fire (679.96°C) and the standard fire (841.8°C) differ significantly, the structural response during heating is governed not only by peak temperature but by the combined effects of heating rate, thermal gradient, and restraint level. Also, retrained steel members are particularly sensitive to the rate of stiffness degradation and the development of thermal gradients rather than peak temperature alone as advocated by the study [45]. In the present study, both fire curves exhibit a broadly similar rate of temperature rise during the early and intermediate heating stages. This leads to comparable through-depth thermal gradients within the section, especially during the temperature range where elastic modulus and yield strength experience rapid reduction (400-700°C) [31-32]. Since the development of restraint thermal expansion forces depends primarily on the mismatch between free thermal strain and support-imposed restraint, similar heating rates generate similar magnitudes of axial restraint forces and thermal curvature. Consequently, the accumulation of plastic strain during heating of the fixed beam remains nearly identical under both fire scenarios. Despite the higher peak temperature in the standard fire, the similarity in thermal gradient evolution results in comparable residual stress fields and permanent distortions after cooling. This explains why the post-fire bending capacity remains almost unchanged between the two fire profiles (Figure 10).

For a simply supported beam, the difference between fire profiles is very small (maximum 0.71 KN-m). This is consistent with prior findings that unrestrained members are less sensitive to fire curve shape because thermal expansion is largely accompanied by free elongation, as advocated by [16].

Moreover, in the present numerical framework, temperature-dependent material properties were restored to their ambient temperature values after cooling in order to isolate structural residual effects. The inclusion of different cooling durations was intended to examine whether the rate of thermal decay influences stress redistribution and geometric distortion during the contraction phase. Even when full-strength recovery is assumed, variations in cooling duration could potentially alter the evolution of restraint forces and residual stress patterns during temperature reductions as reported by [46]. However, the result indicates that under the gradual cooling conditions considered (15, 30, 45 and 60 minutes), thermal contractions occur stably, without introducing additional plastic deformation or significant stress redistribution. Consequently, the residual stress pattern established at the end of heating remains largely unchanged during cooling, and cooling duration does not influence the post-fire bending capacity within the investigated range.



(a)



(b)

Figure 10. Difference between bending moment capacities (External v/s Standard Fire) **(a)** Fixed beam, and **(b)** Simply supported beam

4.4. Effect of Fire Exposure

Across all analyses involving the fixed beam, the lowest reduction in post-fire bending capacity is observed when the beam is heated from two sides. When the exposure increases from one to two sides, the post-fire bending capacity drops by 9.52% (compared to one-sided exposure), after which it remains nearly constant for three and four-sided exposure (Figure 11). A similar trend of increasing residual capacity reduction with increasing exposure area has been reported by [47]. The observed behaviour is governed primarily by the thermo-mechanical history of the section during heating and cooling. Under one-sided heating, a step thermal gradient develops across the depth of the section (Figure 12). Although this induces curvature due to differential thermal expansion, the high-temperature region remains relatively localised. As a result, the extent of plastic strain accumulation and high temperature creep in the cross-section is limited to a smaller zone, leading to comparatively lower residual stress redistribution after cooling, as supported by [48]. Moreover, the non-uniform temperature distribution promotes thermal bowing rather than uniform axial expansion, which partially relieves restraint forces in the fixed-end condition through geometric deformation. Consequently, the residual stresses and cross-sectional distortion after cooling remain moderate, resulting in a relatively smaller reduction in post-fire bending capacity.

When the exposure increases from one to two sides, a significantly larger portion of the cross-section is subjected to elevated temperatures (Figure 12). This reduces the stiffness of both the flange and web regions simultaneously, increasing restrained thermal expansion at the supports. Because the beam ends are fixed, axial expansion is restrained, generating substantial compressive thermal forces during heating. Upon cooling, these compressive forces transform into tensile residual stresses due to inelastic strain accumulation and non-recoverable plastic deformation. The combined effect of higher plastic strain, large residual stress, and permanent cross-sectional distortion leads to a marked reduction in the post-fire bending capacity.

For three and four-sided exposure, the temperature distribution across the section becomes more uniform. This reduces thermal gradients and associated curvature effects. However, because a large portion of the section has already yielded during the two-sided exposure case, further increase in exposure does not proportionally increase residual stress magnitude or plastic strains. The coss-section reaches a saturation state of thermo-mechanical damage, beyond which additional uniform heating does not significantly worsen residual capacity. A similar type of behaviour was also reported by [15]. Therefore, the nearly constant post-fire bending capacity observed for three and four-sided heating (Figure 11) indicates that the dominant damage mechanism; plastic strain accumulation, restrained residual stress, and permanent distortion, have already reached critical levels under two-sided exposure.

In contrast, for simply supported beams, the influence of the number of exposed sides is negligible (Figure 11). This is primarily due to the absence of axial restraint. Without end restraint, the thermal expansion during heating is largely accommodated by free elongation, significantly reducing the development of thermal stresses. Consequently, plastic strain accumulation during heating remains limited. Since residual stresses are minimal, the post-fire bending capacity is governed mainly by geometric imperfections rather than thermo-mechanically induced damage. This explains why variations in exposure conditions do not produce a pronounced difference in residual capacity for simply supported beams, consistent with findings reported [49].

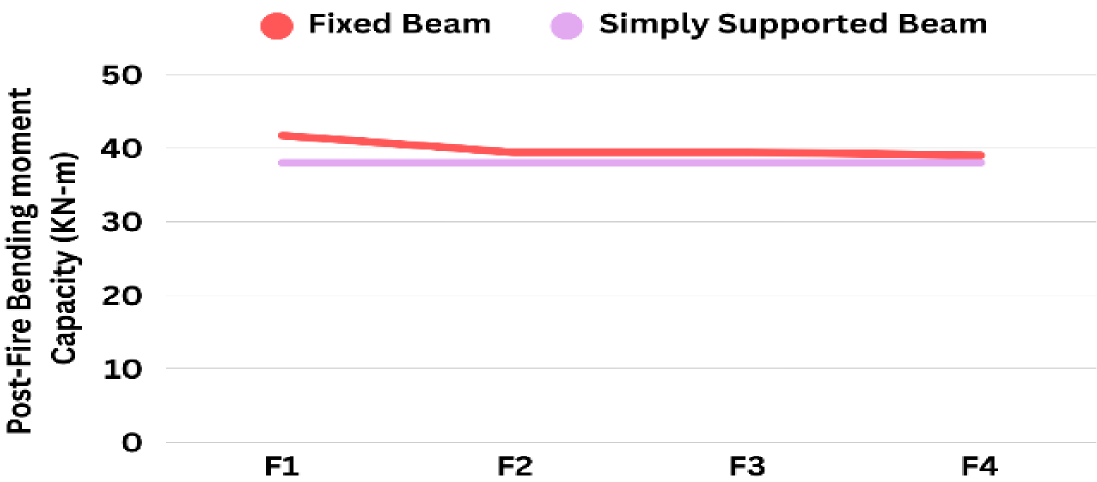


Figure 11. Effect of fire exposure sides on the post-fire bending capacity of the beam

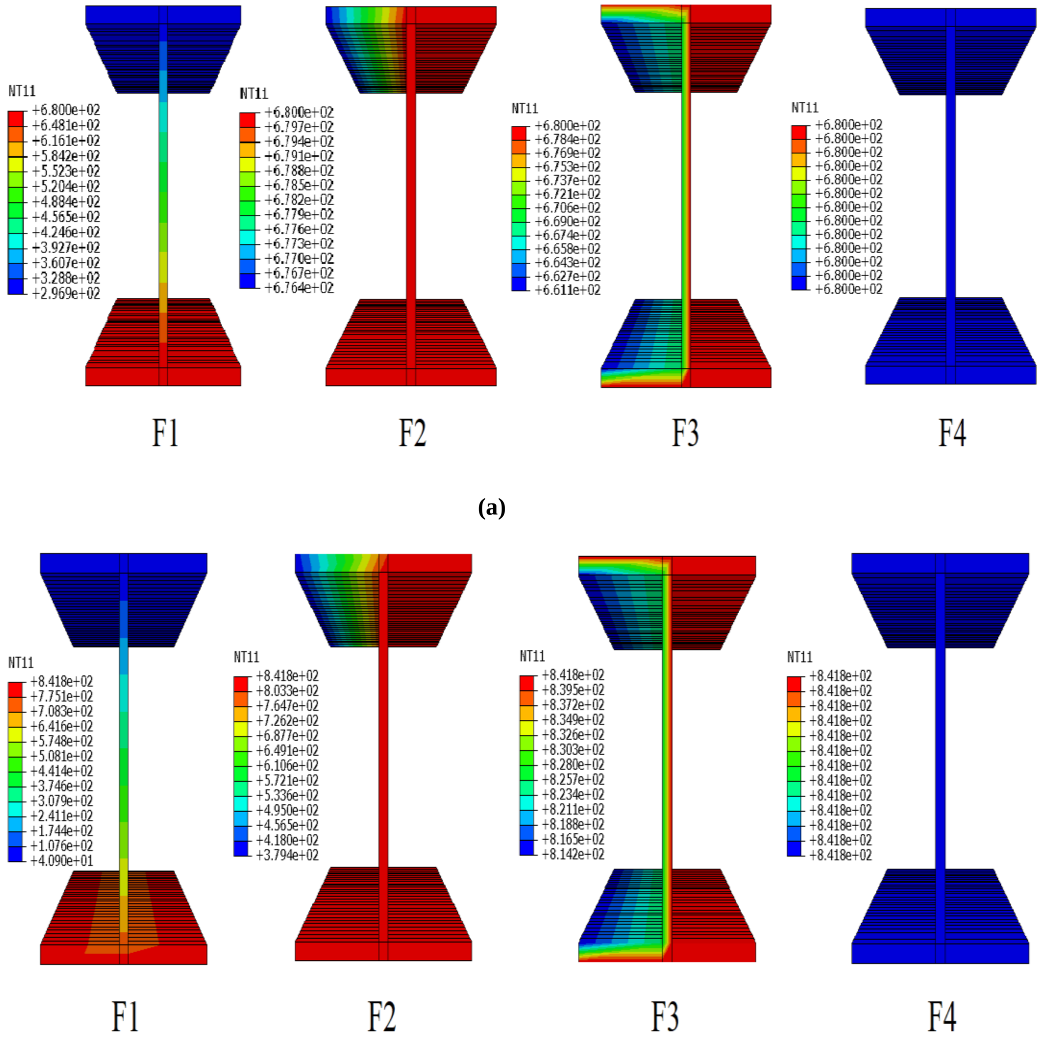


Figure 12. Thermal gradient developed in beam under different fire exposures (F1 to F4) in **(a)** External fire and **(b)** Standard fire

5. CONCLUSIONS

The study presented a methodology for assessing the bending moment capacity of steel beams after fire by incorporating combinations of various variables. The study yielded the following conclusions:

- For the investigated ISWB 150 section of 3 m span, end restraint was identified as the dominant parameter influencing the post-fire bending performance. The fixed beam consistently exhibited a higher reduction in capacity compared to the simply supported beam, demonstrating that restraint conditions significantly govern residual structural behaviour within the studied fire scenarios.
- The observed reduction in the post-fire capacity was observed even after full restoration of temperature-dependent material properties during cooling. This confirms that the capacity loss is primarily associated with irreversible thermo-mechanical effects developed during the heating phase, including residual stress and permanent cross-sectional distortion, rather than permanent material strength degradation alone.
- The fixed beam showed a reduction in bending capacity in the range of 8.8% to 14.7 %, while the simply supported beam showed a decrease of 0.05% to 2.1 %. Although the ranges obtained are specific to this study, they can serve as a baseline, helpful for quick decisions regarding the reuse or repair of the section after fire.
- The fire profile (external v/s standard) and cooling duration had a negligible effect on the post-fire bending capacity. In contrast, exposure sides and support conditions showed a considerable influence: fixed supports produce greater capacity reductions than simple supports, and two-sided exposure caused larger capacity reductions than one-sided exposure.

Scope for Future Work

- Different cross-sections or structural members can be studied to determine their residual load capacities after fire exposure.
- Different durations and profiles of heating and cooling phases can be adopted for a more detailed study.
- Effect of different modes of cooling: intermittent or continuous can be considered.
- The effect of microstructural changes and the regain of strength after cooling on the thickness of the fire protection coating can be determined.
- A machine learning algorithm can be developed for the quick estimation of post-fire load capacities of structural members.

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DECLARATION OF ETHICAL STANDARDS

The authors of this article declare that the materials and methods used in this study do not require ethical committee permission and/or legal-special permission.

AUTHORS' CONTRIBUTIONS

Kuldeep JOSHI: Conceptualization, Methodology, Finite element modelling, Writing-original draft, Review, Editing.

Archana BOHRA GUPTA: Conceptualization, Supervision, Review, Editing.

CONFLICT OF INTEREST

There is no conflict of interest in this study.

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