Fire Induced Progressive Collapse of Multi-storied Steel Structure

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Abstract - *If the main structural elements are removed suddenly and the adjacent structural elements are unable to carry the structural load to be taken by the removal of the main member then collapse of the structure takes place. The removal of column occurs due to vehicle impact, blast and damage of shear wall or column by fire. In this study, a G+14 moment resisting steel frame structure was analysed using ETABS software to predict the sensitivity of the structure to progressive collapse by fire loads. Here columns at different levels were given a temperature of 550̊ C with material properties and yield strength as per IS 800. Load combinations were adopted as per IS 875 part I and II. According to GSA guidelines corner, edge, intermediate and re-entrant columns were applied a fire load separately at different levels or at alternate storeys. Here demand capacity ratio (DCR) and axial load values of all columns are obtained and compared. Here it is observed that top storeys are more susceptible than lower storeys at 550̊ C and the progressive collapse of structure will not occur. Again analysis was done by applying a temperature to edge columns to check the progressive collapse in increments of 100ºC and at 1000ºC the columns started to fail since the demand capacity ratios obtained were exceeding the limit. In that case the structure may be redesigned to avoid progressive collapse with a significant increase in steel consumption. This study can be useful for important structures.*

Key Words: Fire Load, GSA Guidelines, ETABS, Moment Resisting steel Frame, Progressive Collapse.

1. INTRODUCTION

Main structural elements if are eliminated suddenly and the adjacent structural elements are unable to carry the structural load to be taken by the elimination of the main member then collapse of the building takes place. Several accidental and purposeful happenings such as wrong construction practices and order, non-intentional overload, failure due to loads from seismic events and blasts may lead to collapse in a progressive manner. The blast loading having high intensity and having short time may have different effects compared to that from seismic loadings.

In this type of collapse load capacity of a small section of structure is lost due to high load, the damage of the small section leads to failure in the building in the continuous manner. Studying the collapse in progressive way requires us to know the response of building when one or more structural members are damaged. When the damage occurs in a component of building there will be redistribution of load in the building.

When the system of loading on structure and restraint conditions are changing in a way that the structural components are overloaded then structure collapses. The damage to one component leads to load transfer to adjacent components which are trying to find other load paths and if the components do not have sufficient resistance then it leads to total damage of the building. There will be heavy deforming of the components during the process. The damage occurring initially and damage occurring at final stages are not in proportions.

1.1 METHODOLOGY

1. A G+14 storey structure was modelled (Figure 1) for the analysis purpose in ETABS 2015 software, which can design and analyse the buildings.
2. The kind of building is a steel structure with slab of concrete and it is resistant to moment. The building having plan which is irregular and consisting of re-entrant corners.
3. Here the steel sections are taken by doing preliminary design which is done by considering dead load, live load and wind load.
4. For analysing the building the data taken is given below

For columns (built up sections by preliminary design)
From ground to 9th floor:
Depth: 700mm
Flange width: 400mm
Flange thickness: 20mm
Web thickness: 20mm
From 10th to 14th floor:
Depth: 600mm
Flange width: 350mm
Flange thickness: 20mm
Web thickness: 20mm
Primary beams: ISWB250
Secondary beams: ISMB200
Material properties:
Concrete: M25

Steel: Fe345
Rebar: Fe415
Slab thickness: 125mm
Seismic zone: 3
Response reduction factor: 4
Importance factor: 1
Time period(x): 0.4236 seconds
Time period(y): 0.4955 seconds

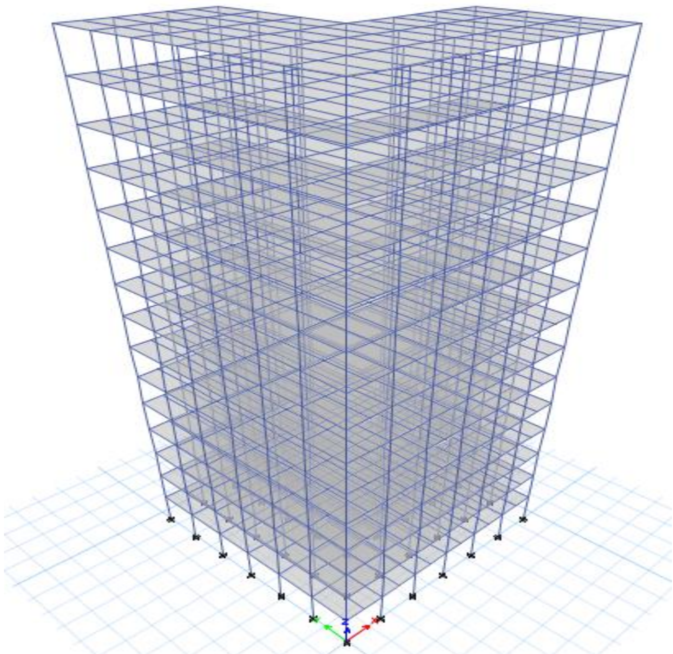

Fig. 1: 3D model of steel structure

5. IS 875 Part I and II have been made use of for taking loads and choosing load combinations. Magnitude of 3 KN/m2 was chosen as Live load and 12.42 KN/m wall load was applied on primary beams. The combinations of load taken are given in below table 1

Table -1: Load combinations

SI No	Load Combinations
1	1.5(DL+LL)
2	1.2(DL+LL±EQ)
3	1.5(DL±EQ)
4	0.9DL±1.5EQ

6. On the columns of the structure, fire load was applied, in starting stages the column is expanding with temperature and with increasing temperature column loses its rigid nature and elasticity modulus lost. This is resulting in collapse of columns. Temperature taken at this stage is 550º C [1].

3. ANALYSIS OF STEEL STRUCTURE USING ETABS SOFTWARE

The analysis of any building is done using standard guidelines so that the building has the capability to resist collapse.

ETABS, the structural analysis finite element program that takes into account difficult geometry and oversees all deformation at hinges to know ultimate deformation. It has default properties for materials and hinges which is also including Indian standard codes. The analysis using ETABS (2015) is involving below steps:

1. Modelling
2. Analysis
3. Designing

Before the analysis of structure a temperature of 550º C was applied to columns at various location of structure as per GSA guidelines. Fire load was given to corner column (C20), edge column (C17), intermediate column (C33) and re-entrant column (C8) of every alternate floor. As per GSA guidelines the demand capacity ratio value of each element should be less than 2. If it is more than 2 then the progressive collapse will occur. Below there are figures showing the plan view of the building, notations and initial deformed shapes of the columns after the analysis was completed by applying fire loads to columns.

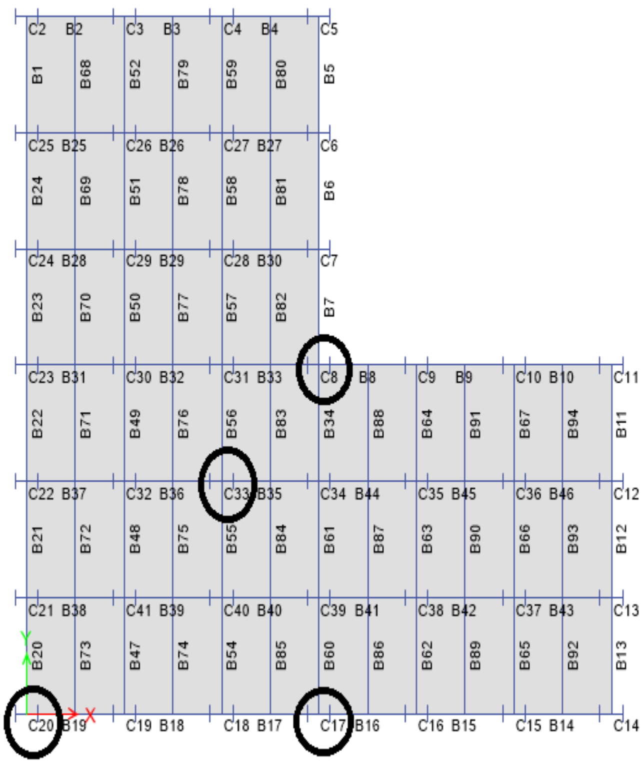

Fig. 2: Plan view of steel structure

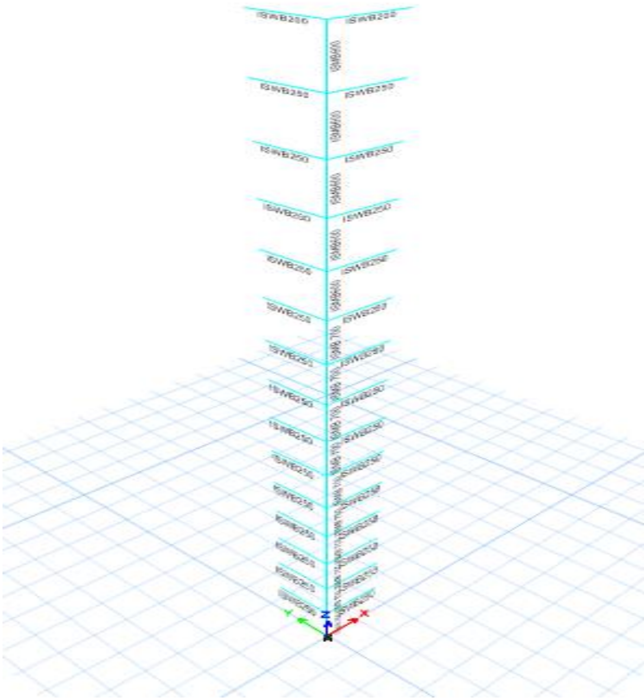

Fig. 3: Notation of corner column

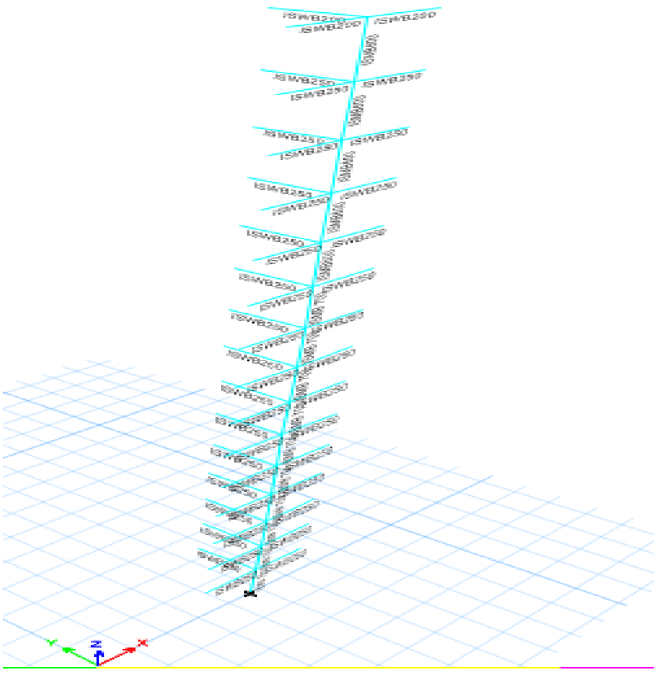

Fig. 5: Notation of edge column

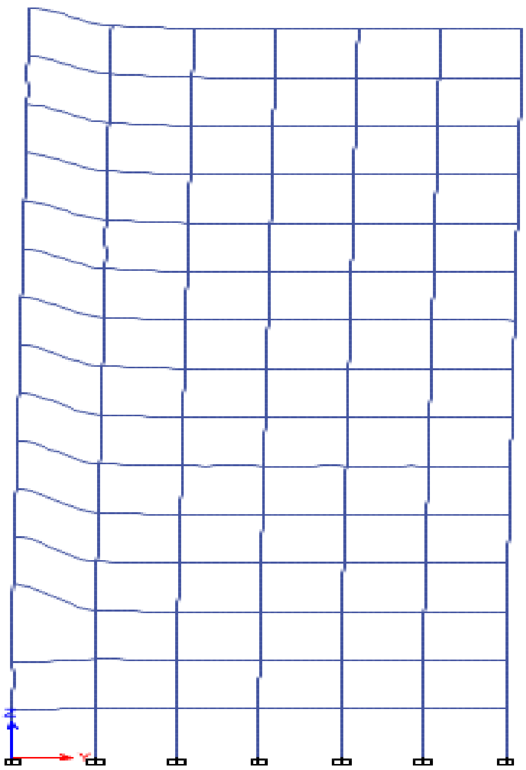

Fig. 4: Deformed shape of second floor corner column

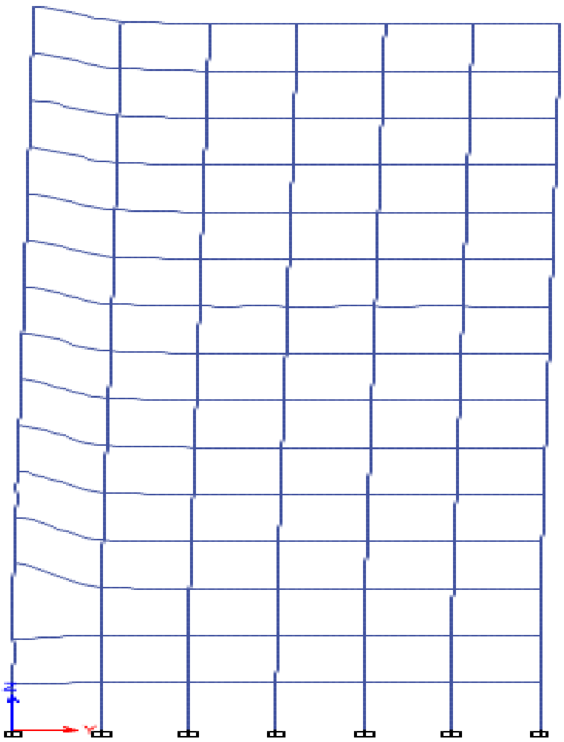

Fig. 6: Deformed shape of second floor edge column

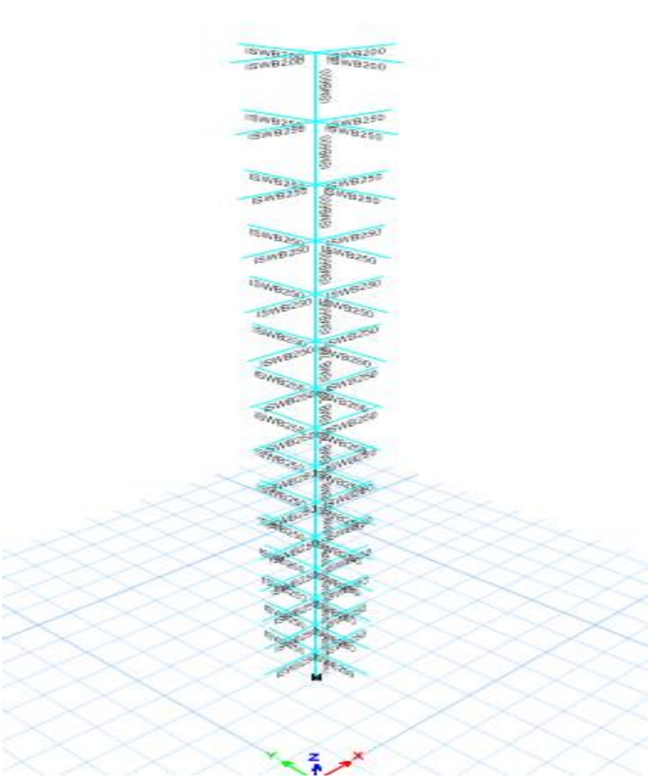

Fig. 7: Notation of intermediate column

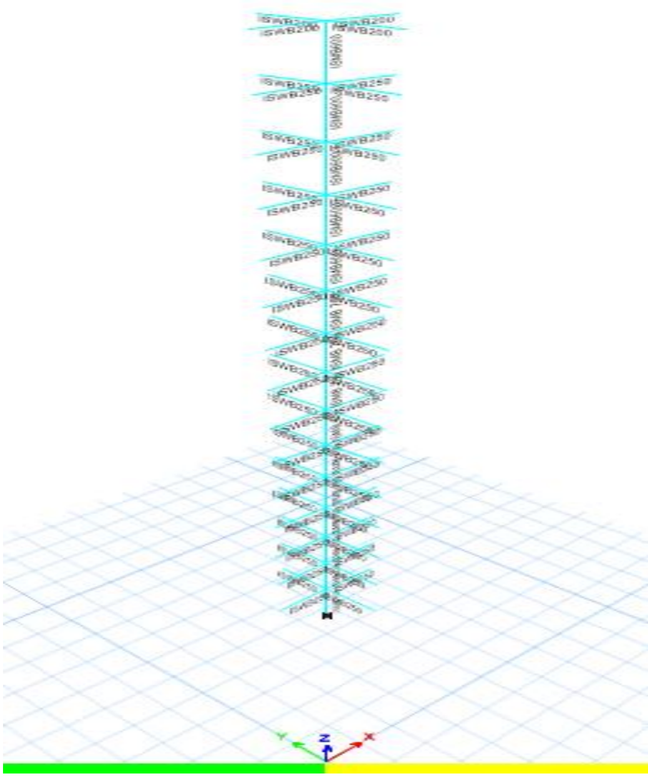

Fig. 9: Notation of re-entrant column

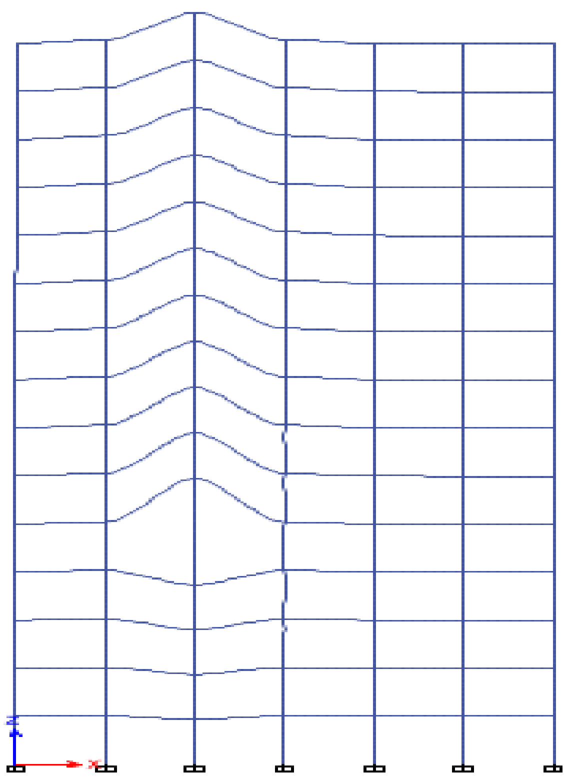

Fig. 8: Deformed shape of fourth floor intermediate column

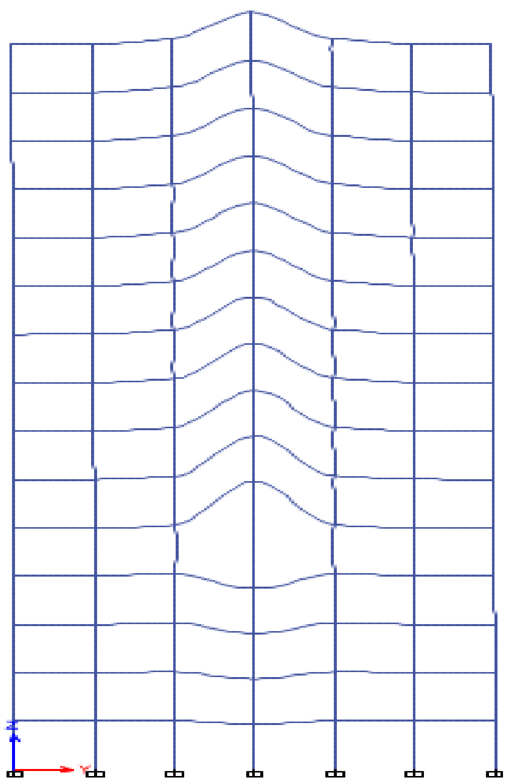

Fig. 10: Deformed shape of fourth floor re-entrant column

4. RESULTS AND DISCUSSIONS

4.1 Comparison of results

After modeling the structure in ETABS (2015), the loads are applied to the model and then the structure was analyzed and designed. The Demand Capacity Ratio (DCR) values and the axial load values are taken before application of the fire load and after application of the fire load. Here the DCR values obtained for columns and adjacent beams are within limit i.e. less than 2 and there is increment in the values of axial load after the application of fire load compared to before application of fire load values. Both DCR and axial load values of corner column, intermediate column, re-entrant column and edge column of each alternate floor are tabulated for both the cases i.e. before and after application of fire load. Below tables shows the obtained values of both DCR and axial load before and after fire cases.

Table -2: DCR values of corner column (C20)

Column location	Before fire	After fire
Ground floor	0.309	0.625
2nd floor	0.329	0.704
4th floor	0.276	0.605
6th floor	0.229	0.514
8th floor	0.180	0.428
10th floor	0.156	0.403
12th floor	0.109	0.320

Table -3: Axial load values of corner column (C20)

Column location	Before fire(kN)	After fire(kN)
Ground floor	1480.70	3153.93
2nd floor	1266.32	2702.58
4th floor	1076.97	2260.78
6th floor	865.10	1845.08
8th floor	652.25	1438.19
10th floor	440.50	1024.63
12th floor	232.29	588.67

By referring table 2 and table 3,

1. Since DCR values obtained are within limit i.e. less than 2, so the progressive collapse is not going to occur under fire load.

2. And also we can see the increment in axial loads but still the columns going to sustain these increased loads because the columns are safe under fire load.

Table -4: DCR values of edge column (C17)

Column location	Before fire	After fire
Ground floor	0.386	0.79
2nd floor	0.411	0.904
4th floor	0.352	0.768
6th floor	0.292	0.645
8th floor	0.23	0.523
10th floor	0.193	0.467
12th floor	0.123	0.314

Table -5: Axial load values of edge column (C17)

Column location	Before fire(kN)	After fire(kN)
Ground floor	1968.83	4180.17
2nd floor	1698.3	3783.01
4th floor	1506.8	3175.05
6th floor	1222.82	2616.92
8th floor	934.83	2071.18
10th floor	644.17	1504.28
12th floor	355.72	848.98

By referring table 4 and table 5,

1. Here also the DCR values obtained are within limit i.e. less than 2, so the progressive collapse is not going to occur under fire load.
2. And also axial loads on columns are increased and here also they are going to sustain these increased loads because the columns are safe under fire load.

Table -6: DCR values of re-entrant column (C8)

Column location	Before fire	After fire
Ground floor	0.398	0.734
2nd floor	0.385	0.609
4th floor	0.327	0.676
6th floor	0.269	0.563
8th floor	0.21	0.454
10th floor	0.189	0.4
12th floor	0.128	0.246

Table -7: Axial load values of re-entrant column (C8)

Column location	Before fire(kN)	After fire(kN)
Ground floor	2200.68	4167.35
2nd floor	2255.77	3627.4
4th floor	1888.5	3978.65
6th floor	1523.02	3290.75
8th floor	1158.84	2624.65
10th floor	665.16	1796.05
12th floor	362.84	1165.98

By referring table 6 and table 7,

1. Here also the DCR values obtained are within limit i.e. less than 2, so the progressive collapse is not going to occur under fire load.
2. And also axial loads on columns are increased and here also they are going to sustain these increased loads because the columns are safe under fire load.

Table -8: DCR values of intermediate column (C33)

Column location	Before fire	After fire
Ground floor	0.433	0.669
2nd floor	0.424	0.63
4th floor	0.355	0.711
6th floor	0.287	0.587
8th floor	0.218	0.467
10th floor	0.176	0.404
12th floor	0.106	0.24

Table -9: Axial load values of intermediate column (C33)

Column location	Before fire(kN)	After fire(kN)
Ground floor	2957.88	4564.62
2nd floor	2543	3776.54
4th floor	2128.85	4263.45
6th floor	1715.57	3518.02
8th floor	1303.14	2799.32
10th floor	891.32	2051.61
12th floor	374.84	1128.11

By referring table 8 and table 9,

1. Here also the DCR values obtained are within limit i.e. less than 2, so the progressive collapse is not going to occur under fire load.
2. And also axial loads on columns are increased and here also they are going to sustain these increased loads because the columns are safe under fire load.

Table -10: Comparison between edge and other located columns

Column	Critical values in %
Corner column	26.73
Re-entrant column	16.83
Intermediate column	41.58

4.2 Analysis by increasing temperature

Since the columns were safe at temperature of 550º C, so we have increased the temperature in increments of 100º C and at a temperature of 1000º C the columns started to fail. The DCR and axial load values are obtained for edge column (since it is a critical column location) at alternate floors by applying fire load of 1000º C and they are compared with the values which are obtained without fire loading. The values are tabulated in below table 11.

Table -11: DCR values of edge column (C17)

Column location	Before fire	After fire
Ground floor	0.386	2.12
2nd floor	0.411	2.204
4th floor	0.352	2.164
6th floor	0.292	1.932
8th floor	0.23	1.748
10th floor	0.193	1.632
12th floor	0.123	1.314

Table -12: Axial load values of edge column (C17)

Column location	Before fire(kN)	After fire(kN)
Ground floor	1968.83	6172.2
2nd floor	1698.3	5421.05

4th floor	1506.8	4543.78
6th floor	1222.82	3761.33
8th floor	934.83	3004.72
10th floor	644.17	2211.27
12th floor	355.72	1288.79

By referring table 11 and table 12,
1. Here the DCR values obtained are exceeding the limit at ground, second and fourth floor i.e. DCR values are more than 2, so the progressive collapse is going to occur under fire load of 1000º C.

2. And also axial loads on columns are increased and here they are not going to sustain these increased loads because the columns are not safe under fire load.

5. CONCLUSIONS

From the above discussions the following conclusions are made
1. By referring table 10, at 550º C the edge columns are 26.73% more critical compared to corner columns at ground floor.

2. By referring table 10, at 550º C the edge columns are 16.83% more critical compared to re-entrant columns at ground floor.

3. By referring table 10, at 550º C the edge columns are 41.58% more critical compared to intermediate columns at ground floor.

4. And also by referring table 2, 4, 6 and 8, the demand capacity ratio (DCR) values obtained are within limit i.e. less than 2 as per GSA guidelines so the progressive collapse is not going to occur at fire load of 550º C.

5. And at 1000º C the DCR values obtained for edge column exceeds the limit as per GSA guidelines, so the structure may fail for this fire load. It can be prevented by using larger steel sections or by increasing bracings.

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