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Structural Robustness of Steel Structures under Exceptional Event: Column Loss Scenario

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Abstract

Structural robustness ensures the safety and integrity of buildings under unforeseen events such as natural disasters or deliberate attacks. This study investigates the behavior of moment-resistant and pinned joint braced steel structures subjected to the extreme column loss scenario, aiming to identify structural requirements for robust design. The reference building in Brussels is a multi-story office building with ductile joints and restrained supports designed according to Eurocode provisions. Using geometrical and material nonlinear analysis, the behavior of the structure is examined under static loss of a column, with contributions from secondary frames and 3D effects included. The alternative load path method is employed, and analyses are conducted using CEPAO software to account for plastic hinges, second-order effects, and buckling. The findings highlight that while column location influences the structural response, the presence of moment-resistant connections, secondary beams, and catenary action significantly enhances robustness. These insights provide valuable guidance for engineers in designing more resilient steel structures against exceptional events.

Keywords: *Robustness, Steel Structure, Moment Resistant frame, Pinned joint, Column loss*

1. Introduction

Steel frames are widely used in commercial and civil buildings for their ease of construction and strong load-bearing capabilities. However, when these buildings face unexpected stresses during their lifespan, certain steel columns may suffer initial damage. These localized failures can then spread throughout the entire structure, potentially leading to its collapse (Zhang *et al.*, 2020). Framed steel structures are widely used as load-bearing systems in residential, office, and industrial buildings, as well as various supporting structures. Recent events such as natural catastrophes or terrorist attacks have highlighted the necessity to ensure the structural integrity of buildings under an exceptional event (Huvelle *et al.*, 2015). While frame systems are generally designed to meet ultimate and serviceability limit states under normal conditions, accidental scenarios like vehicle impacts, explosions, or terrorist attacks may exceed these design limits (Kukla and Kozlowski, 2023). According to Eurocodes and some other national design codes, the structural integrity of civil engineering structures should be ensured through appropriate measures, but in most cases, no precise practical guidelines for achieving this goal are provided (Huvelle *et al.*, 2015).

Robustness is a critical property of structural systems, ensuring public confidence in infrastructure by maintaining safety and functionality even when unforeseen events occur. Over the past century, the importance of designing structures capable of withstanding exceptional incidents has gained significant attention, making structural robustness a key area of research. Various design codes incorporate robustness requirements; however, simple yet accurate methods for practical assessment of building robustness remain scarce, even for static structural behaviour (Comeliau, Rossi and Demonceau, 2012).

This paper examines the behaviour of steel structures in response to the unexpected loss of a column, with a primary focus on the structure's static behaviour. Firstly, it presents the overarching philosophy and strategies for establishing design requirements. It will then discuss the findings from a parametric study of these structures. Moment-resistant and pinned-joint braced multi-storey steel structures are considered for evaluation of structural behaviour.

2. General philosophy and adopted strategy

In recent decades, extensive research has been conducted on the collapse mechanisms of steel frames following column loss. However, due to the complex interactions among structural members during collapse, fully understanding the behavior of the entire frame remains challenging. To address this, researchers have adopted reduced models, often simplifying or neglecting the concrete floor system, to assess the robustness of steel frames under column loss (Stylianidis *et al.*, 2016) (Jiang *et al.*, 2018) (Li *et al.*, 2018). This study focuses exclusively on frames composed of columns and beams, deliberately neglecting the potentially beneficial effects of slabs in the analysis. When a frame experiences the loss of a column, the structure can be divided into two parts: the directly affected part and the indirectly affected part (Jean-François Demonceau, 2008)(Huvelle, Jaspart and Demonceau, 2014).

The *directly affected part* includes all beams, columns, and beam-to-column joints surrounding the lost column (see Figure 1). In contrast, the *indirectly affected part* encompasses the remaining structure, including the lateral portions and the stories beneath the lost column.

When a column is lost (e.g., column AB in Figure 1a), the evolution of the axial force N_{AB} in the column versus the vertical displacement u at its top progresses through three distinct phases, as depicted in Figure 1.

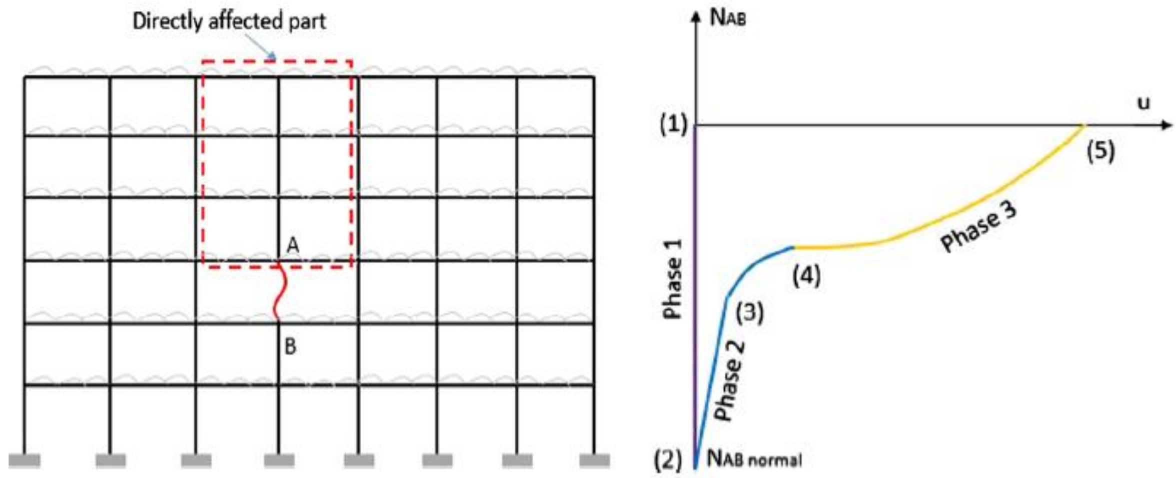
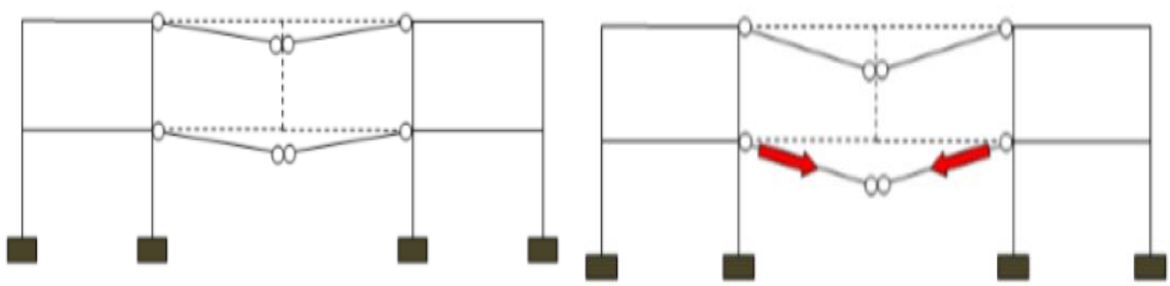


Figure 1: Behaviour of a frame submitted to a column loss (Huvelle *et al.*, 2015)(Jean-François Demonceau, 2008).

Phase 1 (Pre-event: from Point (1) to Point (2) in Figure 1b)

Before the event, the column operates under normal loading conditions, supporting loads transferred from the upper stories. This load is denoted as $N_{ABnormal}$.



(a) (b)

Figure 2: (a) Phase 2, and Phase 3 (Huvelle, Jaspart and Demonceau, 2014)

Phase 2 (Column failure: from Point (2) to Point (4) in Figure 1b)

This phase begins at the moment of the event when the column gradually loses its axial resistance. Concurrently, a plastic mechanism develops in the directly affected part of the structure. Each change in the slope of the curve in Figure 1b corresponds to the formation of a new plastic hinge in the directly affected part, culminating in a complete plastic mechanism at Point (4).

Phase 3 (Post-mechanism: from Point (4) to Point (5) in Figure 1b)

Once the plastic mechanism is fully developed, the vertical displacement at the top of the lost column increases significantly as the structure loses first-order rigidity. During this phase, catenary actions progressively form in the beams of the directly affected part, providing second-order stiffness to the structure. The indirectly affected part plays a crucial role by offering lateral anchorage for these catenary actions. Greater lateral stiffness in the indirectly affected part leads to stronger catenary actions in the directly affected part. Conversely, if the indirectly affected part has no lateral stiffness, catenary actions will not develop, and Phase 3 will not occur.

The behavior of the structure from Point (2) to Point (5) in Figure 1b can be predicted using the simulation shown in Figure 2. In this simulation, the frame—without the lost column AB—is subjected to a concentrated downward load P applied at Node A.

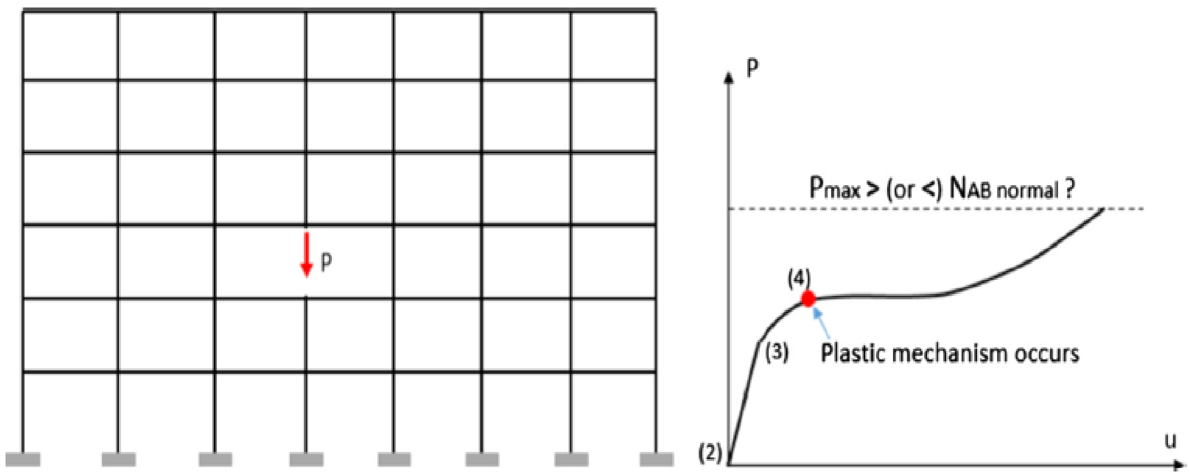


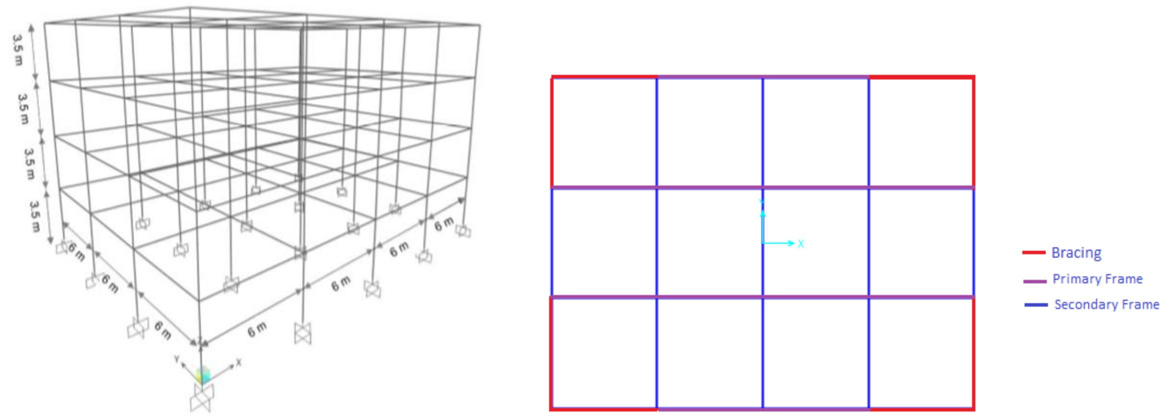
Figure 3: Simulation of column loss (Huvelle *et al.*, 2015).

The analytical method developed in Liège aims to derive a $P\!-\!u$ curve that characterizes the behavior of the simulated structure (see Figure 3), estimate the redistribution of loads during these phases, and determine whether the structure can reach Point (5) (i.e., when $P\!=\!N_{ABnormal}$. Achieving this point indicates that the damaged structure possesses sufficient resistance and ductility to sustain the large displacements and associated forces arising from the activation of alternative load paths (Huvelle *et al.*, 2015).

3. Structural response under column loss

3.1 Geometry configuration of the structure

The reference building selected for this analysis is a four-story office building located in Brussels, Belgium. The choice of this type building was predicated on its alignment with the study's principal aim: to assess structural responses in scenarios involving column loss. Office buildings were specifically chosen due to their typical high occupancy and heightened susceptibility to disproportionate collapse following unexpected incidents such as accidental impacts, explosions, or extreme loading conditions. This renders them pivotal cases for evaluating structural resilience. Moreover, office buildings commonly exhibit open-plan layouts with fewer structural redundancies, thereby amplifying their vulnerability in scenarios involving column loss. The building features a structural layout consisting of four spans in the X-direction, each with a span length of 6 meters, and three spans in the Y-direction, also with a span length of 6 meters. Each story has a height of 3.5 meters, as illustrated in Figure 4.

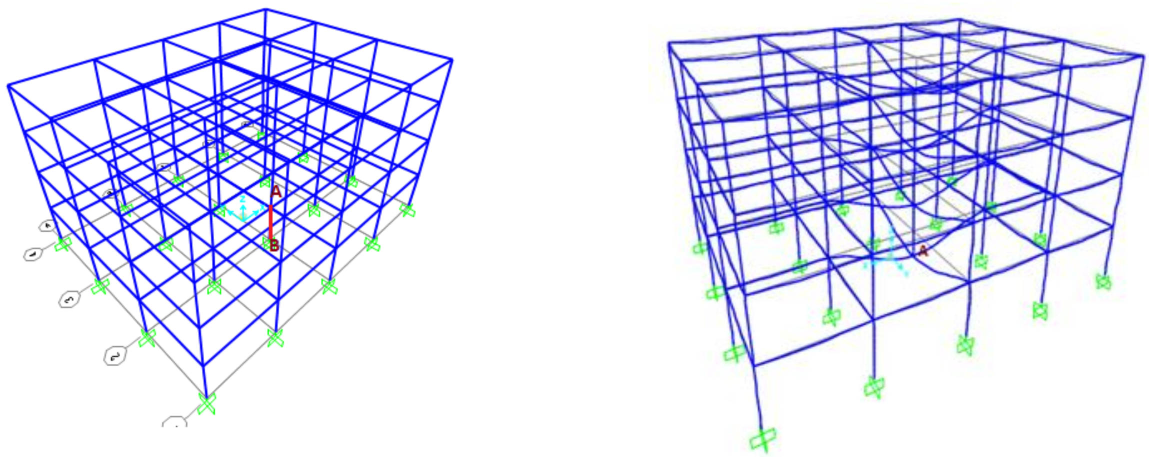


(a) (b)

Figure 4: (a) Reference 3D-Steel Structure of Office Building, and (b) Plan view of the 3D structure

The reference structure is designed and analysis with SAP2000 as per EN1991-1-1 (European Committe for Standardisation, 1994), without considering the seismic effect, with fully moment-resistant and ductile joints supported by restrained connections, ensuring adequate rotational capacity. However, verifying the structure's sufficient ductility to accommodate the maximum deflection required to develop catenary action without failure is essential. The structural design complies with the provisions of the Eurocode (EN 1990) (McKenzie, 2013), meeting the requirements for ultimate and serviceability limit state checks. The material used is mild steel grade S355 according to EN10025 (European Committee for Standardization, no date) with elastic-perfectly plastic behaviour. Figures 5 and 6 show the structural models of moment-resistant and pinned-joint structures in SAP2000 before and after the column loss.

During the structural analysis and verification of structural members using SAP2000, the composite action between the concrete slab and steel beams was not considered. Secondary beams were assumed to be continuously supported and pinned. The secondary beams are positioned between the primary beams, spanning across the four-story office building's three-span (Y-direction) and four-span (X-direction) layout. They are oriented perpendicular to the primary beams, ensuring effective force redistribution across the structure. These beams contribute significantly to load-sharing mechanisms in both moment-resistant and pinned-joint braced structures. The moment-resistant structure utilized IPE270 for external frames and IPE360 for internal frames, conforming to Eurocode standards. Similarly, the pinned joint structure employed IPE360 for external frames and IPE450 for internal frames. Column sections included HE320B, HE240B, HE220B, HE200B, HE180B, HE160B, and HE140B, with bracing provided by IPE120, all verified according to relevant standards.



(a) (b)

Figure 5: Moment resistant structure: a) Structure before column loss, and b) Structure after column loss

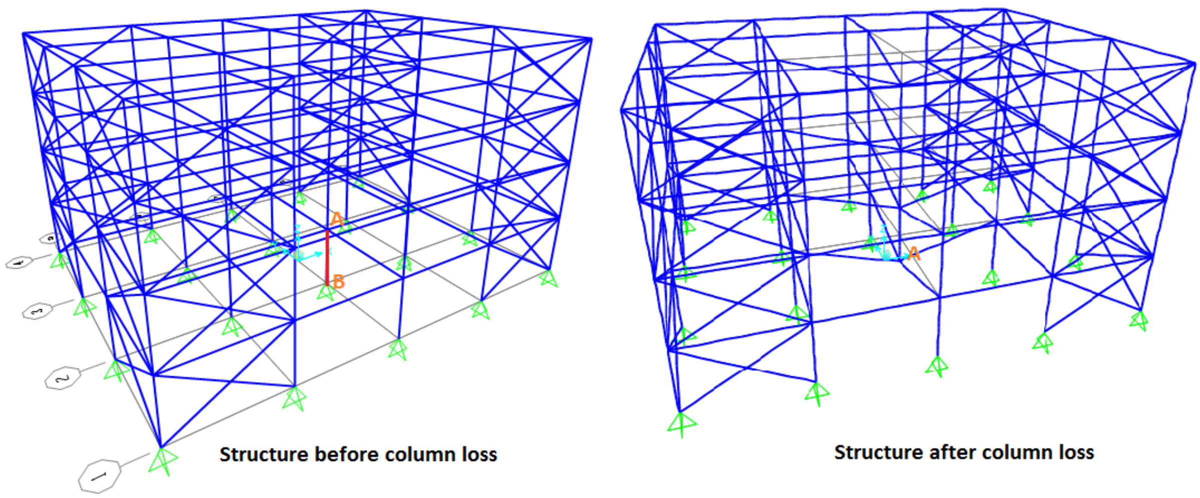


Figure 6: Pinned joint structure before and after column loss

3.2 Methodology

The design approach outlined in the section general philosophy and adopted strategy is employed to study the redistribution of forces in the structure during the static loss of a column. The investigation is performed using a geometrical and material non-linear analysis. The alternative load path method is applied according to EN 1991-1-7 (EN 1991-1-7, 2006), considering elasto-plastic behavior and second-order effects. The study employs the CEPAO program, developed by the University of Liège's Department of Structural Mechanics and Stability, a comprehensive tool for the automated analysis and design of frame structures (Dang Hung, 1984), to perform a geometrical and material non-linear analysis of the moment-resistant and pinned-joint braced steel structures under a column loss scenario. Initially designed by Nguyen-Dang Hung (Dang Hung, 1984), it provided a unified approach for 2D frame structures, incorporating elastic analysis, rigid-plastic limit analysis with proportional loading, step-by-step elastic-plastic analysis, shakedown analysis under variable repeated loads, and optimal plastic design with stability checks and discrete profile selection (Van Long and Dang Hung, 2008). Over time, CEPAO has evolved to support space steel frames, integrating rigid-plastic analysis and design through linear programming (LP) and elastic-plastic analysis using step-by-step methods, considering both first and second-order effects (P-delta) (Van Long and Dang Hung, 2008) (Long and Hung, 2010). The program employs classical finite element methods combined with LP, enabling precise calculations of ultimate load capacity, bending moment distribution, axial and shear forces, and structural collapse mechanisms. CEPAO remains a powerful and evolving tool for structural analysis and optimization, offering advanced capabilities for engineers designing resilient and efficient frame structures (Van Long and Dang Hung, 2008) (Long and Hung, 2010) (Hoang *et al.*, 2015) (Li, 2008).

The main roles of CEPAO in this study are to simulate the progressive collapse response of the structure, incorporating the Plastic hinge formation to capture the redistribution of internal forces, Second-order (P-Δ) effects to account for instability due to large deformations, buckling analysis to evaluate local and global stability under exceptional loads. In addition to this, the software applies the Alternative Load Path Method to assess structural robustness after the sudden removal of a column and the analysis determines the load factor (λ) vs. vertical displacement relationship, identifying critical points where plastic hinges, catenary action, and collapse occur. Following assumptions are made during the CEPAO simulations;

i. Elasto-plastic material behavior: The structural elements follow an elastic-perfectly plastic stress-strain relationship (S355 steel as per EN 10025).
ii. Idealized beam-to-column connections:

- Moment-resistant structure: Fully ductile joints with rotational capacity.
- Pinned-joint braced structure: Hinged connections allowing free rotation.

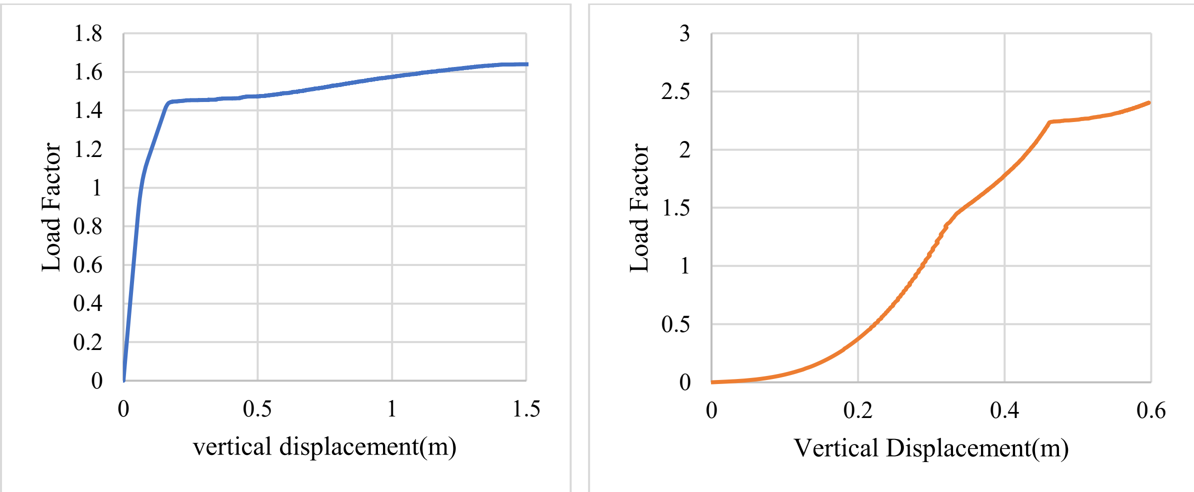
iii. Horizontal bracing system used in place of the concrete slab: Since the real structure lacks horizontal bracing, CEPAO introduces a strong, stiff horizontal bracing system to maintain numerical stability in the absence of diaphragm action.

iv. Load application: The removed column's initial axial force is applied as a downward load (P) at the column location to replicate real-world load transfer.

The methodology follows Eurocode EN 1991-1-7 (EN 1991-1-7, 2006), ensuring compliance with standardized robustness assessment frameworks. CEPAO has been validated against experimental tests and analytical models in prior research (Van Long and Dang Hung, 2008) (Long and Hung, 2010) (Hoang *et al.*, 2015), including PhD studies at the University of Liège (Li, 2008).

3.3 Static Behaviour of the Structure

This section investigates the static behavior of the structure following the loss of a column. The structure's robustness is evaluated based on its global stability during the numerical analysis. The study emphasizes the role of catenary action and load redistribution in enhancing the robustness of moment-resistant and pinned-joint braced steel structures under a column loss scenario. If the structure remains stable until a load factor λ=1 (representing the complete loss of the column), it is deemed robust. Figure 7 presents the evolution of the vertical displacement at the top of the lost column A as shown in Figures 5 and 6 respectively as a function of the load factor λ under accidental limit state load combinations.



(a) (b)

Figure 7: Vertical displacement at the top of the failing column versus load factor, (a) Moment resistant structure, and (b) Pinned Joint structure

After the loss of a column, the structure undergoes a progressive redistribution of loads from the directly affected area to adjacent structural members. The load factor (λ) exceeding 1 across all column loss scenarios confirms the sufficient redundancy of the structure. Load redistribution is primarily facilitated by two key mechanisms, Flexural behavior (Phases 1 & 2), Initial redistribution occurs through beam bending, with plastic hinges forming at beam ends and Catenary action (Phase 3): as displacement increases, axial tension in beams activates catenary forces, further stabilizing the structure.

Flexural Behavior

This behavior is primarily associated with the bending of beams in the directly affected region and serves as the dominant mechanism during Phases 1 and 2.

Phase 1: Elastic deformation

The development of plastic hinges is a fundamental aspect of the analysis, as it indicates the redistribution of loads following a column removal. During the first phase, the structure behaves elastically. A plastic hinge forms at the joint when the moment at the beam ends reaches the resistance moment of the joint, which is lower than the plastic moment of the beam. The elastic stage ends at a load factor λ=0.80 for moment-resistant structure (see Fig. 7a). Similarly, the structure remains within its elastic limit until λ = 1.30, where the first yielding occurs in the beam directly above the lost column due to tension in pinned joint structure (see Fig. 7b). Up to this point, the structure maintains its initial stiffness, and no permanent deformations are observed.

Phase 2: Plastic deformation

Beyond λ=0.80 in the case of moment-resistant structure, plastic hinges begin to form quasi-simultaneously on all floors. Each change in slope in the load-displacement curve corresponds to forming a new plastic hinge. At λ=1.44, the global beam plastic mechanism develops in the directly affected part of the structure, marking the start of Phase 3 (see Fig. 7a). The mechanism is completed when plastic hinges develop at the mid-span of the double beams on each floor in the directly affected area. In the case of a pinned joint braced structure, after λ = 1.30, the structure enters the plastic deformation phase, where permanent deformations begin to accumulate. Initially, yielding spreads to beams in the directly affected part, but the structure remains stable and can still withstand additional loads. As the load factor increases further, secondary plastic hinges begin to form, and redistribution mechanisms such as catenary action become dominant.

A key aspect of the observed flexural behavior lies in the bending of the directly affected region, influenced by the interaction between the primary and secondary frames. In the 3D structure, the extended area of the directly affected region leads to the onset of the beam plastic mechanism at $\lambda = 1.44$, which is the transition point from flexural to catenary action occurs, marking the onset of significant tension in beams. Similarly, the pinned joint braced structure shows greater reliance on axial forces, activating catenary action earlier (around $\lambda = 1.30$) due to its limited moment resistance.

Member Behavior (Catenary Action)

This behaviour involves significant tensile forces (catenary action) developing in beams directly affected by a plastic mechanism. These effects enhance load-carrying capacity, allowing the structure to endure column loss until a complete plastic mechanism forms in the frame.

Additionally, "member behaviour" occurs when tensile forces are generated in beams directly above the lost column, influenced by both primary and secondary frames. In a 3D structure, having more beams above the lost column increases load transfer paths to indirectly affected areas, improving the structure's ability to support exceptional loads after column loss. The catenary mechanism increases the ultimate load-bearing capacity, delaying collapse even after significant deformation. This study highlights that catenary action is crucial for maintaining stability, especially in structures with limited moment-resisting capacity. Incorporating stronger beam-to-column connections and lateral restraints can further enhance load redistribution efficiency.

Phase 3: Catenary Action and Membrane Effects

Once the plastic mechanism forms in the directly affected area, the vertical displacement at the top of the lost column increases rapidly due to the loss of bending stiffness in the joints of both the primary and secondary frames. Tension forces develop in the bottom beam above the lost column, activating the axial stiffness of the beam. These membrane effects reduce the deformation rate until yielding begins in the indirectly affected area.

The failure mode is characterized by forming a plastic mechanism in the indirectly affected area at $\lambda = 1.64$ in moment-resistant structures (see Figure 8). Although the structure can carry additional load, it becomes unstable once the column is removed.

From graph Figure 7b, it is observed that there is a change in the slope of the curve at $\lambda = 2.233$, indicating the yielding of beams in the indirect area and the buckling of adjacent columns next to the column loss, signalling a reduction in structural stiffness. Ultimately, at $\lambda = 2.40$, the structure can no longer sustain additional load and collapses due to the yielding of the braces and beams in the indirectly affected area (see Figure 9). However, the reference structure is sufficiently robust to withstand the additional load resulting from the exceptional event of column loss. The first yielding point in the structure occurs only after the complete loss of the column.

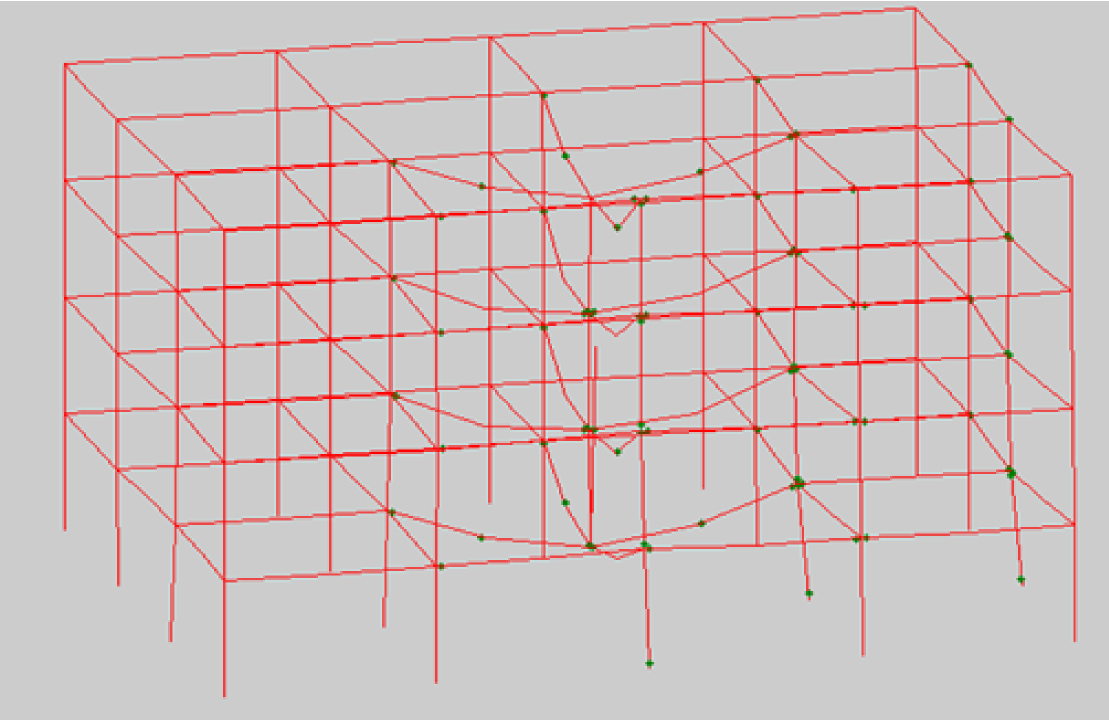


Figure 8: Plastic Mechanism in the indirectly affected parts.

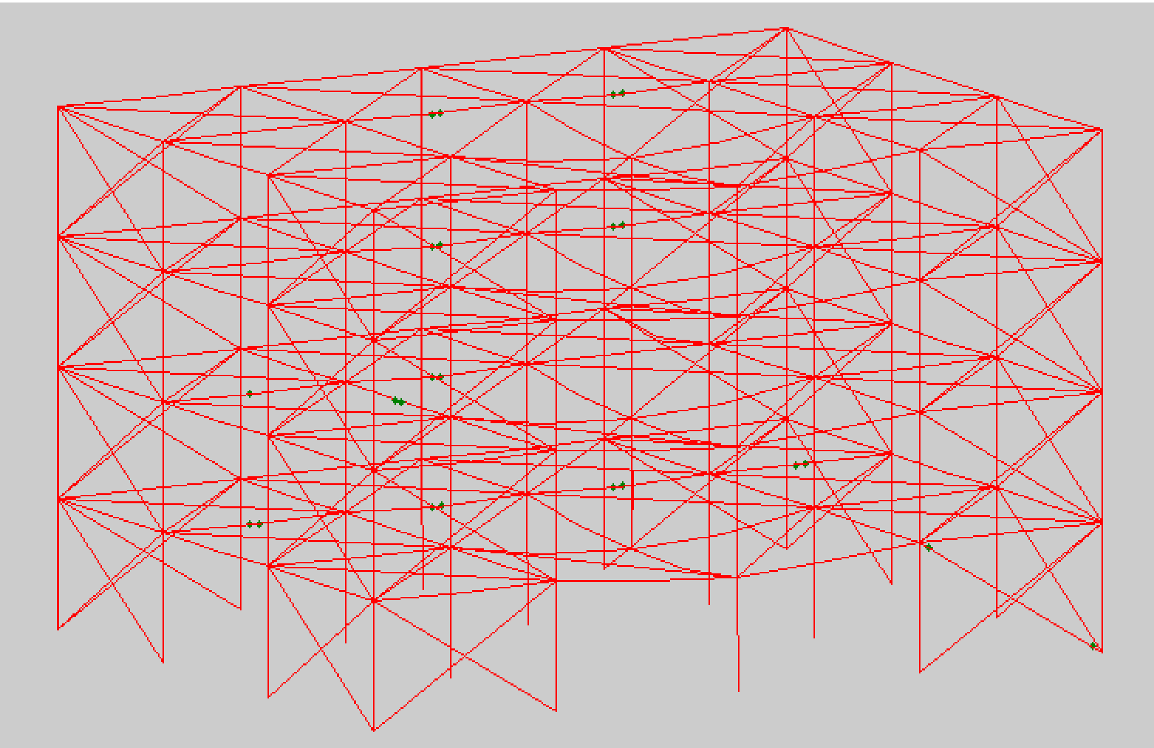


Figure 9: Yielding mechanism in the indirectly affected parts of structure.

3.4 Column Loss at Different Positions

Figures 10, 11, and 12 illustrate the structure's behaviour under the loss of columns located at different positions within the structural grid. Figure 10 highlights the positions of the columns considered for removal (Col-1 to Col-6). Figures 11 presents the corresponding load factor (λ) versus vertical displacement curves for each column loss scenario for moment-resistant structure. Regardless of the column location, the structure shows consistent behaviour, maintaining stability and sufficient load-carrying capacity. The load factor (λ) exceeds unity in all cases, confirming that the structure can withstand the complete loss of any single column without failure.

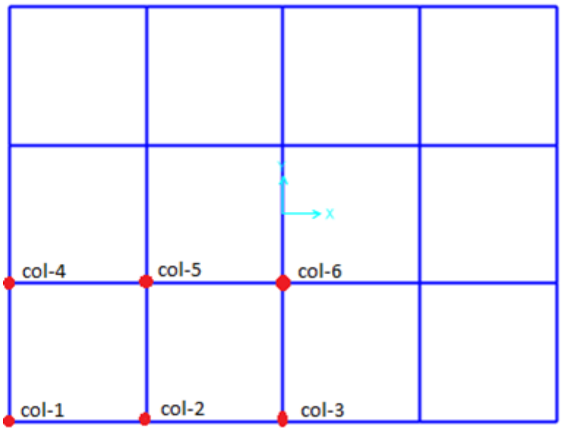


Figure 10: Position of column loss

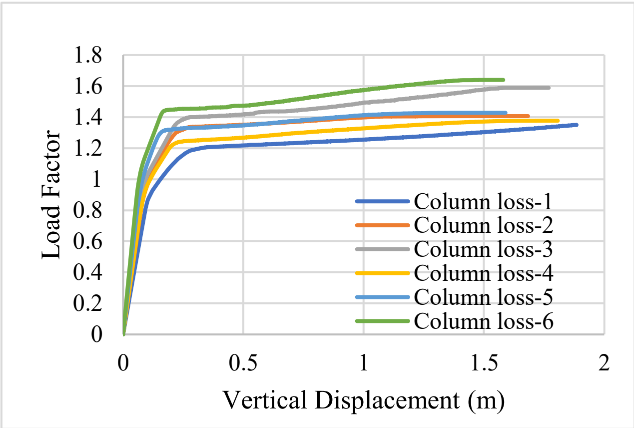


Figure 11: Structure behaviour after the loss of column- Moment Resistant Structure

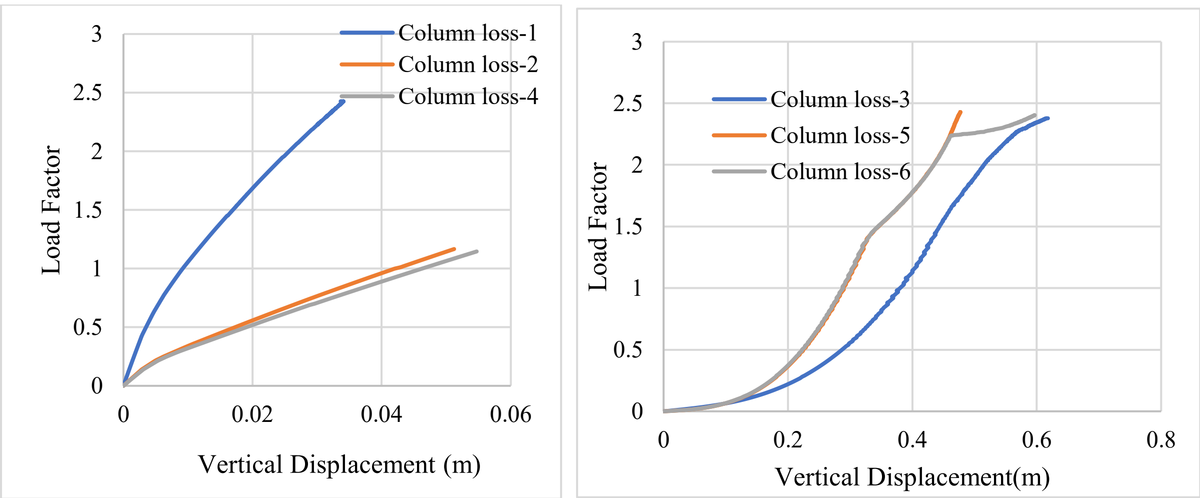


Figure 12: Structure behaviour after loss of column- Pinned Joint Braced Structure.

Figure 12 illustrates the structural behavior of the Pinned Joint Braced Structure under different column loss scenarios by presenting the relationship between load factor (λ) and vertical displacement at the location of the lost column. The curves in the figure 12 represent different column loss positions, highlighting the variations in structural response based on the location of the missing column. The initial phase of each curve shows a gradual increase in vertical displacement, which corresponds to elastic deformation. As the load factor increases, plastic hinges start forming, leading to a nonlinear response, with curves of column 3-5-6 loss showing steeper slopes due to higher flexibility in those specific locations. The structure shows higher displacement at lower load factors, indicating that those positions are more critical for overall stability. The load factor at which the curves flatten represents the onset of significant plastic deformation and redistribution of forces through alternative load paths. Overall, the variations in the curves reflect the influence of column position on structural response. The results demonstrate that the Pinned Joint Braced Structure remains stable beyond column removal, supporting the conclusion that it possesses sufficient redundancy to prevent progressive collapse.

The analysis demonstrates that the structure maintains global stability irrespective of the column loss position, confirming its robustness under accidental scenarios. In all cases, the load factor (λ) exceeds unity, indicating that the structure can withstand the complete removal of a column without immediate failure. While the structure remains stable, the level of deformation and the formation of plastic hinges vary depending on which column is removed. Some column loss positions result in higher displacements and earlier hinge formation, making them more critical in terms of stability. After plastic hinge formation, the structure activates catenary action, allowing tensile forces in beams to assist in load transfer. The secondary frame enhances load redistribution, particularly in cases where column loss occurs near the center of the building.

4. Summary

This study confirms that multi-story steel frames can effectively redistribute loads under different column loss scenarios, ensuring progressive collapse resistance. The findings highlight that while column location influences the structural response, the presence of moment-resistant connections, secondary beams, and catenary action significantly enhances robustness. These insights provide valuable guidance for engineers in designing more resilient steel structures against exceptional events.

The study highlights the critical role of catenary action in ensuring structural stability, particularly in systems with limited moment-resisting capacity. Strengthening beam-to-column connections and incorporating lateral restraints can significantly enhance load redistribution efficiency. However, practical constraints such as the availability of high-ductility steel (e.g., S355) and the quality control of welding and joint fabrication can affect structural performance under extreme conditions. Additionally, implementing alternative load paths, catenary action, and redundancy strategies increases both material and labor costs. To address these challenges, proposed solutions include the use of pre-qualified ductile connections, improved welding procedures, and stricter fabrication tolerances to ensure consistent structural behavior under unexpected loads. Optimized design approaches, such as partial-strength connections, strategically placed reinforcements, and cost-effective retrofitting techniques for existing structures, are recommended.

The study employs simplified assumptions (e.g., neglecting slab contributions and utilizing artificial horizontal bracing in CEPAO) that may not fully capture real-world structural behavior. Future research should incorporate composite slab action, realistic connection behaviors, and dynamic load effects in structural analyses to bridge the gap between theoretical frameworks and practical implementation.

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