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Progressive Collapse of RC Frame Under Different Levels of Damage Scenarios

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ABSTRACT

Progressive Collapse analysis of Reinforced Concrete (RC) frame was carried out using commercial software i.e. SAP2000. The RC frame consisting of nine stories was selected and designed as per Pakistan Building Code. Two damage patterns were considered for the progressive collapse analysis; damage at corner column and damage at edge column. The General Services Administration loading criterion is followed to carry out Linear Static Analysis with 40%, 80% and fully damaged scenarios. In addition to Linear Static Analysis, Nonlinear Static Analysis and Nonlinear dynamic Analysis was also carried out to assess the vulnerability of the structure exposed to progressive collapse. After that results were analyzed to determine the nature and intensity of structural damage due to column failure. It was found that edge column with longer spans has more damage potential as compared with smaller spans in Linear Static Analysis. However, in Non linear Static Analysis and Non linear Dynamic Analysis, the hinges are at their initial stages in all cases and progressive collapse is less critical. Therefore, Linear Static Analysis based on General Services Administration guidelines is more conservative than Nonlinear linear Static Analysis and Non linear Dynamic Analysis.

1. Introduction

In Pakistan terrorism is a menace, which has necessitated this research to make our structures safer from progressive collapse. The term ‘progressive collapse’ can be defined as the sudden/accidental load causing initial local failure in the frame which may initiate a chain reaction from element to element within the structure, leading to full/partial collapse of the structure. Once a column is damaged due to some external effect like blast loading/impact loading, a change of load path occurs and gravity load of the building is transferred to the adjacent columns in the structure. In the United States, the General Services Administration (GSA) [1] provides guidelines and measures to avoid progressive collapse. The GSA criteria/guidelines include a threat independent phenomenon of progressive collapse analysis and describes the procedure for analysis and usage of the Demand Capacity Ratio (DCR), to evaluate progressive collapse potential. As per GSA guidelines, DCR value of typical buildings should not be greater than 2 and for atypical buildings the value should not exceed 1.5. DCR values ranging from 1-1.5 have low collapse potential and values greater than 1.5 have high collapse potential. Marjanishvili [2] estimated the progressive collapse potential for buildings and classified progressive collapse as a dynamic event in which building elements show vibration which is the disturbance of the initial load equilibrium of external loads and internal forces due to member loss and consequently it vibrates until a new equilibrium position is found or until it collapses. Progressive collapse is immanently a non-linear process in which the elements of the structure are in tension until it goes to elastic limit of failure. Nonlinear Dynamic Analysis shows the most accurate results, but with more complexity. Sasani [3] conducted testing of 6 story RC building located in San Diego. The experimental and analytical analysis revealed that damage of columns may lead to partial/complete collapse due to progressive collapse. The San Diego building which was a hotel was equipped with strain gauges to measure strain values on the exterior column which were removed. They provided valuable results on how the structure would respond when faced with abnormal conditions. However, Demand Capacity Ratios values were not calculated to study progressive collapse potential. Therefore Sasani [3] used purely field data and no computer simulation techniques were adopted. Sasani [4] carried out finite element analysis of a model building and compared the results with the DCR method. He concluded that the DCR method is overly conservative. A research study was conducted by Sezen [5] to test progressive collapse potential of the Ohio State Union building which was scheduled for demolition in 2007. The building itself was unique because some of the second floor had collapsed prior to initiation of the experiment. The DCR values and some SAP2000 [6] analysis results were excessively high due to the unique properties of the structure, inaccurate data recording, and demolition site inconsistencies. Therefore Sezen [5] suggested that future structures studied should be fully intact and not damaged. Feng [7] studied the behaviour of a 20 storey steel composite frame building

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under sudden column removal with a 3-D finite element model using the ABAQUS package [8]. Based on this model, parametric studies were carried out to investigate the structural behaviour with variations in: strength of concrete, strength of structural steel and reinforcement mesh size. Through the parametric study, measures to mitigate progressive collapse design were recommended. Lu et.al [9] carried out research for the assessment of progressive collapse resistance of reinforced concrete frame structure. The software used for this purpose was OpenSees software [10]. Investigation was carried out to see the effect of instant column removal as well as column removal duration. Beams containing fiber sections and plastic hinges were used in this research. It was concluded that for assessing complex structures, this method is efficient and can be applied for progressive collapse evaluation. Kim [11] investigated RC frame structures subjected to sudden damage of a first-story column that led to progressive collapse. They carried Out Non-Linear Dynamic Analysis and concluded that the structures with no seismic design are very vulnerable to progressive collapse as compared to structures with proper seismic design provisions. Sagiroglu and Sasani [12] estimated the response of a seven-story reinforced concrete frame structure and noted the effect of 15 simulated column removal scenarios. They found that a top floor column removal is more likely to cause structural collapse than failure on a lower floor. Therefore special concentration should be used to model the frame structure with floor system.

In this research work, a nine-story frame building was selected. The frame was subjected to loading as described by GSA guideline and analyzed under two damage cases which include corner column damage and edge column damage. The demand capacity ratio for beams in linear analysis and deflection at the critical joints were evaluated in all analysis cases with 40%, 80% and fully damaged consideration. Non Linear Static and Dynamic Analysis was also carried out. It was concluded after the analysis that in Linear Static Analysis, the edge column case with long bays are critical in the event of progressive collapse and collapse of building can occur in short interval of time. Therefore, it is mandatory to control deflection under the damaged joint because large deflection will cause collapse of structure.

2. Progressive Collapse Examples in Pakistan

Explosives are the primary weapon for most terrorists. Due to progressive collapse of buildings resulting from any accidental load like blast or fire, a number of commercial buildings, governmental buildings and security offices have become insecure and unsafe. Masonry structures are very common in Pakistan. Most of the buildings in Pakistan which are under blast attack were masonry structures. In most of the cases load bearing walls are the common reason for collapse of structures as in the case of Police and ISI Headquarter Attacks as shown in Fig. 1. Some examples of progressive collapse occurrences in Pakistan are listed in Table 1 below.

Table 1: Progressive Collapse Examples in Pakistan

Incident	Place	Date	Casualties
Police & ISI Headquarter	Lahore	27th May, 2009	Killed 35 and Injured 250
FIA Building	Lahore	15th Oct, 2009	Killed 38 and Injured 20
PC Hotel	Peshawar	10th Jun, 2009	Killed 17 and Injured 46

Fig. 1: Police and ISI Headquarter Attacks (Lahore, 2009)

3. Causes of Progressive Collapse

There are many reasons for the cause of progressive collapse. Often, most of the collapses will happen during construction phase of a building:

1. Misunderstanding between contractors and engineering documents can initiate a progressive collapse. In this scenario, incorrect installment of particular structural member can lead to weakened structural members throughout the building initiating progressive collapse such as in Skyline Towers Building, Virginia, USA, March 2, 1973.
2. Construction equipment may also fail, because the construction with lift slab technology initiates progressive collapse such as in L'Ambiance Plaza in Bridgeport, Connecticut, USA on April 23, 1987.
3. Improper assessment or supervising structural issues also leads to initiation of progressive collapse such as in Harbour Cay Condominiums, Florida, USA on March 27, 1981.
4. Structures without proper routine maintenance, material failure are other reasons and rusting can occur which weakens the structural member or the whole structure initiating progressive collapse such as in Savar, Bangladesh on 24 April 2013.

4. Model Description

A building frame having nine stories with six bays in longer direction and three bays in shorter direction was selected for carrying out progressive collapse analysis. The dead load and live load applied on the slab was 1.91 kN/m^2 and 2.394 kN/m^2 respectively [13]. Lateral earthquake load s considered for analysis. Details of the frame are shown in was Table 2 and its plan view is shown in Fig. 2. The front elevation with member designation is given in Figs. 3 and 4.

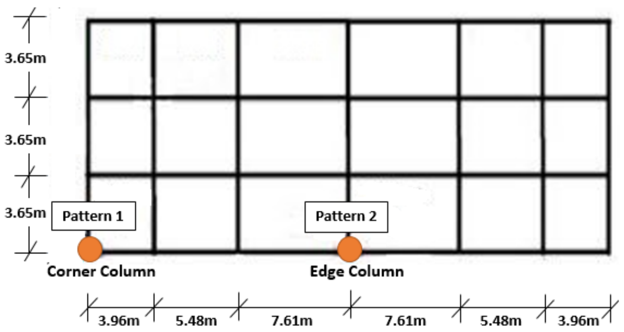


Fig. 2: The Plan of the building

Table 2: Details of selected frame

Sr. No.	Features	Details
1	RC frame	Nine stories
2	Spans	Longer direction – 6 bays Shorter direction – 3 bays
3	Story height	3.3 m
4	Beams	B1 - B9 = 457 x 406 mm B46 – B54 = 457 x 406 mm B10 – B18 = 457 x 457 mm B37 – B45 = 457 x 457 mm B19 – B36 = 635 x 457 mm
5	Columns	C1 – C18 = 457 x 406 mm C46 – C63 = 457 x 406 mm C19 – C45 = 533 x 533 mm
6	Masonary walls	115 mm thick
7	f_c'	27.4 MPa
8	f_y	413.6 MPa

5. Analysis Procedure for Progressive Collapse Potential

GSA guidelines were followed to carry out Linear Static Analysis. The model building was analyzed using a commercial software SAP2000. Damage patterns considered for the progressive collapse analysis are 1) pattern1- damaged corner column, 2) pattern2 - damaged edge column. GSA guidelines provide a threat independent procedure to lessen progressive collapse of the structures. The GSA guidelines criteria are established after performance based design concepts and allow both Linear

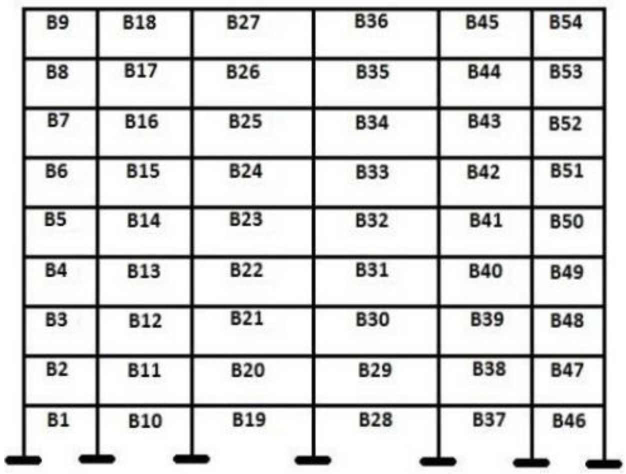


Fig. 3: Front Elevation with member designation of beams

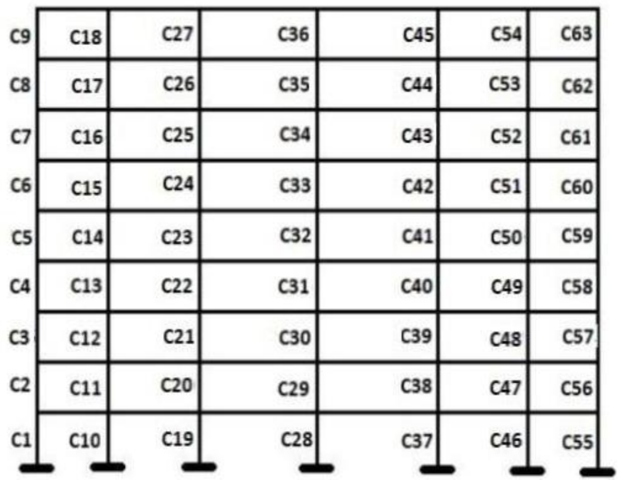


Fig. 4: Front elevation with member designation of columns

and Nonlinear Analysis Procedures. It also gives us load criteria of dead load and live load for Linear / Nonlinear Analysis. The DCR values of members whether beam or column was calculated to conclude progressive collapse potential. When analyzing the structure for progressive collapse potential, GSA guideline mentions a loading factor to be used for every structural member. GSA loading factor is *2 (Dead Load + 0.25 Live Load)*. DCR values provide the basis to analyze which structural members will exceed their loading capacity which may lead to progressive collapse. Using the Linear Static Analysis, the DCR values were found by dividing the demand by capacity, where, Demand equals the moment demand calculated using bending moment diagram in linear static analysis and Capacity equals the Nominal moment capacity. Demand Capacity Ratio (DCR) was used for acceptance criteria for Linear Static Analysis for progressive collapse. If DCR values exceed these criteria then the structure is at risk. Moreover, Non-Linear Static Analysis was also carried out to assess the potential of progressive collapse. The results of Linear as well as Non-Linear Analysis were compared to assess the accuracy of progressive collapse potential.

6. Discussion of Results

The model building was analyzed and its progressive collapse potential evaluated on parameters described in succeeding paragraphs.

6.1 Linear Static Analysis based on GSA Guidelines

A Linear Static Analysis based on GSA guidelines was carried out on commercial software SAP2000. In order to assess the vulnerability of progressive collapse, the DCR ratios were calculated in each structural member for different levels of damages 40%, 80% and fully damaged cases. DCR values for each damage level is discussed in succeeding paragraphs. The DCR values exceeding the acceptable limit are shown with dots in the succeeding figures.

6.1.1 40% column damage level

Figs. 4a and 4b show the computed values of DCR for 40% damaged corner and edge columns respectively. The maximum value of DCR for both the damage patterns are 1.7 and 1.8 respectively. Some beams on the upper part of the frame are critical as compared to lower portion of the frame due to increase in demand after collapse of columns. However, in both damage patterns, the adjacent frame members are subjected to higher progressive collapse potential as their DCR values are exceeding the acceptable limits provided by GSA guidelines. It clearly indicates that the collapse potential is localized and may not lead to complete collapse of building for 40% damage level but the failure in both the patterns is in the middle upper portion of the building. For 40% damage level, pattern 2 – edge column damage is more critical and progressive collapse potential is higher than pattern1 – corner column damage due to larger span length in case of edge column damage pattern.

6.1.2 80% Column damage level

Eighty percent corner damage column case is shown in Fig. 5a. In this damage case, the upper part of the frame is more affected as compared to the lower part of the frame as already observed for 40% damage case. The maximum value of DCR goes up to 1.7 and at many other locations the DCR values are less than 1.5. The beams which are far from damaged column are safe. Fig. 5b shows the 80% damaged edge column, and in this case the maximum value of DCR obtained is 2.1. The damage is present on the upper portion of the frame and is greater as compared with 40% damage level. For 80% damage level, pattern2 – edge column damage is more critical and progressive collapse potential is higher than pattern1 due to larger span length in case of edge column damage pattern.

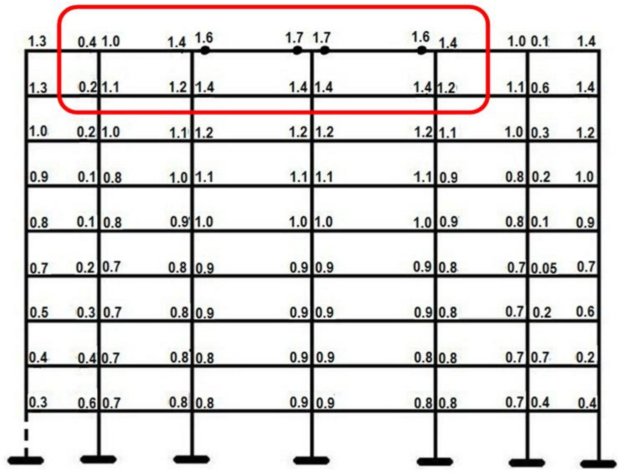


Fig. 4a: DCR values for pattern1 – 40% damage level

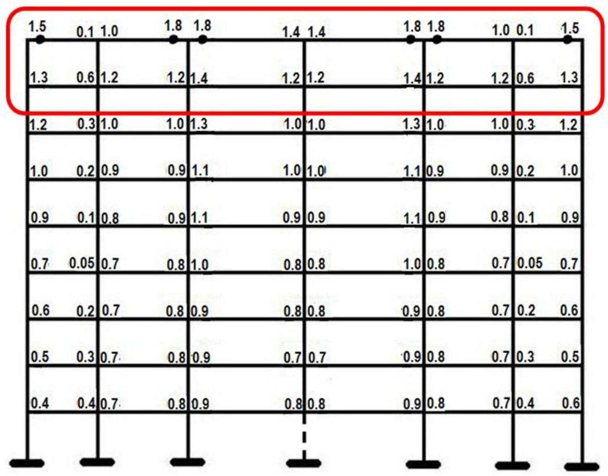


Fig. 4b: DCR values for pattern 2 – 40% damage level

6.1.3 Column full damage level

Fig. 6a shows the DCR values for pattern 1 case with fully damaged corner column. DCR values are greater in the adjacent part of the frame which is at higher risk to progressive collapse. The beam at the top story experienced DCR value of 6.7 which indicates that occurrence of damage is very high in the above portion of the frame. Fig. 6b represents the edge column fully damaged scenario pattern2. It is noticed that longer spans cause more damage than smaller spans. Smaller span bays are helpful in distributing the load on other members in comparison to large span bays. In the first phase members having DCR values greater than 1.5 are going to fail. Afterwards, member having DCR values less than 1.5 tend to fail in the next phase resulting in total structural collapse. The GSA acceptance criterion is exceeded and damage is greater in the central part and maximum value of DCR is 3.1.

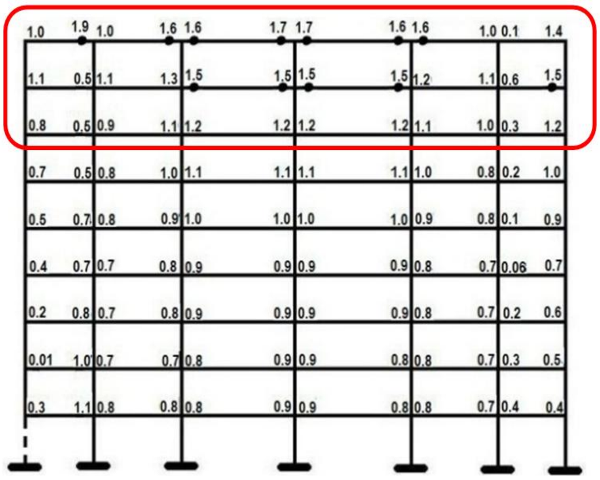


Fig. 5a: DCR values for pattern1 – 80% damage level

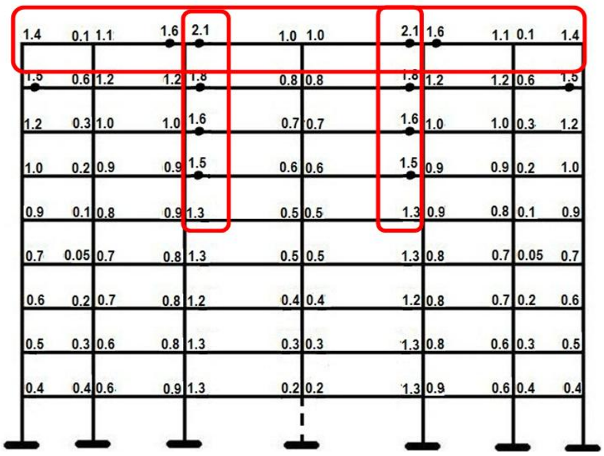


Fig. 5b: DCR values for pattern2 – 80% damage level

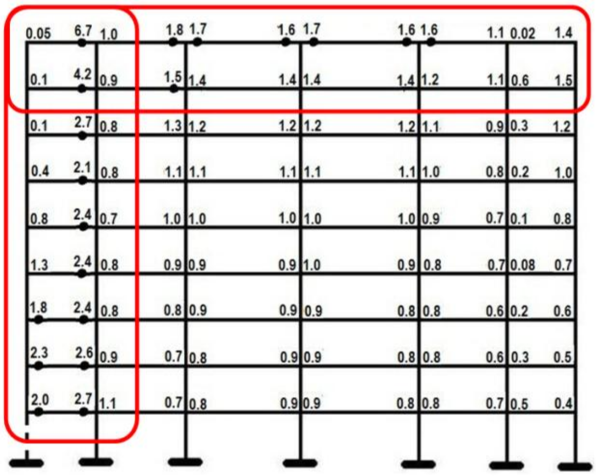


Fig. 6a: DCR values for pattern1 – fully damage level

6.2 DCR Values for Edge Column Damage Scenarios (Side Elevation)

Fig. 7 shows the DCR values of side elevation of structure 40%, 80% and fully damaged column consideration at the edge. It is clearly seen from Fig. 7a that there is no damage present in the frame. All beams are in safe condition and remain elastic without severe damage. No value of DCR has exceeded the GSA criteria of 1.5. In Fig. 7b, it is noticed that beams with DCR values less than 1.5 are mostly located in lower part of the frame. Top most beams have DCR values equal to 1.6 which can be dangerous. In Fig. 7c, the panel with damaged column is more critical and has higher collapse potential. The panel which is near to the damaged column also has more damage and DCR values exceed 1.5. This means that the structural condition is very dangerous in edge column damage case because the effect of large span beams in the front elevation is shifted to the side elevation. The corner damage case is considered critical for Progressive Collapse.

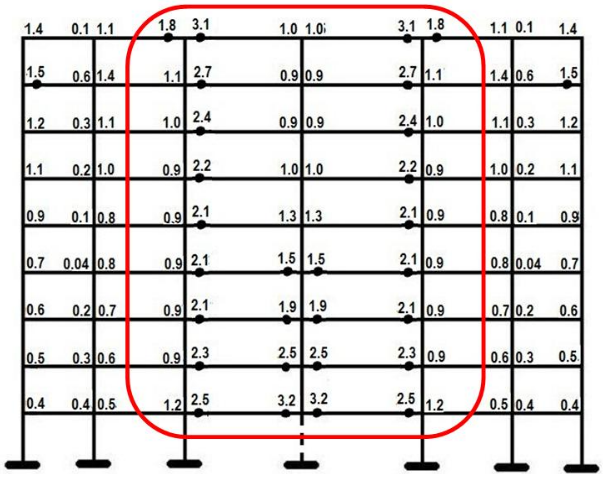
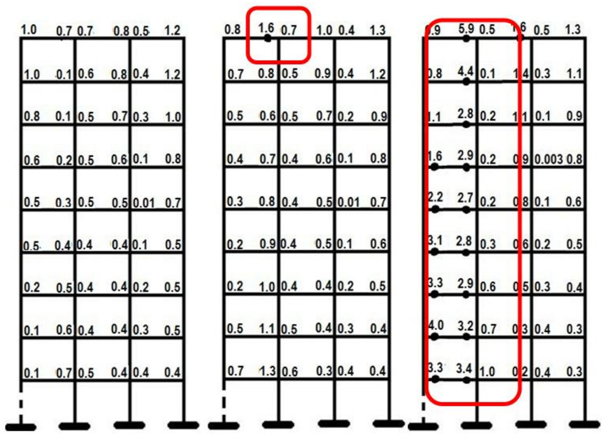


Fig. 6b: DCR values for pattern2 – Full damage level



(a) (b) (c)

Fig. 7: Side view of edge column damage case

This study shows that edge column damaged case is more critical than corner column as noticed in Fig. 7 which can be due to the longer span beams in the front elevation. Therefore, damage in shorter spans is less as compared with the longer spans. The progressive collapse phenomenon is very critical if longer span beams exist in the frame structure.

6.3 Variation in Bending Moment in Progressive Collapse

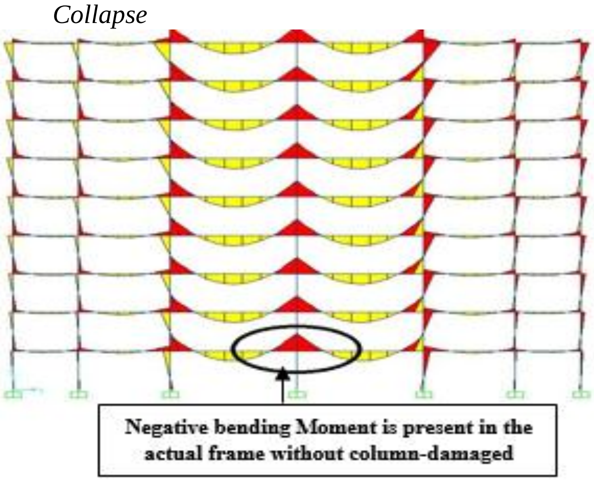


Fig. 8a: Moment variation of original frame without damage for edge column case

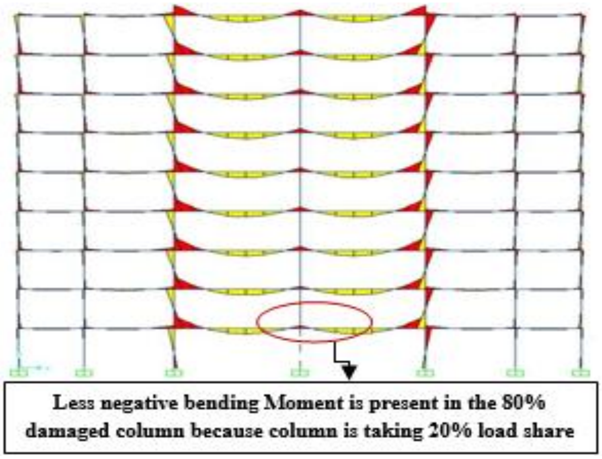


Fig. 8b: Moment variation of 80% Damaged frame for edge column case

Fig. 8a shows the original frame without damage for the edge column case. The bending moment in the encircled portion is negative near the support indicating the contribution of column and uniform distribution of moment in columns and beams. In 80% damage case, (encircled portion of Fig. 8b), the bending moment magnitude is negative and very weak at the damaged joint due to 80% column damage and column is taking 20% load share. When column is considered fully damaged as shown in Fig. 8c, (encircled portion), a large increase in positive bending moment occurs at this place because there is no support underneath. This moment reversal near the column is due to removal of column and is making the beam undergo flexural failure. The reason for this increase is that there is no column underneath and two large span beams are encountered by this fully damaged column. Stress reversal state is present under the damaged joint and joints above that. Reversal of stresses and enhancement in stresses are the two key phenomena that are formed by damages in columns.

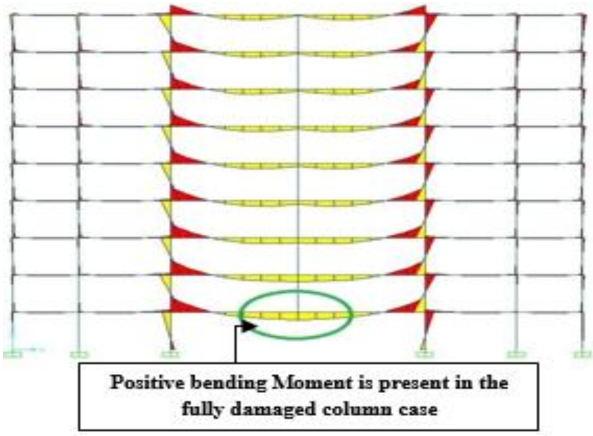


Fig. 8c: Moment variation of fully damaged frame for edge column case

6.4 Damage Index

Damage index is the summation of all the DCR's values greater than 1.5 of beams taken under consideration in which various damage levels are considered. Damage index indicates the collapse potential of the structures. Damage index of corner column with 40%, and 80% damage cases are 6.6, and 19.2 respectively and similarly in edge column case it increases to 10.2, and 20.2 respectively. But in fully damaged case it can be observed that the edge column is more critical than corner column as the damage index value achieved for edge column is 67.8 and for the corner column is 45.8 as shown in Fig. 9. The higher value of damage index in edge column shows that the longer span bay causes more damage than smaller span bay. Moreover, the edge column case has higher progressive collapse potential as compared to the corner column case. Damage index values can help us identify the critical scenario in progressive collapse analysis of the structures.

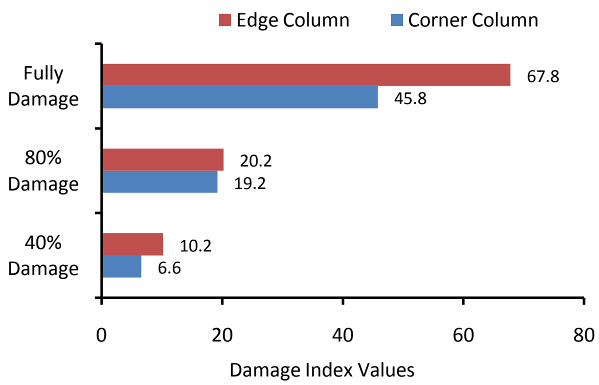


Fig. 9: Damage index values for all the column damage cases

6.5 Non-Linear Static Analysis

Significant research is carried out by researchers which show that the Linear Static Analysis method for Progressive Collapse is very conservative because of many assumptions used to simplify this analysis. The method of Linear Static Analysis is very easy and simple for equal span structures and it is difficult in case of complex and huge structures. It becomes more cumbersome as manual calculation of DCR values are involved in this type of analysis whereas, in Non-Linear Static Analysis the damage of frame is explained by the formation of hinges. In this study, interaction hinges at both ends of column were considered and flexural hinges were considered for the beams.

6.5.1 40% damaged level

Figs. 10a and 10b show the plastic hinge formation in columns and beams on front elevation for 40% corner and edge column damage cases. It is observed from the figure that very few hinges are developed at the upper part of the building and are at their initial stages. No hinge is fully developed because the structure is in immediate occupancy (IO) limit state, so the collapse potential is low. Moreover in case of Linear Static Analysis the collapse potential is quite high as compared to Non-Linear Analysis.

6.5.2 80% damaged level

Figs. 11a and 11b show the plastic hinge formation in front elevation of 80% corner and edge damaged columns. It is observed that hinges have developed in the upper stories but are still in immediate occupancy (IO) limit state. However, hinges have developed at the same location where the DCR values were either equal or greater than 1.5 in Linear Static Analysis. In Linear Static Analysis, demands of number of structural elements have exceeded the available capacities but Non-Linear Static Analysis reveals that the damage potential is still not very significant and is quite low as compared to results obtained from Linear Static Analysis. It is observed that the deflection at corner and edge column damaged joint as observed in Linear Static Analysis is 6.1 mm and 11.5 mm respectively but slightly less deflection is achieved in corner column NSA which is 6.01 mm and edge column with full load factor 2 (D.L+0.25 L.L) which is 11.2 mm.

6.5.3 Fully damaged level

Figs. 12a and 12b show plastic hinge formation in the fully damaged corner / edge column case. Here, all hinges are within IO limit state. The span in which column is fully damaged indicates that the panel is much more stressed after hinge development and the same is observed in Linear Static Analysis case. Deflection at the damaged joint is 18.02 mm which is slightly greater as corner column fully damaged case in Linear Static case. Large deflections with corner column damage cases often make the frame more critical. To overcome this greater deflection the load factor 2 which is multiplied with the dead load and live load $2(D.L + 0.25\ L.L)$ decreases or increases to achieve the same deflection.

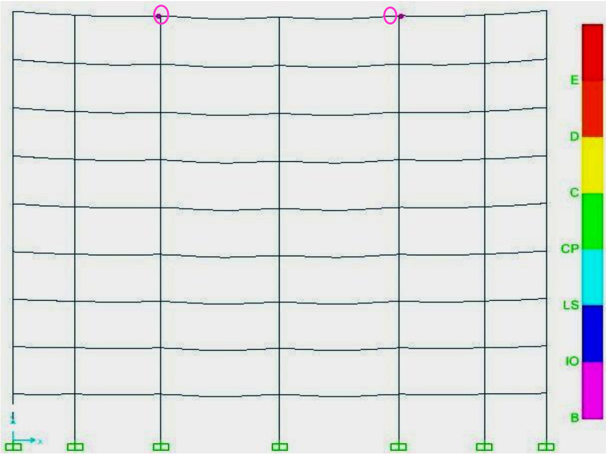


Fig. 10a: NLS of pattern 1 – 40% damaged

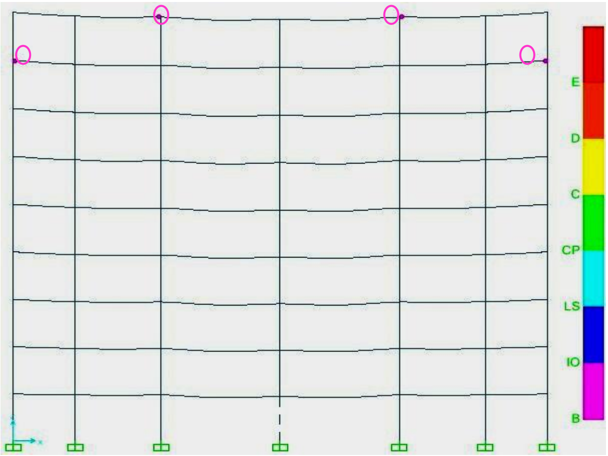


Fig. 10b: NLS of pattern 2 – 40% damaged

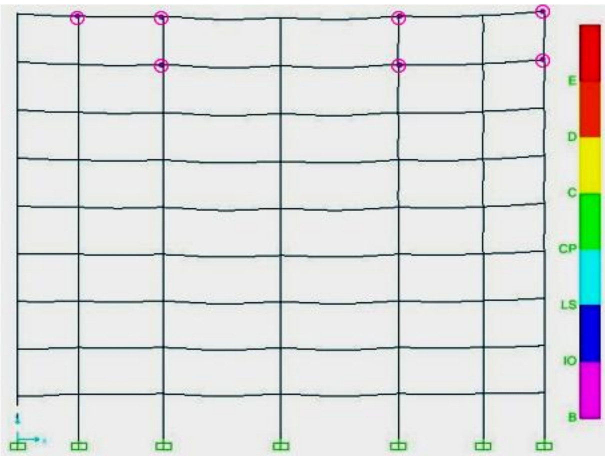


Fig. 11a: NLS of pattern 1 – 80% damaged

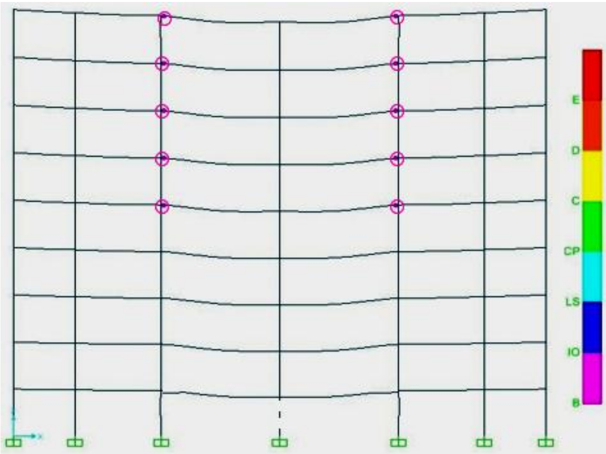


Fig. 11b: NLS of pattern 2 – 80% damaged

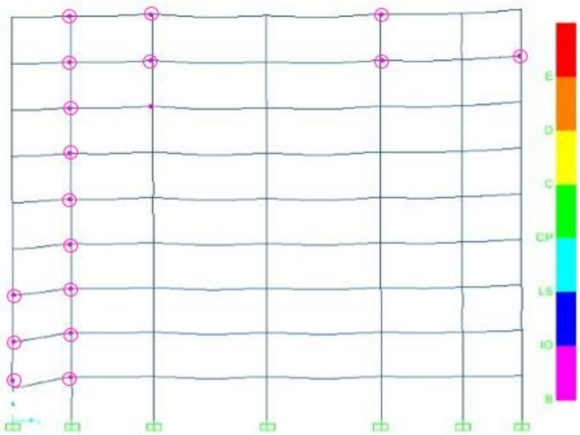


Fig. 12a: NLS of pattern 1 – fully damaged

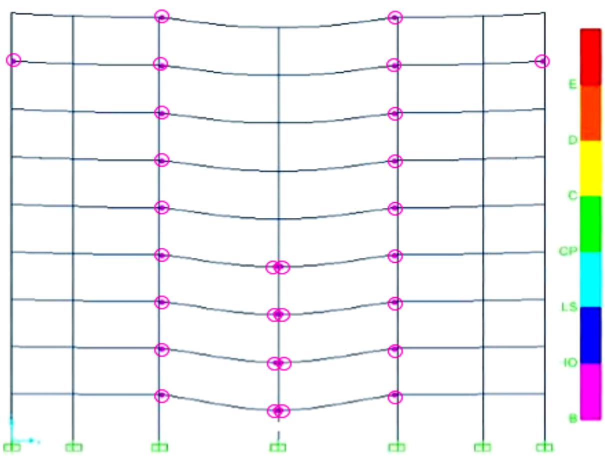


Fig. 12b: NLS of pattern 2– fully damaged

6.6 *Evaluation of Load Multiplier in NLSA for Fully Column Damaged case*

In Nonlinear Static Analysis, GSA's specified load {α(DL+0.25LL)}, the alpha factor of load increases or decreases step by step to attain similar deflection under column fully damaged joint as seen in Linear Static Analysis so that dynamic amplification factor (DAF) is achieved. Then compare the results with linear static analysis. Load combinations of column fully damaged case are given in Table 3. The average factor will be used later for calculation of dynamic amplification factor.

Table 3: Load multiplier in NLSA for fully column damaged case

Cases	Target deflection (mm)	Load combination of GSA (2003) Load {α(DL+0.25LL)}
1	16.3	1.94
2	30.6	1.9
Average		1.92

6.7 *Non-linear Dynamic Analysis*

Non-linear Dynamic Analysis was carried out by considering fully damaged cases. Figs. 13 and 14 show the plastic hinge formation at the corner and edge column cases. It is clear that hinges are at the initial level of formation. The hinges in these cases are not fully developed. So these cases have low potential for progressive collapse. Figs. 15 and 16 show the displacement of damaged joint Vs. time with 5% damping selected for reinforced concrete structures. The time history function is clear to achieve its peak amplitude at time equal to 0.13 sec in corner column case and 0.15 sec in edge column case. At 5% damping state, the maximum multiplier factor for dead load and live load which is applied to obtain the same deflection as observed in linear static analysis is 1.6 for corner column damaged case and 1.31 for edge column case. Maximum deflections achieved in Nonlinear Dynamic Analysis are 16.3 mm for corner case and 30.6 mm for edge case in comparison with Linear Static Analysis. In the start, peak deflection is achieved and afterwords the trend of amplitude of vibrations are gradually reduced because of 5% damping.

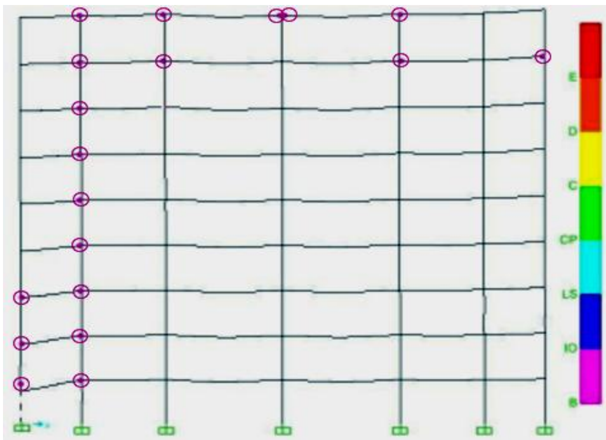


Fig. 13: NLDA Plastic Hinge formation (Corner column fully damaged case)

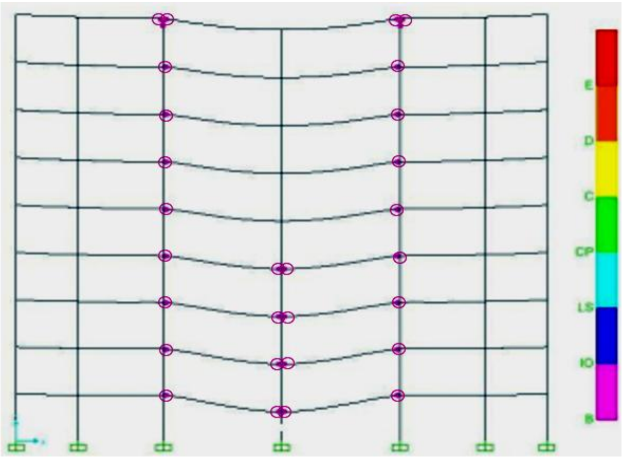


Fig. 14: NLDA Plastic Hinge formation (Edge column fully damaged case)

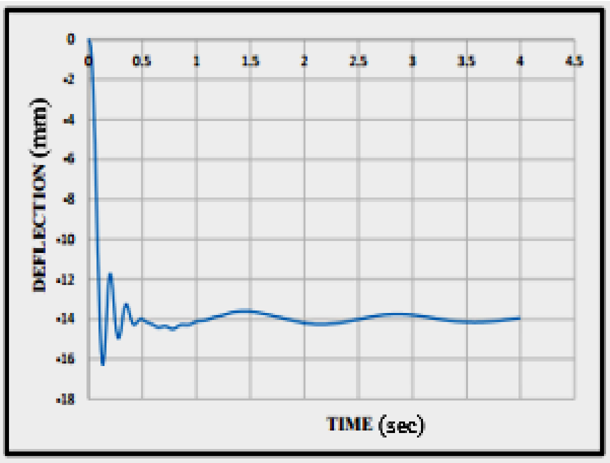


Fig. 15: NLDA Deflection of damaged joint Vs. Time Plot (Corner column fully damaged case)

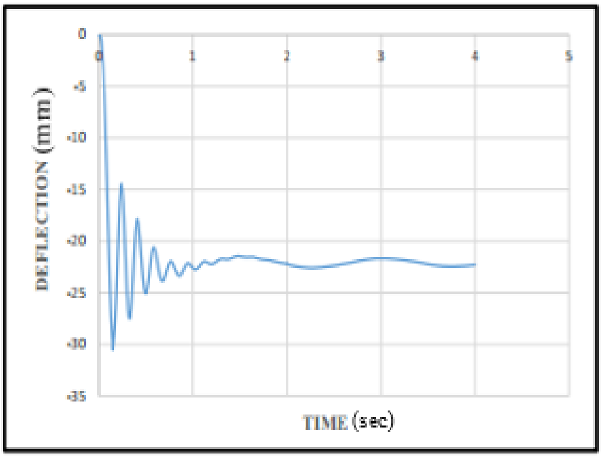


Fig. 16: NLD Analysis: Deflection of damaged joint Vs. time plot (Edge column fully damaged case)

6.8 Dynamic Amplification Factor in Nonlinear Dynamic Analysis for Fully Column Damaged Cases:

GSA Guidelines present Dynamic amplification factor of 2.0 which is multiplied to the linear static load case. Table 4 shows the load combinations of GSA guideline i.e. {α (DL+0.25LL)} is required to obtain the same linear static deflection under column fully damaged joint in Nonlinear Dynamic Analysis. In load combination of GSA, 5% damping is selected for dynamic amplification factor. Damping ratio was assumed to be 5% of the critical damping, which is usually adopted for analysis of structures undergoing large deformation [14].

Table 4: Dynamic amplification factor

Cases	Target deflection (mm)	Load combination of GSA (2003) Load {α(DL+0.25LL)}
1	16.3	1.37
2	30.6	1.0
Average		1.185

The average factor comes out to be 1.185 in Non-linear Dynamic Analysis. The factor achieved in Non-linear Static Analysis is 1.92 (Table 4). So

$$DAF = \frac{1.92}{1.185} = 1.62$$

This Dynamic amplification factor is less than GSA identified factor of 2.0. Tsai and Lin [15] also studied the dynamic amplification factor and also concluded that multiplying factor 2 in linear static load case is somewhat higher and conservative.

6.9 Deflection at Various Damage Levels

Fig. 17 shows the column chart for corner column and edge column damage scenario. The values in this column chart are drawn considering vertical deflection verses damage levels. It is observed from the figure that in 40% damaged case the deflection at corner and edge is 2.7 mm & 5.1 mm respectively. As far as 80% damaged case is concerned the deflection at corner is 6.1 mm and at edge it is 11.5 mm. On comparison the load share taken by 40% damaged column is 60% and similarly it is 20% for 80% damaged column which is evident by increase in deflection. However when column is fully damaged, there is abrupt increase in deflection i.e. 16.3 mm at the corner and 30.6 mm at the edge. The reason for this abrupt increase is that the column cannot bear any load due to complete damage.

7. Conclusions

The conclusions drawn from progressive collapse of RC frame under various level of damage scenarios are as follows:

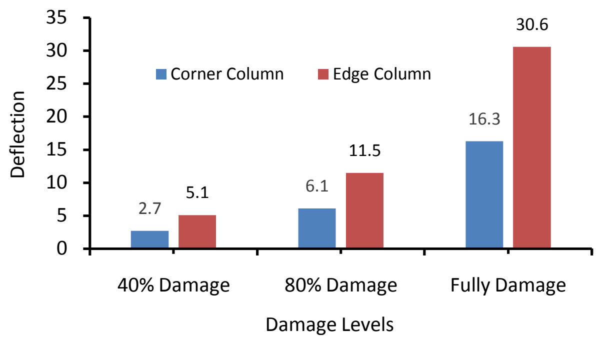


Fig. 17: Deflection Vs. various level of damage

1. Linear Static Method given in GSA Guidelines gives relatively conservative results as compared to Non-Linear Analysis. For this type of structural configuration, the dynamic amplification factor shows that the multiplying factor of 2 taken in Linear Static load case is somewhat higher and conservative. The factor achieved is 19% less than the original factor in Nonlinear Analysis and this factor needs some modification. Non-linear Analysis is found necessary to perform only when more precise and realistic results are required.
2. Reinforced concrete frame has low potential for progressive collapse when partial damage of column is considered and it has high potential when full damage column scenario is considered in Linear Static Analysis. In Nonlinear Static and Dynamic Analysis the potential for progressive collapse does not exist because the formation of hinges in beams and columns are at their initial level.
3. Edge column case with long bays are found critical in the event of progressive collapse because the bays with longer span experience more damage than smaller span as it is seen in Linear Static Analysis. This indicates that the building can fall in short interval of time and there is more possibility of loss of lives in such type of buildings.
4. The area adjacent to the column damage attracts more load and consequently experiences more damage due to different bay sizes the demand exceeds the available capacities.

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