International Journal Of Advance Research, Ideas And Innovations In Technology
IJARIIT
ISSN: 2454-132X
Impact Factor: 6.078
(Volume 7, Issue 4 - V7I4-1740)
Available online at: https://www.ijariit.com

Progressive collapse analysis of steel frame structure in different earthquake zones

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ABSTRACT

Progressive collapse of structures is initiated by the loss of one or more load-carrying members. As a result, the structure will seek alternate load paths to transfer the load to structural elements, which may or may not have been designed to resist the additional loads. Failure of overloaded structural elements will cause further redistribution of loads, a process that may continue until stable equilibrium is reached. Equilibrium may be reached when a substantial part of the structure has already collapsed. The resulting overall damage may be disproportionate to the damage in the local region near the lost member. Loss of primary members and the ensuing progressive collapse are dynamic nonlinear processes.

Keywords: *Alternate Load Path, Progressive Collapse, Dynamic Nonlinear, Etc.*

1. INTRODUCTION

Progressive collapse implies disproportional global structural system failure originated by local structural damage. It is a rare event, as it necessitates an initiation of local element removal criteria either due to the inevitable forces of Nature or due to manmade hazards. The gravity load of the structure is now transferred to neighboring columns; these columns should resist the additional abnormal gravity loads & redistribute loads to avoid failure of the major part of the structure. Present day structure design practices & lesser integral ductility and continuity, gets more prone to progressive collapse. However, there should be certain provisions needed for additional consideration to ascertain the safety of structure after any local failure.

These events define the progressive collapse very well. The provision of the range & type of progressive collapse in different situation provides much important information with particular regard to progressive collapse resistance, by complementing additional measures in the design. In order to secure structural safety against progressive collapse additional considerations such as abnormal loadings must be taken. The abnormal loads arise from vast sources such as explosion of gas, vapor inferno or confined dust, malfunctioning of machines, bomb explosion, the sudden impact of vehicles, etc. Nevertheless, till date, there are no adequate tools that can analyze the progressive collapse with satisfactory reliability.

In this topic study, the behavior of Steel framed structures to progressive collapse located in different seismic zones is investigated. A Structure with 20 stories is analyzed for different seismic zones. As per the provisions of GSA guidelines.

1.1 Progressive Collapse

A structure undergoes Progressive Collapse when a primary structural element fails, resulting in the failure of adjoining structural elements, which in turn causes further structural failure. It is sometimes also called a disproportionate collapse, which is defined as a structural collapse disproportionate to the cause of the collapse. As the small structural element fails, it initiates a chain reaction that causes other structural elements to fail in a domino effect, creating a larger and more destructive collapse of the structure. A good example of progressive collapse is a house of cards; if one card falls near the top, it causes multiple cards to fall below it due to the impact of the first card, resulting in full collapse of the house of cards.

1.2 Causes of Progressive Collapse

There are usually multiple factors that take place in order to initiate a progressive collapse.

1. Improper communication between contractors and engineering documents can cause a progressive collapse. In this case, workers may not install specific structural elements properly that can lead to weakened structural members throughout the structure.
2. Some contractors may be pressed for time to where they may improperly address key connections or finishing techniques to adequately install the structure.
3. Improper inspection or overlooking structural issues also leads to factors that initiate a progressive collapse. In some cases proper inspection may find a faulty member or connection yet may not properly document it or resolve the issue due to poor communication.
4. Another root cause of progressive collapse takes place in the design phase of a structure. If structures go for years without receiving proper maintenance, rust or other material failures can occur which weaken the structure and make it more susceptible to a progressive collapse.

2. OBJECTIVE

1. To determine type of failure and critical section due to earthquake loading for different earthquake zone (Zone II and Zone V) under the guideline of GSA for progressive collapse.
2. To determine the effects on structure caused due to sudden failure of axial member.
3. To decide suitable combination of structure for construction of high rise building by comparing with similar R.C.C. building.
4. To study the response of steel structure for various parameters such as Demand Capacity Ratio, Bending moment, Deflection, Story Drift and cross checking with similar R.C.C. structure.

- Study of Progressive collapse analysis for steel structure.
- Discuss guidelines for column removals in structure.
- Modeling & Analysis of a high-rise steel frame structure using E-TABS.

3. SCOPE OF STUDY

- This study can give us detail idea of failure of steel structure due to earthquake loading.
- Type of failure of structure can also be studied.
- Failure of structure due to earthquake loading can guide us for design of steel structure.

4. LIMITATIONS OF STUDY

- The conclusion which is derived from this project is only for steel structures, as model in this project is considered to be steel frame structure.
- Analysis done in this project is only for G+ 20 structures made of only steel section.
- Results obtained from this project are only valid for G+ 20 structures.
- Results varies as per location of structure for example change in location of structure may change design of structure due to earthquake load or wind load.
- Model selected for structure have specific dimension any change in dimension will change the analysis and hence result of the project.

5. METHODOLOGY

This chapter describes various methods and approaches used for analysis and design of structure. The present study is carried out on analysis and design of high-rise steel building using ETABS 2015 software. Modeling of G+20 storey structure is done in ETABS 2015. The models are analyzed and designed for design loading and load combinations.

5.1 Design methodology

The structures in the present work are designed for progressive collapse according to “GSA Alternate Path Analysis and Design Guidelines for Progressive Collapse Resistance.” The GSA guidelines are applicable in following cases. These Guidelines apply to GSA owned (new and existing) and new GSA lease construction. If stated as a tenant specific requirement within the Program of Requirements (POR), these Guidelines may also apply to new lease acquisitions or succeeding leases that are established through full and open competition. These Guidelines do not apply to lease renewals, extensions, expansions, or superseding and succeeding leases that are established other than through full and open competition.

5.2 Types of Analysis

5.2.1) Linear Static Analysis

1. First, the building is analyzed with the gravity load (Dead Load+ Live Load) … (Eq. 1) and obtains the output results for moment and shear without removing any column.
2. Now remove a vertical support (column) from the position under consideration and carry out a linear static analysis to the altered structure and load this model with 2 {Dead Load + 0.25 Live Load} …. (Eq. 2)
3. The static load combinations were entered into the ETABS 2015V 15.0 program and a model of the structure was generated. An ETABS 2015 computer simulation was executed for each case of different Column removal location on the model and the result are reviewed.
4. Further, from the analysis results obtained, if the DCR for any member end connection or along the span itself is exceeded the allowable limit based upon moment and shear force, the member is expected as a failed member.
5. If DCR value surpasses its criteria then it will lead to progressive collapse.

5.2.2) Dynamic Analysis

1. Static Studies assumes that loads are constant or applied very slowly until they reach their full values. Because of this assumption, the velocity and acceleration of each particle of the model is assumed to be zero. As a result, static studies neglect inertial and damping forces.
2. For many practical cases, loads are not applied slowly or they change with time or frequency. For such cases, use a dynamic study. Generally, if the frequency of a load is a larger than 1/3 of the lowest (fundamentals) frequency, a dynamic study should be used.
3. Linear dynamic studies are based on frequency studies. The software calculates the responses of the model by accumulating the contribution of each model to the loading environment. In most cases, only the lower models contribute significantly to the responses. The contribution of a mode depends on the loads frequency content, magnitude, direction, duration, and location.

Objective of dynamic analysis are include:

- Design structural and mechanical system to perform without failure in dynamic environments.
- Modify systems characteristics (i.e. geometry, damping mechanism, material properties, etc.) to reduce vibration effects.

The design procedures given by GSA (General Services Administration, 1949) Guidelines aim to reduce the potential for progressive collapse by bridging over the loss of a structural element, limiting the extent of damage to a localized area (Alternate Path) and providing a redundant and balanced structural system along the height of the building.

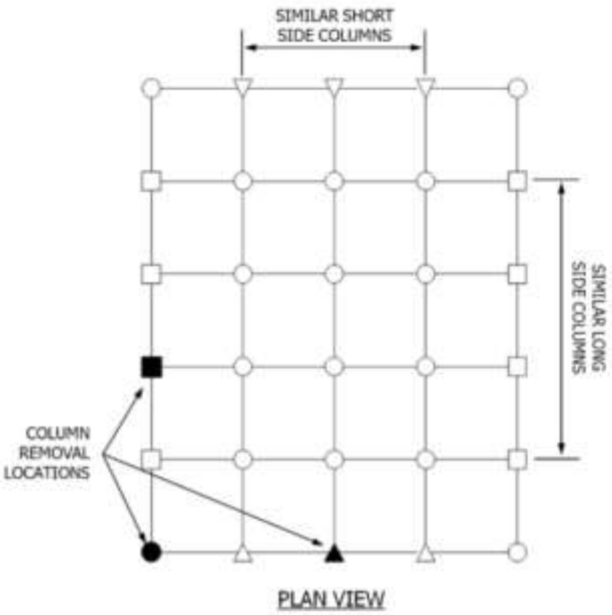

Fig 5.1 Location of External Column Removal

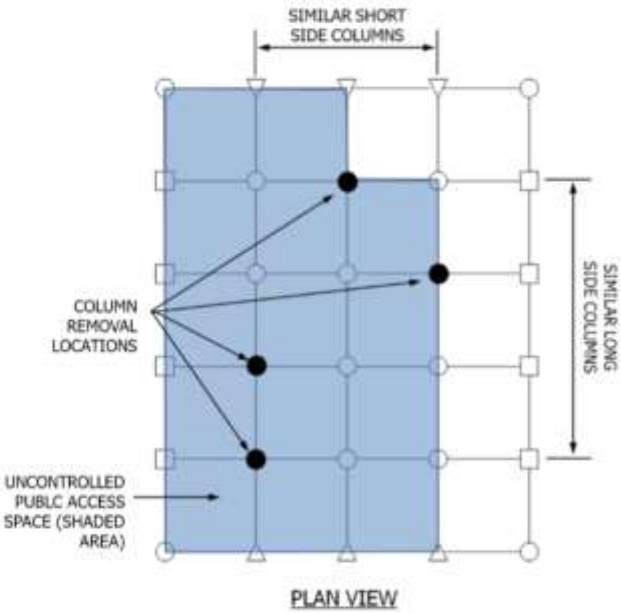

Fig 5.2 Location of Internal Column Removal

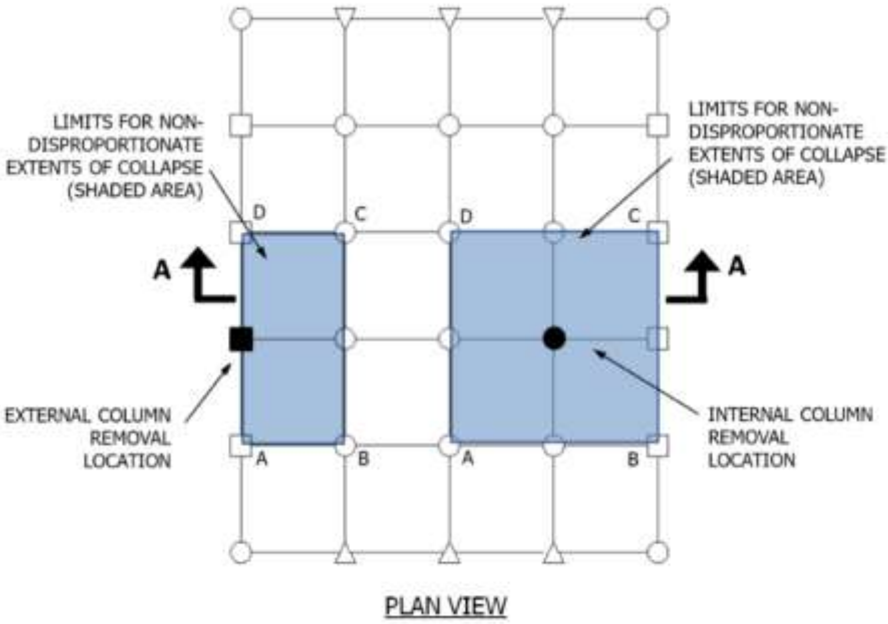

Fig 5.3 Allowable Extents of Collapse for Interior and Exterior Column Removal in Plan

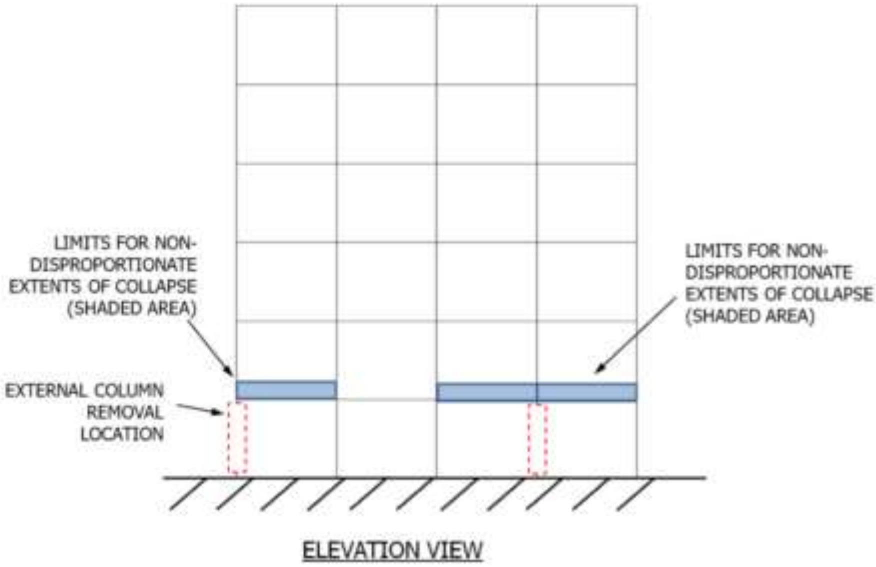

Fig 5.4 Allowable Extents of Collapse for Interior and Exterior Column Removal in Elevation

5.2.3) Preliminary data of structure for analysis:

Table No. 5.1 Preliminary data required for analysis (Validation)

Sr. No.	**Particulars**	**Details**
1	Type of Structure	High Rise Steel Frame Structure
2	Type of building	Public building
2	Seismic Zone	(IS:1893-2002) [Zone II & Zone V]
3	Number of Stories	G+20
4	Floor Height	3000 mm
5	Spacing of grid	5000 mm in both direction
6	Imposed load	2.5 KN/m^2
7	Floor Finish	1.25 KN/m^2
8	Wall Load	10 KN/m^2
9	Wall Load at Roof	3 KN/m^2
10	Materials	Concrete M25, Reinforcement Fe 415 & Structural steel (Fy-345)
11	Depth of slab	150 mm thick

5.3 Structural modeling, analysis and design

Modeling of building structure is done by using ETABS 2015. The complete modeling, analysis and design of structure is done in three phases namely preprocessing, processing and post processing.

5.4 Pre-processing
In preprocessing phase, building is modeled by forming a grid, defining material properties, defining of section properties and defining support conditions. Loading and load combinations are defined as per IS:1893-2002 (Part-I) & IS:875-1987 and assigned to model. Response Spectrum function is defined as per IS: 1893-2002 for the purpose of seismic analysis.

5.5 Processing
After carried out all the steps involved in pre-processing phase, structure are ready to analysis and design. There is option of Run analysis cases. As per requirement, specific analysis cases are chosen and analysis of structure is performed. After analysis, design is carried out to get, DCR, Joint Displacement, Axial Force Shear force and Bending Moment for different position of column removal and bracing system.

5.6 Post-processing
In post- processing phase, obtained results of analysis and design are verified.

6. ANALYSIS RESULT

6.1 For Seismic Zone II:

6.1 A] Case: 1 Analysis for the sudden loss of a column situated at the corner of building.

Case 1: Column C1 Remove at ground floor.

Table 6.1 Demand capacity ratio of column – C1

Sr. No.	Before PC	After PC
1	0.392	0.00
2	0.782	0.274
3	0.744	0.116
4	0.624	0.137
5	0.914	0.162
6	0.481	0.377
7	0.536	0.6
8	0.541	0.6
9	0.518	0.651
10	0.488	0.65
11	0.539	0.674
12	0.491	0.57
13	0.455	0.741
14	0.609	0.913
15	0.576	0.857
16	0.516	0.446
17	0.462	0.449
18	0.502	0.429
19	0.498	0.613
20	0.449	0.639

Table 6.2 Demand capacity ratio of column – C2

Sr. No.	Before PC	After PC
1	0.525	1.008
2	0.699	1.354
3	0.871	1.106
4	0.791	1.804
5	0.748	1.127
6	0.654	1.265
7	0.714	2.816
8	0.707	1.347
9	0.67	1.248
10	0.31	1.137
11	0.6	1.25
12	0.547	0.43
13	0.518	0.856
14	0.694	1.123
15	0.774	1.158
16	0.671	0.422
17	0.597	0.303
18	0.529	0.677

19	0.559	0.75
20	0.52	0.829

Table 6.3 Demand capacity ratio of column – B1

Sr. No.	Before PC	After PC
1	0.494	2.717
2	0.548	2.261
3	0.582	2.773
4	0.612	1.968
5	0.632	1.926
6	0.527	2.409
7	0.538	5.662
8	0.577	2.243
9	0.624	2.074
10	0.627	1.763
11	0.687	1.565
12	0.626	1.615
13	0.604	1.02
14	0.557	1.245
15	0.707	1.112
16	0.671	1.048
17	0.624	0.998
18	0.594	0.982
19	0.389	1.642
20	0.556	1.69

Table 6.4 Axial Force of column - C1

SR NO	Before PC	After PC
1	-3303.8112	0
2	-3155.7074	-36.2100
3	-3003.5864	-57.4770
4	-2847.6625	-57.9500
5	-2688.4543	-41.6721
6	-2272.2884	-19.3591
7	-2119.1279	-257.9737
8	-1964.4963	-446.2174
9	-1808.6515	-588.4344
10	-1649.3612	-715.4028
11	-1488.7317	-814.0322
12	-1334.8382	-815.0266
13	-1173.5790	-887.0778
14	-1013.5980	-928.9767
15	-853.9983	-919.5095
16	-713.3261	-781.7634
17	-566.6691	-631.7423
18	-420.8595	-476.5738
19	-286.4382	-311.1278
20	-131.5210	-198.4176

Table 6.5 Axial Force of column - C2

Sr. No.	Before PC	After PC
1	-4419.5443	-10264.8825
2	-4195.3987	-9711.5403
3	-3968.3430	-9162.6108
4	-3747.0344	-8635.1888
5	-3526.6313	-8123.9970
6	-3303.3583	-7622.5161
7	-3082.8293	-6984.2260

8	-2864.5932	-5780.8263
9	-2645.8620	-5786.2298
10	-2426.3777	-5208.2855
11	-2205.8700	-4646.3840
12	-1984.2614	-4137.2332
13	-176.9044	-3590.8181
14	-1537.8776	-3057.5611
15	-1320.2630	-2552.8084
16	-1098.7538	-2126.5630
17	-874.9203	-1706.7597
18	-650.2122	-1289.1108
19	-427.1543	-871.5408
20	-213.3389	-415.1585

Table 6.6 Maximum Bending Moments of Beam - B1

Sr. No.	Before PC	After PC
1	45.4523	527.0487
2	47.5916	517.5778
3	51.7648	500.5599
4	54.9241	481.9039
5	56.6998	470.8375
6	68.3265	773.8402
7	70.7054	720.3918
8	73.2746	675.5212
9	77.6009	656.5257
10	78.7481	455.4261
11	73.2448	508.227
12	81.0534	535.0541
13	78.2223	550.8052
14	72.7401	486.3326
15	66.0751	340.9353
16	67.4923	325.6487
17	65.6726	321.2621
18	59.9548	303.9256
19	67.7122	365.0766
20	45.7047	256.6419

Table 6.7 Shear Force of Beam – B1

Sr. No.	Before PC	After PC
1	59.7191	332.8879
2	60.6094	330.5021
3	62.402	321.5768
4	63.842	313.193
5	64.4906	307.9465
6	70.1843	439.0498
7	71.1547	415.4873
8	72.1515	393.9709
9	73.8854	386.3197
10	74.5648	343.528
11	71.883	322.4195
12	75.6818	334.21
13	74.621	343.641
14	72.5977	317.2487
15	68.5525	252.7525
16	69.04	246.4601
17	68.7668	243.6991
18	66.9936	238.0457
19	71.2116	266.8499
20	62.4387	223.1374

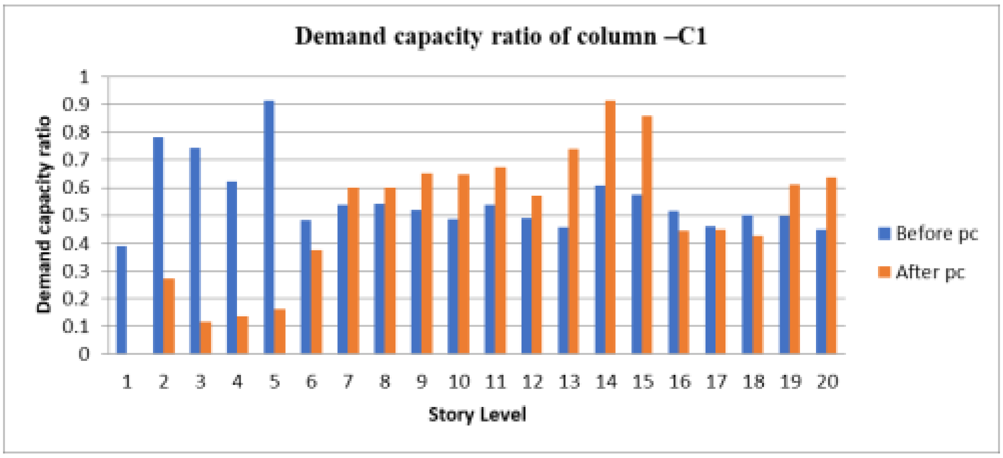

Chart 6.1 Demand capacity ratio of column – C1

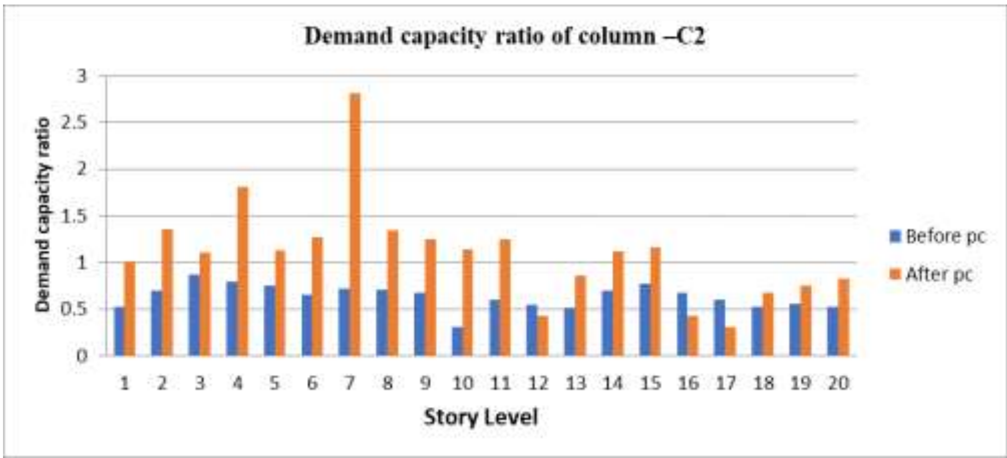

Chart 6.2 Demand capacity ratio of column – C2

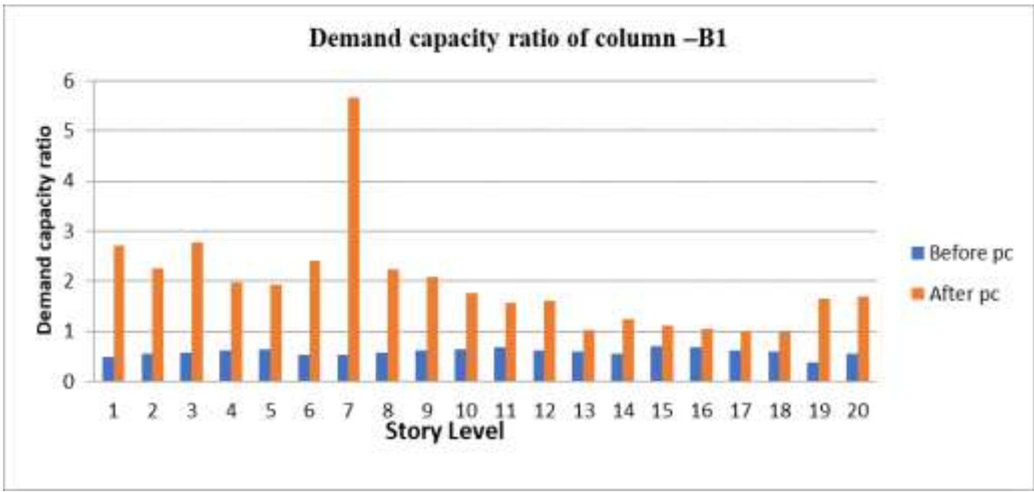

Chart 6.3 Demand capacity ratio of column - B1

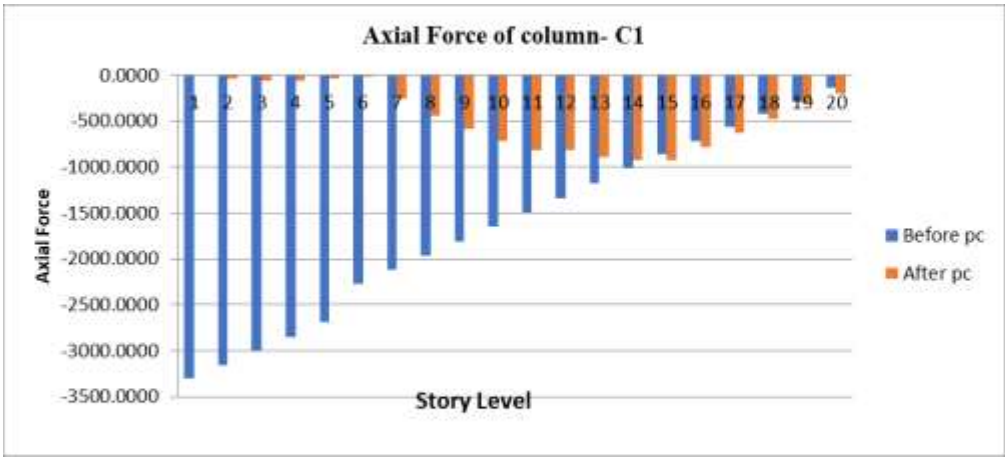

Chart 6.4 Axial Force of column - C1

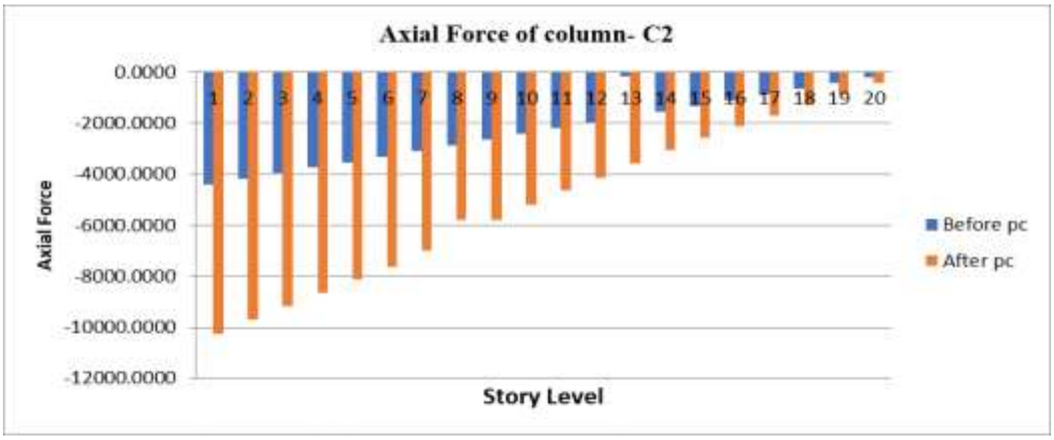

Chart 6.5 Axial Force of column- C2

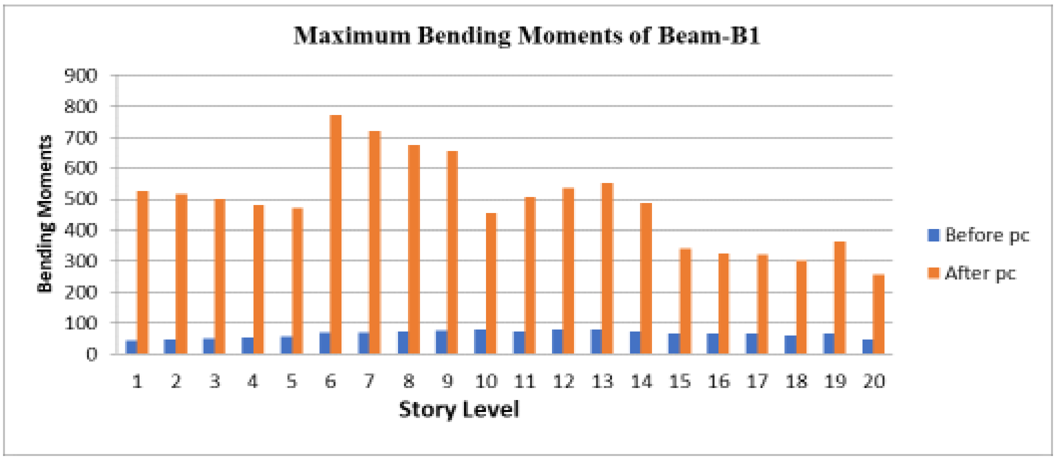

Chart 6.6 Maximum Bending Moments of Beam - B1

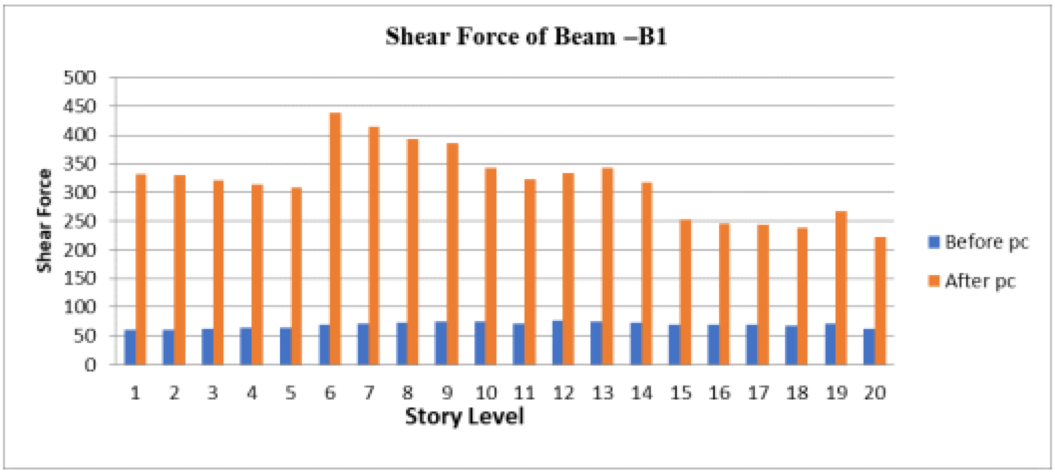

Chart 6.7 Shear Force of Beam – B1

6.2 For Seismic Zone V:

6.2 A] Case: 1 Analysis for the sudden loss of a column situated at the corner of building.

Case 1: Column C1 Remove at ground floor.

Table 6.8 Demand capacity ratio of column – C1

Sr. No	Before PC	After PC
1	0.519	0
2	0.878	0.178
3	0.811	0.076
4	0.68	0.09
5	1.01	0.107
6	0.671	0.243
7	0.623	0.386
8	0.648	0.384
9	0.635	0.415
10	0.601	0.413

11	0.652	0.427
12	0.585	0.359
13	0.556	0.465
14	0.755	0.572
15	0.673	0.534
16	0.523	0.274
17	0.463	0.273
18	0.527	0.257
19	0.531	0.37
20	0.449	0.378

Table 6.9 Demand capacity ratio of column – C2

Sr. No	Before PC	After PC
1	0.657	0.695
2	0.713	0.932
3	0.909	1.153
4	0.825	1.049
5	0.794	0.98
6	0.683	0.877
7	0.746	0.966
8	0.768	0.934
9	0.748	0.87
10	0.707	0.795
11	0.663	0.734
12	0.595	0.655
13	0.58	0.607
14	0.792	0.794
15	0.866	0.385
16	0.711	0.676
17	0.609	0.594
18	0.565	0.509
19	0.565	0.56
20	0.52	0.513

Table 6.10 Demand capacity ratio of column – B1

Sr. No	Before PC	After PC
1	0.494	1.54
2	0.548	1.456
3	0.582	1.343
4	0.612	1.24
5	0.632	1.179
6	0.527	1.537
7	0.538	1.404
8	0.577	1.262
9	0.624	1.917
10	0.627	1.757
11	0.687	1.801
12	0.626	1.668
13	0.604	1.574
14	0.557	1.374
15	0.707	1.63
16	0.671	1.554
17	0.624	1.525
18	0.594	1.458
19	0.389	1.033
20	0.556	1.273

Table 6.11 Axial Force of column - C1

Sr. No.	Before PC	After PC
1	-2984.3491	0
2	-2845.6278	-30.4496
3	-2705.1318	-47.1643
4	-2561.0592	-49.7980
5	-2414.1148	-41.0231
6	-2272.2884	-29.3636
7	-2119.1279	-184.0074
8	-1964.4963	-305.2092
9	-1808.6515	-395.9780
10	-1649.3612	-475.9979
11	-1488.7317	-537.1427
12	-1334.8382	-536.4470
13	-1173.5790	-579.6265
14	-1013.5980	-603.1967
15	-858.9983	-594.5211
16	-713.3261	-505.5732
17	-566.6691	-408.5052
18	-420.8595	-307.9823
19	-280.4382	-201.3710
20	-131.5210	-126.6569

Table 6.12 Axial Force of column - C2

Sr. No.	Before PC	After PC
1	-4419.5443	-7059.0575
2	-4195.3987	-6680.0131
3	-3968.3430	-6302.8754
4	-3747.0344	-5940.6856
5	-3526.6313	-5588.7475
6	-3303.3583	-5242.4533
7	-3082.8293	-4808.0454
8	-2864.5932	-4392.4013
9	-2645.8620	-3991.3652
10	-2426.3777	-3595.3429
11	-2205.8700	-3211.5276
12	-1984.2614	-2861.2204
13	-1760.9044	-2485.9795
14	-1537.8776	-2119.6721
15	-1320.2463	-1773.4058
16	-1098.1538	-1476.2331
17	-874.9203	-1182.8970
18	-650.2122	-890.6506
19	-427.1543	-599.4942
20	-213.3389	-285.7741

Table 6.13 Maximum Bending Moments of Beam - B1.

Sr. No.	Before PC	After PC
1	45.4528	343.4403
2	47.5916	336.7745
3	51.7648	325.0049
4	54.9241	312.2664
5	56.5998	304.4569
6	68.3265	497.9329
7	70.7054	462.4171
8	73.2746	432.5161
9	77.6009	419.1889
10	78.7481	398.3663
11	73.2448	323.5238

12	75.6818	378.9385
13	78.2223	348.7001
14	72.7901	307.4566
15	66.0551	216.1341
16	67.3923	206.0662
17	65.6726	2021.8534
18	59.9548	191.9021
19	67.7122	229.047
20	45.7047	162.8348

Table 6.14 Shear Force of Beam – B1

SR NO	Before PC	After PC
1	59.7191	214.1974
2	60.6094	212.4175
3	62.402	206.148
4	63.842	200.4379
5	84.49	196.769
6	70.1843	280.5768
7	71.1547	264.7776
8	72.1515	250.3533
9	73.8854	244.9612
10	74.5658	235.5476
11	71.883	203.7277
12	75.6818	226.184
13	74.621	216.4756
14	72.5977	199.4478
15	68.5525	158.9419
16	69.04	154.7869
17	68.7668	152.9033
18	66.9936	149.395
19	71.2116	166.9587
20	62.4387	140.2364

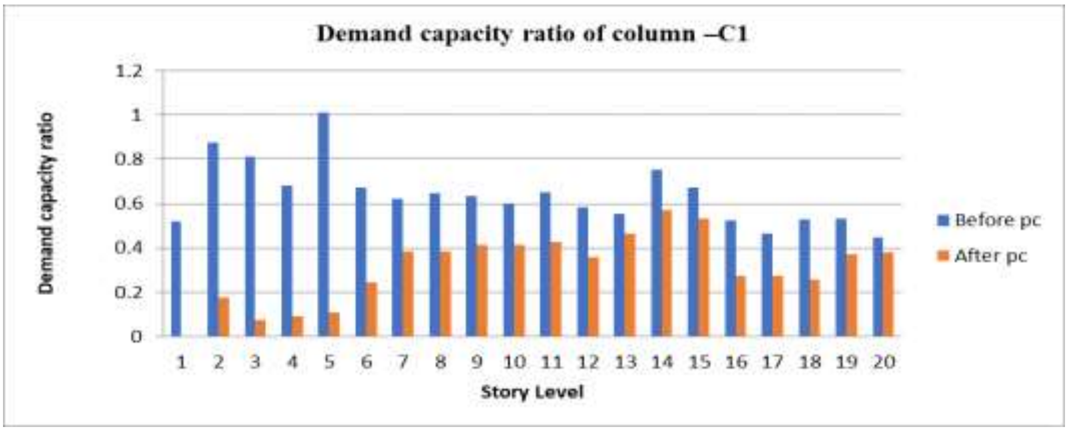

Chart 6.8 Demand capacity ratio of column – C1

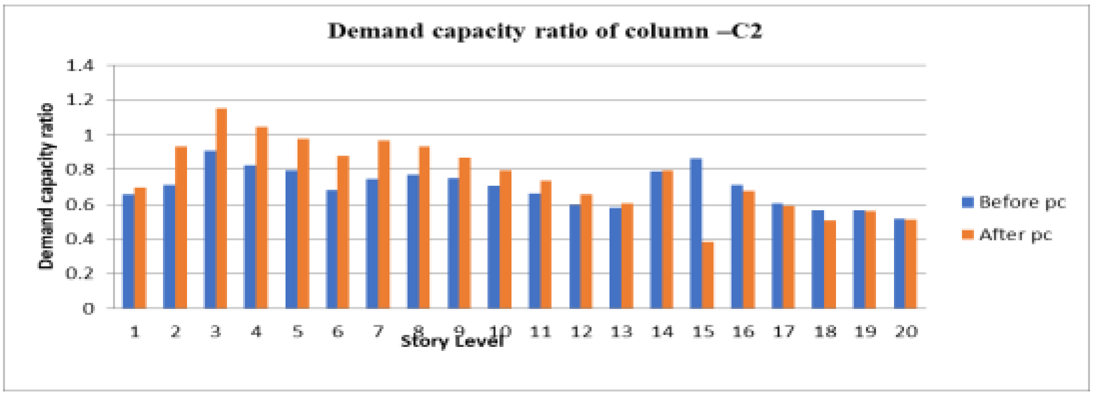

Chart 6.9 Demand capacity ratio of column - C2

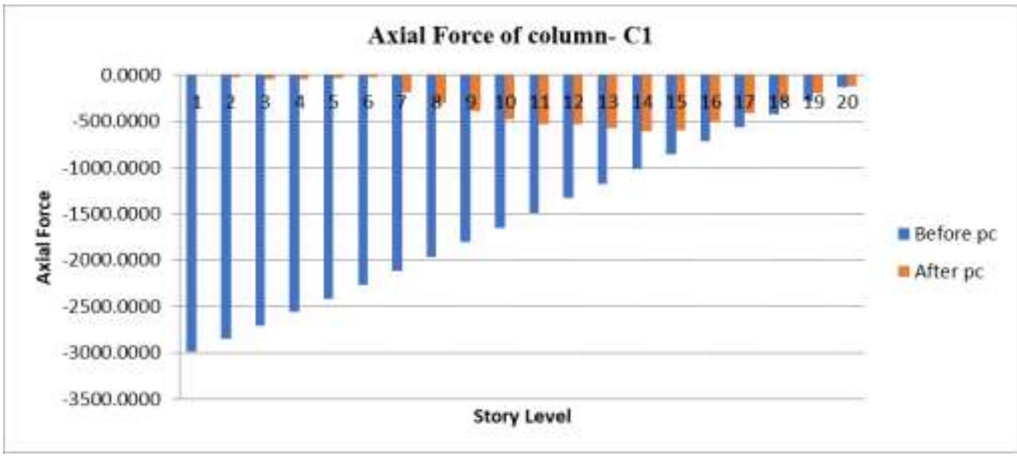

Chart 6.10 Axial Force of column - C1

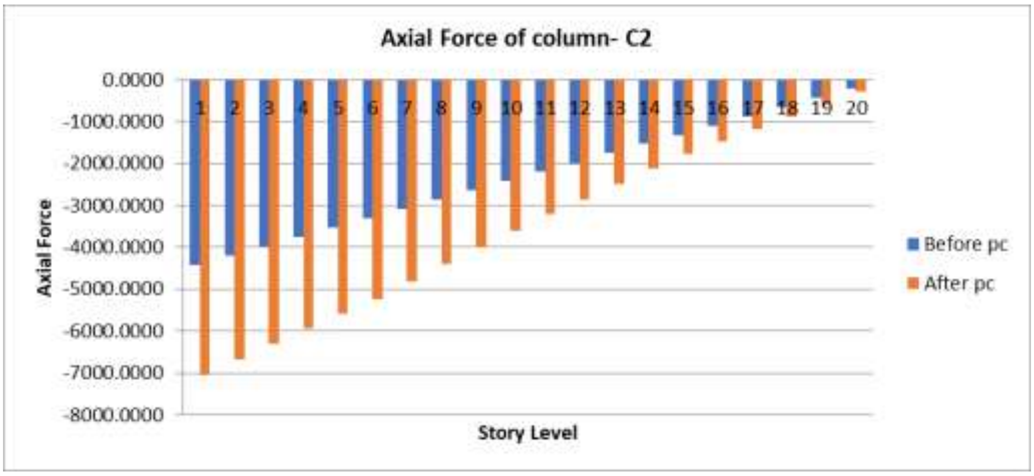

Chart 6.11 Axial Force of column- C2

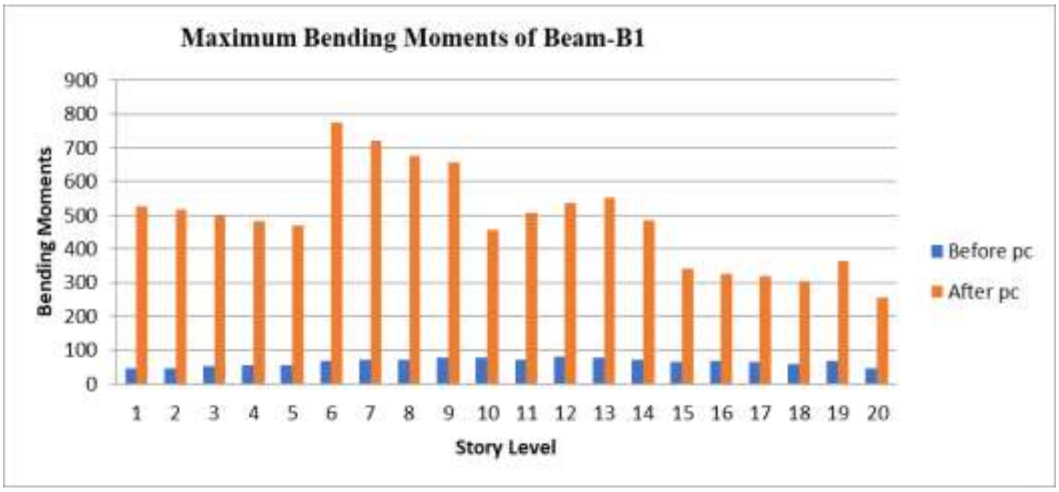

Chart 6.12 Maximum Bending Moments of Beam - B1

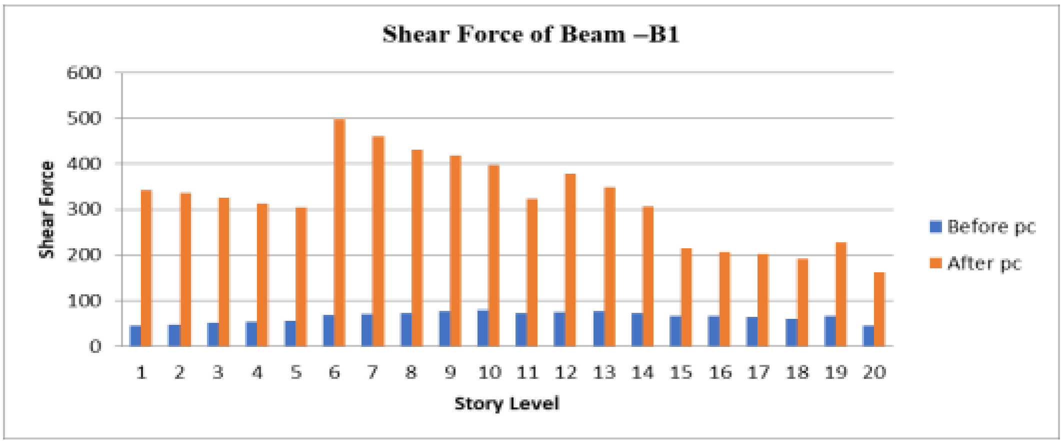

Chart 6.13 Shear Force of Beam – B1

7. CONCLUSION

After the study of response of G+20 steel structure in ETABS for Progressive collapse considering various parameters like Demand Capacity Ratio, Bending Moment, Shear Stress and Axial Forces on columns and comparing with similar R.C.C. structure, we came to the following conclusion that:

- When any Column of ground floor fails due to earthquake loading, most of the columns above that member fails the check for demand capacity ratio as the axial forces increases drastically.
- Due to sudden removal of specific column, the surrounding columns gets affected and there is an increase in axial forces of about 20% to 25% accordingly.
- Due to the sudden failure/removal of column, the surrounding beams are also affected resulting in an increase in Maximum Bending Moment of about 5 times of which acting before Collapse of axial member.
- There is an increase in shear forces in surrounding beams of about 50% to 65% near the failed column.
- The steel structure does not show sudden failure as compared to similar R.C.C. structure as the plasticity state of steel is being used after elastic limit. The Plastic limit of R.C.C. structure is relatively low as compared to steel structure.
- The retrofitting of failed members in steel structure is comparatively easy rather than R.C.C. structure.

Hence, preferring steel structure for high rise building in higher seismic zones rather than R.C.C. structure is a bit safer.

8. REFERENCES

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