

KARADENİZ TECHNICAL UNIVERSITY
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

DEPARTMENT OF CIVIL ENGINEERING

**PROGRESSIVE COLLAPSE SIMULATION OF A BUILDING UNDER
MULTIPLE FIRE SCENARIOS USING CARBON STEEL, PROTECTED STEEL
AND STAINLESS-STEEL MATERIALS**

MASTER'S THESIS

Athraa Muttair Nuhair ALSAADI

SEPTEMBER 2025
TRABZON



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Athraa Muttair Nuhair ALSAADI

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DECLARATION

This research thesis, titled “Progressive Collapse Simulation of a Building Under Multiple Fire Scenarios Using Carbon Steel, Protected Steel, and Stainless-Steel Materials”, which I, Athraa Muttair Nuhair ALSAADI presented as a Master's Thesis, was originally written by me and fully supported by my advisor, Assoc. Prof. Murat GÜNAYDIN, that I have gathered the data presented therein myself ,that no results in this thesis have been fabricated, that I have fully documented the information I have obtained from other sources both in the text as well as the list of references, that I acted in compliance with scientific research and code of ethics during the research, and that no result from this thesis has been submitted/accepted for a degree at any other institution. In the event that the aforementioned declaration is violated, I accept full responsibility. 17/09/2025

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ÖZET

KARBON ÇELİK, KORUNMUŞ ÇELİK VE PASLANMAZ ÇELİK MALZEMEDEN İNŞA EDİLMİŞ BİR BİNANIN FARKLI YANGIN SENARYOLARI ETKİSİNDE AŞAMALI GÖÇME SİMÜLASYONU

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Birçok çelik bina yangın kaynaklı aşamalı göçmeye maruz kalmıştır. Bu kapsamda özellikle orta ve yüksek katlı çelik binaların farklı yangın senaryoları için aşamalı göçme simülasyonlarının irdelenmesi, risklerin belirlenmesi ve önlemlerin irdelenmesi önem arz etmektedir. Bu tez kapsamında; karbon çelik, korunmuş çelik, paslanmaz çelik ve hibrit (paslanmaz-karbon çelik) binaların farklı yangın senaryoları altındaki aşamalı göçme davranışları sayısal olarak incelenmiştir. Bu kapsamda, ABAQUS sonlu elemanlar paket programında aşamalı göçme analizleri yapılmıştır. Analizler yangının farklı bölgelerde meydana gelmesi, yangının sıçraması gibi parametrelere bağlı gerçekleştirilmiş ve irdelenmiştir. Çalışmada, her bir durum için sıcaklık profilleri, eleman eksenel kuvvetleri, yer değiştirmeler ve göçme dizileri detaylı izlenmiştir. Çalışma sonucunda; korumasız karbon çelik binaların yangın altında kısa sürede göçmeye karşı son derece hassas oldukları, korunmuş karbon çeliğin göçme süresini sınırlı ölçüde uzattığı, buna karşılık paslanmaz çelik ve hibrit sistemlerin yüksek sıcaklıklarda dahi taşıma kapasitelerini koruyarak çok daha yüksek yangın dayanımı ve göçme direnci gösterdikleri tespit edilmiştir.

Anahtar Kelimeler: ABAQUS, Östenitik Paslanmaz Çelik, Sonlu Elemanlar Analizi, Yangın , İntümesan Kaplama ,Aşamalı Göçme Analizi

Master's Thesis

SUMMARY

PROGRESSIVE COLLAPSE SIMULATION OF A BUILDING UNDER MULTIPLE FIRE SCENARIOS USING CARBON STEEL, PROTECTED STEEL AND STAINLESS-STEEL MATERIALS

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Many steel buildings have experienced fire-induced progressive collapse. In this context, investigating progressive collapse simulations for medium- and high-rise steel buildings under different fire scenarios, identifying risks, and evaluating preventive measures are of critical importance. In this thesis, the progressive collapse behavior of buildings constructed with carbon steel, protected carbon steel, stainless steel, and hybrid (stainless-carbon steel) systems was numerically examined under various fire scenarios. For this purpose, progressive collapse analyses were conducted using the ABAQUS finite element package. The analyses were carried out by considering parameters such as the location of the fire and traveling fire, and the results were systematically evaluated. For each case, temperature profiles, member axial forces, displacements, and collapse sequences were tracked in detail. The findings reveal that unprotected carbon steel buildings are highly vulnerable to rapid collapse under fire, protected carbon steel extends collapse time only to a limited degree, whereas stainless steel and hybrid systems retain their load-carrying capacity even at elevated temperatures and demonstrate significantly greater fire resistance and robustness against progressive collapse.

Key Words: ABAQUS, Austenitic stainless steel, Finite element analysis, Fire, Intumescent coating, Progressive collapse simulation

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LIST OF SYMBOLS AND ABBREVIATIONS

IC	: Intumescent coating
CS	: Carbon steel
SS	: Stainless steel
PCS	: Protected carbon steel.
FEM	: Finite Element Model.
$k_{y,\theta}$	: The reduction factor for the yield strength of steel with temperature dependent
f_y	: The steel yield strength at 20°C
$f_{y,\theta}$	: The effective yield strength of steel at elevated temperature
$k_{p,\theta}$	: Proportional-limit reduction factor of steel
$k_{E,\theta}$	: Elastic-modulus reduction factor of steel
θ	: The temperature
c_a	: The specific heat of steel
λ_a	: The thermal conductivity
c_p	: The specific heat of the fire protection material
χ_{fi}	: The reduction factor for flexural buckling in the fire design situation

1. GENERAL INFORMATION

1.1. Introduction

Progressive collapse is a disproportionate structural failure in which a localized deficiency propagates through the load-carrying system, ultimately resulting in partial or total collapse. Among the various initiating hazards, fire is particularly severe due to the accelerated degradation of strength and stiffness at elevated temperatures, especially in critical components such as columns and beam-column connections(EN 1993-1-2, 2005). This occurs particularly in steel structures. As known, Carbon-steel members may lose 50 % of their ambient-temperature yield strength by 550 °C. Historic conflagrations from the Great Fire of London in 1666, which destroyed approximately 70 000 homes, through the Great Chicago Fire(1871) and the Alfred P. Murrah Federal Building bombing and fire(1995) have repeatedly demonstrated the severe social, economic and regulatory consequences of fire-induced structural failure. More recent disasters, such as the Grenfell Tower façade fire (2017) and the 2023 Maui wildfires, confirm that such risks persist despite advancements in modern codes and construction materials. Moreover, three contemporary high-rise cases further illuminate the mechanisms by which fire can trigger a cascading loss of load paths:

The collapse of the World Trade Center Building 7 (WTC 7) on September 11, 2001 was a unique incident in structural engineering history, as it was the first known case of a tall steel frame building collapsing primarily due to uncontrolled fires(NIST, 2004) .The 47-storey building to the north of the main WTC complex was approximately 100 meters long, 43 meters wide and 190 meters high(Shyam-Sunder et al., 2008). The building was originally planned as a substation, but the final design called for a much larger structure that required a system of beams and trusses between the fifth and seventh floors to distribute the loads over a limited foundation area (NIST, 2005).

Some floors set on fire because of the accident and the falling debris from the neighboring towers. The sprinkler system installed in the World Trade Center 7 was, however, not working because the fire pumps that were electrically operated could only be operated manually, and hence they were unable to function. Since it depended on only one main water line and fire pumps that were electrically powered, the system had to be operated manually, therefore it was not working. Besides, the water pressure was low,

which made it easier for the fire to spread in the building (NIST, 2005). The eastern part of the building began to collapse after a critical column was damaged. The failure was due to the loss of lateral support when a girder at the 13th floor disconnected, thus the column's effective length was increased, and it buckled. This local failure extended both vertically and horizontally and thus the entire collapse within around eight seconds was the result (Shyam-Sunder et al., 2008).

The inquiry had uncovered many reasons why the building collapsed, such as unexpected thermal expansion at low temperatures, long-span floor systems that exacerbated the movement of the building, and connection designs that only allowed load transfer from gravity but not from thermally induced lateral forces. In addition, the whole structural system was not designed to have the capability to absorb and reduce the effects of progressive collapse in case of fire (Shyam-Sunder et al., 2008),(NIST, 2005). As illustrated in (Figure 1), the fire on the North Face of the World Trade Center was concentrated on Floors 7 and 12, with visible flames and smoke.

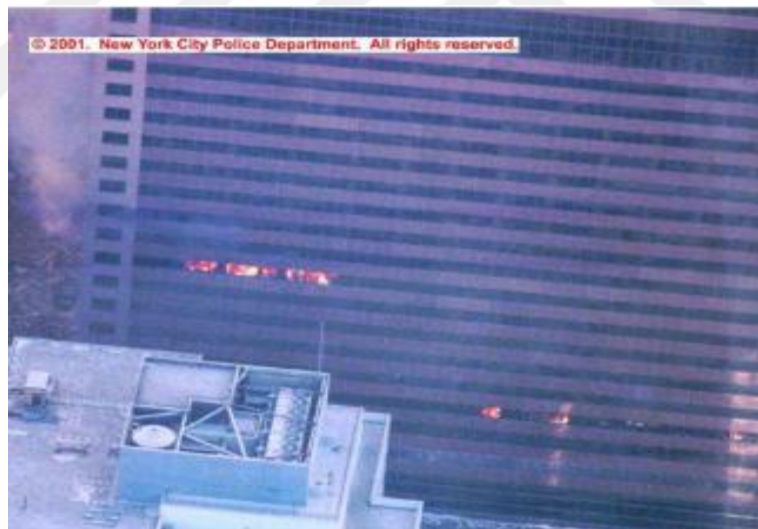


Figure 1. The World Trade Center fire (2001)(NIST, 2004)

On 12 February 2005, a fire broke out in the Windsor Building in Madrid, Spain. The fire started on the 21st floor of the 32-storey (106-meter) building at around 11 p.m. and quickly spread to the top floor by 1 a.m. the following morning. The flames reached the fourth floor by 9:00 and began to engulf the entire upper ten floors. The fire was not brought under control until around 2:00 p.m. and lasted between 18 and 20 hours. The footprint of the building was 40 x 25 meters. As shown in (Figure 2), the Windsor Building is depicted

both before and after the fire incident. The building was completed in 1979 to Spanish design standards of the 1970s and had a composite steel and reinforced concrete structure. These standards had limited fire safety requirements. At the time of the fire, the building was undergoing refurbishment, including the installation of fire suppression systems and fire protection for structural components. The refurbishment extended from the ground floor to the 17th floor, but the floors ≥ 18 not protected. The lack of fire protection resulted in significant structural damage to the upper eleven floors. The perimeter steel columns, including the external waffle slabs, collapsed almost completely. However, the reinforced concrete structure retained its strength and prevented total collapse. According to a study by (NIWA, 2006), the causes of the partial collapse were as follows:

- The steel columns near the fire buckled due to high temperatures.
- The axial loads on the buckled columns were redistributed to nearby structural elements.
- The fire intensified but the waffle slabs prevented structural collapse.
- The fire spread and the waffle slabs exceeded their load bearing capacity as support and finally collapsed.
- The floor collapse led to the failure of other floors and the waffle slabs were torn off at their connections to the central core. The mechanical floor between the 16th and 17th floors provided sufficient redundancy to prevent progressive collapse.



Figure 2. Windsor building fire (2005) (Luminex Inc, 2024)

Grenfell Tower is a 24-storey, 67 m-tall residential blocks completed in 1974 and refurbished in 2012-16 with ACM/PE rain screen cladding experienced a catastrophic façade fire on 14 June 2017 (Grenfell Tower Inquiry, 2019). Ignition in a fourth floor flat progressed to full-height external flame spread in approximately 20 minutes because the polyethylene core of the ACM cassettes softened and delaminated, allowing flames to exploit an unprotected ventilation cavity. As shown in (Figure 3) Grenfell Tower is depicted during the fire, with flames engulfing the external cladding system.

The absence of sprinklers, a common alarm, and continuous cavity barriers further accelerated fire growth and hampered control efforts. Although the reinforced-concrete frame remained standing, 72 occupants died, more than 70 were seriously injured, and the façade suffered near-total loss of integrity.

Large-scale façade tests conducted for the public inquiry show that ACM/PE panels lose over 90 % of their mechanical strength within the first 15 minutes of exposure, generating high external heat fluxes (600-700 °C) capable of inducing elastic-plastic buckling in unprotected steel members. These findings underscore the necessity of integrating façade-induced thermal loads and cavity-chimney effects into progressive-collapse simulations of steel-framed buildings under fire (Grenfell Tower Inquiry, 2024).



Figure 3. Grenfell Tower façade during post-fire inspection (Dan Woodland, 2024)

In recent years, the occurrence of major fire incidents and their devastating consequences have prompted many countries to develop or revise fire safety codes and

regulations. At the same time, they have encouraged the scientific community to conduct both numerical and experimental investigations into the behavior of structures under fire exposure, with a particular emphasis on progressive collapse mechanisms. This growing attention to fire-induced structural failures has led to a significant increase in research focused on understanding how structures respond to fire conditions. Within this context, several notable studies have been conducted, some of which are reviewed below.

During the last thirty years, experimental work and numerical modelling have transformed our understanding of how steel buildings behave in fire, especially with respect to progressive collapse. Full-scale furnace tests and laboratory studies have shown in detail how rising temperatures weaken steel, alter connection behavior, and drive distinct failure modes. Meanwhile, finite-element analyses carried out with ADAPTIC, SAFIR, VULCAN, ABAQUS, ANSYS, Open Sees, and MIDAS have explored key questions about critical steel temperatures, heat-induced strength loss, and the effect of boundary restraints. Foundational simulations by (Burgess & Plank, 1991); (Bailey, 1995) and (Huang et al., 2000) reproduced the celebrated Cardington fire tests and revealed that tensile membrane action allows composite floors to keep carrying loads even beyond 600 °C. (Yin & Wang, 2004) later examined large deflections in restrained beams and introduced refined strength-reduction factors now embedded in modern design codes. These combined experimental and numerical insights have led to the establishment of test protocols such as (ISO 834, 1999) and standards such as (BS 5950-8, 2003) and the Eurocodes. Yet day-to-day practice remains largely prescriptive: designers still tend to rely on passive insulation rather than assessing the fire performance of the whole structure. The accumulated evidence therefore provides a strong basis for moving toward genuinely performance-based fire design in steel buildings.

Rubert & Schaumann, (1986) investigated the performance of complete structural frameworks subjected simultaneously to fire, vertical and horizontal loading. The principal objective was to correlate failure temperatures of heated systems with ambient-temperature design parameters specifically the applied load factor and the global slenderness ratio. Several steel frames were examined, and two were later selected to validate the modelling methodology presented in Chapter 2.

The Broken Hill Proprietary (BHP) Research Laboratories in Melbourne conducted four -scale compartment tests in 1990 on a purpose-built corner bay representing a 41-storey office tower. Peak gas temperatures of 1228 °C and steel temperatures of 632 °C were recorded; mid-span beam deflection reached 120 mm yet largely recovered on cooling,

demonstrating the large beneficial thermal shielding provided by a conventional suspended ceiling (Steel, 1999). Four fire tests were carried out. A third test was designed to assess the fire resistance of the existing composite slab, while the other two tests were designed to assess the effectiveness of the current light hazard sprinkler system. The fourth test assessed the fire resistance of the exposed steel beams, considering the effect of the thermal shielding provided by a traditional suspended ceiling system by a simulated office fire. The steel temperature reached 632°C and the air temperature reached 1228°C at the height of the office fire. The steel temperatures of the insulated beams increased to 632°C. It was observed that most of the peak beam displacement detected at mid-span and measured at 120mm was recovered after the test. According to the results of the study, the fire resistance of a steel frame floor system during fire exposure could be significantly enhanced by the thermal shielding provided by a conventional suspended ceiling system.

As mentioned above, the Cardington Programme, which investigated the fire performance of steel buildings in the mid-1990s, involved the fire exposure of a full-scale, eight-story steel frame building at the Building Research Establishment's research facility in Cardington, UK (Bailey et al., 2006). Numerous steel joists in the composite floors of the steel building tested at Cardington were not fire-rated and would not have met the prescriptive fire protection standards set by the United States. The floor system and the entire structure could withstand significant fire exposure without collapsing, even in the absence of fire protection. The Cardington trials demonstrated that significant cost reductions in fire protection were possible without compromising the safety of steel structures under fire exposure. One of the key findings of the Cardington research was that the development of tension in the cables and membranes as a result of the large vertical displacements that typically occur in a fire is the primary behavioral factor that influences the ability of a floor system to survive a fire.

Lien et al., (2009) stated that they extended the Vector Form Intrinsic Finite-Element (VFIFE) method to capture the non-linear response of steel frames through both the heating and subsequent cooling phases of a fire. After validating the VFIFE model against published tests and analytical solutions for post-fire cooling, they applied temperature-dependent steel constitutive laws including strain-reversal effects and showed, through several illustrative examples, that the scheme can reliably track large-deflection behavior and strength recovery as temperatures fall, providing accurate prediction of frame performance in both fire stages.

Sun et al., (2012) embedded an explicit-dynamic module in the Vulcan finite-element code, enabling automatic switches between static and dynamic solution schemes so that analyses could ride through local instabilities and trace behavior up to and beyond fire-induced collapse. Working with validated 2-D frame models, they observed that unbraced frames with low gravity-load ratios and larger beams endure higher critical temperatures, whereas heavily loaded frames with slender beams fail sooner; introducing bracing curtailed column pull-in and vertical drift but left the global failure temperature largely unchanged. They therefore identified gravity-load ratio, beam size, and bracing configuration as the key levers for improving the fire robustness of multi-story steel frames.

Kilic & Selamet, (2013) were studied about the 49-storey moment-resisting steel tower and subjected it to several gravity-plus-fire scenarios, letting the fire front spread symmetrically or asymmetrically across a single floor. The uncoupled thermal-structural simulations showed that the pattern of fire spread-not its floor location-governs the frame's progressive-collapse mode, while confinement to one floor limits the influence of impact point. The study concluded that high-rise fire-robustness assessments must prioritize how flames traverse a story, because symmetrical versus asymmetrical spread can dictate whether collapse propagates globally or remains localized.

Agarwal & Varma, (2014) qualitatively assessed two alternative ten-story composite office buildings one braced by perimeter moment frames, the other by an RC-shear-wall core to isolate the role of interior gravity columns during corner-compartment fires. The finite-element simulations showed that gravity columns carry the highest utilization ratios, reach critical temperatures first and thus govern global stability; once a gravity column fails, axial load must be rerouted to neighbors through alternate paths such as catenary action. The study also demonstrated that providing slab reinforcement beyond the minimum shrinkage steel markedly improves the slab's flexural-tensile capacity, enabling more uniform load redistribution and preventing progressive collapse after gravity-column failure.

Sun et al., (2015) embedded a component-based connection element and an alternating static explicit-dynamic solver into the Vulcan program, enabling step-by-step tracking of local fractures developing in beam-to-column connections during fire exposure and tracing the progression of these failures toward frame-level collapse. Each connection is idealized as a series of "bolt-row" springs whose stiffness, strength, ductility, and fracture properties vary with temperature; when any spring fails, the solver automatically switches from static to dynamic analysis, allowing the model to continue through otherwise destabilizing

singularities. Validation against published tests showed close agreement in stiffness loss, fracture sequence, and beam detachment. Using a two-story and a multi-story exemplar, the authors observed that both tensile and compressive ductility within the connections raise the beams' fire resistance and that enhancing connection ductility is a practical way to prevent beam drop-off and the ensuing progressive collapse. It was concluded that the method, together with detailed component data, provides a reliable, performance-based tool for determining ductility requirements for steel joints exposed to fire.

B. Jiang et al., (2015) used an explicit-dynamic model that can keep running after local buckling to see how a planar steel moment frame reacts when one fire-heated column suddenly fails. Once mesh size and loading-rate effects were shown to be negligible, the validated model revealed three distinct collapse patterns: a cantilever-beam mechanism, a pull-in-force mechanism, and a newly identified high-load-ratio member-failure mechanism that dominated when heavily loaded interior columns were lost. Checking the first two mechanisms is recommended for fires affecting edge columns, while the latter two apply when fires reach interior columns. The most dangerous scenarios arise when first-story or top-storey edge columns are exposed.

B. Jiang et al., (2017) reported, on the basis of three full-scale frame tests that captured the dynamic buckling of a heated column, that their calibrated finite-element models could accurately reproduce progressive-collapse behavior of steel moment frames exposed to a localized fire. It was observed that varying the viscous-damping ratio from 0 % to 10 % had virtually no influence on collapse resistance, whereas strain-rate effects markedly reduced the capacity of the heated column and, by extension, the frame whenever dynamic buckling governed. The strain-rate sensitivity was confined to the fire-exposed members, being negligible elsewhere in the structure. Having validated the models against the experiments, the authors concluded that the tool is suitable for parametric studies aimed at developing robust design guidance for steel frames under localized-fire scenarios.

Using a validated 3-D finite-element model, (J. Jiang & Li, 2017) explored how compartment-fire location (corner, edge, interior) and three fire-protection levels (low, medium, high) affect disproportionate-collapse behavior in steel-framed buildings. Simulations showed that frames seldom fail immediately after a column buckles; instead, floor-slab sagging drives lateral drift in adjacent cool columns, providing a 60-minute grace period beyond the member-level fire rating. Medium fire protection (2-hour rating for columns) preserved stability, while low protection led to collapse. When comparing

modelling approaches, the study found that 2-D frames underestimate load redistribution, especially along short spans, resulting in overly conservative collapse predictions. To enhance robustness, the study recommended raising the perimeter-column rating to at least 2 hours and insulating slabs to delay global failure.

Memari & Mahmoud, (2018) created multi-resolution finite-element models in which detailed 3-D solid representations of reduced-beam-section (RBS) connections were seamlessly embedded within line-element SAC moment frames designed for Los Angeles seismic demands. Simulations of several localized-fire scenarios showed that both beams and columns violated the Appendix 4 temperature–capacity limits of AISC 360-16, while the RBS region experienced flange-and-web buckling that imposed exceptionally high axial-moment demands at the beam-to-column interface. Based on these findings, the study argued that current structural-fire design must explicitly account for the severe local demands generated in RBS connections and cannot rely on line-element models alone to ensure robustness.

Qin & Mahmoud, (2019) developed a 3-D finite-element model of a six-story composite steel building (moment, gravity, and concentrically braced frames) to trace collapse under ISO 834 and parametric fire curves applied as a corner-compartment fire versus an entire first-floor fire. The analyses revealed twist-induced failure with lateral sway for the corner scenario, but a progressive global-vertical collapse when the whole floor was engulfed, confirming that collapse mode is highly sensitive to fire location and extent. Overall, the study demonstrates that advanced 3-D FE modelling can faithfully capture member-connection behavior up to collapse and underscores the need to represent realistic fire patterns in performance-based design

Suwondo et al., (2019) showed, through 3-D nonlinear FE analysis, that gravity load shed by a fire-weakened column redistributes along two perpendicular floor directions behavior that 2-D frames cannot capture. Residual earthquake drift satisfying the Collapse-Prevention limit had virtually no effect on either the load-transfer path or the overall fire resistance of the composite frame. Building survival became highly sensitive once the fire front was allowed to travel between compartments; the spatial progression and inter-zone delay of the travelling fire largely governed the onset of progressive collapse.

Shan et al., (2019) studied the influence of masonry infill walls on the collapse behavior of steel frames subjected to fire scenarios. The masonry panels provided alternative load paths that drastically redistributed axial forces, offered lateral resistance during edge-

bay fires, and simultaneously lowered the buckling temperature while increasing the ultimate collapse temperature of the heated columns.

Ren, (2020) reported a numerical study using 3-D finite-element model of steel frames, adding fire protection to beams had little influence on the heated columns because the simulated fire-spread phase was brief. The seven travelling-fire scenarios were detailed. It was found that the collapse was governed mainly by where and how the fire front moved: when flames originated in corner or long-edge compartments and spread horizontally, the frame failed, whereas other layouts survived. A stable catenary action in floor beams lengthened collapse time and confined damage, but this transferred two-way tensile forces to the corner column, making its lateral stiffness a critical design parameter. In un-insulated frames, the most dangerous moment coincided with the peak gas temperature in the source compartment, since column temperature and axial load maxima occurred simultaneously. Vertical fire spread had less effect on columns than horizontal spread, yet it markedly reduced beam fire-resistance. As a result, it was highlighted that designers must focus on spread range, duration, and peak temperature when selecting the most adverse horizontal-spread scenario and should ensure that corner columns and beam catenary capacity are adequate for those conditions.

Based on simulations of a three-dimensional gravity frame, Jian Jiang et al., (2020) found that a building's survival hinges on the duration of the heating stage: the model withstood a brief "short-hot" flash-fire but collapsed under milder or "long-cool" curves, often during cooling. Collapse time could be conservatively estimated when column steel reached approximately 550 °C. Raising the fire-resistance rating (FRR) of members delayed failure under slow fires but sometimes merely shifted it into the cooling phase for faster-moving fire fronts. Tensile-membrane forces in heated slabs were resisted either by a perimeter "ring" or by yielding lines running to edge columns. To exploit these load paths, the study recommended (i) full-story hat bracing at roof level, (ii) combined perimeter and internal vertical bracing, and (iii) beam-to-column joints detailed for tensile forces up to the beam's yield strength and for half-yield compression during cooling.

J. Jiang et al., (2021) extended a three-dimensional gravity-frame model in Open Sees to examine how a travelling fire across a floor alters the failure mechanisms of a steel building. By varying the heating curve ("short-hot," "mild," and "long-cool") and fire-front speed (2.5-10 mm s⁻¹), the study found that the duration of the heating phase is the critical trigger: a slow-burning "long-cool" curve combined with a fast-moving front can lead to

global collapse, even during the cooling stage. Added fire protection or horizontal/vertical bracing prevented collapse under slow fires but often only delayed failure to the cooling phase under faster-moving fires; even a heavily protected frame collapsed after approximately 130 minutes—well beyond the rating of its individual members. To ensure structural safety, the study recommended designing for a 3-hour rating under “long-cool” travelling fires and detailing beam-to-column connections to resist tensile forces equal to the beam’s full yield strength and compressive forces equal to half that value.

Venkatachari & Kodur, (2021) developed an ABAQUS system-level model for a ten-story braced steel frame that explicitly tracks temperature-induced transient creep strains during fire. Using the validated model, a parametric study was conducted by varying (i) peak fire temperature, (ii) fire-exposure duration, and (iii) the number of simultaneously burning compartments or floors. Results showed that transient creep governs the onset of instability under severe fires: when peak temperatures exceed 800 °C and burn for two hours or more, significant creep strains accumulate, triggering frame buckling much earlier than conventional analyses predict. The threat becomes more severe when multiple compartments on a single story ignite simultaneously, as high axial stresses coupled with prolonged heating amplify creep deformations. The study concluded that design checks for progressive collapse must incorporate explicit transient-creep modelling, particularly for scenarios involving severe, long-duration, or widespread fires.

According to Nan et al., (2022), a 3-D LS-DYNA model of a steel–composite office floor slice was created and validated against a Veselí travelling-fire test, followed by 32 parametric simulations to investigate how design decisions influence structural response under travelling fires. The analysis revealed that relying solely on temperature or deflection limits for members is insufficient for safety, as failure is governed by steel-stress utilization as the fire front progresses. Fire-spread rate and fuel-load density emerged as the primary factors determining the most critical scenarios, rendering the inverse-opening factor negligible. A slow-moving fire front (0.5 mm/s) combined with higher fire-resistance ratings (FRR) can mitigate cooling-phase failures, whereas faster fronts (≥ 2.5 mm/s) may still induce collapse despite increased FRR. The study further showed that slab reinforcement ratio has a greater influence on performance than slab thickness. Moreover, a full 3-D model can capture internal-force reversals and collapse triggers that are not evident in 2-D models, which tend to overestimate strength by more than 80%.

Kültür et al., (2022) numerically compared two four-story buildings with identical plans but framed respectively in reinforced concrete and structural steel, subjected to a compartment fire that heated a corner bay and a mid-bay from 25 °C to 1,000 °C in 100 °C increments. ABAQUS simulations were used to track column forces, critical-temperature times, and plastic-hinge formation. Results showed that the reinforced concrete frame maintained its structural integrity throughout the heating history, while the steel frame reached its critical temperature and collapsed after approximately 60 minutes.

Shan & Pan, (2022) used finite-element analysis to examine a six-story steel modular building under various fire scenarios, focusing on global deformation, load transfer, and collapse mechanisms. Simulations indicated that a structure designed according to Hong Kong codes can withstand a single-module fire but fails when a travelling fire spreads through adjacent modules. In such cases, overturning actions, bracing effects, and the system of paired floor beams become the primary load-carrying mechanisms. Damage was observed to propagate progressively from columns nearest the fire compartment to those farther away, with collapse resistance proving highly sensitive to the gravity-load ratio and column cross-section size. Based on these findings, a practical, accuracy-checked design method was proposed to help engineers identify the most vulnerable members and predict the final collapse state of modular frames under fire.

Shakeri et al., (2023) conducted finite-element simulations of 3, 7, and 12-story intermediate moment-resisting frames (IMRFs) to assess fire-induced column-loss scenarios. Frames proportioned for high-seismic demands and consequently carrying low gravity-load ratios remained robust against all examined failure scenarios. However, increasing the gravity-load ratio—typical in low-seismic regions—sharply raised the likelihood of progressive collapse, particularly in low-rise frames and in cases where failures were triggered in the upper stories, with the top floor being the most critical. First-story column failures were found to be more hazardous in low-rise than in high-rise frames. A previously undocumented “combined” global failure mode was also identified, involving simultaneous catenary and compression–flexure actions. These findings emphasize the importance of explicitly considering gravity-load ratio, frame height, and the position of heated columns in the fire-robustness design of IMRFs.

Venkatachari & Kodur, (2023) conducted a parametric finite-element study on a 10-story braced steel frame, systematically varying six factors: fire-intensity curve, horizontal/vertical fire spread, alternate load-path activation, high-temperature transient

creep, temperature-induced local instability, and nonlinear analysis regime, to isolate each factor's influence on collapse behavior. Results demonstrated that more severe or widespread fires significantly accelerate progressive collapse, while neglecting transient creep leads to non-conservative (delayed) predictions of failure time. At elevated temperatures, upper-story columns may transition from non-slender to slender classification, triggering early local buckling. These findings highlight the necessity for performance-based fire design to explicitly account for realistic fire growth, transient creep strains, and temperature-dependent sectional slenderness.

Qi et al., (2024) conducted an experimental fire test on a two-story unprotected steel frame building to investigate fire-induced collapse behavior. This test, which continued until total collapse, provided important information about the collapse process as a whole. The temperature distribution and structural response were measured and plotted in detail. According to the test results, the frame collapsed after 82 minutes of heating at an average component temperature of about 900 °C. This time it was 215.4% longer than for a single frame column. Damaged slab-beam connections and significant rotation in beam-column connections were the causes of the final collapse. A sink collapse mode, significant membrane action in the slabs and significant horizontal displacement were among the notable findings. Although the column buckled after approximately 40 minutes, the structural integrity was not compromised. For displacements of less than 20 mm, a 3D numerical technique using explicit dynamic analysis was presented and confirmed by the experimental results. Three possible collapse modes for steel frame structures are identified by numerical analysis considering different fire scenarios: the flip collapse mode from edge bay fire, the sink collapse mode from edge bay fire and the center bay fire. Compared to the flip collapse mode, which requires more precise conditions, the sink collapse mode is more likely to occur. The collapse process, which includes three separate stages: thermal expansion, column buckling and collapse stage, is the same for multi-story steel frame buildings as for the tested frame.

1.2. Objectives and Scope of Research

Building on the research gaps highlighted in the preceding review namely the limited quantitative data for mid-rise steel building subjected to standard fires and the scarcity of comparative studies across alternative steel solutions the present investigation seeks to advance performance-based fire design for steel buildings. The primary objective is to assess the vulnerability of a six-story office building to progressive collapse under multiple fire scenarios, using ISO 834 standard fire exposure. Additionally, the study aims to evaluate the relative performance of carbon steel, coated steel, stainless steel, and hybrid structural systems. To achieve this aim, five inter-related objectives are formulated:

- Develop and validate a detailed three-dimensional, coupled thermo-mechanical Finite Element (FE) model of the six-story benchmark frame reported by Shan and Li (2021). The model's accuracy will be confirmed by replicating their published bare-frame results. Validation will be considered successful if axial column displacements and mid-span beam deflections deviate by no more than $\pm 10\%$ from Shan and Li's numerical benchmarks.
- Conduct a comparative behavioral assessment to determine the critical steel temperatures, axial forces, and displacements for four material configurations under eight ISO 834 fire scenarios: (a) unprotected carbon steel (CS), (b) protected carbon steel with a 0.5mm intumescent coating (PCS), (c) austenitic stainless steel (SS), and (d) hybrid CS–SS columns.
- Identify the dominant failure mechanisms and benchmark the fire resistance of each configuration by determining collapse time and primary failure modes (e.g., column buckling, catenary fracture, progressive floor failure) under each fire scenario, while using utilization factors to indicate the onset of element-level failure.
- Evaluate the effectiveness of the 0.5 mm intumescent coating by comparing temperature histories and collapse times between configurations (a) unprotected carbon steel and (b) coated carbon steel.

These objectives collectively translate insights from three decades of experimental and numerical fire research into a focused evaluation of mid-rise steel buildings robustness, offering a pathway toward holistic, performance-oriented fire safety design.

To achieve these objectives, the study will follow a sequence of integrated research tasks. Three-dimensional, coupled thermo-mechanical FE models will be developed using ABAQUS (Dassault Systèmes, 2025). The six-story benchmark building will be analyzed under both time-to-failure conditions and multiple ISO 834 fire scenarios. Temperature-dependent material properties for carbon steel, austenitic stainless steel, and the intumescent coating will be defined in accordance with EN 1993-1-2 (2005), Inox (2007), and EN 13381-8 (2013), respectively.

1.3. Steel Behavior at Fire

When carbon steel is heated during a fire it passes through four successive phases:

- temperature rise
- deformation
- loss of stability and load-bearing capacity
- and destruction

Quantitative data from (EN 1993-1-2, 2005) show that the yield-strength reduction factor $k_{y,\theta}$ falls to 0.5 at about 600 °C and to 0.2 by 700 °C, while the elastic modulus plunges even faster; at 700 °C only 13 % of its ambient value remains. These losses directly precipitate the large deflections observed at approximately 300 °C and the subsequent rapid degradation of stiffness between 500-650 °C.

In addition to basic strength degradation, two additional phenomena further accelerate structural failure:

- Transient thermal creep inelastic strains, which accumulate under sustained loading once steel exceeds 450 °C, have been shown-through parametric studies of multi-story frames-to trigger global instability earlier than predicted by conventional temperature-dependent plasticity models (Venkatachari & Kodur, 2021).
- Connection degradation becomes critical above 400 °C. Experimental studies on welds and high-strength bolts have reported strength reductions to 50% for weld metal and as low as 40% for Grade 10.9 bolts compared to ambient values-

compromising joint integrity before adjacent members reach their critical temperatures (Rezaeian, Keshavarz, et al., 2020)(Rezaeian, Shafiei, et al., 2020).

Accordingly, this study investigates methods to retard the thermal exposure of structural steel and decelerate the degradation of its mechanical performance during fire.

1.4. Fire Simulation

Engineers and researchers use fire simulation to gain deeper insight into how fires spread and impact buildings and infrastructure. By leveraging advanced simulation tools, fire behavior can be analyzed in virtual environments that account for critical variables such as temperature, airflow, fuel distribution, and material properties. These simulations help evaluate how fire affects structural components, enabling the development of safer, more fire-resistant building designs.

Fire is caused by a chemical process known as combustion, in which fuel is converted into heat energy and light. According to (Walls & Zweig, 2017) the fire triangle (Figure 4) illustrates that three elements-fuel, oxygen, and an ignition source-are essential for combustion to occur. When these components are present in the right proportions and under suitable conditions, a fire can ignite, spread rapidly, and become increasingly difficult to control. Fires can cause extensive damage to buildings and infrastructure, as various construction materials respond differently to heat. Among them, steel is particularly vulnerable to high temperatures, making it a critical focus in fire safety design.

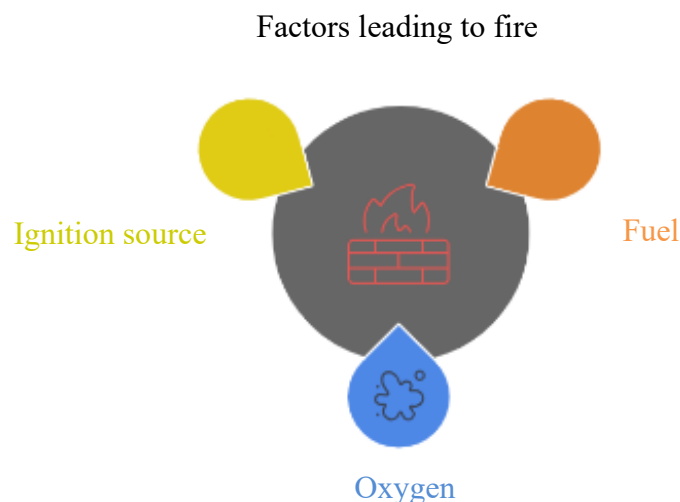


Figure 4. The fire triangle modified to suit informal settlement dwellings

(Walls & Zweig, 2017)

When steel used in structural frameworks is exposed to the high temperatures generated by fire, it undergoes both physical and chemical changes. Its mechanical properties-such as strength and stiffness-begin to deteriorate significantly. At elevated temperatures, steel can lose more than half of its original strength, severely compromising its ability to support structural loads. This degradation increases the risk of partial or complete structural collapse during a fire.

Due to the severe risks fire poses to structural integrity and public safety, engineers and researchers continue to develop advanced techniques for fire resistance and mitigation. Modern approaches include the application of fire-resistant coatings to enhance the heat tolerance of steel structural elements. Sprinkler systems are also widely implemented to suppress fires during their early stages. Another effective method involves thermal barrier systems, which insulate structural components by reducing direct heat transfer through protective layers. Additionally, sophisticated computer simulation tools are now used to model fire development and assess its impact on various structural materials. These simulations play a crucial role in designing safer, more fire-resilient buildings. The mechanism of fire development and its effect on steel elements is illustrated in Figure 5.

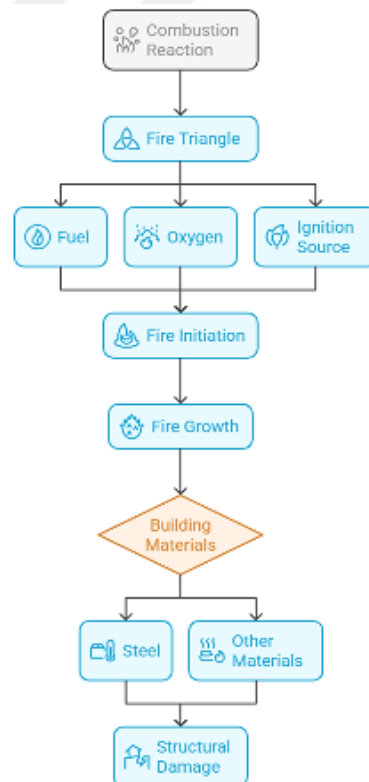


Figure 5. Flowchart of fire dynamics leading to material-specific structural damage

1.5. Standard Fire Curves

Standard time-temperature fire curves form the basis of fire resistance testing by establishing benchmark furnace conditions. The most widely used curves include ASTM E119 in the United States, ISO 834 internationally, and the Eurocode parametric curve in Europe. Both ASTM E119 (ASTM, 2020) and ISO 834 (ISO 834, 1999) define continuously rising temperature profiles that simulate a fully developed cellulosic fire, providing a standardized framework for evaluating the fire resistance of structural elements such as walls, beams, columns, and floor assemblies. In contrast, the Eurocode parametric curve (EN 1991-1-2, 2002) incorporates both heating and cooling phases, offering a more realistic representation of compartment fires. This enables performance-based assessments of residual structural capacity during the cooling stage. Detailed derivations of the ISO 834 and Eurocode parametric equations, along with their implementation in the current numerical model, are provided in Chapter 2.

1.6. The Principles of Heat Transmission

Heat transfer is the conversion of a temperature difference into thermal energy, or the movement of heat. This movement can occur in three main ways: (i) conduction, (ii) convection and (iii) radiation (see Figure 6). Each of these processes is governed by different physics and is based on specific mathematical models (Bergman et al., 2011).

Conduction is the transfer of heat through a solid or stationary fluid by molecular vibration and electron movement. Fourier's law of conduction is the method that compares the process of heat transfer to a material with a negative gradient of temperature and the area through which the heat flow passes. Materials with high conductivity, such as metals, transfer heat effectively, while insulators, such as foam, do so minimally.

Convection refers to the process of heat transfer between a surface and a moving fluid (either a liquid or a gas). Convection involves both conduction within the fluid and the motion of the fluid. It can be divided into natural convection, which is caused by differences in density in the fluid (usually temperature), and forced convection, where a fan or pump creates a more significant movement of the fluid.

Radiation differs from conduction and convection in that it does not require a medium for heat transfer. It involves the emission of heat through electromagnetic waves, primarily

in the infrared spectrum, which is also associated with light. All objects emit thermal radiation in proportion to their temperature.

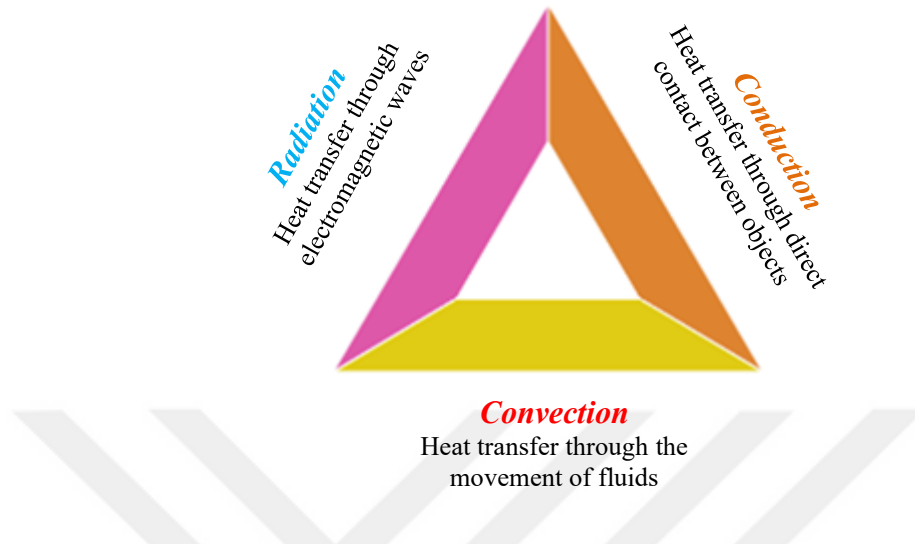


Figure 6. Conduction, convection and radiation

In real-world conditions, heat transfer typically occurs through a combination of conduction, convection, and radiation. For instance, in building environments, heat may be radiated from sun-exposed surfaces, convected through air movement, and conducted through walls and other solid materials.

When radiant heat encounters a steel plate coated with an intumescent material, the outer layer absorbs the infrared energy and rapidly swells to form a thick, foam-like char. This char possesses extremely low thermal conductivity. As a result, most of the heat attempting to pass through is impeded and can only proceed via slow conduction. A portion of the incident energy is reflected into the surroundings as re-radiated heat, while a smaller fraction is carried away by a thin layer of rising hot air.

Due to the porous and insulating nature of the char, only a small fraction of the original heat flux reaches the steel surface. Consequently, the steel heats up more slowly and remains below critical temperatures where mechanical strength begins to degrade. This thermal delay provides the structure with crucial additional time before any loss of load-bearing capacity occurs (see Figure 7). Understanding the interaction between these heat transfer mechanisms is essential for accurate thermal analysis in engineering and fire safety applications.

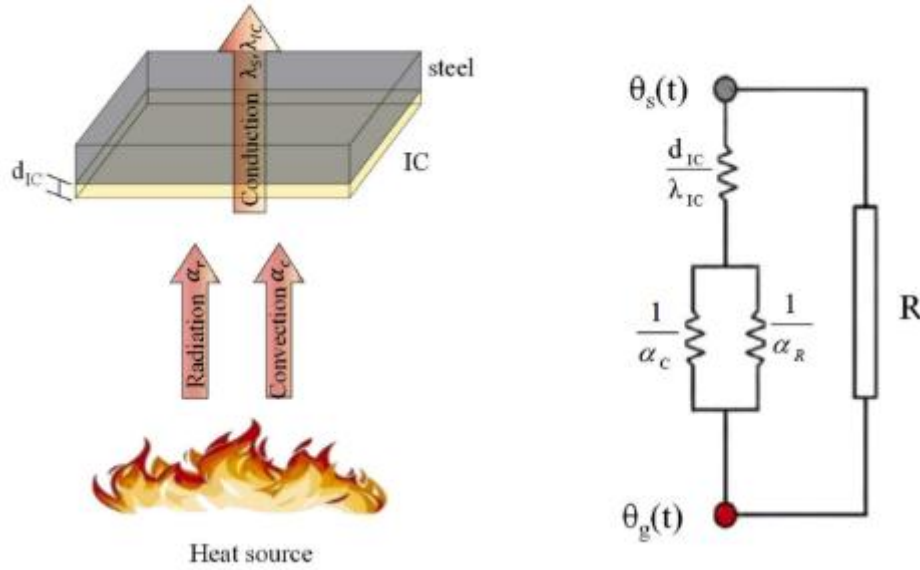


Figure 7. Heat transfer model for steel protected by IC (de Silva et al., 2020)

1.7. Passive Fire Protection of Steel with Intumescent Coatings

In modern structural fire engineering, safety measures are grouped into three complementary categories: passive fire protection (PFP), active fire protection (AFP), and organizational fire-safety management. PFP systems such as fire-rated walls, firestops, and thermally reactive coatings function without external power to limit heat transfer and contain flames, thereby preserving the load-bearing capacity of structural members during fire exposure (Hurley et al., 2016). AFP detects and suppresses a blaze through alarms, sprinklers, or gaseous extinguishing systems, while organizational measures (evacuation planning, staff training, and scheduled maintenance) ensure that both passive and active installations perform as intended.

Among PFP solutions, intumescent thin-film coatings have become a preferred option for exposed or architecturally complex steelwork. When the steel surface temperature exceeds roughly 200-250 °C, the coating undergoes a chemical reaction and can expand up to fifty times its original thickness, forming a microporous carbonaceous char that insulates the substrate and delays the steel from reaching the critical failure temperature of about 550°C (Soler Palau, R.; Matthew, 2025). Compared with heavier fireproofing methods vermiculite sprays, gypsum board encasement, or concrete jackets intumescent paints add minimal dead load, preserve architectural aesthetics, and can be applied to intricate

geometries with relative ease, making them attractive for both new construction and retrofit projects.

For these reasons, the present study adopts a thin-film intumescent coating as the primary passive fire protection system for the multi-story steel building analyzed under performance-based design scenarios. The selection and verification of the coating are based on (EN 13381-8, 2013) the European reference standard for qualifying reactive fire protection systems on structural steel. The standard's furnace testing procedures, time-temperature curve, and thickness-to-rating tables are integrated into the numerical model to simulate the thermal and structural response of coated members. This approach ensures that the investigation aligns with current best practices in structural fire engineering.

(J. Jiang & Li, 2018) emphasized that enhanced passive fire protection can delay progressive collapse by at least 60 minutes; however, they did not quantify the thermal properties of the coatings used. As a result, subsequent experimental and numerical studies have sought to isolate and characterize these thermal parameters. (Eremina & Korolchenko, 2020) identified a link between binder chemistry, phosphate grade, and the expansion efficiency of intumescent coatings, while Wang et al. (2013) demonstrated that aging reduces char thickness and increases its thermal conductivity.

A series of furnace test campaigns conducted by (Li et al., 2016), (Cirpici et al., 2016; de Silva et al., 2019, 2020), and (Zhang et al., 2013) provided several key findings: *(i)* ultra-thin coatings exhibit non-uniform swelling, *(ii)* effective thermal conductivity increases with both heating rate and section factor, and *(iii)* temperature-averaged constant conductivity values can replicate plate temperature profiles within $\pm 10\%$. While these studies offer valuable correlations for heat transfer behavior, they stop short of evaluating how such thermal characteristics influence global collapse mechanisms in multi-story steel frames.

The present thesis addresses this gap by integrating experimentally derived laws for thermal conductivity and expansion-applicable to both bare and protected carbon steel-into a coupled 3-D progressive collapse model. This model is then used to compare the fire performance of a conventional carbon steel frame with that of an equivalent structure constructed from austenitic stainless steel.

1.8. Stainless Steel in Fire

Stainless steel has emerged as a high-value construction material because it combines mechanical robustness with exceptional durability and corrosion resistance; unlike carbon steel, its protective chromium-rich oxide film virtually eliminates routine maintenance and confers a service life that can exceed a century in aggressive environments (Waller & Cochrane, 1990). In addition, stainless steel is fully recyclable without loss of properties, offering a closed-loop life cycle and high residual value at decommissioning (Unified Alloys, n.d.). Architects also prize its aesthetic finish, while engineers benefit from its favorable strength-to-weight ratio, which can translate into lighter primary members and reduced foundation loads (American Stainless & Supply, 2025). Fire performance provides a further and often underestimated advantage. Austenitic and ferritic grades retain both strength and stiffness to markedly higher temperatures than structural carbon steel; critical load-bearing capacity is typically maintained beyond 600 °C, whereas equivalent carbon-steel sections experience severe softening at 550 °C. (Sakumoto et al., 1996) substantiated. This behavior was investigated through a comprehensive programme of elevated-temperature tensile, modulus, and creep tests, from which constitutive equations were developed to support fire-safe structural design. The results revealed significant post-yield work-hardening in the stress–strain response of stainless steel—a characteristic identified as fundamental to the superior fire resistance of stainless-steel frames. In addition, the researchers quantified thermal expansion and thermal conductivity, underscoring the importance of incorporating these parameters into structural fire models for accurate performance prediction.

Taken together, the corrosion resistance, recyclability, aesthetic versatility, and—critically—the enhanced high-temperature strength retention demonstrated by (Sakumoto et al., 1996) position stainless steel as a highly viable alternative to carbon steel. Its suitability is particularly compelling for performance-based structural applications where long-term durability and inherent fire resilience are of primary importance.

Gardner & Baddoo, (2006) and Gardner, (2007) demonstrated through a series of complementary experimental, numerical, and analytical investigations that unprotected austenitic and ferritic stainless steel members retain significantly greater strength and stiffness than carbon steel at temperatures exceeding 600 °C. Their full-scale fire tests on columns and beams—supported by extensive FE back-analysis and parametric studies—

revealed slower deflection rates and higher ultimate load capacities in stainless steel. These findings led to refinements in the reduction factors and thermal expansion allowances specified in the ECCS Model Code. The resulting design recommendations were later adopted almost verbatim into Euro (Inox, 2007) and, with minor harmonization, incorporated into in (EN 1993-1-2, 2005).

Tondini et al., (2013) reported the first full-scale fire tests on structural members fabricated from EN 1.4003 ferritic stainless steel, a grade increasingly specified for thin-walled tubular applications yet lacking elevated-temperature data. Within the RFCS “SAFSS” programme, they loaded three hollow-section columns two square and one rectangular axially in compression and then subjected them to a standard fire curve, repeating each test at room temperature for baseline comparison. The authors noted that the ferritic tubes maintained significant load-bearing capacity well into the heating phase, providing benchmark temperature–displacement histories and failure modes that had not previously been available for this material class. These results, they argued, furnish essential validation data for emerging design rules and numerical models targeting ferritic stainless-steel columns under fire loading.

Kucukler et al., (2020) developed and validated nonlinear FE models capable of accurately reproducing the flexural-buckling behavior of stainless-steel I-section columns at elevated temperatures. Following model verification, the study conducted an extensive parametric analysis encompassing multiple stainless-steel grades (austenitic, duplex, and ferritic), a broad range of cross-section geometries and slenderness ratios, and various fire-level temperature conditions. Based on this numerical database, the fire design rules for stainless steel columns currently specified in EN 1993-1-2 were evaluated. The assessment revealed that existing provisions often yield scattered and inaccurate strength predictions. To address this limitation, an alternative design approach was proposed-fully compatible with the Eurocode framework but calibrated to reflect the findings of the parametric study. Comparative analyses demonstrated that the new design rules offer significantly improved predictive accuracy while maintaining appropriate safety margins across all investigated scenarios. As a result, the proposed approach is scheduled for inclusion in the forthcoming revision of EN 1993-1-2.

Molkens et al., (2021) investigated the post-fire reuse potential of stainless-steel members by evaluating residual mechanical properties following exposure to elevated temperatures and subsequent cooling. Tensile coupons were first extracted from a Grade 1.4301 austenitic cellular beam that had undergone fire testing under load. Additional “virgin” coupons were then subjected to peak temperatures ranging from 200 °C to 1000 °C under various controlled cooling regimes. Results showed that the proof strength, ultimate strength, and Young’s modulus of austenitic stainless steel exhibited only minor degradation below approximately 700 °C and retained higher values than equivalent carbon-steel retention factors even at higher temperatures. To build a broader perspective, these findings were combined with existing data for ferritic and duplex stainless-steel grades. A statistical reliability analysis was then conducted, leading to the proposal of reduced partial safety factors and a lower reliability index-intended to balance the economic and social costs of post-fire replacement. The study concluded that most stainless-steel structural elements can be reinstated after fire with minimal remedial work, provided that residual strengths are verified and the updated safety factors are applied.

(Quan & Kucukler, (2023) developed and validated FE models for cold-formed and hot-rolled stainless steel square and rectangular hollow sections (SHS and RHS) subjected to fire-level temperatures. These models were benchmarked against physical test results before being used to conduct an extensive parametric study comprising 13,860 simulations. The study covered austenitic, duplex, and ferritic stainless-steel grades across a broad range of cross-section slenderness levels and included combined loading scenarios involving compression, bending, and shear at temperatures approaching the melting point of steel. Based on the simulation data, a new cross-section design approach was proposed. This method incorporates the forthcoming plate-buckling provisions of EN 1993-1-2 and extends their application to SHS and RHS stainless-steel members. Comparative evaluations against current Eurocode rules demonstrated that the proposed approach offers significantly improved accuracy while maintaining adequate safety margins, thereby providing a more reliable and robust framework for the design of stainless-steel hollow sections under elevated-temperature conditions.

2. NUMERICAL METHODOLOGY AND MODEL DEVELOPMENT

2.1. Overview

This chapter establishes the numerical framework used to evaluate fire-induced progressive collapse in a six-story prototype building. Section 2.2 presents the temperature-dependent mechanical and thermal properties for carbon steel, Grade 1.4301 stainless steel, concrete, and the intumescent coating. Section 2.3 outlines the global building layout, member sizing, and the gravity and accidental load combinations defined according to GSA guidelines that govern the ambient temperature demand. Section 2.4 introduces the three benchmark structural variants-bare carbon steel, fire-protected carbon steel, stainless steel, and a hybrid system-that serve as the basis for comparative analysis.

Section 2.5 describes the FE modelling strategy implemented in ABAQUS (2025), including element formulations, boundary conditions, and mesh convergence checks. Section 2.6 defines the applied thermal actions, specifically the ISO 834 standard fire and the EN 1991-1-2 parametric compartment fire curve. Section 2.7 maps these fire curves to compartment-specific scenarios within the prototype building. Section 2.8 outlines how inter-zone delay is incorporated within the sequentially coupled thermal-structural analysis framework used to simulate progressive collapse. Finally, Section 2.9 presents verification and validation exercises conducted at the member, sub-assembly, and global structural levels.

2.2. Material Properties

2.2.1. Temperature-Dependent Mechanical Properties of Steel

At ambient temperature (20 °C), the carbon steel grade considered in this study exhibits a density of 7850 kg/m³, a Young's modulus of 206000 MPa, a Poisson's ratio of 0.30, and a nominal yield strength of 235 MPa.

The austenitic Grade 1.4301 stainless steel has the same density as carbon steel (7850 kg/m³) but a slightly lower Young's modulus of 200000 MPa. It retains Poisson's ratio of 0.30 and has a nominal yield strength of 230 MPa. The reinforcing steel is modeled as an elastic-perfectly plastic material with a yield strength of 415 MPa and a modulus of elasticity

of 200000 MPa. All baseline material properties are subsequently adjusted using the temperature-dependent reduction factors specified in EN 1993-1-2, (2005) for structural fire design.

The temperature-dependent stress-strain curve for structural carbon steel at elevated temperatures is obtained by multiplying the ambient-temperature mechanical properties by temperature-specific reduction factors (see Figure 8). These coefficients are defined as follows:

- Yield-strength reduction factor: $k_{y,\theta} = f_{y,\theta} / f_y$ where f_y is the nominal yield strength at 20°C and $f_{y,\theta}$ is the corresponding value at temperature θ .
- Proportional-limit reduction factor: $k_{p,\theta} = f_{p,\theta} / f_y$ which $f_{p,\theta}$ denoting the proportional limit at θ .
- Elastic-modulus reduction factor: $k_{E,\theta} = E_{a,\theta} / E_a$ where E_a is the elastic modulus at room temperature and $E_{a,\theta}$ is its value at θ .

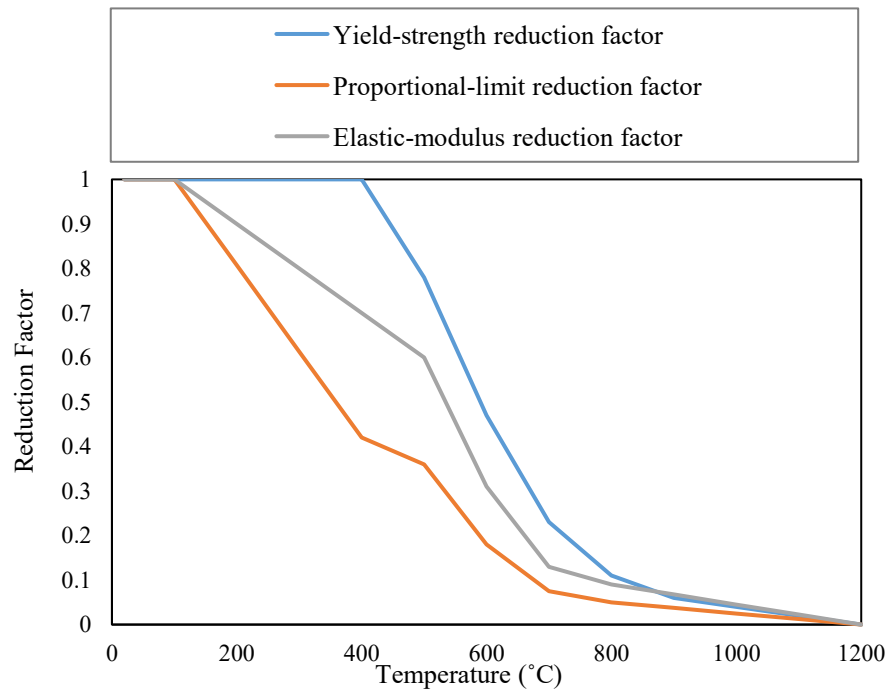


Figure 8. Reduction factors for the stress-strain relationship
of carbon steel at elevated temperature

The elevated temperature stress–strain response (Figure 9) of austenitic stainless steel is described using a set of non-dimensional reduction factors, which are applied to the room-temperature mechanical properties detailed below:

- Elastic stiffness: $k_{E, \theta} = E_{a, \theta} / E_a$, the ratio of the elastic modulus at temperature θ to its ambient value.
- 0.2 % proof strength: $k_{0.2p, \theta} = f_{0.2p, \theta} / f_y$
- Ultimate tensile strength: $k_{u, \theta} = f_{u, \theta} / f_{uk}$

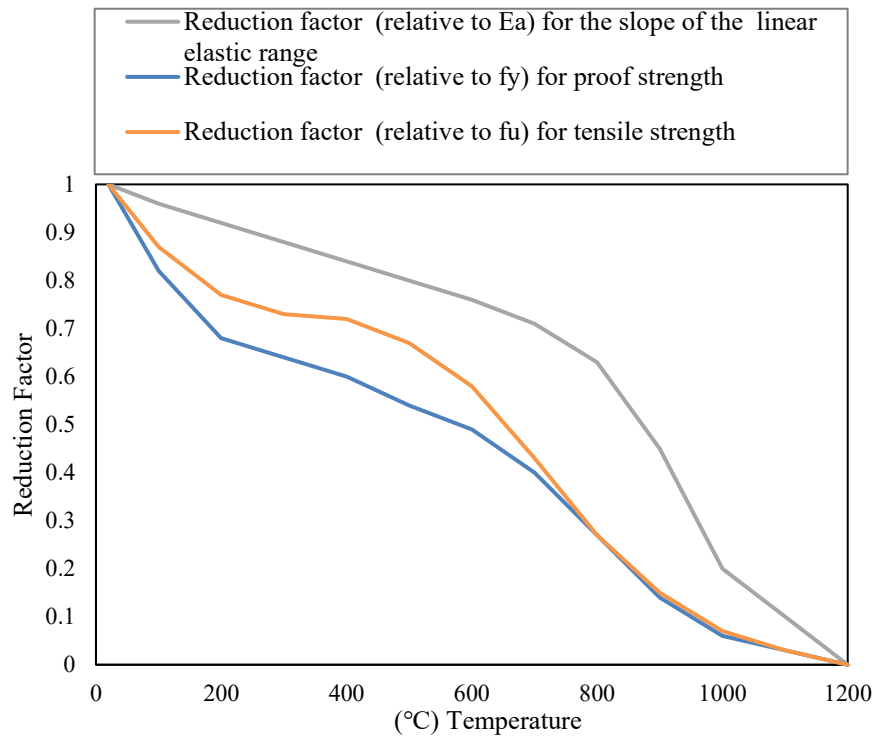


Figure 9. Reduction factors for the stress-strain relationship of austenitic stainless steel at elevated temperature

The temperature-dependent stress-strain relationships adopted in this study are presented in (Figure 10) for carbon steel and (Figure 11) for stainless steel. Both curves were generated directly from Eurocode 3 – Design of steel structures, Part 1-2: General rules – Structural fire design EN 1993-1-2, (2005). For carbon-steel, the reduction factors were taken from Table 3.1, whereas the corresponding factors for austenitic stainless steel were extracted from Informative Annex C of the same standard.

For carbon steel (Figure 10), the yield plateau that characterizes room-temperature behavior remains visible up to approximately 400 °C but contracts rapidly and disappears above 500 °C. The effective yield strength decreases to around 40 % of its ambient value at 600 °C and drops to below 10 % at 700 °C. Beyond this point, the stress-strain curve shows pronounced softening and very limited plastic capacity.

The austenitic stainless-steel curves (Figure 11) exhibit continuous strain hardening across all temperatures, along with a significantly slower loss of strength compared to carbon steel. At 700 °C, the material retains more than half of its yield strength at 20 °C, and meaningful hardening behavior persists even up to 900 °C. This gradual strength degradation, combined with sustained ductility, indicates that stainless steel members are capable of accommodating larger deformations while maintaining load-carrying capacity — a key advantage for structural systems relying on catenary or membrane action during fire exposure.

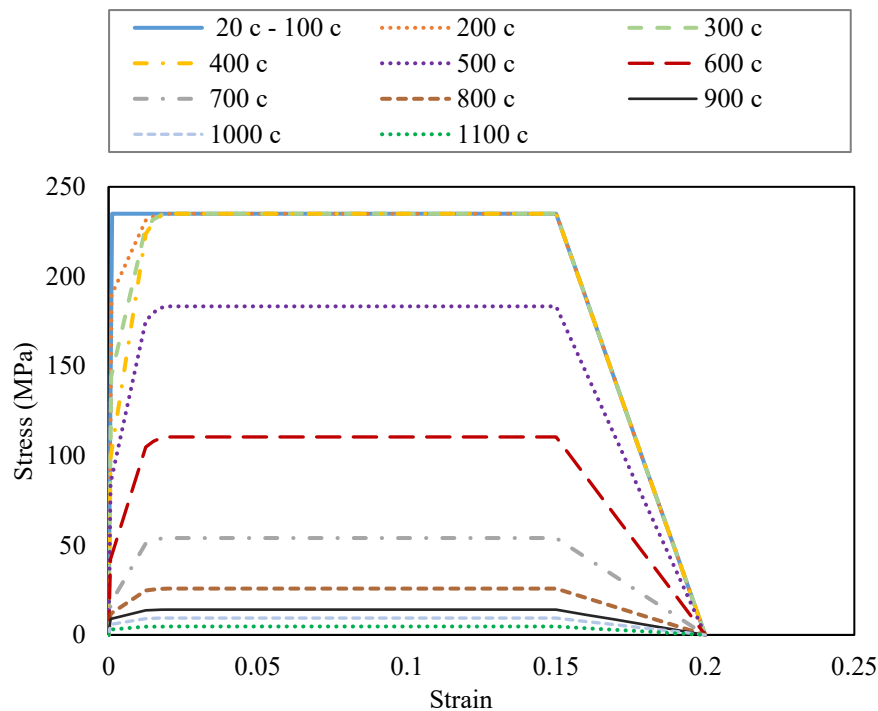


Figure 10. Stress-strain relationship for carbon steel at elevated temperatures

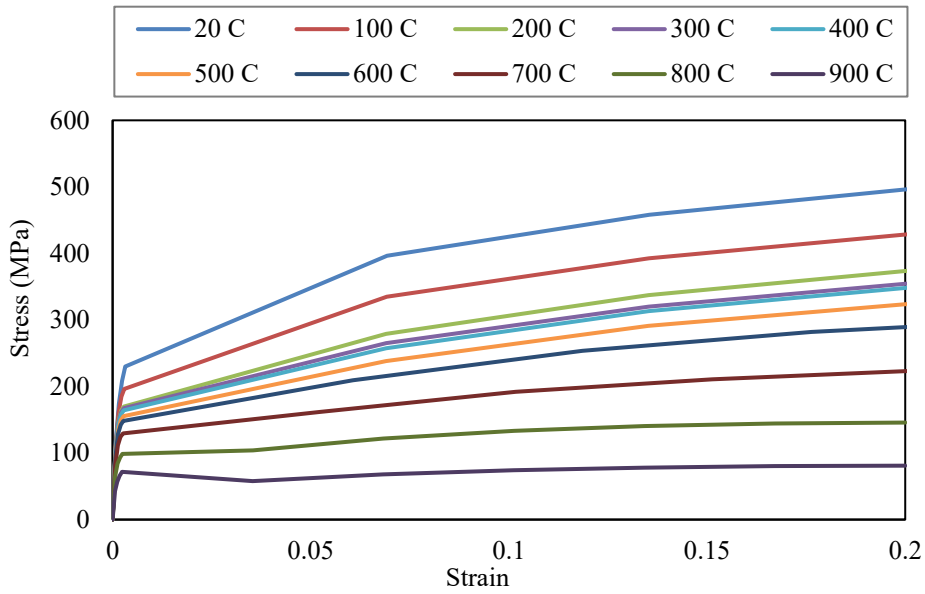


Figure 11. Stress-strain relationship for grade 1.4301 stainless steel
at elevated temperatures

2.2.2. Temperature-Dependent Thermal Properties of Steel

Accurate prediction of the steel temperature field and the associated thermally induced strains require four temperature-dependent thermal properties: (i) thermal elongation, (ii) specific heat capacity, (iii) thermal conductivity, and (iv) surface emissivity. These properties govern, respectively, free thermal expansion, transient heat storage, conductive heat transfer through the section, and radiative heat exchange with the fire environment. In this study, all values were taken from EN 1993-1-2, (2005).

- Thermal Elongation: The dimensionless thermal elongation of carbon steel, expressed as $\Delta l / l$, is calculated based on the following set of equations:

For $20\text{ }^{\circ}\text{C} \leq \theta_a < 750\text{ }^{\circ}\text{C}$:

$$\Delta l / l = 1.2 \times 10^{-5} \theta_a + 0.4 \times 10^{-8} \theta_a^2 - 2.416 \times 10^{-4} \quad (1)$$

For $750\text{ }^{\circ}\text{C} \leq \theta_a \leq 860\text{ }^{\circ}\text{C}$:

$$\Delta l / l = 1.1 \times 10^{-2} \quad (2)$$

For $860\text{ }^{\circ}\text{C} < \theta_a \leq 1200\text{ }^{\circ}\text{C}$:

$$\Delta l / l = 2 \times 10^{-5} \theta_a - 6.2 \times 10^{-3} \quad (3)$$

The thermal elongation of austenitic stainless steel $\Delta l / l$ is calculated using the following equation:

$$\Delta l / l = (16 + 4.79 \times 10^{-3} \theta_a - 1.243 \times 10^{-6} \theta_a^2) \times (\theta_a - 20) \times 10^{-6} \quad (4)$$

where l is the length at $20\text{ }^{\circ}\text{C}$, Δl is the temperature induced elongation, θ_a is the steel temperature ($^{\circ}\text{C}$). (Figure 12) shows the temperature-elongation curve for carbon steel and stainless steel

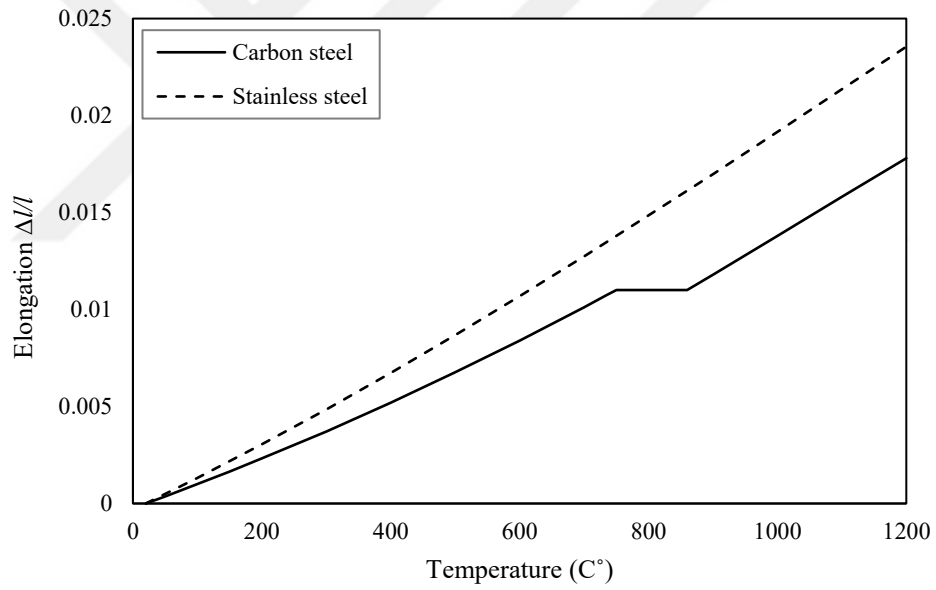


Figure 12. Temperature-dependent relative thermal elongation
for carbon steel and stainless steel

- Specific Heat: The temperature-dependent specific heat capacity of steel (Figure 13), c_a , is obtained using the following relationship:

For carbon steel:

For $20\text{ }^{\circ}\text{C} \leq \theta_a < 600\text{ }^{\circ}\text{C}$:

$$c_a = 425 + 7.73 \times 10^{-1} \theta_a - 1.69 \times 10^{-3} \theta_a^2 + 2.22 \times 10^{-6} \theta_a^3 \text{ J / kgK} \quad (5)$$

For $600\text{ }^{\circ}\text{C} \leq \theta_a < 735\text{ }^{\circ}\text{C}$:

$$c_a = 666 + \frac{13002}{738 - \theta_a} \text{ J / kgK} \quad (6)$$

For $735\text{ }^{\circ}\text{C} \leq \theta_a < 900\text{ }^{\circ}\text{C}$:

$$c_a = 545 + \frac{17820}{\theta_a - 731} \text{ J / kgK} \quad (7)$$

For $900\text{ }^{\circ}\text{C} \leq \theta_a \leq 1200\text{ }^{\circ}\text{C}$:

$$c_a = 650 \text{ J / kgK} \quad (8)$$

For stainless steel:

$$c_a = 450 + 0.280 \times \theta_a - 2.91 \times 10^{-4} \theta_a^2 + 1.34 \times 10^{-7} \theta_a^3 \text{ J/kgK} \quad (9)$$

where: θ_a is the steel temperature ($^{\circ}\text{C}$).

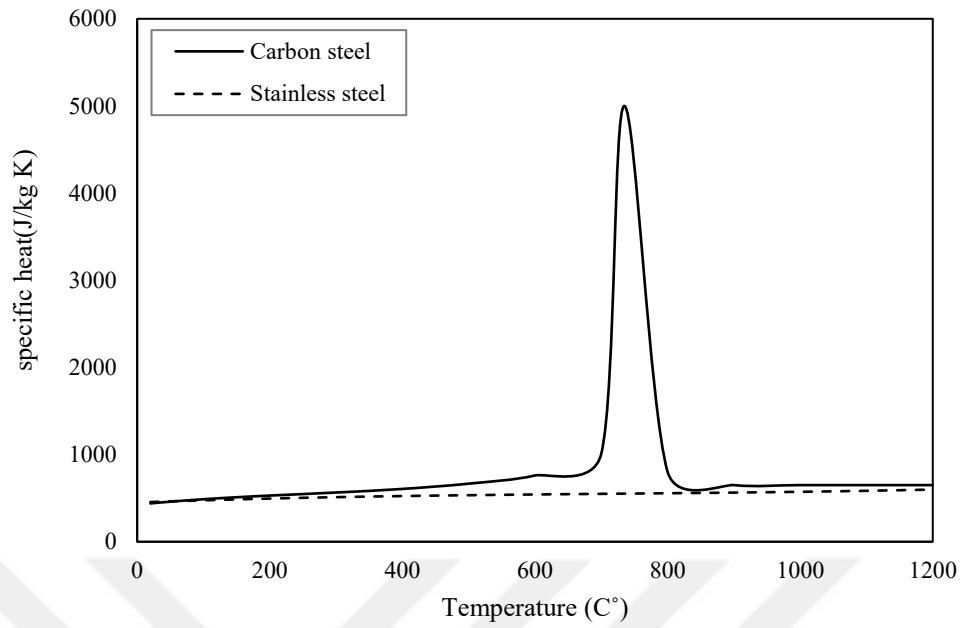


Figure 13. Specific heat of carbon steel and stainless steel

- Thermal Conductivity: The thermal conductivity of steel λ_a (Figure 14) is determined using the following:

For carbon steel:

For $20\text{ }^{\circ}\text{C} \leq \theta_a < 800\text{ }^{\circ}\text{C}$:

$$\lambda_a = 54 - 3.33 \times 10^{-2} \theta_a \text{ W/mK} \quad (10)$$

For $800\text{ }^{\circ}\text{C} \leq \theta_a \leq 1200\text{ }^{\circ}\text{C}$:

$$\lambda_a = 27.3 \text{ W/mK} \quad (11)$$

For stainless steel:

$$\lambda_a = 14.6 + 1.27 \times 10^{-2} \theta_a \text{ W/mK} \quad (12)$$

where: θ_a is the steel temperature ($^{\circ}\text{C}$).

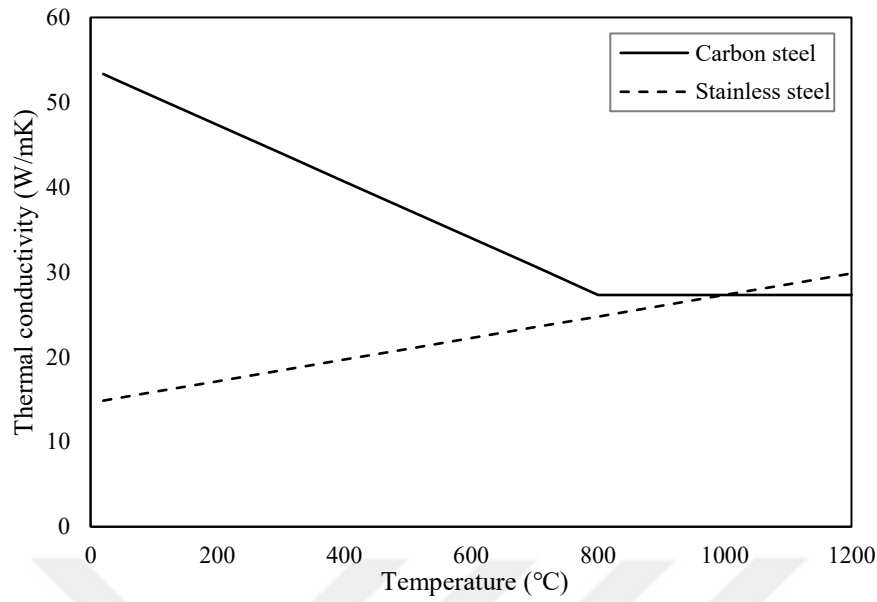


Figure 14. Thermal conductivity of carbon steel and stainless steel

- **Emissivity:** Emissivity is defined as the ratio of a material's thermal radiation to that of a black body at the same temperature, ranging ($0 \leq \varepsilon \leq 1$). For carbon steel, emissivity increases with temperature due to the formation of an oxidized surface layer above 300 °C. This oxidation increases surface roughness and enhances heat absorption. In contrast, stainless steel forms a passive chromium-rich oxide layer even at room temperature. This layer remains stable at elevated temperatures and inhibits further oxidation, resulting in lower surface roughness and reduced emissivity. According to EN 1993-1-2, (2005), the recommended emissivity values are 0.7 for carbon steel and 0.4 for stainless steel.

2.2.3. Concrete - Slab Material Properties

The composite floor slab of the steel building studied is represented by an 80 mm normal-weight concrete layer reinforced with $\phi 6$ mm bars at 200 mm spacing. A characteristic cube compressive strength of $f_c' = 32$ MPa is adopted, consistent with the range reported by Shan & Li, (2021). Concrete non-linearity is captured through the concrete-damage plasticity (CDP) formulation available in ABAQUS. The stress-strain backbone curves in compression and tension follow the modified Kent-Park model (Kent & Park, 1971; Park & Paulay, 1991), while the damage parameters are calibrated according to

the procedure proposed by Hafezolghorani et al., (2017). (Figure 15) shows the adopted constitutive law, and the corresponding CDP coefficients dilatation angle, viscosity parameter, compressive and tensile-damage factors, and inelastic-strain breakpoints are summarized in Table 2.1.

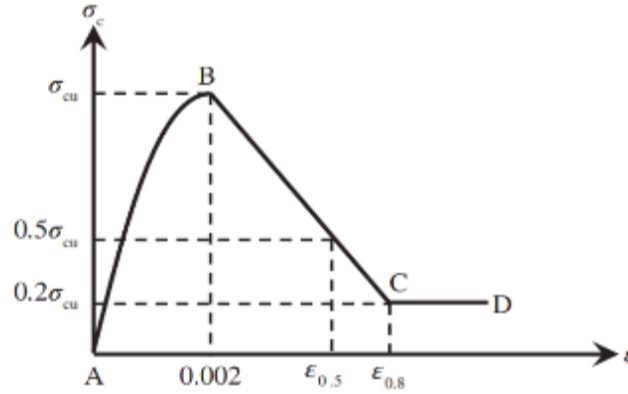


Figure 15. Kent and Park parabolic constitutive model

Table 2.1. Parameters of CDP model

Dilation Angle, ψ	Eccentricity, (e)	f_{b0} / f_{c0}	K	Viscosity Parameter, (μ)
38	0.1	1.156	0.667	0.261

2.2.4. Intumescent Coating Thermal Properties

Because the intumescent layer is extremely thin (0.5 mm), its density and specific heat have only a marginal effect on the calculated steel temperature. Constant values from Annex E of EN 13381-8, (2013) were therefore adopted $c_p=1000 \text{ J / kgK}$ and $\rho_p=100 \text{ kg / m}^3$. The temperature-dependent effective thermal conductivity, $\lambda \text{ (T)}$ was taken from the fire-test correlation developed by Wang et al., (2013) and is plotted in (Figure 16). Charring-induced expansion was neglected at this stage because the target thickness ($\leq 1 \text{ mm}$) remains within the pre-expansion regime for most commercial epoxy-based intumescent systems; the resulting error in conductive heat flow is below 5 % for heating rates typical of the ISO-834 curve Cirpici et al., (2016). Heat flux was applied to the coated surface in line with the experimental configuration, using a convective coefficient of $25 \text{ W / m}^2 \text{ K}$ and the emissivity 0.8 taken EN 1991-1-2, (2002).

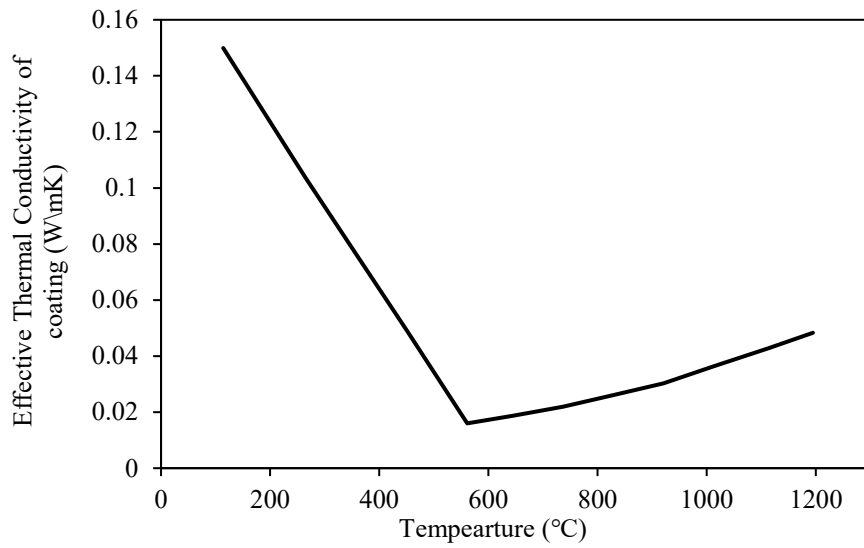


Figure 16. Effective thermal conductivity-temperature curve

2.3. Prototype Steel Building Details and Loading Scheme

2.3.1. Global Layout

The benchmark structure is a six-story moment-resisting steel building originally developed by Shan & Li, (2021) for a medium-seismicity region in China. As shown in (Figure 17), the plan consists of five bays in the longitudinal direction and three bays in the transverse direction, each bay measuring 6 m. Story heights are 4.2 m at ground level and 3.3 m for the remaining floors (see Figure 18). A composite floor system 80 mm concrete slab cast on profiled steel decking and acting compositely with the surrounding beams spans between the primary girders. Reinforcement comprises 6 mm diameter bars at 200 mm centers with a 30 mm bottom cover.

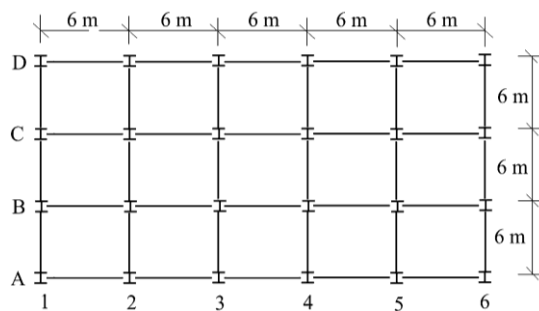


Figure 17. Plan view of steel building

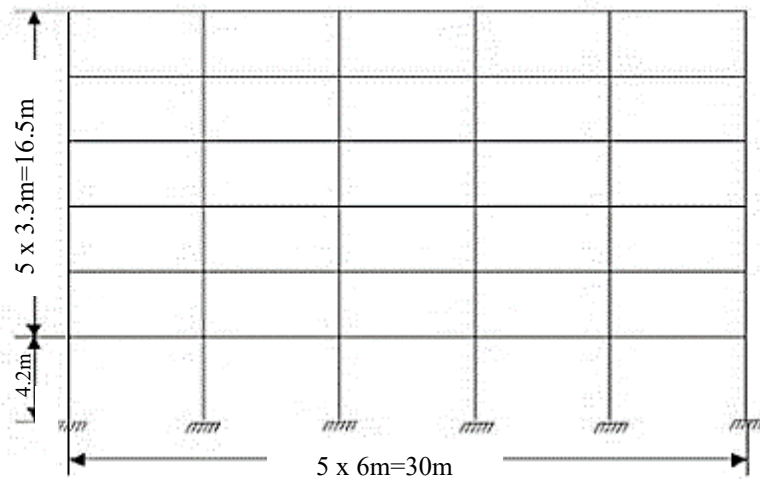


Figure 18. Elevation view of steel building

2.3.2. Cross-Sectional Property of Members

All gravity and lateral-resisting members employ rolled H-sections selected in accordance with the Chinese design codes (GB 50003-2011, 2011; GB 50011-2010, 2010; GB 50017-2017, 2017; GB50009-2012, 2012). Column sections are oriented so that their strong axes align with the short-span direction, thereby compensating for the relatively low lateral stiffness in that plane. The complete section catalogue is summarized in Table 2.2.

Table 2.2. Cross-sections of frame members adopted in steel building

Member	Story		
	1	2-3	4-6
Corner Column	HW250×250×9×14	HW250×250×9×14	HW250×250×9×14
Edge Column	HW350×350×12×19	HW350×350×12×19	HW300×300×10×15
Interior Column	HW400×400×13×21	HW350×350×12×19	HW300×300×10×15
Edge Beam	HN 298×149×5.5×8	HN 298×149×5.5×8	HN 298×149×5.5×8
Interior Beam	HN399×199×7×11	HN399×199×7×11	HN350×175×7×11

Note: The four numbers indicate, in order, the section's height, width, web thickness, and flange thickness.

2.3.3. Gravity & Service-Load Combinations

Design dead loads (exclusive of member self-weight) are 1.5 kN / m^2 on the roof and 3.5 kN / m^2 on typical floors. Live-load allowances follow the same codes: 2.0 kN / m^2 on the roof and 3.5 kN / m^2 on occupied stories. The gravity load combination according to GSA is set to $1.2\text{DL} + 0.5\text{LL}$.

2.4. Benchmark Orientations of Building

Four distinct variants were practiced for every fire scenario (Table 2.3). Case CS is the unprotected carbon-steel baseline; Case SS replaces all members with austenitic stainless steel of identical geometry; Case HYB combines stainless-steel columns with carbon-steel beams, reflecting a practical hybrid alternative; and Case PCS applies a 0.5 mm thin-film intumescent coating to the carbon-steel frame to quantify the benefit of passive fire protection. This systematic set-up enables direct, like-for-like comparisons of thermal resistance, deformation capacity, and progressive-collapse propensity across the four material strategies.

Table 2.3. Benchmark orientation of building considered in the study

Case No	Case Name	Beams	Columns	Exposed Fire Curve	Fire-Scenario Diagram
1	CS	CS	CS	ISO-834 (unprotected)	Figure 26 - 29
2	SS	SS	SS	ISO-834 (unprotected)	Figure 26 - 29
3	HYB	CS	HYB	ISO-834 (unprotected)	Figure 30 - 33
4	PCS	PCS	PCS	Protected curve derived from IC Model	Figure 26 - 29

2.5. FE Modelling Strategy

2.5.1. Mesh & Element Types

A 3-D FE model of the six-story building was developed using ABAQUS, (2025). All primary load-carrying members-columns, girders and floor beams were idealized with two-node Timoshenko beam elements (type B31). These line elements capture axial, flexural, shear and torsional actions while keeping the number of degrees of freedom to a minimum, thereby reducing computational cost without compromising global accuracy. Concrete floor slabs were discretized with 4-node shell elements featuring reduced integration (type S4R). This choice limits hour-glass modes and is well suited to thin plate-like components subject to membrane and bending effects. Steel reinforcement in the slabs was represented by rebar layers embedded within the shell formulation, allowing temperature-dependent material properties and bonds to be assigned consistently. Composite action between the steel beams and the floor slabs was enforced through ABAQUS tie constraints, which couple all translational and rotational degrees of freedom at the beam–slab interface. Column bases were modelled as fixed supports, and beam-to-column joints were assumed fully rigid. This simplification removes rotational stiffness degradation or connection failure from the analysis; however, the study focuses on system-level fire response, so the influence of these second-order connection effects on the principal findings is expected to be limited. A Rayleigh damping ratio of $\xi = 0.05$ is assigned to the whole frame, consistent with national design guidance for ordinary steel structures. The mesh adopted for the multi-story frame is illustrated in (Figure19).

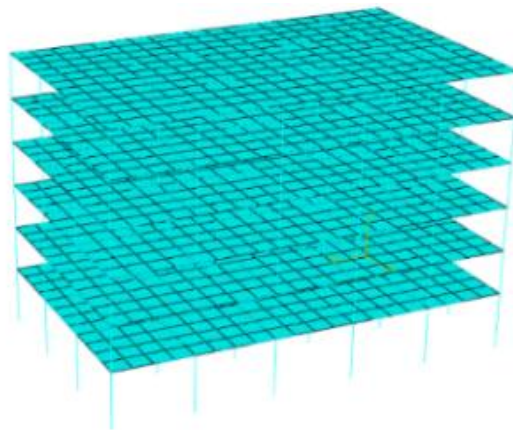


Figure 19. 3-D Finite element model of the steel buildings

2.5.2. Fire Induced Progressive Collapse Analysis

A nonlinear dynamic explicit analysis was conducted. The design gravity load was gradually ramped up to its full value over 3 seconds and held constant for an additional 2 seconds to allow inertial effects to dissipate. Once the system stabilized, the gravity load remained active while fire-exposed structural members were heated in accordance with the parametric temperature-time curve specified in Eurocode EN 1991-1-2. Throughout the analysis, the ratio of kinetic to internal energy was monitored to ensure a quasi-static response was maintained until the onset of failure in the heated columns.

To lessen computational expense, the physical fire-exposure time was uniformly compressed by a factor of 120, so that 1 min of real heating corresponds to 0.5 s of simulation time. Benchmark tests and prior studies (B. Jiang et al., 2015; J. Jiang et al., 2018) indicate that this scaling leaves the structural response essentially unchanged, provided the ratio of kinetic to internal energy remains below the 0.1 threshold recommended for quasi-static explicit analyses.

2.5.3. Intumescent-Coating Modelling Technique

To simulate the progressive collapse of the intumescent-protected frame (Case 4, Table 2.3), a two-stage procedure was adopted:

- Member-level heat-transfer sub-models: Because line-beam elements (B31) cannot accommodate a finite coating thickness, every fire-exposed column and girder was re-modelled in an independent thermal sub-model. The steel core was discretized with eight-node, linear heat-transfer bricks DC3D8, while the 0.5 mm intumescent layer was represented by four-node, reduced-integration shell elements S4R (see Figure 20). Both meshes were coupled with an ABAQUS *TIE* constraint, a technique explicitly recommended for shell-to-solid connections as it suppresses relative slip and rotation along the interface (Dassault Systèmes SIMULIA, 2006).
- Derivation and application of protected-steel fire curves: In each sub-model, the ISO 834 temperature-time curve was applied to the coating surface via a convection-radiation (*film*) boundary during an ABAQUS / Standard transient heat-transfer step. Each column was exposed to the fire from 4 sides, while the

beams were burned from only 3 sides, as in (Figure 21) and (Figure 22) respectively. The resulting nodal steel temperatures were exported and used to generate member-specific protected-steel fire curves, which capture the delayed heating and lower peak temperatures produced by the intumescent layer. These curves are summarized in Figure 41. They were imported as predefined temperature fields and assigned to the corresponding B31 elements in the global ABAQUS / Explicit progressive-collapse analysis.

This layered solid-and-shell formulation preserves through-thickness temperature gradients, accounts for the steel's heat-sink effect, and faithfully reproduces the lag in thermal response introduced by the intumescent coating thereby providing a realistic basis for assessing the progressive-collapse resistance of the protected frame.

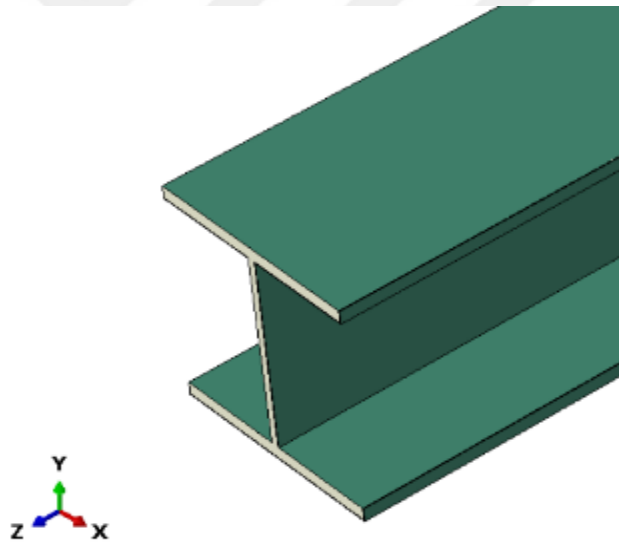


Figure 20. 3-D FE model of an I-section member with a 0.5 mm IC

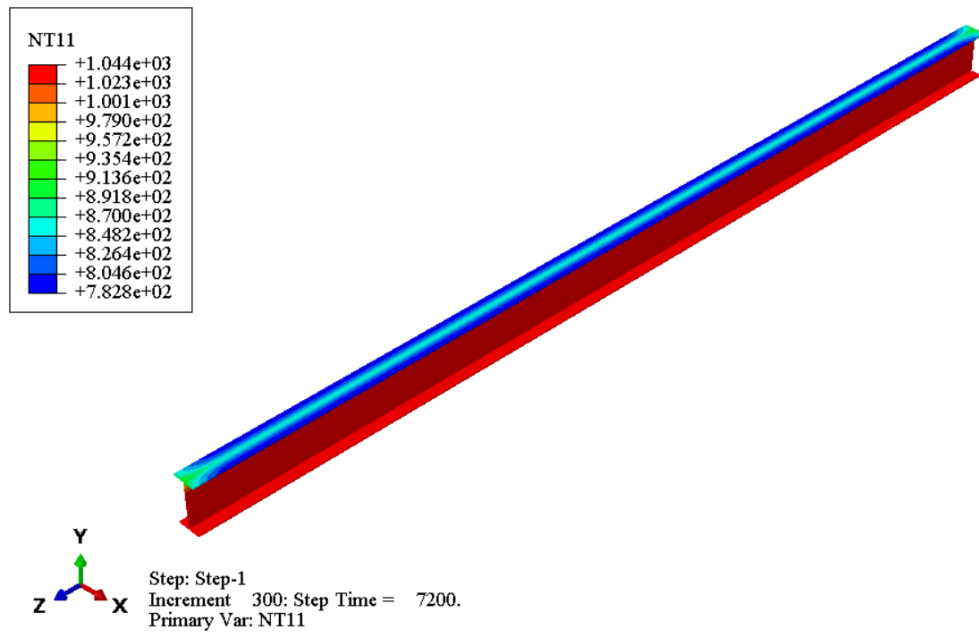


Figure 21. Temperature contour for the HN 298×149×5.5×8 edge beam
(illustrative example)

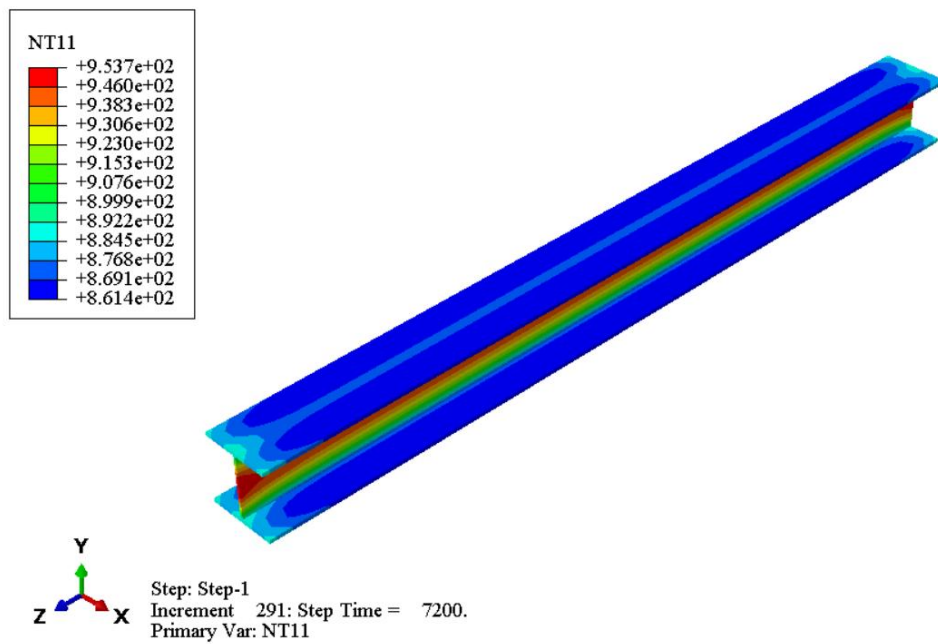


Figure 22. Temperature contour for the HW350×350×12×19 edge column
(illustrative example)

2.6. Fire-Loading Curves

2.6.1. ISO-834 Standard Fire Curve

To represent a fully developed cellulosic compartment fire, the present study employs the ISO 834 standard temperature-time curve (Figure 23) for all unprotected-steel scenarios (Cases 1-3 in Table 2.3). The curve is defined in EN 1991-1-2, (2021) using the logarithmic expression:

$$\theta_g = 20 + 345 \log_{10}(8t + 1) \quad (13)$$

where: θ_g is the gas temperature (°C) and t is the time (min).

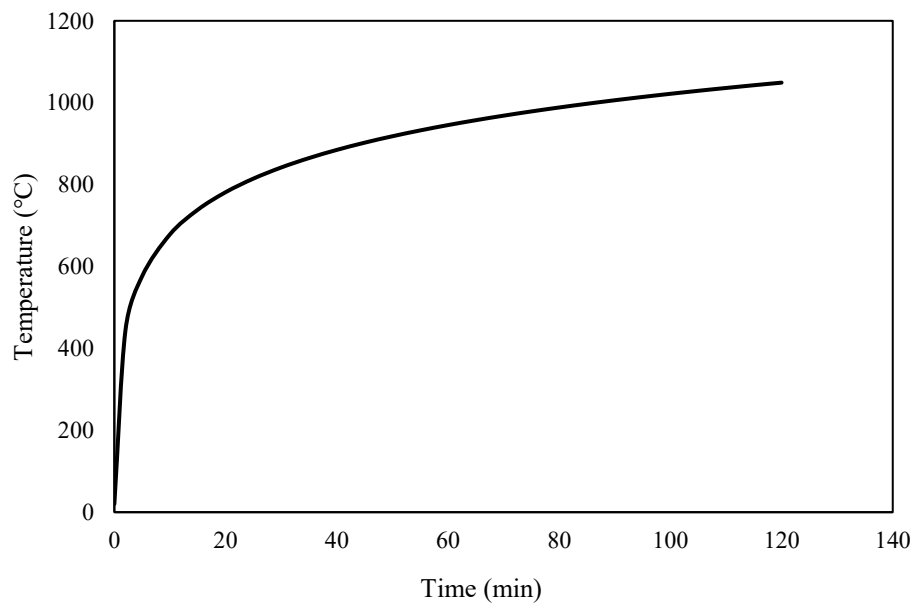


Figure 23. ISO-834 standard fire curve

2.6.2. EN 1991-1-2 Parametric Fire Curve

The Eurocode parametric fire curve (EN 1991-1-2, 2021), which accounts for ventilation and thermal inertia, was adopted for model validation. The curve consists of three distinct phases: (i) an initial heating ramp, (ii) a subsequent cooling ramp, and (iii) a final plateau at constant temperature.

- The temperature-time relationship during the heating phase is expressed by the following curves:

$$\theta_g = 20 + 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) \quad (14)$$

where: θ_g : is the gas temperature (°C), t: time (h) and

$$t^* = t * \Gamma \quad (15)$$

$$\Gamma = (O / b)^2 / (0.04 / 1160)^2 \quad (16)$$

$$b = \sqrt{(\rho c \lambda)} \quad (\text{J/m}^2\text{s}^{1/2}\text{K}) \quad (17)$$

where: ρ : Density of boundary of enclosure (kg / m³), c : Specific heat of boundary of enclosure (J / kgK), λ : Thermal conductivity of boundary of enclosure (W / Mk), O : opening factor (m^{1/2}) and Γ : time factor function.

For the validated model, (Shan & Li, 2021) adopted the following parameters : (i) a fire load density ($q_{t,d}$) of 180 MJ / m² , (ii) an opening factor (O) of 0.045 m^{1/2}, and (iii) a compartment-surface thermal parameter (b) of 1407 J / m² s^{1/2} K

- The maximum temperature in the heating phase when $t^* = t_{max}$

$$t_{max} = \max [0.2 \times 10^{-3} \frac{qt,d}{O}, t_{lim}] \quad (18)$$

t_{lim} =20 min for medium fire growth according to Eurocode.

- The temperature-time curves in the cooling phase are given by:

$$\theta_g = \theta_{max} - 625 (t^* - t_{max}^*) \text{ for } t^*_{max} \leq 0.5\text{h} \quad (19)$$

Using the specified input parameters, the resulting parametric temperature-time curve (Figure 24) rises to a peak of 887 °C at 48 minutes during the heating phase, then cools back to ambient temperature after 153 minutes.

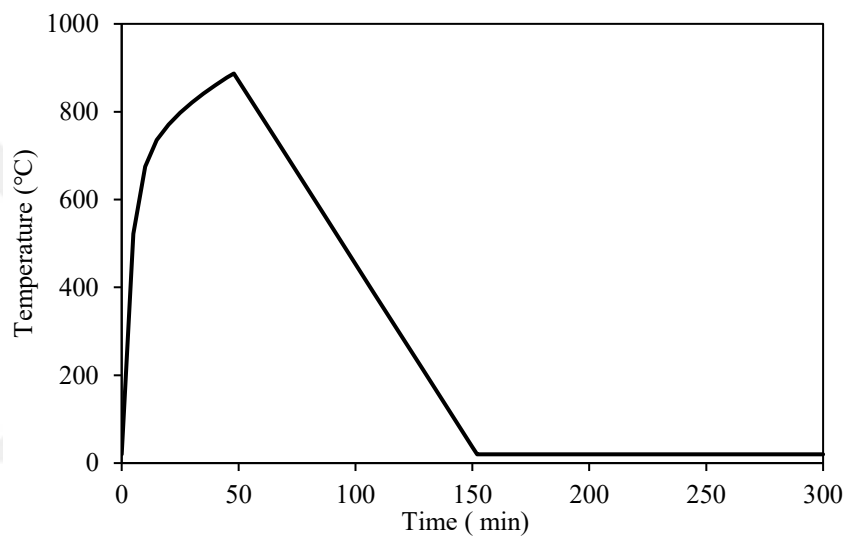


Figure 24. Eurocode parametric fire curve

2.6.3. Analytical Temperature-Time Curve for Fire-Protected Steel

An analytical expression for the transient steel temperature within members encased by passive fire-protection material is provided in EN 1993-1-2, (2005). Assuming a uniform cross-section temperature, the incremental rise of steel temperature, $\Delta\theta_{a,t}$, over a time step Δt (≤ 30 s) is governed by:

$$\Delta\theta_{a,t} = \frac{\lambda A_p / V}{d_p c_a \rho_a} \frac{(\theta_{g,t} - \theta_{a,t})}{\left(1 + \frac{\phi}{3}\right)} \Delta t - (e^{\frac{\phi}{10}} - 1) \Delta\theta_{g,t} \quad (20)$$

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p A_p / V \quad (21)$$

where:

- A_p / V : is the section factor for steel members insulated by fire protection material
- A_p : is the appropriate area of fire protection material per unit length of the member (m^2 / m)
- V : is the volume of the member per unit length (m^3 / m)
- c_a : is the temperature dependent specific heat of steel ($\text{J} / \text{kg K}$)
- c_p : is the temperature independent specific heat of the fire protection material ($\text{J} / \text{kg K}$)
- d_p : is the thickness of the fire protection material (m)
- Δt : is the time interval (seconds)
- $\theta_{a,t}$: is the steel temperature at time t ($^{\circ}\text{C}$)
- $\theta_{g,t}$: is the ambient gas temperature at time t ($^{\circ}\text{C}$)
- $\Delta\theta_{g,t}$: is the increase of the ambient gas temperature during the time interval Δt (K)
- λ_p : is the thermal conductivity of the fire protection system (W / mK)
- ρ_a : is the unit mass of steel (kg / m^3)
- ρ_p : is the unit mass of the fire protection material (kg / m^3)

In the present work this closed-form solution was used to generate an analytical fire-protected curve, which serves as a benchmark for validating the thermal response predicted by the detailed FE heat-transfer model of the intumescent-coated frame. Agreement (or lack thereof) between the analytical curve and the numerical steel temperature history serves as an immediate check on the accuracy of the coating's material properties and the adequacy of the mesh discretization, prior to executing progressive collapse simulations. (Figure 25) shows the protected-steel fire curve compared with the ISO 834 standard fire curve.

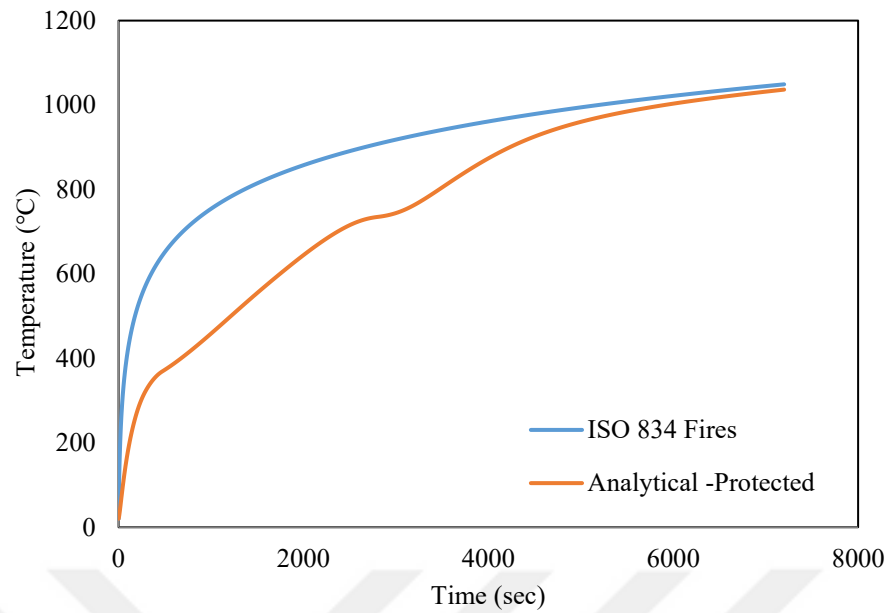


Figure 25. Protected-steel analytical fire curve and ISO-834

2.7. Fire Scenarios

To capture the range of realistic thermal exposures in multi-story building, several compartment-fire scenarios were considered: (i) two stationary fires (corner, central bay), (ii) one two vertical travelling fire, and (iii) four horizontal travelling-fire cases with different spread direction. Table 2.4 lists the scenarios together with the corresponding Figs. references, where the temperature-time curves and ignition footprints are illustrated in detail.

Table 2.4. Fire scenarios considered in the study

ID	Name	Description	Figure Reference
F-1	a-Compartment	Single-Corner Compartment	Figure 26a
F-2	b-Compartment	Single-Central Compartment	Figure 26b
F-3	a-Compartment	Vertical Fire Traveling	Figure 27a
F-4	b-Compartment	Vertical Fire Traveling	Figure 27b
F-5	Central-Plus	Fire Traveling	Figure 28a
F-6	E-W	Fire-Travelling (East-West)	Figure 28b
F-7	N-S	Fire-Travelling (North-South)	Figure 29a
F-8	L	Fire-Travelling in L shape (N-S) & (E-W)	Figure 29b

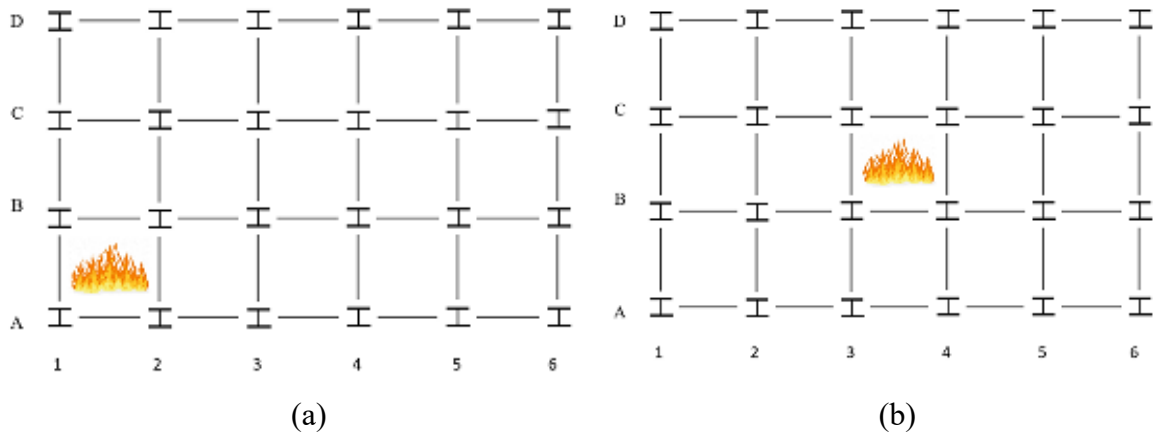


Figure 26. (a) F-1 fire scenario and (b) F-2 fire scenario

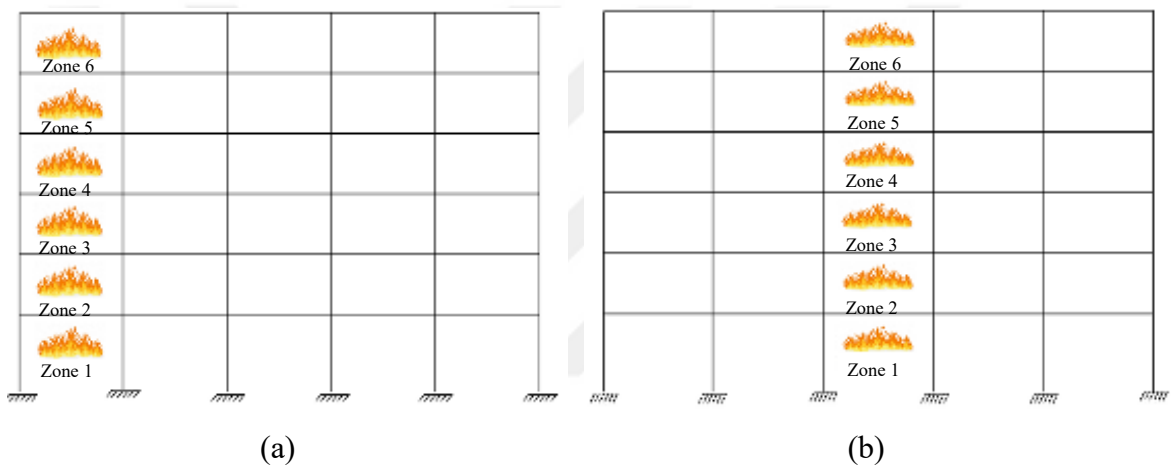


Figure 27. (a) F-3 fire scenario and (b) F-4 fire scenario

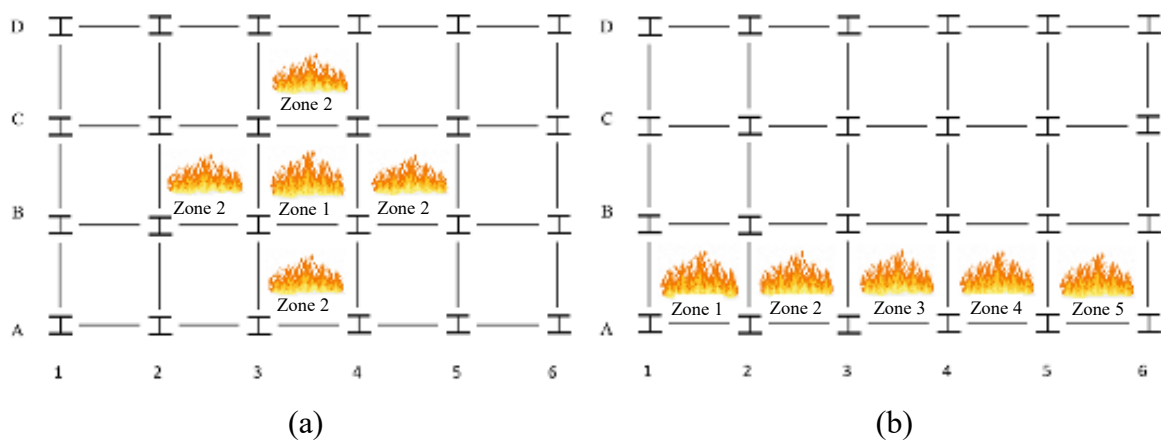


Figure 28. (a) F-5 fire scenario and (b) F-6 fire scenario

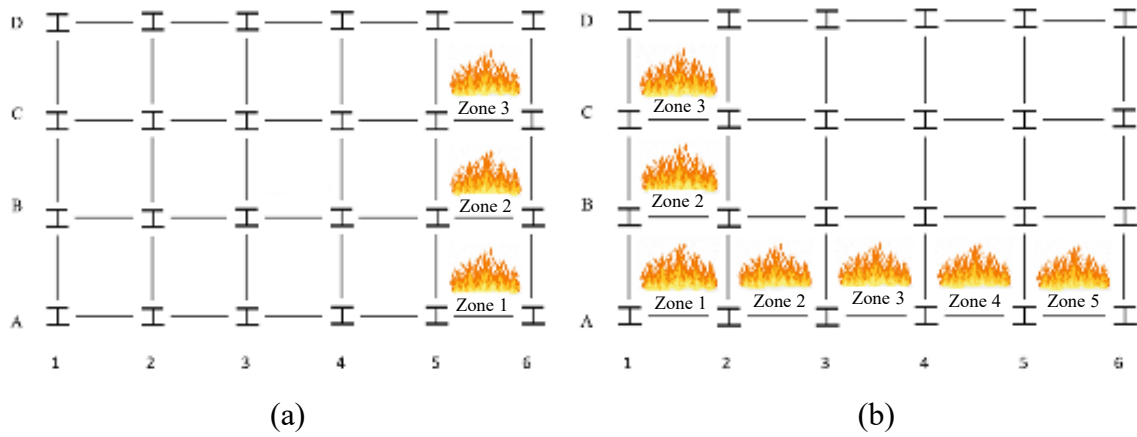


Figure 29. (a) F-7 fire scenario and (b) F-8 fire scenario

Moreover, the relevant schematics of (Figures. 30-33) are detailed. Elements depicted in red—including those enclosed by a red circle—represent stainless-steel sections, whereas black elements correspond to carbon-steel members.

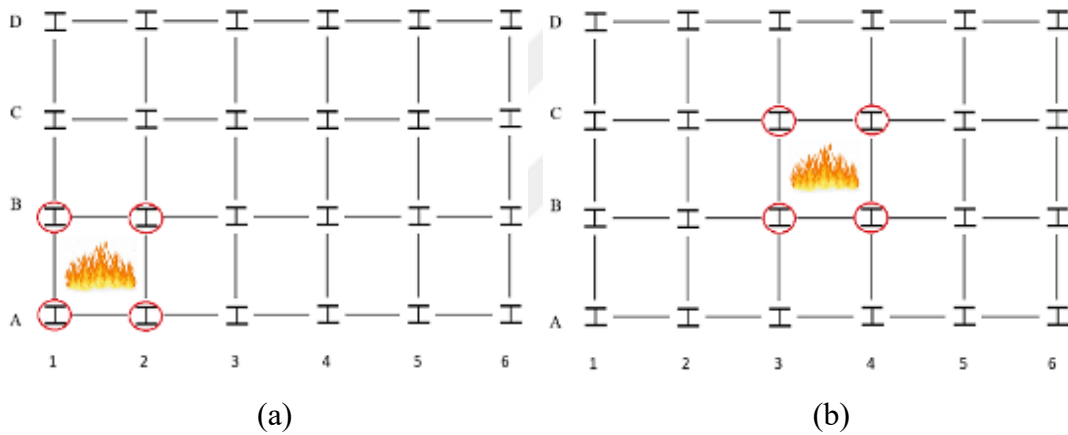


Figure 30. (a) F-1 fire scenario and (b) F-2 fire scenario

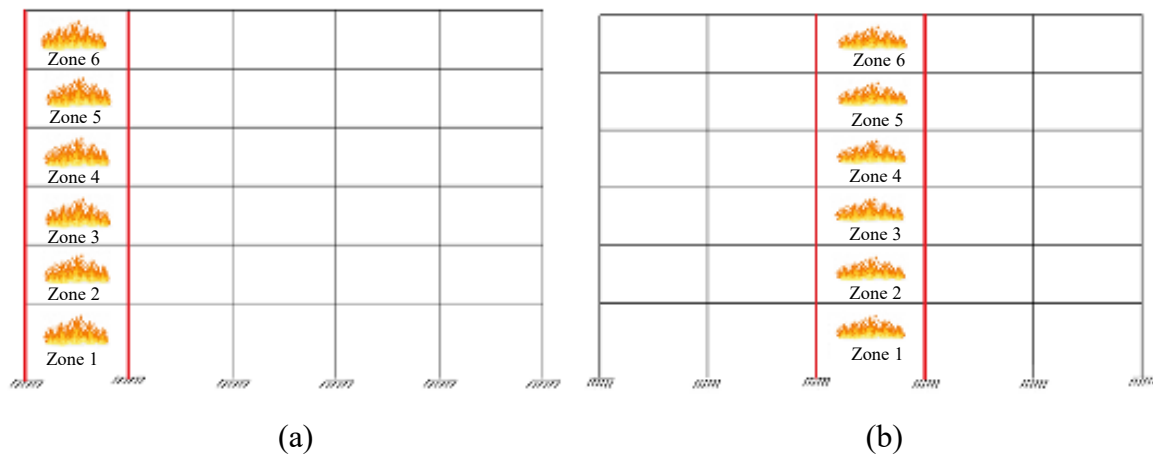


Figure 31. (a) F-3 fire scenario and (b) F-4 fire scenario

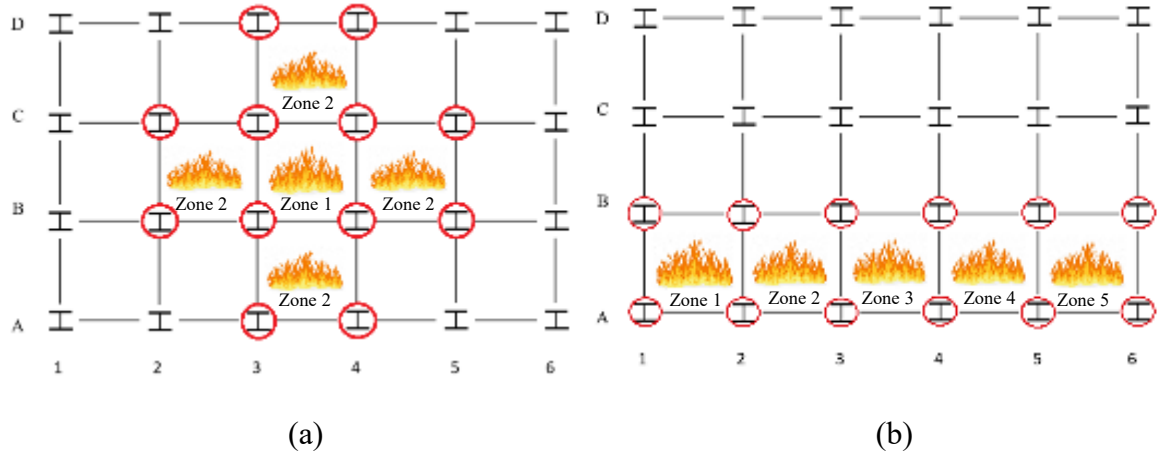


Figure 32. (a) F-5 fire scenario and (b) F-6 fire scenario

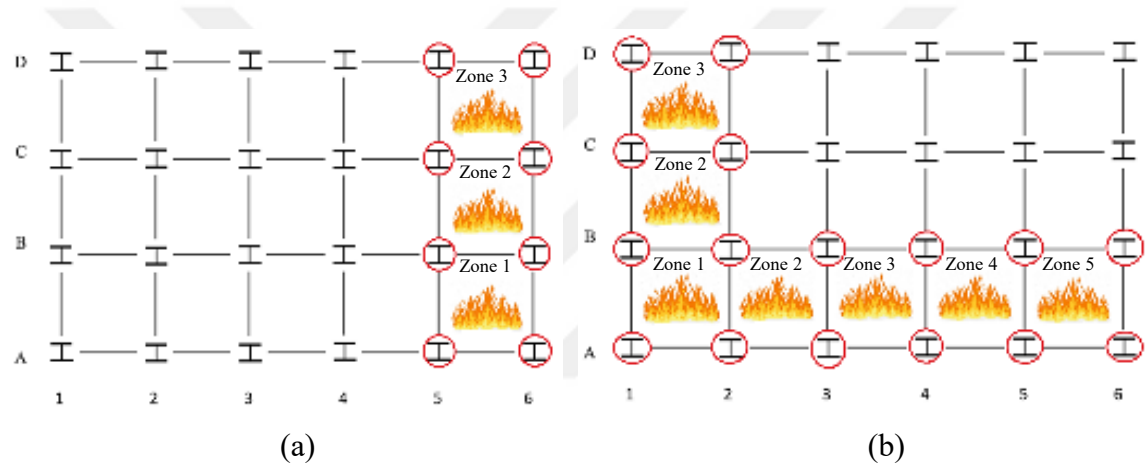


Figure 33. (a) F-7 fire scenario and (b) F-8 fire scenario

2.8. Inter-Zone Delay in Travelling-Fire Scenarios

All travelling-fire cases in this study adopt a uniform inter-zone delay of 30 minutes, following the empirical spread rates reported by Suwondo et al., (2019). To avoid redundant descriptions, the procedure is illustrated once using the six-zone vertical model (Figure 34): Zone 1 ignites at $t = 0$ min, Zone 2 at 30 min, and so on until Zone 6; each zone follows the ISO 834 curve independently. The remaining scenarios-vertical (6 stories), east-west (5 zones), north-south (3 zones), central “PLUS” (2 zones), and L-shaped (5 + 3 zones)-were generated by the same algorithm, differing only in the number and spatial arrangement of zones. Thus, the composite temperature-time histories retain identical 30-minute offsets but are truncated once the last zone in each pattern ignites. These amplitudes are subsequently imported as user-defined fire curves for the non-linear explicit analyses.

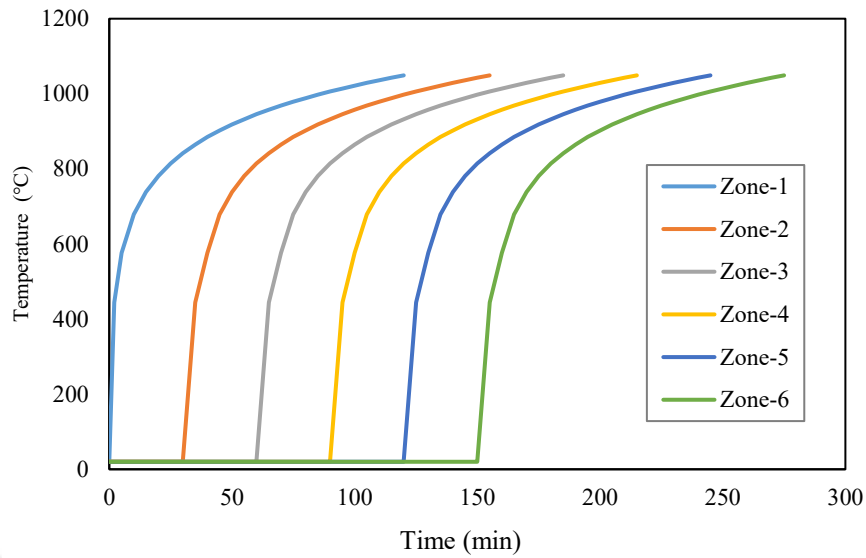


Figure 34. ISO-834 fire curve for 30 min inter-zone time delay

2.9. Model Verification and Validation

2.9.1. Validation using 2-D Line Element

To verify the reliability of the proposed numerical procedure, two laboratory specimens originally tested by (Rubert & Schaumann, 1986) were validated using FE method:

- EHR frame-an L-shaped single-bay configuration
- ZSR frame-a two-span portal configuration

Both frames were built from IPE 80 sections (Figure 35). All members were exposed to the ISO 834 standard fire curve, and temperature-dependent material properties were assigned in accordance with EN 1993-1-2, (2005), as outlined in Chapter 2. The applied vertical and horizontal loads matched the experimental protocol.

Model accuracy was assessed by comparing (i) the mid-span beam and column deflections in the EHR frame and (ii) the lateral top-column displacement in the ZSR frame. As shown in Figure 36 and Figure 37, the displacement-temperature histories predicted by the present FE models follow the experimental records closely and fall within the scatter of previous numerical studies of Lien et al., (2010) and Sun et al., (2012). This agreement confirms that the adopted wire-element formulation can accurately capture the thermally induced behavior of both the EHR and ZSR frames, providing confidence in the simulations conducted for the main parametric study.

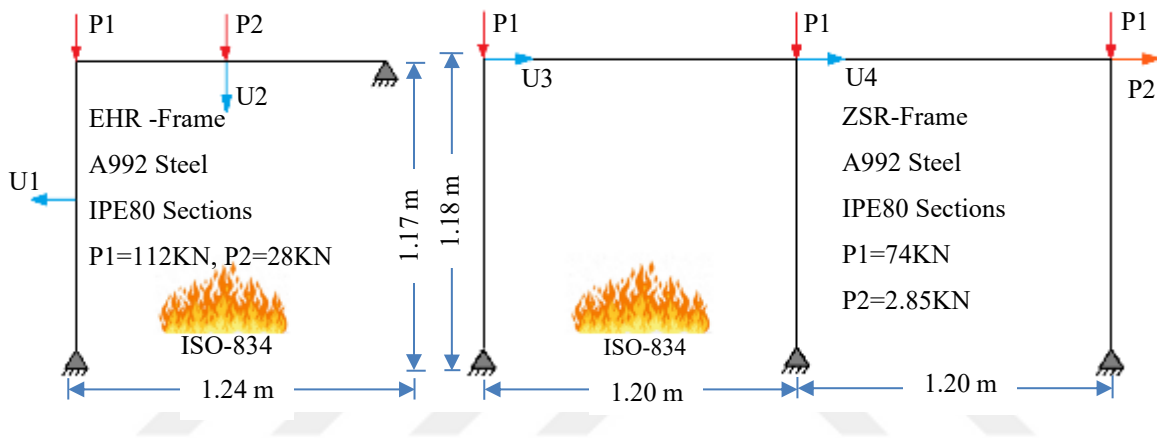


Figure 35. The EHR and ZSR frames

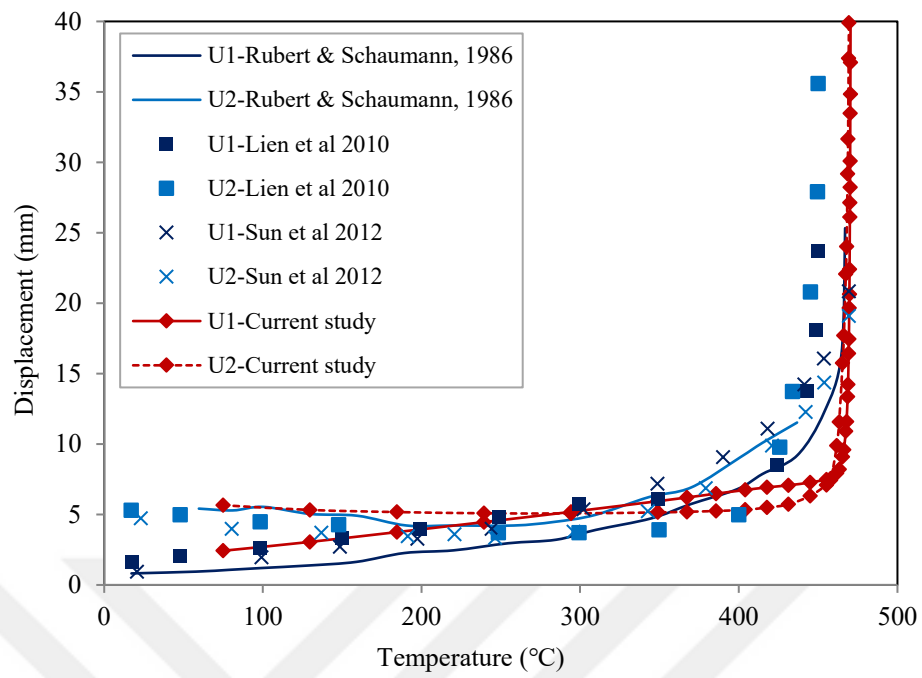


Figure 36. The results obtained from EHR frame

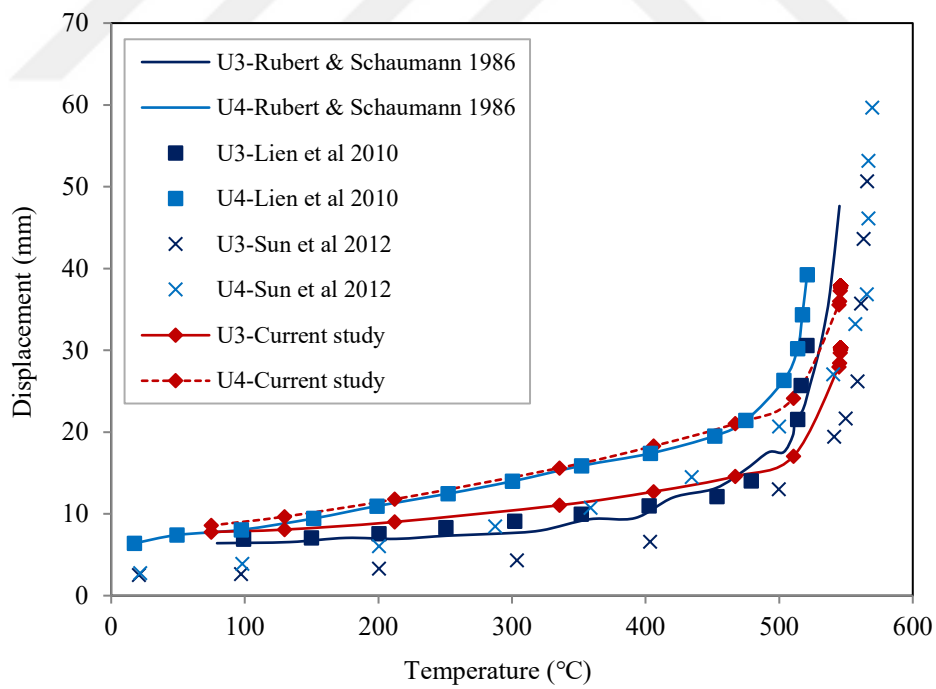


Figure 37. The results obtained from ZSR frame

2.9.2. Validation using 3-D Line Element

The validation of the numerical procedure under a non-standard fire scenario was carried out using the six-story benchmark building model of Shan and Li (2021) (see Figure 38). The geometric layout, member sizes and gravity loads are identical to those specified in Section 2.3; the sole difference lies in the thermal action, the corner compartment was heated by the Eurocode-1 parametric fire curve (Figure 24). Gravity loading (1.2 DL+0.5 LL) was applied quasi-statically prior to heating, and the temperature-dependent steel properties followed to EN 1993-1-2, (2005).

Figure 39 and Figure 40 compares the simulated axial-force and vertical-displacement histories of the corner columns with the results given by Shan and Li (2021). The present model reproduces the measured response to within $\pm 5\%$ throughout the heating phase, confirming that the adopted wire-element formulation and material model remain accurate. This result provides additional confidence in the model's applicability to the eight fire scenarios analyzed in the subsequent chapters.

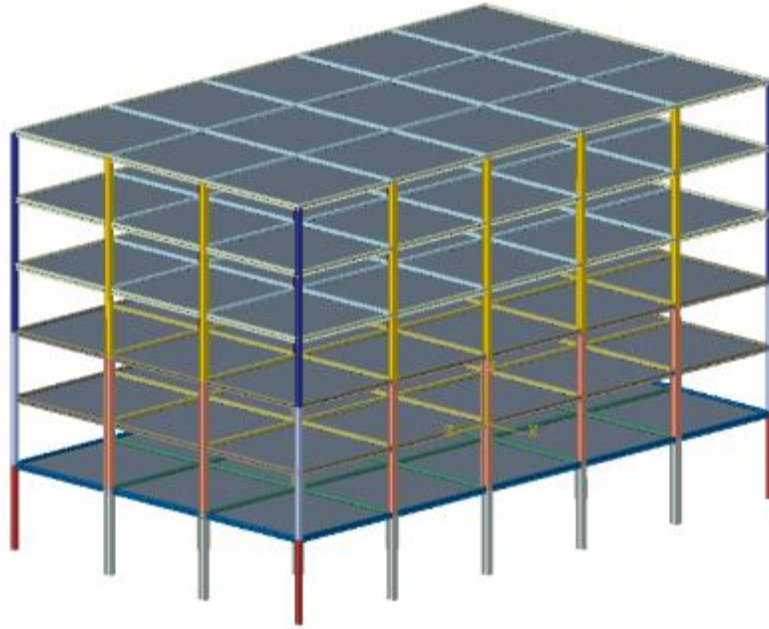


Figure 38. The FE model of the building using validation procedure

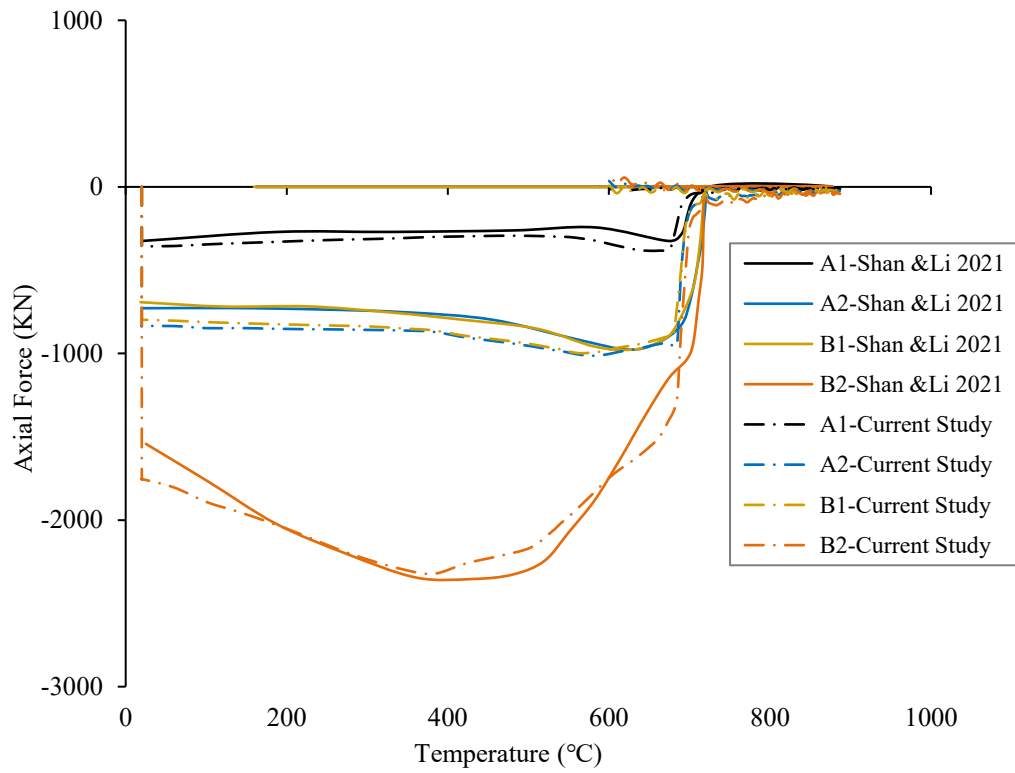


Figure 39. Axial force-temperature curves for heated columns

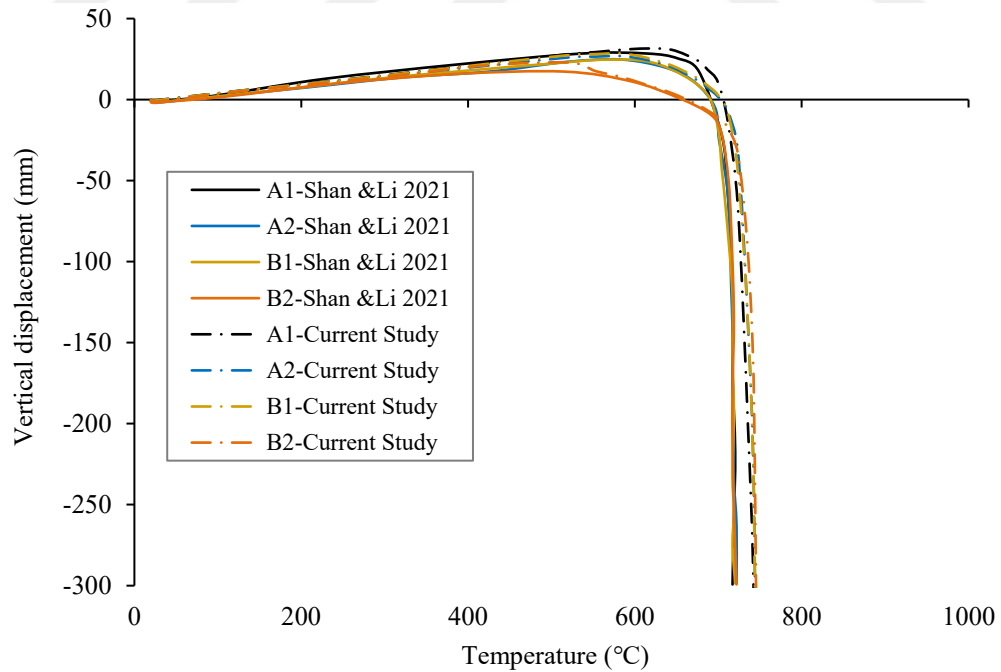


Figure 40. Vertical displacement-temperature curves for heated columns

3. RESULTS AND DISCUSSION

3.1. Overview

This chapter presents the coupled thermomechanical response of a six-story steel building subjected to multiple fire scenarios. The results are organized by material system—unprotected carbon steel (CS), intumescent-protected carbon steel (PCS), austenitic stainless steel (SS), and a hybrid CS–SS configuration—and are discussed along two complementary tracks: (i) global performance metrics (time to collapse and collapse sequence for each scenario), and (ii) member-level diagnostics (temperature histories, vertical displacement trends, axial force trajectories, and utilization factors). For each scenario, the analysis first traces the thermal evolution of the most exposed members, then relates this to column buckling, the initiation of catenary action in floor systems, and the subsequent load redistribution. The failure indicators used for interpretation are defined in Section 3.3.

3.2. Temperature-Time Curves for Intumescent-Coated Steel

The predicted temperature–time responses for the 0.5 mm intumescent-protected steel members subjected to the eight ISO-834 fire scenarios are first presented. All coating properties, char-layer parameters, and numerical implementation details were established in Chapter 2 (Sections 2.2.4 and 2.5.3), where the material dataset was calibrated against the requirements of EN 13381-8 (2013). (Figure 41) presents the temperature–time curves for intumescent-coated steel sections exposed to a two-hour fire, which serve as the thermal input for evaluating the protected frame (Case 4 in Table 2-3). In each case, three curves are shown: the reference ISO fire curve, the analytical-protected prediction, and the numerical model. The ISO fire curve (blue) rises sharply in the early stages, reaching temperatures above 1000 °C within 120 minutes. By contrast, both the analytical-protected and numerical predictions (orange and gray) exhibit a much slower rate of temperature increase, reflecting the protective action of the intumescent coating. Comparing the analytical and numerical results shows strong agreement, with only minor deviations at intermediate times (between ~2000 and 4000 s), where the analytical solution occasionally predicts a slightly higher steel temperature. By the end of the two-hour exposure, both methods converge closely, suggesting that the numerical implementation reliably captures the essential thermal

behavior. This agreement indicates that the calibrated intumescent-coating model reproduces the performance expected under the ISO 834 standard fire and is consistent with the assessment framework of EN 13381-8:2013 for reactive coatings on structural steel.

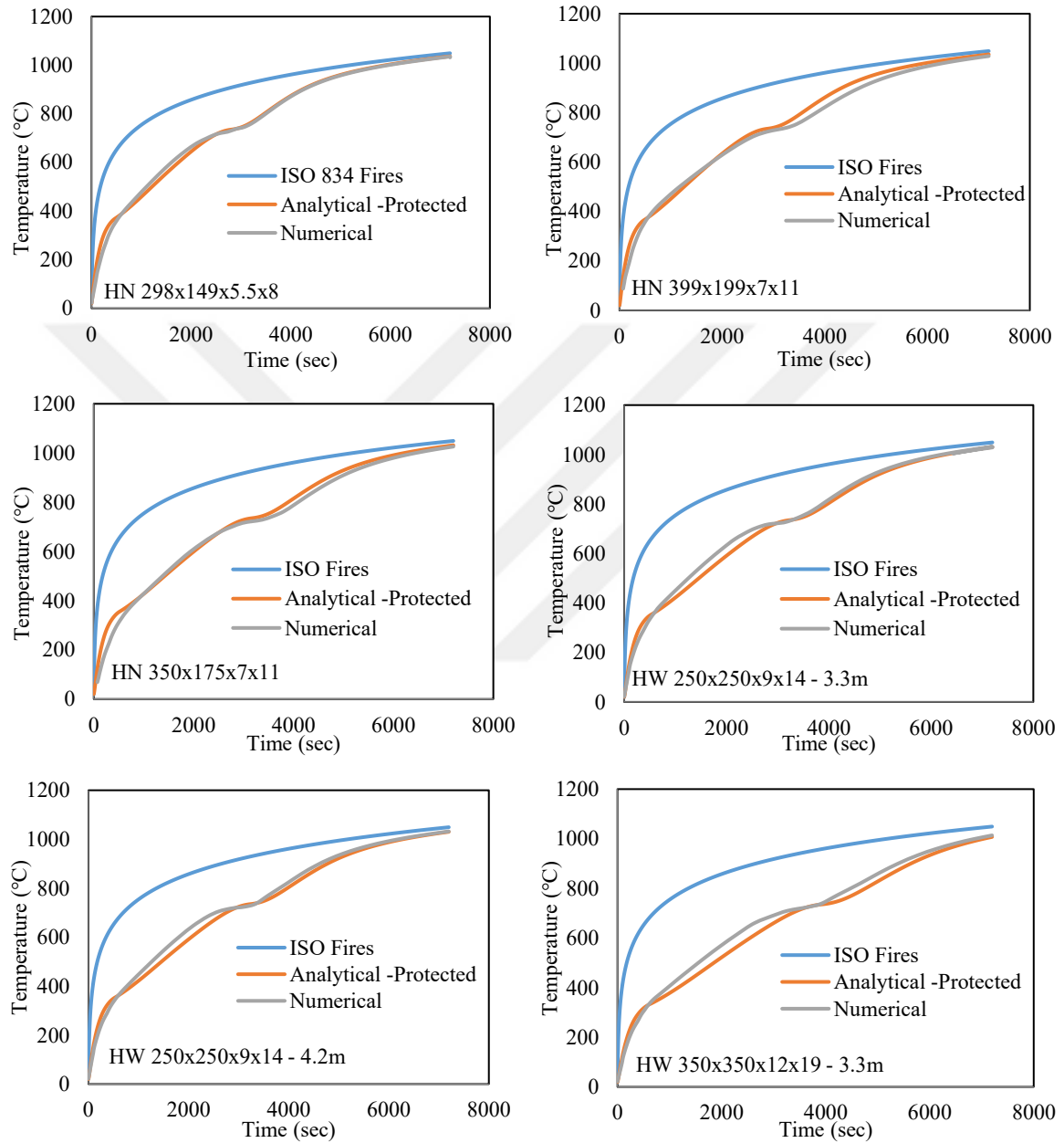
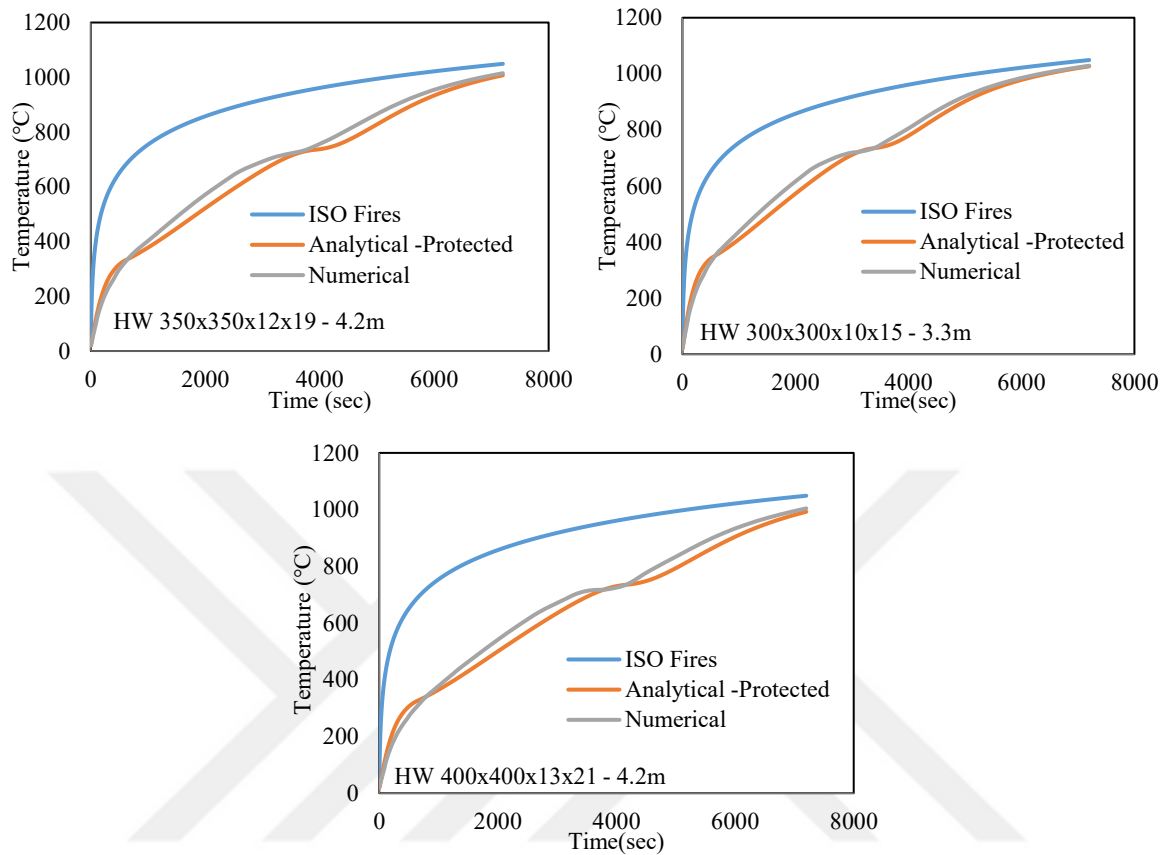


Figure 41. Simulated temperature-time curves for the intumescent-coated steel members exposed to a two-hour ISO-834 fire

Figure 41. (continued)



Because the composite slab in this study is 80 mm thick ($h_e < 120$ mm), the EN 1994-1-2 simple critical-temperature model for composite beams (Clause 4.3.4.2.3) falls outside its formal scope. Consequently, steel temperatures are taken from the transient heat-transfer analysis. For downstand beams with $h \leq 500$ mm, a uniform through-depth steel temperature ($\theta_{web} = \theta_{bottom\ flange}$) is assumed, as permitted by EN 1994-1-2, Clause 4.3.4.2.2(10).

The temperature-time histories for the coated columns are shown in Figure 42, while those for the corresponding beams are presented in Figure 43. The greater scatter among the column curves reflects variations in the section factor (A/V): members with higher exposed-surface-to-volume ratios heat more rapidly, even as the intumescent layer expands to form an insulating char. By contrast, the beams-designed with nearly uniform A/V values and identical boundary conditions-exhibit markedly less variation.

To keep the frame analysis tractable-particularly under traveling-fire scenarios, the HW 350 profile is adopted as the representative section for all columns, and the HN 399 profile for all beams. On average, the intumescent coating delays the attainment of 550 °C by 20–40 minutes compared with bare steel, thereby providing the protected building with an additional thermal buffer for the subsequent structural response analysis.

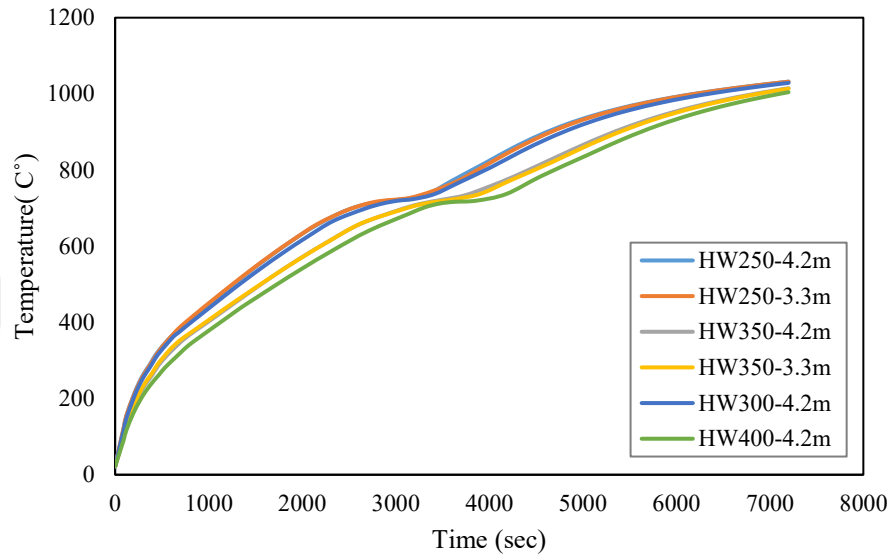


Figure 42. Numerically predicted temperature-time curves for intumescent-coated steel columns subjected to a two-hour ISO-834 fire exposure

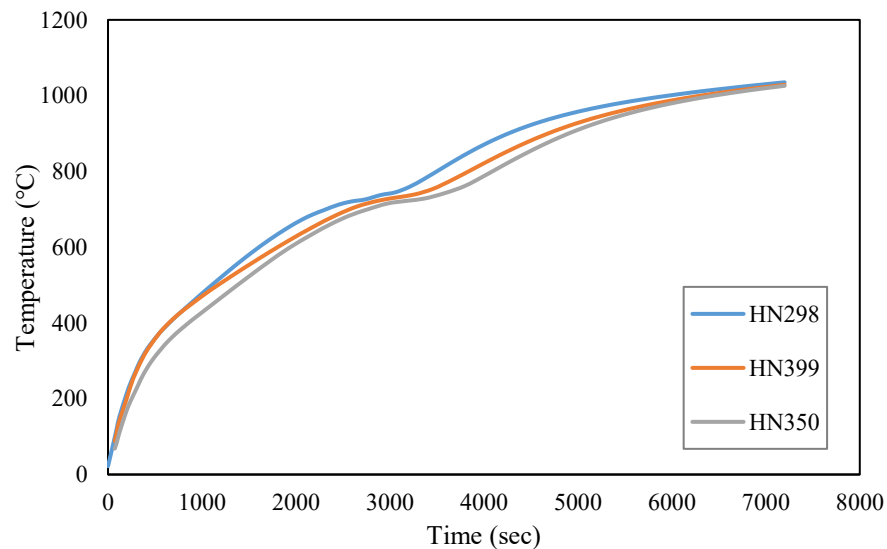


Figure 43. Numerically predicted temperature-time curves for the intumescent-coated steel beams subjected to a two-hour ISO-834 fire exposure

3.3. Failure Indicators Adopted in the Analysis

The analysis interprets structural response using three operational criteria consistent with Eurocode fire design:

- temperature-dependent utilization $\eta_\theta = N_{Ed,\theta} / N_{Rd,\theta}$ with a limit state recorded when $\eta_\theta > 1.0$ persists for at least three consecutive output steps.
- sudden loss of compressive capacity, defined as a $\geq 90\%$ drop of the prior peak axial compression within a temperature band $\Delta\theta \leq 150^\circ\text{C}$, and
- the first zero-crossing of axial force, used to mark load-path reversal and the onset of catenary action in the floor system.

Global collapse is deemed to occur when two or more of these indicators coincide, together with sustained and accelerating vertical shortening of the heated region. Resistance verification at elevated temperature-including the reduction factors $k_{y,\theta}$, $k_{E,\theta}$, and for columns, the fire buckling factor χ_{fi} in $N_{Rd,\theta}$ is carried out in accordance with EN 1993-1-2 (2005). The interpretation of tensile-membrane (catenary) action and robustness follows Eurocode-aligned industry guidance (Simms, 2012) as well as research on membrane action in composite floors.

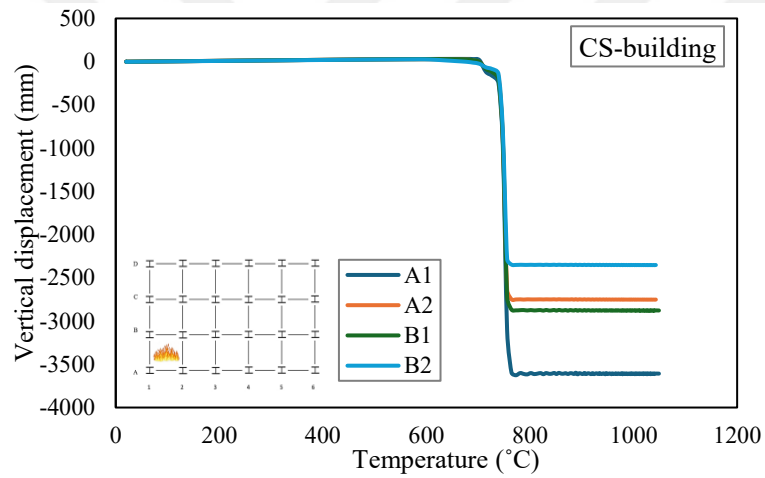
3.4. Vertical Displacement of Columns

Across all scenarios, the vertical-displacement histories of the heated columns exhibit a consistent sequence: modest thermal expansion at low temperatures; accelerated downward movement as stiffness and strength degrade; and, most notably in PCS and HYB cases, partial moderation once floor catenary action develops and gravity loads are redistributed to cooler members. Stainless steel frames show the least sag for the same exposure duration, reflecting the higher retention of elastic modulus and yield strength at elevated temperatures. Scenario-specific results are presented in (Figures 44-51). The displacement results were detailed for the fire-compartment columns or nearby members.

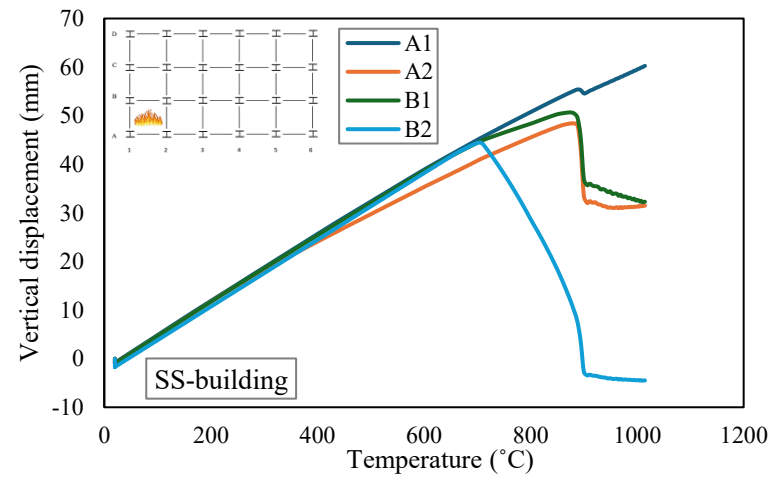
3.4.1. Case F-1

(Figure 44) compare the vertical displacement-temperature responses of four building systems-unprotected carbon steel (CS), stainless steel (SS), protected carbon steel (PCS),

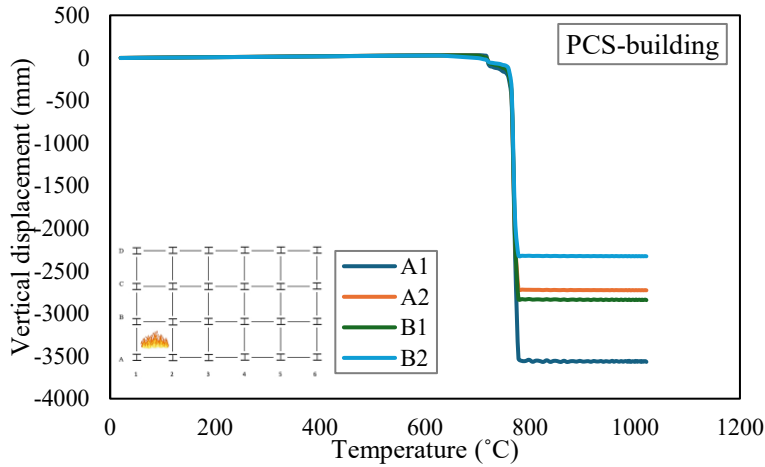
and a hybrid configuration (HYB)-under Case F-1 fire scenario. The CS and PCS buildings show a sudden and severe downward displacement once the critical steel temperature (750-800 °C) is reached, reflecting rapid strength loss and buckling of heated columns; intumescent protection in PCS delays this failure point but does not alter the abrupt collapse mechanism. In contrast, the SS and HYB buildings exhibit more gradual displacement trends, with stainless steel sustaining stiffness and strength at higher temperatures and thus limiting vertical shortening. The HYB system, which combines stainless steel only for the columns at the fire compartment, demonstrates intermediate behavior: displacement develops progressively, and partial recovery is observed once catenary action in the cooler members redistributes gravity loads. Overall, these results highlight the superior thermal robustness of stainless steel and hybrid solutions compared with conventional unprotected or solely protected carbon steel frames.



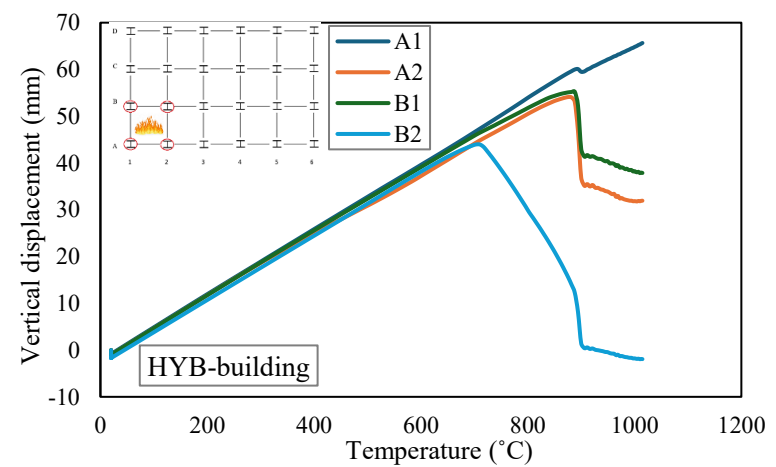
(a)



(b)



(c)

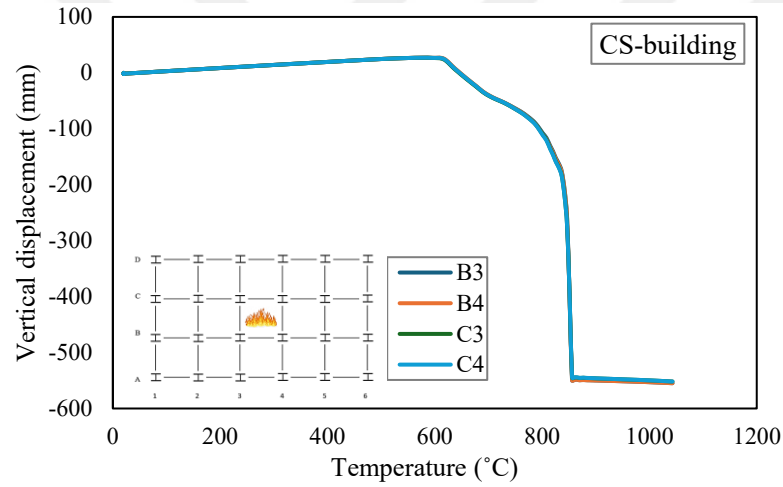


(d)

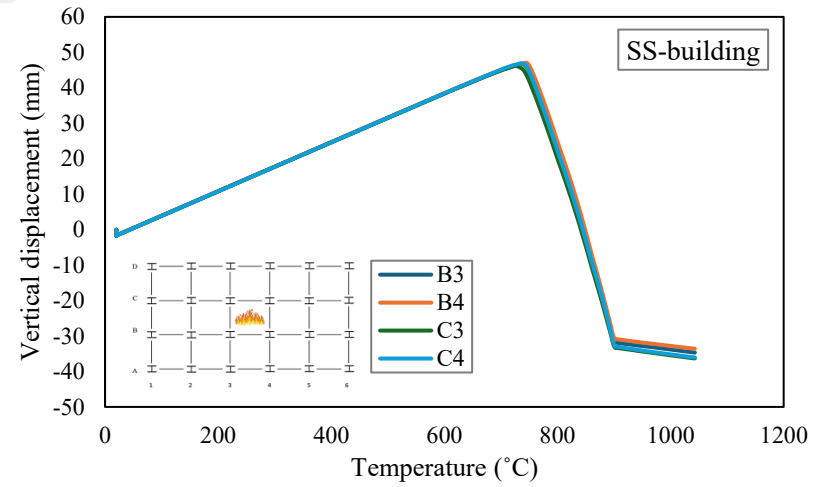
Figure 44. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-1 fire scenario

3.4.2. Case F-2

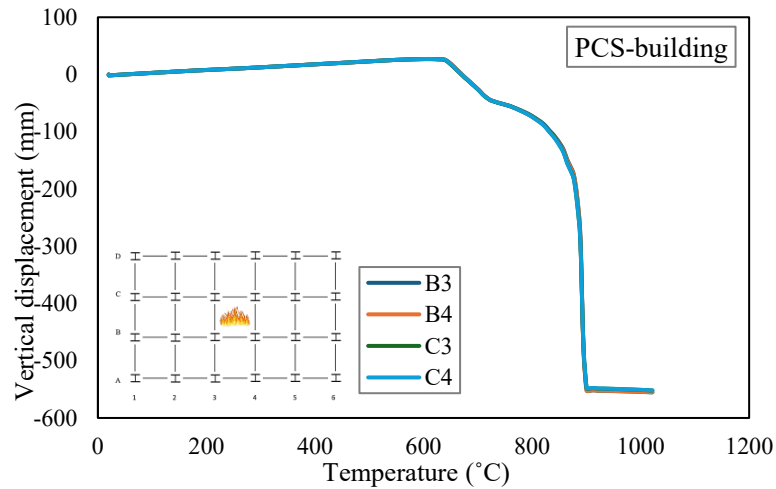
(Figure 45) illustrates the vertical displacement–temperature responses of columns B3, B4, C3, and C4 in the central fire compartment scenario (Case F-1). These columns, located at the fire zone, experience different deformation patterns depending on the material system. In the carbon steel (CS) and protected carbon steel (PCS) buildings, the columns show slight thermal expansion up to approximately 500-600 °C, followed by a sharp and sudden downward displacement due to rapid stiffness loss and buckling around 800-900 °C. While the intumescent coating in PCS provides some delay in failure onset, the overall collapse mode remains abrupt. In contrast, the stainless steel (SS) and hybrid (HYB) buildings exhibit gradual elongation as temperatures rise to 750°C, maintaining alignment and resisting early shortening. Only at elevated temperatures do these columns begin to shorten slightly, and even then, the magnitudes are smaller than those in CS and PCS systems. This reflects the superior thermal stability of stainless steel, while the HYB system benefits from combining stainless and protected carbon steel to achieve similar post-buckling resilience. Overall, these findings confirm that advanced material systems reduce displacement severity and help delay or prevent progressive collapse under fire exposure.



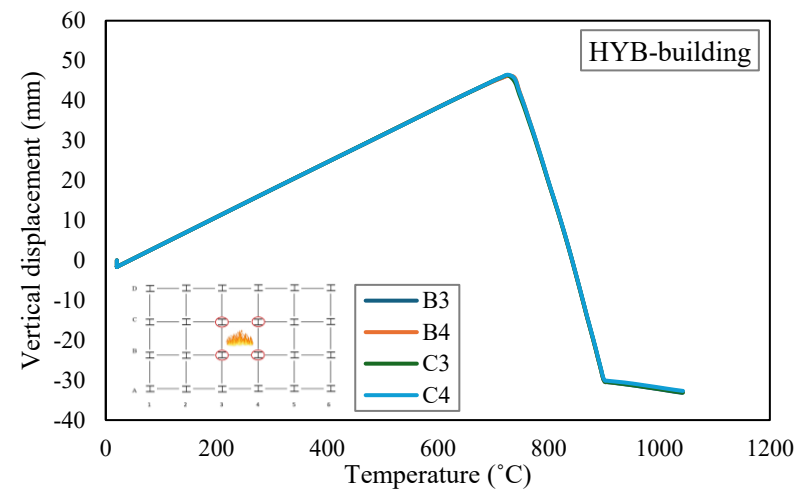
(a)



(b)



(c)

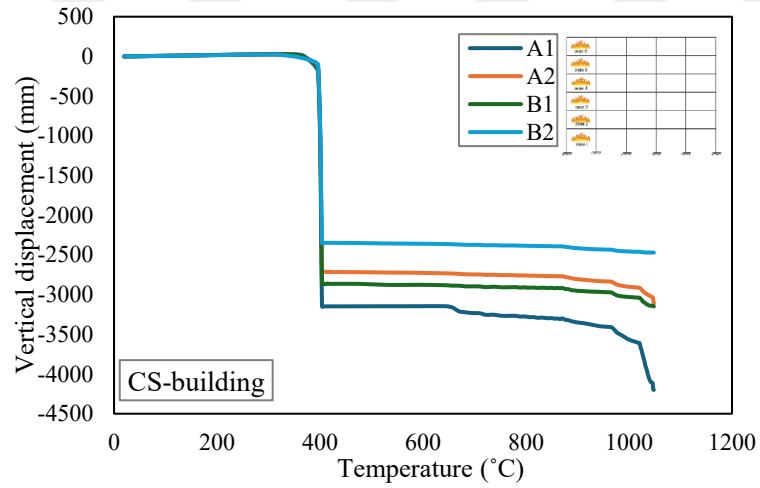


(d)

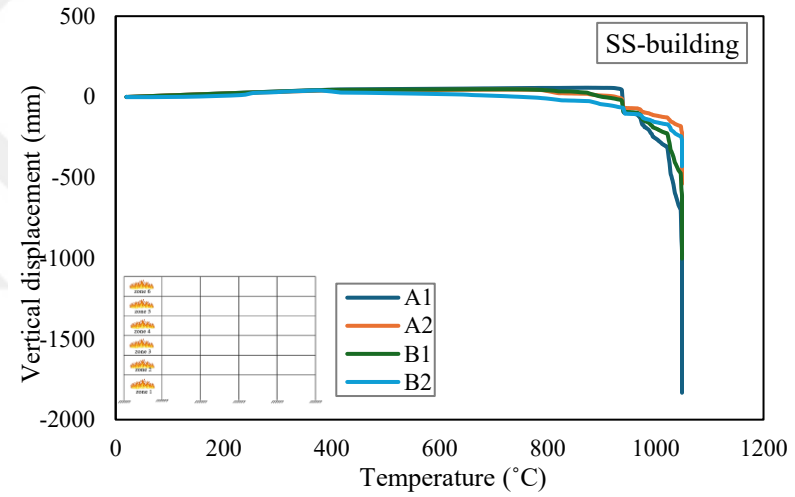
Figure 45. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-2 fire scenario

3.4.3. Case F-3

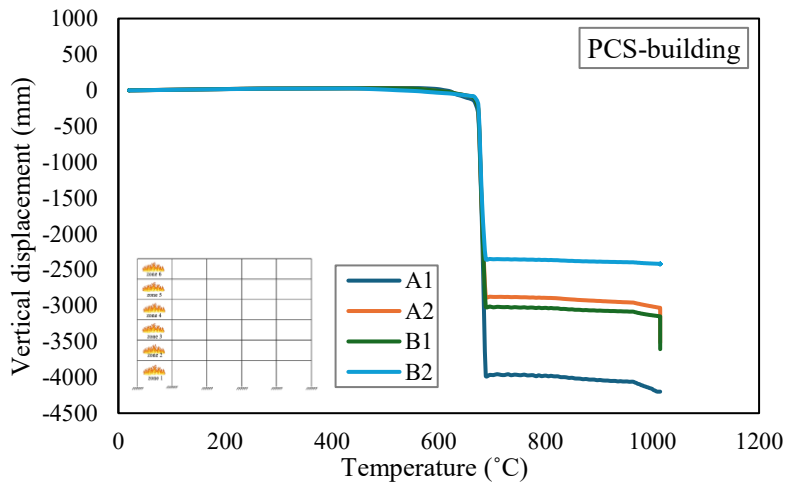
(Figure 46) presents the displacement-temperature responses of perimeter columns A1, A2 and interior columns B1, B2 under the corner travelling fire scenario (Case F-3). In the carbon steel (CS) building, all columns initially expand slightly up to $\sim 400^\circ\text{C}$, then quickly transition into severe downward displacement due to rapid strength and stiffness degradation. Column A1 shows the most dramatic shortening, exceeding 3000 mm, indicating early instability. The protected carbon steel (PCS) system follows a similar pattern, but the intumescent coating delays the onset of buckling to higher temperatures (600-700 $^\circ\text{C}$), without changing the abrupt failure mode. By contrast, the stainless steel (SS) frame maintains column stability up to $\sim 900\text{-}1000^\circ\text{C}$, with displacement magnitudes remaining lower than in CS or PCS. The hybrid (HYB) frame shows the most robust performance: columns undergo moderate expansion followed by gradual shortening only at high temperatures, even beyond 1000°C . These results confirm the advantage of advanced material configurations in delaying progressive failure and maintaining structural integrity under fire conditions.



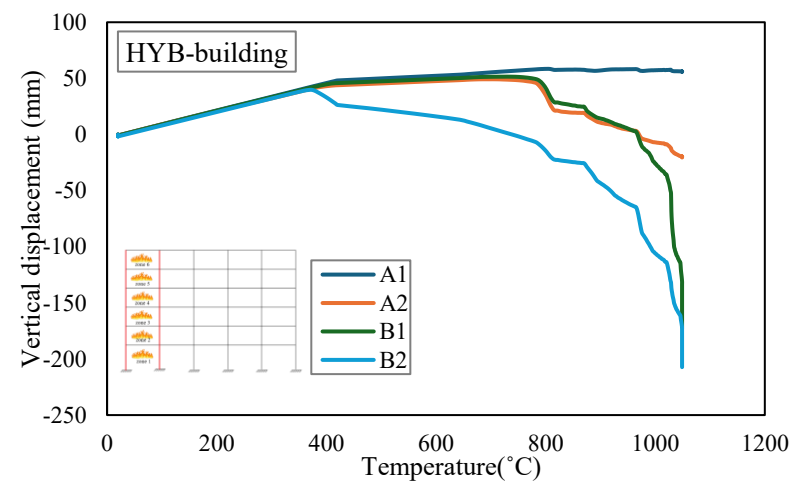
(a)



(b)



(c)

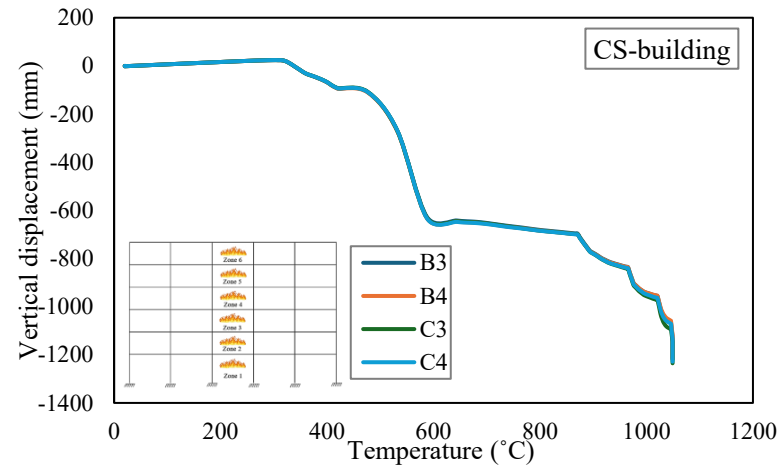


(d)

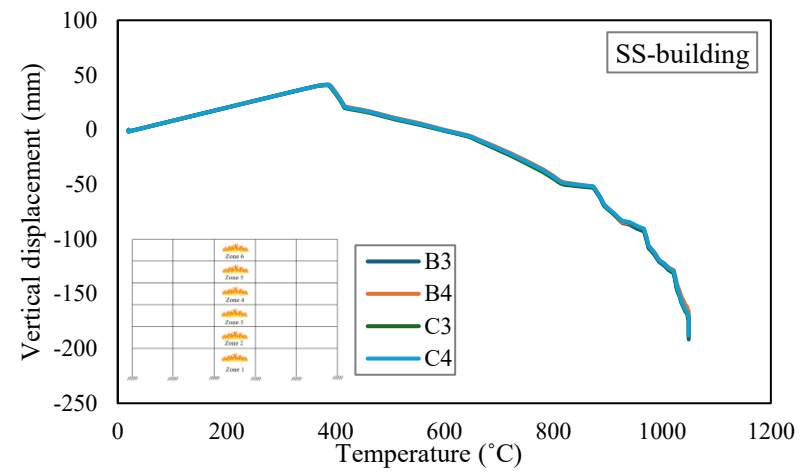
Figure 46. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-3 fire scenario

3.4.4. Case F-4

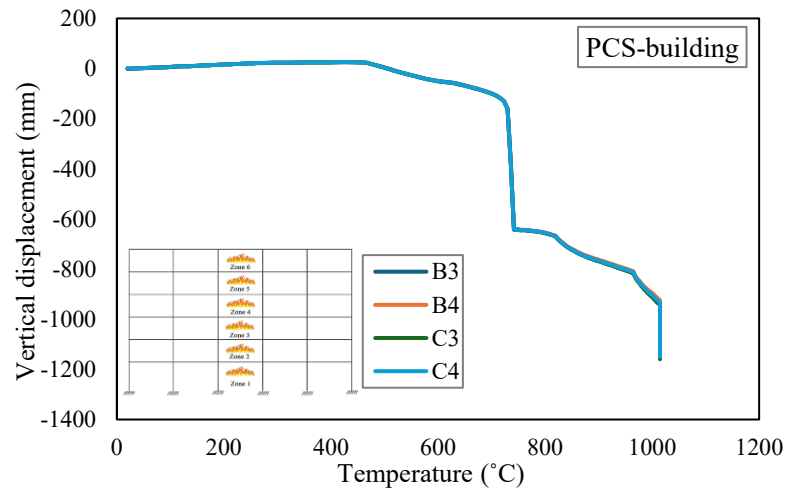
The displacement–temperature curves in (Figure 47) illustrate the behavior of columns B3, B4, C3, and C4 under Case F-4 fire exposure (central-vertical fire travelling). In the CS and PCS buildings, the columns show a short phase of slight expansion up to $\sim 200\text{--}300\text{ }^{\circ}\text{C}$, followed by progressive vertical shortening. Once steel strength deteriorates beyond $\sim 500\text{--}600\text{ }^{\circ}\text{C}$, displacement accelerates sharply, with failure occurring around $900\text{--}1000\text{ }^{\circ}\text{C}$ and maximum shortening exceeding $\sim 1200\text{ mm}$. The intumescent coating in the PCS system delays this instability, but the failure mechanism (buckling-driven failure mode) remains abrupt. By contrast, the SS and HYB-buildings display more stable and ductile responses: columns expand modestly to $\sim 300\text{--}400\text{ }^{\circ}\text{C}$ before gradual shortening sets in. Even at $1000\text{ }^{\circ}\text{C}$, displacements remain much smaller ($\sim -150\text{ to }-200\text{ mm}$), with no evidence of sudden loss of stability. These results confirm that while carbon steel systems-protected or not-are vulnerable to rapid instability once critical temperatures are reached, stainless steel and hybrid systems sustain load-carrying capacity longer and undergo less severe displacements, demonstrating superior fire resilience. Furthermore, a fire in the center of the building poses a lower risk than a fire in a corner in terms of progressive collapse action.



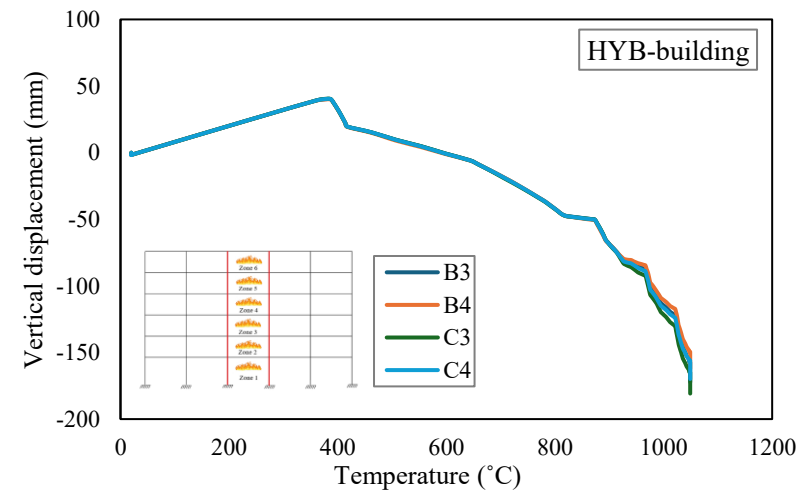
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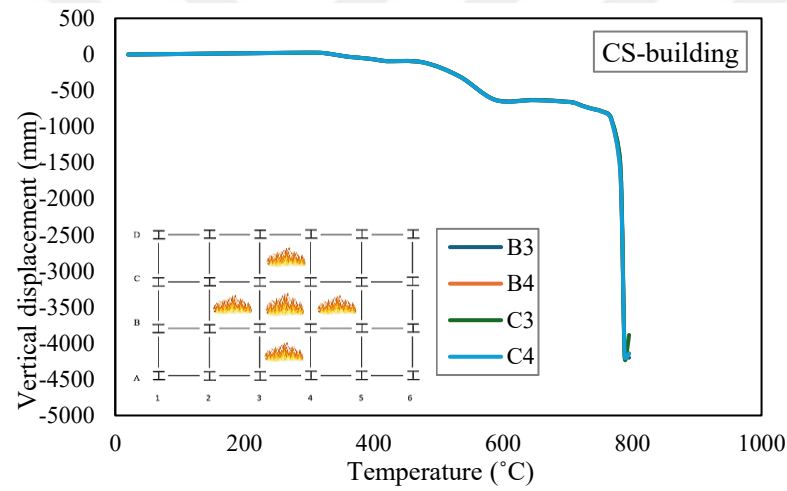
(d)

Figure 47. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-4 fire scenario

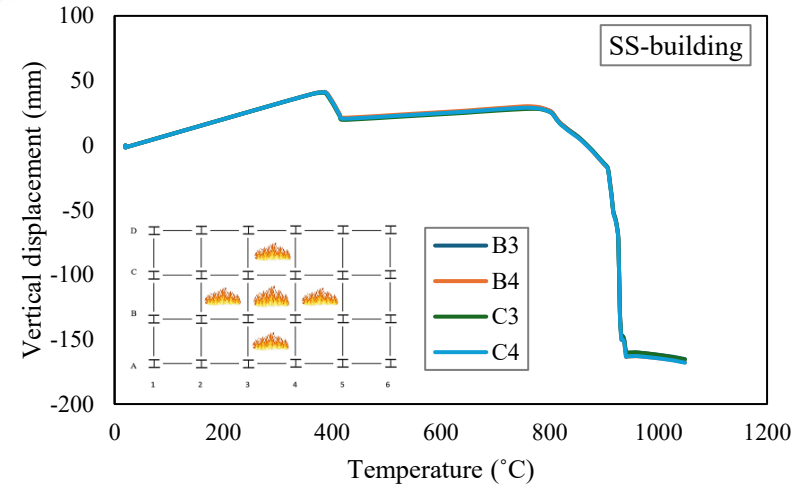
3.4.5. Case F-5

The vertical displacement-temperature curves in (Figure 48) (Case F-5, central-plus fire travelling) highlight distinct performance differences between the four material systems for columns B3, B4, C3, and C4. In the CS building, the columns exhibit negligible displacement until ~ 400 °C, followed by gradual shortening and then a sharp, catastrophic drop at ~ 750 - 800 °C, reaching displacements beyond -4000 mm. The PCS building shows a similar pattern but with collapse shifted slightly later, as the intumescent protection delays the critical temperature; however, once the coating's insulating capacity is exceeded, displacement accelerates rapidly to large negative values, indicating abrupt failure. In the SS building, the columns first expand modestly up to ~ 300 °C, then remain relatively stable with only gradual shortening; even near 1000 °C, maximum displacements remain limited to about -100 to -150 mm, demonstrating stainless steel's superior retention of mechanical properties at elevated temperatures. Finally, the HYB building shows intermediate performance: expansion is followed by delayed shortening, and while the onset of instability occurs around ~ 900 - 1000 °C, the magnitude of displacement (≈ -4000 to -4500 mm) is still significantly lower than in unprotected or protected buildings at earlier stages. Overall, these results confirm that carbon steel systems-whether bare or protected-are highly vulnerable to sudden collapse once critical temperatures are reached, whereas stainless steel and hybrid systems provide much greater robustness and delay the onset of instability.

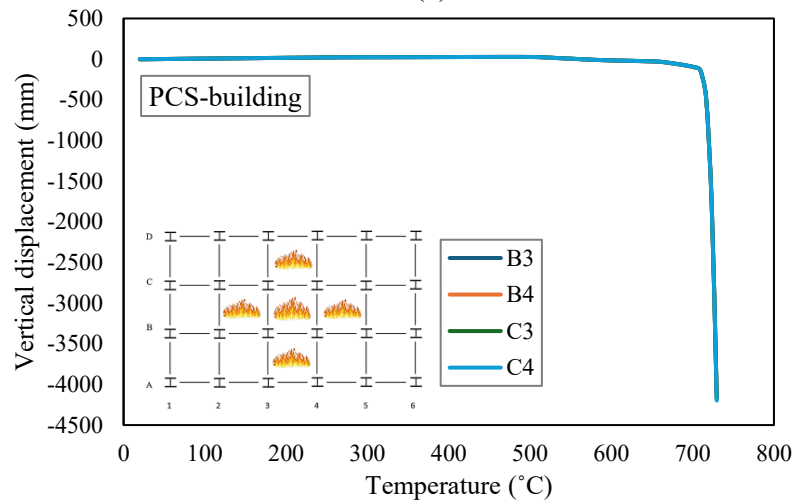
Bottom line; for these centrally heated interior columns, CS and PCS fail abruptly once critical temperature is reached (coating only buys time), whereas SS and HYB keep B3/B4/C3/C4 in a low-magnitude, ductile regime to much higher temperatures, with no single column governing due to nearly identical exposure and restraints.



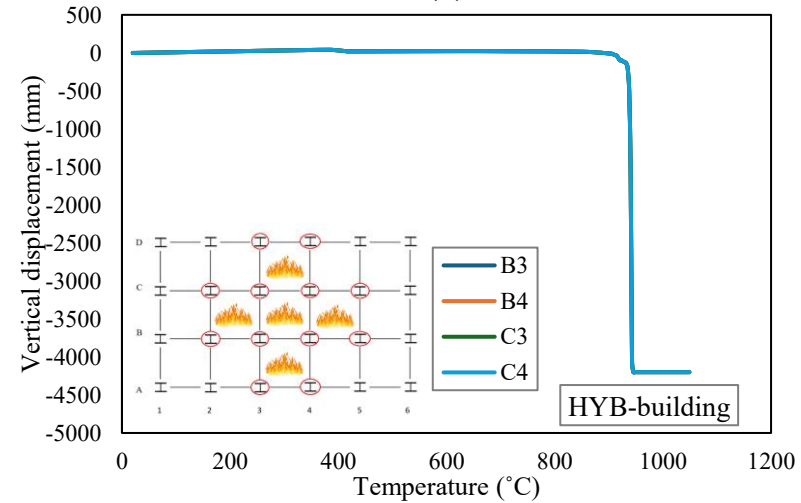
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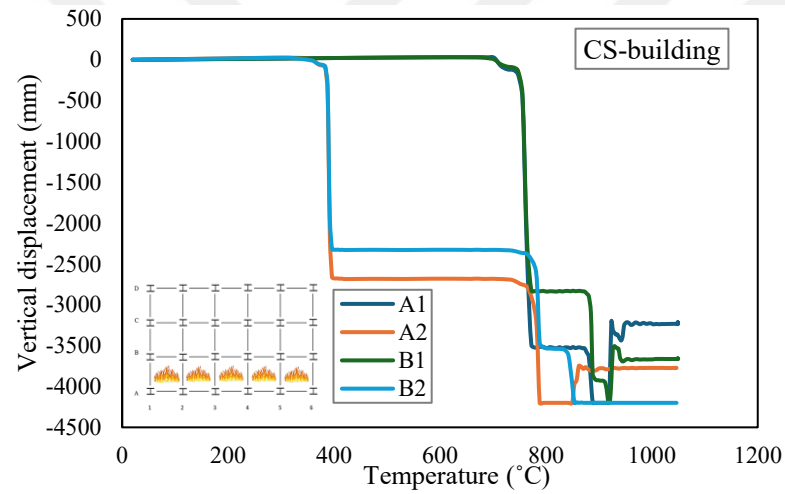
(d)

Figure 48. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-5 fire scenario

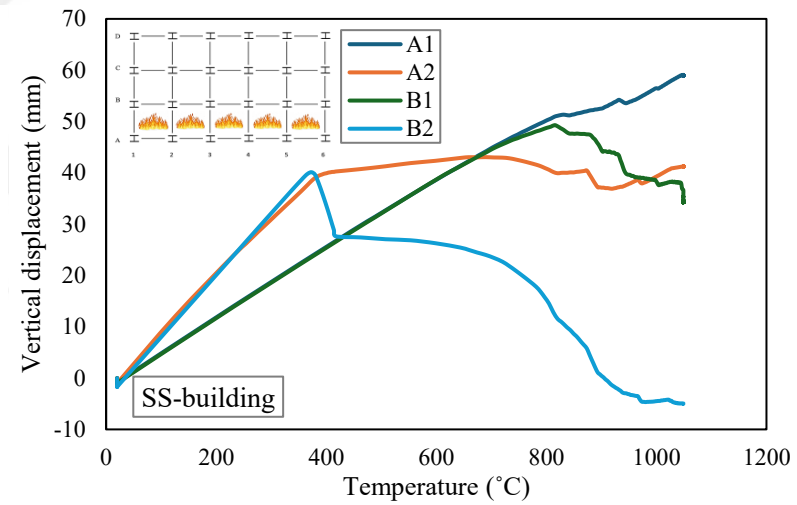
3.4.6. Case F-6

(Figure 49) shows the vertical displacement of columns A1, A2, B1, and B2 as a traveling fire moves east–west across five bays. In the CS and PCS buildings, columns A1, A2, B1, and B2 initially remain stable but then undergo sudden and large downward displacements once critical steel temperatures are reached ($\approx 600\text{--}800\text{ }^{\circ}\text{C}$), with abrupt drops exceeding 3000 to 4000 mm, indicating collapse. The intumescent protection in PCS delays the onset of failure compared with CS, but the failure mechanism remains abrupt. In contrast, the SS and HYB buildings display a much more ductile response: all four columns experience steady thermal expansion up to $\sim 300\text{--}400\text{ }^{\circ}\text{C}$, followed by gradual shortening. Even beyond $1000\text{ }^{\circ}\text{C}$, their displacements remain limited (tens of millimeters rather than thousands), and no sudden instability is observed. Both systems provide markedly improved stability compared with carbon steel building. These differences emphasize that while CS/PCS are vulnerable to rapid collapse once heated, SS and HYB retain stiffness and redistribute loads effectively, ensuring far greater robustness under severe fire exposure.

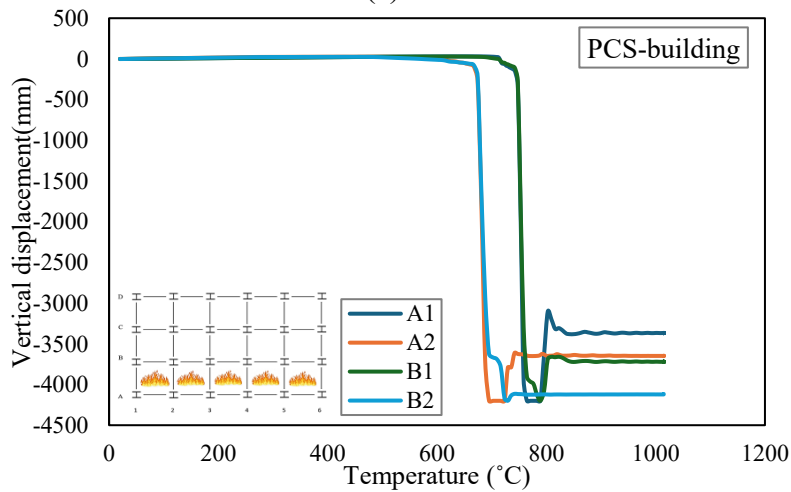
As in the previous cases, A1, A2, B1, and B2 fail abruptly in CS and PCS systems once the critical temperature is reached, whereas in SS and HYB systems they deform gradually and maintain stability for much longer—demonstrating that stainless and hybrid solutions are significantly more fire-resilient.



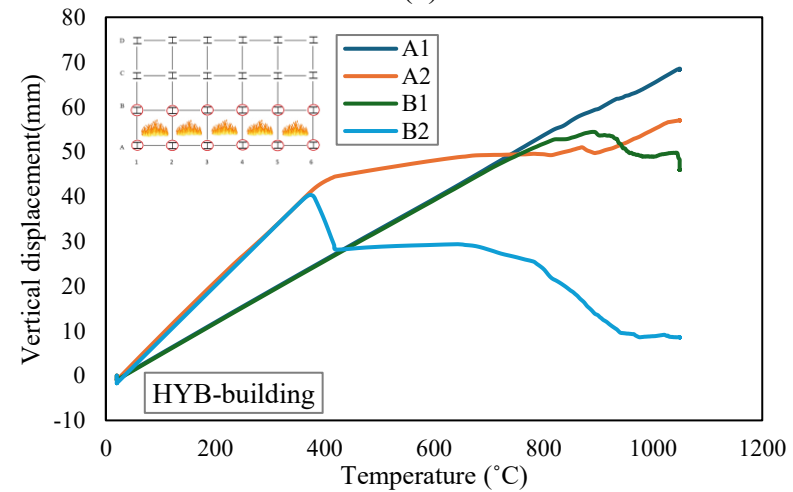
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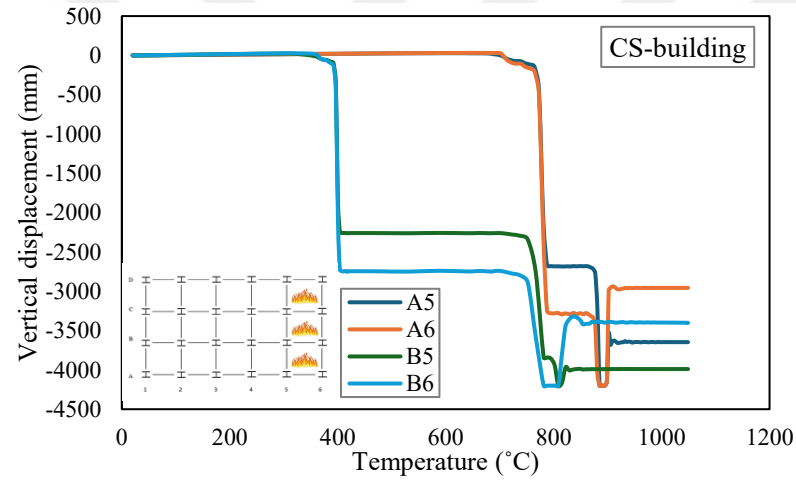


(d)

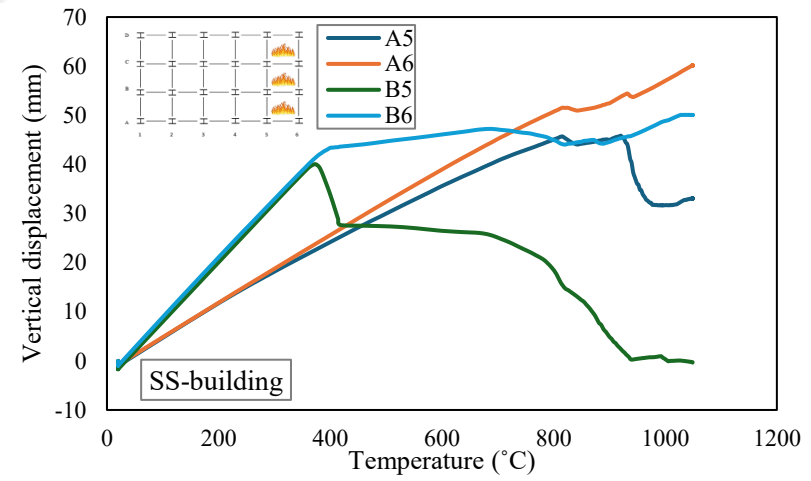
Figure 49. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-6 fire scenario

3.4.7. Case F-7

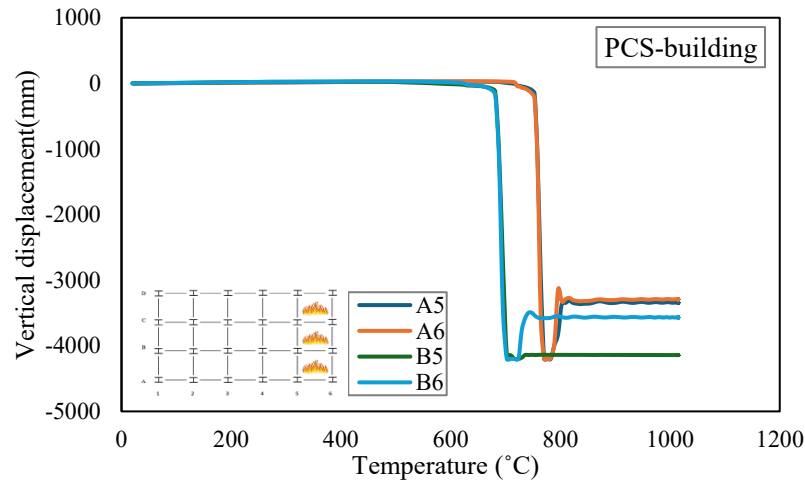
The results in (Figure 50) (Case F-7) reveal pronounced differences in the displacement-temperature responses of columns A5, A6, B5, and B6 across the four building systems. In the CS building, the columns remain stable only up to $\sim 400\text{-}500\text{ }^{\circ}\text{C}$, after which they experience sudden and severe vertical shortening, with multiple abrupt drops exceeding 3000 mm, indicating sudden failure. The PCS building shows the same collapse mechanism but with a modest delay, as the intumescent coating temporarily slows heating; however, once the coating is exhausted, the columns still undergo sharp, simultaneous displacements. By contrast, the SS-building exhibits a ductile response: all four columns expand steadily with rising temperature, reaching elongations of 40-60 mm, before gradually shortening at higher temperatures, with no abrupt instability. The HYB building shows similar behavior to SS, though B-columns (B5, B6) tend to accumulate slightly larger displacements than A-columns, reflecting load redistribution effects. Overall, while CS and PCS columns fail suddenly and catastrophically once critical temperatures are exceeded, SS and HYB systems provide gradual, stable deformation and sustain structural capacity much longer, confirming their superior fire resilience.



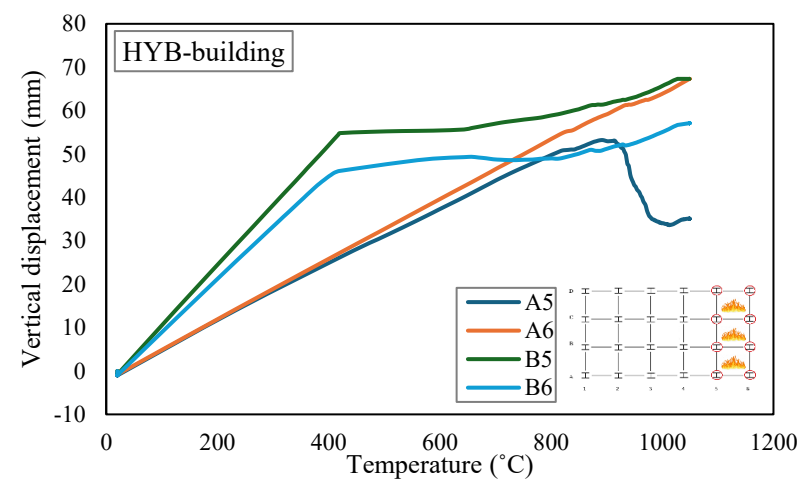
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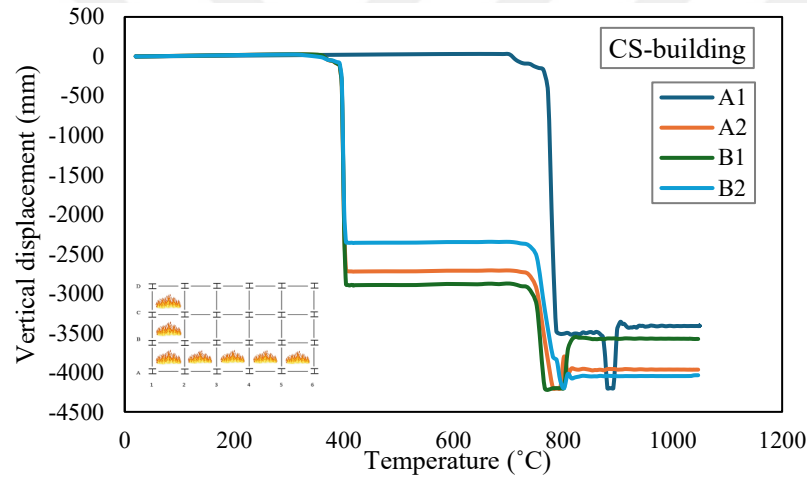
(d)

Figure 50. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-7 fire scenario

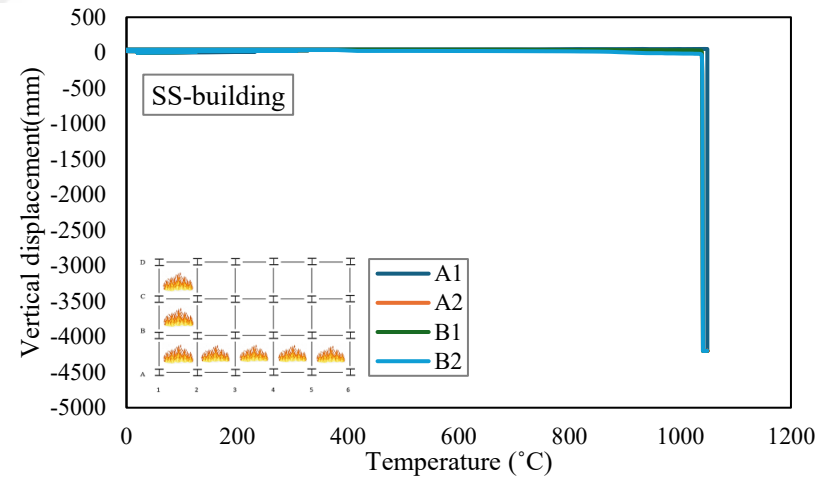
3.4.8. Case F-8

The displacement–temperature curves in (Figure 51) (Case F-8, L-shaped traveling fire) show distinct differences among the four building systems. In the CS- and PCS-buildings, columns A1, A2, B1, and B2 undergo limited movement at low temperatures, but once steel properties degrade beyond $\sim 500\text{--}600\text{ }^{\circ}\text{C}$, they experience sudden and severe shortening, with abrupt drops of ~ 3000 to ~ 4500 mm. PCS protection slightly delays this instability, but the collapse mechanism remains abrupt and brittle. In contrast, the SS and HYB buildings exhibit a markedly different response: their columns remain almost unaffected up to very high temperatures ($\sim 900\text{--}1000\text{ }^{\circ}\text{C}$), after which failure occurs suddenly but with much less cumulative shortening. Compared to the gradual, ductile deformation seen in earlier cases, the SS and HYB systems here reveal greater resistance to heating but ultimately undergo a more synchronized, late-stage instability. These results highlight that while CS/PCS are prone to early catastrophic collapse, SS and HYB systems maintain stability much longer, though under this specific fire scenario their failure still occurs in a sudden, collective manner at extreme temperatures.

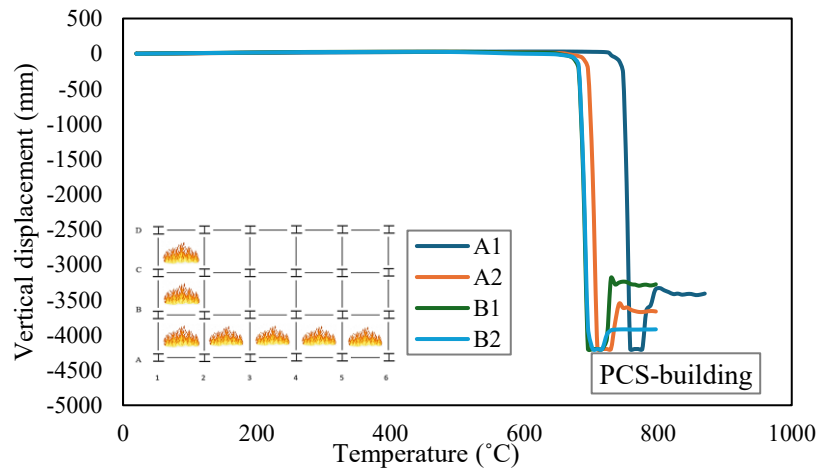
Across the eight fire scenarios, the collapse situation is strongly governed by the location of fire within the structural frame. Edge fires (F-1, F-3, F-6, F-7) are particularly critical, since perimeter columns provide both vertical load-bearing and lateral stability; once these members are heated, redistribution options are limited, leading to rapid and often abrupt collapse in carbon steel and only slightly delayed failure in protected carbon steel. Middle fires (F-2, F-4) expose interior columns simultaneously, causing severe displacements and global instability, though stainless steel and hybrid systems show improved ductility and delayed failure through catenary action. The distributed or multi-fire cases (F-5, F-8) represent the most dangerous condition overall: multiple columns are heated at once, resulting in near-simultaneous failures in carbon and protected steel, while stainless and hybrid systems still delay collapse but eventually lose stability under extreme uniform heating. These comparisons confirm that while material type governs failure mode, the location and extent of fire exposure are also decisive in determining the onset and severity of collapse.



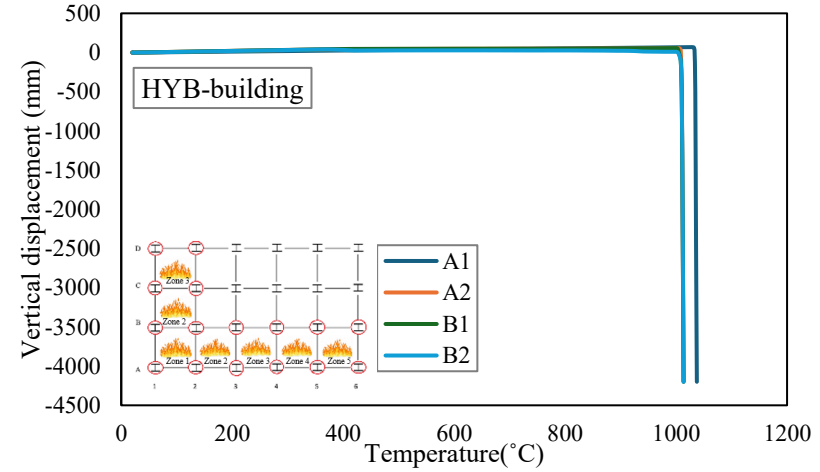
(a)



(b)



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Figure 51. The vertical displacement-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-8 fire scenario

3.5. Axial-Load Histories

This section presents the axial load distribution of four buildings (CS, SS, PCS and HYB) under eight fire scenarios. Each scenario was detailed as follows.

3.5.1. Case F-1

The graphs in (Figure 52) compare the axial force–temperature responses of columns A1, A2, B1, and B2 in CS, PCS, SS, and HYB buildings under the Case F-1 edge-fire scenario. In the CS and PCS buildings, the axial forces in fire-exposed columns (especially B2) decrease rapidly from around -1800 kN at ambient conditions to nearly -2600 kN at about 600 °C, followed by a sudden recovery toward 0 kN near 750-800 °C before complete failure. This highlights their abrupt and unstable load redistribution behavior. By contrast, the SS and HYB buildings show a much more gradual force reduction, with the axial force in B2 decreasing steadily to about -2200 kN near 800 °C, then recovering only gradually beyond 900 °C. The decline in axial force is less severe, and the recovery occurs later and in a smoother manner compared to CS and PCS. This indicates that stainless steel and hybrid systems can sustain and redistribute loads more effectively under fire, while carbon steel and protected carbon steel systems are prone to sudden instability once critical temperatures of around 600-800 °C are reached.

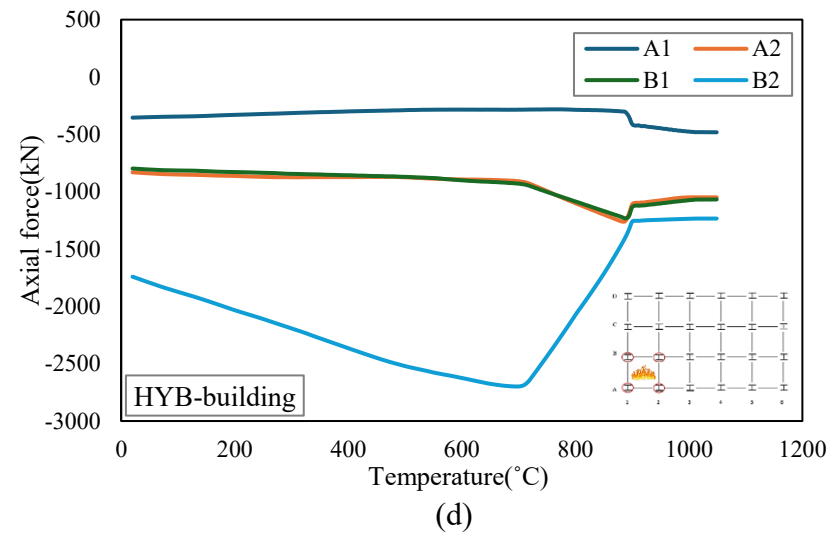
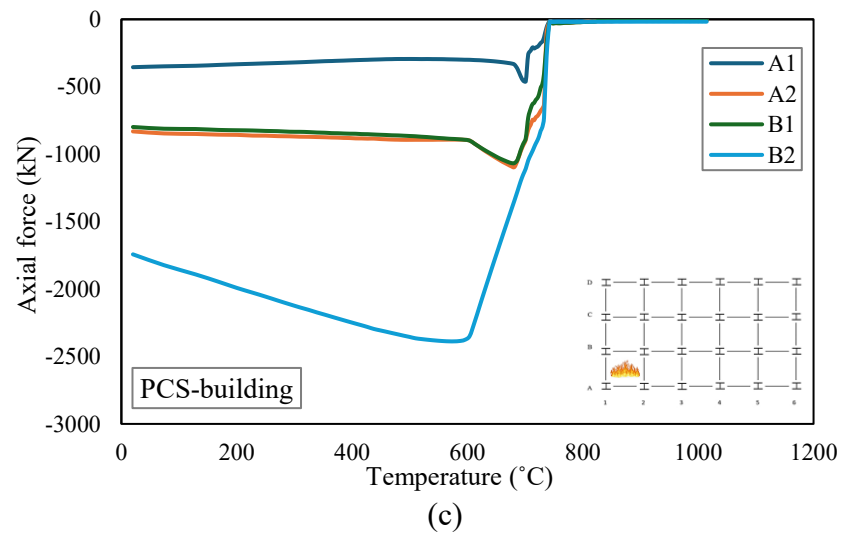
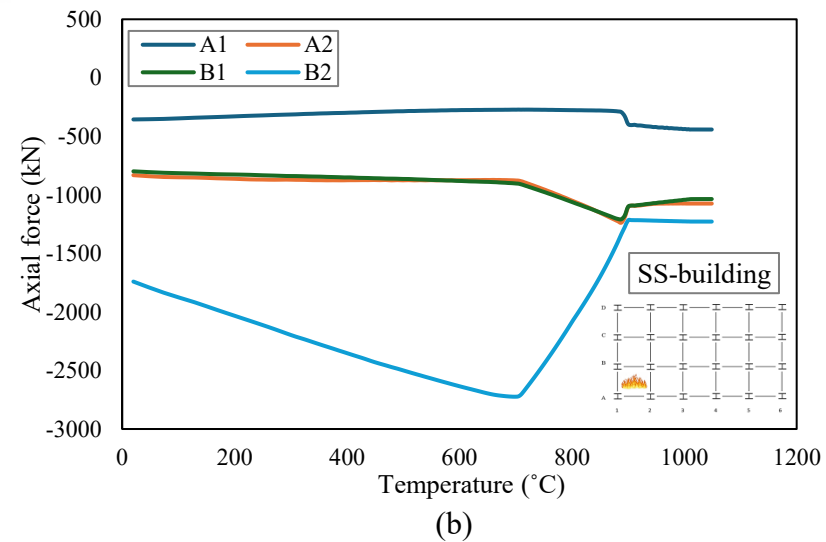
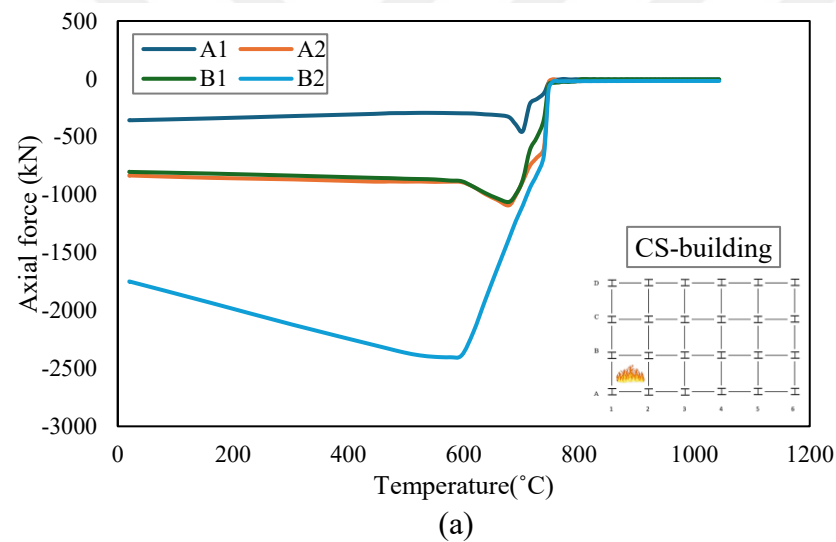
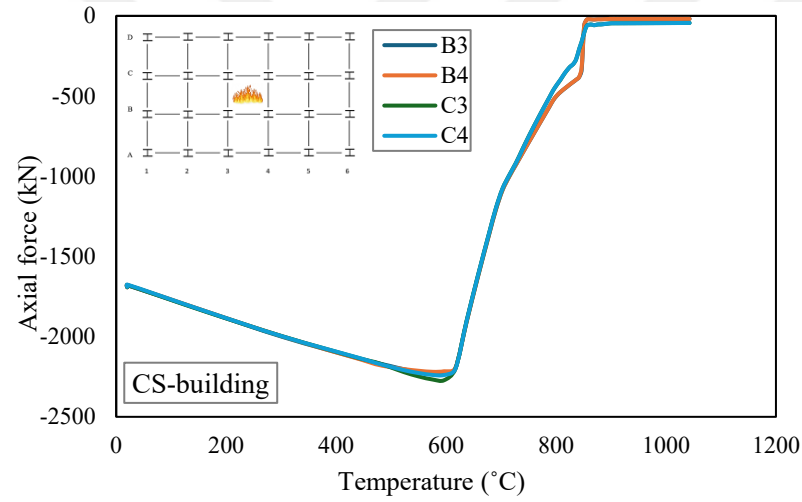


Figure 52. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-1 fire scenario

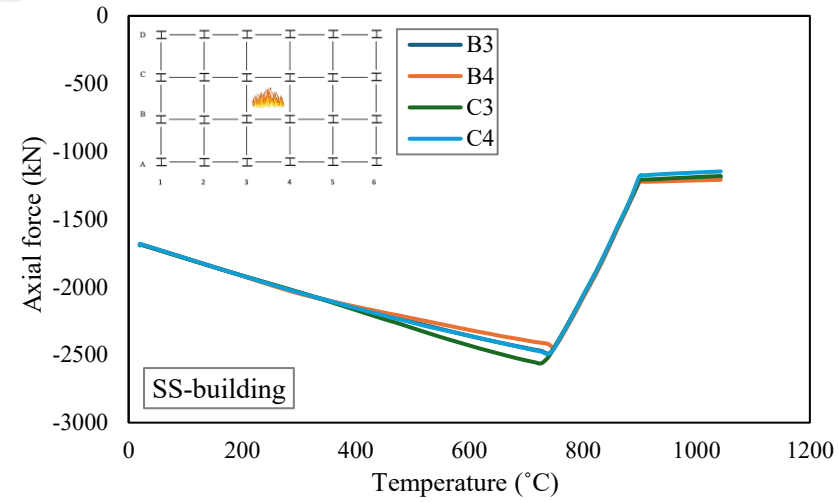
3.5.2. Case F-2

The graphs in (Figure 53) illustrate the axial force-temperature responses of columns B3, B4, C3, and C4 in CS, PCS, SS, and HYB buildings under the Case F-2 middle-fire scenario. In the CS and PCS buildings, the axial force in fire-exposed columns decreases rapidly from about -1600 kN at ambient to nearly -2200 kN by 600 °C, before a sharp recovery to 0 kN around 800-850 °C, reflecting abrupt instability and sudden redistribution. Also, none of the columns shift into tension, which means no catenary action develops, and the gravity loads are carried mainly by the cooler surrounding bays. By contrast, the SS and HYB buildings display a more controlled response: forces decline gradually to about -2000 to -2300 kN at 700-800 °C and then recover smoothly, stabilizing near -1200 to -1000 kN at 1000 °C. The smoother trends in SS and HYB systems indicate greater ductility and redistribution capacity, whereas CS and PCS systems show brittle-like behavior with a sudden force drop and recovery. These results highlight that in middle-fire scenarios, stainless steel and hybrid structures maintain significantly higher residual load-bearing capacity compared to carbon steel-based systems.

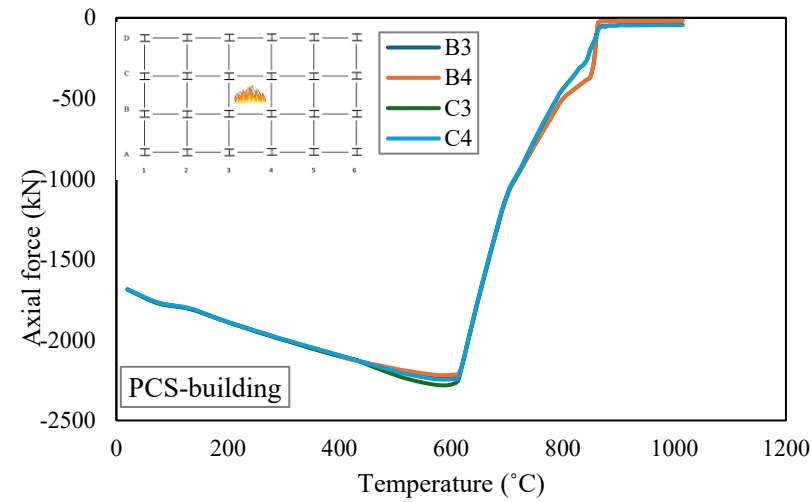
Overall, edge-fires scenario are structurally more critical than middle-fire for CS and PCS systems, as the simultaneous loss of multiple interior columns causes rapid global instability. By contrast, SS and HYB systems demonstrate stable redistribution and retain significant axial capacity under both edge- and middle-fire conditions, confirming their superior fire resilience.



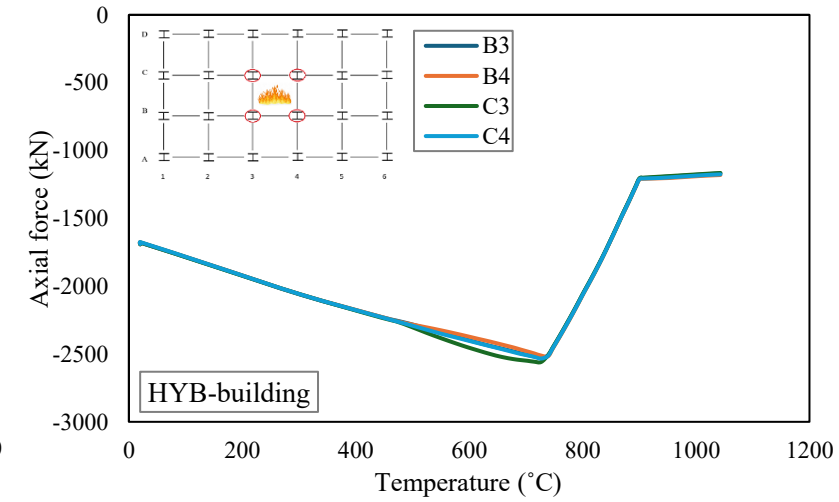
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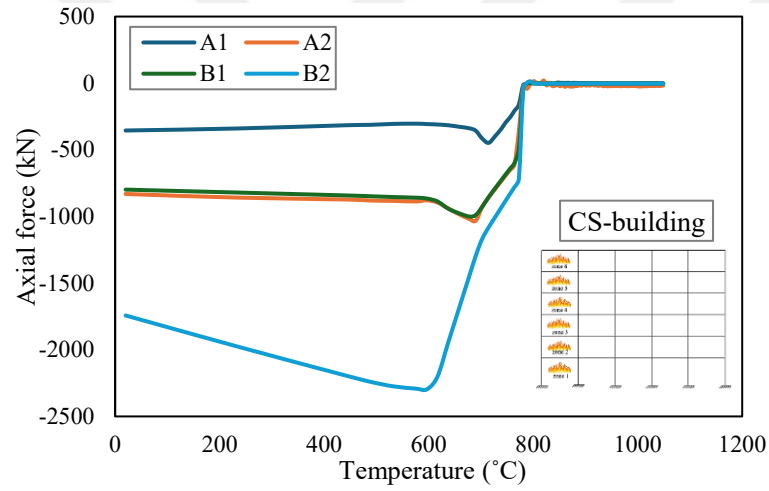


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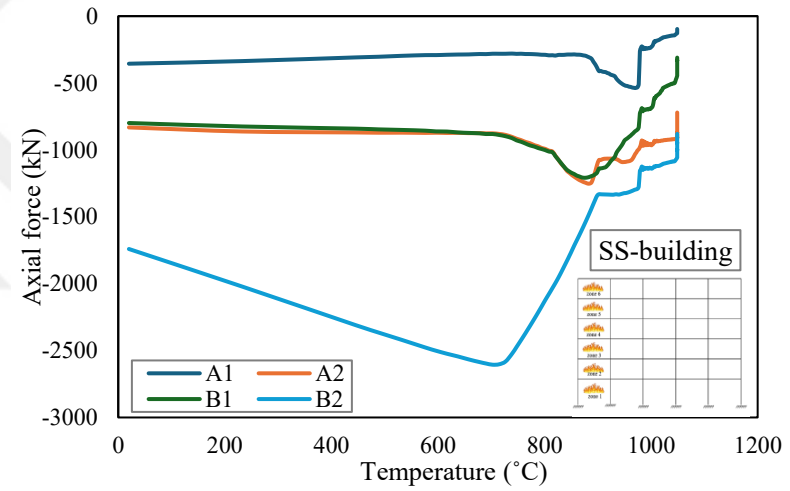
Figure 53. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-2 fire scenario

3.5.3. Case F-3

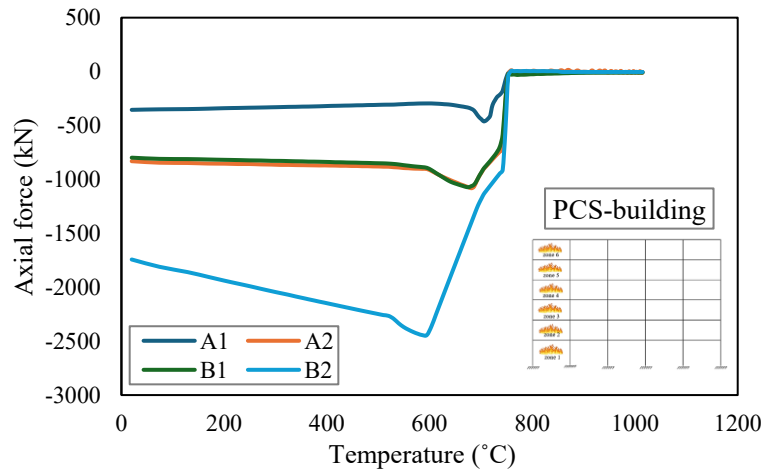
In (Figure 54), the axial force–temperature responses of columns A1, A2, B1, and B2 under the Case F-3 edge-fire scenario reveal distinct material-dependent behaviors. In the CS and PCS buildings, the fire-exposed column B2 experiences a steep axial force reduction from approximately -1700 kN down to nearly -2300 kN by around 650 °C, followed by an abrupt recovery toward zero, indicating sudden and unstable redistribution. In addition, catenary action can be observed with a short tensile response, but the effect is weak and provides little tying capacity. By contrast, the SS and HYB buildings show a more gradual decline in axial forces: in SS, the reduction extends to about -2600 kN near 750 °C, but the recovery is smoother and more progressive compared to CS/PCS. Similarly, in the HYB case, axial force decreases to around -2600 kN and redistributes steadily, delaying collapse to nearly 1000 °C. The catenary action is more dominant for SS and HYB buildings. These results highlight that while CS and PCS systems are prone to sudden force drops and abrupt instability at lower temperatures (~600-700 °C), SS and HYB structures maintain axial load capacity for longer durations, showing enhanced resilience and more controlled redistribution under edge-fire conditions.



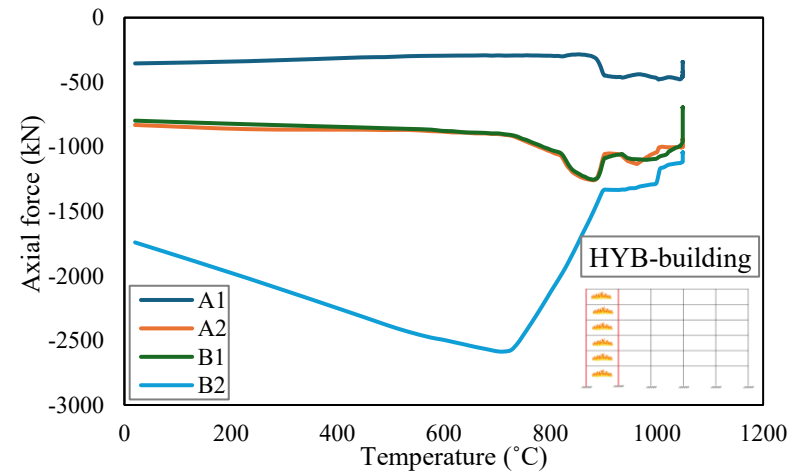
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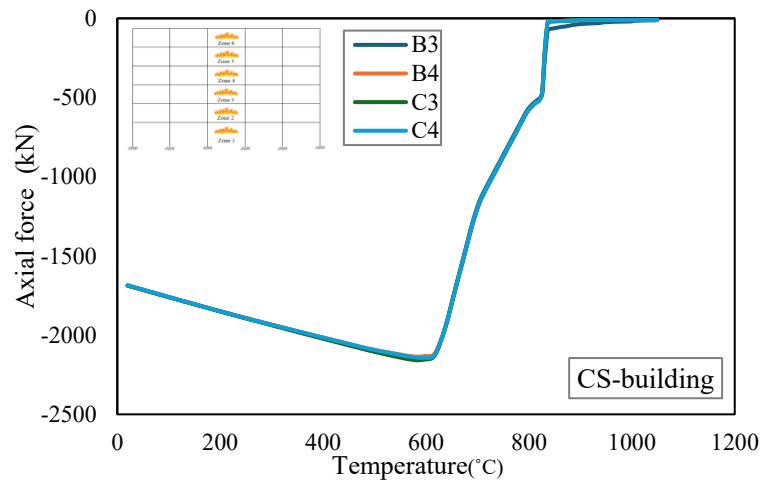
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Figure 54. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-3 fire scenario

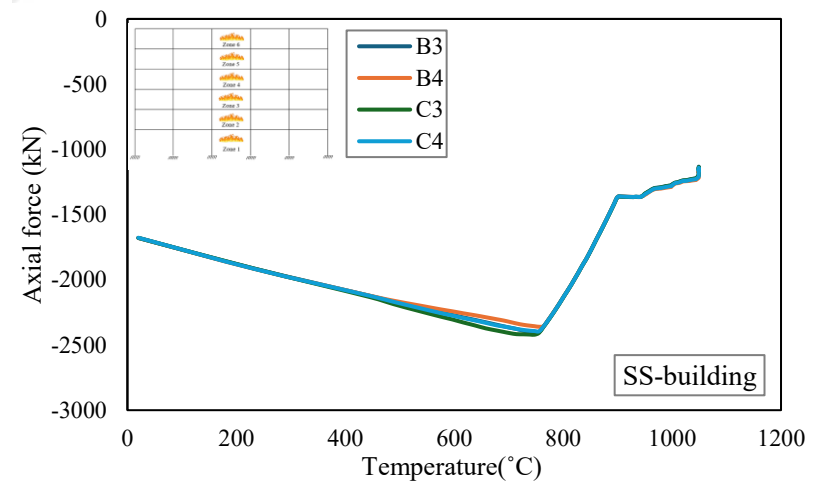
3.5.4. Case F-4

The axial force responses of centrally located columns in Case F-4 (Figure 55) highlight the sharp contrast between carbon steel systems (CS and PCS) and stainless-based systems (SS and HYB). In the CS building, columns begin around -1690 kN and rise to nearly -2150 kN by 600 °C as restrained thermal expansion increases compressive demand. Beyond 650 °C, the steel softens rapidly, and by 800 °C axial forces drop to only -40 to -70 kN, less than 5% of their peak, leaving the frame almost entirely dependent on catenary floor action for stability. The PCS building follows the same pattern with insulation only delaying the critical drop. Peak forces reach about -2260 kN at 600 °C, but once the coating fails, axial capacity collapses within a narrow ~200 °C window, stabilizing below -100 kN-again under 5% of peak load. In contrast, the SS building shows far superior stability: axial forces rise nearly linearly to ~-2400 kN at 750 °C and, even during unloading between 780-930 °C, the columns retain ~-1200 kN at 1050 °C (~50% of peak). The HYB system, with stainless steel in the fire-exposed columns, performs comparably to SS: peak values near -2400 kN at 750 °C, unloading gradually to ~-1150 kN beyond 1000 °C, without any column entering tension.

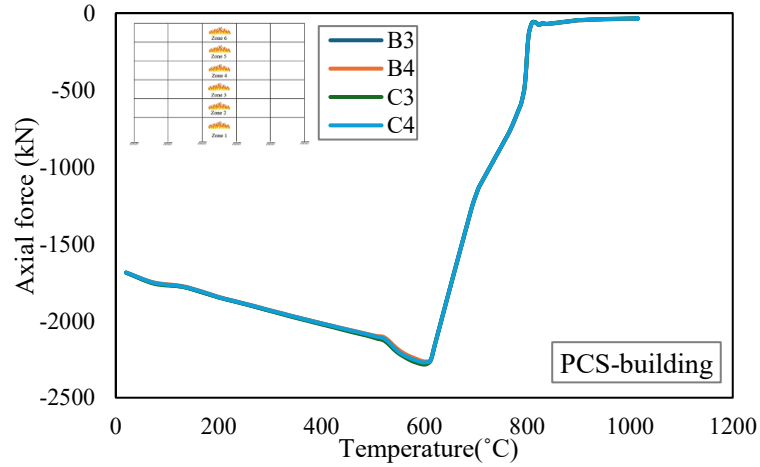
In summary, CS and PCS systems experience rapid loss of axial capacity and depend on catenary floor action, making them highly vulnerable to early collapse. By contrast, SS and HYB systems retain substantial compressive resistance (40-50% of peak loads) even at >1000 °C, demonstrating far greater robustness under severe fire exposure.



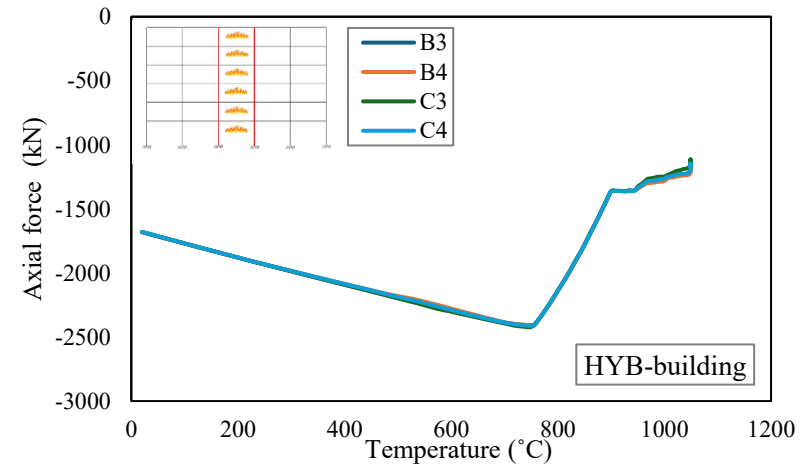
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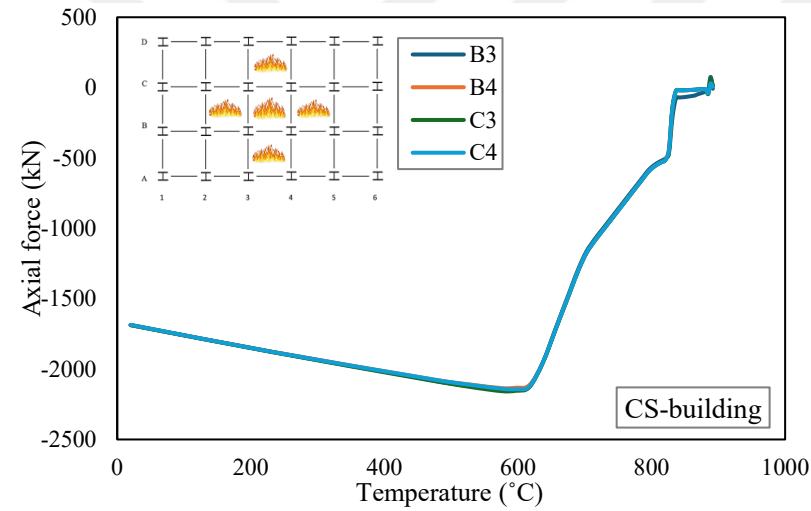


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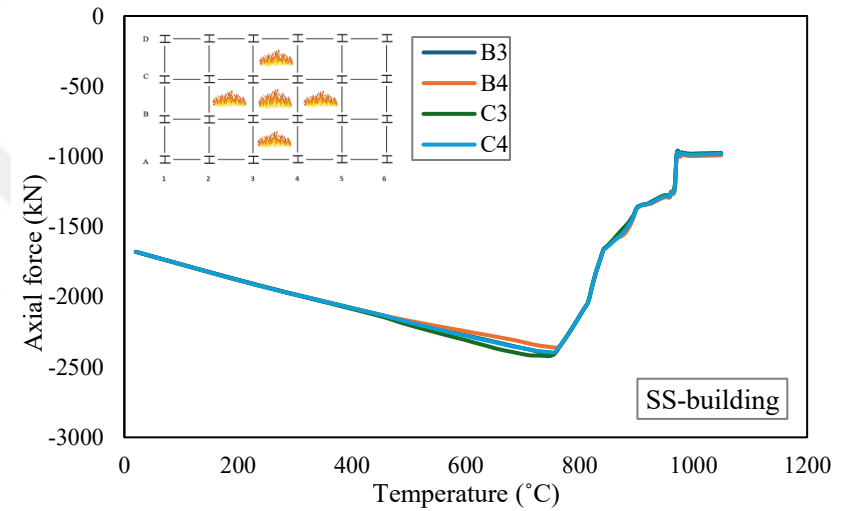
Figure 55. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-4 fire scenario

3.5.5. Case F-5

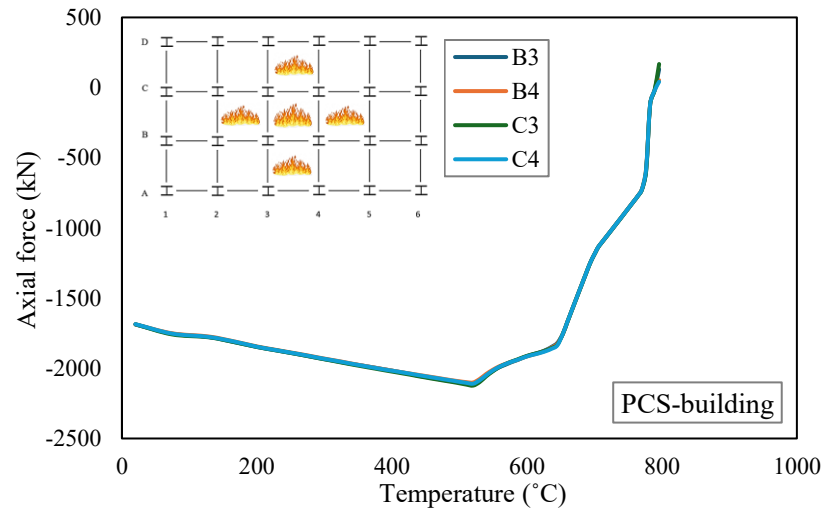
In Case F-5, where a plus-shaped multi-fire scenario is applied, the axial force histories of the centrally located columns demonstrate sharp contrasts between the four material systems (Figure 56). In the CS building, column forces rise from about 1690 kN to nearly 2170 kN by 600 °C, but then drop rapidly once the beams begin to soften beyond 650 °C. Within a short temperature range, compression is almost entirely lost, with some columns even shifting into light tension, which highlights the severe vulnerability of bare steel. The PCS building delays this progression but ultimately follows the same mechanism: peak loads of around 2110-2130 kN are reached before 550 °C, after which forces decrease steeply, in some cases passing into tensile values after 700-800 °C. By contrast, SS maintains stability much more effectively, with forces increasing to about 2400 kN near 750 °C and still sustaining close to 1000 kN even beyond 1000 °C, without any transition into tension. The HYB building mirrors the stainless-steel response, as its fire-exposed stainless components carry high compressive loads up to 2400 kN and then unload gradually to residual values above 200 kN by 1050 °C, while avoiding tensile instability. Overall, these results confirm that stainless and hybrid systems retain significant axial capacity and structural integrity under multi-fire exposure, whereas carbon steel systems, whether protected or not, exhibit rapid strength loss and depend heavily on catenary floor action for survival.



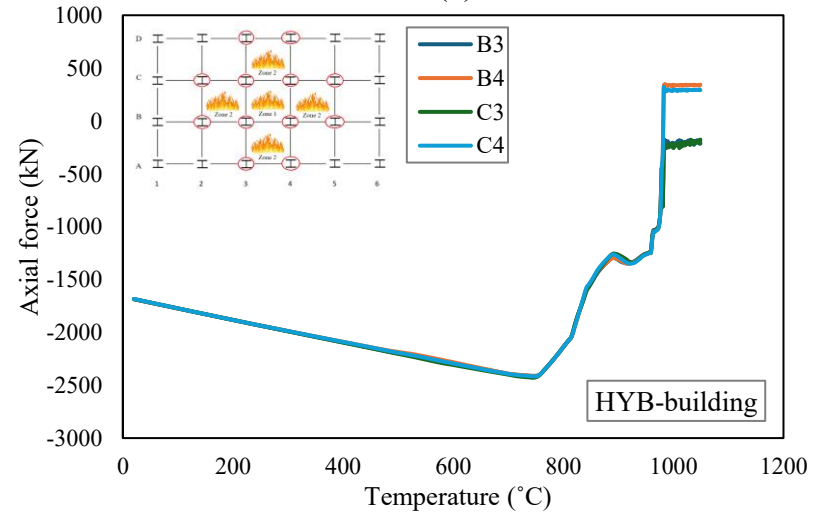
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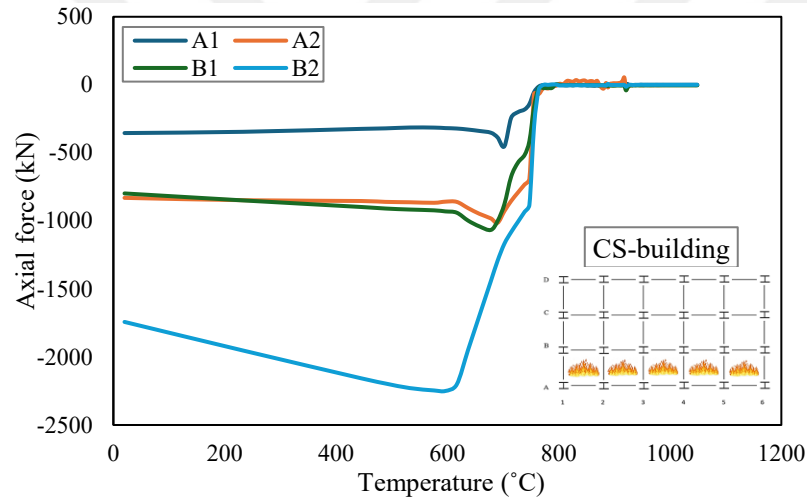


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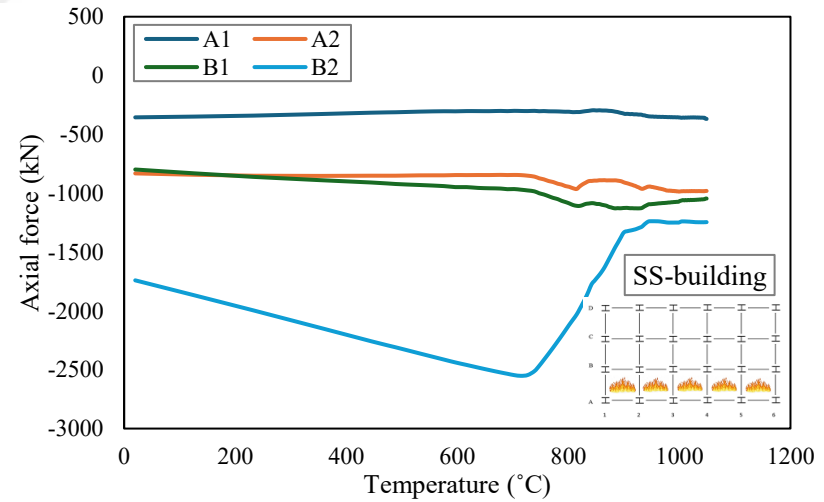
Figure 56. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-5 fire scenario

3.5.6. Case F-6

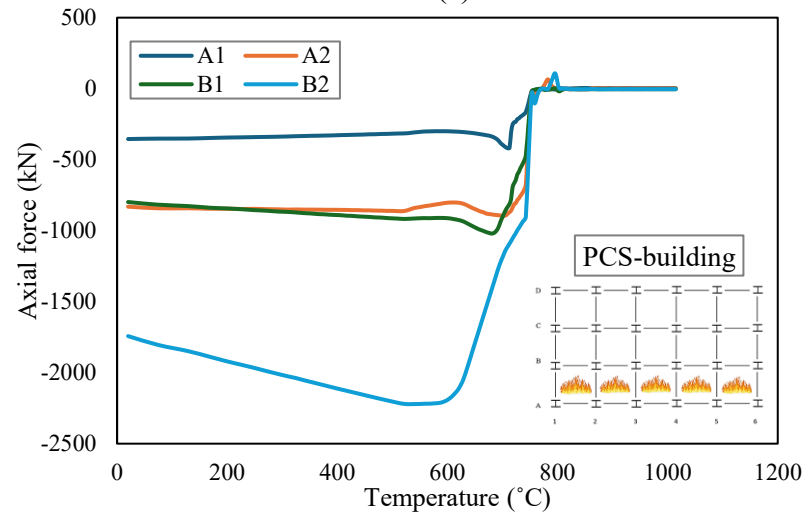
(Figure 57) illustrates the axial force responses of columns A1, A2, B1, and B2 under the Case F-6 travelling fire scenario moving eastward across five bays. In the CS building, column forces increase sharply, reaching peak compression around 600-700 °C before unloading rapidly as material softening occurs. The interior column B2, which initially carries the highest load, loses almost all compressive resistance and briefly enters tension, indicating severe instability and reliance on floor catenary action for load transfer. The PCS building exhibits a similar pattern but with delayed onset: compressive strength is maintained until roughly 680-700 °C, after which unloading is abrupt and tensile phases again appear in B2. By contrast, the SS system retains significant compressive resistance throughout heating. Even after 1000 °C, all columns remain in compression, and none show instability, confirming the superior high-temperature performance of stainless steel. The HYB building behaves comparably, as its stainless fire-zone columns sustain compression while surrounding carbon steel members redistribute loads safely. Overall, the results demonstrate that both unprotected and protected carbon steel frames are prone to abrupt collapse once critical temperatures are reached, whereas stainless and hybrid systems maintain axial stability, offering far greater fire resilience under a travelling fire.



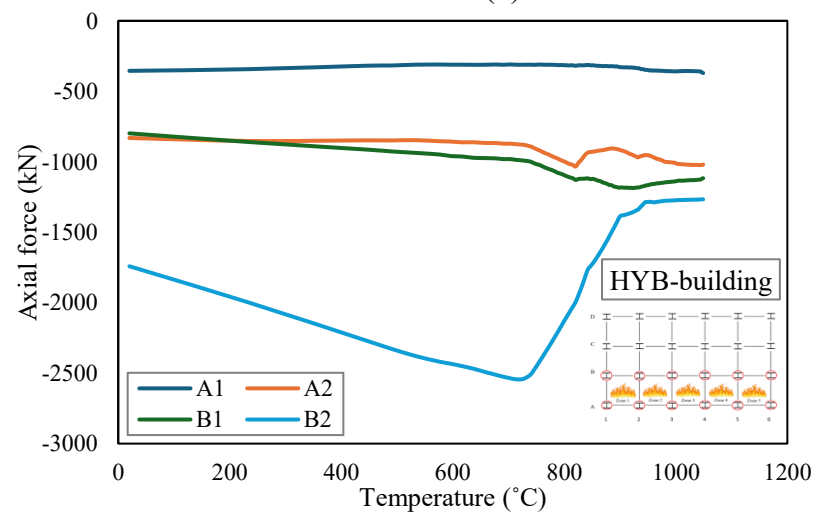
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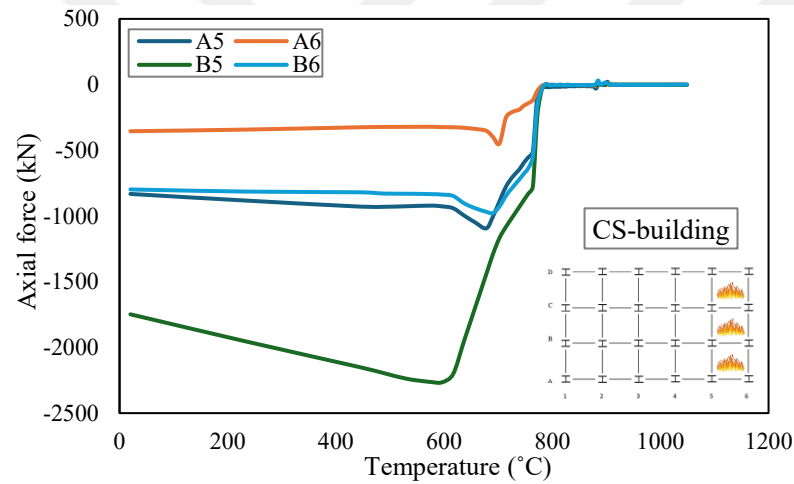


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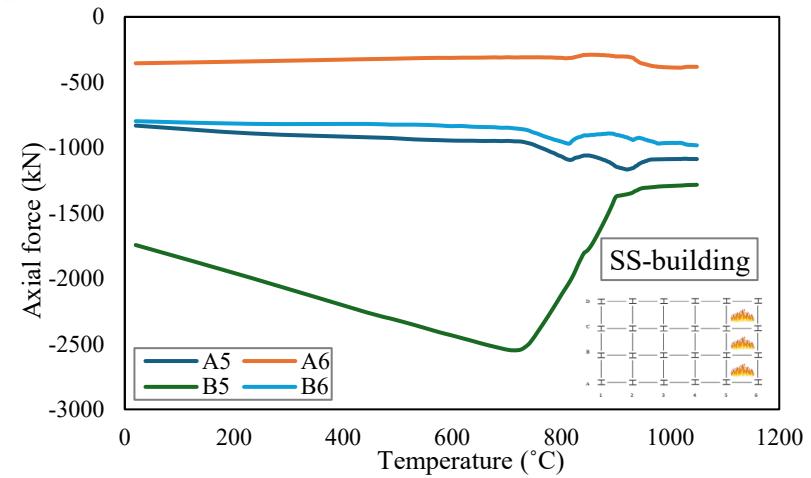
Figure 57. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-6 fire scenario

3.5.7. Case F-7

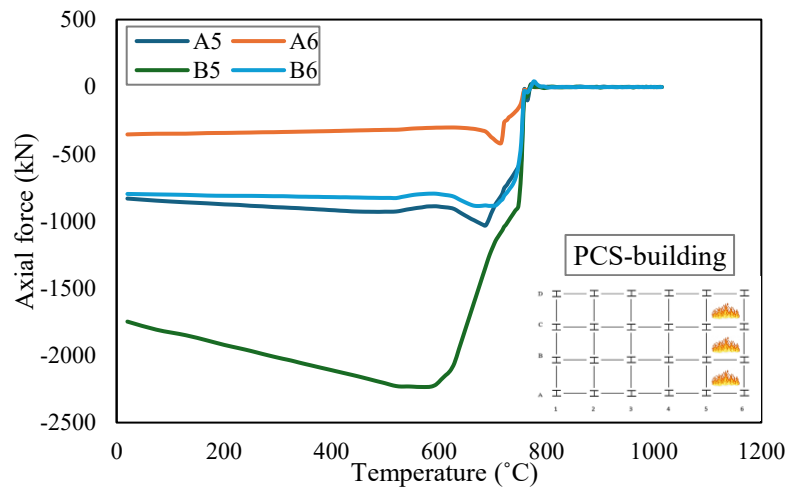
(Figure 58) shows the axial force-temperature responses of columns A5, A6, B5, and B6 under the Case F-7 corner fire scenario. In the CS building, compression rises initially, with the most loaded column (B5) reaching about -1740 kN, but rapid unloading begins beyond 600 °C. By 770 °C, the edge columns (A5 and A6) have lost nearly all axial force, while interior columns (B5 and B6) approach zero or slightly tensile states by 900 °C. This abrupt reduction highlights the high collapse risk of bare steel frames. The PCS building exhibits the same pattern but with delayed heating: column B5 peaks above -2200 kN near 600 °C before unloading sharply. By 770-800 °C, several members again pass through short tensile phases, showing that insulation slows degradation but does not alter the ultimate failure mode. In contrast, the SS system maintains stability. Column B5 reduces gradually from -1740 kN to -1280 kN at 1050 °C, while A6 still carries around -380 kN; no member moves into tension. The HYB building, with stainless fire-zone columns, behaves similarly: B5 peaks near -2540 kN before unloading steadily to ~-1280 kN, while all other columns remain in compression. Overall, CS and PCS frames suffer abrupt capacity loss and rely heavily on floor catenary action, whereas SS and HYB frames preserve significant axial resistance and demonstrate far superior resilience under corner fire exposure.



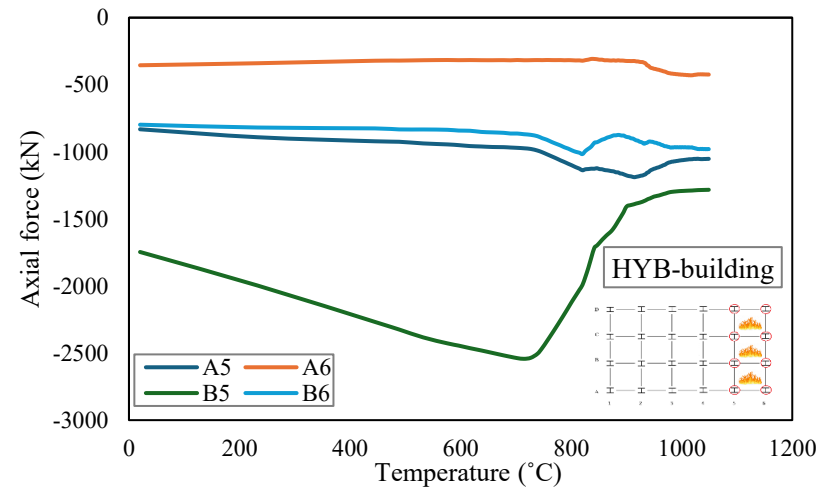
(a)



(b)



(c)



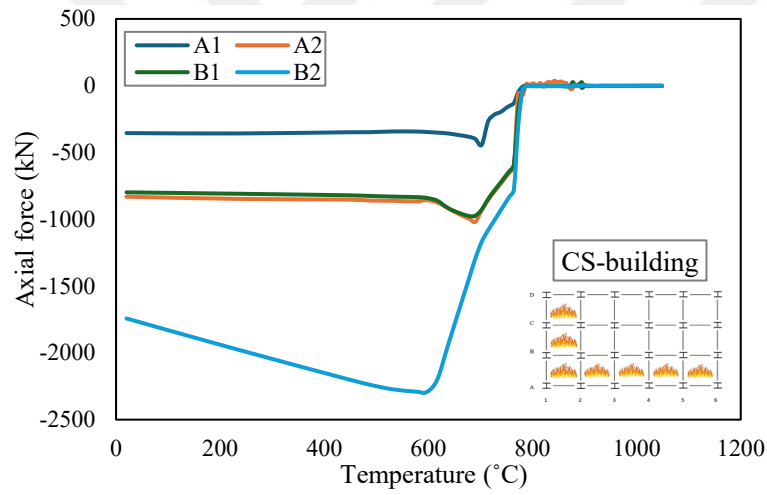
(d)

Figure 58. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-7 fire scenari

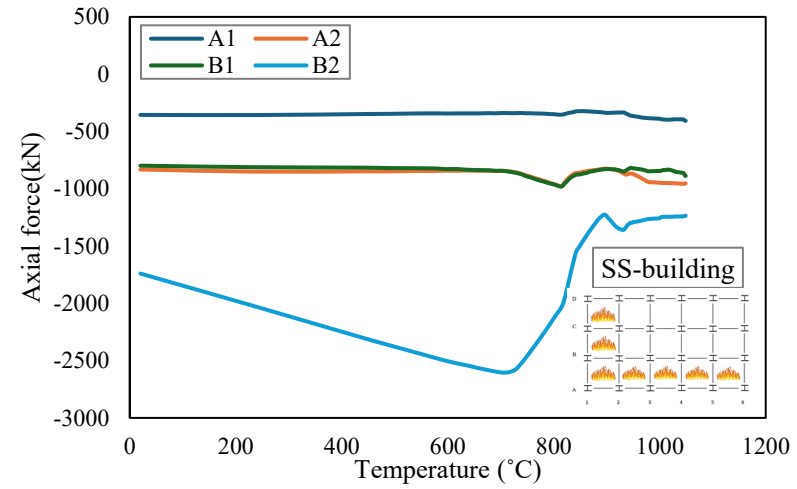
3.5.8. Case F-8

In the CS building (Figure 59), all columns begin in compression, with column B2 carrying the highest initial demand at about 1743 kN. As heating progresses, compressive forces rise briefly before unloading abruptly. B2 peaks at 2294 kN near 576 °C, then falls to almost 30 kN by 781 °C, while A1 fluctuates around zero and both A2 and B1 reverse into mild tension between 840–880 °C. This sharp unloading and tension reversal highlights the rapid stiffness loss and early instability of bare carbon steel under the corner travelling fire. The PCS building follows a similar trend, though with delayed onset due to insulation. B2 reaches about 2262 kN at 520 °C before unloading begins. Between 770–900 °C, all columns transition into tension, ranging from 5 kN in A1 to 30 kN in B1, showing that while fire protection slows heating, it cannot prevent the eventual loss of axial stability once the coating fails. By contrast, the SS building demonstrates much greater stability. B2 peaks at 2600 kN at 700 °C, then smoothly declines to 1234 kN at 1049 °C, while the other columns remain in safe compression between 340 and 957 kN throughout heating. The HYB structure also shows enhanced stability, with B2 peaking at 2574 kN and gradually decreasing to 1266 kN at 1030 °C. Unlike CS and PCS, the HYB system delays tension reversal and maintains compression in most columns until very high temperatures, although residual strength is eventually lost beyond 1000 °C.

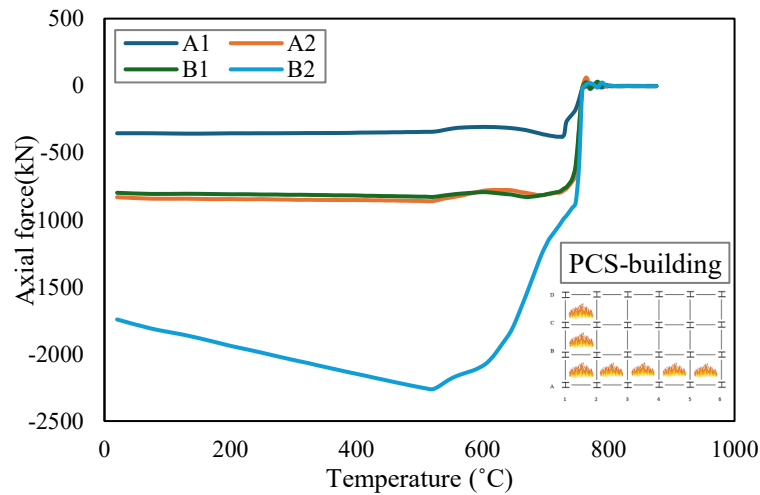
In summary, CS and PCS frames suffer from sudden unloading and column reversals near 800 °C, making them highly vulnerable to collapse, while SS and HYB retain a large portion of their axial capacity at the same temperature and only approach full capacity loss near 1000 °C, offering far greater resilience against fire-induced failure.



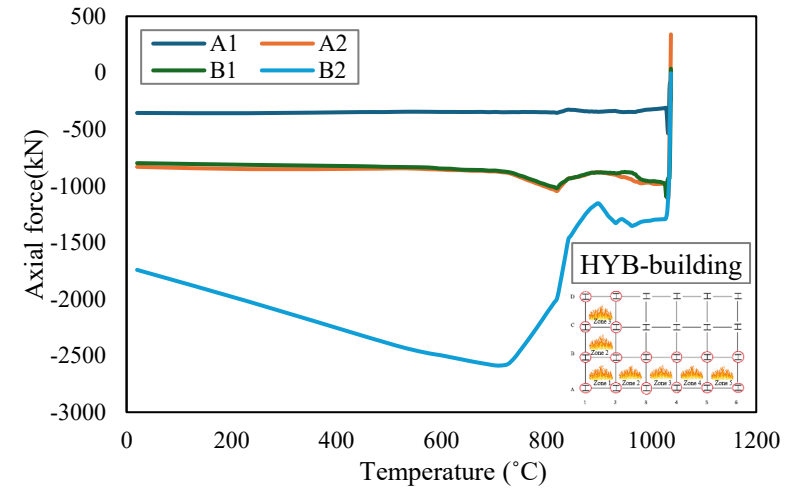
(a)



(b)



(c)



(d)

Figure 59. The axial force-temperature responses of four building systems- CS, SS, PCS, and HYB under Case F-8 fire scenario

3.6. Temperature-Dependent Utilization Factors (η_θ) and Collapse - Sequence Correlation

This section compiles temperature-dependent utilization factors: $\eta_\theta = N_{Ed, \theta} / N_{Rd, \theta}$ for the heated columns and correlates them with the collapse sequences. The axial design resistance in fire follows EN 1993-1-2, $N_{Rd, \theta} = \chi_{fi} A_{k, \theta} f_{y, \theta} / \gamma_{M, fi}$. For each scenario and material system, the tables report the critical steel temperature θ_{crit} , the design resistance at temperature $N_{Rd, \theta}$, the maximum utilization $\eta_{\theta, max}$ (or the governing value at θ_{crit}), and the critical time t_{crit} . The accompanying collapse-sequence figures are then used to verify the timing and location of failure propagation indicated by the utilization data.

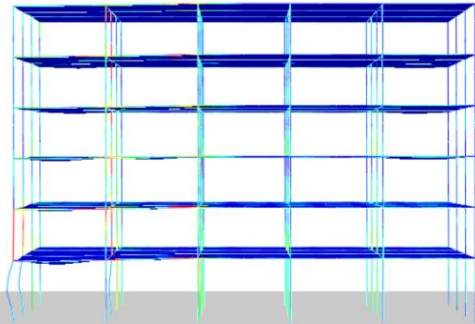
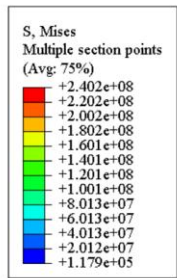
3.6.1. Utilization and Collapse Sequence of Case F-1 Fire Scenario

Table 3.1 presents the summary of utilization and fire resistance for the heated columns A1, A2, B1, and B2 under case F-1 fire scenario, highlighting significant performance differences between CS, PCS, SS, and HYB. The results indicate that CS columns exhibit the lowest fire resistance, with critical temperatures (θ_{crit}) falling to approximately 617-702 °C and corresponding critical times (t_{crit}) limited to 17–22 minutes for most members. Although fire protection delays the onset of failure, protected carbon steel still demonstrates a relatively modest improvement, with θ_{crit} values around 604-701 °C and t_{crit} extending to 39-53 minutes. In contrast, SS and HYB show markedly superior performance. For instance, A1 columns in both SS and HYB systems reach θ_{crit} values of about 900 °C and sustain loads for up to 94 minutes, while interior columns such as A2 and B1 retain critical times of approximately 79 minutes. Even the most heavily loaded column B2 maintains compressive resistance until 42 minutes in these systems. Overall, the findings confirm that stainless steel and hybrid systems provide significantly enhanced robustness against fire, while carbon steel—whether protected or not—remains highly vulnerable to early strength loss and abrupt collapse. In addition, (Figures 60-63) illustrate the progressive collapse sequence of the CS, SS, HYB and PCS buildings under Case F-1, providing visual confirmation of critical time-temperature relationships for the four building systems. As can be seen, the CS building collapses the earliest and most abruptly, the PCS building delays but does not

prevent collapse, and the SS and HYB buildings maintain stability throughout, clearly proving their superior fire resilience.

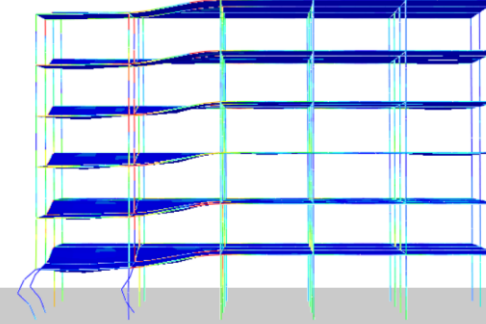
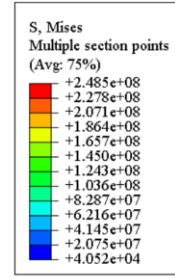
Table 3.1. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-1

Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
A1	CS	702	344.9	1.3072	22
	SS	900.1	289.7	1.3613	94
	HYB	900.1	289.7	1.4203	94
	PCS	701	344.9	1.3324	53
A2	CS	702	735.2	1.2010	22
	SS	809.4	1058.4	1.0081	80
	HYB	802.6	1058.4	1.0455	79
	PCS	701	735.2	1.2172	53
B1	CS	702	735.2	1.1831	22
	SS	802.6	1058.432	1.0063	79
	HYB	802.6	1058.4	1.0296	79
	PCS	701	735.2	1.1999	53
B2	CS	616.8	1985.1	1.1004	17
	SS	603.7	2417.9	1.0915	42
	HYB	603.7	2417.9	1.0871	42
	PCS	603.7	1985.1	1.1834	39



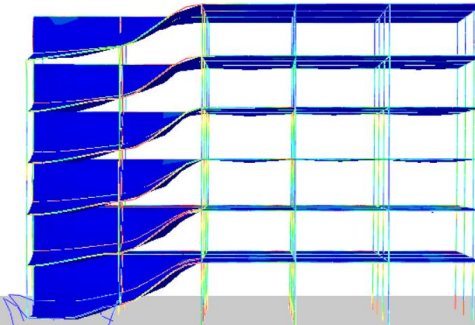
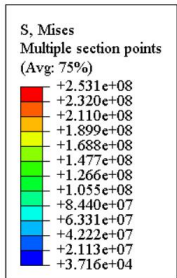
Step Time = 19 min

Temperature =726.8°C



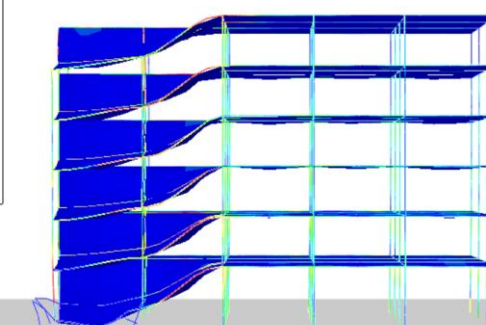
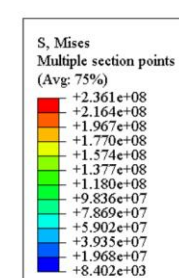
Step Time = 21 min

Temperature =747.4°C



Step Time = 23 min

Temperature =764.2°C



Step Time = 120 min

Temperature =1043°C

Figure 60. Progressive collapse sequence of the CS building under Case F-1 fire scenario

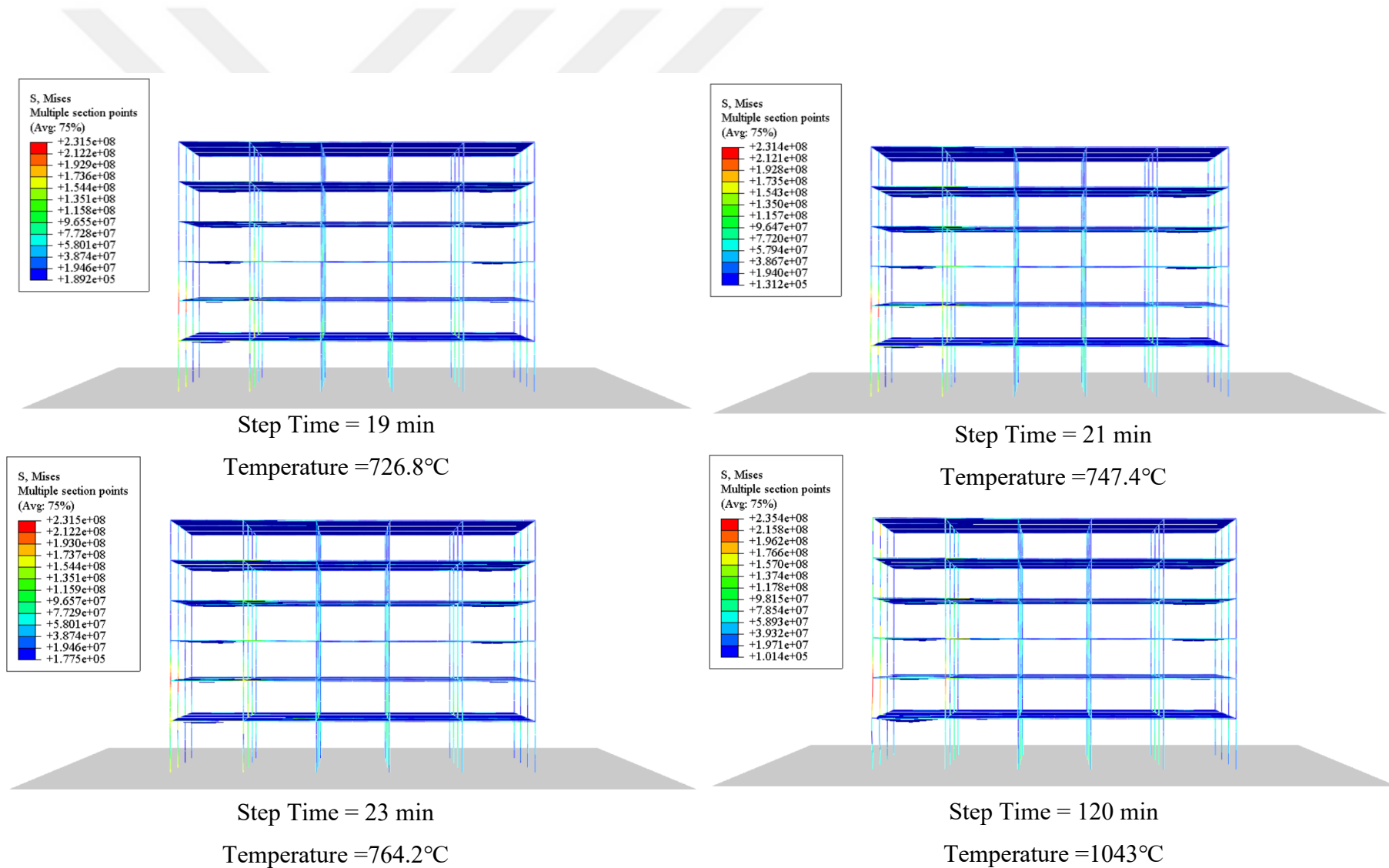
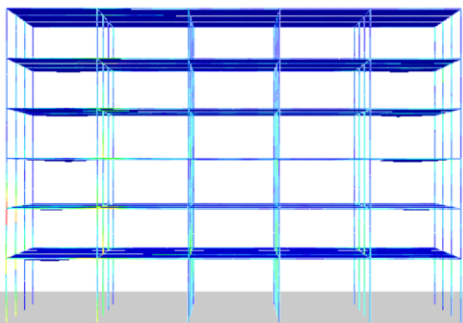
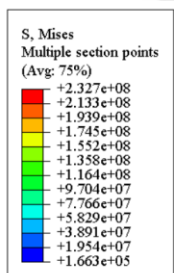
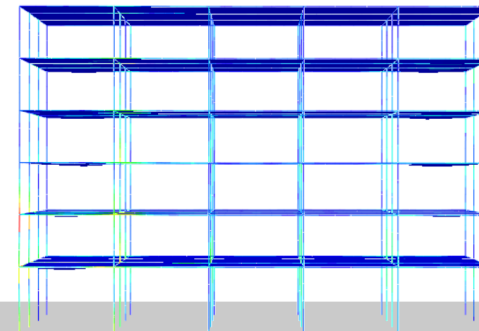
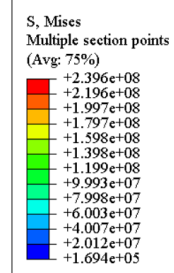


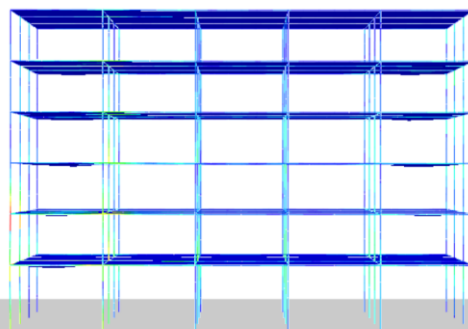
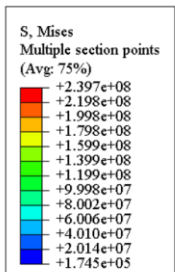
Figure 61. Progressive collapse sequence of the SS building under Case F-1 fire scenario



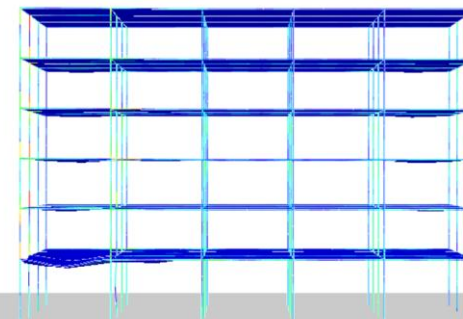
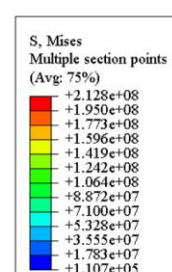
Step Time = 19 min
Temperature = 726.8°C



Step Time = 21 min
Temperature = 747.4°C

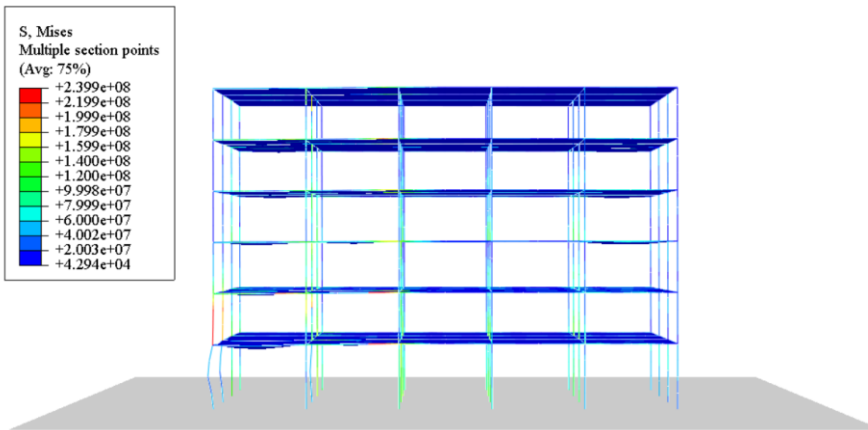


Step Time = 23 min
Temperature = 764.2°C

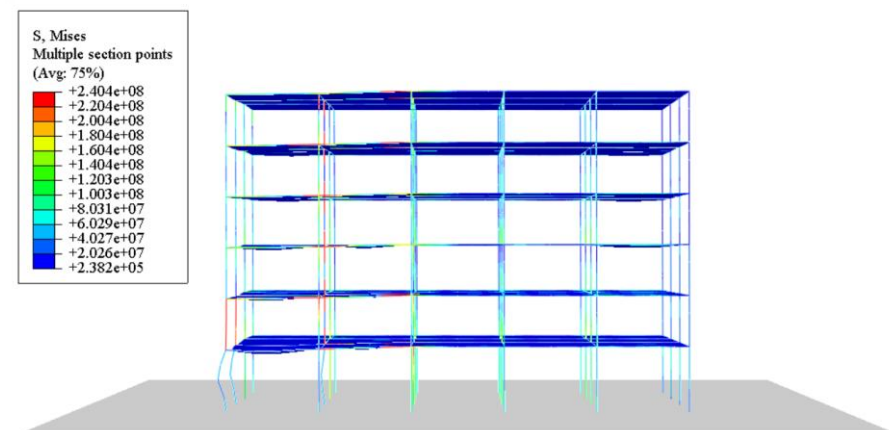


Step Time = 120 min
Temperature = 1043°C

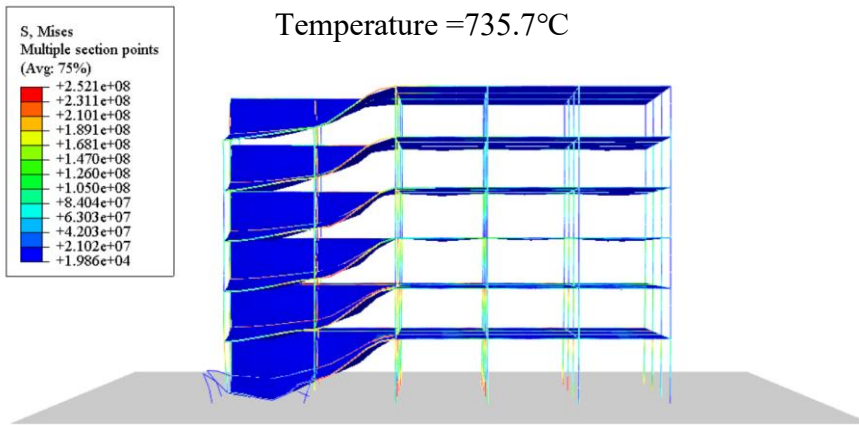
Figure 62. Progressive collapse sequence of the HYB building under Case F-1 fire scenario



Step Time = 53min

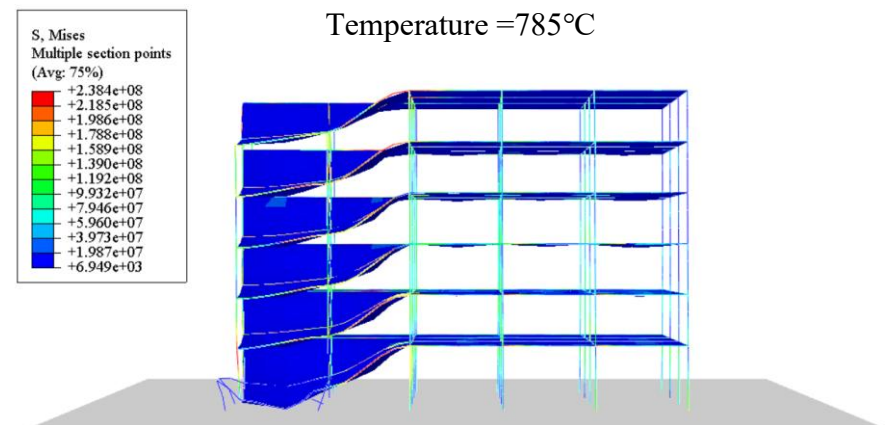


Step Time = 62 min



Step Time = 65 min

Temperature = 808.3°C



Step Time = 120 min

Temperature = 1029°C

Figure 63. Progressive collapse sequence of the PCS building under Case F-1 fire scenario

3.6.2. Utilization and Collapse Sequence of Case F-2 Fire Scenario

Table 3.2 summarizes the fire resistance and utilization factors of heated columns B3, B4, C3, and C4 under Case F-2 for (CS, PCS, SS, and HYB buildings. The results clearly show that carbon CS columns have the lowest critical temperature, around 616.8 °C, with short critical times of ~17 minutes, reflecting their vulnerability to early strength loss. PCS provides a notable improvement, extending the critical time to ~39 minutes despite slightly lower critical temperatures (~603.7 °C), demonstrating that insulation effectively delays failure but does not significantly increase material capacity. SS achieves higher critical temperatures, between 678-702 °C, and longer critical times of 17-21 minutes. Importantly, SS also shows higher utilization factors ($\eta\theta$ up to 1.213), indicating a more efficient load-carrying capacity at elevated temperatures. HYB columns perform comparably to stainless steel, with critical temperatures around 616-657 °C and critical times of 17 - 19 minutes, supported by strong load resistance values above 2400 kN. Overall, the table emphasizes that while CS fails earliest, PCS extends resistance through delayed heating, and SS and HYB systems stand out as the most robust solutions, maintaining higher load capacity and longer fire resistance under severe fire exposure. (Figures 64-67) show progressive collapse sequence of the CS, SS, HYB and PCS buildings under Case F-2. The bare carbon steel building show the earliest instability. At only 17 minutes (≈ 724 °C), visible deformations emerge in heated columns, and by 34–38 minutes (≈ 836 -856 °C), severe buckling initiates. The fire protection delays damage and SS shows superior stability. Also, the HYB building mirrors the SS case.

Table 3.2. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-2

Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
B3	CS	616.8	1985.1	1.1081	17
	SS	678.0	2417.9	1.0053	20
	HYB	616	2417.9	1.0001	17
	PCS	603.7	1985.1	1.1269	39
B4	CS	616.8	1985.1	1.1060	17
	SS	702.4	1973.8	1.2135	21
	HYB	657.6	2417.9	1.0052	19
	PCS	603.7	1985.1	1.1157	39
C3	CS	616.8	1985.1	1.1094	17
	SS	616.8	2417.9	1.0137	17
	HYB	616.8	2417.9	1.0238	17
	PCS	603.7	1985.1	1.1450	39
C4	CS	616.8	1985.1	1.1094	17
	SS	678.0	2417.9	1.0062	20
	HYB	616.8	2417.9	1.0014	17
	PCS	603.7	1985.1	1.1298	39

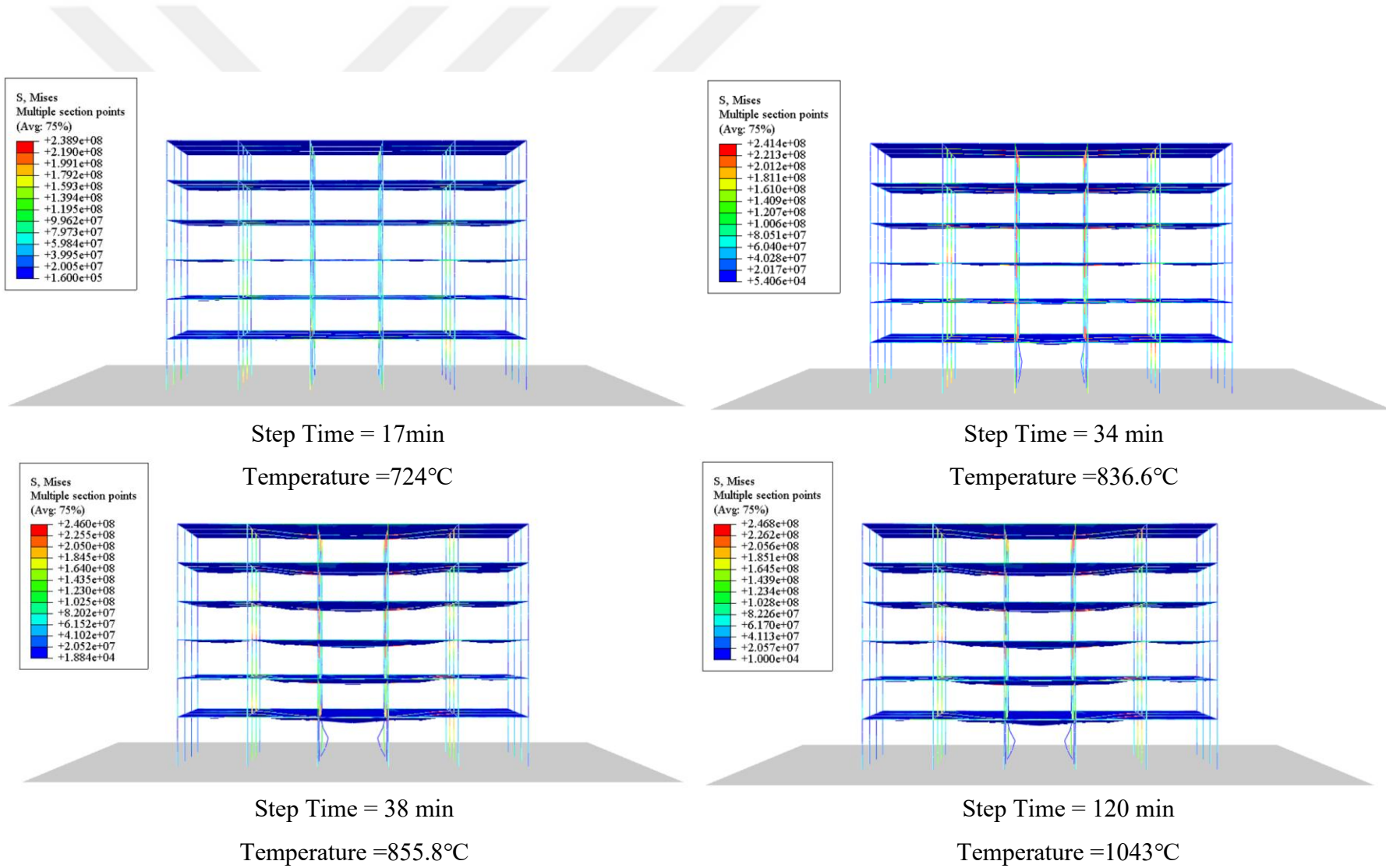
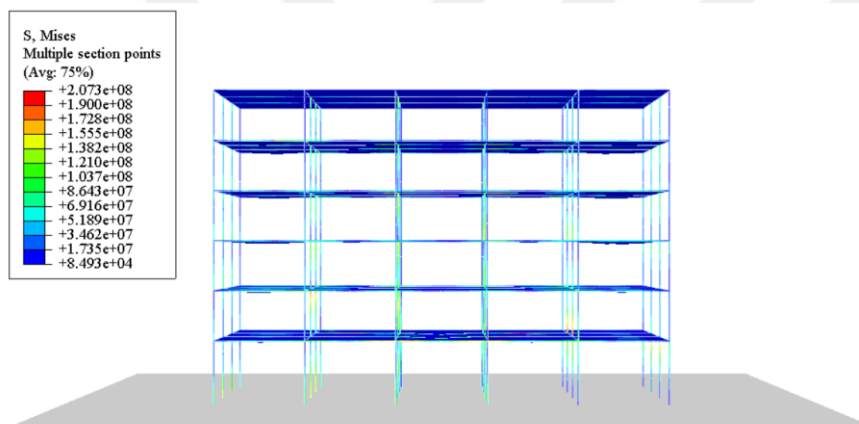
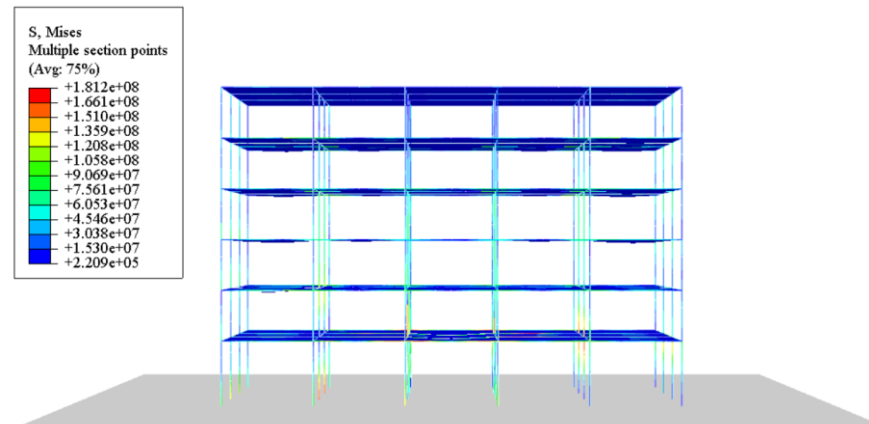


Figure 64. Progressive collapse sequence of the CS building under Case F-2 fire scenario



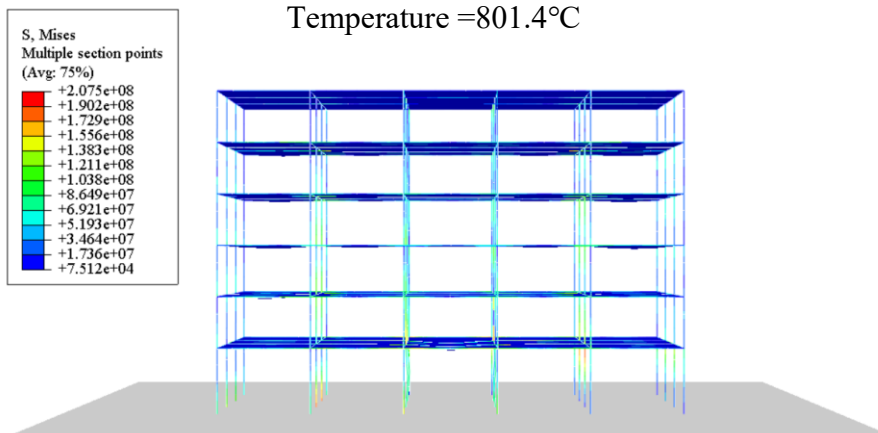
Step Time = 28min

Temperature =801.4°C



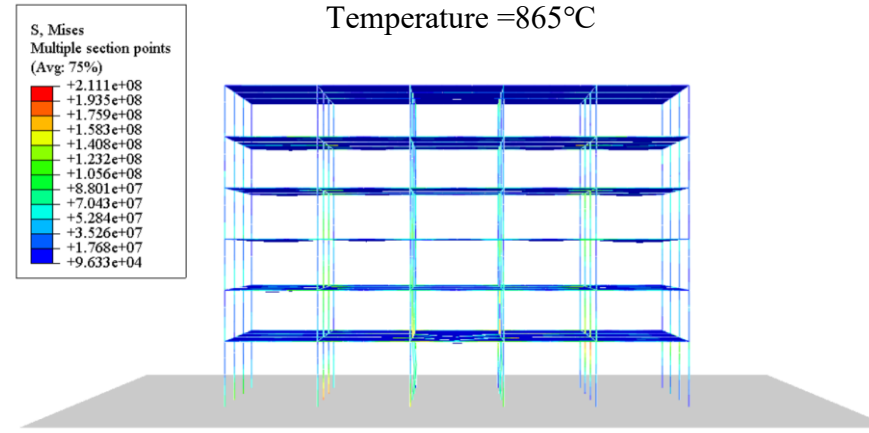
Step Time = 40 min

Temperature =865°C



Step Time = 84 min

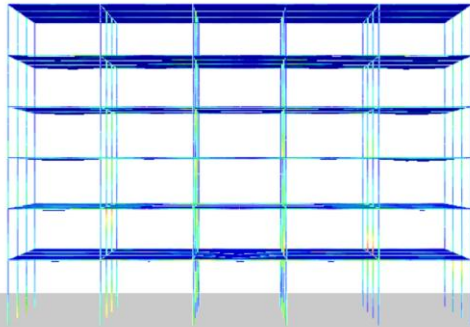
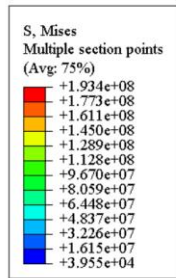
Temperature =986.2°C



Step Time = 120 min

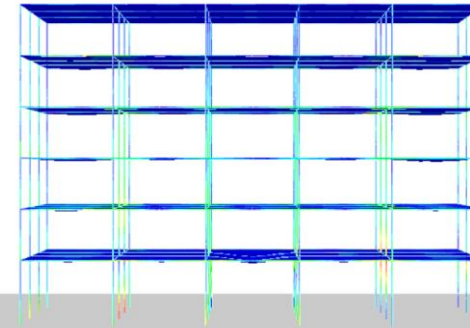
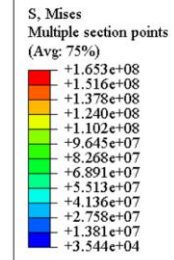
Temperature =1043°C

Figure 65. Progressive collapse sequence of the SS building under Case F-2 fire scenario



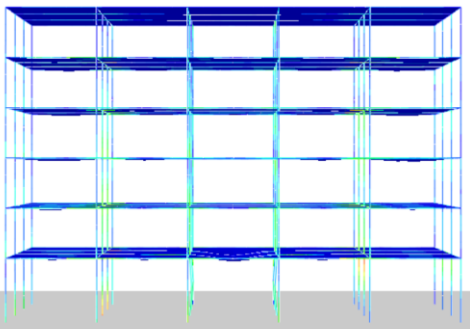
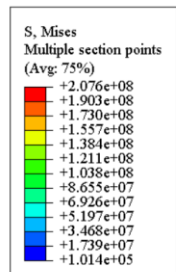
Step Time = 28min

Temperature =801.4°C



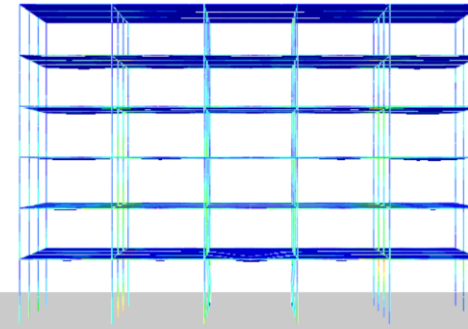
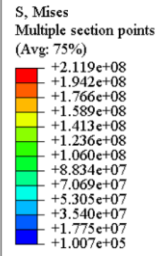
Step Time = 40 min

Temperature =865°C



Step Time = 84 min

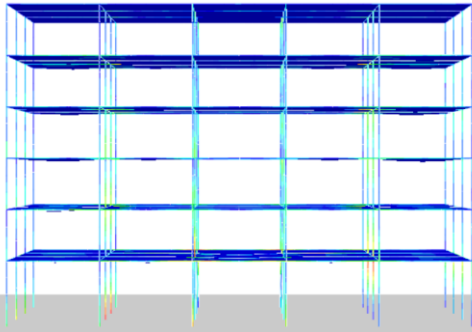
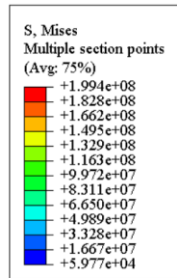
Temperature =986.2°C



Step Time = 120 min

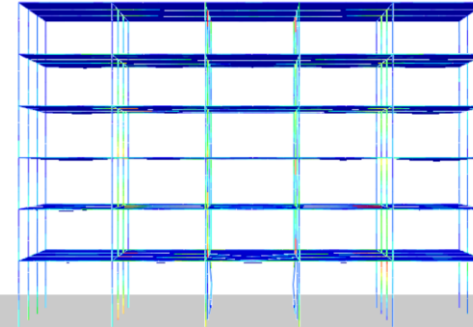
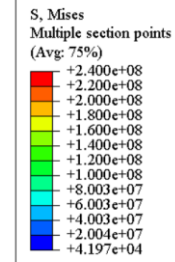
Temperature =1043°C

Figure 66. Progressive collapse sequence of the HYB building under Case F-2 fire scenario



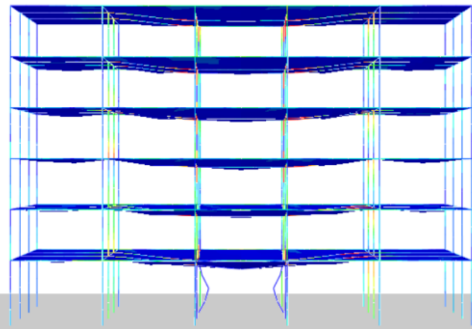
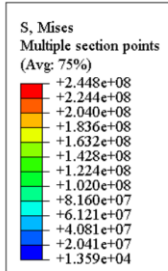
Step Time = 28min

Temperature =801.4°C



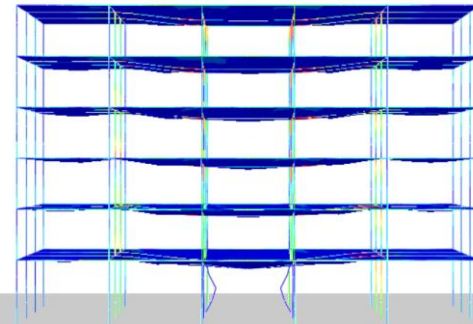
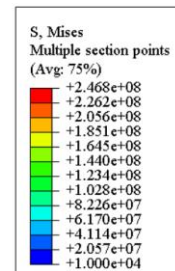
Step Time = 40 min

Temperature =865°C



Step Time = 84 min

Temperature =986.2°C



Step Time = 120 min

Temperature =1043°C

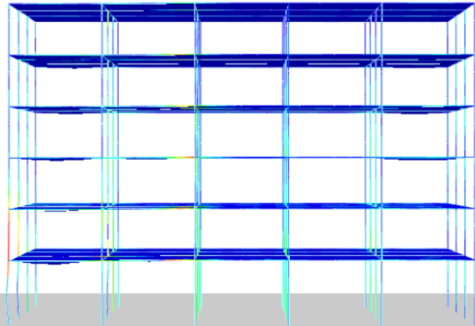
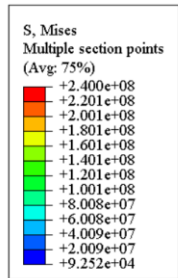
Figure 67. Progressive collapse sequence of the PCS building under Case F-2 fire scenario

3.6.3. Utilization and Collapse Sequence of Case F-3 Fire Scenario

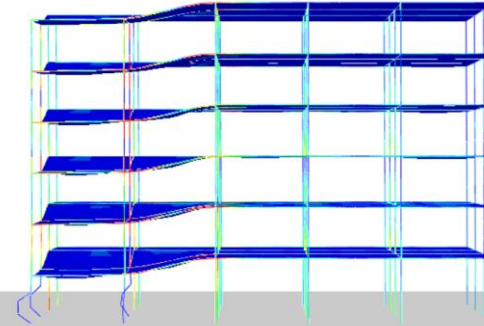
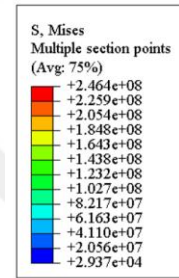
Table 3.3 summarizes the utilization and fire resistance of heated columns (A1, A2, B1, B2) in different structural systems (CS, SS, HYB, PCS) under the Case F-3 scenario. The results show clear material-dependent differences in critical temperature (θ_{crit}), load resistance ($NR_{d,\theta}$), and critical time (t_{crit}). For the corner column A1, stainless SS and HYB buildings demonstrate the highest fire endurance, reaching critical temperatures around 902 °C and sustaining load up to 67-68 minutes, compared to only 39 minutes for CS. The PCS extends the resistance further to 74 minutes, though the mechanism is still governed by coating performance. For interior columns A2 and B1, stainless and hybrid frames again outperform carbon steel, with critical times of ~50-51 minutes versus 39 minutes in CS. In the case of column B2, which carries the highest axial demand, the results reveal distinct differences among the systems. The CS frame shows the earliest failure, losing capacity at 34 minutes when the steel temperature reaches about 617 °C, with an axial load of 1985.1 kN. The PCS system improves fire endurance significantly, sustaining the same load for 59 minutes at ~604 °C, highlighting the protective effect of the intumescent coating. By contrast, both SS and HYB systems achieve much higher load resistance of 2417.9 kN but fail at around 34 minutes once critical temperatures are reached. These findings indicate that while stainless and hybrid systems provide superior load-carrying capacity under fire, PCS offers the longest endurance time by delaying heating. Unprotected carbon steel remains the weakest option, confirming its poor fire resistance under heavy axial loading. (Figures 68-71) reveal distinct variations in the fire performance of the four structural systems. The CS frame deteriorates most rapidly, collapsing in just 25 minutes, underscoring its poor resistance. The PCS frame performs better by delaying collapse until about 68 minutes, though it still fails due to rising axial forces and material degradation. In contrast, the SS and HYB systems display much greater resilience. In the SS system, failure is manifested as column buckling at around 147 minutes, yet the structure avoids collapse and continues to deform until 270 minutes. The HYB system demonstrates the best performance, maintaining overall integrity for the full 270-minute simulation, even under temperatures exceeding 1000 °C. These results confirm that stainless steel and hybrid systems provide superior stability and fire resistance, effectively preventing progressive collapse compared with conventional carbon steel or partially protected systems.

Table 3.3. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-3

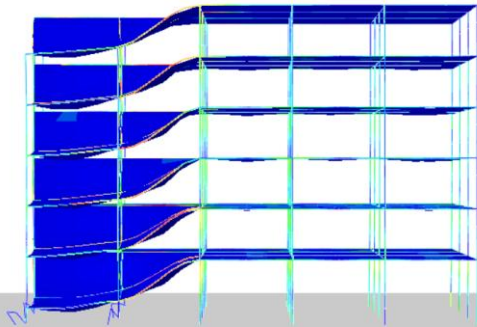
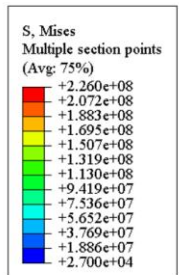
Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
A1	CS	702.4	344.9	1.2029	39
	SS	902	289.7	1.4158	67
	HYB	902	289.7	1.5481	68
	PCS	701	344.9	1.2819	74
A2	CS	702.4	735.2	1.2630	39
	SS	825.8	1058.4	1.0121	51
	HYB	815	1058.4	1.0009	50
	PCS	701	735.2	1.2759	74
B1	CS	702.4	735.2	1.2613	39
	SS	825.8	1058.4	1.0135	51
	HYB	825.8	1058.4	1.0192	51
	PCS	701	735.2	1.2697	74
B2	CS	616.8	1985.1	1.1092	34
	SS	616.8	2417.9	1.0437	34
	HYB	616.8	2417.9	1.0386	34
	PCS	603.7	1985.1	1.1899	59



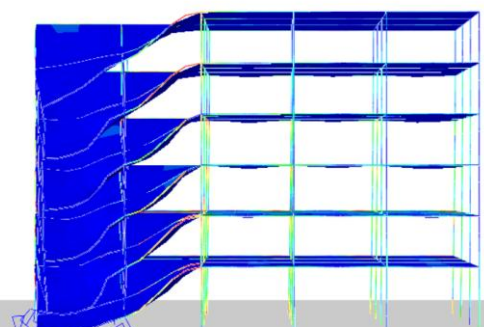
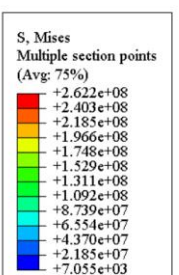
Step Time = 15min
Temperature = 739°C



Step Time = 20 min
Temperature = 781°C

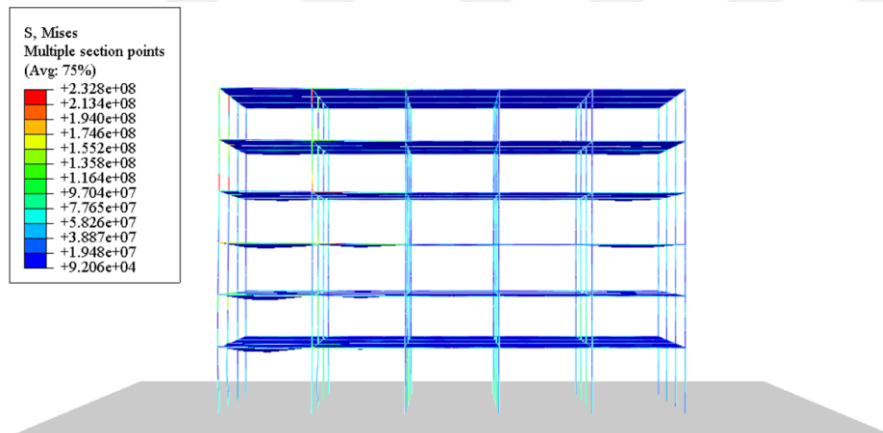


Step Time = 25 min
Temperature = 815°C



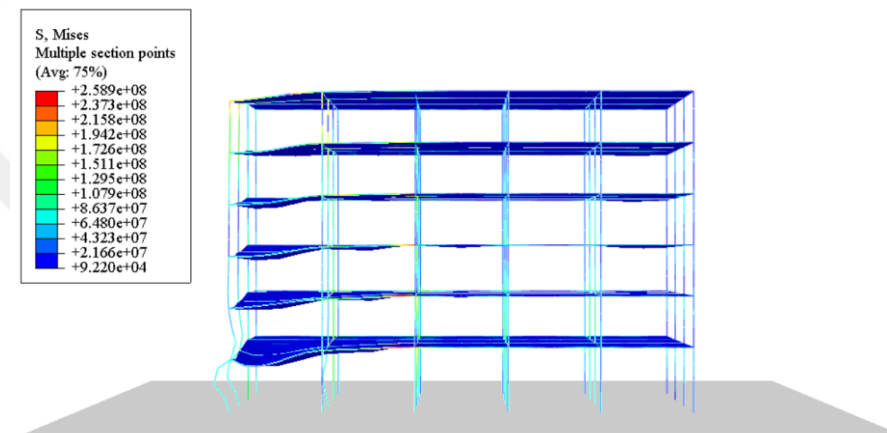
Step Time = 270 min
Temperature = 1049°C

Figure 68. Progressive collapse sequence of the CS building under Case F-3 fire scenario



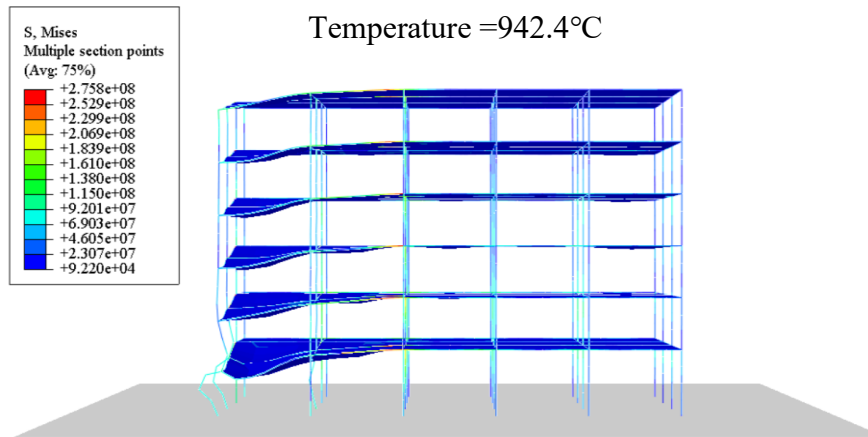
Step Time = 59min

Temperature = 942.4°C



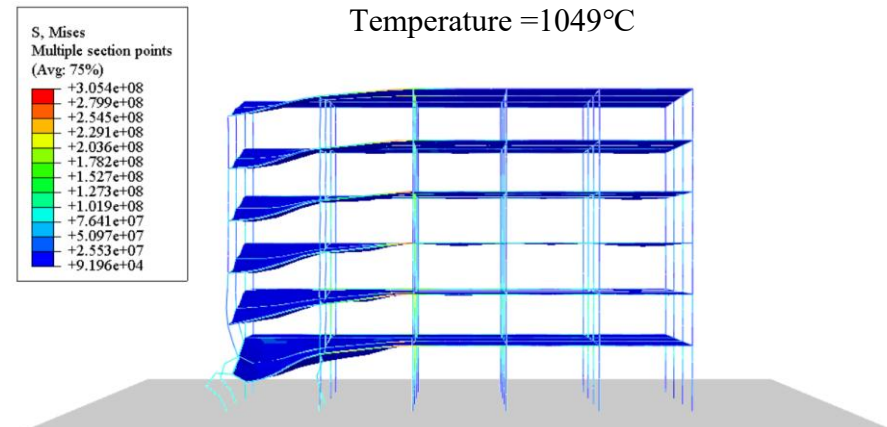
Step Time = 147 min

Temperature = 1049°C



Step Time = 173min

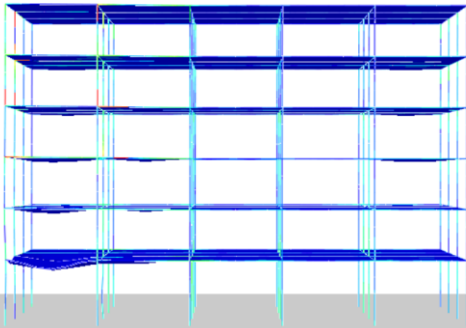
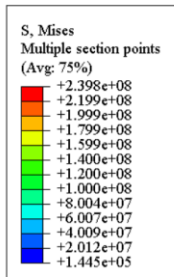
Temperature = 1049°C



Step Time = 270 min

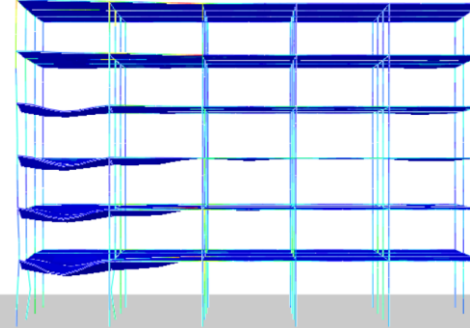
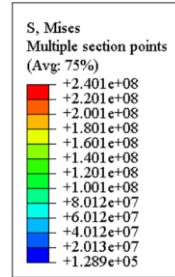
Temperature = 1049°C

Figure 69. Progressive collapse sequence of the SS building under Case F-3 fire scenario



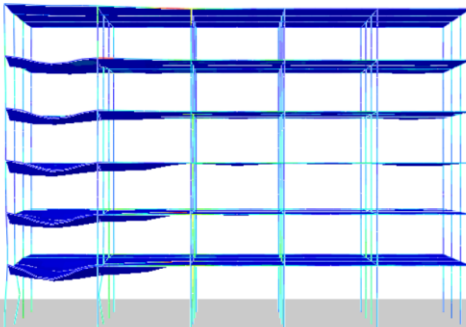
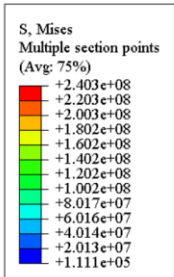
Step Time = 59min

Temperature = 942.4°C



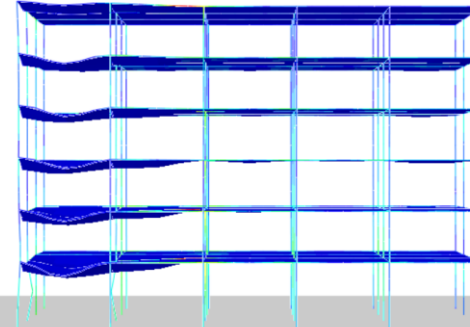
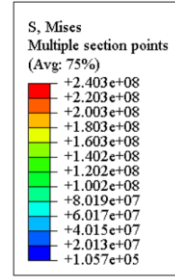
Step Time = 147 min

Temperature = 1049°C



Step Time = 173min

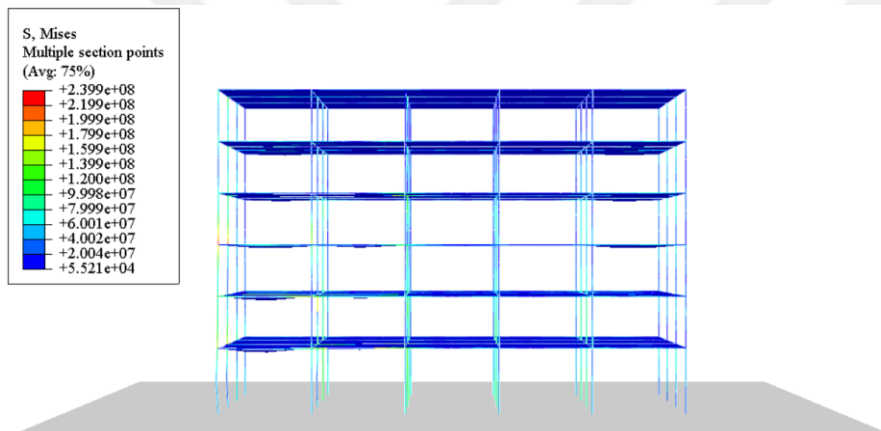
Temperature = 1049°C



Step Time = 270 min

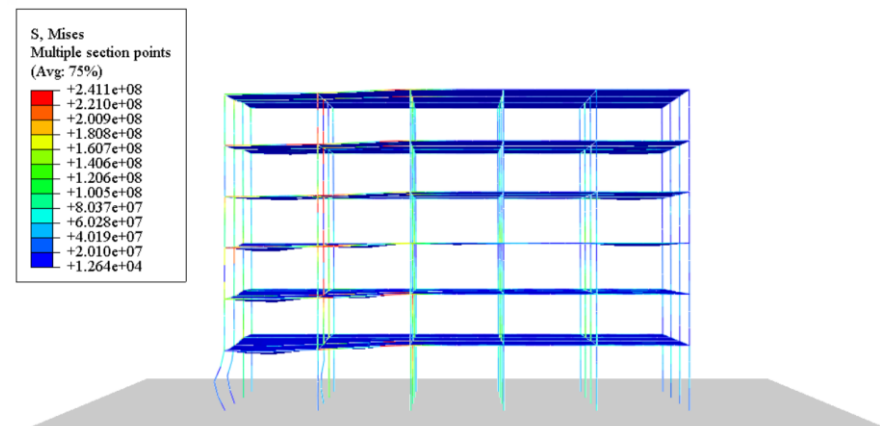
Temperature = 1049°C

Figure 70. Progressive collapse sequence of the HYB building under Case F-3 fire scenario



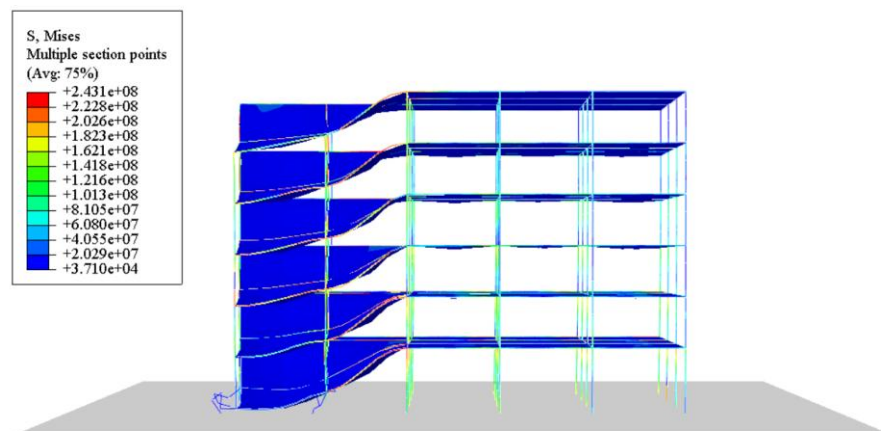
Step Time = 57min

Temperature =718.4°C



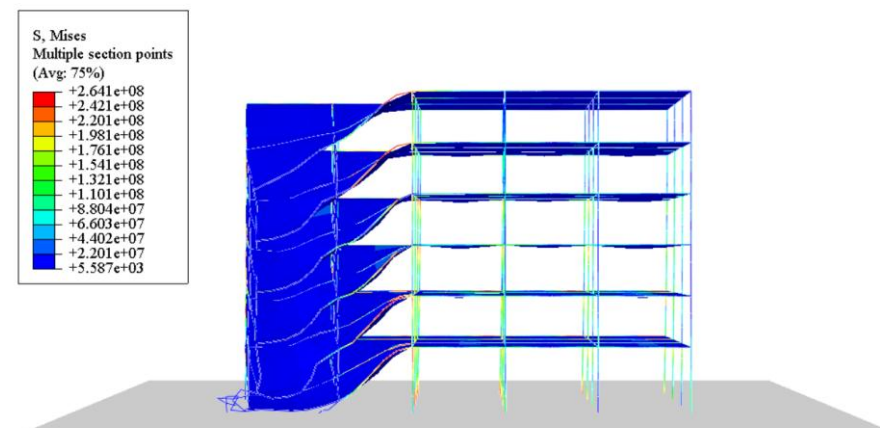
Step Time = 65 min

Temperature =747.8°C



Step Time = 68min

Temperature =764.5°C



Step Time = 270 min

Temperature =1015°C

Figure 71. Progressive collapse sequence of the PCS building under Case F-3 fire scenario

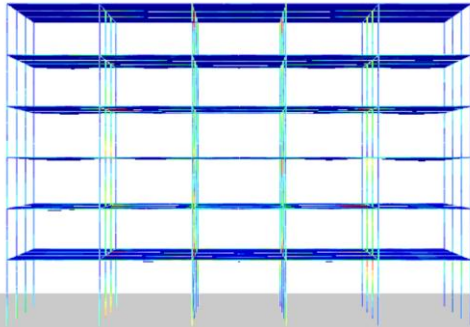
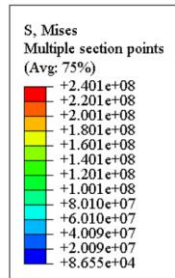
3.6.4. Utilization and Collapse Sequence of Case F-4 Fire Scenario

Table 3.4 presents the utilization and fire resistance of columns B3, B4, C3, and C4 under the Case F-4 fire scenario for four structural systems. The results reveal that CS exhibits the shortest endurance, with critical times around 34 minutes at $\theta_{crit} \approx 616$ °C. The SS and HYB columns show moderate improvement, sustaining fire exposure up to 39 minutes at higher critical temperatures of 702 °C. The best performance is achieved by the PCS system, which prolongs resistance significantly to 59 minutes despite a lower θ_{crit} of 604 °C, highlighting the role of insulation in delaying collapse rather than increasing material temperature capacity. The axial load capacity remains relatively similar across systems (~1974–1985 kN), indicating that the differences in performance are governed mainly by thermal resistance and load redistribution mechanisms. Overall, PCS provides the longest protection time, while SS and HYB maintain improved thermal strength over bare steel, with CS remaining the most vulnerable under F-4 conditions.

As can be in (Figures 72-75) the CS system (building) is the most vulnerable, with early failure and severe instability. PCS delays the failure. SS and HYB systems exhibit outstanding fire resistance, maintaining structural stability even under prolonged fire exposure.

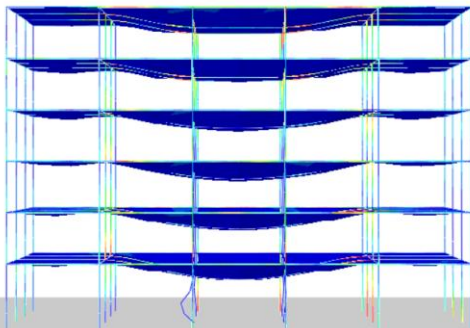
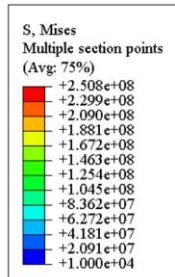
Table 3.4. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-4

Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
B3	CS	616	1985	1.0685	34
	SS	702	1974	1.1993	39
	HYB	702	1974	1.2100	39
	PCS	604	1985	1.1444	59
B4	CS	616	1985	1.0655	34
	SS	702	1974	1.1772	39
	HYB	702	1974	1.2101	39
	PCS	604	1985	1.1407	59
C3	CS	616	1985	1.0730	34
	SS	702	1974	1.2207	39
	HYB	702	1974	1.2140	39
	PCS	604	1985	1.1498	59
C4	CS	616	1985	1.0705	34
	SS	702	1974	1.1993	39
	HYB	702	1974	1.2101	39
	PCS	604	1985	1.1432	59



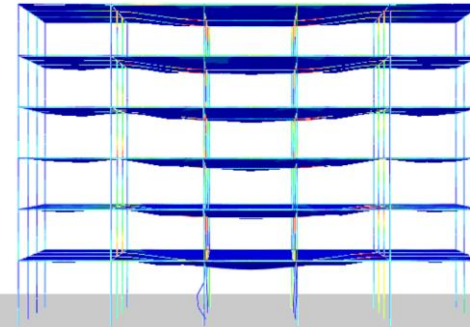
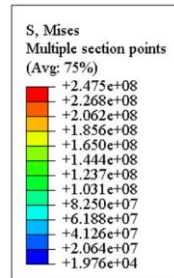
Step Time = 20min

Temperature =781°C



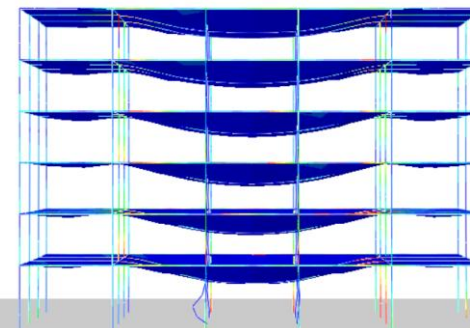
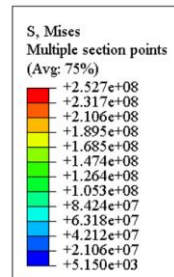
Step Time = 128min

Temperature =1049°C



Step Time = 32 min

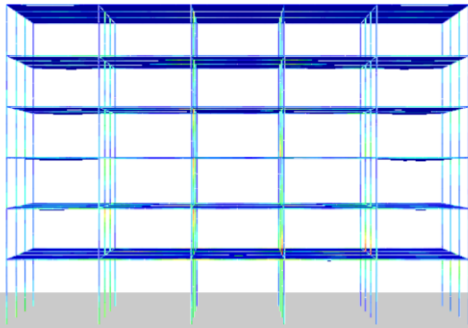
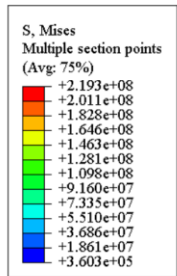
Temperature =851.2°C



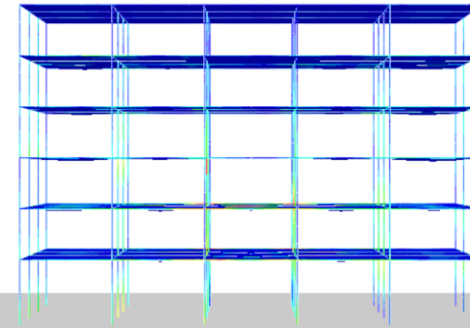
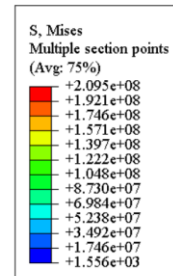
Step Time = 270 min

Temperature =1049°C

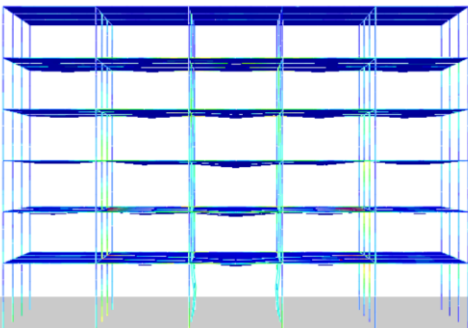
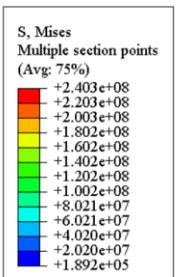
Figure 72. Progressive collapse sequence of the CS building under Case F-4 fire scenario



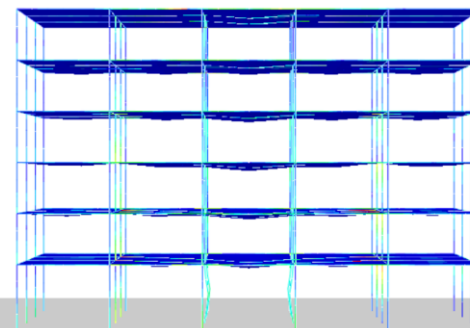
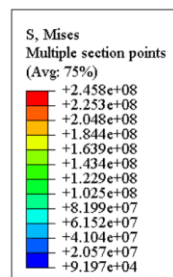
Step Time = 20min
Temperature = 781°C



Step Time = 33 min
Temperature = 855.8°C



Step Time = 126min
Temperature = 1049°C



Step Time = 270 min
Temperature = 1049°C

Figure 73. Progressive collapse sequence of the SS building under Case F-4 fire scenario

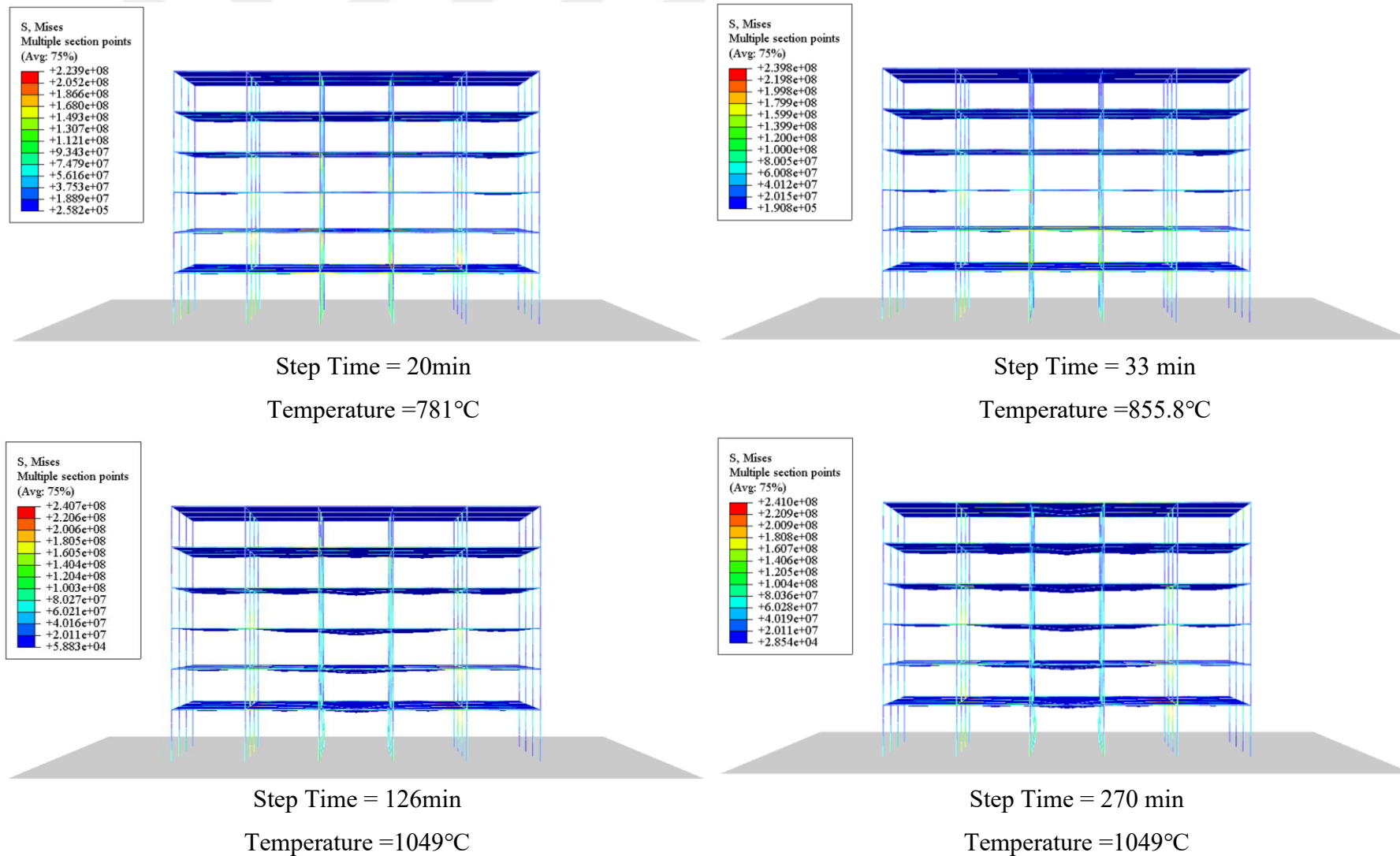
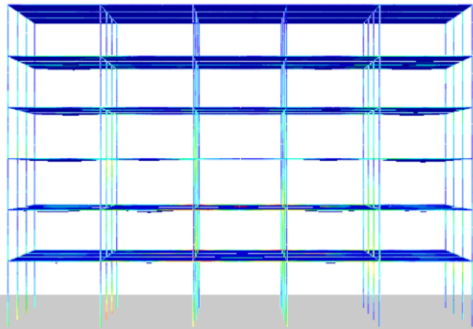
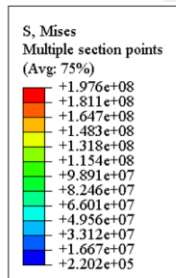
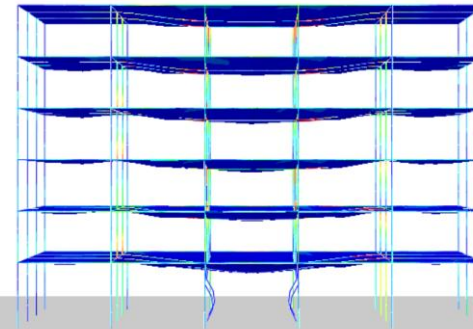
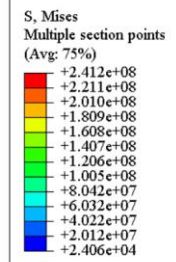


Figure 74. Progressive collapse sequence of the HYB building under Case F-4 fire scenario



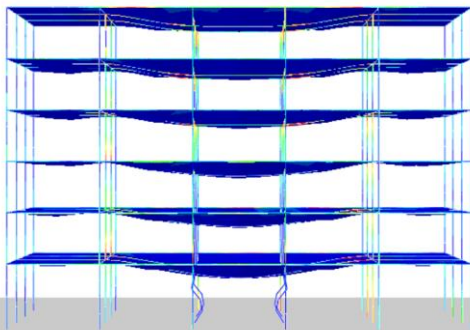
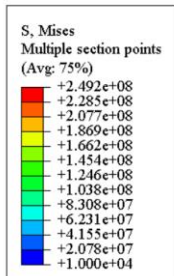
Step Time = 44min

Temperature = 660°C



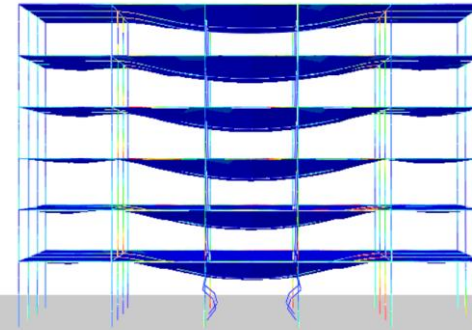
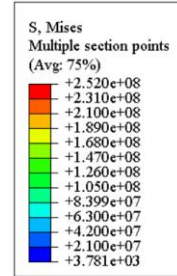
Step Time = 78 min

Temperature = 829.7°C



Step Time = 130min

Temperature = 1015°C



Step Time = 274 min

Temperature = 1015°C

Figure 75. Progressive collapse sequence of the PCS building under Case F-4 fire scenario

3.6.5. Utilization and Collapse Sequence of Case F-5 Fire Scenario

Table 3.5 summarizes the fire resistance and utilization of heated columns B3, B4, C3, and C4 for the four buildings under Case F-5 (central plus - fire travelling). The results indicate that the critical temperature (θ_{crit}) of CS column is the lowest at around 616.8 °C, with a corresponding fire resistance time (t_{crit}) of only 34.5 minutes, showing their vulnerability to rapid strength degradation. In contrast, stainless SS and HYB systems reach higher θ_{crit} values of 702.4 °C and achieve longer resistance (39 minutes), benefiting from stainless steel's superior high-temperature performance. The PCS system shows a lower θ_{crit} of 701 °C but achieves the longest resistance, with t_{crit} extending to 74 minutes due to the insulation layer, despite its reduced axial resistance. In summary, CS columns fail the fastest, SS and HYB offer improved thermal stability, while PCS provides the longest endurance due to protection, despite operating closer to their ultimate capacity.

(Figures 76-79) compare the progressive collapse sequences of CS, PCS, SS, and HYB buildings under Case F-5 fire scenario. The CS system collapses fastest, losing stability by around 42 minutes at ~892 °C, with rapid inward buckling of heated bays. The PCS system shows improved performance, maintaining stability until ~73 minutes at ~796 °C. This delay reflects the effect of insulation, which slows down strength degradation, though full collapse still occurs. The SS system exhibits the best fire resistance, preserving structural integrity for more than 150 minutes up to ~1049 °C, with no signs of global collapse. The HYB system also performs robustly, resisting failure until ~76 minutes at ~981 °C, better than CS and PCS but less effective than the fully stainless structure.

Table 3.5. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-5

Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
B3	CS	616.8	1985.1	1.0685	34.5
	SS	702.4	1973.8	1.1993	39
	HYB	702.4	1973.8	1.2129	39
	PCS	701	956.0	1.2333	74
B4	CS	616.8	1985.1	1.0655	34.5
	SS	702.4	1973.8	1.1772	39
	HYB	702.4	1973.8	1.2120	39
	PCS	701	956.0	1.2322	74
C3	CS	616.8	1985.1	1.0730	34.5
	SS	702.4	1973.8	1.2207	39
	HYB	702.4	1973.8	1.2168	39
	PCS	701	956.0	1.2333	74
C4	CS	616.8	1985.1	1.0705	34.5
	SS	702.4	1973.8	1.1993	39
	HYB	702.4	1973.8	1.2131	39
	PCS	701	956.0	1.2322	74

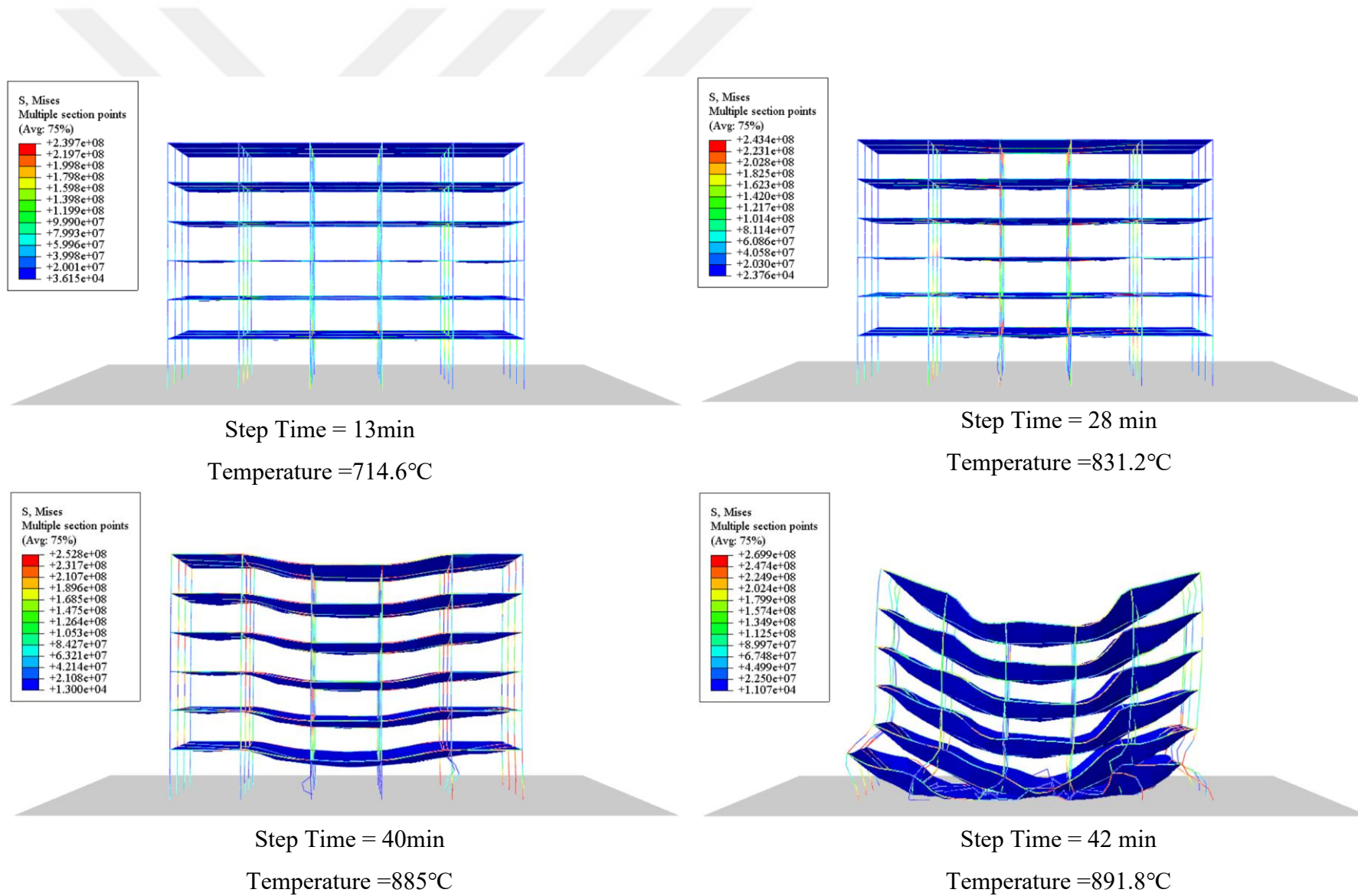


Figure 76. Progressive collapse sequence of the CS building under Case F-5 fire scenario

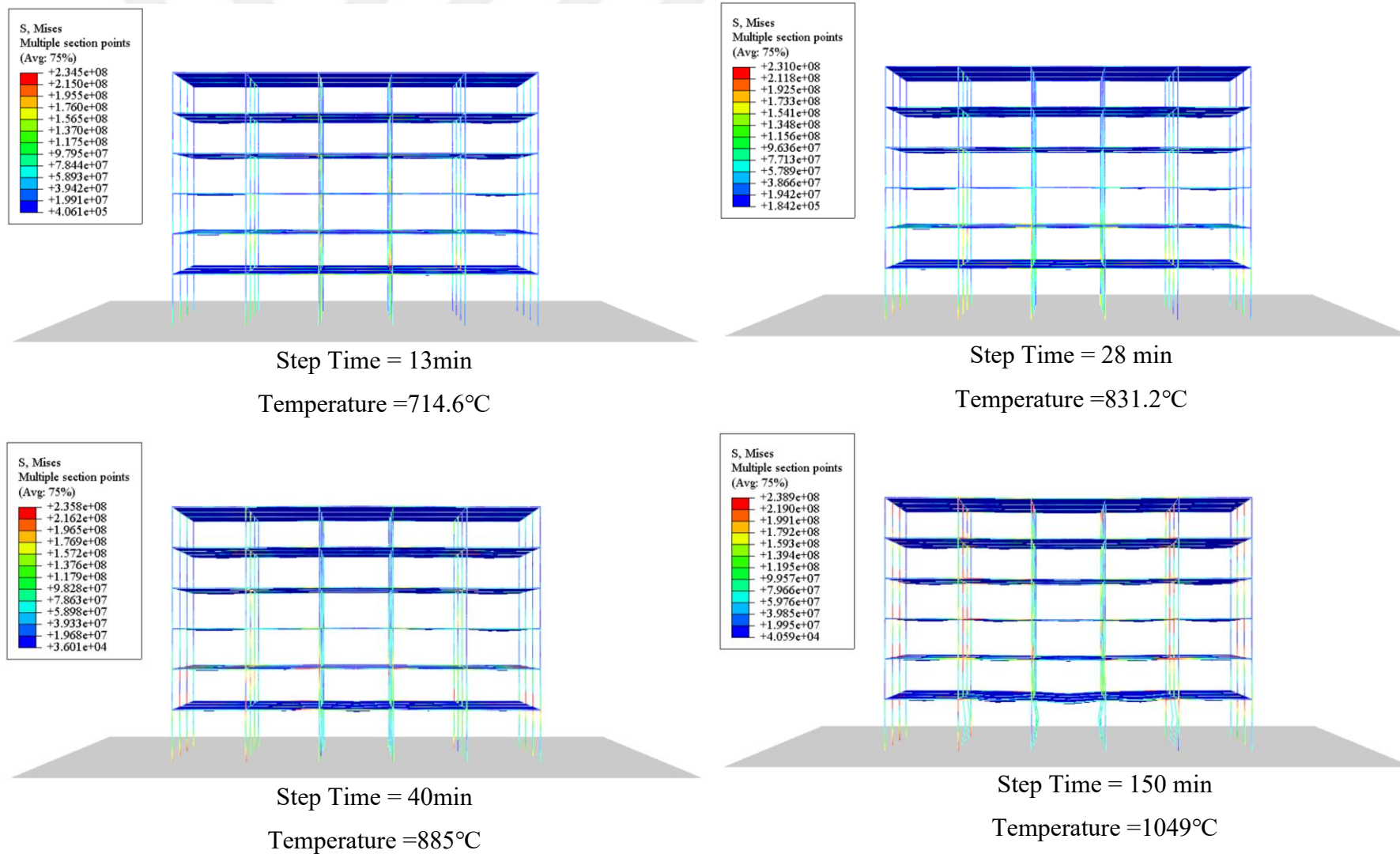
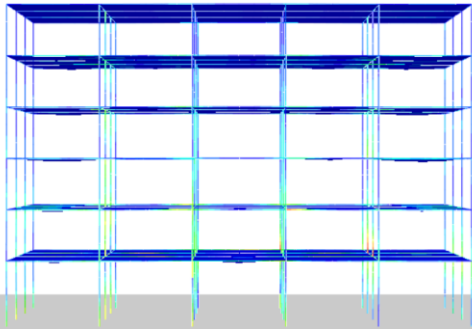
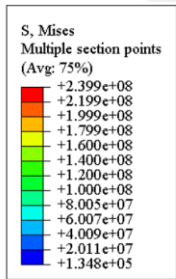
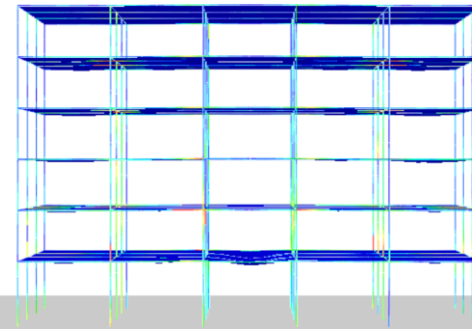
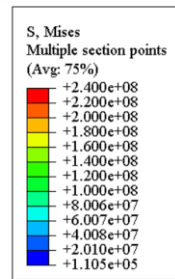


Figure 77. Progressive collapse sequence of the SS building under Case F-5 fire scenario



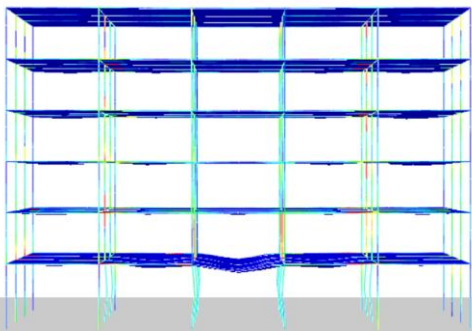
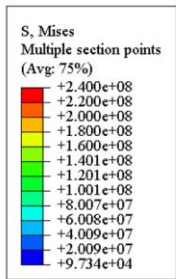
Step Time = 28 min

Temperature =831.2°C



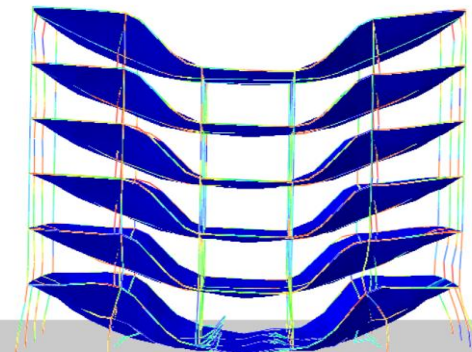
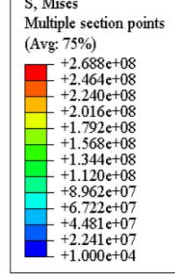
Step Time = 38 min

Temperature =877°C



Step Time = 68 min

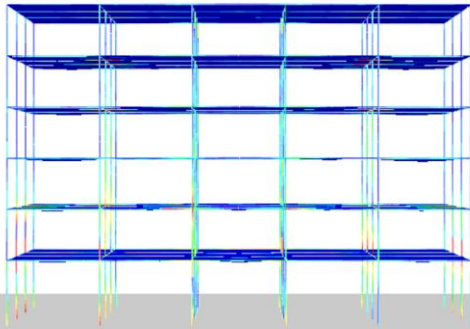
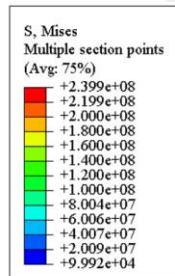
Temperature =963.6°C



Step Time = 76 min

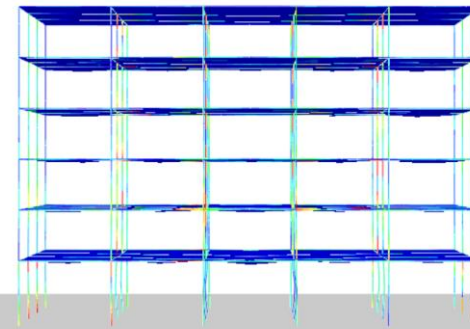
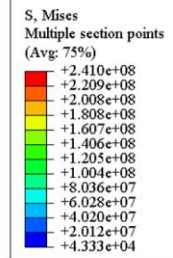
Temperature =980.8°C

Figure 78. Progressive collapse sequence of the HYB building under Case F-5 fire scenario



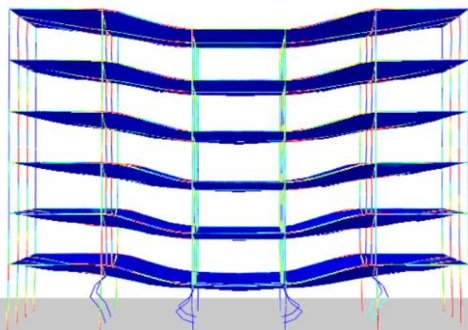
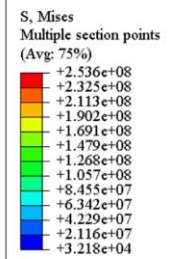
Step Time = 53 min

Temperature = 705.3°C



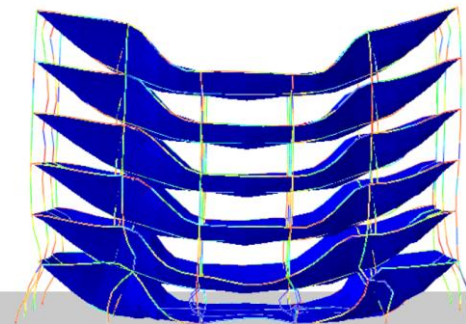
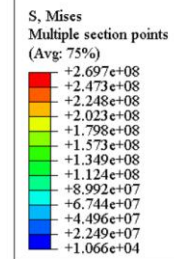
Step Time = 70 min

Temperature = 776.6°C



Step Time = 72 min

Temperature = 789.1°C



Step Time = 73 min

Temperature = 795.9°C

Figure 79. Progressive collapse sequence of the PCS building under Case F-5 fire scenario

3.6.6. Utilization and Collapse Sequence of Case F-6 Fire Scenario

Table 3.6 presents the fire resistance and utilization of heated columns A1, A2, B1, and B2 for the four buildings under Case F-6 (fire-travelling east-west). The results clearly show that carbon steel CS columns reach critical conditions quickly, with very short fire resistance times (17-39 min), confirming their vulnerability. The PCS performs better, extending the fire endurance of columns up to 57–74 minutes, owing to the insulating effect of protection. The SS and HYB systems exhibit superior thermal capacity, reaching high critical temperatures (≈ 902 °C for columns A, 801 °C for columns B1, and 617 °C for columns B2) and providing moderate resistance times ranging from 28 to 68 minutes.

(Figures 80-83) illustrates the progressive collapse sequences of the four structural systems under the F-6 fire scenario, and they reveal clear differences in fire resistance effectiveness. The CS and PCS systems show the most rapid loss of stability. In the case of CS, collapse begins very early (around 20 minutes at ~ 781 °C) and progresses quickly, with total failure occurring within 54 minutes. PCS performs slightly better in terms of timing, with collapse extending to about 76 minutes at ~ 816 °C. By contrast, the SS and HYB systems demonstrate significantly improved resistance. Both systems maintain stability for much longer durations, with the SS building resisting up to 240 minutes at 1049 °C, and HYB showing comparable robustness with no full collapse in the same timeframe. Their deformation patterns indicate better redistribution of loads and delayed critical failure. Overall, the comparison highlights that SS and HYB systems are the most effective in delaying progressive collapse under the F-6 fire condition, while CS and PCS provide only short-term resistance, making them more vulnerable to early structural failure.

Table 3.6. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-6

Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
A1	CS	702.4	344.9	1.3111	17
	SS	902	289.7	1.1262	50
	HYB	902	289.7	1.1357	50
	PCS	701	344.9	1.1532	57
A2	CS	702.4	735.2	1.2639	39
	SS	902	548.8	1.6600	67
	HYB	902	548.8	1.6744	68
	PCS	701	735.2	1.2154	74
B1	CS	702.4	735.2	1.1951	17
	SS	801.4	1058.4	1.0262	28
	HYB	801.4	1058.4	1.0374	28
	PCS	701	735.2	1.2190	57
B2	CS	616.8	1985.1	1.1097	34
	SS	616.8	2417.9	1.0172	34
	HYB	616.8	2417.9	1.0133	34
	PCS	603.7	1985.1	1.0994	59

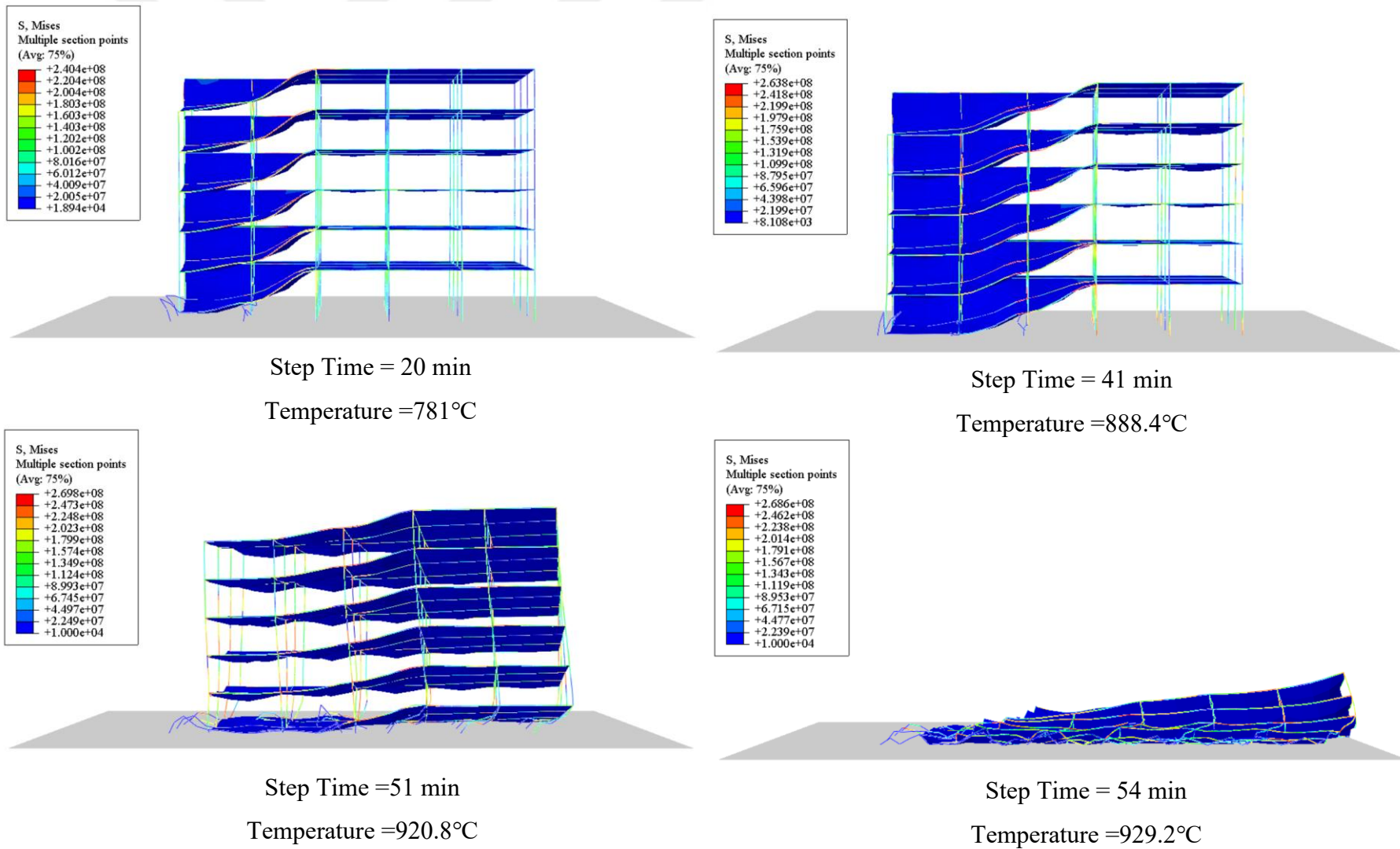


Figure 80. Progressive collapse sequence of the CS building under Case F-6 fire scenario

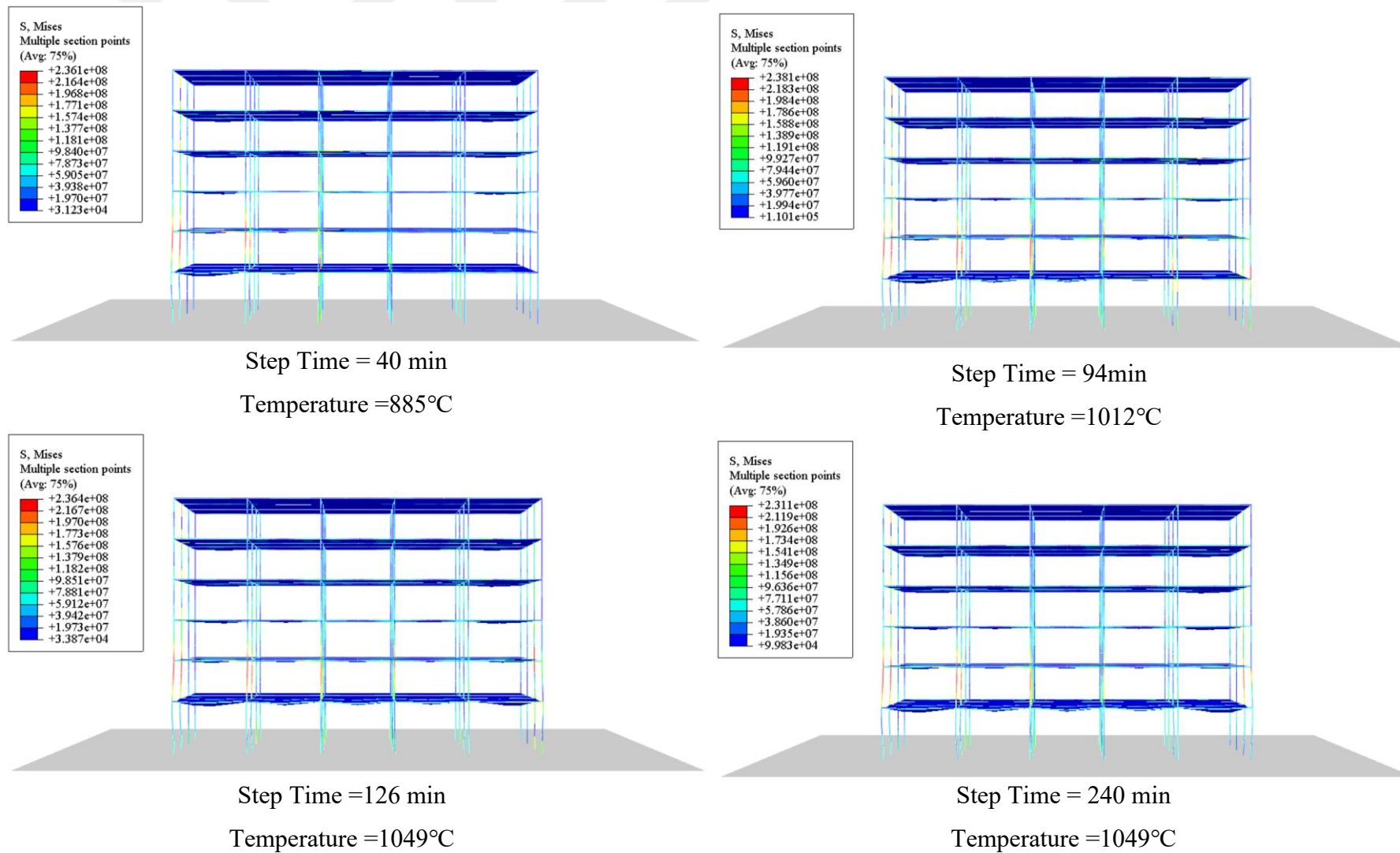
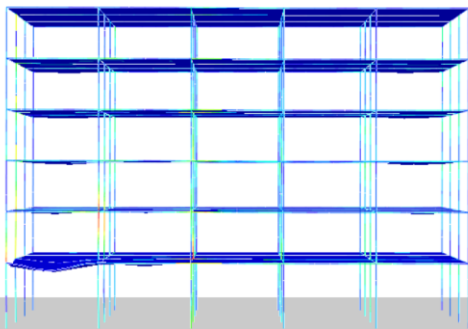
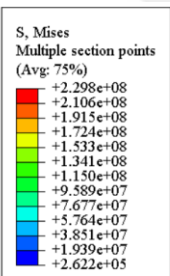
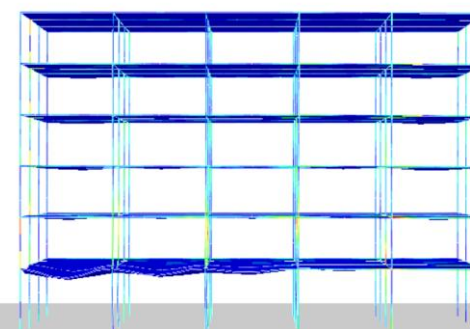
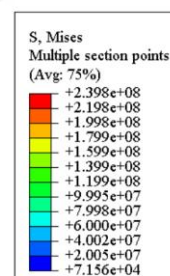


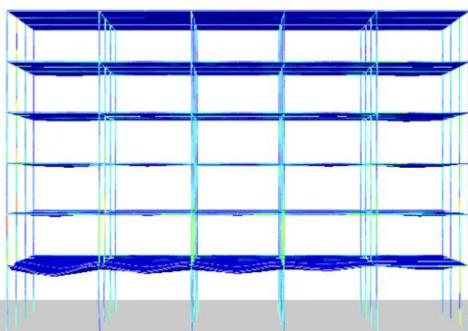
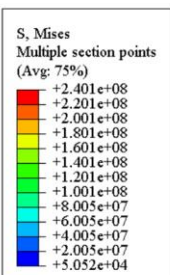
Figure 81. Progressive collapse sequence of the SS building under Case F-6 fire scenario



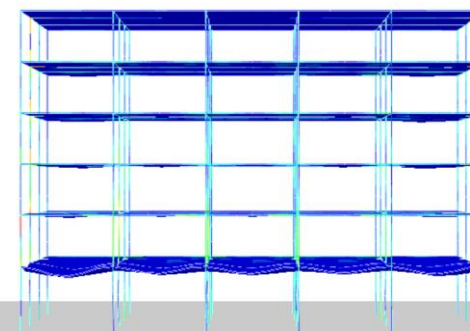
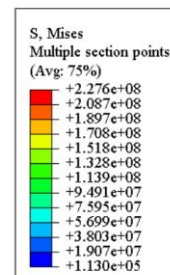
Step Time = 40 min
Temperature = 885°C



Step Time = 94min
Temperature = 1012°C



Step Time = 126 min
Temperature = 1049°C



Step Time = 240 min
Temperature = 1049°C

Figure 82. Progressive collapse sequence of the HYB building under Case F-6 fire scenario

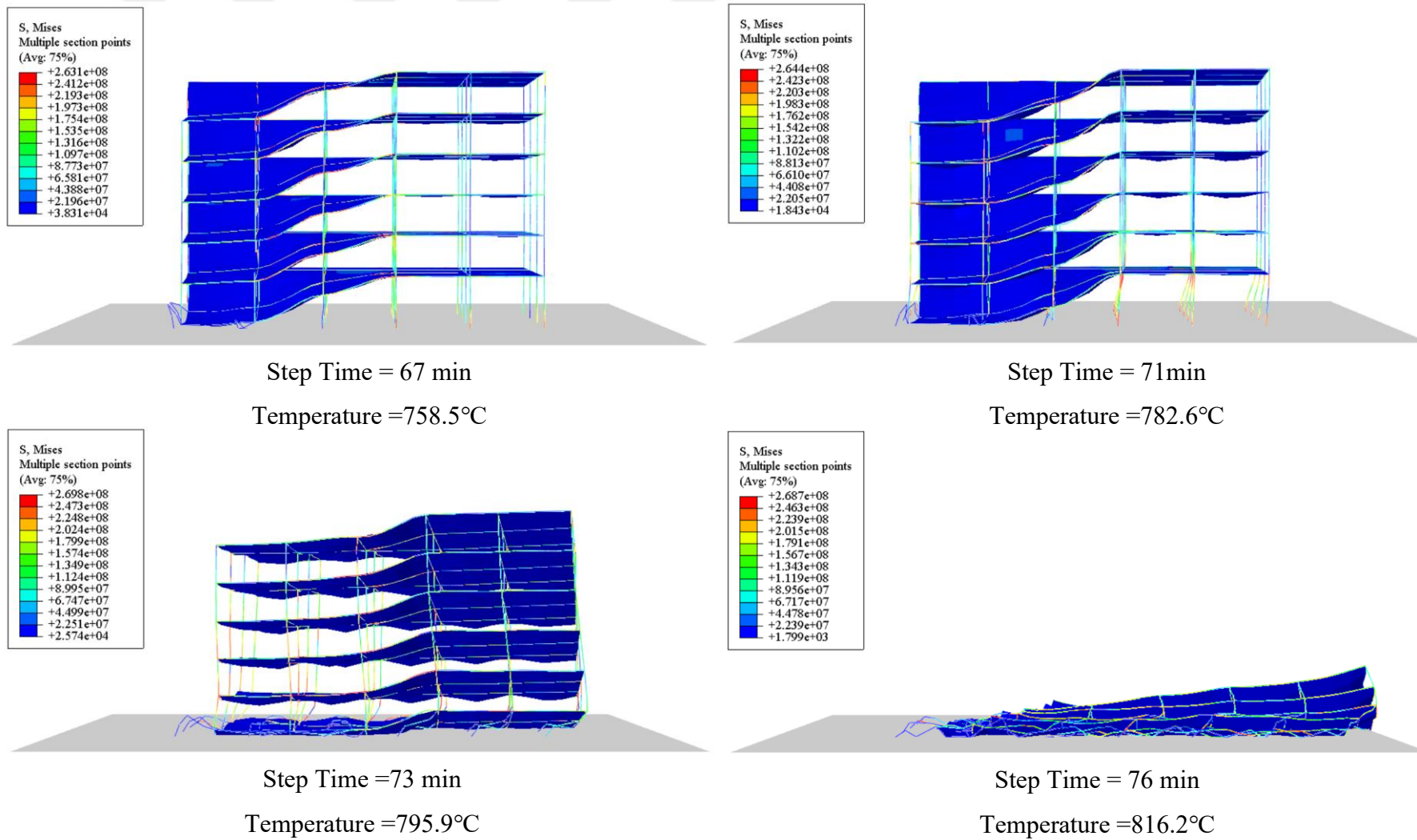


Figure 83. Progressive collapse sequence of the PCS building under Case F-6 fire scenario

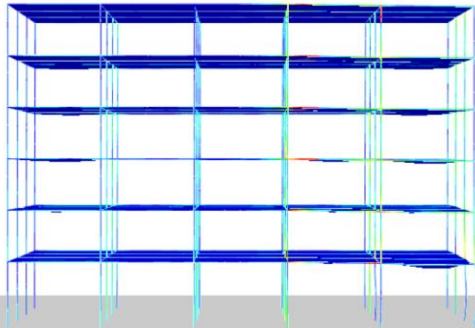
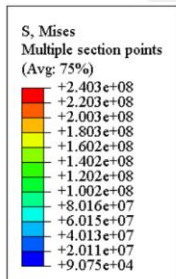
3.6.7. Utilization and Collapse Sequence of Case F-7 Fire Scenario

Table 3.7 presents the utilization and fire resistance characteristics of columns under Case F-7 for four buildings. The results show that critical temperatures (θ_{crit}) vary between approximately 603-902 °C depending on the system and column, with SS and HYB generally maintaining higher thresholds, particularly in A6 and B6 columns. The critical times (t_{crit}) also reveal distinct performance differences: SS and HYB consistently provide longer fire endurance, with values reaching up to 68 minutes, while CS and PCS often fall within a lower range of 17–74 minutes. Overall, the table highlights that SS and HYB systems demonstrate superior fire resistance compared to CS and PCS, especially in heavily loaded columns, while PCS benefits from lower axial loads but remains more vulnerable to collapse over extended fire exposure.

(Figures 84-87) compare the progressive collapse behavior of the four building systems under the F-7 fire scenario. The CS building experienced rapid instability and total collapse within 47 min, with PCS lasting slightly longer but still showing full structural failure around 75 minutes at ~800°C. In contrast, the SS and HYB systems demonstrated much higher fire resistance, withstanding exposure up to 240 minutes without total collapse, despite significant stress redistribution and localized damage. Overall, SS and HYB are the most effective systems under fire conditions, maintaining structural stability for extended durations, while PCS, although slightly better than CS, could not prevent complete collapse under prolonged fire exposure.

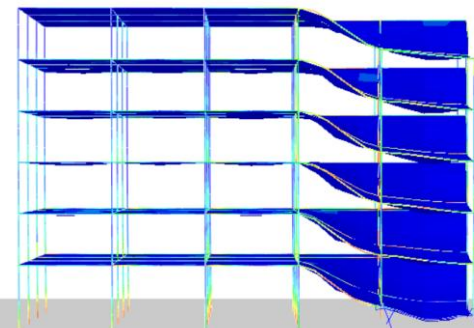
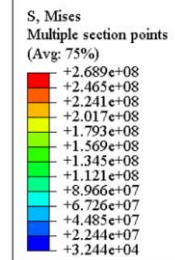
Table 3.7. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-7

Column	System	θ_{crit} (°C)	$N_{Rd,\theta}$ (kN)	η_θ	t_{crit} (min)
A5	CS	702.4	735.2	1.2086	17
	SS	801.4	1058.4	1.0089	28
	HYB	801.4	1058.4	1.0402	28
	PCS	701	735.2	1.2382	57
A6	CS	702.4	344.9	1.3024	17
	SS	902	289.7	1.0445	50
	HYB	902	289.7	1.1149	50
	PCS	701	344.9	1.1186	57
B5	CS	616.8	1985.1	1.1094	34
	SS	616.8	2417.9	1.0149	34
	HYB	616.8	2417.9	1.0188	34
	PCS	603.7	1985.1	1.0984	59
B6	CS	702.4	735.2	1.2626	39
	SS	902	548.8	1.6442	67
	HYB	902	548.8	1.6223	68
	PCS	701	735.2	1.2050	74



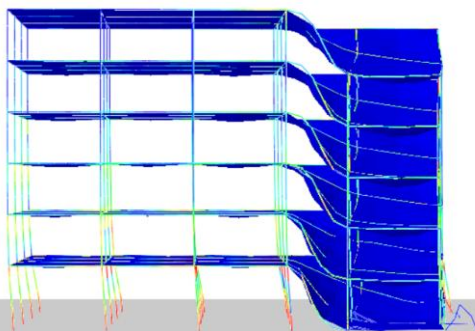
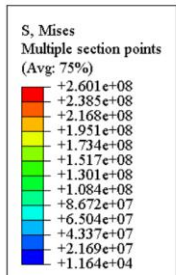
Step Time = 16 min

Temperature = 747.4°C



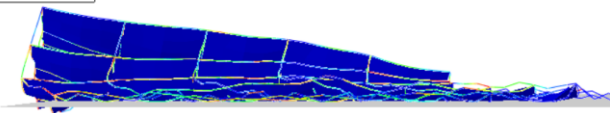
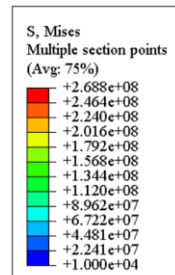
Step Time = 21 min

Temperature = 787.8°C



Step Time = 42 min

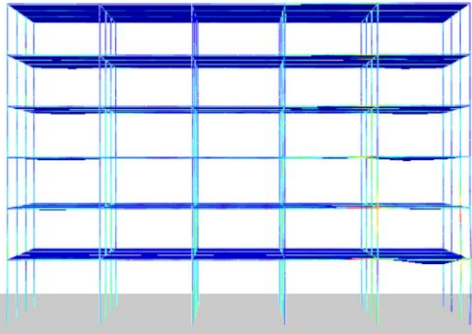
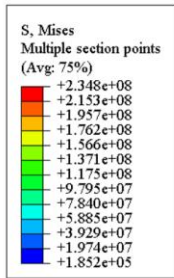
Temperature = 891.8°C



Step Time = 47 min

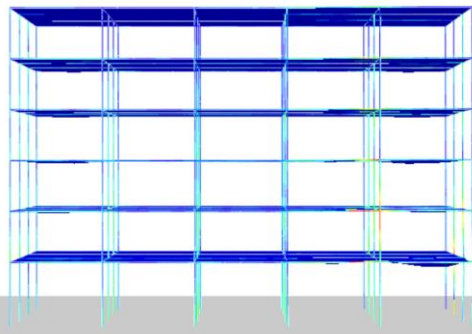
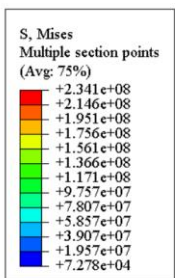
Temperature = 908.4°C

Figure 84. Progressive collapse sequence of the CS building under Case F-7 fire scenario



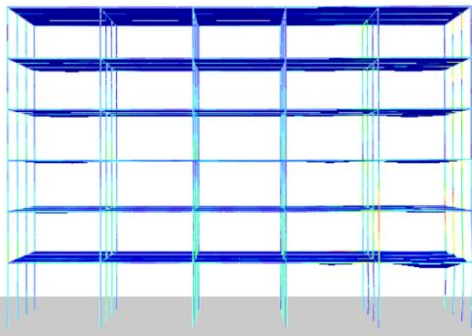
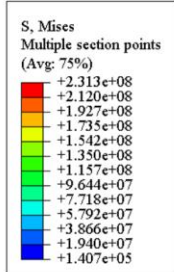
Step Time = 16 min

Temperature = 747.4°C



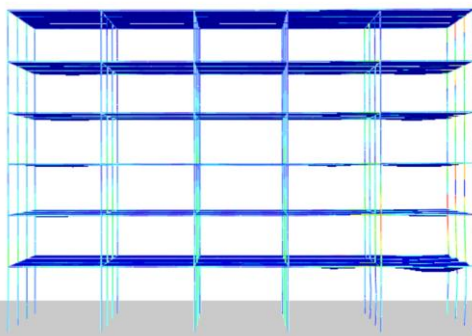
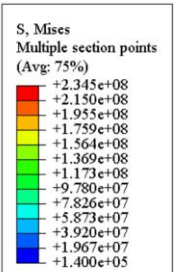
Step Time = 28min

Temperature = 831.2°C



Step Time = 100min

Temperature = 1022°C



Step Time = 180 min

Temperature = 1049°C

Figure 85. Progressive collapse sequence of the SS building under Case F-7 fire scenario

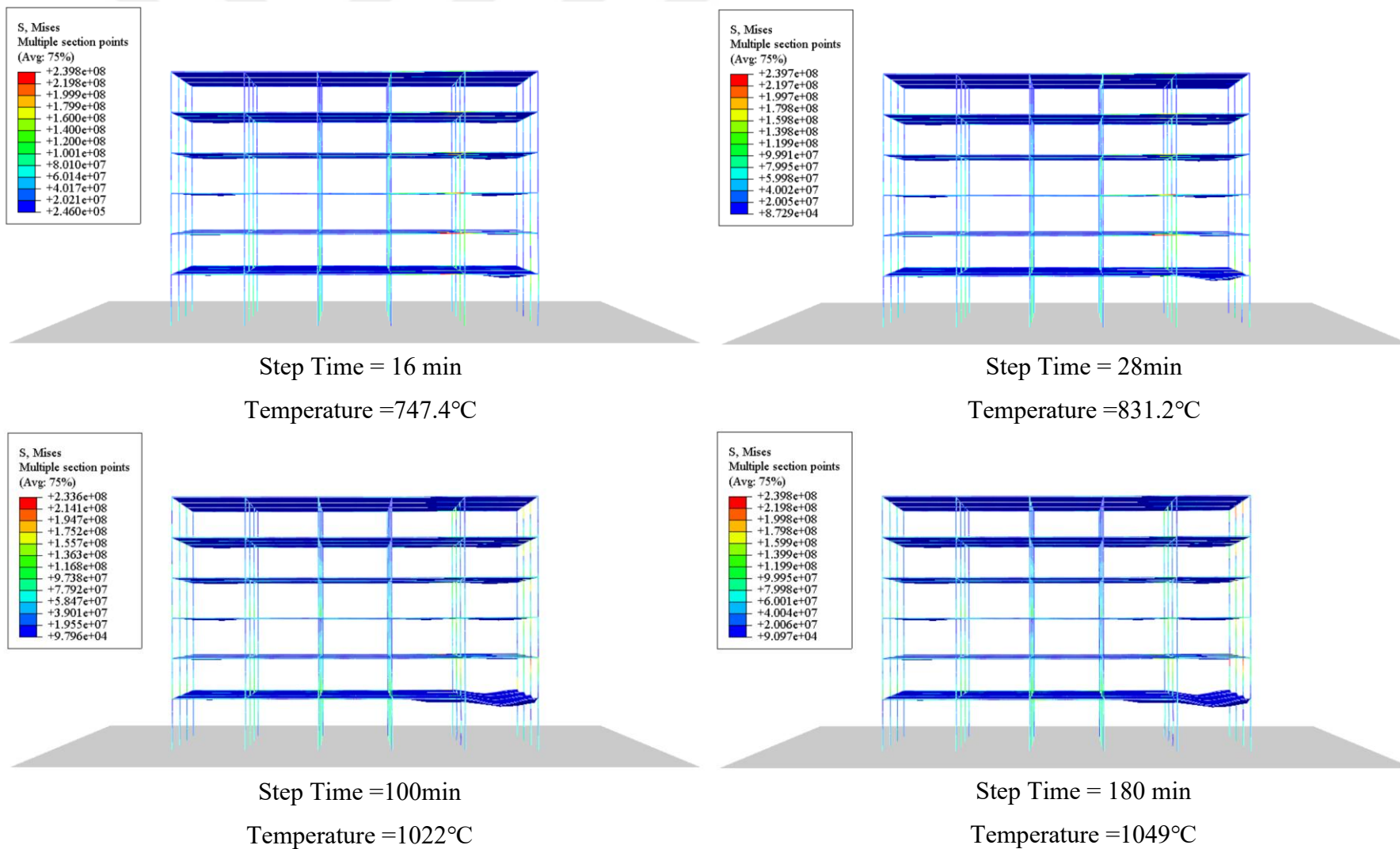


Figure 86. Progressive collapse sequence of the HYB building under Case F-7 fire scenario

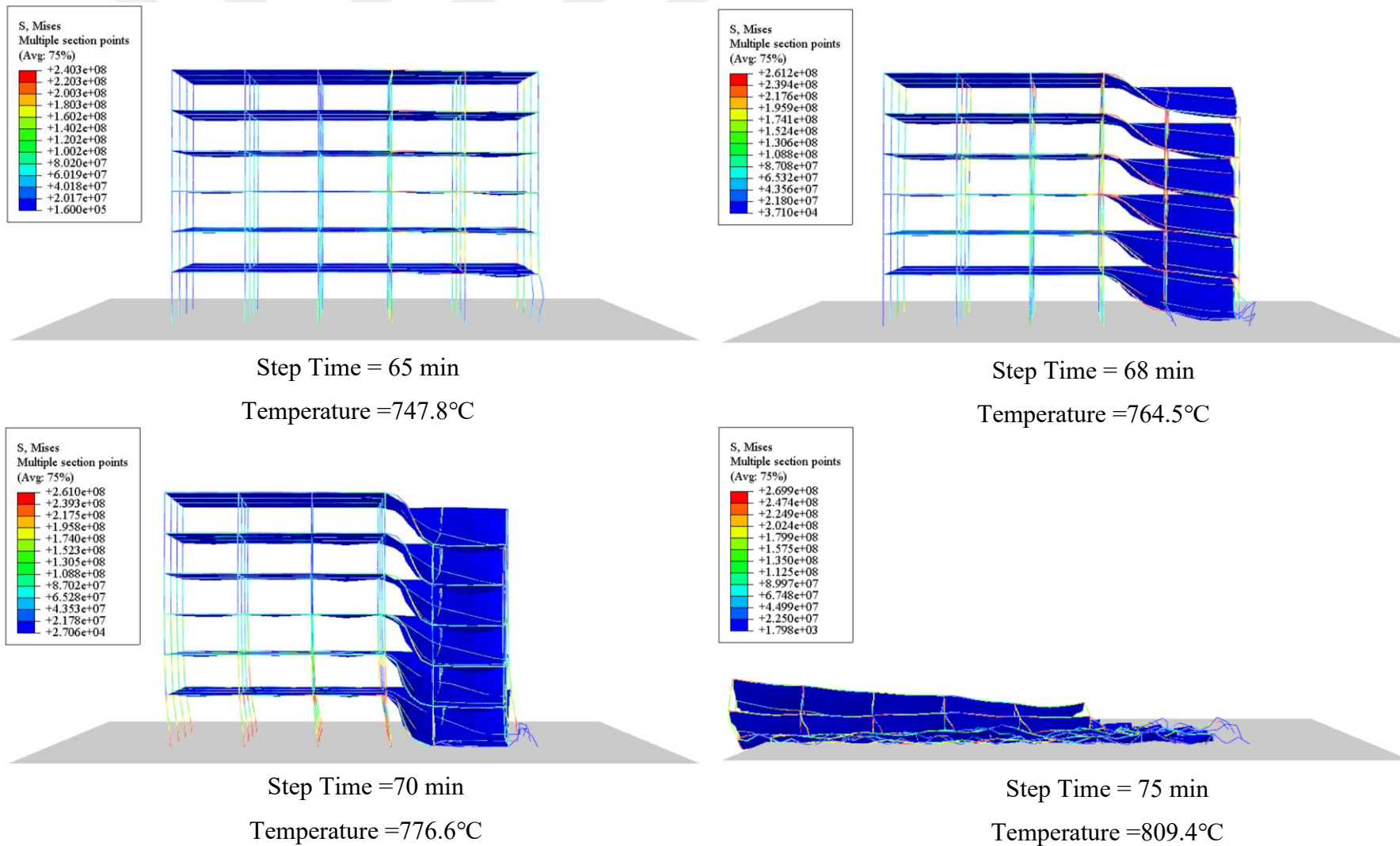


Figure 87. Progressive collapse sequence of the PCS building under Case F-7 fire scenario

3.6.8. Utilization and Collapse Sequence of Case F-8 Fire Scenario

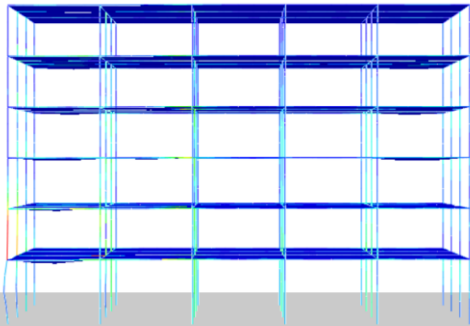
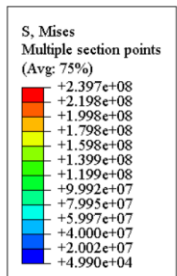
Table 3.8 summarizes the fire resistance of heated columns under Case F-8, highlighting the differences among CS, SS, HYB, and PCS systems. The results indicate that CS columns generally show the lowest fire resistance, with critical times as short as 17-39 minutes. By contrast, SS and HYB systems achieve significantly better performance, with critical times reaching 50-68 minutes due to their higher critical temperatures (902°C). The PCS system demonstrates the best overall stability, particularly in columns A2 and B1, where critical times extend up to 74 minutes, showing improved resistance compared to the other systems. Overall, while SS and HYB enhance fire endurance relative to CS, PCS provides the longest resistance times, confirming its effectiveness in delaying failure, even though ultimate collapse remains unavoidable.

The progressive collapse behavior of four structural systems (CS, SS, HYB, and PCS) under the F-8 fire scenario are given in (Figures 88-91). The results show significant differences in fire resistance and structural robustness. The CS system exhibited the most rapid collapse, with failure occurring in less than 50 minutes at temperatures below 910 °C, indicating limited fire endurance. The PCS system demonstrated better resistance compared to CS, maintaining stability until approximately 75 minutes and higher critical temperatures (~809 °C), although it eventually failed due to progressive loss of load-bearing capacity. The SS system performed more effectively, sustaining the fire for up to 144 minutes withstanding temperatures exceeding 1049 °C before collapse, showing considerably improved fire resistance. Finally, the HYB system showed the cloase performance to SS, delaying collapse to about 111 minutes at temperatures above 1037 °C, indicating enhanced structural integrity under severe thermal exposure.

Table 3.8. Summary of utilization and fire resistance of four building systems- CS, SS, PCS, and HYB for heated columns under case F-8

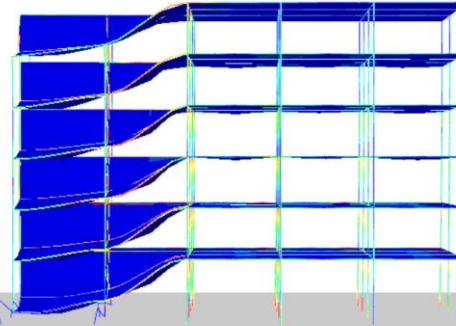
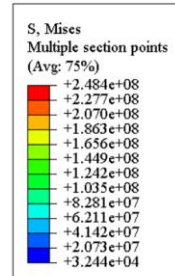
Column	System	θ_{crit} (°C)	$N_{rd,\theta}$ (kN)	η_θ	t_{crit} (min)
A1	CS	702	344.9	1.2851	17
	SS	902	289.7	1.1685	50
	HYB	902	289.7	1.1865	50
	PCS	701	344.9	1.0615	58
A2	CS	702	735.2	1.2643	39
	SS	902	548.8	1.5049	67
	HYB	902	548.8	1.5982	68
	PCS	701	735.2	1.1039	74
B1	CS	702	735.2	1.2630	39
	SS	902	548.8	1.5103	67
	HYB	902	548.8	1.6057	68
	PCS	701	735.2	1.1011	74
B2	CS	616.8	1985.1	1.1092	34
	SS	616.8	2417.9	1.0437	34
	HYB	616.8	2417.9	1.0394	34
	PCS	603.7	1985.1	1.0469	59

In general, for the travelling fire scenarios from F-3 to F-8, the critical time (t_{crit}) is often reached earlier in systems without thermal protection, as temperature rise occurs rapidly under direct fire exposure. In contrast, systems with thermal insulation—such as intumescent coatings—delay this temperature escalation, resulting in the utilization factor η_θ exceeding 1.0 at a later stage. Therefore, protected systems typically exhibit longer t_{crit} values, even if this does not always translate into delayed total collapse. This general trend was observed consistently across all travelling fire scenarios from F-3 to F-8.



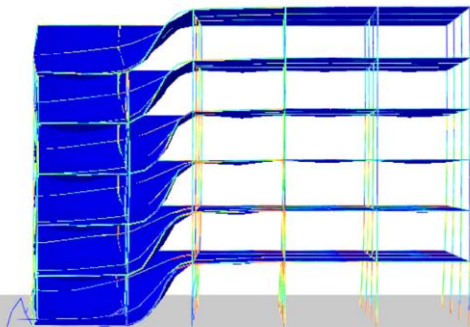
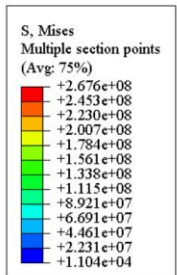
Step Time = 13 min

Temperature = 714.6°C



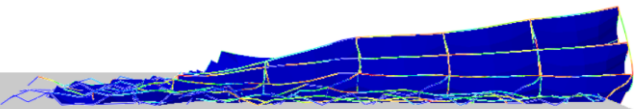
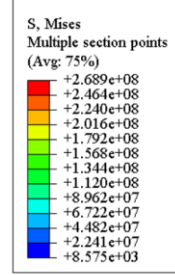
Step Time = 21 min

Temperature = 787.8°C



Step Time = 40 min

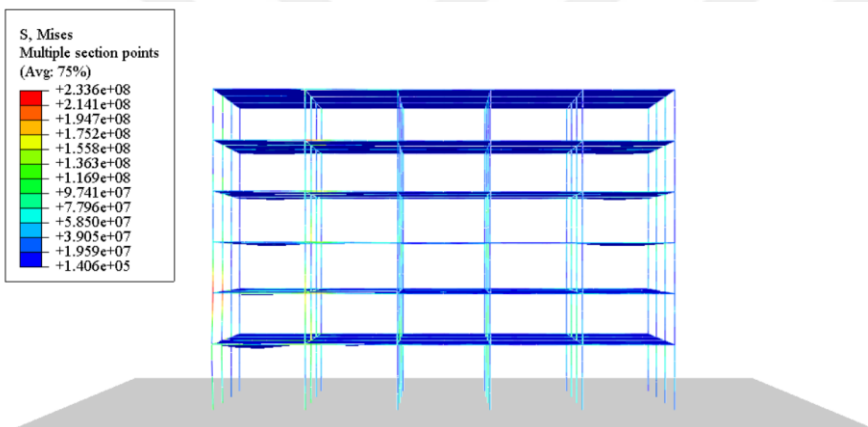
Temperature = 885°C



Step Time = 46 min

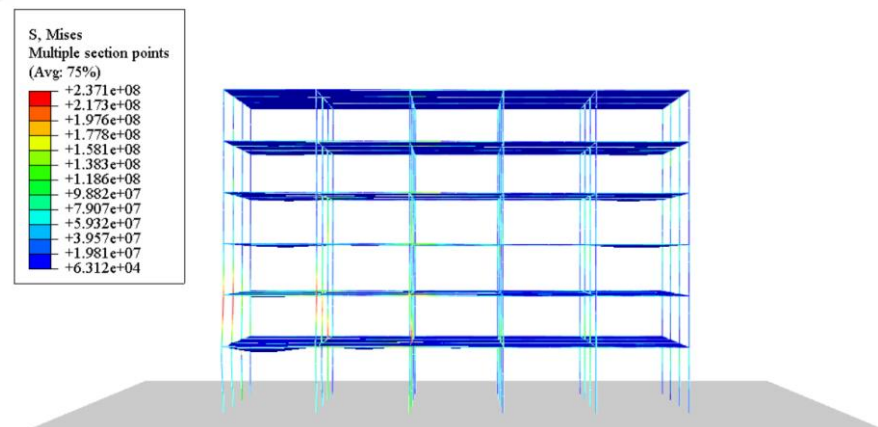
Temperature = 905.2°C

Figure 88. Progressive collapse sequence of the CS building under Case F-8 fire scenario



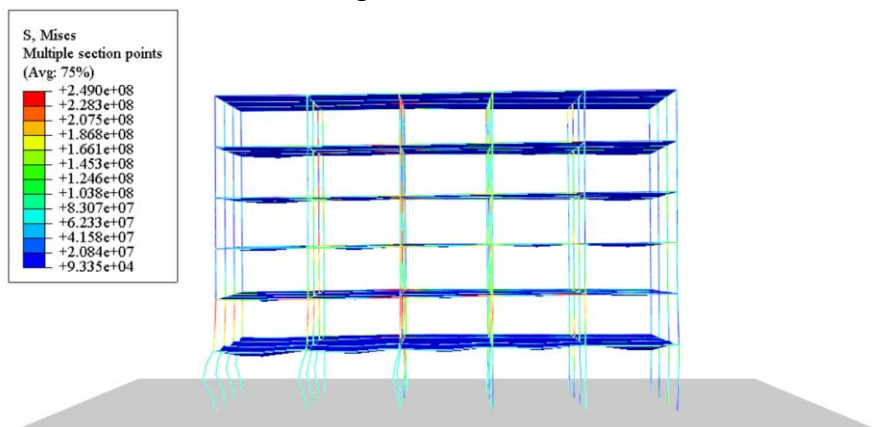
Step Time = 23 min

Temperature =801.4°C



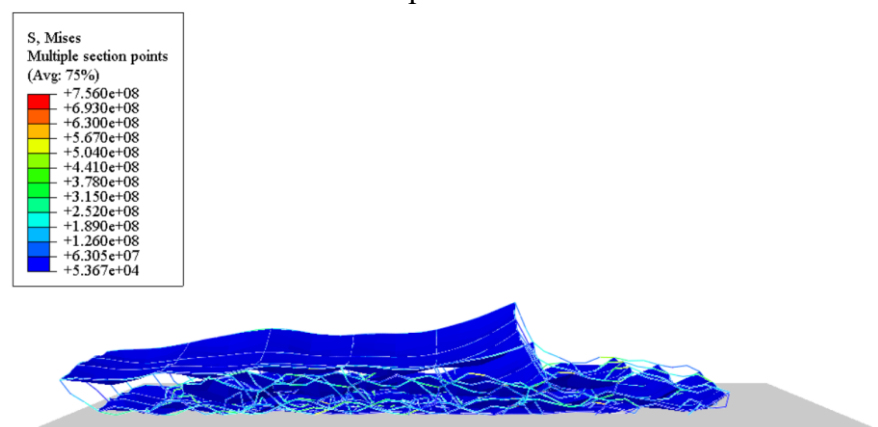
Step Time = 40min

Temperature =885°C



Step Time =136min

Temperature =1049°C



Step Time = 144 min

Temperature =1049°C

Figure 89. Progressive collapse sequence of the SS building under Case F-8 fire scenario

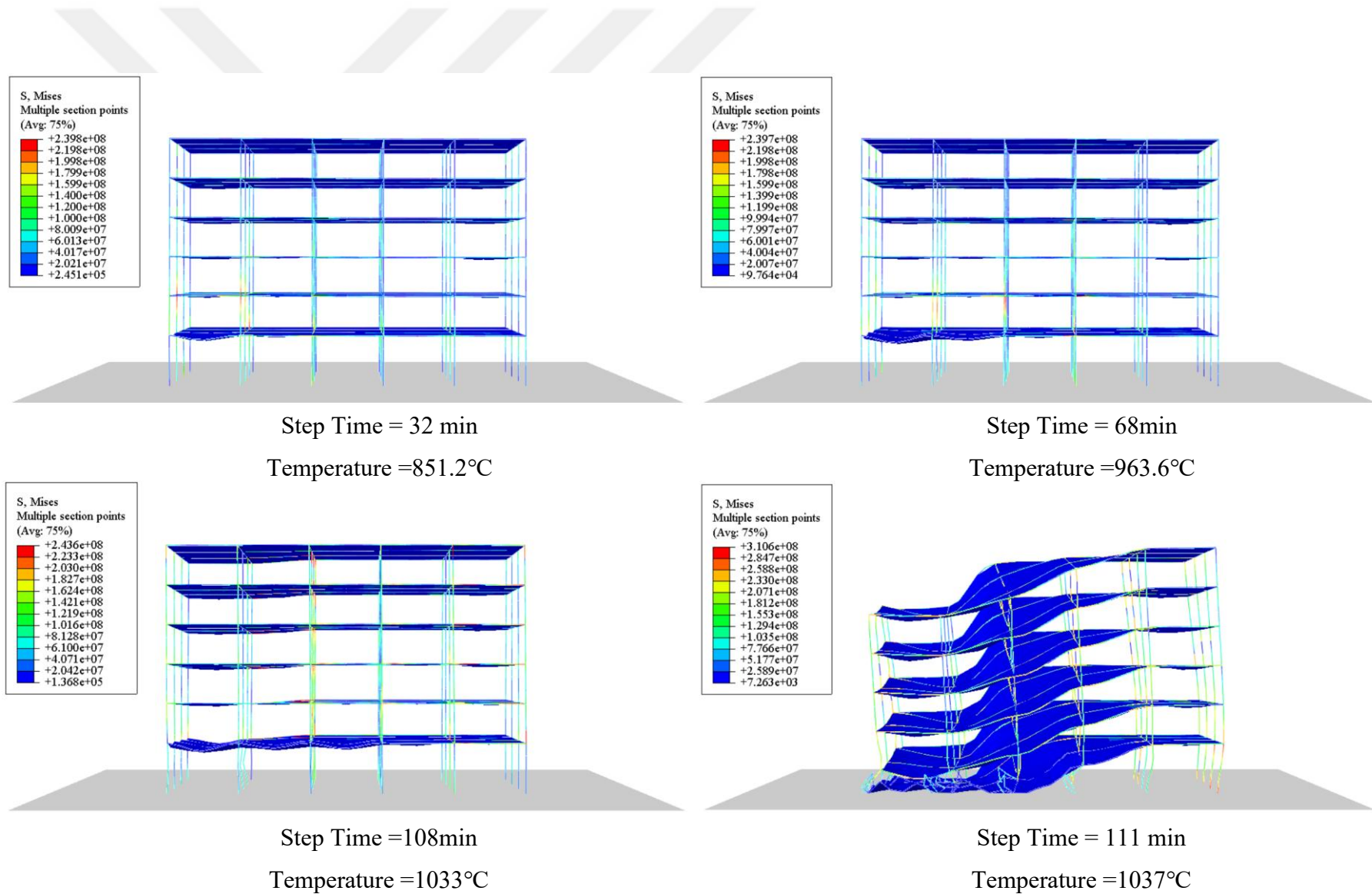


Figure 90. Progressive collapse sequence of the HYB building under Case F-8 fire scenario

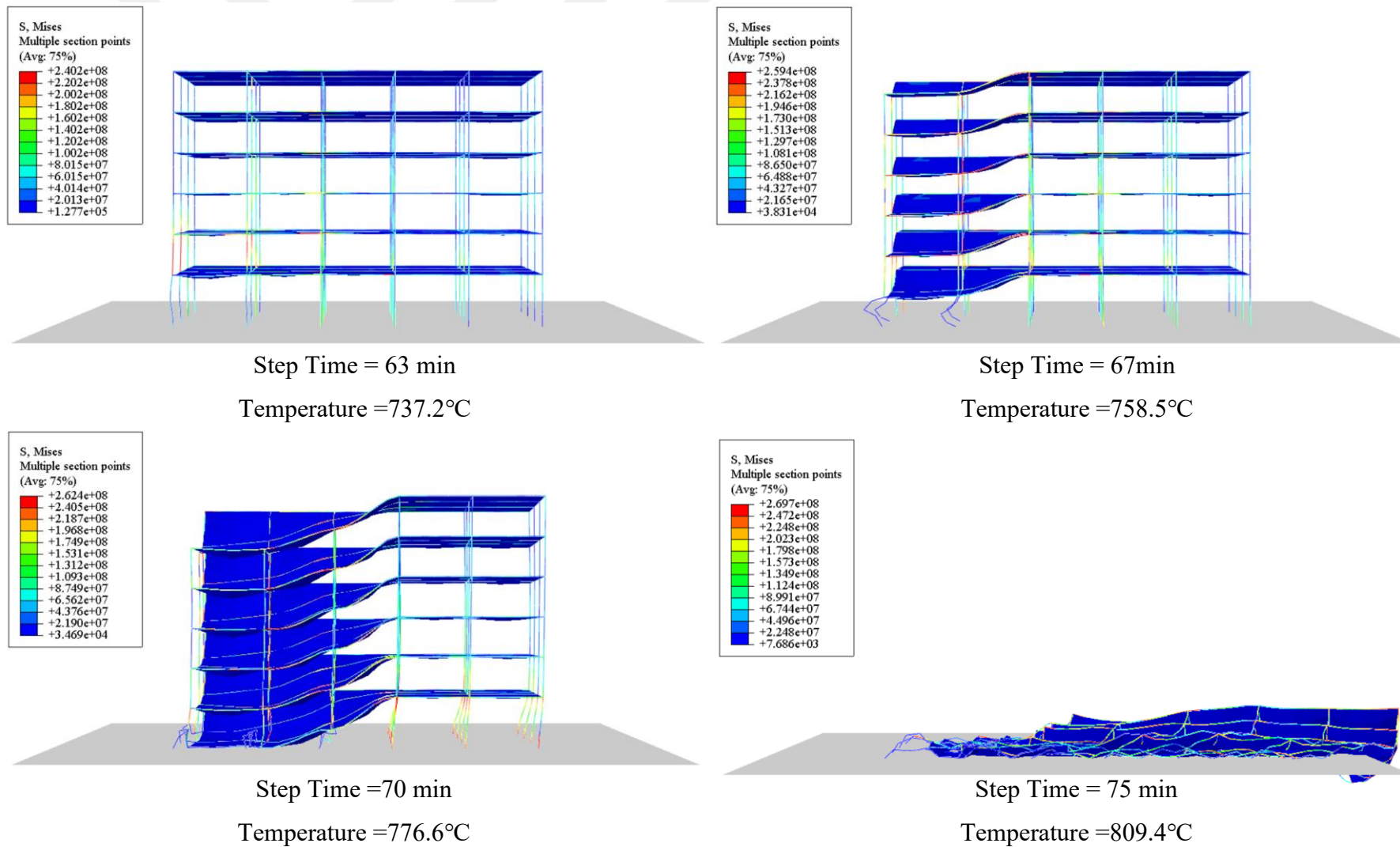


Figure 91. Progressive collapse sequence of the PCS building under Case F-8 fire scenario

4. CONCLUSION AND RECOMMENDATIONS

This study examined fire-induced progressive collapse of a six-storey steel building under different fire scenarios for four material strategies: unprotected carbon steel (CS), intumescent-protected carbon steel (PCS), full austenitic stainless steel (SS), and a hybrid system (HYB) with stainless columns confined to the fire zone and carbon-steel members elsewhere. The key conclusions of this study are as follows:

- It is concluded that numerical progressive collapse analyses can be effectively carried out using the finite element method (FEM) under different fire scenarios
- The comparative evaluation of four structural systems (CS, SS, HYB, and PCS) under different fire scenarios shows that SS and HYB systems provide the most reliable fire resistance and structural stability, while PCS offers intermediate performance, and CS is the least effective
- SS and HYB systems exhibit the best performance, maintaining stability up to 240–270 minutes and resisting collapse even at temperatures exceeding 1000°C.
- PCS performs moderately well, benefiting from lower axial loads but still unable to prevent collapse under prolonged exposure
- CS is the weakest, with rapid collapse occurring within 20–50 minutes in most scenarios
- In edge fires, the heated columns often carry significant gravity loads and are part of the lateral stability system. Once they lose strength, load redistribution is limited because fewer alternative load paths exist at the perimeter. This can trigger progressive collapse earlier, even if fewer columns are heated compared to central or multi-fire cases
- In central fires, interior columns (like B3, C3, etc.) fail, but surrounding cooler columns and perimeter frames often provide alternate paths and redistribute loads more effectively. The collapse may not be as abrupt, even though displacements can be large
- In multi-fire cases, yes, risk is very high because many columns fail simultaneously. But often this scenario is more “severe but less probable.” In reality, a single edge fire may produce faster and less controllable collapse compared to a central fire

- It is recommended that materials posing a higher fire risk be located in central zones of the building rather than along the perimeter, since edge columns provide limited alternative load paths and their early failure can accelerate progressive collapse
- In terms of performance-to-cost efficiency, adopting stainless steel columns only in fire-prone bays (HYB) provides the most balanced and robust solution, while full stainless systems offer the greatest time-to-collapse but at substantially higher material cost, as confirmed by the scenario comparisons and the quantified performance–cost–carbon matrix.

Collectively, the results confirm that CS and PCS frames suffer from early collapse mechanisms in every fire case, while SS and HYB provide markedly enhanced resistance, offering reliable pathways for fire-safe structural design.

In future studies, this work can be extended by:

- examining different building typologies, varied fire scenarios, and alternative material strategies to enhance the generalizability of the results
- taking into account $P-\Delta$ effect
- taking into account the different connection types of beams / slabs and columns
- taking into account different types of coating materials
- examining multi-hazard and post-fire assessment.

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RESUME

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