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PROGRESSIVE COLLAPSE ANALYSIS OF SEISMICALLY DESIGN LOW RISE STEEL FRAME STRUCTURE

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ABSTRACT

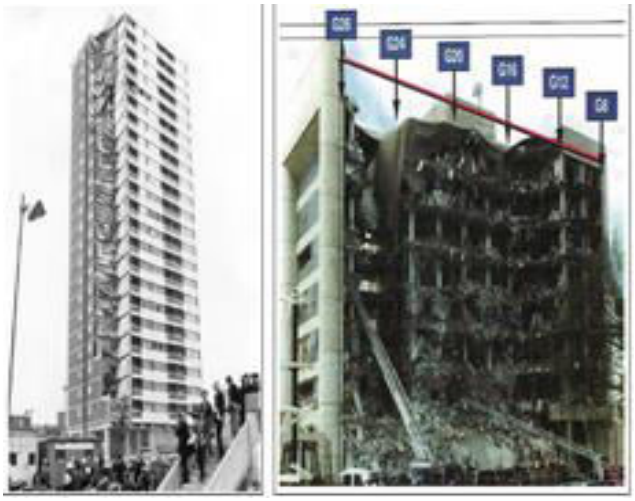
The remarkable partial collapse of the Ronan point apartment tower in 1968initiatedan intellectual discussion among the engineering community on the possible ways to design buildings against such catastrophic progressive types of failure. There are, in general, three alternative approaches to designing structures to reduce their susceptibility to disproportionate collapse such as redundancy or alternate load paths, local resistance and interconnection or continuity from these one or more approaches should be used to avoid progressive collapse. For this study, we consider, simple, low rise three-dimensional steel frame is to be prepared using ETABS 2015 software and are designed according to the Indian Standard Codes, for all load combinations. This model is to be an analysis for progressive collapse analysis as per latest GSA guideline 2013. The main objective of this study is to determine and understand the critical locations of columns in three-dimensional steel frames, which will cause the structure to undergo progressive collapse or maximum damage. The location of column removal largely affects the joint displacement and deformation behaviour. Nodal displacement of joint changes abruptly, which indicates that beam-column junction becomes critical. Out of two location corner column removal is more critical as compare to Edge middle location due larger cantilever effect and less connecting members present to transfer extra load. Sudden increased in Shear force and Bending Moment values indicate increased the strength of Beam to avoid the progressive collapse in a structure. The alternative path method would be one of the best remedies or precaution to overcome the progressive collapses apart from the other methods mentioned by various researchers in the past.

Keywords: progressive collapse, steel frame, static analysis, remarkable partial collapse, alternate load paths, GSA guideline 2013.

INTRODUCTION

Progressive collapse has been one of the issues in building failures since the collapse of the Ronan Point apartment building in 1968 (Griffiths, *et al*. 1968). Progressive collapse is a failure sequence that relates local damage to large scale collapse in a structure. The local failure can be defined as a loss of the load carrying capacity of one or more structural components that are part of the whole structural system. Preferably, once any structural component fails, the structure should enable an alternative load-carrying path. After the load is redistributed through a structure, each structural component will support different loads. If any load exceeds the load-carrying capacity of any member, it will cause another local failure.

Such sequential failures can propagate through the structure. If a structure loses too many members, it may lead to partial or total collapse. This type of collapse behaviour may occur in framed structures, such as buildings (Griffiths, *et al*. 1968, Burnett, *et al.* 1973, Ger, *et al*. 1993, Sucuoglu, *et al*. 1994, Ellis and Currie 1998, Bazant and Zhou 2002), trusses (Murtha-Smith 1988, Blandford 1997), and bridges (Ghali and Tadros 1997, Abeysinghe 2002). On the morning of 16 May 1968, Mrs. Ivy Hodge, a tenant on the 18th floor of the 22-story Ronan Point apartment tower in Newham, east London, struck a match in her kitchen. The match set off a gas explosion that knocked out load-bearing precast concrete panels near the corner of the building. The loss of support at the 18th floor caused the floors above to collapse. The impact of these collapsing floors set off a chain reaction of collapses all the way to the ground. The ultimate result can be seen in Figure 1(a): the corner bay of the building has collapsed from top to bottom. The Murrah Federal Office Building in Oklahoma City was destroyed by a bomb on 19 April 1995. The bomb, in a truck at the base of the building, destroyed or badly damaged three columns. Loss of support from these columns led to failure of a transfer girder. Failure of the transfer girder caused the collapse of columns supported by the girder and floor areas supported by those columns. The result was the general collapse evident in Figure-1(b).



(a) (b)

Figure-1. (a) Ronan point building after 16 May 1968 collapse. (b). Murrah Federal Office Building after 19 April 1995 attack.

Each of the twin towers of World Trade Centre 1 and 2 collapsed on 11 September 2001 following this sequence of events: A Boeing 767 jetliner crashed into the tower at high speed; the crash caused structural damage at and near the point of impact and also set off an intense fire within the building the structure near the impact zone lost its ability to support the load above it as a result of some combination of impact damage and fire damage; the structure above collapsed, having lost its support; the weight and impact of the collapsing upper part of the tower caused a progression of failures extending downward all the way to the ground.

LITERATURE REVIEW

After the collapse of Ronan Point building, Ferahian (1972) reviewed the changes that were made in the British and Canadian codes to prevent progressive collapse. The author argued that it could be possible for a building designed for earthquakes to resist progressive collapse after a loss of a load carrying component. It was also recommended that ductility and continuity between the structural elements/joints for structural integrity should be provided to enhance the toughness of the structure. In another study, McGuire (1975) concluded that alternate path method and specific local resistance should not be used as the main methods for preventing progressive collapse. The author recommended that codes should provide adequate guidelines to reduce the risk of progressive collapse to within acceptable limits.

Lewicki *et al.* (1974) advocated that although the problem of progressive collapse was more critical for large panel type structures, there was a potential for progressive collapse in other structural systems also. They asserted that although it was possible to design a building to resist progressive collapse after loss of local load carrying capacity, it was not economically practicable to prevent local failure from occurring due to the uncertainties present in the loading environment and the strength of the structure. They also concluded that, with the available knowledge, it was not possible to define the size of a local failure that the building should resist economically. The need for further theoretical and experimental work was stressed for safe and economical structures by limiting the probability of progressive collapse.

Leyendecker *et al*. (1977) proposed guidelines for preventing progressive collapse in buildings. The authors outlined three methods to prevent progressive collapse: 1) Event control method – in which abnormal loading on the structure is prevented by indirect measures; 2) Indirect design method - in which the structure is designed to have minimum strength, ductility and redundancy, and then assumed to perform adequately in the presence of local failures; 3) Direct design method – in which structural members are made adequately strong so that they can resist abnormal loading or the structure is designed so that it can tolerate local loss carrying capacity e.g. loss of a critical column. Ellingwood *et al*. (1978) further examined the design criteria to control progressive collapse and presented a probabilistic framework for their implementation in existing standards at that time. The main objective for such design criteria was to minimize the loss of life and to permit safe evacuation of occupant from the damaged structure.

The first study involving progressive collapse analysis of steel frames was presented by Gross et al. (1983). In this study, the behavior of 2-D moment resisting steel frames with the loss of one of the columns or increased load on the beams representing fallen debris was examined numerically. The nonlinear analysis program included the modelling of inelastic beam column behavior, beam to column connection behavior, and the effect of shear infill panels.

Both material and geometric nonlinear effects were taken into account in an updated Lagrangian formulation. The authors analysed a four-story, three-bay steel frame representing a low-rise apartment or a small office building, designed according to 1978 AISC specifications. Two cases were considered, one with the second story external column removed and other with the second story interior column removed. Pretlove (1986) studied the dynamic effects that occur in the progressive failure of a simple uniaxial tension structure and concluded that a structure that appears to be safe under static load redistribution may actually be unsafe if the transient dynamic effects are taken into account. Mlakar *et al.* (2003) presented the findings from a study of the 2001 Pentagon attack and gave their recommendations for the future design and research needs for collapse prevention. Astaneh-asl (2003) carried out an experimental investigation of the viability of steel cable based systems to prevent progressive collapse of buildings. The tests were conducted on a full-scale specimen of a one-story building. One side of the floor of the specimen had steel cables placed within the floor representing new construction and the other side had cables placed on the outside as a measure of retrofit of the existing building. The author claimed that the test results showed that the system could economically and efficiently prevent progressive collapse of the floor in the event of removal of one of the exterior columns.

Kaewkulchai *et al.* (2004) presented a beam element formulation and solution procedure for dynamic progressive collapse analysis of planar frame structures. To illustrate the importance of dynamic effects, static and dynamic analyses of two-bay, two-story frame were carried out. The analysis results showed a significant increase in nodal displacements, number of plastic hinges and plastic rotations when inertial effects were included. It was concluded that static analysis might not provide conservative estimates of the collapse potential of frame structures. Kim Jinkoo, Kim Taewan (2009) presented the progressive collapse-resisting capacity of steel moment resisting frames was investigated using alternate path methods recommended in the GSA and DoD guidelines. The linear static and nonlinear dynamic analysis procedures were carried out for comparison. It was observed that, compared with the linear analysis results, the nonlinear dynamic analysis provided larger structural responses and the results varied more significantly depending on the variables such as applied load, location of column removal, or number of building story. M. A.

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Hadianfard & M. Wassegh (2012) In this paper, the intermediate steel moment frames structures with different levels of height designed for moderate and very high level of seismic zones of Iran are studied. The results show that, generally, for the steel structures designed for higher seismicity there is higher capacity for progressive collapse and in the low height steel structures, there is not enough redundancy to redistribute loads of the failed elements so, the potential of progressive collapse increases with decreasing the height of the structure.

MODEL DESCRIPTION

There have been many numerical methods which have been developed in the past and currently updated continuously in simulating the actual behaviour for various conditions. Few of them have been outstandingly well in analysing and predicting actual behaviour of structures namely finite element method, applied element method, spectral element method and boundary element methods. In this paper, ETABS software is used to simulate the building with various conditions to understand the phenomenon of progressive collapse.

For this study, simple three-dimensional 6 bays 5 Storey steel structures is considered with span length of 5m and Storey height of 3m as shown in Figure-2. The sectional properties of column and beam are given in the Table-1.

Table-1. The loading information applied to structures.

Level	Dead load (KN/m^2)	Floor finish (KN/m^2)	Live load (KN/m^2)	Weight of wall (KN/m)
Floor level	3.75	1.25	2.5	10
Roof level	3.75	2	1.5	3

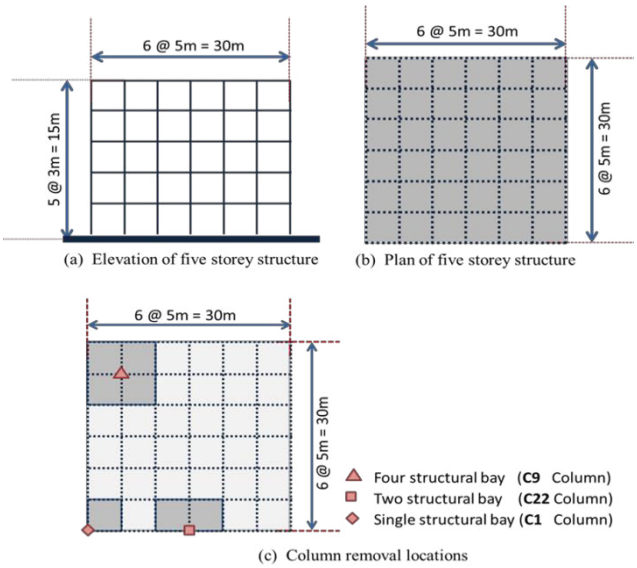

Figure-2. Illustration of 5 storey frame in elevation views (a), plan (b) and (c) column removal according to GSA, 2013.

METHODOLOGY

Even though there are several methods (Specific local resistance, alternate load path method, prescriptive design rules) for analysing a structure for progressive collapse analysis, alternate path method has gained lot of popularity in scientific and engineering community due to its simplicity in application, the same has been recommended by GSA (2003, 2013) and UFC (2005, 2013) guidelines. In alternate path method, key structural members (typically a single column) are removed, and the structure analysed to determine its capacity to span across or "bridge" across that "missing" member. For this study, GSA 2013 guide lines are used for progressive collapse behaviour simulations for the procedure Linear static.

Linear Static Analysis Procedure (LSA): In general, LSA is effective for the structures less than or equal to 10-stories. To calculate the force-controlled actions, simultaneously following combination of gravity loads are calculated using equation (1) and applied to those bays immediately adjacent to the removed element and at all floors above the removed element as shown in Figure-3

$$GLF = \Omega LF\ [1.2\ D + (0.5\ L \text{ or } 0.2\ S)] \quad (1)$$

Where GLF = Increased gravity loads for linear Static analysis ΩLF = Load increase factor for linear Static analysis = 2 Gravity loads for floor areas away from removed column or wall are calculated using equation (2).

$$G = 1.2\ D + (0.5\ L \text{ or } 0.2\ S) \quad (2)$$

Where G = Gravity loads, D = Dead loads, L = Live loads and S= Snow loads.

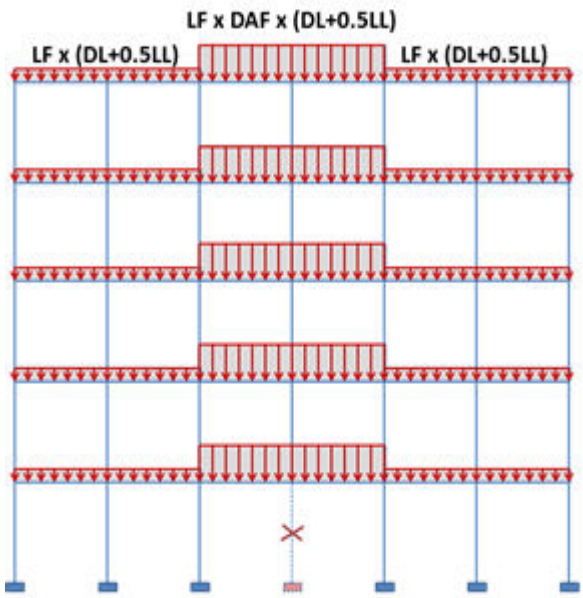

Figure-3. Illustration of Load Factor (LF) and Dynamic Amplification Factor (DAF) considered for removal of an interior column.

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ANALYSIS AND RESULTS

Two Iterations have been carried out for this study. Demand Capacity Ratio, Joint Displacement, Axial force in column, Shear Force, Bending Moments and Axial force in Beams are the comparative parameters before and after removal of column for a various position in Low-rise steel structure. In these cases, we have performed Progressive collapse analysis of Five Story Steel Frame using latest GSA-2013 guidelines. As per guidelines all central and corner columns are critical for progressive collapse analysis therefore in this study we included two cases for analysis. Following GSA, 2013 guidelines linear static analysis is carried out in several steps. Initially the structure has been analysed for the load combinations given by GSA 2013. Extra loads calculated from equation-1 are applied on a slab for corner column removal (C7), double slab on peripheral internal column removal (C4) Figure-4 shows the deformed shapes obtained for 5- storey structures for C4. The location of column removal largely affects the joint displacement and deformation behaviour.

Case-1: Central column (C4) removal at first floor

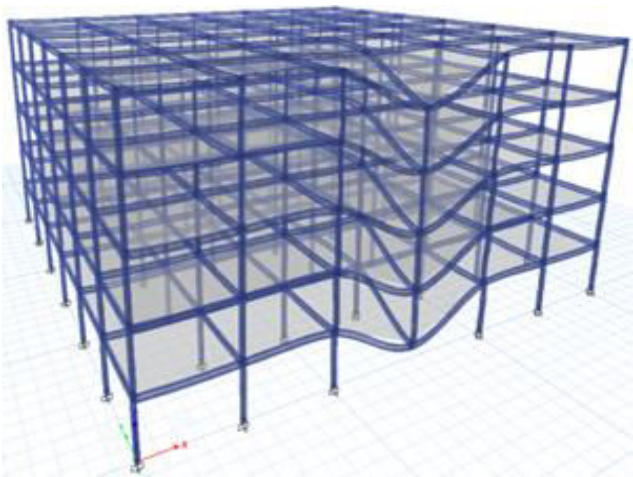
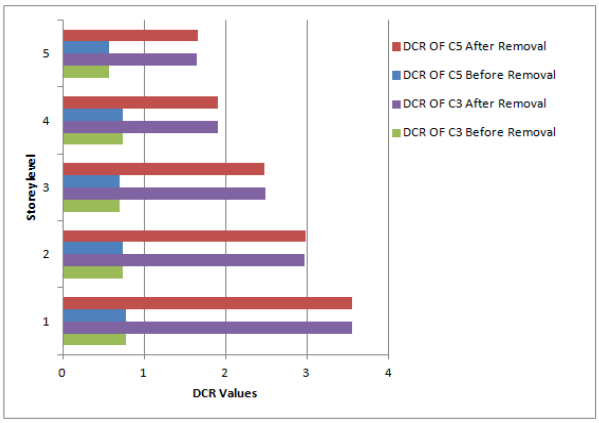
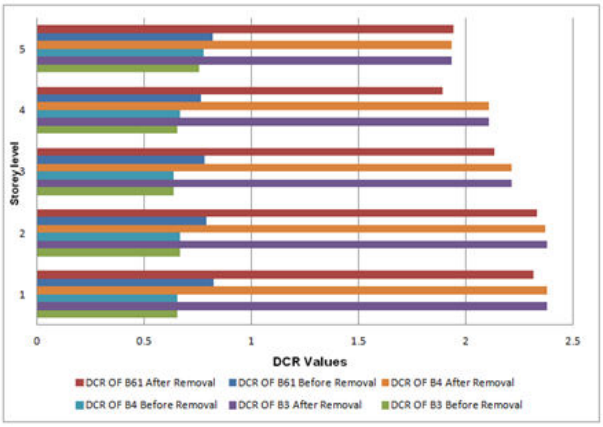


Figure-4. Elevation of after removal of column (C4).



(a)



(b)

Figure-5. Change in DCR for columns and beams after removal of column (C4).

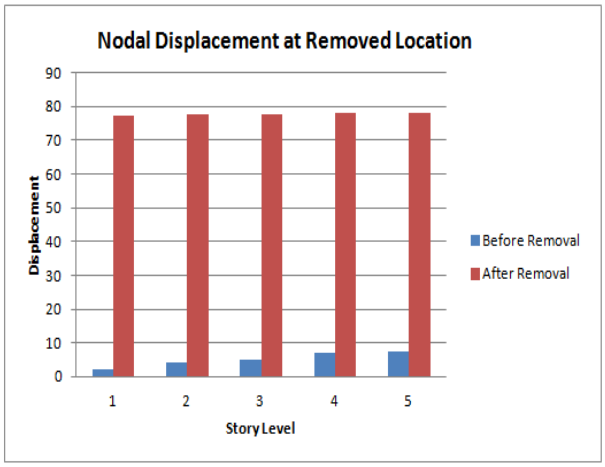


Figure-6. Nodal displacement of joint (C4) before and after removal of column.

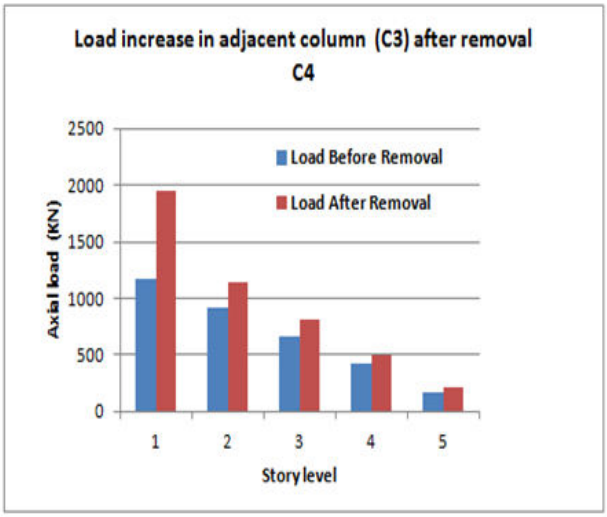


Figure-7. Axial force in column (C4) before and after removal.

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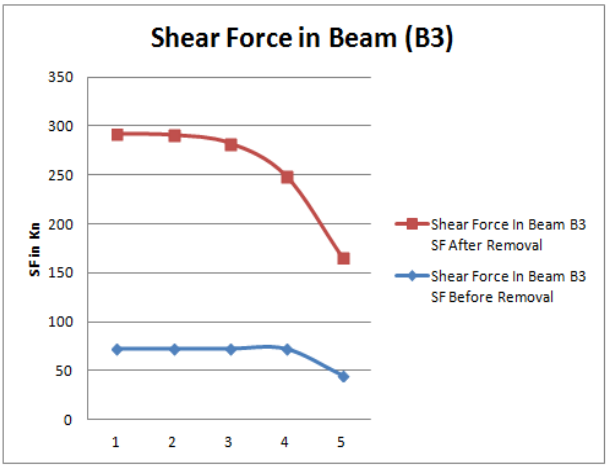


Figure-8. Change in shear force in beams (B3) after removal of column (C4).

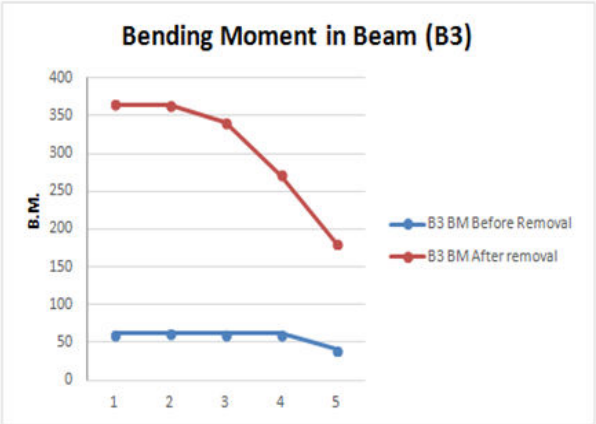


Figure-9. Change in shear force in beams (B3) after removal of column (C4).

Variation of DCR, Nodal displacement, axial force, Shear Force, Bending Moment values for respective case-1following observation seen. Figure 1(a) & 1(b) shows abrupt change in DCR values of nearby location of Columns and Beam respectively. Which shows nearby columns and beams capacity suddenly increased more than 50% of before removal values.The DCR of column is largely increasing second storey indicate the partial collapse of that vertical load bearing element. The load transfer elements on each storey are failing due column removal.Figure-6 shows drastic change in Nodal Displacement at removed Location. This indicates localised failure of member from first story to last story. Figure-7 shows axial load on nearby column after sudden removal of central column (C4) was largely increased.Figure- 8& Figure- 9 shows sudden variation of values of shear force and bending moment in Beam (B3) & similarly for Beam (B4).Progressive collapse seen in this case is larger in adjacent bays and storey of structures.

Case-2 Corner column (C7) removal at first floor :

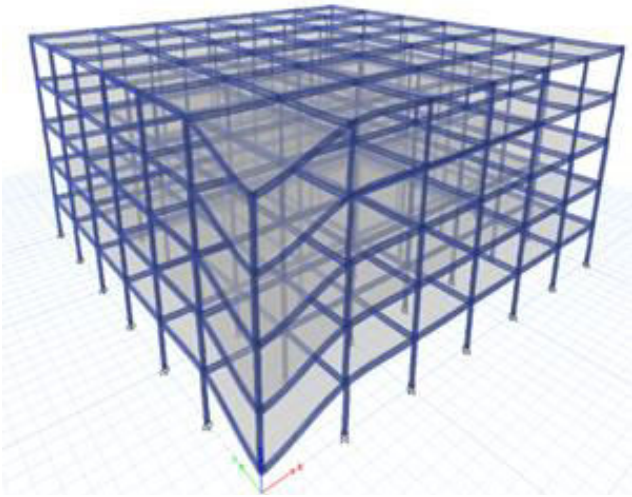
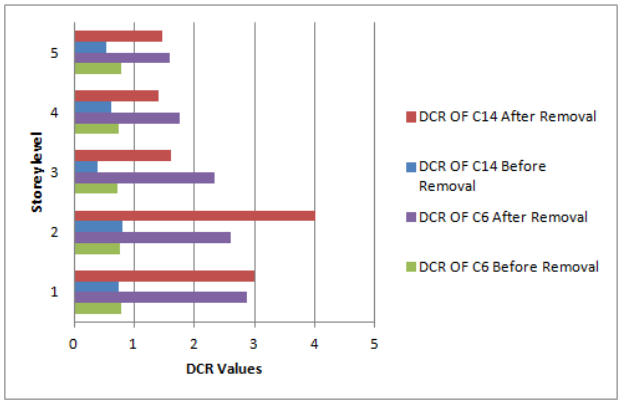
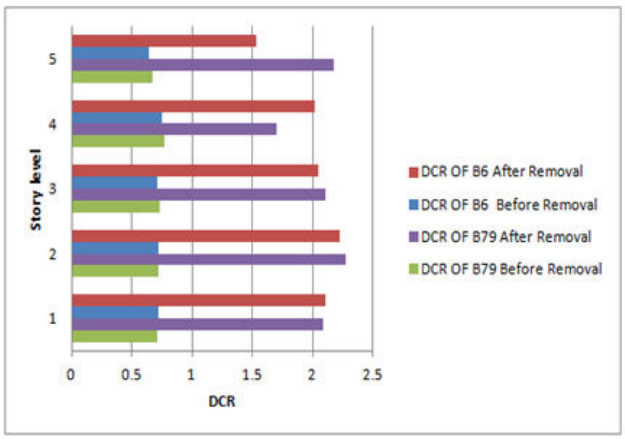


Figure-10. Elevation of before and after removal of column (C7).



(a)



(b)

Figure-11. Change in DCR for columns and Beams after removal of column (C7).

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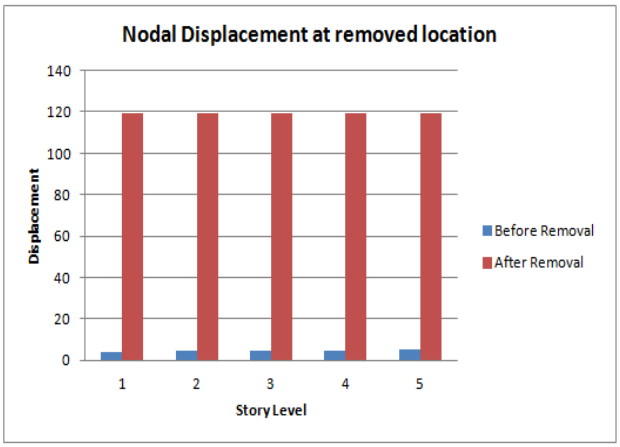

Figure-12. Nodal displacement of joint (C7) before and after removal of column.

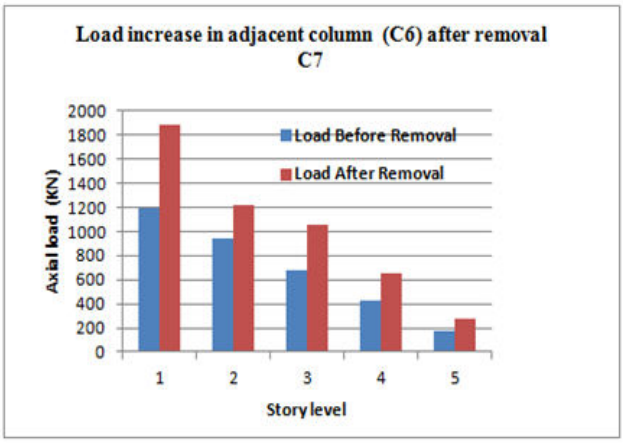

Figure-13. Axial force in column (C7) before and after removal.

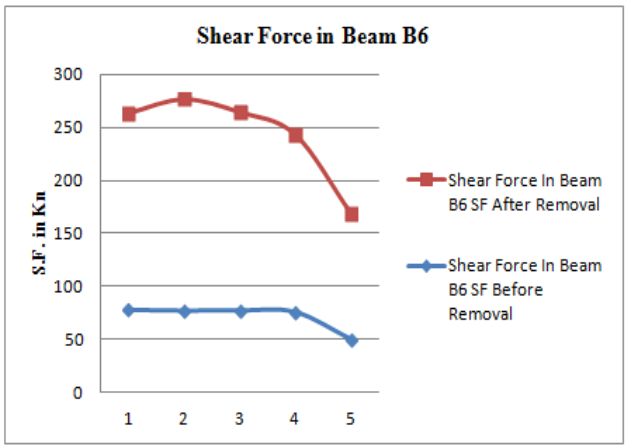

Figure-14. Change in shear force in beams (B6) after removal of column (C7).

Figure 10 shows deformation shape of 5-storey structure for corner column(C7) removal. Variation of DCR, Nodal displacement, axial force, Shear Force, bending Moment values for respective case- 2 Figure-11 to Figure-15.

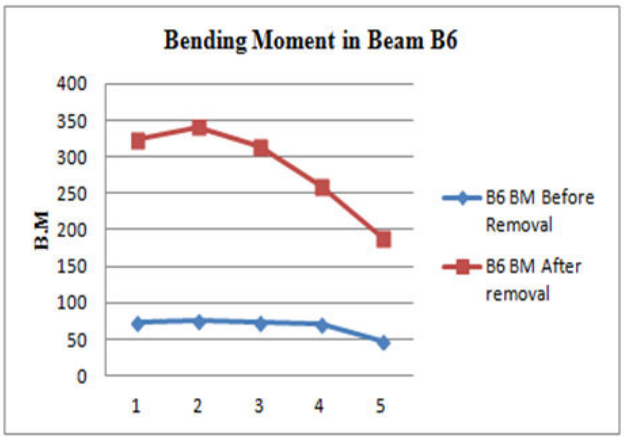

Figure-15. Change in shear force in beams (B6) after removal of column (C7).

Figure-11(a) & 11(b) shows abrupt change in DCR values of nearby location of Columns and Beam respectively. Figure-12 shows drastic change in Nodal Displacement at removed Location. This indicates localised failure of member from first story to last story. Figure-13 shows axial load on nearby column after sudden removal of corner column (C7) was largely increased. Figure-14 and Figure-15 shows sudden variation of values of shear force and bending moment in Beam (B6) & similarly for Beam (B79). The results obtained prove that one of the most effect elements when any vertical elements removal are the just adjacent elements, that could be both column or beam, in fact sometime it is the beam column junctions. If this study is done with material non-linearity and more loads than a clear phenomenon of progressive collapse would be seen.

CONCLUSIONS

A simple modelling is done by considering a six bay by five storey to understand the fundamental behavior of progressive collapse when vertical elements are removed, which could occur due to various manmade or natural disasters. From this study, observations were, when an element is removed loads are distributed to its surrounding elements and almost effect will be seen on complete structure even though maximum effect is on immediate horizontal and vertical elements, which is what actually happens and defines a progressive collapse. Nodal displacement of joint changes abruptly which indicates that beam-column junction becomes critical. Comparing between C4 removal and C7 removal, the reason for more displacement in former case due to larger cantilever effect. Sudden increased in Shear force and Bending Moment values indicate increased the strength of Beam to avoid the progressive collapse in a structure. The location of column removal largely affects the joint displacement and deformation behaviour. Finally, even though is a very basic model simulation, but it gives in depth fundamental understanding about the progressive collapse. As all the graphs discussed show the increased in studied parameters in the member just immediate to the vertical element

removed. Surely, alternative path method would be one of the best remedies or precaution to overcome the progressive collapses apart from the other methods mentioned by various researchers in the past.

FURTHER STUDY

The fundamental progressive behavior is understood using is a simple three-dimensional portal frame. Further extensions of this research work are currently under progress which includes similar portal frame with different height of frame structures including different Braced system. The progressive collapse analysis is a dynamic event so analytical method such as Non-linear static, linear dynamic and Non-linear dynamic are consider.

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