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Evaluation of Moment Resisting Steel Frame Structure Resistance to Progressive Collapse Using Quantitative Algorithms and Force-Based Approaches

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ABSTRACT

This paper proposes a new quantitative algorithm to evaluate the structural robustness of moment-resisting steel frame structures against progressive collapse. The objective of the study is to enhance existing assessment methods by introducing two new damage criteria: the total number of remaining overstressed members and the mean change in Demand Capacity Ratio (DCR) before and after pre-selected column removal. The methodology involves using SAP2000 software for structural design and MATLAB for algorithm development. A ten-storey steel frame structure serves as the case study, analysing a total of 180 column removal scenarios. Key findings indicate that the proposed algorithm effectively identifies critical column locations that significantly affect the structure's collapse potential. Nonlinear dynamic implicit analysis using ABAQUS further investigates these critical scenarios, examining displacement and energy changes under various column removal rates and locations. The conclusions demonstrate that this quantitative approach provides a robust framework for evaluating the progressive collapse potential of steel frame structures, offering valuable insights for future structural design and safety assessments.

Keywords: Progressive collapse analysis; steel frame structures; alternate load path; removal time; column removal

INTRODUCTION

The Alternate Load Path (ALP) method evaluates the extent of structural failure resulting from the hypothetical loss of load-bearing elements, replicating localized damage conditions. The GSA progressive collapse guidelines (2016) and UFC4-023-03 (2024) outline several analytical techniques for implementing the ALP approach, including linear static, nonlinear static, and nonlinear dynamic analyses.

Given that progressive collapse is inherently a dynamic event, the most suitable evaluation method is nonlinear dynamic analysis. However, this approach is computationally intensive and requires a deep understanding of dynamic mechanics (Izzuddin 2022; Stephen, Lam, Forth, Ye & Tsavdaridis 2019). Researchers have identified various factors that must be taken into account to ensure accurate and reliable assessments, which induces strain rate effect on material strength (Xie, Gu, & Qian 2020; Qiao, Luo, Wei, & Chen 2020) and appropriate rate of removal on the load bearing members to initiate localised damage (Kiakojouri, Sheidaii, Biagi, & Chiaia 2020; Stephen, Lam, Forth, Ye & Tsavdaridis 2019; Liu, Davison & Tyas 2012). Furthermore, accurate representation and modelling of the steel connections is another determinant of the reliability of the assessment output (Subki, Mansor, Hamid & Parke 2022). The GSA progressive collapse guidelines (2016) and UFC4-023-03 (2024) propose the use of the Dynamic Increase Factor (DIF) in nonlinear static analysis to approximate results like those obtained from nonlinear dynamic analysis. However, research by Kiakojouri et al. (2020), Massimiliano (2019), Li, Cai et al. (2017), and Tsai and You (2012) has shown that the currently recommended DIF values fail to adequately capture the structural resistance at advanced stages of deformation during collapse.

Defining the extent of localized damage in the Alternate Load Path (ALP) method is a critical issue. The 2016 guidelines and the UFC4-023-03 standards of 2024, recommend simulating localized damage in structural frames by removing one column at a time. Similar approaches are used in EN1991-1-7 (2006), and the Approved Document A: Structure for UK Buildings (The Building Regulation, 2013). However, recent studies propose removing multiple columns simultaneously. Zhang et al. (2020) Lee et al. (2021), Elsanadedy et al. (2022), and Lee et al. (2024). According to Lee et al. (2024),) showed that removing two columns in a seven-story Ordinary Moment Frame (OMF) can cause progressive collapse, irrespective of their location. Zhang et al. (2020) suggested removing two or three columns in nonlinear dynamic analyses to better capture instability-induced collapse. However, determining the number of columns to remove is subjective, as factors like structural redundancy, connection ductility, and non-structural element contributions affect collapse risk. Elsanadedy et al. (2022), Tian et al. (2021), and Song and Sezen (2013).

The location of column removal is critical in the Alternate Load Path (ALP) method, with the GSA progressive collapse documents (2016) and UFC4-023-03 (2024) suggesting several ground floor positions. However, studies by (Fu, 2010; 2009; Tavakoli & Afrapoli, 2018). Fu (2010; 2009), show that removing columns at higher levels induces greater vertical displacements than ground-level removals Fu (2010; 2009). Tavakoli and Afrapoli study (2018) further emphasized that the progressive collapse potential depends significantly on the location of column loss, highlighting the need to analyze various possible scenarios due to the randomness of triggering events Tavakoli and Afrapoli (2018). Despite extensive research, no specific column locations have been identified as universally triggering the most severe structural damage, underscoring the importance of evaluating structural robustness comprehensively. To date, there are still no actual incidents that can be justified as to which of the column locations will trigger the most damaging effect on a structure. Summary of the column removal selection and common research methods are tabulated in Table 1.

TABLE 1. Summary of previous research on assessment of steel frame structures against progressive collapse.

Authors	Methods	Findings	Limitations
Fu (2010, 2009)	Investigated column location for removal in collapse analysis.	Removing columns at higher levels results in larger vertical displacements.	No consensus on which column removal location causes the most damage.
Liu et al. (2010a, 2010b); Gerasimidis & Baniotopoulos (2011); Khandelwal et al. (2008, 2012)	Studied column removal strategies in ALP analysis.	Highlighted the importance of specific column removal locations for assessing collapse.	Insufficient focus on factors such as continuity and redundancy in structure.
Tasi and You (2012)	Investigated the Dynamic Increase Factor (DIF) for nonlinear static analysis.	DIF underestimates resistance in large deformations, leading to conservative results.	Current DIF recommendations are inadequate at large-deformation states.
Song and Sezen (2013)	Studied column removal strategies in ALP analysis.	Removal of columns does not always lead to collapse, depending on structural factors.	Case studies highlight variability in outcomes based on specific structure configurations.
Tavakoli and Afrapoli (2018)	Investigated column location for removal in collapse analysis.	Removing columns at higher levels induced larger displacements compared to lower-level removals. Progressive collapse depends on the column location and the randomness of triggering events.	No consensus and specified method on which column removal location causes the most damage.
Li, Cai et al. (2017) & Kiakojouri et al. (2020)	Investigated the Dynamic Increase Factor (DIF) for nonlinear static analysis.	DIF underestimates resistance in large deformations, leading to conservative results.	Current DIF recommendations are inadequate at large-deformation states.
Zhang et al. (2020) & Lee et al. (2021, 2024)	Studied column removal strategies in ALP analysis.	Removing two or more columns provides a more variety of collapse scenario.	Determining the appropriate number of columns to remove is subjective and case dependent.

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Dimopoulos (2024)	Studied damping effects in progressive collapse analysis.	Damping effects are not crucial for elastic structures in progressive collapse analysis.	Elastic structure-specific; does not consider non-elastic behaviours.
Elsanadedy et al. (2022) & Ozgan et al. (2022)	Evaluated linear and nonlinear static analysis procedures.	Linear static analysis often produces conservative estimates due to m-factor inaccuracies.	Simplified methods fail to capture structural nonlinearity adequately.
Xie et al. (2020)	Examined analysis procedures for the ALP method.	Strain rate effects and member removal rates crucial for accurate results.	Damping effects are not sufficiently accounted for in elastic structures.

In EN 1991-1-7 (2006), it is assumed that the removal of a column results in the collapse of all beams it supports, subsequently leading to the failure of all slabs reliant on those beams. The failure criteria are determined based on the assumed collapsed floor area, which is limited to either 15% of the total floor area of the affected storey or 100 m², whichever is smaller. However, this approach has a significant limitation: it does not account for the nonlinear structural capacity to resist collapse. Instead, it evaluates the probability of progressive collapse by comparing the assumed collapsed area with the permissible collapse threshold

This work implements the concept from EN 1991-1-7 (2006) of sequentially removing all supporting members through linear static analysis. It evaluates each removal's impact on structure robustness using the Demand Capacity Ratio (DCR) and proposes new damage assessment criteria. These quantitative criteria help identify critical column locations within a specific structure, facilitating subsequent detailed analyses such as dynamic assessments impractical for removing every individual column.

PROPOSED QUANTITATIVE ALGORITHM

The quantitative methodology presented in this study, as illustrated in Figure 1, follows a structured approach consisting of four main phases. Initially, the structural model was developed and analysed in SAP2000, adhering to standard design principles that incorporate appropriate dimensions, material properties, section sizes, boundary conditions, and applied loads (Figure 1).

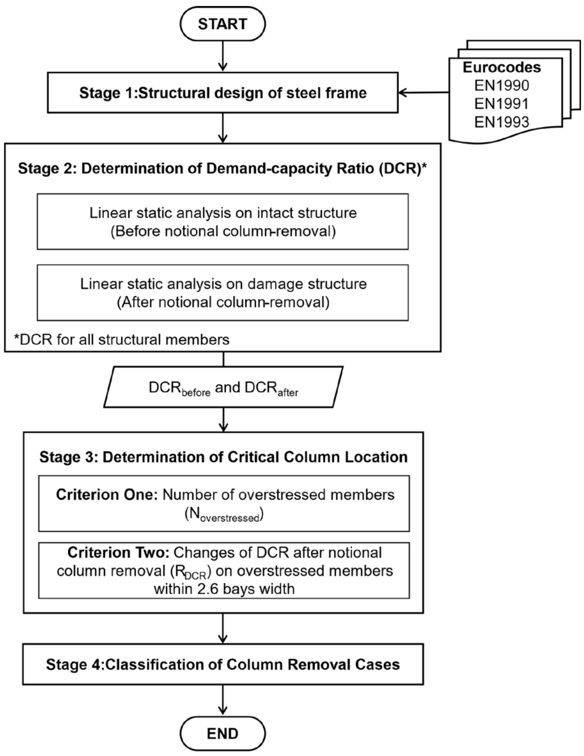


FIGURE 1. Proposed quantitative algorithm.

Following the structural design, a systematic process was implemented to identify and remove specific structural components, including columns and transfer beams supporting columns, in accordance with UK and Eurocode standards. To enhance computational efficiency and streamline the analysis, an automated technique was introduced by integrating SAP2000 with MATLAB through the Open Application Programming Interface (OAPI), significantly reducing processing time.

Next, the identification of critical column locations was carried out by analysing the extent of overstressed members and computing the mean variation in the Demand-Capacity Ratio (DCR) before and after column removal. Finally, based on the established performance criteria, the classified critical column locations were systematically evaluated to assess their impact on structural stability.

DEMAND CAPACITY RATIO (DCR)

A strength based DCR for structural member is proposed to quantify the vulnerability of the structure. This is to align with the limit of the interaction equation given in Eurocode 3: BS EN 1993-1-1 (2005). The related clauses according to strength resistance to calculate DCR are available in previous report (Mansor 2014).

DETERMINATION OF THE TOTAL COUNT OF OVERSTRESSED MEMBERS BASED ON DEMAND-TO-CAPACITY RATIOS (DCR)

The impact severity of column removal on a structure is assessed by quantifying the total number of overstressed structural members. The likelihood of progressive collapse is directly correlated with the number of failed or overstressed members, where a higher count indicates increased vulnerability, while a lower count suggests a reduced risk of collapse. Column removal scenarios are ranked according to the number of overstressed members and classified into three categories: low, medium, and high. The thresholds for these classifications can be established using a straightforward mathematical formula:

$$Range = \frac{N_{max,overstressed} - N_{min,overstressed}}{3} \tag{1}$$

ASSESSMENT OF THE VARIATION IN DEMAND-TO-CAPACITY RATIOS (DCR) OF OVERSTRESSED ADJACENT MEMBERS PRE- AND POST-COLUMN REMOVAL

The second measure for assessing the likelihood of collapse focuses on examining the fluctuations in strength-to-demand ratios of overloaded structural elements before and after eliminating a specific column. Components adjacent to the removed column are scrutinized, as they are anticipated to experience the most pronounced shifts in these ratios. This change is quantified by computing the proportion of values prior to and following column removal, utilizing the designated formula :

$$R_{DCR} = \frac{DCR_{after}}{DCR_{before}} \tag{2}$$

To identify surrounding members to consider after a column removal, the concept of 'vicinity' is defined by members whose centroids fall within a sphere of radius R, centred at the removed column's centroid. The radius is derived from Clause 5.1 of the Approved document A (2013), which limits the collapsed area to 15% of the floor area or 100m², whichever is smaller (see Figure 2). Practically, this corresponds to one bay width horizontally and one- story height vertically above and below the removed column. However, Lu et al. (2011), noted this limit is impractical for internal column removal in steel frame buildings with large column spacing, such as 6m×6m grids. In this study, the collapse area limit was extended to a 2.6 bay width radius (see Figure 3), accounting for beams supported by the removed column and adjacent bays, which are also affected to varying degrees. Members within this 2.6 bay width radius were evaluated based on the total number of overstressed members and the mean change in DCR values before and after column removal. This serves as the figure of merit to identify key elements and assess structural vulnerability.

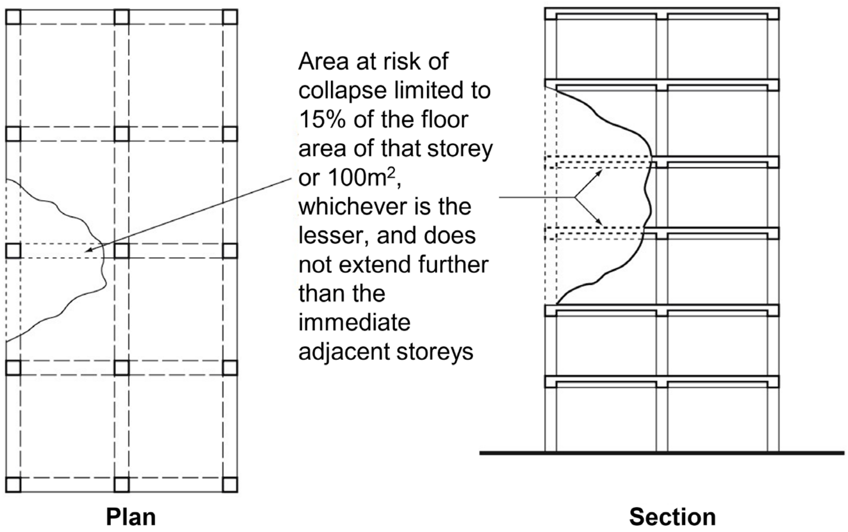


FIGURE 2. Area at risk of collapse proposed by UK progressive collapse guideline (UK Approved Document A, 2013).

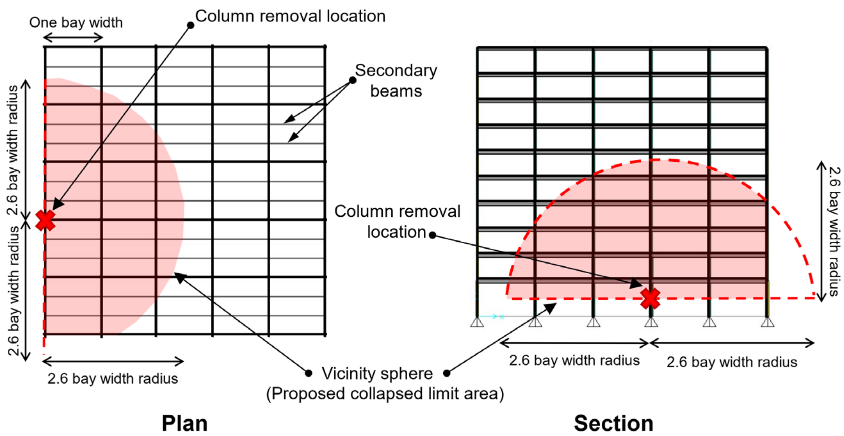


FIGURE 3. Extended area for the collapsed limit proposed in this study.

In this approach, the bay width is defined as the maximum distance between two side elevations. The average variation in Demand to Capacity ratio ($\bar{R}_{DCR}$) before and after column removal, calculated for all overstressed elements within the defined region, is selected as the key parameter for evaluating each column removal scenario. This can be expressed as:

$$\bar{R}_{DCR} = \sum_{i=1}^{n} \frac{R_{DCR}}{n} \tag{3}$$

In this context, n denotes the total count of structurally impacted members within the specified radius after the key column is eliminated. Additionally, the peak variation in the strength-to-demand ratio among the affected elements in this zone is computed. This approach also necessitates categorizing the average alteration in these ratios across all column removal cases into three levels: low, moderate, and high. The range of each class can simply be defined through the formula below:

$$\text{Range} = \frac{\bar{R}_{DCR,max} - \bar{R}_{DCR,min}}{3} \tag{4}$$

The greatest mean variation in the strength-to-demand ratio signifies the most substantial overall shift in the structural performance of elements within the defined region. This reflects the most pronounced reduction in their load-bearing capacity, resulting from the destabilization of the entire framework.

CATEGORISATION OF COLUMN ELIMINATION SCENARIOS USING THE TWO ESTABLISHED CRITERIA

The classification of column elimination scenarios, which identifies the most critical column locations, was conducted based on the robustness categories established by both evaluation criteria. Consequently, a total of nine distinct groups can be formed through the combination of these two metrics. Within this framework, the most severe column removal scenario is expected to correspond to the highest classifications in both the *'total count of overstressed members'* and the *'variation in DCR values'* categories. Conversely, the least significant cases will fall into the lowest classifications of these groups. By systematically assigning all column removal scenarios to their respective categories, detailed nonlinear dynamic analysis can be performed exclusively on the most critical cases for further validation.

IMPLEMENTATION OF THE PROPOSED ALGORITHM

DESIGN OF PROPOSED STRUCTURE

A ten-story steel moment-resisting frame, designed for a commercial office building, was selected to illustrate the proposed methodology (Figure 4). The structure consists of 4.0 m floor heights, except for the ground-to-first floor, which measures 5.5 m. Beam-to-column connections are assumed to be fully rigid and aligned at the centroid. The frame was modelled using SAP2000, incorporating isotropic material properties, including a Young's modulus of E = 210GPa, a Poisson's ratio of 0.3, a minimum yield strength of 355MPa, and an ultimate tensile strength of 482MPa. All column bases were represented with fixed boundary conditions. Detailed member dimensions are provided in Table 2 and Table 3.

To account for floor stiffness, cross- bracing diaphragms (UKB203x133x25) were modelled at each storey. The frame was analysed under two load combinations, with Load Combination One (Table 4) identified as the critical case for determining optimum section sizes. The characteristic actions (unfactored loads) are summarised in Table 4.

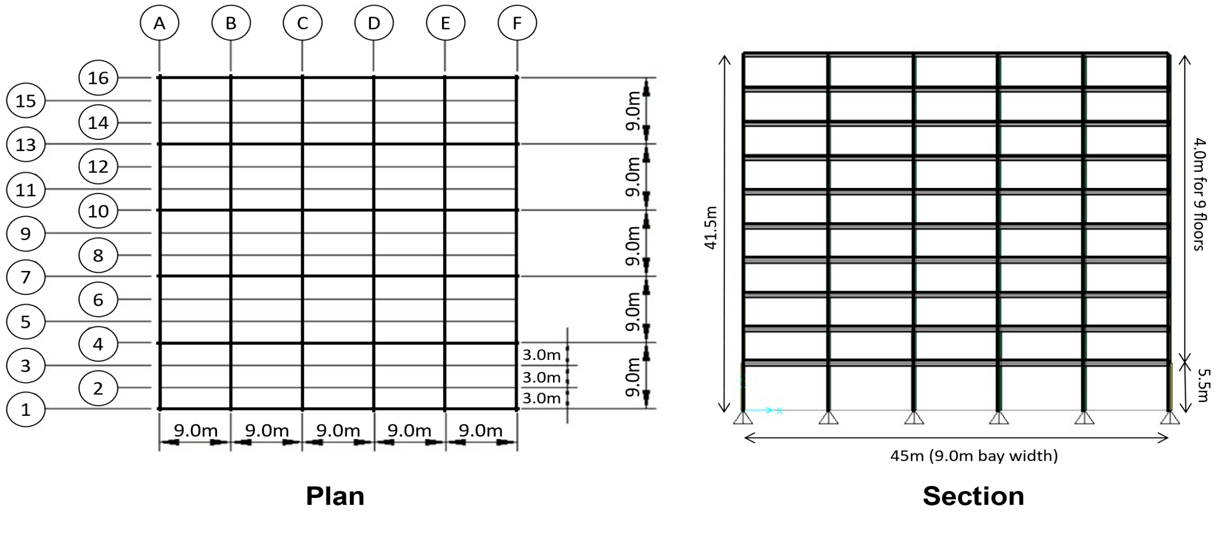


FIGURE 4. Proposed moment-resisting steel frame.

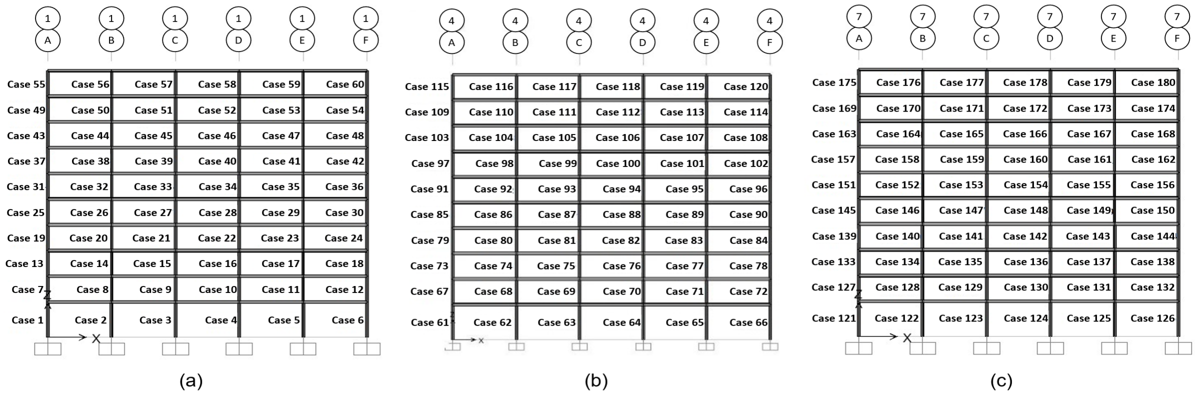


FIGURE 5. Column removal cases: (a) Grid 1, (b) Grid 4, and (c) Grid 7.

TABLE 2. Column section sizes for the proposed structure.

Level	Grid A1/Grid F1	Grid A4/Grid F2	Grid A7/Grid F7	Grid A10/Grid F10	Grid A13/Grid F13	Grid A16/Grid F16
L 0-1	UKC356x368x153	UKC356x406x235	UKC356x406x235	UKC356x406x235	UKC356x406x235	UKC356x368x153
L 1-2	UKC356x368x129	UKC356x368x177	UKC356x368x177	UKC356x368x177	UKC356x368x177	UKC356x368x129
L 2-3	UKC356x368x129	UKC356x368x153	UKC35 x368x153	UKC35 x368x153	UKC35 x368x153	UKC356x368x129
L 3-4	UKC356x368x129	UKC356x368x153	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129
L 4-10	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129
Level	Grid B1/Grid E1	Grid B4/ Grid E4	Grid B7/Grid E7	Grid B10/Grid E10	Grid B13/Grid E13	Grid B16/Grid E16
L 0-1	UKC356x406x340	UKC356 x406x551	UKC356 x406x551	UKC356 x406x551	UKC356x406x551	UKC356x406x340
L 1-2	UKC356x406x235	UKC356x406x393	UKC356x406x393	UKC356x406x393	UKC356x406x393	UKC356x406x235
L 2-3	UKC356x368x202	UKC356 406x340	UKC356 406x340	UKC356 406x340	UKC356 406x340	UKC356x368x202
L 3-6	UKC356x368x177	UKC356x40x 340	UKC356x40x 340	UKC356x40x 340	UKC356x40x 340	UKC356x368x177
L 4-6	UKC356x368x177	UKC356x406x287	UKC356x406x287	UKC356x406x287	UKC356x406x287	UKC356x368x177
L 6-7	UKC356x368x153	UKC356x368x177	UKC356x368x177	UKC356x368x177	UKC356x368x177	UKC356x368x153
L 7-9	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129
L 9-10	UKC356x368x129	UKC356x406x551	UKC356x406x551	UKC356x406x551	UKC356x406x551	UKC356x368x129
Level	Grid C1/ Grid D1	Grid C4/Grid D4	Grid C7/Grid D7	Grid C10/ Grid D10	Grid C13/GridD13	Grid C16/Grid D16
L 0-1	UKC356x406x393	UKC356x406x551	UKC356x406x551	UKC356x406x551	UKC356x406x551	UKC356 x406x393
L 1-3	UKC356x368x202	UKC356x406x340	UKC356x406x340	UKC356x406x340	UKC356x406x340	UKC356x368x202
L 3-7	UKC356x368x177	UKC356x406x287	UKC356x406x287	UKC356x406x287	UKC356x406x287	UKC356x368x177
L 7-8	UKC356x368x153	UKC356x368x153	UKC356x368x153	UKC356x368x153	UKC356x368x153	UKC356x368x153
L 8-10	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129	UKC356x368x129

TABLE 3. Beam section sizes for the proposed structure.

Beam	Level 1-9	Roof level
Along G1-G6	UKB457x191x82	UKB406x178x67
Along G2-G15	UKB457x191x133	UKB406x178x85
Along GA-GF	UKB457x191x74	UKB457x191x67
Along GB-GE	UKB457x191x89	UKB533x210x138

TABLE 4. Summary of the assigned characteristic actions on the floor and roof level.

Member	1st - 9th floor	10th floor
Permanent Actions (G_k)		
Internal	10.8 kN/m	10.8 kN/m
External	5.4 kN/m	5.4 kN/m
Variable Actions (Q_k)		
Internal	18 kN/m	1.8 kN/m
External	9 kN/m	0.9 kN/m
Load Combination One = 1.25 G_k + 1.5 Q_k + 0.75Wind		
Load Combination Two = 1.25 G_k + 1.05 Q_k + 1.05Wind		

MEMBER REMOVAL PROCESS

A total of 180 distinct column locations were designated for sequential removal, resulting in 180 individual scenarios for analysis. The subsequent step involved systematically eliminating each identified column. This removal process was executed using the '*auto-operation*' technique, developed by Mansor (2014). Columns were automatically removed one at a time, following a predefined sequence. For Grid 1, the sequence commenced with column Case 1 (refer to Figure 5), followed by Cases 11, 21, 31, 41, 51, 2, 12, 22, and continued up to Case 60. A similar removal order was applied to Grid 4 (Figure 5(b)) and Grid 7 (Figure 5(c)). Prior to initiating the elimination process, the designer must specify the desired output parameters, such as joint response data and steel frame design details, ensuring that results are generated automatically.

IDENTIFICATION OF THE MOST VULNERABLE COLUMN POSITION

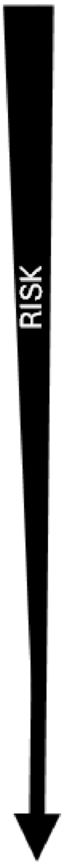
The possible critical column locations are determined, and the analysis results generated are sorted accordingly based on the two proposed criteria described earlier and presented in Table 5.

CATEGORISATION OF COLUMN REMOVAL SCENARIOS

Table 5 outlines the grouping of 180 column elimination scenarios for the moment-resisting frame, determined using the two established assessment criteria. Although a large volume of results was produced, only the most relevant cases have been selected for discussion and analysis. To ensure clarity and conciseness, one representative case from each category was chosen for in-depth evaluation. For this structural investigation, the column removal cases examined include Case 125, 65, 2, 81, 56, 167, and 6. The assessment involves identifying the location of critically stressed members within a 2.6-bay-width region and analysing their strength-to-demand ratios (DCR) before and after the column's removal. These outcomes are illustrated in Figure 6 to Figure 12.

Figure 6 illustrates the distribution of highly stressed elements within the 2.6-bay-width zone for column removal Case 125. Within this designated region, 68 structural members exhibited excessive stress. To represent the changes in DCR values before and after column removal, Figure 6(b) presents a bar chart visualization. The findings indicate that 15 of the 68 overstressed members (approximately 22.1%) displayed a DCR greater than 2 following the column removal.

TABLE 5. Classification of the 180 column removal cases.



Total no of overstressed member	Average change in DCR_{after} and DCR_{before}	Case Number
High	High	No Cases were Identified
	Medium	125
	Low	65, 62, 71, 63, 68, 64, 77, 122, 74, 128, 131, 69, 123, 124, 70, 83, 80, 134, 137, 75, 76, 89, 129, 130, 135, 140, 143, and 86
Medium	High	There are no cases found in this category
	Medium	2, 26, 107, and 32
	Low	81, 136, 146, 82, 149, 5, 95, 92, 155, 141, 11, 152, 142, 8, 87, 17, 88, 3, 4, 66, 101, 14, 98, 147, 72, 126, 148, 9, 23, 61, 161, 10, 78, 67, 93, 121, 132, 158, 20, 15, 16, 94, 127, 73, 29, 138, 84, 21, 133, 153, 22, 144, 154, 79, 27, 35, 28, 104, 90, 99, 100, 150 and 139
Low	High	56, 59, 55, 57, 58, 60, 115, 120, 175, and 180
	Medium	164, 38, 113, 44, 47, 110, 167, 145, 91, 151, 102, 173, 157, 170, 37, 97, 50, 51, 53, 168, 52, 108, 103, 163, 49, 54, 174, 114, 116, 119, 176, 179, 109, and 169.
	Low	6, 12, 33, 34, 156, 159, 160, 7, 18, 85, 24, 13, 96, 30, 105, 106, Case 19, 39, 40, 165, 166, 36, 162, 25, 31, 111, 112, 172, 171, 117, 118, Case 45, 46, 42, 48, 43, 177 and 178

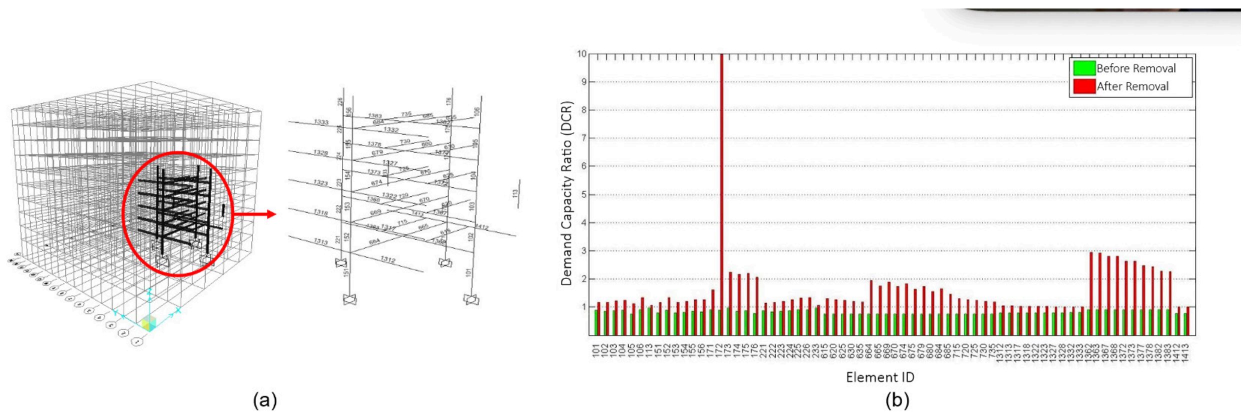
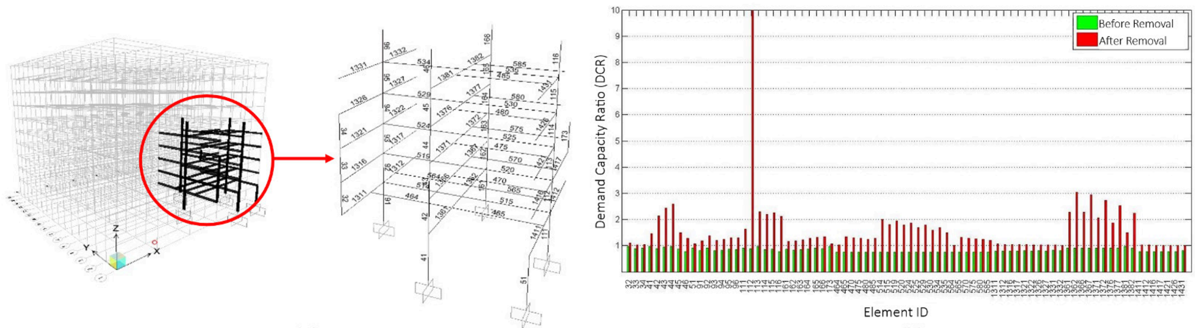
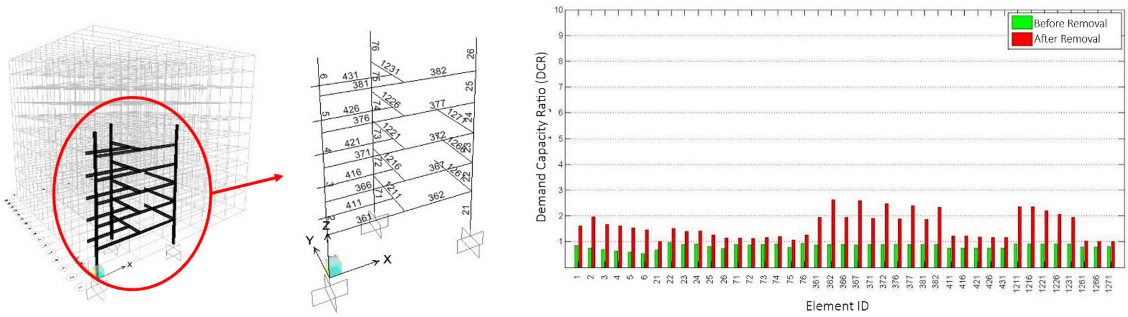


FIGURE 6. Classification of column removal Case 125: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.



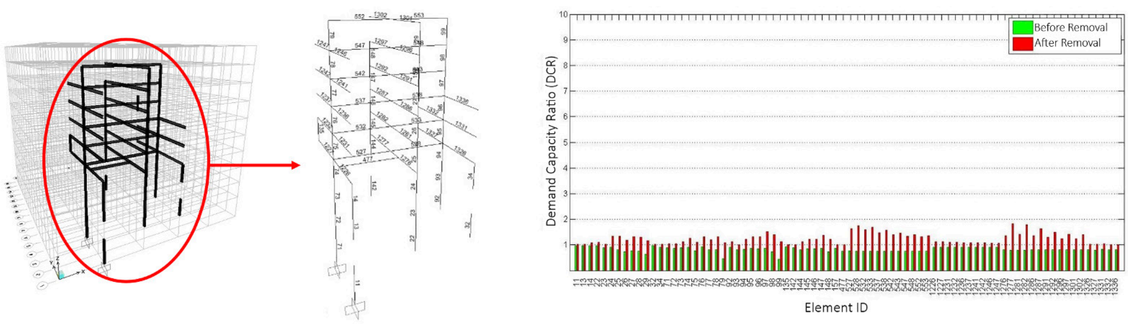
(a) (b)

FIGURE 7. Classification of column removal Case 65: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.



(a) (b)

FIGURE 8. Classification of column removal Case 2: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.



(a) (b)

FIGURE 9. Classification of column removal Case 81: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.

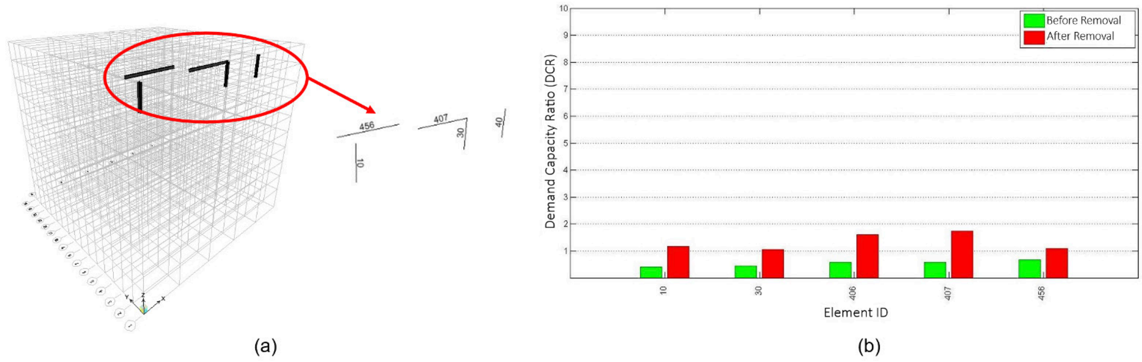


FIGURE 10. Classification of column removal Case 56: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.

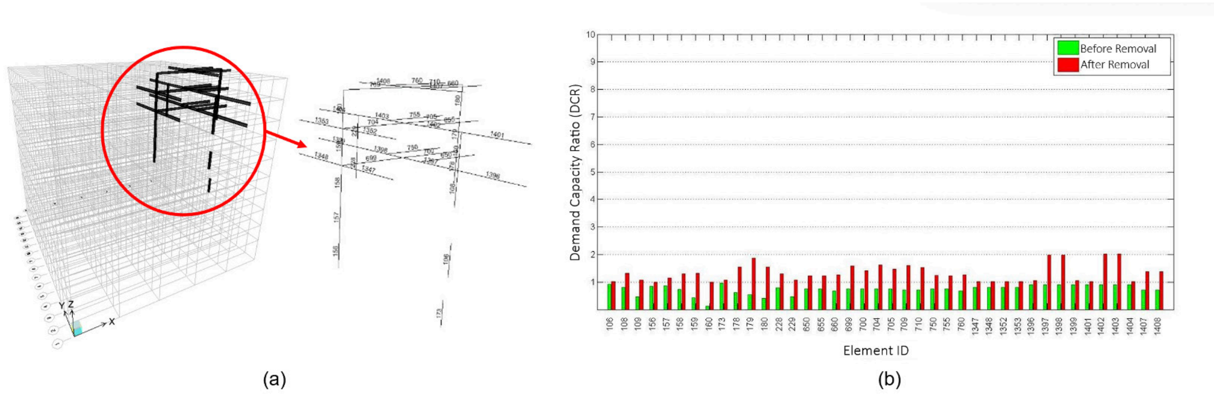


FIGURE 11. Classification of column removal Case 167: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.

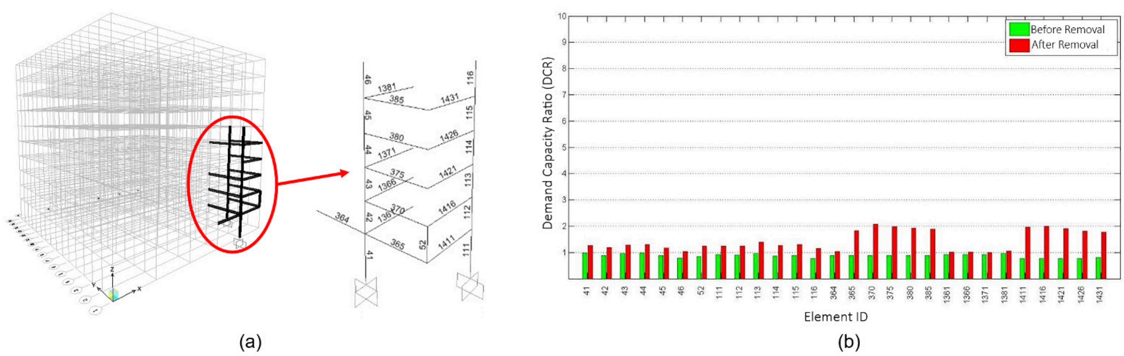


FIGURE 12. Classification of column removal Case 6: (a) Position of critically loaded elements within a 2.6-bay-width range, and (b) Strength-to-demand ratio (DCR) of affected members prior and following column elimination.

TABLE 6. Summary of analysis results of the selected key cases.

Column removal cases	Total overstressed members within 2.6 bays-width radius	Total members with DCR>2	Percentage of members with DCR>2
Case 125	68	15	22.1%
Case 65	78	17	21.8%
Case 2	41	9	21.9%
Case 81	78	-	-
Case 56	5	-	-
Case 167	40	-	-
Case 6	28	-	-

Similarly, Figure 7(a) and Figure 7(b) depict the positioning and demand-to-capacity ratios (DCR) of highly stressed members prior to and following column elimination for Case 65. This case exhibits 78 overstressed elements, which is slightly greater than the 68 identified in Case 125. However, only 21.8% of these members in Case 65 had DCR values exceeding 2 (17 out of 78), while none of the overstressed members in Cases 2 (Figure 8) and 81 (Figure 9) exhibited DCR values over 2. Case 56 (Figure 10) had just 5 overstressed members, with no DCR exceeding 2, consistent with its lower-risk classification (Refer to Table 5). Case 167 (Figure 11) showed 40 overstressed members with no significant anomalies. Finally, Case 6 (Figure 12) demonstrated the lowest risk, with no overstressed members exceeding a DCR value of 2. A summary of these key cases is provided in Table 6.

IDENTIFICATION OF COLUMN REMOVAL SCENARIOS FOR DETAILED EXAMINATION IN ABAQUS FINITE ELEMENT SOFTWARE

Two specific scenarios, Case 125 and Case 167, were chosen for advanced nonlinear dynamic analysis using ABAQUS software to assess the effectiveness of the proposed algorithm. Case 125 was presumed to represent the most structurally vulnerable column removal scenario. In contrast, Case 167 was selected to explore the impact of a reduced number of overstressed members, as it falls within the lowest-risk classification. It was anticipated that the sudden removal of this column would result in a less critical structural response compared to Case 125. Additionally, Case 167 was positioned at level 8, whereas Case 125 was located at the ground level. Although the primary reason for selecting Case 167 was to examine the consequences of a lower count of overstressed members, it also provides an opportunity to observe the potential collapse behaviour of upper-story damage in comparison to Case 125, where the failure occurs at the base of the structure.

COLUMN REMOVAL ANALYSIS

FINITE ELEMENT MODEL

The finite element representation of the ten-story steel moment-resisting frame was developed in ABAQUS. The primary beams were rigidly attached to the columns, while the secondary beams were similarly connected to the primary beams. Horizontal x-diaphragms were linked to the columns using hinge connectors, permitting only axial force transfer. Fixed supports were assigned at the base of all ground-floor columns. All structural components were modelled using the beam element type, with a dependent meshing approach and an approximate global mesh size of 0.1 m. The material properties of all structural members followed an elastic-plastic behaviour, characterised by a Young's modulus E = 210GPa and a Poisson's ratio of 0.3. Details of plastic work hardening parameters are provided in Table 7.

TABLE 7. Stress-strain data for steel isotropic plastic behaviour

Stress (MPa)	Strain (%)
0	0.00
355	0.17
357	1.19
355	2.00
389	3.33
453	7.33
482	14.00
479	17.00

TABLE 8. Rate of removal for each column removal case.

Rate of removal	Upward forces, P (kN)	Time, T (s)
	Case 125	
	0.00	0.00
0.1 T	11256.00	1.00
	0.00	1.133

continue ...

... cont.

	0.00	0.00
0.01 T	11256.00	1.00
	0.00	1.013
	0.00	0.00
0.001T	11256.00	1.00
	0.00	1.001
	Case 2	
	0.00	0.00
0.1 T	5379.99	1.00
	0.00	1.133
	0.00	0.00
0.01 T	5379.99	1.00
	0.00	1.013
	0.00	0.00
0.001T	5379.99	1.00
	0.00	1.001
	Case 167	
	0.00	0.00
0.1 T	2840.76	1.00
	0.00	1.13315
	0.00	0.00
0.01 T	2840.76	1.00
	0.00	1.013315
	0.00	0.00
0.001T	2840.76	1.00
	0.00	1.0013315

LOADING AND SOLUTION CONTROL

The assessment of progressive collapse commenced with identifying the natural period (T_{assoc}) corresponding to the structural response mode. The structure was first analysed under static loads without removing columns, followed by the deletion of pre-selected columns and the application of upward and downward axial forces based on specific cases. These forces were ramped down to zero using amplitude curves, activating the dynamic response by removing forces at the nodes above the removed columns within durations of $0.1T_{assoc}$, $0.01T_{assoc}$, and $0.001T_{assoc}$. The process faced challenges, such as structural instability and numerical convergence issues, requiring very small time increments to ensure accurate results. The ramp amplitude data for Case 125, 2 and 167 are tabulated in Table 8.

COLUMN REMOVAL ANALYSIS RESULTS

EFFECT OF COLUMN REMOVAL LOCATIONS

The simulation outcomes indicated that the moment-resisting steel frame building is likely more susceptible to progressive collapse when the column in Case 125 is suddenly removed. This is evident in Figure 13, where, for all column removal durations in Case 125, the structural response failed to reach an equilibrium state. For example, when analysing the time-displacement response at the node directly above the removed column (See Figure 13), the results terminated at the maximum displacement point. In the case of a removal time of $0.1T_{assoc}$ seconds, only an initial vibration was observed (refer to Figure 13). However, for shorter removal times of $0.01T_{assoc}$ and $0.001T_{assoc}$, there was no indication from the numerical results that the structure would continue oscillating. The analysis of energy release demonstrates a discrepancy between the external work applied and the internal energy developed within the structure at $0.001T_{assoc}$ seconds in Case 125 (Figure 14). This difference stems from both vertical and horizontal displacements resulting from gravitational acceleration acting on the structural mass after the abrupt column removal. A higher level of external work leads to increased deformation.

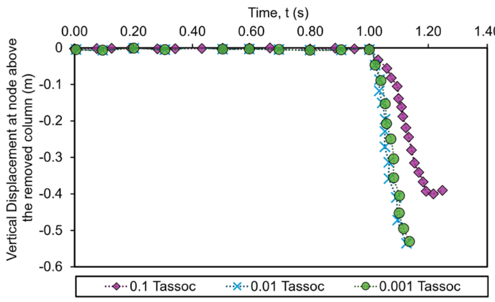


FIGURE 13. Vertical displacement response of the node in Case 125 under different column removal intervals.

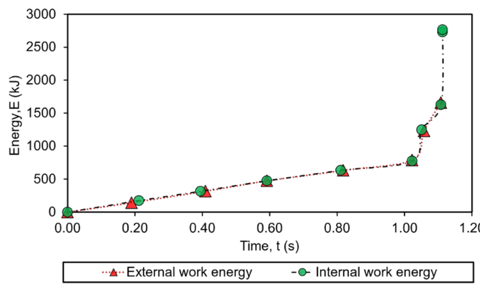


FIGURE 14. Comparison of external work and internal energy dynamics of the entire structure in case 125 at $0.001T_{assoc}$ seconds.

The imbalance led to localized failure at a perimeter column (Figure 15). The column's axial compression capacity was $N_{c,Rd}$ = 6730kN. Before the column removal, it carried an axial load of N = 3525 kN, which increased to N=7948 kN after the removal. This 18% overload, caused by load redistribution from primary and secondary beams, exceeded the column's capacity, resulting in buckling failure.

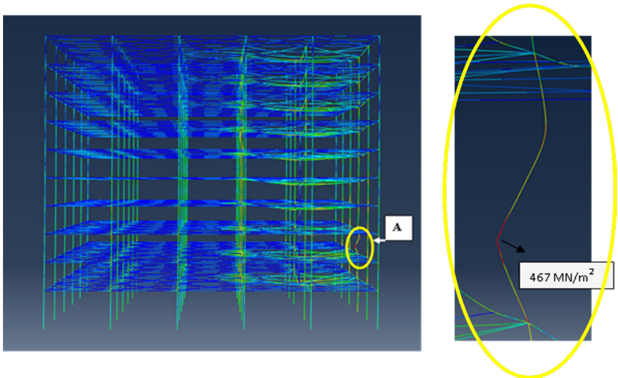


FIGURE 15. Localised failure at the perimeter column.

Unlike Case 125, the structural analysis for Case 167 reached a stable equilibrium across various removal durations, as illustrated in Figure 16. The energy response history (Figure 17) confirms the stability of the structure in Case 167. Following the column removal, the energy flow distribution was balanced, with external work equalling internal work and kinetic energy, indicating stability. This equilibrium is due to the reduced axial forces in upper-level columns compared to those at lower levels, along with the greater resistance of adjacent members, likely resulting from the larger section sizes utilized for the top columns.

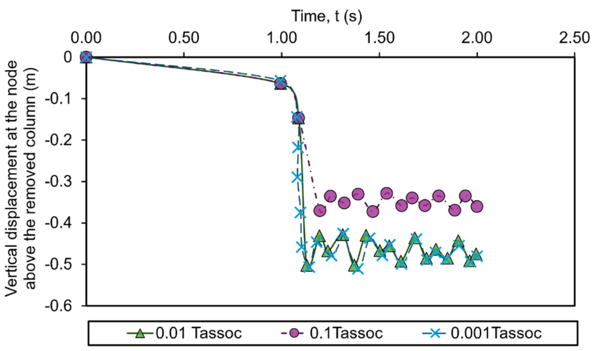


FIGURE 16. Vertical displacement response of the node position above the eliminated column in Case 167 across various scenarios.

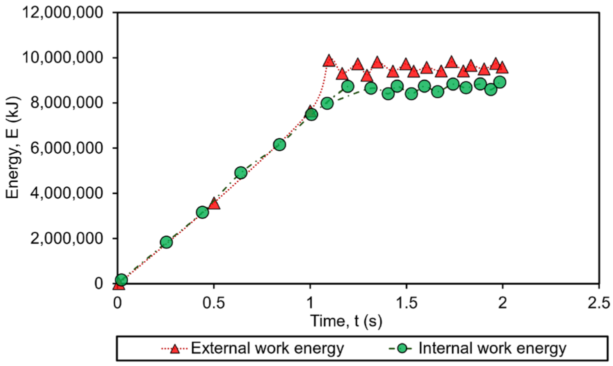


FIGURE 17. Comparison of External and Internal Works energy for Case 167

Contrastingly to the previous findings by Fu (2010; 2009) and Tavakoli (2018) this study did not show that removing an upper-level column would lead to a severe structural response. Detailed analyses of Case 167 under various column removal times demonstrated that the structure was successful. redistributed the load and achieved equilibrium after the removal of a high-level column. Nevertheless, it is essential to recognize that the reaction to the removal of an upper-level column is contingent upon the structural configuration. A quantifiable and justifiable method is essential for identifying the most vulnerable columns in a structure.

RESULTS AND ANALYSIS DISCUSSION

The study shows that the rate of column removal significantly impacts structural response, with pronounced changes in displacement and energy when removal time decreases from $0.1T_{assoc}$ to $0.01T_{assoc}$, but minimal differences between $0.01T_{assoc}$ and $0.001T_{assoc}$. For instance, in Case 167, internal energy increased by approximately 0.9×10^6 Joules between removal times of $0.1T_{assoc}$ and $0.01T_{assoc}$, with no further change at $0.001T_{assoc}$. Similarly, in Case 125, the difference in internal energy between $0.1T_{assoc}$ and $0.01T_{assoc}$ was about 3.2×10^7 Joules. These findings validate the GSA guideline's suggestion that column removal should not occur faster than 1/10 of the natural period. They also emphasize the structural response's sensitivity to removal duration, underscoring the necessity of evaluating multiple time intervals to accurately assess collapse behaviour. (Refer to Figure 16 and Figure 17)

This research investigated column elimination at different positions, with particular emphasis on Cases 125 and 167, to evaluate the potential for progressive collapse in steel-framed structures. A new quantitative algorithm, validated through nonlinear dynamic analysis using

ABAQUS, demonstrated high accuracy in predicting structural responses, such as time-displacement and energy transfer, and identified Case 125 as the most vulnerable, enabling targeted reinforcement strategies. By analysing 180 scenarios, the algorithm pinpointed high-risk areas, and its integration with SAP2000 and MATLAB ensured computational efficiency for evaluating large, complex structures. However, the algorithm's reliance on linear static analysis may result in overly conservative estimates, as it does not fully account for material and geometrical nonlinearities or the underutilized safety margins of structural components. Addressing these limitations in future work could enhance the accuracy and reliability of progressive collapse assessments.

CONCLUSION

In conclusion, the proposed quantitative algorithm represents a significant advancement in evaluating progressive collapse risks in steel frame structures, offering an effective and efficient method for identifying vulnerable columns and assessing structural robustness. Validation through dynamic analysis, using a ten-story moment-resisting steel frame structure as a case study, confirms the algorithm's reliability. Its computational efficiency also makes it practical for widespread use. However, future research should focus on incorporating nonlinearities and refining collapse prediction thresholds to address the algorithm's conservative approach. Additionally, further testing on a variety of steel frame structures is needed to better understand their limitations and capabilities in assessing structural collapses. These improvements will be critical for enhancing the safety and resilience of structures against progressive collapse.

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DECLARATION OF COMPETING INTEREST

None.

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