https://doi.org/10.31436/iiumej.v27i3.4607

Progressive Collapse Analysis of 2D RC Frames Under the Impact of Single Interior and Exterior Columns

TAHARA RAMADZAN MD KASSIM[1*], MUHAMMAD FAIZ MOHAMAD NOR[1], TAKSIAH A. MAJID[2]

[1]*Department of Civil Engineering, Kulliyyah of Engineering, International Islamic University Malaysia, Kuala Lumpur, Malaysia*
[2]*School of Civil Engineering, Universiti Sains Malaysia, Penang, Malaysia*

**Corresponding author: tahara@iium.edu.my*

(Received: 16 July 2026; Accepted: 21 August 2026; Published online: 11 September 2026)

ABSTRACT: Structural elements are susceptible to both local and global failures triggered by progressive collapse, making it essential in structural vulnerability assessments. This research investigates and predicts the progressive collapse behavior of two-dimensional (2D) reinforced concrete frames designed for gravity loadings, considering the removal of columns in both interior and exterior locations. Linear static and nonlinear dynamic analyses are conducted to investigate and predict the development of alternate load paths (ALPs) and dynamic structural response under column-loss scenarios using SAP2000 software. Three cases are considered: Case 1 (no column removal), Case 2 (removal of one interior column), and Case 3 (removal of one exterior column). Linear static analysis reveals that the General Services Administration (GSA) yields slightly higher, more conservative axial forces than the British Standard (BS). Regardless of where the column was removed, ALP under progressive collapse significantly altered the structural response in Case 2 and Case 3. Nonlinear dynamic analysis revealed that Case 3 exhibited the highest displacement and interstorey drift ratio (IDR), particularly under Ground Motion (GM5). The findings also demonstrate that exterior column removal triggered the highest drift demands, indicating critical structural vulnerability to progressive collapse.

ABSTRAK: Elemen struktur terdedah kepada kegagalan lokal dan kegagalan menyeluruh boleh mencetuskan keruntuhan progresif, menjadikannya satu aspek penting dalam penilaian struktur. Kajian ini menyiasat dan meramalkan tingkah laku keruntuhan progresif bagi kerangka konkrit bertetulang dua dimensi (2D) yang direkabentuk untuk menampung beban graviti sahaja dengan mengambil kira penyingkiran tiang yang berada di dalam dan tiang luaran. Analisis linear statik linear dan analisis tidak linear dinamik dilakukan untuk mengkaji dan meramalkan laluan "Alternate Load Path" (ALP) serta tindak balas dinamik struktur di bawah senario kehilangan tiang tersebut menggunakan perisian SAP2000. Tiga kes telah dipertimbangkan, iaitu Kes 1 (tiada kegagalan tiang), Kes 2 (penyingkiran satu tiang dalaman) dan Kes 3 (penyingkiran satu tiang luaran). Hasil analisis statik linear menunjukkan bahawa General Services Administration (GSA) menghasilkan daya yang sedikit lebih tinggi dan lebih konservatif berbanding British Standard (BS). Tanpa mengira lokasi tiang yang disingkirkan, ALP mengakibatkan keruntuhan progresif telah mengubah tindak balas struktur dengan ketara, khususnya bagi Kes 2 dan Kes 3. Analisis tidak linear dinamik pula menunjukkan bahawa Kes 3 merekodkan anjakan dan 'interstorey drift ratio' (IDR) yang paling tinggi, terutamanya apabila dikenakan gempa bumi 'Ground Motion' (GM5). Kajian turut menunjukkan bahawa penyingkiran satu tiang luaran menghasilkan IDR yang paling tinggi, sekali gus menunjukkan bahawa keadaan tersebut merupakan senario yang paling kritikal dan menjadikan struktur lebih terdedah kepada keruntuhan progresif.

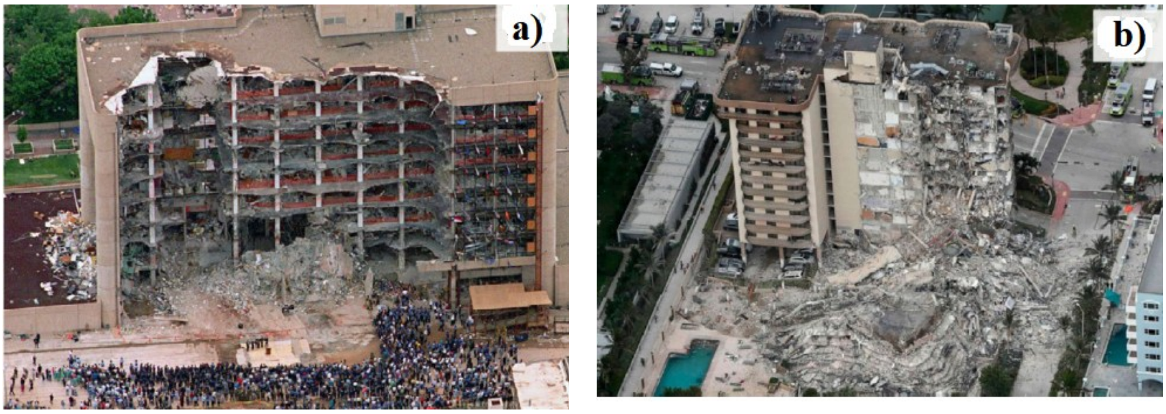
KEYWORDS: *Progressive Collapse, Column Removal, Alternate Load Path, Nonlinear Dynamic Analysis*

1. INTRODUCTION

Structures are designed to be safe and robust according to design code guidelines. However, extreme events such as earthquakes, explosions, and terrorist attacks usually cause local damage that can lead to total collapse. Initial local damage can be caused by one or more columns (load-bearing) or a portion of the bearing wall failing [1]. These localized structural failures can propagate across adjacent components, potentially triggering a total or disproportionate collapse of the entire system, which is classified as progressive collapse, as shown in Fig. 1. Several progressive collapses have been reported, such as the A. P. Murrah Federal Building (Oklahoma, 1995) and the World Trade Center (New York, 2001), which caused not only economic but also social impacts [2]. Adam et al. [3] stated that the failure of one or more walls or columns, which are vertical load-bearing elements regarded as the most devastating local damage, will set off a series of events that ultimately result in the complete collapse of the building or a significant portion of it. Feng [4] stated that the General Services Administration (GSA), the U.S. Department of Defense (DoD), and several other building codes provide guidelines for progressive collapse analysis. These codes offer detailed information and techniques, such as the alternate path method (ALP), to prevent the gradual collapse of building structures. Nevertheless, the risk of progressive collapse can be assessed using linear static (LS) and nonlinear static (NS) analysis according to the U.S. General Services Administration (GSA) and the U.S. Department of Defense. A threat-independent method can assess a structure's resistance to progressive collapse when high loading leads to the loss of several columns [5]. Fig. 2 shows the most common examples of alternate load paths that may develop during a progressive collapse event. Elkady et al. [6] stated that studying progressive collapse and mitigation has become a major research topic in structural engineering, driven by the increasing number of studies and the development of new design standards. However, comprehensive structural design should focus on the interaction among structural elements to better understand and predict structural behavior under extreme loading conditions [7].

Progressive collapse is a dynamic phenomenon and is reliably evaluated by adopting nonlinear dynamic analysis [8]. Izzuddin et al. [9] studied progressive collapse assessment using nonlinear static response with dynamic effects in multi-story buildings under sudden column-loss scenarios. Arshian et al. [10], Hefez et al. [11], and Kim and Kim [12] studied progressive collapse frames using alternate path methods according to GSA [13] and DoD [14] guidelines. Progressive collapse of steel moment-resisting frames was evaluated using linear static and nonlinear dynamic analysis procedures based on the same design codes. The results indicated that nonlinear dynamic analysis provided better structural performance predictions, while the linear static procedure produced more conservative outcomes [12]. Tsai and Lin [15] performed the progressive collapse analysis using the linear static analysis method recommended by the US General Services Administration (GSA). In progressive collapse, most studies focused on single-column failure scenarios [16-17]. However, multiple-column removal may result in heavy damage or destruction under different time frames [18]. The studies found that removing consecutive corner columns induces more severe damage. Undeniably, corner columns are considered key load-bearing members in a building structure [19-20]. Jin et al. [21] studied the collapse resistance of RC frames, focusing on corner column removal. Compared to interior or edge columns, corner columns may have a higher probability of failure due to their position, especially under vulnerable abnormal load events. Furthermore, corner column loss would be more significant because there is no column to support it. Buitrago et al. [22] stated that DoD regulations for frame structures require removing corner or exterior middle columns on the long and short sides one by one for collapse analysis.

Using 2D models focuses the analysis on a single structural plane, providing a simplified yet effective way to analyze load redistribution during collapse, as in studies by Ren et al. [23]. Progressive collapse analysis involves geometric and material nonlinearities, making detailed 3D simulations computationally demanding. As a result, 2D models are often used as a practical screening tool to quickly identify the most critical failure modes [37]. This study investigates the progressive collapse behavior of non-seismically designed reinforced concrete (RC) 2D frames under single interior and exterior column removal scenarios. These two column-loss cases are examined because they represent different in-plane load-redistribution mechanisms under both linear and nonlinear dynamic analyses. Sudden column removal is simulated to evaluate the structural vulnerability and robustness of simplified 2D RC frames. A 2D frame provides a computationally efficient approach for conservatively estimating in-plane load redistribution and catenary action, while offering insights into the progressive collapse response that can support structural assessment, especially in analysis and design.



a) Collapse of the A. P. Murrah Building [24] b) Collapse of Champlain Towers South [25]

Figure 1. Examples of progressive collapse events.

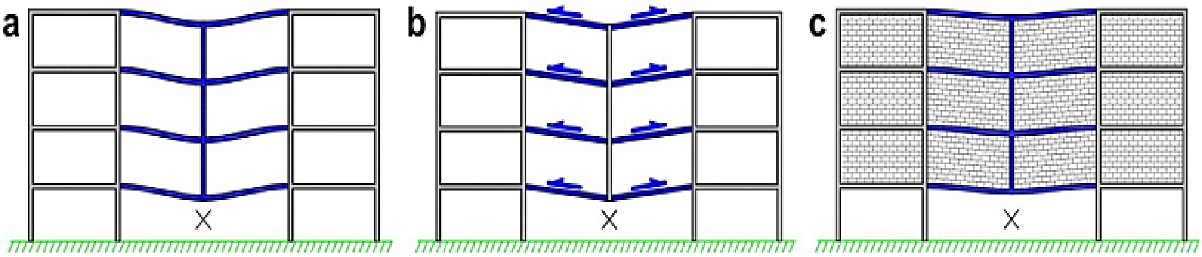


Figure 2. Alternate load paths may occur as (a) Vierendeel action; (b) catenary action; and (c) contribution of non-structural elements [3]

2. MODELING 2D RC FRAMES

This study focuses on the progressive collapse of a non-seismically designed existing building: a 4-story staff quarters at SMK Ranau in Sabah. The building has a symmetrical layout. Meng [26] provides further details on the structural reinforcements. The beam size used is 250 mm × 500 mm, taken for all levels. Each joint is assigned a diaphragm. The column size is taken as 250 mm × 250 mm. The 2D RC frames were modeled using SAP2000 software

[27]. Fig. 3 and Fig. 4 illustrate the front view and layout plan of staff quarters, respectively. Fig. 4 shows the layout of the structure (7-bay frame) selected for the current study. The study assumed the critical section for the 2D RC frame is in the internal section. The total span and height of the 7 frames are 28.06 m and 13.45 m, respectively. The structure height is 13.45 m. The concrete compressive strength, fck, is 30 N/mm². The structure is assumed to be fixed at the supports. The frames were designed to meet the specifications of British Standard BS 8110 [28], considering gravity loading using a safety factor of 1.4 for dead load (Gk) + 1.6 for live load (Qk). The floor slab thickness is assumed to be 150mm. Table 1 shows the loading details for the frames.

Figure 3. Front view of staff quarters of SMK Ranau.

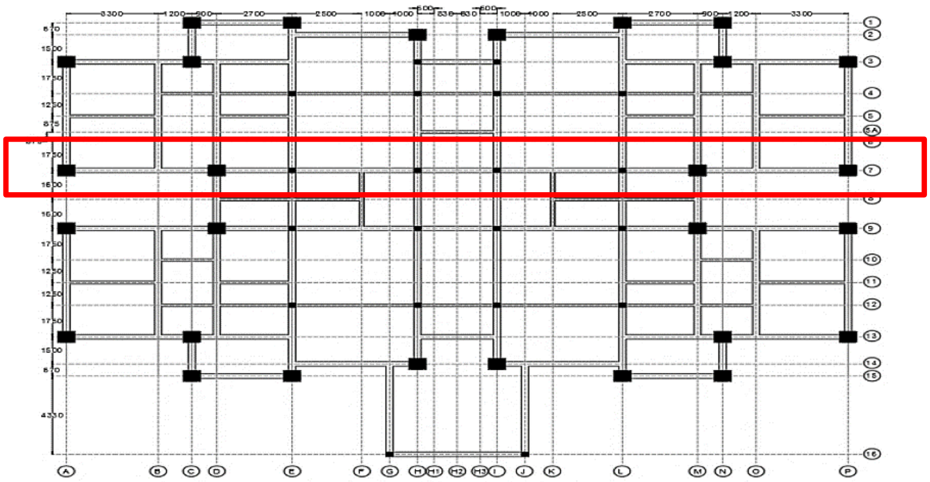


Figure 4. Layout plan of staff quarters of SMK Ranau [26].

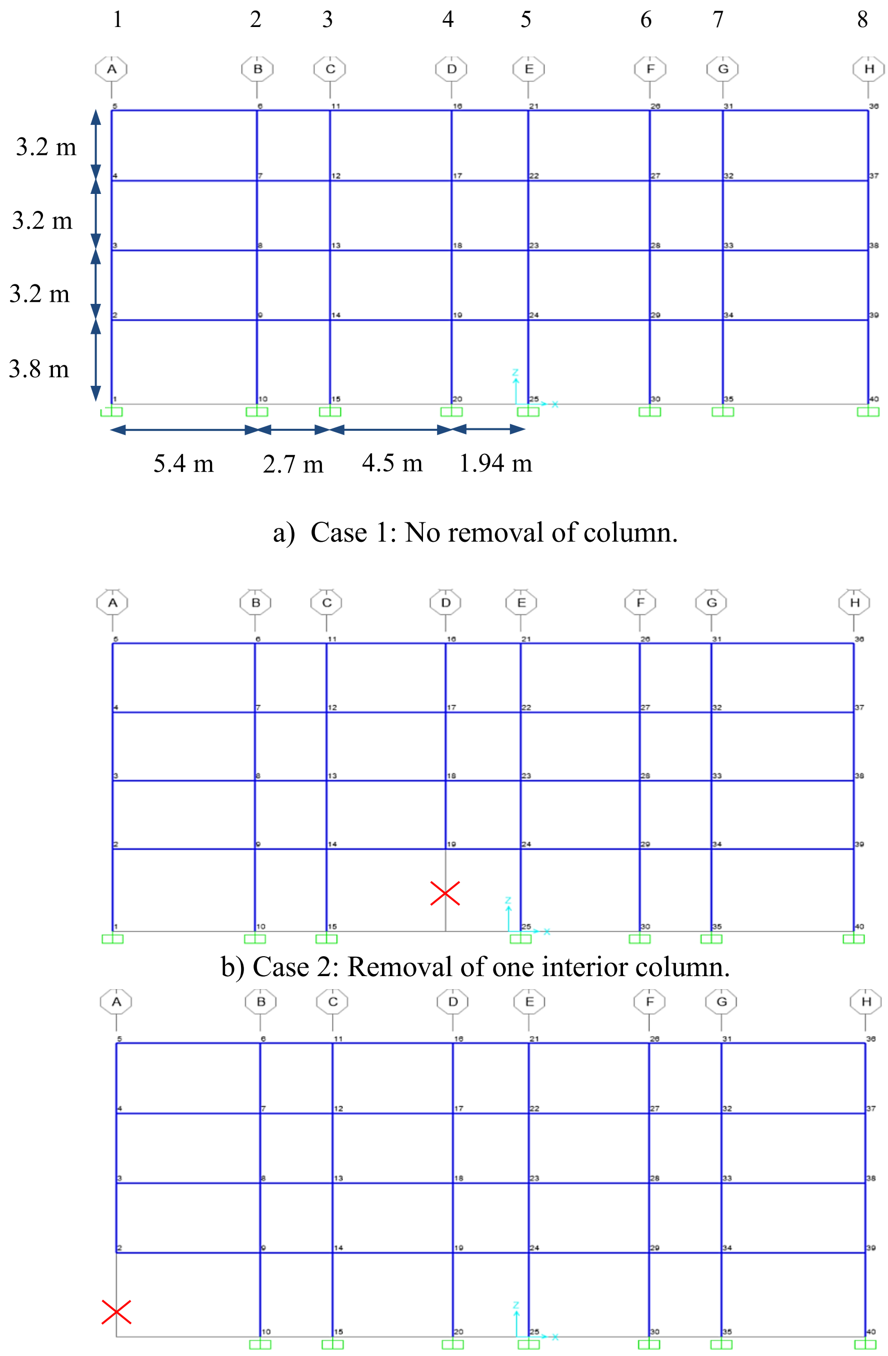
Table 1. Detailed loading information on the frames

Building Level	Dead Load (kN/m²)	Live Load (kN/m²)
1	1.65	1.5
2	1.65	1.5
3	1.65	1.5
4	15 (Water Tank)	1.5

https://doi.org/10.31436/iiumej.v27i3.4607

2.1. Progressive Collapse Analysis

In this study, both interior and exterior columns were deliberately removed to simulate and predict the building's response to progressive collapse using linear and nonlinear analyses. The focus was on removing vertical elements, specifically a single column in the interior and exterior sections of the ground floor.



a) Case 1: No removal of column.

b) Case 2: Removal of one interior column.

c) Case 3: Removal of one exterior column (left).

Figure 5. 2D frames without and with column removal.

Three distinct cases are considered as outlined below:

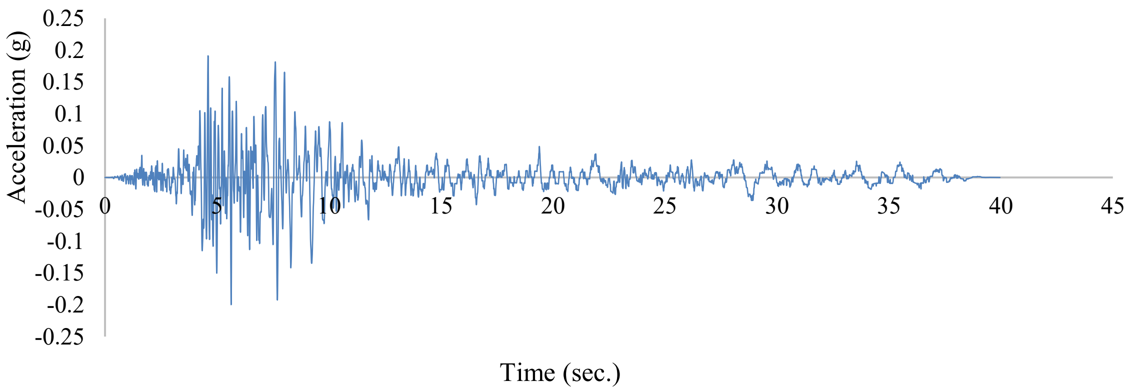
- Case 1: No removal of column.

- Case 2: Removal of one interior column (middle).
- Case 3: Removal of one exterior column (left).

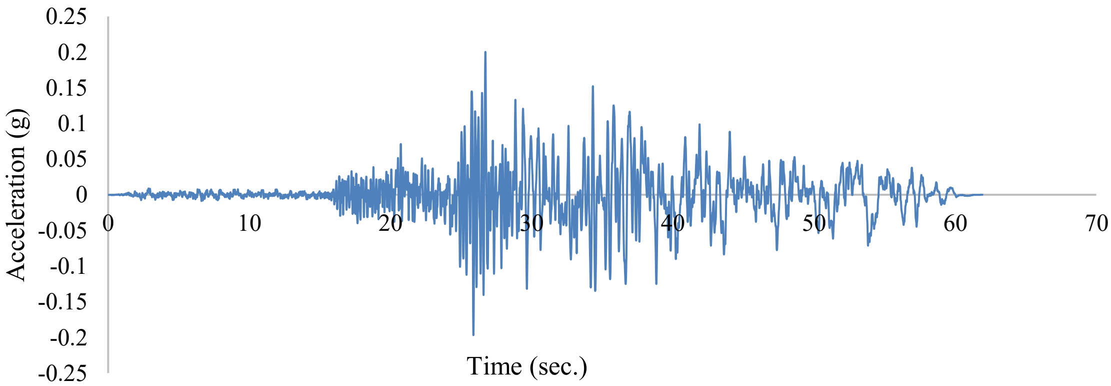
Fig. 5 shows that three distinct cases are analyzed for progressive collapse in a 2D RC frame. For the progressive collapse analysis, the frames were loaded with a safety factor of 0.2 for dead load (Gk) + 2.0 for live load (Qk), as suggested by GSA [13]. Both GSA [13] and DoD [14] recommend using linear static analysis to address a structure's ability to resist progressive collapse and to observe and assess alternate load path distribution within the existing frame components when columns at different locations are removed. Removing this column denotes a local failure triggered by progressive collapse during earthquakes.

2.2. Nonlinear Dynamic Analysis

This study focused on the progressive collapse of 2D reinforced concrete (RC) frames subjected to seven earthquake datasets. Rock-soil conditions were assumed for all seismic inputs. Fig. 6 illustrates representative earthquake records, with the complete list and details in Table 2. All of these earthquakes are normalized to 0.2g peak ground acceleration (PGA) according to Hatzigeorgiou et al. [29], assuming low- to medium-magnitude earthquakes in Malaysia. The 2D frames are subjected to earthquakes to assess outcomes such as displacements and inter-story drift, mainly under progressive collapse. A damping ratio of 5% was applied, which is typical and appropriate for RC structures. The seven earthquakes are compatible with EC8 [36] for Soil B local conditions with 0.2 PGA, as shown in Fig. 7.



a) Chalfant Valley earthquake



b) Chi-Chi earthquake

Figure 6. Example of an earthquake used in the current study.

Table 2. List of earthquakes used by Hatzigeorgiou et al., [29]; Abrahamson et al., [30]

GM	Earthquake	Station	Magnitude	PGA (g)
1	Chalfant Valley	Zack Brothers Ranch	6.3	0.0586
2	Chi-Chi	CHY025	6.2	0.0129
3	Coalinga	46T04 CHP	6.0	0.059
4	Imperial Valley	Holtville P.O.	6.6	0.258
5	Irpinia	Bagnoli Irpino	6.2	0.021
6	Mammoth Lakes	Convict Creek	6.1	0.314
7	Whittier Narrow	San Marino	5.3	0.138

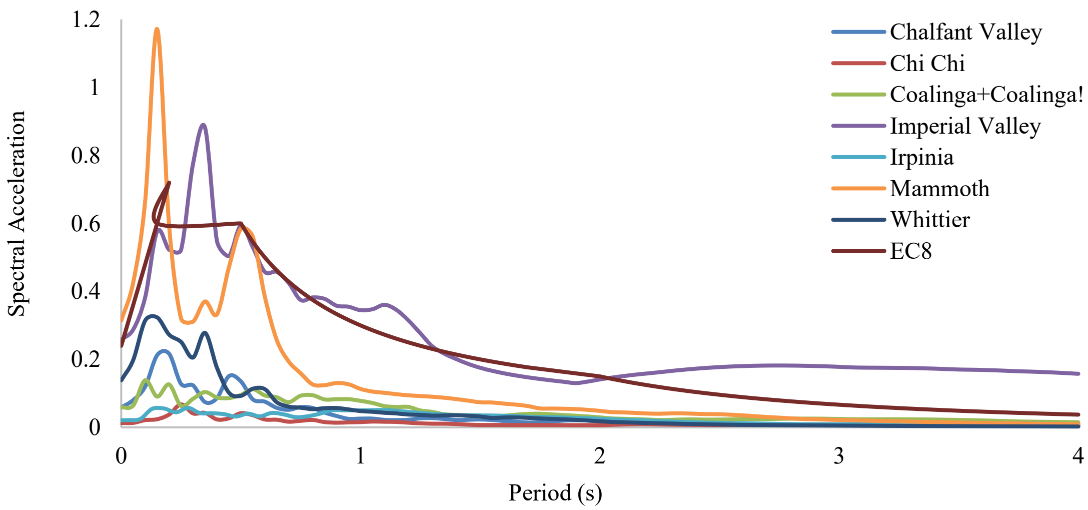
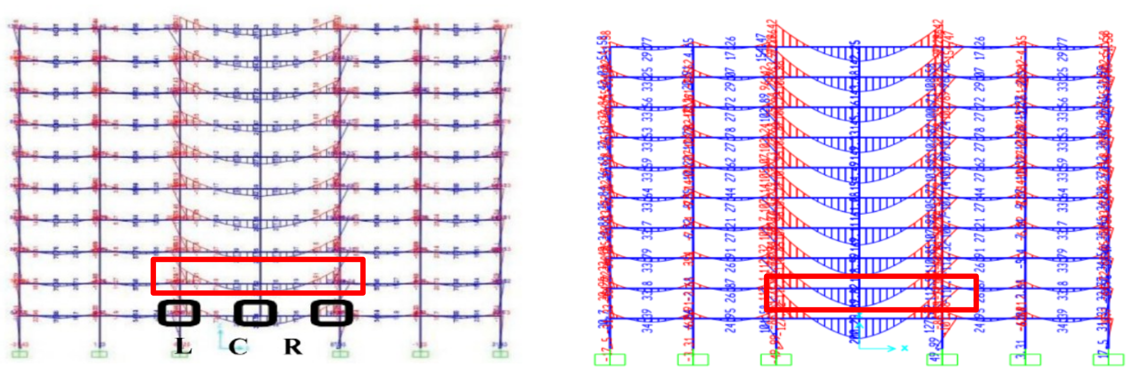


Figure 7. Response spectra of the selected earthquakes with the EC8 design spectrum

3. Results and Discussion

For validation, this study compared results from Vinay et al. [31], who investigated progressive collapse by removing a single internal column using SAP2000. This study successfully modeled and reproduced the bending moment distribution with the alternate load path, with a 5.5% difference in the second-floor internal beam, as shown in Fig. 8. The difference may be attributed to software implementation, the modeling approach and assumptions, material properties, and numerical discretization, including differences between the modeling approaches adopted. The bending moment patterns were also similar, supporting the adoption of the current study's modeling approach for further analysis.



a) A bending moment value of 307 kNm was obtained from Vinay et. al. [31]

b) A bending moment value of 290 kNm was obtained from the validation analysis.

Figure 8. Comparison of bending moment values for validation analysis.

https://doi.org/10.31436/iiumej.v27i3.4607

3.1. Alternate Load Path under Progressive Collapse

A linear static analysis was conducted across three distinct scenarios to evaluate the alternative load path mechanism and load redistribution capacity of a 2D reinforced concrete frame following the sudden removal of an interior or exterior column. The analytical findings were validated against the provisions of the British Standard (BS 8110) and the General Services Administration (GSA) guidelines. The structural response indicates that sudden column loss induces abrupt stress concentrations in adjacent columns. If these neighboring structural elements lack the requisite redundant design capacity to sustain these transferred loads, localized overstressing may trigger progressive collapse. Fig. 9 presents a comparative analysis of the resulting bending moments under both BS and GSA standards. An evaluation of the bending moment distributions across the seven spans reveals distinct geometric trends and establishes Case 3 as the most severe scenario for localized flexural demand. Case 1 represents a highly stable baseline across both design frameworks, exhibiting a flat trend line where bending moments remain uniform and minimal, fluctuating strictly between 50 kNm and 100 kNm. Conversely, Case 2 demonstrates a distinct parabolic or bell-shaped trend, wherein flexural demands originate at lower magnitudes at the outer spans, progressively climb to a maximum peak at the central 4th span, and symmetrically taper off toward the terminal spans. Under the British Standard (BS), this central peak reaches approximately 215 kNm, whereas the General Services Administration (GSA) guidelines amplify the entire curve, raising the peak to roughly 245 kNm, a 13.9% increase in peak flexural demand over BS.

Case 3 emerges as a critical vulnerability within the flexural profile, characterized by a sharply downward-sloping trend that originates with a massive, highly concentrated stress concentration at the 1st span. Under BS, this initial peak reaches approximately 300 kNm and increases to around 345 kNm under GSA provisions. This represents a 15.0% increase in localized bending stress compared to BS, constituting the highest absolute flexural magnitude recorded across the entire study and indicating severe localized beam vulnerability at the initial span. The redistribution of axial forces across the eight columns outlines a highly localized load-transfer mechanism, identifying Case 2 as the most severe scenario for critical column overloading. For both Case 1 and Case 3, the axial force profiles follow a symmetric, wave-like trend that peaks near the interior columns (Columns 3, 4, and 5) before gradually tapering down toward the perimeter. For example, the axial load sustained by Column 4 measures approximately 680 kN under BS but scales up to roughly 780 kN under GSA, representing a 14.7% vertical shift in the trend line.

In conclusion, the GSA code consistently projects a trend line that sits between 13.9% and 15.0% higher than the British Standard. This uniform upward shift underscores that the GSA code incorporates a structurally more conservative safety factor or dynamic amplification factor when evaluating alternate load paths (ALP) during progressive collapse scenarios. Case 3 constitutes the most severe condition for beam flexure (1st span peak), while Case 2 represents the most severe condition for column axial capacity (Column 5 peak). These findings are consistent with those reported by Parisi and Scalvenzi [32], Alshaikh et al. [33], Alogla et al. [34], and Jian and Li [35]. The authors observed that the loss of a column causes the surrounding region, which was originally designed to resist tensile forces, to experience significant compressive stresses, thereby increasing the load demand on adjacent structural members. Similarly, Elkady et al. [6] stated that the progressive collapse response varies with the structural system and collapse mechanism, particularly when assessed using the Alternate Load Path (ALP) method. This behavior is also evident in the results of the present study.

https://doi.org/10.31436/iiumej.v27i3.4607

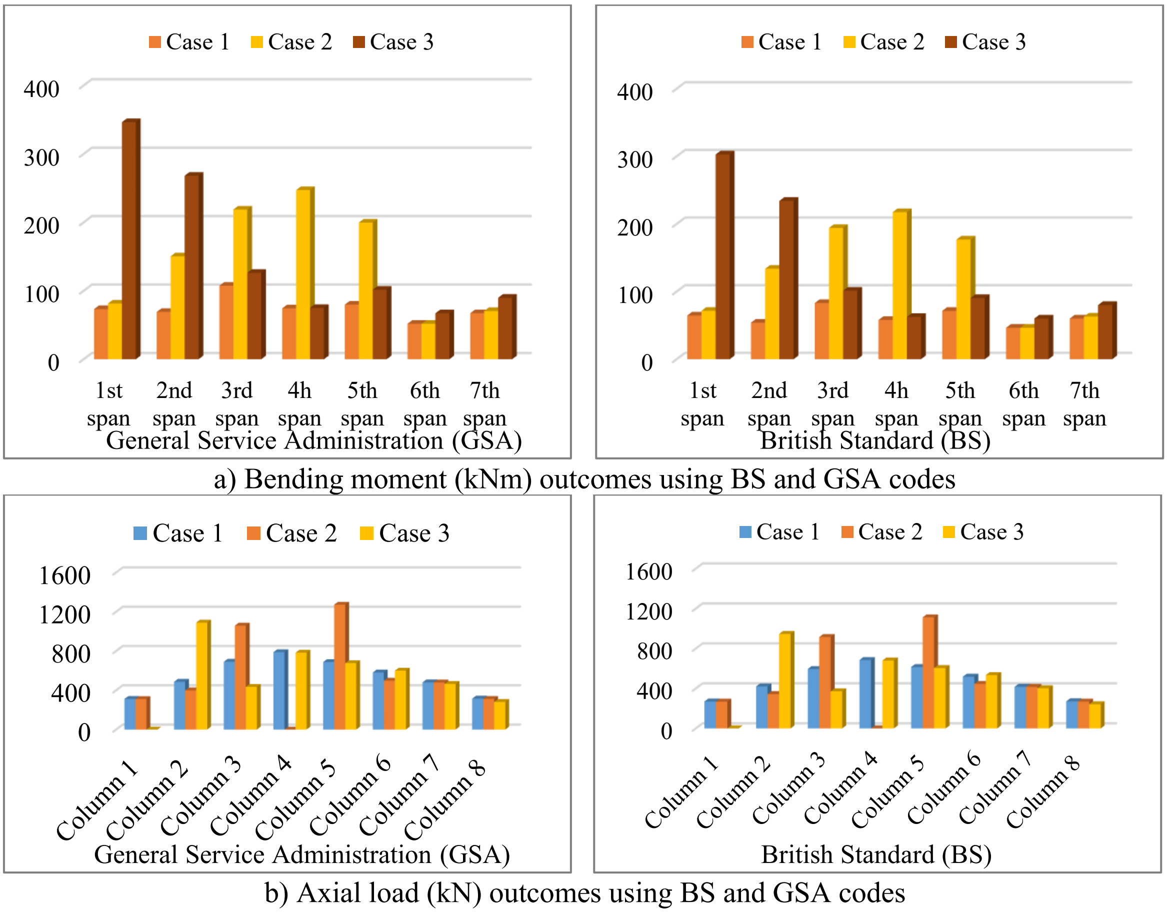
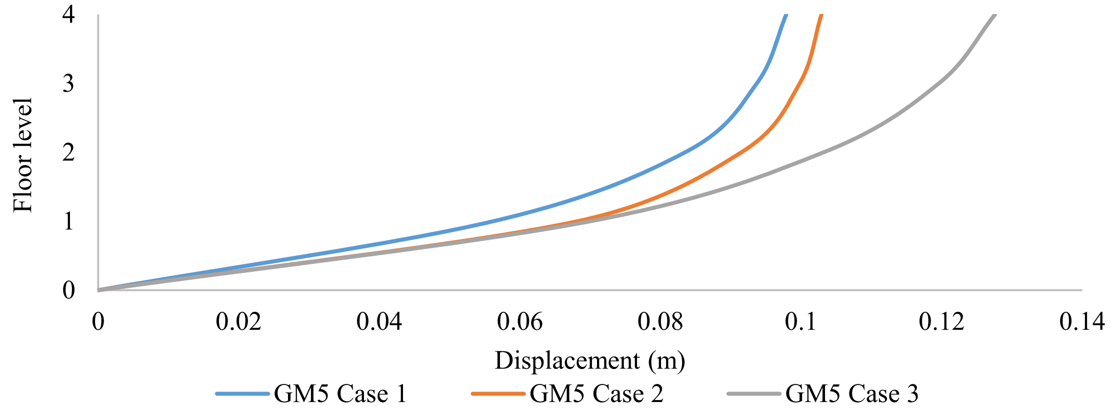


Figure 9. Alternate load path (ALP) under linear static analysis for different cases

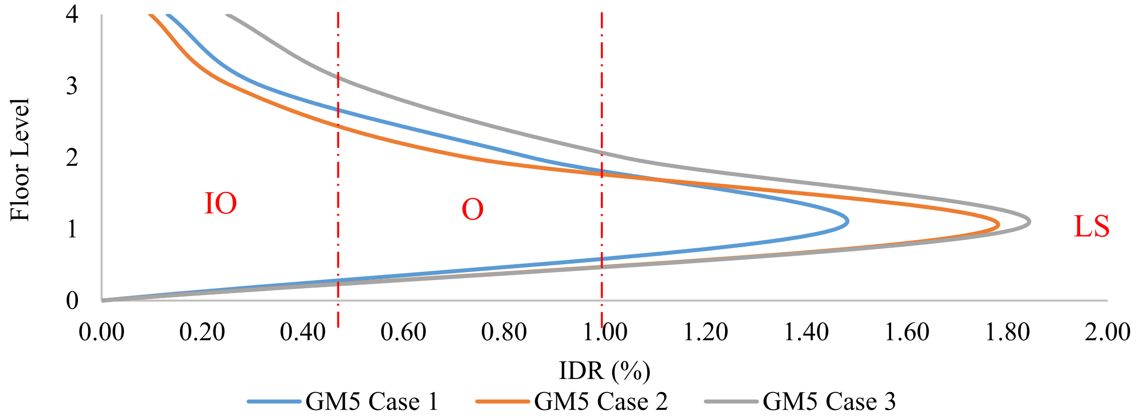
3.2. Nonlinear Dynamic Analysis

The seismic response of the two-dimensional reinforced concrete (RC) frame was evaluated under seven earthquake records, with particular emphasis on story displacements and inter-story drift ratio (IDR). The IDR was determined as the ratio of the relative lateral displacement between two consecutive stories to the corresponding story height. This study investigates the impact of different column-removal scenarios on structural response and evaluates the potential for progressive collapse. Fig. 10 presents the displacement and inter-story drift under the GM5 ground motion for three cases: Case 1 (no column removal), removal of one interior (middle) column, and removal of one exterior (left) column. The structural response under progressive collapse induced by Ground Motion 5 (GM5) was evaluated for three design cases. As shown in Fig. 10a, the lateral floor displacement increased with building height, reaching a maximum at the roof (Floor 4). Case 1 exhibited the highest lateral stiffness, with a peak roof displacement of 0.098 m. In contrast, the roof displacement increased to 0.103 m in Case 2 (5.1% higher than Case 1) and to 0.128 m in Case 3, representing a 30.6% increase, indicating a significant reduction in structural stiffness. Fig. 10b presents the interstory drift ratio (IDR), which displays a similar parabolic distribution for all cases, with the maximum drift concentrated at the first floor and gradually decreasing toward the upper stories. The peak IDR was 1.48% for Case 1, increasing to 1.78% in Case 2 (20.3%) and 1.85% in Case 3 (25.0%).

https://doi.org/10.31436/iiumej.v27i3.4607



a) Floor level vs displacement (m).



b) Floor level vs IDR (%)

Figure 10. Nonlinear dynamic analysis under progressive collapse for all cases subjected to GM5

Under GM5, comparing the optimized baseline (Case 1) with the most severe scenario (Case 3) shows a 30.6% increase in peak roof displacement and a 25.0% increase in the maximum first-story interstory drift ratio (IDR). These results indicate that drift demand increases as structural robustness decreases. Removing the exterior column significantly reduces structural stability, causing deformation to concentrate at the first story and producing the highest drift demands. In all cases, the first-story IDRs exceed the Immediate Occupancy (IO, 0.50%) and Operational (O, 1.00%) performance limits but remain within the Life Safety (LS) range. This deformation concentration highlights the need to enhance continuity ties and provide redundant load paths to improve progressive collapse resistance. Fig. 11 further confirms that Case 3 exhibits the most critical progressive collapse response under all seven earthquake records. Peak IDRs consistently occur at the first story, although their magnitude depends on the ground-motion frequency characteristics. Among the records, GM5 produces the most critical response, with a peak IDR of 1.85% and a roof displacement of 0.128 m. Within Case 3, the peak IDR under GM5 is 71.6% higher than GM2, 131.3% higher than GM3, and 640.0% higher than GM7 (0.25%), demonstrating the structure's strong sensitivity to seismic excitation and its increased vulnerability to first-story failure following exterior column loss.

According to Ghobarah [37], IDR limits classify progressive collapse performance as IO (≤0.5%), O (0.5-1.0%), LS (1.0-2.5%), and collapse (>2.5%). Most ground motions (GM1-GM4, GM6, and GM7) remained within the O–LS range, indicating controlled damage and structural stability. However, GM5 produced significantly higher IDR demands at lower stories, exceeding the LS limit and approaching collapse levels, indicating soft-story development and increased vulnerability to progressive collapse.

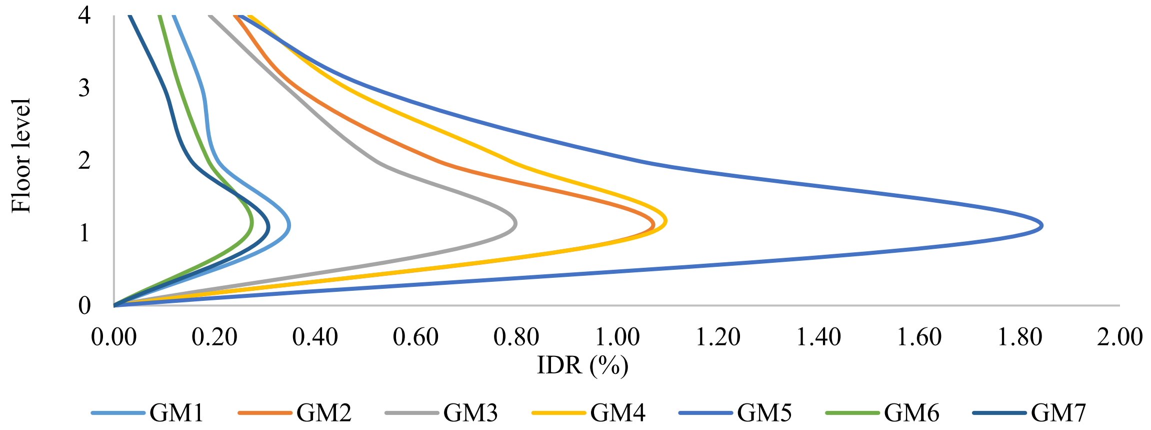


Figure 11. 2D RC frames under seven earthquakes for Case 3

4. CONCLUSIONS

This study used linear static and nonlinear dynamic analyses to predict the progressive collapse resistance of 2D reinforced concrete (RC) frames, accounting for single-column removal in interior and exterior frames. Three cases are considered: Case 1: no column removal, Case 2: removal of one interior column (middle), and Case 3: removal of one exterior column (left). The study's aims and findings can be summarized as:

For alternate load path (ALP), abrupt column removal caused load redistribution, resulting in significant stress increases in adjacent members. Under linear static analysis, the GSA code consistently projects a trend line that sits between 13.9% and 15.0% higher than the British Standard. This uniform upward shift suggests that the GSA code applies a more conservative safety factor or dynamic amplification factor when evaluating alternate load paths (ALP) during progressive collapse scenarios. Case 3 constitutes the most severe condition for beam flexure (1st span peak), while Case 2 represents the most severe condition for column axial capacity (Column 5 peak).

Next, for nonlinear dynamics, Case 3 is the most critical and vulnerable, with the highest IDR of 1.85% under GM5. This triggers a soft-story-like response at level 1, about a 25% increase in lateral sway compared to Case 1. These findings suggest that both internal and exterior column removal should be considered when assessing structural vulnerability, mainly exterior, under progressive collapse.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the financial support provided by the Ministry of Higher Education (MoHE), Malaysia, through the Fundamental Research Grant Scheme-Early Career Researcher (FRGS-EC), under Grant No. FRGS-EC/1/2024/TK06/UIAM/03/2.

REFERENCES

[1] Ellingwood BR (2006) Mitigating risk from abnormal loads and progressive collapse. J. Perform. Constr. Facil., 20(4):315-323.

[2] Russell JM, Sagaseta J, Cormie D, Jones AEK (2019) Historical review of prescriptive design rules for robustness after the collapse of Ronan Point. Structures, 20:365-373. https://doi.org/10.1016/j.istruc.2019.04.011

[3] Adam JM, Parisi F, Sagaseta J, Lu X (2018) Research and practice on progressive collapse and robustness of building structures in the 21st century. Eng. Struct., 173:122-149. https://doi.org/10.1016/j.engstruct.2018.06.082

[4] Feng F (2010) 3-D nonlinear dynamic progressive collapse analysis of multi-storey steel composite frame buildings: Parametric study. Eng. Struct., 32:3974-3980. https://doi.org/10.1016/j.engstruct.2010.09.008

[5] American Society of Civil Engineers (2023) ASCE 76-23: Standard for mitigation of disproportionate collapse potential in buildings and other structures. ASCE.

[6] Elkady N, Nelson LA, Weekes L, Makoond N, Buitrago M (2024) Progressive collapse: Past, present, future and beyond. Structures, 62:106131. https://doi.org/10.1016/j.istruc.2024.106131

[7] Li H, El-Tawil S (2014) Three-dimensional effects and collapse resistance mechanisms in steel frame buildings. J. Struct. Eng., 140. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000839

[8] Marjanishvili S, Agnew E (2006) Comparison of various procedures for progressive collapse analysis. J. Perform. Constr. Facil., 20(4):365-374. https://doi.org/10.1061/(ASCE)0887-3828(2006)20:4(365

[9] Izzuddin BA, Vlassis AG, Elghazouli AY, Nethercot DA (2008) Progressive collapse of multi-storey buildings due to sudden column loss—Part I: Simplified assessment framework. Eng. Struct., 30(5):1308-1318. https://doi.org/10.1016/j.engstruct.2007.07.011

[10] Arshian AH, Morgenthal G, Narayanan S (2016) Influence of modelling strategies on uncertainty propagation in the alternate path mechanism of reinforced concrete framed structures. Eng. Struct., 110:36-47. https://doi.org/10.1016/j.engstruct.2015.11.019

[11] Hefez I, Khalil A, Mourad S (2013) Alternate path method analysis of RC structures using applied element method. Int. J. Protect. Struct., 4(1):45-64. https://doi.org/10.1260/2041-4196.4

[12] Kim J, Kim T (2009) Assessment of progressive collapse-resisting capacity of steel moment frames. J. Constr. Steel Res., 65(1):169-179. https://doi.org/10.1016/j.jcsr.2008.03.020

[13] General Services Administration (2003) Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. U.S. General Services Administration.

[14] Department of Defense (2009) Unified Facilities Criteria (UFC): Design of structures to resist progressive collapse. U.S. Department of Defense.

[15] Tsai MH, Lin BH (2008) Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure. Eng. Struct., 30(12):3619-3628. https://doi.org/10.1016/j.engstruct.2008.05.031

[16] Gudmundsson GV, Izzuddin BA (2008) The sudden column loss idealisation for disproportionate collapse assessment. Struct. Eng., 86(6):22-26.

[17] Iribarren B, Berke P, Bouillard P, Vantomme J, Massart TJ (2011) Investigation of the influence of design and material parameters in the progressive collapse analysis of RC structures. Eng. Struct., 33:2805-2820. https://doi.org/10.1016/j.engstruct.2011.06.005

[18] Parisi F, Scalvenzi M (2019) Progressive collapse assessment of gravity-load designed European RC buildings under multi-column loss scenarios. Eng. Struct., 209:110001. https://doi.org/10.1016/j.engstruct.2019.110001

[19] Yang T, Liu Z, Lian J (2021) Progressive collapse of RC flat slab substructures with unbonded post-tensioning strands after the loss of an exterior column. Eng. Struct., 234:111989. https://doi.org/10.1016/j.engstruct.2021.111989

[20] Mohammadi A, Pachenari A, Sadeghi B (2021) Numerical study on the behavior and bearing mechanism of flat slabs in column loss events. Adv. Civ. Eng., 2021:1-12. https://doi.org/10.1155/2021/8834638

[21] Jin X, Wang S, Li K, Qian X, Deng F (2021) Study on collapse resistance of RC frame under the corner column removal scenario. Materials, 14(23):7157. https://doi.org/10.3390/ma14237157

[22] Buitrago M, Bertolesi E, Sagaseta J, Calderón PA, Adam JM (2021) Robustness of RC building structures with infill masonry walls: Tests on a purpose-built structure. Eng. Struct., 226:111384. https://doi.org/10.1016/j.engstruct.2020.111384

[23] Ren P, Li Y, Lu X, Guan H, Zhou Y (2015) Experimental investigation of progressive collapse resistance of one-story reinforced concrete specimen under penult column removal. Eng. Struct., 87:88-105. https://doi.org/10.1016/j.engstruct.2015.01.011

[24] Jenkins JP (2001) Oklahoma City bombing. Encyclopaedia Britannica.

[25] Winsor M (2021) Surfside building collapse: Death toll rises to 18 after 2 children found. ABC News. Available: https://abcnews.go.com/US/surfside-building-collapse-latest-12-dead-149missing/story?id=78574497

[26] Meng HK (2019) Dynamic characteristics and seismic performance of reinforced concrete buildings experiencing 5 June 2015 and 8 March 2018 Sabah earthquakes. Master’s thesis, School of Civil Engineering, Universiti Sains Malaysia.

[27] Computers and Structures, Inc. (2025) SAP2000 (Version 26). Computers and Structures, Inc.

[28] British Standards Institution (1997) BS 8110-1:1997 Structural use of concrete-Part 1: Code of practice for design and construction. BSI.

[29] Hatzigeorgiou GD, Liolios AA (2010) Nonlinear behaviour of RC frames under repeated strong ground motions. Soil Dyn. Earthq. Eng., 30(10):1010-1025. https://doi.org/10.1016/j.soildyn.2010.04.013

[30] Abrahamson N, Atkinson G, Boore D, et al. (2008) Comparisons of the NGA ground-motion relations. Earthq. Spectra, 24(1):45-66. https://doi.org/10.1193/1.2924363

[31] Vinay M, Rao PKR, Dey S, Swaroop AHL, Sreenivasulu A, Rao KV (2022) Evaluation of progressive collapse behavior in reinforced concrete buildings. Structures, 45:1902-1919. https://doi.org/10.1016/j.istruc.2022.10.001

[32] Alshaikh IM, Bakar BA, Alwesabi EA, Akil HM (2020) Experimental investigation of the progressive collapse of reinforced concrete structures: An overview. Structures, 25:881-900. https://doi.org/10.1016/j.istruc.2020.03.018

[33] Alogla K, Weekes L, Augusthus-Nelson L (2017) Theoretical assessment of progressive collapse capacity of reinforced concrete structures. Mag. Concr. Res., 69(3):145-162. https://doi.org/10.1680/jmacr.16.0031

[34] Jian H, Li S (2016) Progressive collapse resistance of RC frames under a side column removal scenario: The mechanism explained. Int. J. Concr. Struct. Mater., 10:237-247. https://doi.org/10.1007/s40069-016-0134-y

[35] Ghobarah A (2001) Performance-based design in earthquake engineering: State of development. Eng. Struct., 23:878-884.

[36] European Committee for Standardization (2004) Eurocode 8: Design of structures for earthquake resistance-Part 1: General rules, seismic actions and rules for buildings (EN 1998-1:2004). CEN.

[37] El-Ariss B, Elkholy S, Shehada A (2022) Benchmark numerical model for progressive collapse analysis of RC beam-column sub-assemblages. Buildings, 12(2):122. https://doi.org/10.3390/buildings12020122