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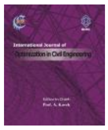

# PROGRESSIVE COLLAPSE ANALYSIS OF RCC STRUCTURES

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## ABSTRACT

The study aims to investigate the progressive collapse behaviour of RCC building under extreme loading events such as gas explosion in kitchen, terroristic attack, vehicular collisions and accidental overloads. The behavioural changes have been investigated and node displacements are computed when the building is subjected to sudden collapse of the load bearing elements. Herein, a RCC building designed based on Indian standard code of practice is considered. The investigation is carried out using commercially available software. The node displacement values are found under the column removal conditions and collapse resistance of building frame is studied due to increased loading for different scenarios. This simple analysis can be used to quickly analyse the structures for different failure conditions and then optimize it for various threat scenarios.

**Keywords:** progressive collapse; RCC building frame; moment; force.

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## 1. INTRODUCTION

Awareness on the issue of progressive collapse took place after the structural failure of Ronan point in 1968 [1]. After the terrorist attack on Murrah federal office building in 1995, more and more research efforts were put to understand the progressive collapse [2]. This was further concreted with the several terrorist attacks around the world. But is important to note that collapse of the World Trade Centre (commonly known as 9/11) has led to the detailed investigations for the enhancement of robustness of structures in order to save precious loss of life and property under such attacks [1].

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As per ASCE progressive collapse is defined " The spread of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as disproportionate collapse" [2]. The General Services Administration, USA adopt the basic definition of that "Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse" [3]. Department of defense (DoD) offers another definition as "A progressive collapse is a chain reaction of failure of building members to an extent disproportionate to the original localized damage" [4]. Progressive collapse is deformation of any load bearing element which initiate the local failure and transfer of additional load progression to the adjoining elements to generate disproportionate collapse [5]. An increasing number of progressive collapse around the world lead more disastrous event leading to loss of life, injuries and large number of death and not dealt with common codal provision to address the progressive collapse in conventional design. Considering this an important issue, United States Department of Defense (DOD) and United States General Services Administration (GSA), and Euro codes published a string of various guidelines and specifications [7]. Two design approaches were recommended for design of new and existing building against the progressive collapse as: direct approach and indirect approach. Further, four levels of protection were recommended for the building according to department of defense i.e. HLOP (High level of protection), MLOP (Moderate level of protection), LLOP (Low level of protection) and VLOP (Very low level of protection) to classify the severity of the collapse. Based on the analysis, it was suggested that alternate load path analysis is necessary to perform for building to have high and moderate level of protection (HLOP and MLOP) and secure the tie forces on buildings which have low and very low level of protection (LLOP and VLLOP) [8]. Alternate load path analysis is more adoptable because of its risk free approach and mainly focus on the performance of building after removal of critical support to ensure the safety of the building. There are four substitute analytical techniques drawn in alternate load path approach i.e. linear static analysis, nonlinear static analysis, linear dynamic and nonlinear dynamic analysis. In linear static analysis full factored load is applied on the damaged structure at once. The response of structure after removal the component of structure is dynamic and nonlinear; so dynamic effect is indirectly considered by taking the constant amplification factor. After the static analysis DCR (Demand capacity ratio) can be computed to determine the extent of damage zone. This method is inconvenient if structure elements and joints connection have the DCR value less than 2 i.e. the structure have possesses several cracks and damage in that case other method is suitable [9]. The advantage of this conservative method lie in its simplicity, fast to complete it and this method is application for the building with maximum of 10 floors. GSA. Nonlinear static analysis accounts for the nonlinearity of material and geometry, consist step by step iteration thus making this method time consuming. Herein, analysis is done based on load history from zero to full factored load applied on the structure and iterations are continued until the structure model gets stabilized whereas nonlinear dynamic analysis represent the nonlinearities of material and geometry and express the actual behavior of structure while undergoes inelastic deformation [10-11].

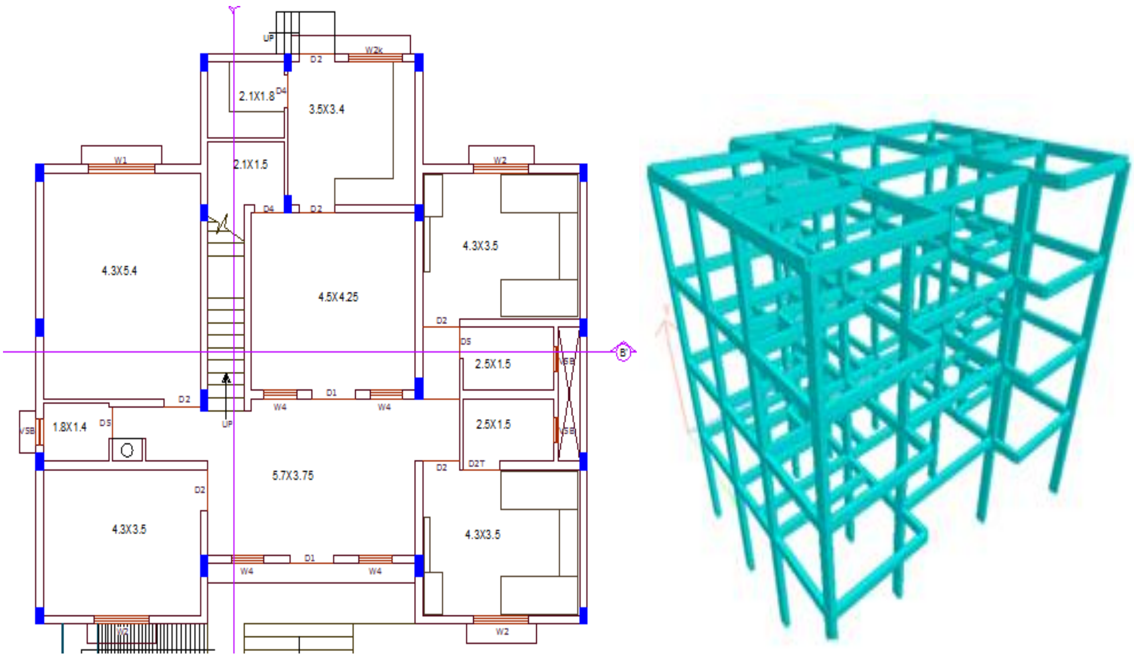
Fu [12] observed the load redistribution, increment of force in column to a peak with steady value and peak axial force versus increase in column force for all column removal cases and determined the amount of energy required to be absorbed by remaining building.

Fu [13] studied the 3D behavior of composite steel frame building under sudden column removal scenario and observed that lower the steel grade, larger the maximum vertical dynamic deflection and higher the steel grade, results in higher bending moment and axial force. Tsai and Huang [14] investigated linear and nonlinear analysis for the RC frame and observed that the exterior wall is a better opinion than the parapet type and panel type wall with a constant opening rate of 60%. Wang et al. [8] studied about the design analysis, method of reinforced concrete structure to resist progressive collapse which included conceptual design, tie strength design and removal of component design to provide overall stability and concluded that damage of local structural element do not lead to large scale collapse. Li et al. [7] found that the correction factor β, taking into account of nonlinear effect instead of linear elastic approach and present internal force correction β versus the deformation capacity. Salem et al. [15] found that the stresses were tensile for uppers bar and compressive for the lowers before removal of column the tensile stresses in upper bar started to decrease and become compressive, compressive stresses in lower bar decrease and changed to tensile due to downward deformation. They observed that additional reinforcement helps to prevent the progressive collapse which is used in above removed column. Tavakoli and Alashti [6] studied the buildings that have been designed according to seismic design specification are able enough to resist progressive collapse with damaged column in different location. Helmy et al. [16] examined that neither the increment in slab thickness nor increase its reinforcement helped to preventing the progressive collapse in case of an edge shear wall loss. Guo et al. [17] observed plastic hinge action and catenary action played an important role to carry the load rather than in preventing progressive collapse. They observed that structure is less influenced by the horizontal restraining stiffness before it begins to go into catenary stage. The catenary action would increase with an increment of horizontal restraining increment. Kaveh and Behnam (2013) studied optimal design of three dimensional multi-story reinforced concrete structures using meta-heuristic algorithms. Based on this investigation they concluded that meta-heuristic algorithms simplifies the optimization process [18].

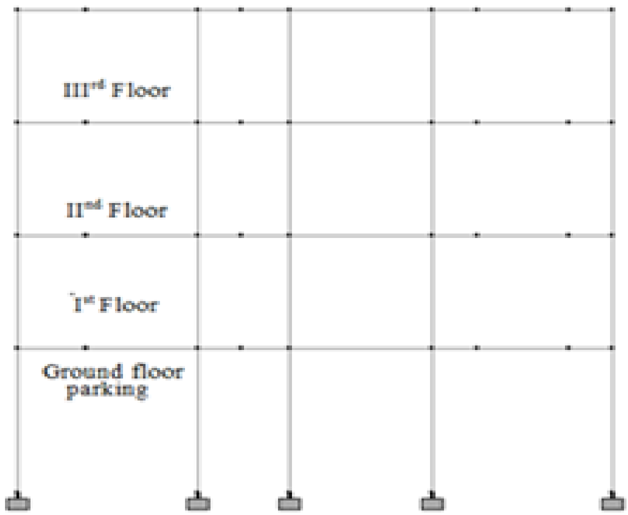
Based on literature review, it is found that there exist several investigations for progressive collapse analysis of structures but all of these are related to high rise buildings. Hence, in the present investigation, an attempt is made to understand the effect of column removal on a medium rise building which is a common scenario in most of the developing countries. Moreover, the objective is to study the effect on the building after adopting the column removal approach under extreme events and to understanding the behavior of building, so that engineer can easily adopt the suitable analysis approach and material after understanding the behavior of building under progressive collapse without much complex analysis as proposed by earlier researchers.

## 2. ANALYTICAL MODELING OF STRUCTURE

In the present investigation, a 4 story concrete frame building having 3×3 bays in longitudinal and transverse direction with same plan throughout the whole height is considered. Fig. 1 shows plan of the building which is symmetrical throughout the whole height and perspective view of concrete frame, the elevation and plan of building is symmetrical respectively along with their respective column and beam. The height of the storey is 3 m except for the ground floor which is 4.5 m high to be used to serve as parking space. For each building floor, size of column were kept constant for every story along the height and also size of beam were designed and kept constant for the whole height. The complete design of building is as per Indian code of practice and typical reinforcement of column and beam and the slab is shown in Fig. 2. Slab is 20 cm thick and building is designed according to the specifications of Indian code of practice for dead load with sunk load and live load condition. The building is designed for a live load of 3.5 kN/m$^2$. The value of $f_y$ as 415 x 10$^3$ kN/m$^2$ is considred in the analysis. The amount of reinforcement required for building component is obtained by structural design analysis software (Staad Pro 2006) as per Indian code of practices [19-21]. The plinth level diagram of the guest house with prescribed node and member is shown in Fig. 3. The load combination is assumed to be DL+LL, 1.5DL +1.5LL, DL+0.25LL. The concrete frame is designed to resist gravity loads and progressive collapse is considered in accordance with the Indian code of practice.

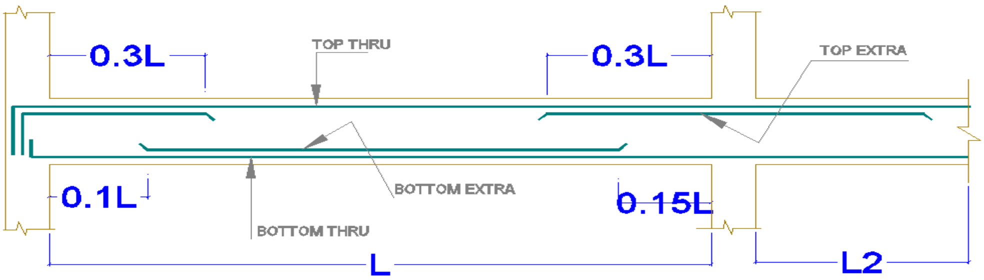



(a) (b)



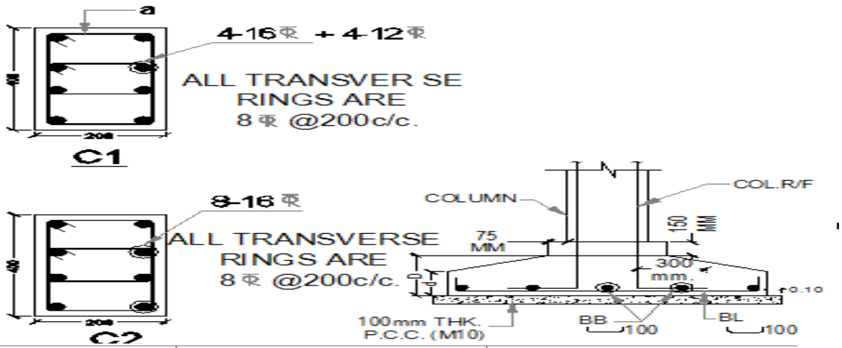

(c)

Figure 1. Model structure: (a) Plan of building; (b) Perspective view of analytical frame (c) Elevation diagram of analytical frame



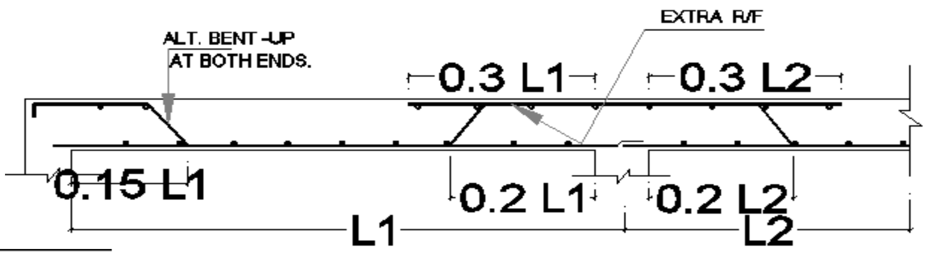

| BEAM MKD. | SIZE | REINFORCEMENT THRU BARS | | REINFORCEMENT EXTRA BARS | | STIRRUPS TWO LEGGED | |
|---|---|---|---|---|---|---|---|
| | | TOP THRU. | BOT THRU. | TOP EXTRA | BOT EXTRA | NEAR SUPPORT L/4 OF SPAN | REST OF SPAN |
| PB1 | 200 X 400 | 2 - 12φ | 2 - 12φ | 2- 10 φ | 1 - 10φ | 8φ@ 100 c/c | 8φ@ 200 c/c |
| PB2 | 200 X 400 | 2 - 12φ | 2 - 12φ | 2 - 12φ | 2 - 16φ | 8φ@ 100 c/c | 8φ@ 200 c/c |
| PB3 | 200 X 400 | 2 - 16φ | 2 - 16φ | 3- 16 φ | 2 - 16φ | 8φ@ 100 c/c | 8φ@ 200 c/c |
| PB4 | 200 X 400 | 2 - 12φ | 2 - 12φ | 2 - 16φ | 2 - 16φ | 8φ@ 100 c/c | 8φ@ 200 c/c |
| PB5 | 200 X 400 | 2 - 12φ | 2 - 12φ | - | 2 - 12φ | 8φ@ 100 c/c | 8φ@ 200 c/c |
| PB6 | 200 X 400 | 2 - 12φ | 2 - 12φ | - | - | 8φ@ 150 c/c | 8φ@ 150 c/c |
| PB7 | 200 X 400 | 3 - 16φ | 3 - 12φ | 3 - 16φ | - | 8φ@ 100 c/c | 8φ@ 100 c/c |
| PB8 | 200 X 400 | 2 - 12φ | 2 - 12φ | 2 - 16φ | 1 - 16φ | 8φ@ 100 c/c | 8φ@ 200 c/c |

(a)




| COL. MKD | SIZE (MM) | R/F |
|---|---|---|
| C1 | 200 X 400 | 4-16 ₹ +4-12 ₹ |
| C2 | 200 X 400 | 8-16 ₹ |

(b)




| SLAB MKD. | THK. (mm) | TYPE | R/F ALONG SHORTER SPAN | R/F ALONG LONGER SPAN | DIST.R/F ALONG LONGER SPAN | EXTRA R/F ALONG SHORTER SPAN | EXTRA R/F ALONG LONGER SPAN |
|---|---|---|---|---|---|---|---|
| S1 | 125 | ONE WAY | 10 ₹ @ 200c/c | — | 8 ₹ @ 200c/c | 10 ₹ @ 400c/c | — |
| S2 | 125 | TWO WAY | 10 ₹ @ 200c/c | 10 ₹ @ 200c/c | — | 10 ₹ @ 400c/c | 10 ₹ @ 400c/c |
| S3 | 125 | TWO WAY | 10 ₹ @ 175c/c | 10 ₹ @ 190c/c | — | 10 ₹ @ 350c/c | 10 ₹ @ 380c/c |

(c)

Figure 2. Typical reinforcement details (a) Typical reinforcement detail of beam (b) Typical reinforcement detail of column and (c) Typical reinforcement detail of slab

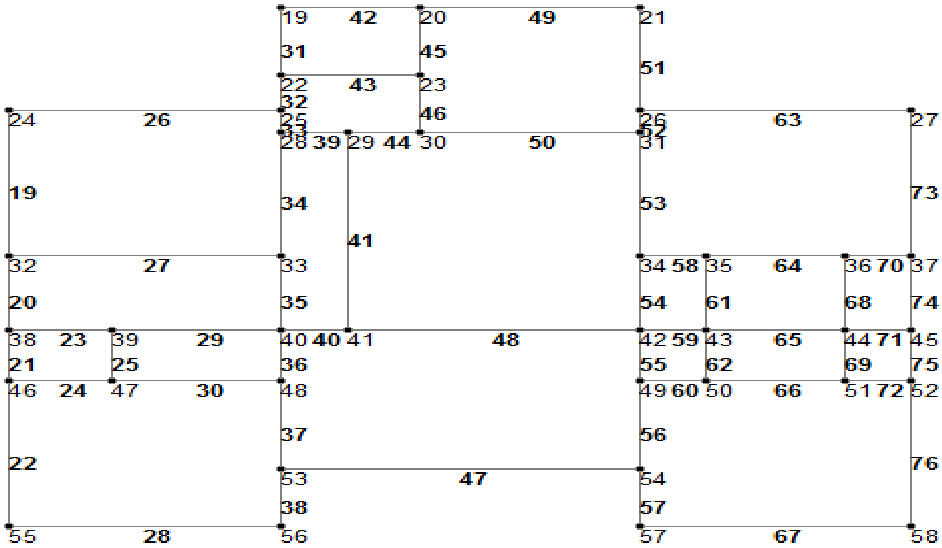


Figure 3. Plinth with node and member

## 3. ANALYSIS OF STRUCTURES UNDER COLUMN REMOVAL

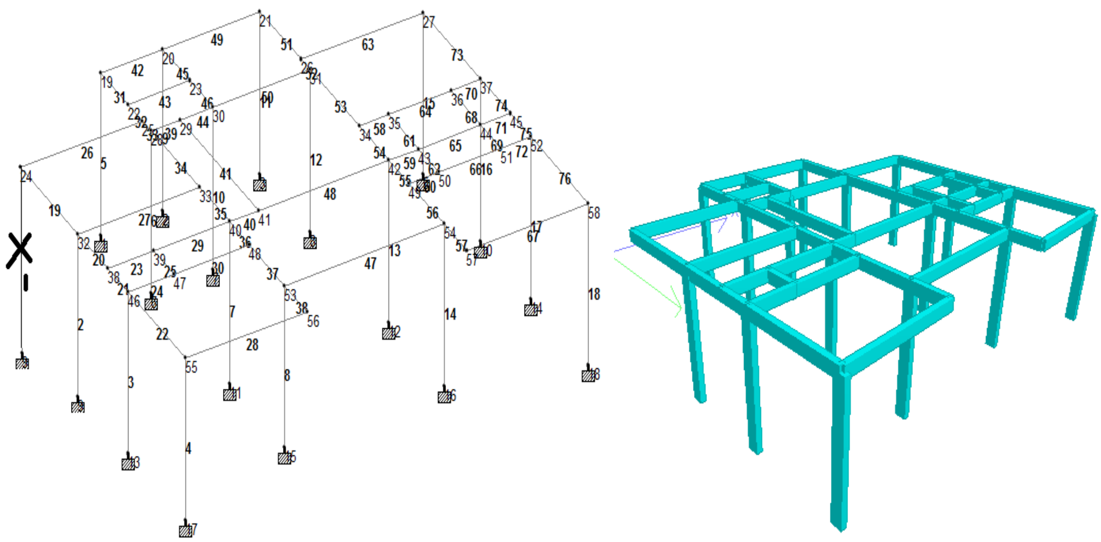
### *3.1 Scenario 1: one column removal*

In this case column at ground floor is suddenly removed and additional forces is conveyed to the surrounding member resulting eventually in the increase in bending moment and forces from base analysis wherein there was no removal of structural member. This resulted in redistribution of forces and the loads are transferred to nearby members. Considering this scenario, column number 1 is removed as shown in Fig. 4 (a) and it is observed that node on the top of removed column reaches displacement values as shown in Table 1. A large redistribution of forces is observed which took place due to removal of this single column. Fig. 5 (a) shows the increase in percentage of additional loading due to the accidental collapsing of structural member number 2 and results in transfer of load to adjacent member is about 60%. Fig. 5 (b) shows the approximately percentage increase of reaction at adjacent column number 5 due to removal of column number 18 and it is found to be 35%. The comparison between the forces in Y direction and bending moment in Z direction with all columns and the structure when one column is removed is reported in Table 3 for load combination (1.5 DL+1.5 LL) as per Indian code of practice.

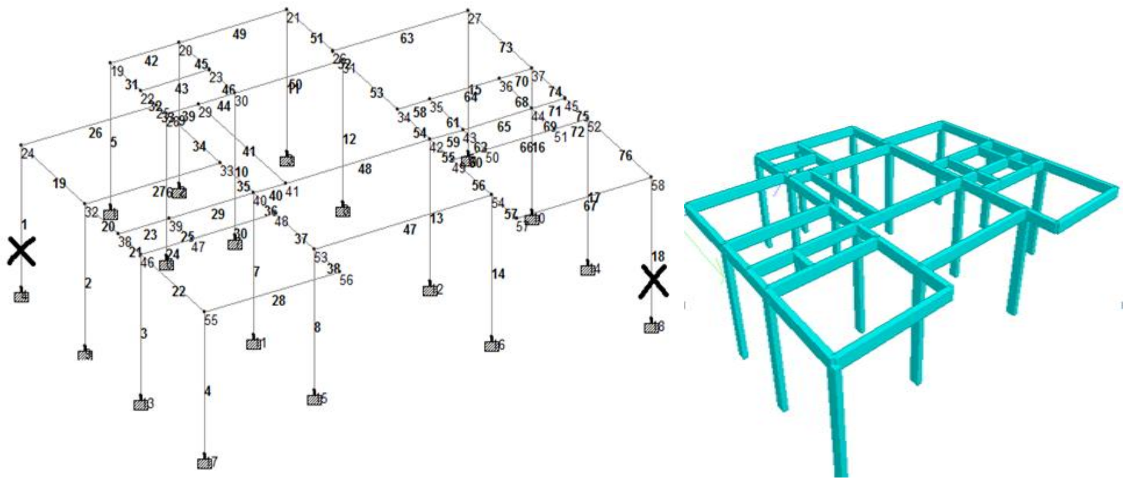
### *3.2 Scenario 2: two column removal*

Herein, column number 1 and column number 18 is removed as shown in Fig. 4 (c) and in Fig. 4 (d). Based on the analysis it is observed that node 58 on the top removed column reached node displacement values as shown in Table 4. A large redistribution of forces is observed. Fig. 5(c) shows increase in percentage of additional load in node 14 due to the accidental collapsing of structural member number 18 transferring the loading to adjacent member is 66%. Fig. 5 (d) show the percentage increase of reaction at adjacent column number 16 due to removal of column number 18 and it is found to be 12%. The comparison of shear forces and bending moment between the frame with no removal of member and after removal of two columns is reported in Table 4 for load combination (1.5 DL+1.5 LL) as per Indian code of practice.

Further based on the analysis Figs. and Tables, it is observed that a large difference occurs in forces and bending moment from the initial condition (when no column is removed). The reason may be attributed to the transfer of the instantaneously applied load to the remaining undamaged structure as well as joints. Moreover, it is observed that effect of transferring the load is more on the nearest member of the removed member and negligible when moved away from removed column. Also, it is found that the increment in the joint displacement of the neighbouring member of removed element gets approximately 27 times of an initial node displacement from the initial values after removal of one column and with an increment of approximately 30 times of initial value after removal of two columns due to large redistribution of forces. Further, shear force in Y direction ($F_Y$) at that member which is located just above the removed column generally give negative value at the point of zero shear forces in initial condition. Sometime positive value is also seen instead of zero value of shear forces in Y- direction after removal of columns. Also, it is observed that shear force in X direction ($F_X$) at that member which is located just above the removed column after removal of two columns give value in positive zone instead of negative zone initially before removal of any columns.



(a) (b)



(c) (d)

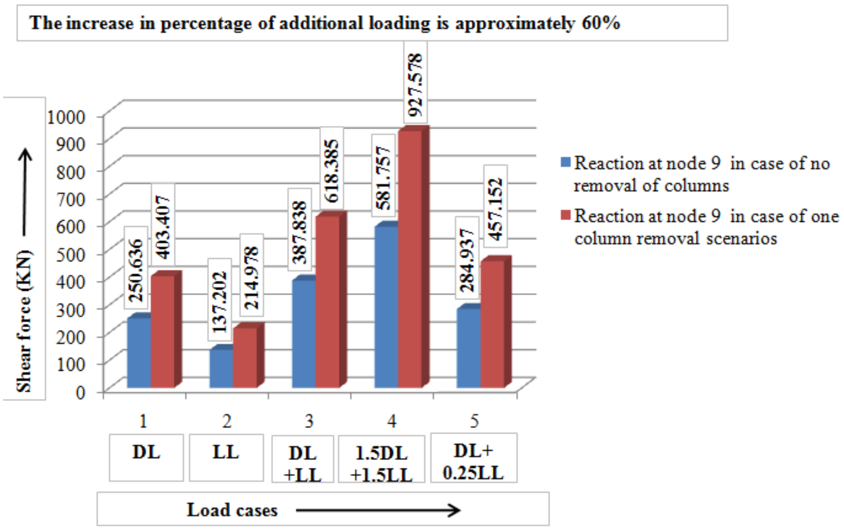
Figure 4. (a) One column removal scenario (b) Perspective view of one column removal scenarios (c) Two column removal scenario and (d) Perspective view of two column removal scenario

Table 1: Node displacement for no column removal and one column removal scenario

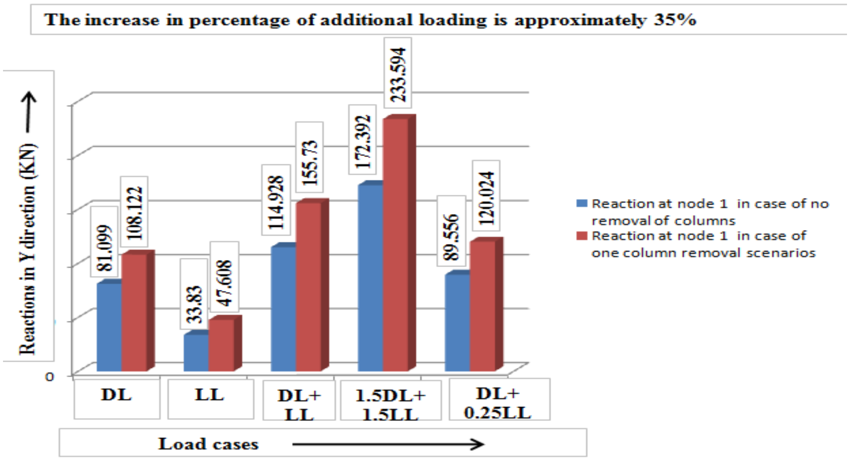
| **Node No.** | **Load cases** | **Node displacement** | |
|---|---|---|---|
| | | **At Y direction** | **At Y direction** |
| | | **No column removal** | **One column removal scenario** |
| 24 | 1 Load case 1 | -0.387 | -11.088 |
| | 2 Load case 2 | -0.192 | -5.635 |
| | 3 Combination load case 3 | -0.579 | -16.723 |
| | 4 Combination load case 4 | -0.869 | -25.085 |
| | 5 Combination load case 5 | -0.435 | -12.497 |

Table 2: Node displacement for no column removal and two column removal scenario

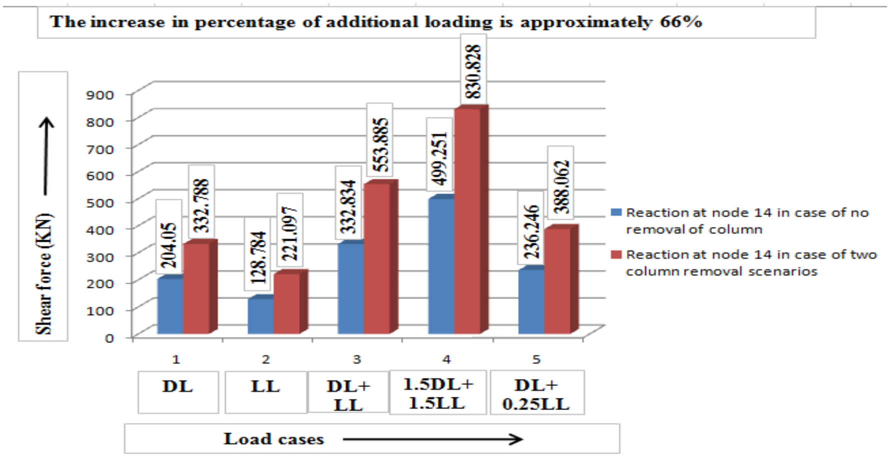
| **Node No.** | **Load cases** | **Node displacement** | |
|---|---|---|---|
| | | **At Y direction** | **At Y direction** |
| | | **No column removal** | **Two column removal scenarios** |
| 58 | 1 Load case 1 | -0.305 | -9.425 |
| | 2 Load case 2 | -0.209 | -6.72 |
| | 3 Combination load case 3 | -0.514 | -16.145 |
| | 4 Combination load case 4 | -0.771 | -24.217 |
| | 5 Combination load case 5 | -0.357 | -11.105 |


The increase in percentage of additional loading is approximately 60%
Shear force (KN)
1000
900
800
700
600
500
400
300
200
100
0
250.636
403.407
137.202
214.978
387.838
618.385
581.757
927.578
284.937
457.152
1
2
3
4
5
DL
LL
DL +LL
1.5DL +1.5LL
DL+ 0.25LL
Load cases
Reaction at node 9 in case of no removal of columns
Reaction at node 9 in case of one column removal scenarios

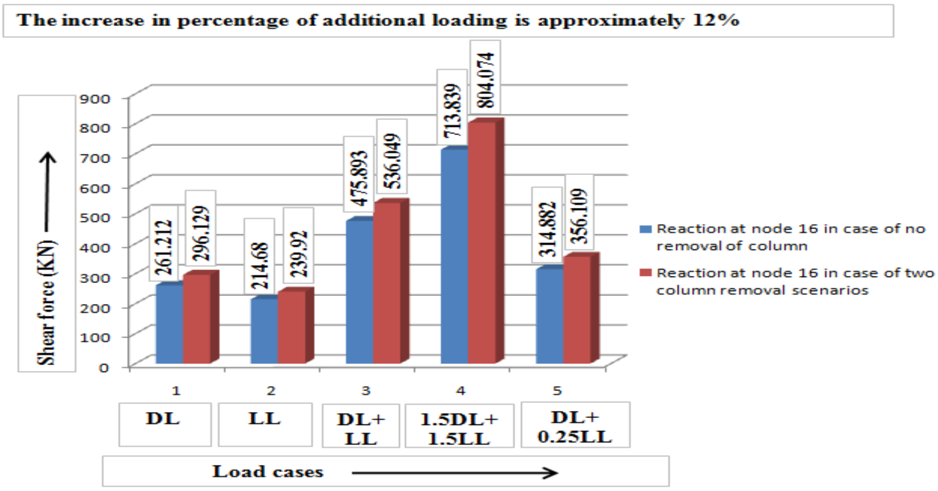
(a)


The increase in percentage of additional loading is approximately 35%
Reactions in Y direction (KN)
0
81.099
108.122
33.83
47.608
114.928
155.73
172.392
233.594
89.556
120.024
DL
LL
DL+ LL
1.5DL+ 1.5LL
DL+ 0.25LL
Load cases
Reaction at node 1 in case of no removal of columns
Reaction at node 1 in case of one column removal scenarios

(b)


The increase in percentage of additional loading is approximately 66%
Shear force (KN)
900
800
700
600
500
400
300
200
100
0
204.05
332.788
128.784
221.097
332.834
553.885
499.251
830.828
236.246
388.062
1
2
3
4
5
DL
LL
DL+ LL
1.5DL+ 1.5LL
DL+ 0.25LL
Load cases
Reaction at node 14 in case of no removal of column
Reaction at node 14 in case of two column removal scenarios

(c)

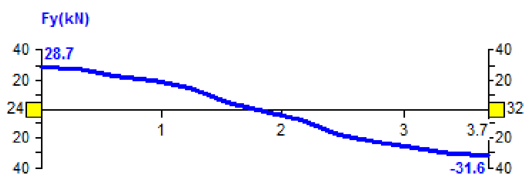
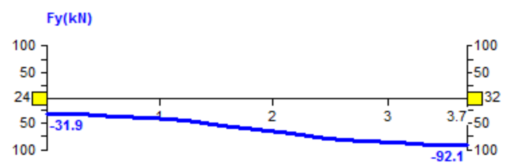
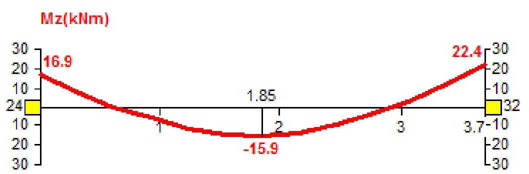
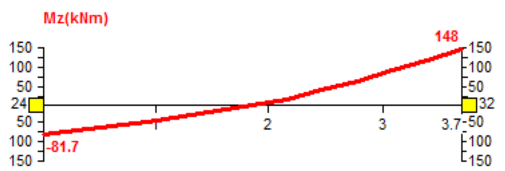
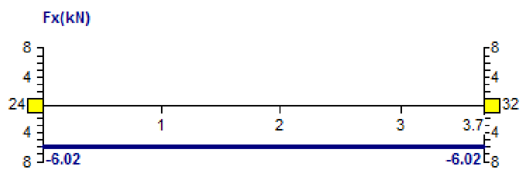
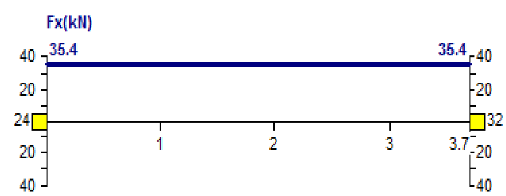



(d)

Figure 5. Comparision of node reaction for various column removal scenarios of three dimensional structures (a) Reaction at node 2 for one column removal scenario (b) Reaction at node 1 for one column removal scenario (c) Reaction at node 16 for two column removal scenarios and (d) Reaction at node 18 for two column removal scenario

Table 3: Comparison the results of shear force and bending moment with no removal of columns and after removal of one column scenario

Beam No.19 (Load Case 4)

| No column removal | One column removal scenarios |
|---|---|
|   |   |
|   |   |
|   |   |

Beam No.26 (Load Case 4)

No column removal | One column removal scenarios

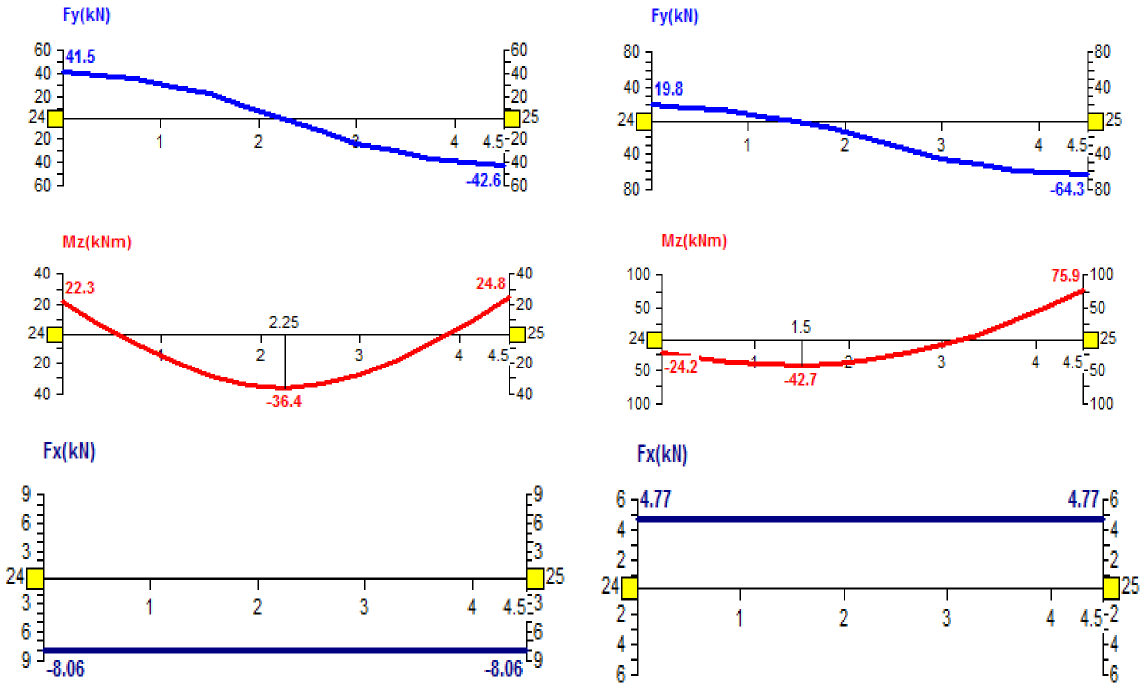
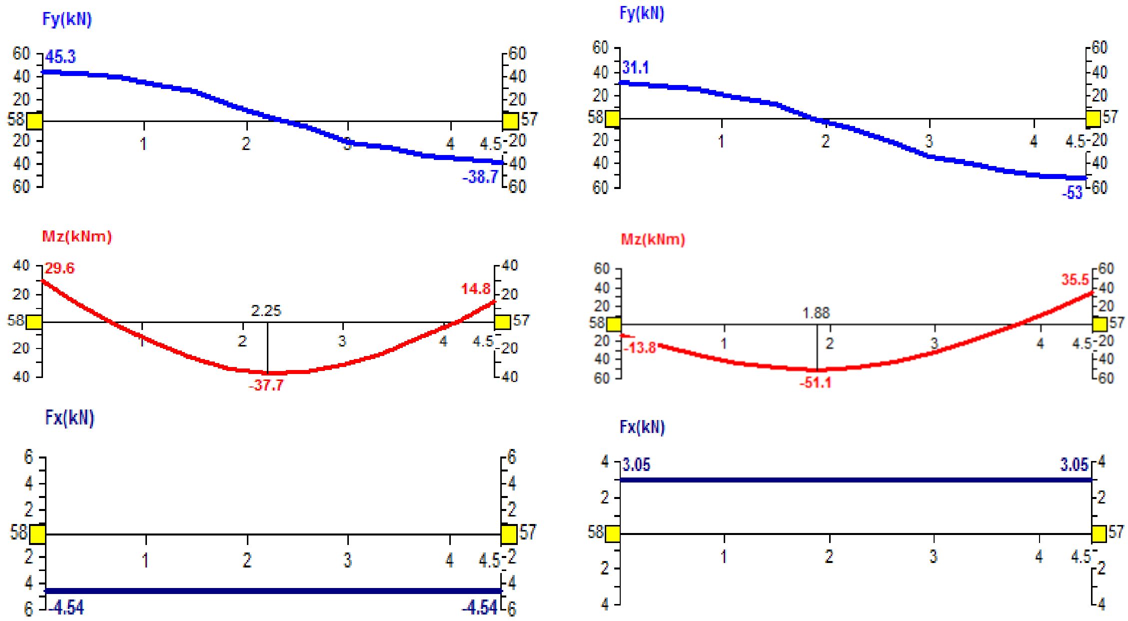



Table 4: Comparison the results of shear force and bending moment with no removal of columns and after removal of two column scenario

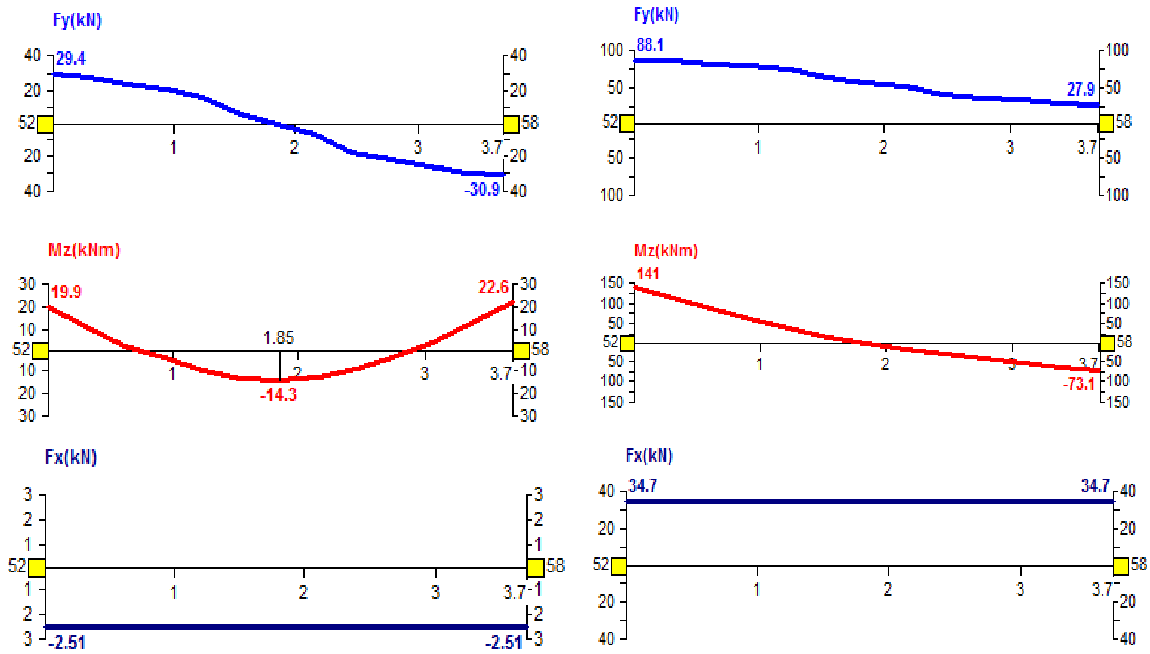
Beam No.67 (Load Case 4)

No column removal | Two column removal scenarios



Beam No.76 (Load Case 4)

No column removal Two column removal scenarios




## 4. CONCLUSIONS

In this work, the main objective was to investigate the behaviour of the four storey RCC building due to progressive collapse. Parametric studies are carried out to investigate the behaviour of progressive collapse under the umbrella of changes in such as shear forces, bending moment, node displacement, reactions at nodes, beam forces and induced beam stresses subjected to sudden loss of a vertical support member. Two different diagonally opposite columns were removed one by one, and the study (both static and dynamic) of progressive collapse initiation on a typical reinforced concrete frame is done with the help of a commercial software. This simple analysis can be used to quickly analyse the structures for different failure conditions and then optimize it for various threat scenarios. Based on this investigation, following conclusions are drawn:

1. It is found that the reaction of the neighbouring member of the removed element gets approximate 60% increment of the initial values due to large redistribution of forces which took place in short way direction whereas 12% to 35% increment of initial reaction is transferred to the adjacent column in longer way direction.
2. From the comparison, it is found that a large difference occurs in forces and bending moment from the initial condition (when no column is removed). The reasons is due to transfer of the instantaneously applied load to the remaining undamaged structure as well as joints.
3. The effect of transferring the load is more on the nearest member of the removed member and negligible when moved away from removed column.

4. It is found that the increment in the joint displacement of the neighbouring member of removed element gets approximately 27 times of an initial node displacement from the initial values after removal of one column and with an increment of approximately 30 times of initial value after removal of two columns due to large redistribution of forces.
5. Shear force in Y direction ($F_Y$) at that member which is located just above the removed column generally give negative value at the point of zero shear forces in initial condition. Sometime positive value is also seen instead of zero value of shear forces in Y- direction after removal of columns.

Shear force in X direction ($F_X$) at that member which is located just above the removed column after removal of two columns give value in positive zone instead of negative zone initially before removal of any columns.



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