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# Assessment for the Progressive Collapse of Moment Resisting Frame Structures Using a Practiced-Oriented Method

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Abnormal loads that include explosions, vehicle accidents, bombings, and earthquakes that cause column failure may lead to progressive collapse. This study investigated the potential of progressive collapse of column removal in order to evaluate the tendency of progressive collapse between Moment Resisting Concrete Frames (MRCF) and Moment-Resisting Steel Frames (MRSF) related to the deformation of rotation degree. This study also evaluated the drift limit of damage measurement. The moment resisting frames were designed based on Eurocode (EC3) and (EC8). The response of 4-, 6- and 9-storey MRCF structures and MRSF structures using the Alternative Path Method (APM) was studied whereas the locations of columns removed at the corner and at the centre of the structures were specified as Case 1 and Case 2, respectively. Pushover Analysis (POA) and Incremental Dynamic Analysis (IDA) were performed using the SAP 2000 program. Two types of framed structures with single column loss in two different locations, i.e. corner and centre were considered in this investigation. The results showed that MRCF has a larger potential of experiencing progressive collapse than MRSF. A greater Peak Ground Acceleration (PGA) value was obtained by Case 2 before it reached the Collapse Prevention Limit. Therefore, this finding revealed that column loss at the corner of buildings causes a higher risk of progressive collapse compared to column loss at the centre of buildings.

**Keywords:** column loss, corner column, center column, IDA, POA, plastic hinges, progressive collapse

## I. INTRODUCTION

Progressive collapse begins when a vertical load carrying member is cast away due to man-made causes or natural hazards. Abnormal forces are transferred to the neighbouring columns in the structure which initiate beam failure and cause partial or the whole structure.

Generally, design practices do not consider abnormal events such as gas explosions, vehicle impacts, bomb attacks, and other hazards unless the structures are exposed to these events. This scenario leads to the production of guidelines on reducing the sensitivity of buildings towards progressive collapse such as General Service Ad-

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ministration (GSA, 2003), and Department of Defense (DoD, 2005). Progressive collapses of a structure occur when loading patterns or boundary conditions are affected. For example, structural failure tends to happen when the carrying load exceeds the ultimate capacities. This failed element will eventually lead to failure mechanisms (Rakshith, 2013).

Progressive studies have become important after the occurrence of accidental cases such as the gas explosion at Ronan Point in 1968, terrorist attacks on Murrah Federal Building in 1995 and the World Trade Center in 2001 and so on (Wang *et al.*, 2014). The collapse of Ronan Point Tower in 1968 was due to structural engineering issues. According to Jalali Larijani *et al.* (2013), three main columns of Murrah Federal Building were damaged leading to the failure of the transfer girder. This event ended with the collapse of columns supported by girders and floor areas supported by damaged columns. Kyei and Braimah (2017) used the LS-DYNA software to study the effect of transverse reinforcement spacing on reinforced concrete column performance under blast loading.

Lu *et al.* (2011) suggested that the robustness of frame structures can be determined by taking the residual reserve strength ratio as the quantitative index. Robustness of structure can be clarified as the ability of structures to resist progressive collapse. Hosseini *et al.* (2014) noted that the elimination of columns will affect the adjacent column subjected to additional imposed loads which significantly increase both stress and force. These extra forces imposed on columns need to be checked to ascertain whether the structure has the ability to bridge the missing elements.

Tavakoli and Alashti (2013) carried out a study on the potential of progressive collapse for 5- and 15-storey buildings with four and six bays by applying the Alternate Path Method recommended by Unified Facilities Criteria (UFC). They found that as the number of storeys and bays increased, the resistance of progressive collapse also increased.

Chidambaram *et al.* (2016) studied the effect of fire loads on a (G+7) moment resisting steel frame structure (MRSF) residential building as one of the main reasons of structural failure. The fire load was given at different column locations (corner, edge, intermediate, and re-entrant column) under a given temperature of $550^{o}$C. As per GSA guideline, the demand capacity ratio (DCR) was the index of the progressive collapse for each element. Elshaer *et al.* (2017) made a parametric study to investigate the effect of different parameters on progressive collapse. These parameters include the location of column removal, level of the removed column, case of loading, and the slab in a progressive collapse. This investigation was performed by using the "Applied Element method" for a structure under seismic load, and UFC guideline requirements.

Sideri *et al.* (2017) considered blast loading as one of the main reasons for the progressive collapse. They have investigated and evaluated the structural robustness of adjacent structural member damage distribution induced by blast loads. Yu *et al.* (2017) studied the effect of corrosion reinforcement of an aging reinforced concrete structure subjected to the scenario of middle column loss. Based on the dynamic load-displacement curve pushdown analysis, the results showed that old buildings with high severe corrosion are more vulnerable to progressive collapse than the newly constructed ones.

Goel *et al.* (2017) investigated the critical load path or load transfer and the collapse behaviour of an RC building subjected to sudden blast load. This investigation was based on the removal of certain columns and observes the load path. On the other hand, the behaviour was investigated in terms of joint displacement, vertical reaction, and the axial force after removal due to blast load. Rezvani *et al.* (2017) investigated the effect of using the inverted V-bracing on enhancing steel moment resisting frames (SMRF) that were subjected to the loss of exterior columns that would lead to failure progression. The study was based on the Dynamic Increase Factor (DIF) that suggests the estimation required for the steel bracing cross-sectional area for strengthening the structure.

Gerasimidis *et al.* (2015) conducted a progressive collapse analysis of a 20-storey steel frame with the removal of corner columns according to the alternate load path approach. At the moment of removal, two adjacent columns failed due to elastic flexural-torsional buckling. Al-Salloum *et al.* (2016) investigated the failure mechanism and the vulnerability of medium-rise circular RC buildings against the progressive collapse generated from the blast load.

Kandil *et al.* (2013) emphasised different parameters affecting the behaviour of steel frames under progressive collapse. The study concluded that increasing values of damping in a dynamic analysis will result in decreasing maximum lateral deflection. Moreover, the potential of progressive collapse reduces as the number of storeys increases due to structural members involved in the resistance of progressive collapse behaviour. Asgarian and Hashemi Rezvani (2010) studied the progressive collapse of 10-storey buildings designed with Concentrically Braced Steel Frames (CBF). In order to clarify the critical location of element removal, the impact factor of structural number for different storeys was determined.

Feng *et al.* (2017) proposed a novel method to improve the lateral collapses due to the earthquake and progressive collapses. Kinked rebars were placed in the beams of a six-storey RC frame to improve the seismic behaviour and the progressive collapse resistance. Hadi and Saeed Alrudaini (2012) have proposed a scheme for retrofitting RC buildings to resist progressive collapse resulting from first floor column failure. Vertical cables were connected at the end of beams and hung on hat steel braced frames on top of the building. Alrudaini and Hadi (2010) proposed another novel method to increase the progressive collapse resistance of RC buildings by using embedded vertical cables in the column and hanging those cables at the top to a hat-steel braced frame placed on the top of the building.

This research was conducted for Moment Resisting Concrete Frame (MRCF) and Moment Resisting Steel Frame (MRSF) buildings measuring 4, 6 and 9 storeys. Two cases of column removal at the corner and at the centre were considered as Case 1 and Case 2 respectively. The primary focus was on evaluating the tendency of progressive collapse potential related to the deformation limit of rotation degree and the drift limit of damage measurement. The Alternate Path Method (APM) which consists of POA and IDA non-linear dynamic analyses were employed in this study, as Dinar *et al.* (2013) and other researchers highlighted the important of using in-elastic pushover analysis as well the incremental dynamic analysis for demand predictions. Moreover, the most critical scenario relative to the location of column removal was evaluated through damage measurement.

## II. METHODOLOGY

4-, 6-, and 9-storey frame buildings were selected because the evaluation of progressive collapse must be 4 storeys or greater as stated in the GSA guideline. Two cases were considered in this study. Case 1 represents the removal of columns at the corner of the structure whereas Case 2 represents the removal of columns at the centre of the structure.

The procedure started with the design of the moment resisting frames based on Eurocode. By using the SAP2000 software(CSI, 2004), pushover analysis (POA) and Incremental Dynamic Analysis (IDA) were performed. The number of plastic hinges and drift limit states were determined. In order to assess the structural performance of these structures, five performance levels were considered according to Xue *et al.* (2008). Figure 1 shows the flowchart methodology.

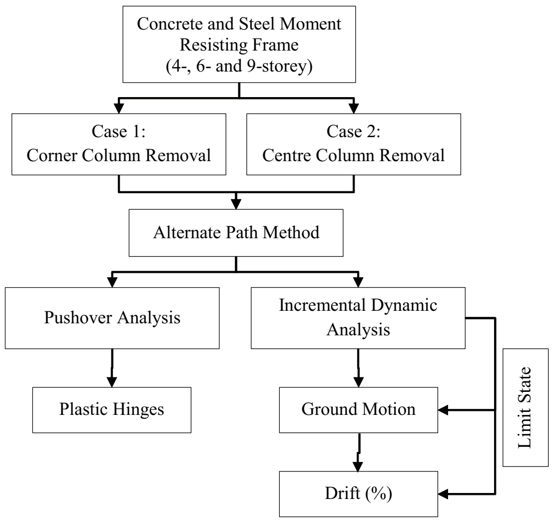



Figure 1. Flow Chart Methodology

### A. Material properties and loading

In this study, three buildings of different sizes (4-, 6-, and 9-storeys) were investigated. The height of each storey was 3.0m with a bay width of 6.0m and slab thickness of 15cm. As shown in Figure 2 (a), 2 (b), and 2 (c). The structure had no irregularities in its elevation.

The Moment Resisting Concrete Frames were designed based on Eurocode, and the designed ground acceleration is assumed to be 0.5g of ground type A which refers to rocks in a geological formation. The gravity loads consisting of dead loads (Gk) and live loads (Qk) were applied to all floors which were taken as 5.3kN/m$^2$, and 4kN/m$^2$ respectively. Table 1 illustrates the structural modelling data input for analysing MRCF and MRSF. The column size selected measured 500mm x 500mm, whereas the beam size measured 300mm x 700mm with a compressive strength of 30MPa. Based on the frame design, the columns and beams were reinforced with T20 bars and T10 link reinforcement with a yield stress of 460N/mm$^2$.

For Moment-Resisting Steel Frames (MRSF), the design followed EC3 and associated with EC8, the steel grade was assumed as S275 for the steel frame structure. Based on the universal columns (UC) and beams (UB) standard sections that refer to the specifications of the H-shape, the column section sizes used were 305mm x 305mm x 198mm for 4- and 6-storey buildings and 305mm x 305mm x 240mm for

9-storey buildings. For beams, the selected dimensions were 533mm x 210mm x 92mm for 4- and 6-storey buildings and UB533mm x 210mm x 101mm for 9-storey buildings. Table 2 and 3 illustrate the section properties for MRCF and MRSF respectively.

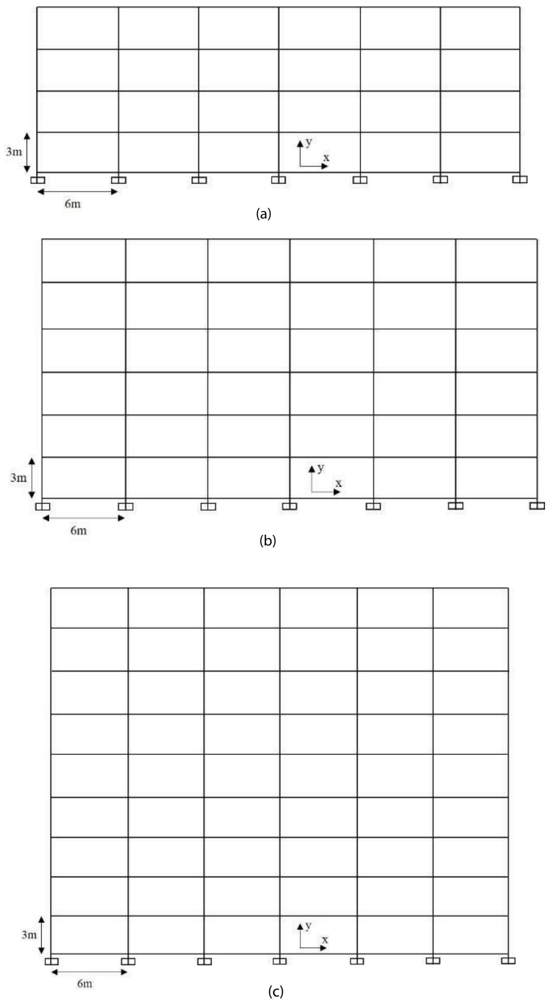



Figure 2. Moment resisting frame (MRF) elevation view (a) 4-storey frame; (b) 6-storey frame; and (c) 9-storey frame

## III. ANALYSING AND MODELLING THE PROGRESSIVE COLLAPSE

The potential for progressive collapse of these buildings is due to the removal of columns based on the Alternative Path Method (APM). Two cases of column removal were applied in the analysis. Case 1 referred to the removal of corner columns while Case 2 referred to the removal of centre columns of buildings as shown in Figure 3 (a) and (b). Both cases demonstrated the removal of columns at ground floor level. In this paper, the analyses were done using pushover analysis (POA) and incremental dynamic analysis (IDA). Therefore, the mentioned procedure has been applied for analysing the structure for three different storey heights. According to Unified Facilities Criteria (UFC) guidelines, the following load combinations; equation (1) and equation (2) have been applied to the entire structure.

*For non-linear static analysis (POA):*

$$2(Gk + 0.25Qk) \tag{1}$$

$$(Gk + 0.25Qk) \tag{2}$$

### A. Plastic Hinges and Drift limits

Pushover analysis and incremental dynamic analysis were conducted after column removal. The increase in load due to column removal will cause excessive load to the residual structure. Plastic hinges are formed and the drift values increase as the load increases. The value of the plastic rotations at the plastic hinges will be assessed whether it exceeds the limit (0.05rad).

Table 1. Structural modelling data input for MRCF and MRSF

| Input structural data analysis parameters | |
|---|---|
| Slab thickness | 15cm |
| Bay width | 6m |
| Storey height | 3m |
| Dead load (Gk) | 5.3kPa |
| Live load (Qk) | 4kPa |
| Ground Type | A |
| Design ground acceleration on type A ground | $ag = agR.\gamma I$ |
| Important factor ($\gamma I$) | 1.0 |
| Peak ground acceleration ($agR$) | 0.5g |
| Type of analysis | Nonlinear analysis |

Table 2. Section properties of Moment Resisting Concrete Frame (MRCF)

| Description | Column | Beam |
|---|---|---|
| Section size | 500mm x 500mm | 300mm x 700mm |
| Concrete Strength | 30N/mm$^2$ | 30N/mm$^2$ |
| Steel Yield Stress | 460 N/mm$^2$ | 460 N/mm$^2$ |
| Longitudinal Bar Diameter | 20mm (T20) | 20mm (T20) |
| Link Bar diameter | 10mm (T10) | 10mm (T10) |

Meanwhile, the % drift after performing the IDA analysis will be compared with the drift limit suggested by FEMA-273 and Xue *et al.* (2008). The recommended limit states are OP, IO, LS, DC, and CP at % drift equivalent to 0.5%, 1%, 1.5%, 2%, and 2.5% respectively.

### B. Ground motion records

IDA analyses require a suitable set of ground motion records. According to Nazri (2011), a few parameters to consider during the selection of ground motion are event magnitude, peak ground acceleration (PGA), distance, and soil type. Ground motion time-histories recommend a minimum of three sets of ground motion records as stated in FEMA 450. Thus, for this study, three sets of ground motion records were used. Smerzini *et al.* (2014) suggested that the range of magnitude to be considered should be between 5.0 to 7.3. Table 4 represents the selected ground motion events from the PEER NGA website.

## IV. RESULTS AND DISCUSSION

### A. Pushover analysis

Pushover analyses performed consisted of two load cases applied to the structure. Gravity loads and lateral loads were applied over the height of the structure. The gravity load on the members is a combination of Dead Load (DL) and Live Load (LL). Meanwhile, load distribu-

Table 3. Section properties of Moment Resisting Steel Frame (MRSF)

| **Number of Storey** | **4- and 6-Storey** | | **9-Storey** | |
|---|---|---|---|---|
| Section | Beam | Column | Beam | Column |
| Size | 533x210x92 | 305x305x198 | 533x210x101 | 305x305x240 |
| Area Section, A($cm^2$) | 117 | 252 | 129 | 306 |
| Depth of Section, D (mm) | 533.1 | 339.9 | 536.7 | 352.2 |
| Width of section, B (mm) | 209.3 | 314.5 | 210 | 318.4 |
| Flange thickness, tf (mm) | 15.6 | 31.4 | 17.4 | 37.7 |
| Web thickness, tw (mm) | 10.1 | 19.1 | 10.8 | 23 |
| Mass (kg/m) | 92.1 | 198.1 | 101 | 240 |

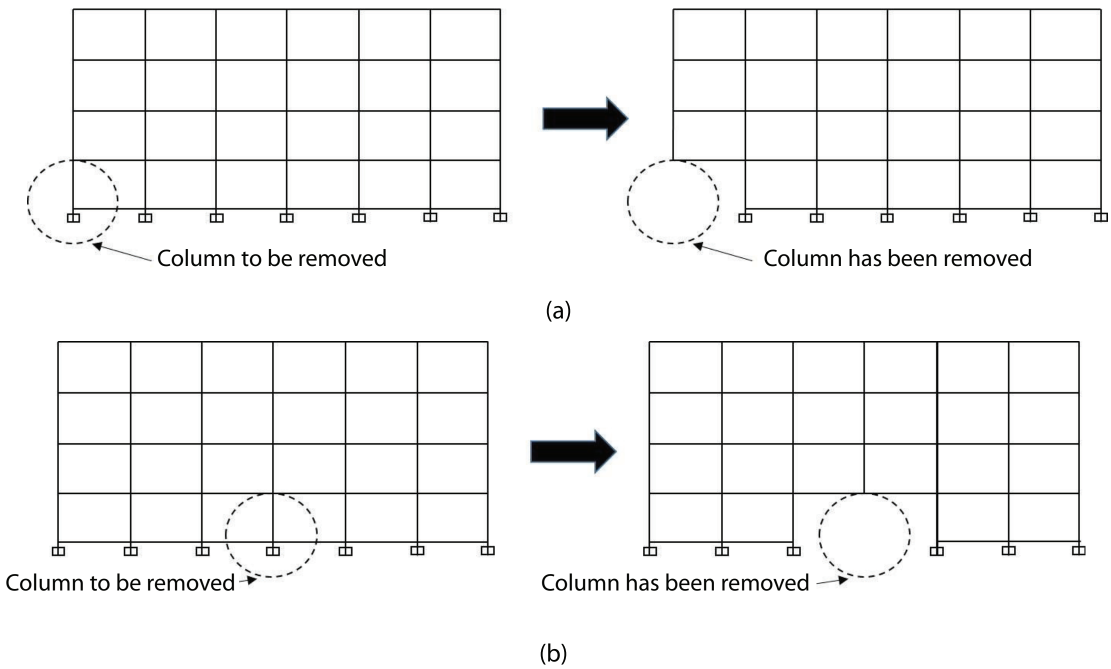



Figure 3. Removal of column (a) Case 1: Corner Column; (b) Case 2: Center Column

tion pattern for lateral loading used is triangular distribution. Through the analysis, the pushover curve of base shear versus displacement was plotted.

There were 12 structures analysed using Pushover Analysis (POA) including Moment Resisting Concrete Frame (MRCF) for Case 1 and Case 2, and Moment Resisting Steel Frame (MRSF) for Case 1 and Case 2 with three different storey heights. The base shear demands from these analyses were compared.

Based on the pushover curves in Figure 4(a) and 4(b), the base shear values of 4-, 6-, and 9-storey MRCF structures and 4-, 6-, and 9-storey MRSF structures computed from the analysis were 1919.89kN, 1875.16kN, and 1553.97kN, 4523.59kN, 3987.78kN and 3728.58kN, respectively for Case 1. In this case, the highest

Table 4. Selection of Ground Motion

| Event | Magnitude | Year |
|---|---|---|
| San Fernando | 6.61 | 1971 |
| Imperial Valley | 6.53 | 1979 |
| Morgan | 6.19 | 1984 |

base shear value was obtained by the 4-storey MRSF structure. The highest base shear value obtained was 4523.59kN. Meanwhile, 9-storey MRCF structure carried the lowest value of base shear is 1553.97kN.

For Case 2, the base shear values of 4-, 6-, and 9-storey MRCF structures and 4-, 6-, and 9-storey MRSF structures computed from the analysis were 1374.56kN, 1189.33kN, 978.03kN, and 4550.27kN, 4056.09kN and 3847.86kN, respectively. The same scenario occurred for Case 2 where the highest base shear value obtained was obtained by the 4-storey MRSF structure (4550.27kN) while the lowest value was obtained by the 9-storey MRCF structure (978.03kN).

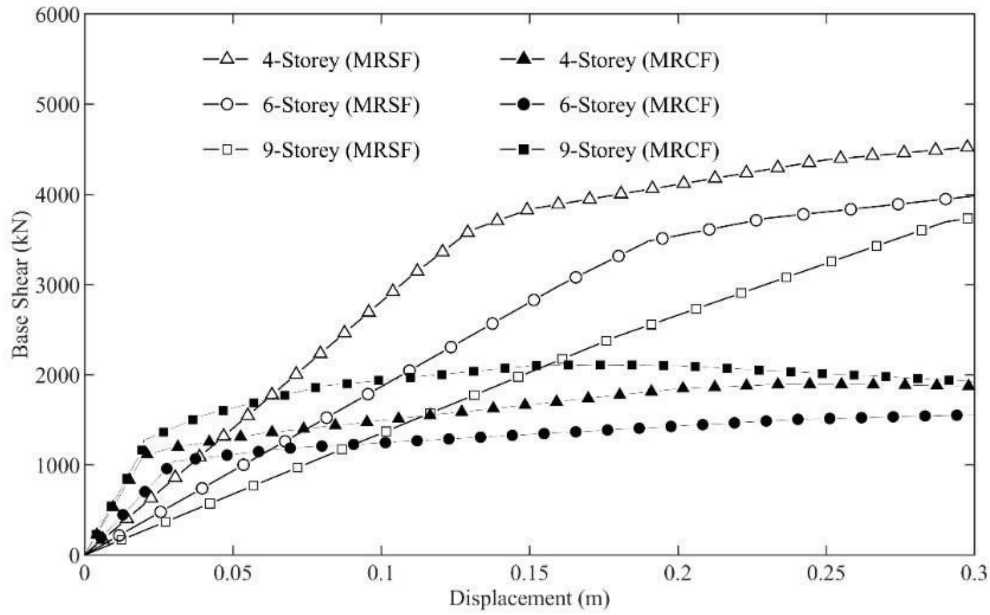
The result showed that MRSF has the ability to carry a higher value of shear force compared to MRCF. This is due to the higher moment capacity possessed by MRSF compared to MRCF. In the case of column removal, the base shear value of 4-storey structures is greater than that of 6- and 9-storey structures. This is explained by the fact that the weight of structural elements due to column removal is transferred to other elements. Therefore, this situation leads to an increase in structural pressure.

### B. Plastic Hinges

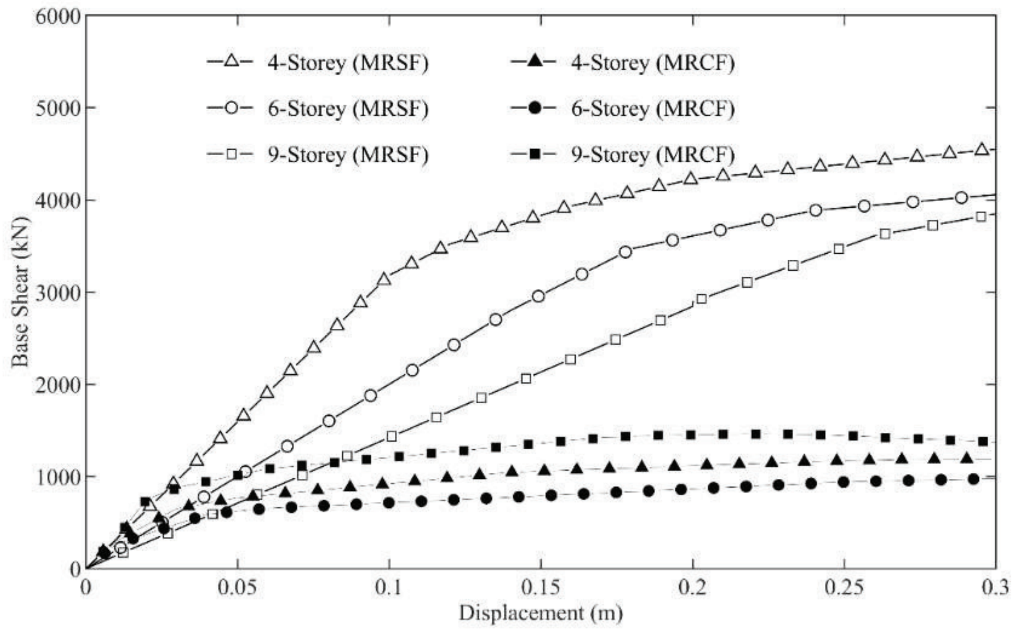
As a result of the pushover analysis, the formation of hinges can be viewed graphically on a step-by-step basis. Figure 5 and Figure 6 show the distribution of plastic hinges for Moment-Resisting Concrete Frame (MRCF) Structure for Case 1 and Case 2. The results presented in the figures show the value of plastic rotation of each hinge formed. As shown, most of the values are larger than the limit value recommended by FEMA (0.05 rad). Therefore, structure failure is represented by the occurrence of plastic hinges at the end of the beams. This instability of the structure system enhances the collapse mechanism.

However, for the Moment Resisting Steel Frame (MRSF), the sudden removal of the column at the first floor causes the downward displacement at the point in different storeys above the removed column as shown in Figure 5 and Figure 6. Moreover, no plastic hinges formed in MRSF due to the removal of a single column.

Based on the results, it can be concluded that the Moment Resisting Steel Frame (MRSF) will not suffer progressive collapse as it successfully absorbed the loss of the first-floor column which was due to the removal of a single column (either corner or centre column). Therefore, MRSF is more sustainable in resisting single column loss compared to MRCF which has a larger potential for progressive collapse.




(a) Case 1: (Corner Column)




(a) Case 1: (Center Column)

Figure 4. Pushover Curve of MRCF and MRSF

### C. Incremental Dynamic Analysis (IDA)

It is very important to determine whether structure due to column loss has adequate durability by referring to the drift limit state. In this study, two schemes (Case 1 and Case 2) of IDA analysis were considered. Only MRCF was applied in this analysis due to a higher potential for progressive collapse.

According to the observation of the mean

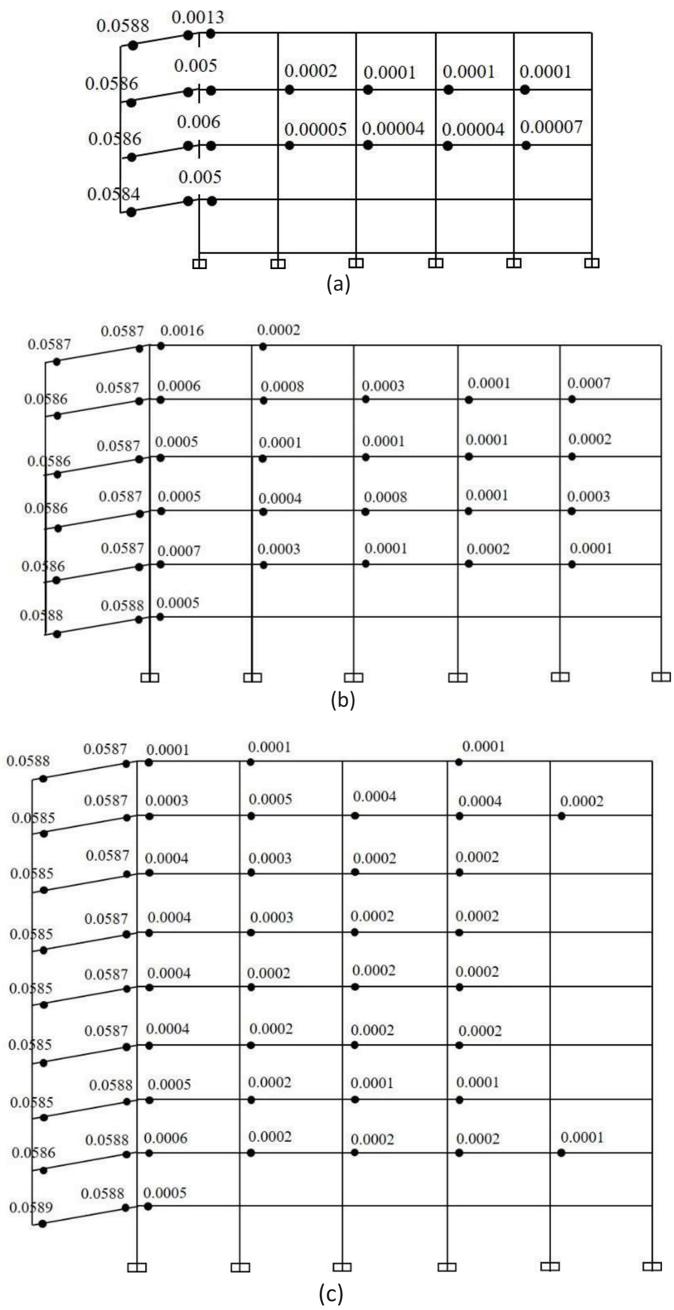



Figure 5. Distribution of Plastic Hinges of Moment-Resisting Concrete Frame (MRCF) for the Removal of Corner Columns in (a), (b) and (c), Case 1

IDA curves in Figure 7(a) and Figure 7(b), the incremental dynamic analysis (IDA) shows that the total height of the structure can influence the potential of the structure to collapse. In the first case (Case 1), the element removed at the corner of the structure showed that the 4-storey MRCF structure reaches the collapse prevention limit state (CP) at peak ground acceleration (PGA) of 3g. This was followed by PGA values of 4.2g and 5.1g for 6-storey and 9-storey

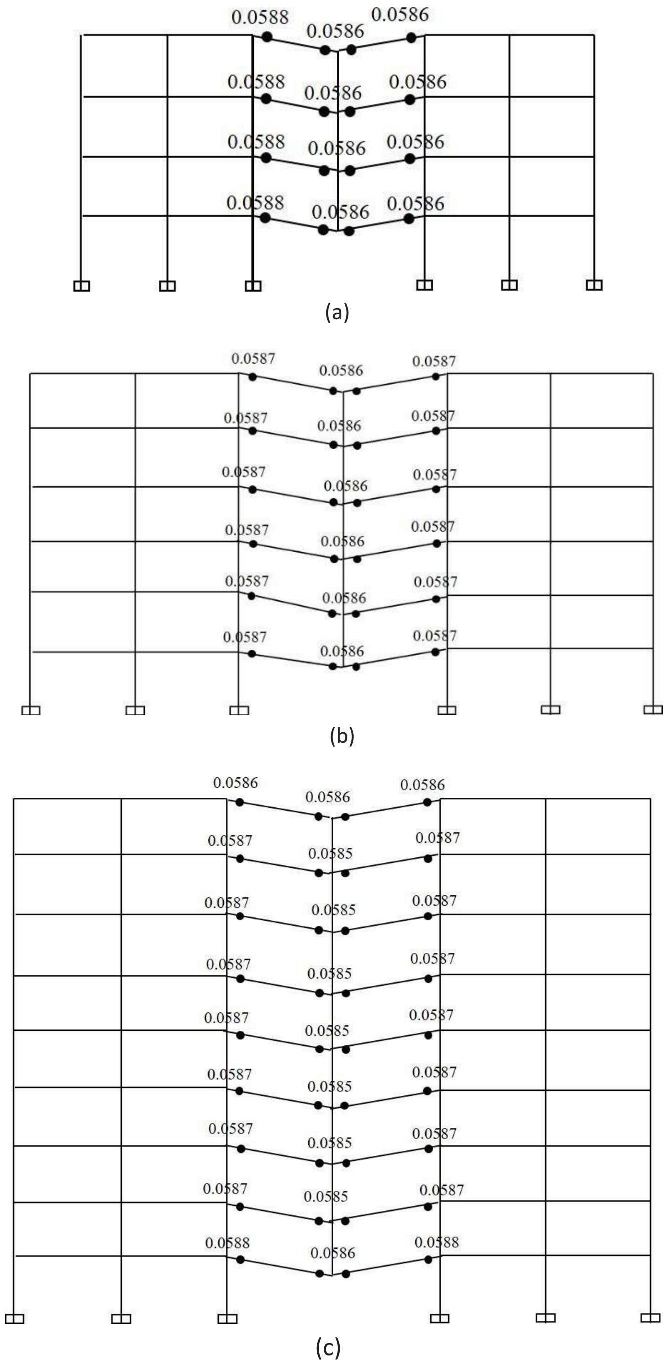



Figure 6. Distribution of Plastic Hinges of Moment-Resisting Concrete Frame (MRCF) for the Removal of Center Columns in (a), (b) and (c), Case 2

buildings respectively. For the second case (Case 2), the element removed at the centre of the structure showed that the 4-storey MRCF structure reaches the collapse prevention limit state at a PGA of 3.8g, followed by PGA values of 4.3g and 5.4g for 6-storey and 9-storey buildings respectively. Therefore, it can be concluded that higher buildings can better withstand shaking due to higher PGA. Moreover, the potential for progressive collapse reduces as the number of storeys increases. This result is in agreement with a previous study done by Kandil *et al.* (2013) where it was found that the potential of progressive collapse reduces as the number of storeys increases because of structural members which resist progressive collapse behaviour. In addition, from the analysis, structures which lose a centre column was found to be more sustainable compared to structures which lose a corner column as corner column loss leads to a higher tendency of progressive collapse.

## V. CONCLUSION

This study investigates the behaviour of Moment-Resisting Concrete Frame (MRCF) and Moment-Resisting Steel Frame (MRSF) towards progressive collapse. This study emphasises the elimination of external structural columns in two different locations. In the first case, the column at the corner of a structure is removed. For the second case, the column at the centre of the structure is removed. SAP 2000 was used to analyse both cases using Pushover Analysis (POA) and Incremental Dynamic Analysis (IDA).

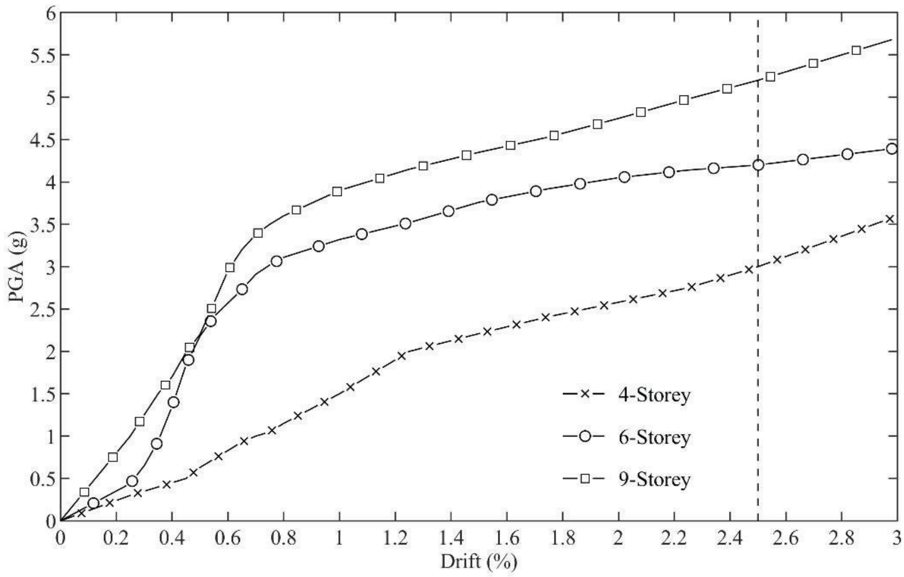
Based on the results, the following conclusions can be made:

1. Based on the Pushover Analysis (POA), MRSF was found to be more resistant towards damage or failure due to a smaller displacement induced to the structure when a larger shear force is applied.

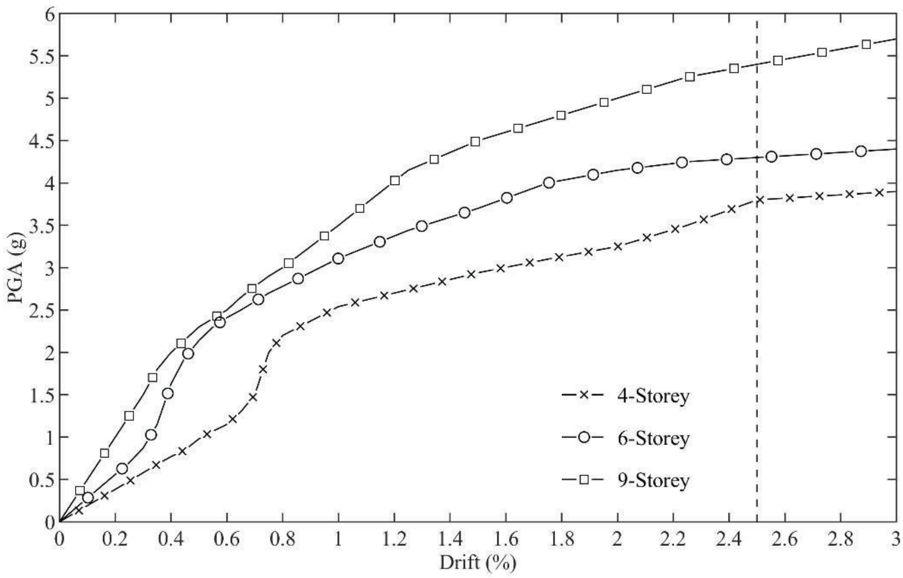
2. The highest value of plastic rotation recorded (0.0589 rad) occurred in the 9-storey MRCF structure of Case 1 and Case 2. MRCF has a larger potential of experiencing progressive collapse than MRSF.

3. Higher buildings are better able to resist shaking due to higher Peak Ground Acceleration (PGA). Moreover, the potential for progressive collapse reduces as the number of storeys increases.

4. The critical condition of corner column removed from the structure is observed. Therefore, the removal of corner columns increases the potential for progressive collapse compared to the removal of centre columns.




(a) Case 1: (Corner Column)




(b) Case 2: (Center Column)

Figure 7. Mean IDA curves of (a): Case 1, and (b): Case 2

## VI. ACKNOWLEDGMENTS

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