DESIGN AND EVALUATION OF A STEEL STRUCTURE FOR GRADUAL COLLAPSE

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Article History: Received: 12.02.2023 **Revised:** 26.04.2023 **Accepted:** 10.06.2023

Abstract

Progressive collapse is a structure's entire or significant partial collapse after localized damage to a limited building area. Structures gradually collapse due to explosions, car accidents, fires, or other manufactured risks. The primary goal of the current research is to evaluate how steel constructions behave under unintended loads that might cause the structure to collapse gradually. The performance of the steel structure will be evaluated for sudden column loss as per the current guideline available for critical column removals like GSA or DOD. To study the behavior of steel building structures on the special moment resting frame (SMRF) under the progressive collapse G+10 structure is modelled in E-Tab (2018). In this work, the linear static (LS) analysis approach for single-column removal has been used to comprehend better the components taken into consideration, learn about progressive collapse, and get accurate findings for Demand-Capacity ratio (DCR), displacement of removal location, column's axial stress, notably in columns next to the obliterated column.

Keywords: Gradual Collapse, Apm, Dcr, Omrf, Lsa, Gsa, Fem, Etabs

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DOI: 10.31838/ecb/2023.12.s3.474

1. Introduction

The phrase "gradual/progressive collapse" has been used to describe how an initial local failure spreads in a way similar to a chain reaction that causes a structure to partially or completely collapse. Progressive collapse is characterized by a condition of failure that is disproportionately worse than the original failure.

Types of progressive collapse

Progressive collapse may be separated into different sections based on the cause of the progressivity, even though it is controlled in the design rules and norms as a single occurrence. The kind of building and the first incident determine the cause of the gradual collapse. Five different categories of gradual collapse are outlined below. Pancake, zipper, domino, instability, and section-type destruction are the given collapse modes.

Problem Statement

Restricted studies of progressive collapse on high rise multi-storey structure have been reported out of this lot of analytical approaches involve two dimensional analysis .in the present study special attention is given on simulating of structural response of three dimensional structures is considered for various type of framing system like OMRE In this study ,three dimensional (G+10) SMRF type of frames system are considered for all severe load combination with the structure having Demand-Capacity ratio (DCR/PMM) between 0.5 to 0.9 .the same model is analyses for progressive for progressive collapse guidelines which is specify by GSA (2013) .after analyzing model ,reading are taken on same model for nodal displacement , axial load, Demand-Capacity ratio in column bending moment & shear force in beam with &without progressive collapse ,and hence next step is to give remedial measure for this

2. Methodology

This technique does not include threats or missing members into the design process. It genuinely stipulates minimal standards of strength, continuity, and ductility to important structural parts, including implicit concerns to reduce gradual collapse. As a result, the structural system is deemed to be capable of withstanding a supposed anomalous loading if certain "minimum conditions" are met. Moreover, alternative pathways for the system to disperse its gravity loads should be available in the event that a crucial structural component fails. The goal of this technique is to develop extraneous structures that can handle all assumed loads. As a result, many building standards and specifications now include this strategy since it is thought to enhance structural response overall. Nevertheless, some academics have criticised this strategy for failing to give specific thought to how a building would behave when a crucial structural component is eliminated, which prevents a thorough understanding of progressive collapse prevention. The detection of tie forces is a need for this methodology's main feature. The Tie Force (TF) approach involves binding the building's structural components together. By needing links to hold the structural elements together in the case of anomalous loads, the method improves structural continuity, ductility, and redundancy. Several horizontal connections are needed for this, including internal ties, external ties, and ties to edge columns, corner columns, and walls. Moreover, load-bearing walls and columns need to have vertical ties installed. Fig. 2 shows the placement and direction of ties needed to keep structural parts together when they experience localised damage. It should be noted that this approach makes a lot of assumptions, therefore in order to ensure the method's safety, the empirical aspects need to be thoroughly examined.

Probable analysis and loading criteria for assessment of PC

The probable building collapse scenarios that should be taken into account should be complete and take into account all distinctive structural variations. It may have an impact on the choice on whether to design a PC with low or high potential. The following analysis scenarios should be utilised for framed structures with regular, predictable layouts and no exceptional structure combinations.

Exterior consideration

1. Study of the immediate damage of a column for a level above grade at or near the centre of the building's short and long sides.
2. Study of the unexpected collapse of a column at the corner of the structure, one storey above grade.

   Examine the possibility of an immediate loss of a column for a level above grade that is in or near the center of the building's short side. Look for the sudden disappearance of a column in your analysis for a floor above grade that is either in the middle or close to the center of the building. Investigate the possibility of a column in the corner of the structure, one storey above grade, suddenly collapsing.

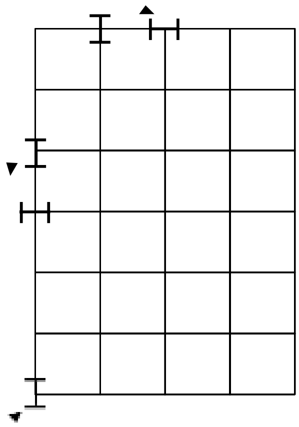


Figure 2.1 Plan view columns to be removed for assessment

Interior Considerations

The technique described in Assess for the sudden loss of 1 column that extends from the level of the subterranean parking area or uncontrolled public ground floor area to the next floor must be used by facilities that have underground parking and/or uncontrolled public ground floor areas (1 story). The column under consideration need to be within the lines of the surrounding columns.

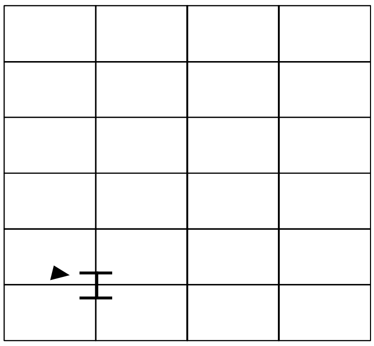


Figure 2.2 Plan view showing column to be removed interiorly

3. Modeling & Its details

For above objective a steel framed structure is modelled in ETABS 2016 software with following details

Table 3.1 Detail of Model Structures.

Type of structure	**Special moment resisting frame (SMRF)**
Number of stories	**Ground +10**
Location assumed	**Mumbai**
Average wind speed	**43m/s**
Soil type	**II**
Live load for floor &terrace floor	**2.5 KN/m^2**
Floor finish for floor &terrace floor	**1.25 KN/m^2**
Wall load on beams	**10 KN/m^2**
Concrete with unit weight	**25**
Steel with unit weight	**77.008**
Grade of steel used are	**Fy345Pa and E=200GPa**
Plan dimensions	**40X40 and 4X4**
Floor to floor height	**3m**
Height of the building	**30m**

Slab thickness assumed	**150mm**
Seismic zone	**Zone III**
Importance factor	**1**
Response reduction factor	**5**
Section used	**BOX and I-section**

Progressive collapse analysis by GSA 2013

According to GSA nonlinear analysis should be performed on high rise (greater than 10 story) building so as to consider the effect of dynamic impact due to sudden loss of element in the structure. since the pushdown analysis is performed in the current project with due consideration of P-delta effect .as discuss in the last chapter APM is the best suitable method for analysis against PC under removal of primary structure element on one level at a time. Typically, and at a minimum, exterior columns must be removed from buildings around the centre of the short side, the centre of the long side, and the corner. columns must also be removal at location where the plan geometry of the structure changes significantly such as abrupt decrease in bay size and re-entrant corners or at location where adjacent columns are lightly loaded the bays have different tributary sizes members frame in at different orientation or elevations and other similar situation in the current study we removed column at corner middle of the longer side, center column of the building at different floor location for this

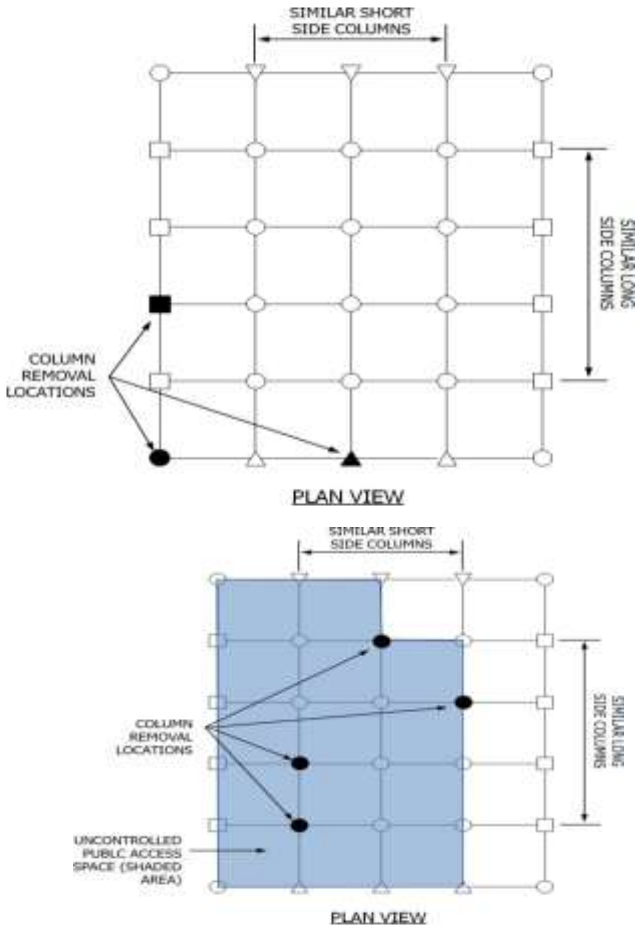


Figure 3.2 Possible locations for internal column removal case

we decided that AP is taken on ground fifth, ninth levels. As the results a very negligible variation in the member due to it higher stiffness at the collapse part, AP is taken frequent level. It is also advised that internal columns be removed from buildings with subterranean parking or other uncontrolled public ground floor sections around the center of the short side, the middle of the long side, and at the corner of the uncontrolled space. The removed column extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (i.e. a one-story height must be removed) Internal columns must also be removed at other critical location within the uncontrolled public access area, as determined with engineering judgment. for both external and internal column removal continuity must be retained across the horizontal elements that connect to ends of the column The following gures show the possible location for the column and the areas to be loaded with increased load as per GSA guideline

2. Results and Discussion

The various parameters are compared before and after sudden column strength loss by different analysis. The results obtained are discussed with due consideration of cases. In this current study, we performed the progressive collapse analysis using GSA-2013 guidelines for this the sudden removal of any column due to any abnormal loads which are mentioned earlier in chapter 1. For these various cases performed as explain for various parameter and their graphs are plotted under sudden column removal effect to study the behaviour of the model structures. If the DCR of the member goes beyond unity it shows red colour which means that particular member reaches its maximum strength (capacity). For the current chapter all the cases are combined for the purpose of comparisons and graphs are plotted and are mention under the same topic. For the ease in understanding the results are taken only for critical member in the structure which gives the best results for the progressive collapse load combination which was already defined in the model for analysis and design against PC according to GSA-2013

Case I :(C1) analysis for the sudden loss of a column situated at the corner of building
Case II.a: Column C1 Remove at ground floor,
Case I.b: Column C1 Remove at fifth floor
Case I. c: Column C1 Remove at ninth floor
Case II: (C6) analyses for the sudden loss of a column situated at the middle of the one of the directions (X direction in this case) of the building
Case II.a: Column C6 Remove at ground floor,
Case II.b: Column C6 Remove at fifth floor
Case II.c: Column C6 Remove at ninth floor
Case III: (C61) analyses for the sudden loss of a column situated at or near middle removal at any suitable location should be carried out for building in these case column next to middle position.
Case III.a: Column C61 Remove at ground floor,
Case III.b: Column C61 Remove at fifth floor
Case III.c: Column C61 Remove at ninth floor
Case I.b: Column C1 Remove at fifth floor

Table 4.1 Demand-Capacity ratio of column –C1

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.788	0.318	0.302	0.285
2	0.758	0.307	0.307	0.392
3	0.758	0.253	0.253	0.415
4	0.725	0.226	0.226	0.384
5	0.674	0	0	0
6	0.632	1.675	1.675	0.267
7	0.542	1.301	1.016	0.355
8	0.466	1.630	1.599	0.337
9	0.391	1.281	1.073	0.245
10	0.183	2.875	1.877	0.124

Table 4.2 Demand-Capacity ratio of column –C2

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.856	0.777	0.757	0.705
2	0.808	0.727	0.726	0.65
3	0.721	0.673	0.673	0.606
4	0.812	0.753	0.749	0.73
5	0.848	1.094	0.946	0.720
6	0.744	1.011	0.964	0.538
7	0.800	1.272	1.090	0.575
8	0.604	1.31	1.203	0.419

9	0.479	0.826	0.775	0.27
10	0.217	1.919	0.951	0.136

Table 4.3 Demand-Capacity ratio of column –B1

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.803	1.21	0.137	1.21
2	0.803	1.096	1.096	1.107
3	0.819	1.118	1.118	1.152
4	0.834	1.140	1.14	1.152
5	0.720	1.050	1.05	1.0135
6	0.720	0.983	0.883	0.983
7	0.772	1.056	1.056	1.055
8	0.772	1.055	1.055	1.055
9	0.772	1.059	1.06	1.055
10	0.685	0.834	0.834	0.835

Table 4.4 Axial Force of column- C1

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	-1040.5063	-620.5856	-623.5183	-640.7281
2	-931.0112	-464.3895	-464.3895	-858.8703
3	-821.5162	-308.1935	-908.1935	-673.0237
4	-712.2352	-153.3979	-153.3979	-391.7032
5	-606.1648	0	0	0
6	-498.1661	-9.5125	-9.5387	-286.7679
7	-390.1673	-21.9662	-22.0184	-394.3137
8	-281.8779	-35.4092	-35.4862	-366.7709
9	-173.1885	-48.6746	-48.7914	-237.2626
10	-64.6991	-62.127	-62.102	-88.7571

Table 4.5 Axial Force of column- C2

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	-1781.7711	-2114.036	-2116.9064	-2006.8265
2	-1596.5924	-1929.607	-1929.5443	-1833.3604
3	-1411.41370	-7545.1777	-1745.1150	-1710.0457
4	-1225.89290	-1560.26	-1560.1974	-1643.0009
5	-855.78710	-1375.3699	-1375.3073	-1236.2441
6	-670.60330	-1132.7407	-1132.6885	-915.4437
7	-670.60330	-891.1612	-8091.1194	-680.4272
8	-486.94420	-651.1035	-651.0721	-482.8928
9	-302.79310	-410.6449	-410.624	-294.4447
10	-118.15350	-169.788	-169.7776	-112.9428

Table 4.6 Maximum Bending Moments of Beam-B1

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	54.4072	73.9029	75.2777	73.9029

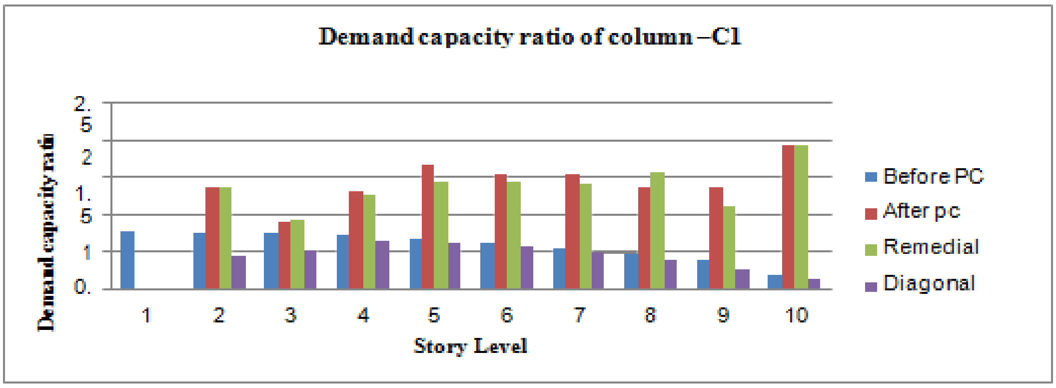
2	54.4072	73.9029	73.9029	73.9029
3	55.0499	74.8208	74.8208	74.8208
4	55.6918	75.7407	75.7407	75.7404
5	56.4632	79.5636	79.5636	79.5636
6	56.4632	76.7669	76.7669	76.7669
7	57.0263	77.6292	77.6292	76.6292
8	57.0263	77.6292	77.6292	77.6292
9	57.0263	77.9292	77.6292	77.6292
10	36.4883	44.3425	44.3425	44.3425

Table 4.7 Shear Force of Beam –B1

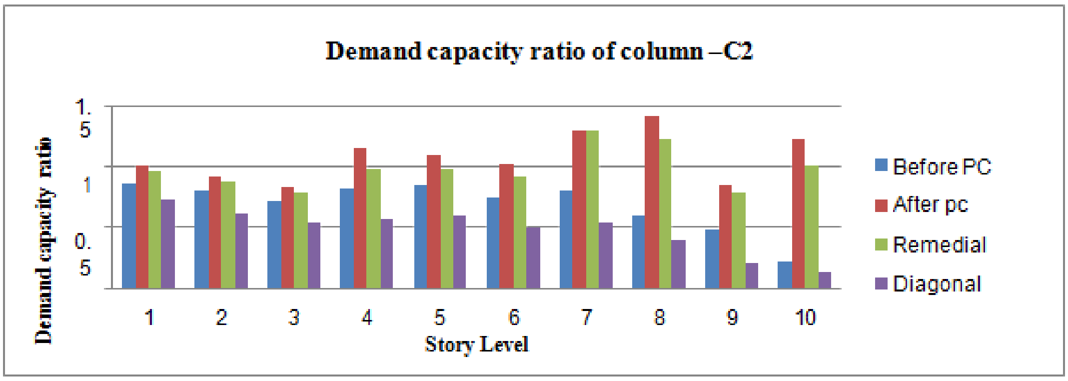
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	51.4435	73.352	74.8784	73.352
2	51.4435	73.352	73.352	73.352
3	51.7856	73.8403	73.8403	73.8403
4	52.1259	74.3771	74.3271	74.3271
5	52.3247	76.1461	76.1461	76.1461
6	52.3247	74.6937	74.6926	74.6926
7	52.3011	74.8799	74.8799	74.8799
8	52.3011	74.8799	74.8799	74.8799
9	52.3011	74.8799	74.8799	74.8799
10	31.1945	39.2711	39.2711	29.2711

Table 4.8 Story Drifts

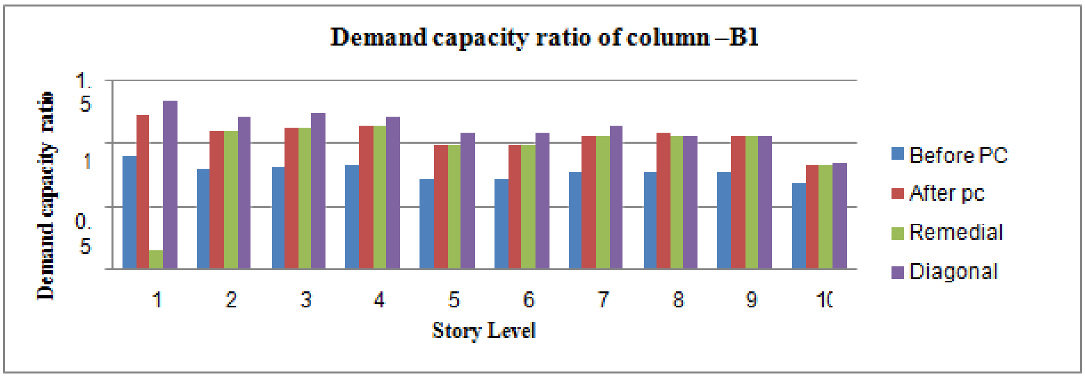
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.006394	0.000331	0.000298	0.000065
2	0.019369	0.001333	0.00116	0.000056
3	0.031735	0.002364	0.002639	0.000028
4	0.042882	0.003489	0.003309	0.000061
5	0.055461	0.005115	0.004929	0.000267
6	0.071067	0.007516	0.007402	0.000187
7	0.087048	0.010396	0.010204	0.00021
8	0.102665	0.013907	0.013714	0.00015
9	0.117916	0.018690	0.018496	0.000138
10	0.125788	0.022670	0.022476	0.000142



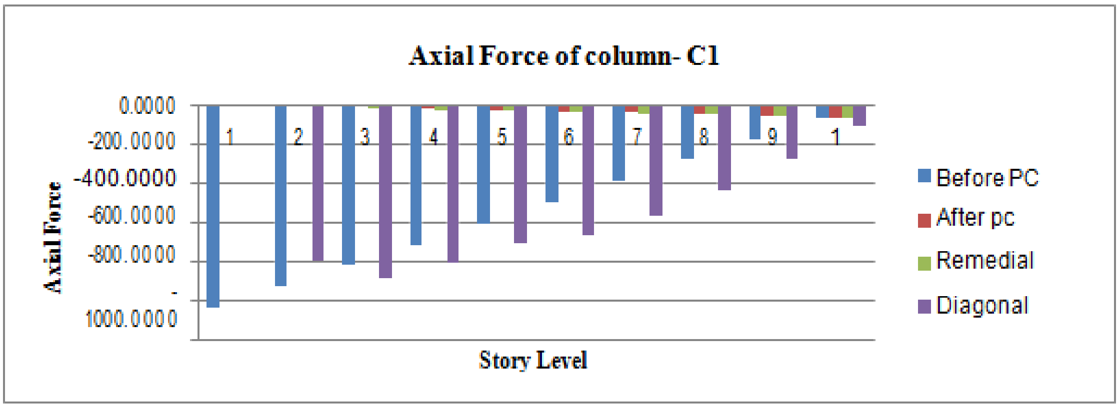
a. Demand-Capacity ratio of column –C1



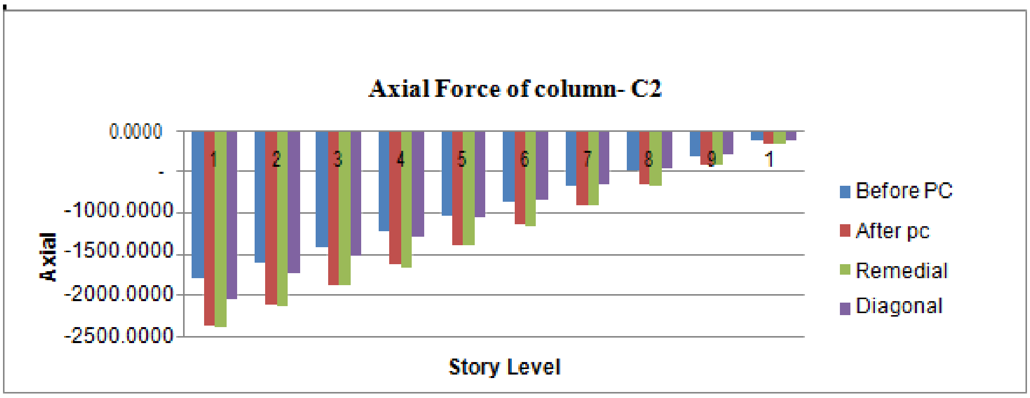
b. Demand-Capacity ratio of column –C2



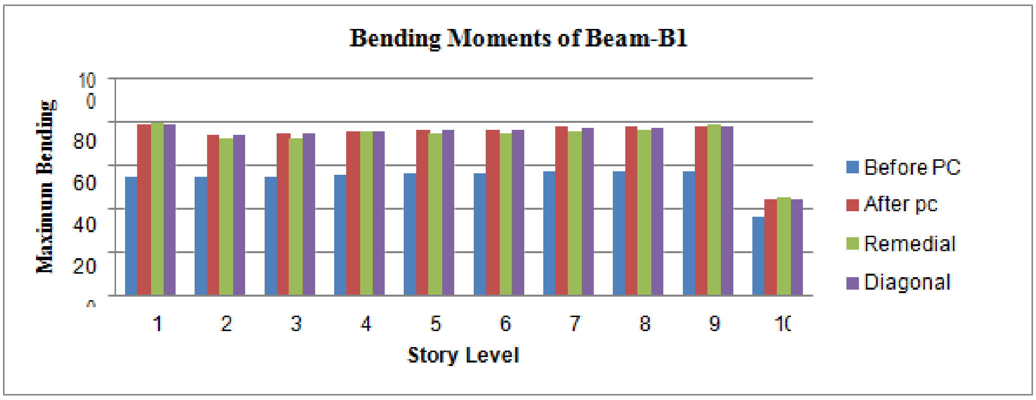
c. Demand-Capacity ratio of column –B



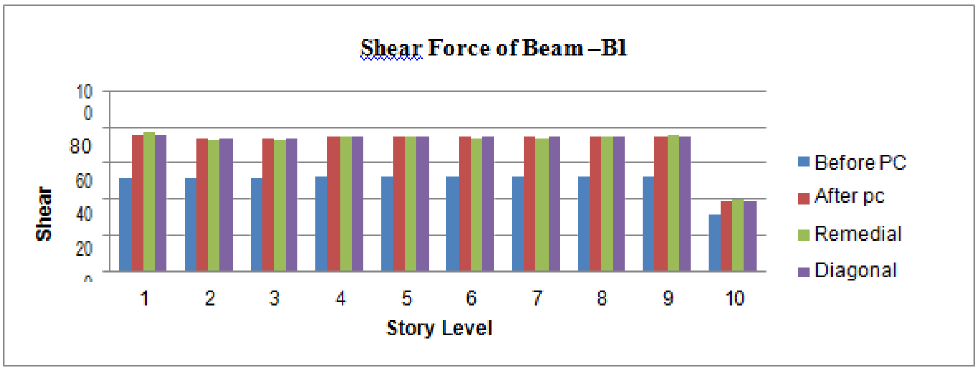
d. Axial Force Column C1



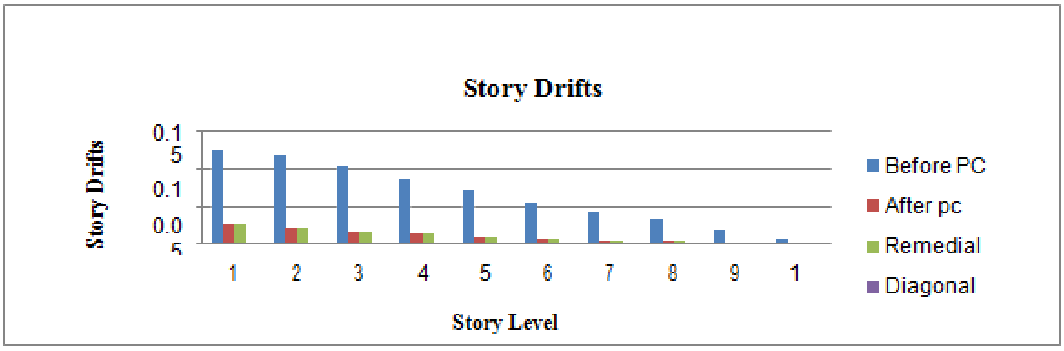
E. Axial Force Column C2



F. Maximum Bending Moment of beam B1



g. Shear Force of Beam B1



h. Story Drift

Graph 4.1 (a-h) comparisons of various parameters for removal of column C1 at fifth floor

In this case we perform the progressive collapse analysis using GSA-2013 guidelines for this the sudden removal of any column due to abnormal load, in this particular case the corner column of the first story was failed then the failure patterns of structure from local failure to global failure After failure structure we are using same remedial to increases failure time

i.e. decreases the DCR i.e. complete failure was invested.. We can use linear static analysis. The DCR increases when remove column C1 at 5st story for linear static which means structure fails at column and beam position.

The DCR is the ratio of load coming on the element to the ultimate capacity of the element. The structure member is safe if the DCR is below 1. And it said assumed failed when the ratio exceeds the limit of unity. Extent of damage can be quantifiable by observing the DCR values of members. DCR of adjoining structural members to removed column can be column are calculated using linear static method for column strength loss cases considered as per GSA guideline. Using remedial after removal of column C1 column the DCR of critical column is changed in case of LSA analysis. This means after using remedial frames and diagonal bracing are capable of taking load up to certain limit before collapse. So it is concluded that remedial frame and diagonal bracing are stronger as compared to normal frame. But as compared to the remedial and diagonal bracing system is stronger. Also from the graph 4.1 (a, b & c) it is observed that effect of column strength loss on the beam go on decreasing for beam at upper level DCR values for exterior column strength loss scenario are less because of the fact that external beam contribute to less slab area as compare to internal beam. The change in bending moments of beams observed helps to conclude the above statements. The bending moment of beams go on decreasing at higher levels for three column strength loss cases considered. Hence the DCR values of beams go on decreasing. Graph 4.1 (f) shows the comparison of bending moments when column strength loss takes place at ground level. Comparison of magnitudes of the bending moment of beam immediate above removal column is summarized in table. It is observed that at near starting point of the beam, the shear force changes its nature and increases in magnitude whereas the shear force increases considerably after column strength loss suddenly but does not changes its nature. Though shear force changes its nature of increases considerably after column strength loss suddenly it does not lead to failure of the member, because sections used have sufficient capacity to resist shear force increased. It has been observed that there is no effect on shear force of beams for column strength loss at different level. Also we concluded in the above graph 4.1(d ,e & f) there Is change in axial force and bending moment as in axial force when we removed the critical column there is drastic decrease in axial force at the critical column whereas in other column there is increase in axial force. Whereas in bending moment case there is increase in moment in clockwise direction for all adjoin beams near the critical column linear static analysis.

Case III.a: Column C61 Remove at fifth floor

Table 4.9 Demand-Capacity ratio of column –C61

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.807	0.234	0.06	0.237
2	0.919	0.231	0.231	0.231
3	0.791	0.154	0.154	0.154
4	0.866	0.092	0.092	0.092
5	0.865	0	0	0
6	0.879	0.04	0.004	0.04
7	0.884	0.015	0.015	0.016
8	0.792	0.033	0.032	0.032
9	0.652	0.061	0.061	0.061
10	0.23	0.078	0.078	0.078

Table 4.10 Demand-Capacity ratio of column –C60

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.807	0.481	0.476	0.481
2	0.919	0.577	0.577	0.577
3	0.791	0.527	0.527	0.523
4	0.866	0.586	0.586	0.578
5	0.865	0.751	0.751	0.648
6	0.879	0.84	0.841	0.798
7	0.884	0.934	0.934	0.906
8	0.792	1.024	1.024	1.024
9	0.652	1.021	1.021	1.021
10	0.23	1.522	1.522	1.522

Table 4.11 Demand-Capacity ratio of beam –B55

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.837	1.087	0.138	1.087
2	0.837	1.087	0.991	1.087
3	0.837	0.991	0.991	0.991
4	0.837	0.991	0.991	0.991
5	0.703	0.925	0.925	0.925
6	0.628	0.863	0.863	0.863

7	0.698	0.823	0.827	0.827
8	0.719	0.853	0.854	0.853
9	0.741	0.879	0.879	0.879
10	0.862	0.978	0.978	0.979

Table 4.12 Axial Force of column- C61

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	-2991.8246	-1551.5661	-1591.7052	-1551.5661
2	-2680.3946	-1162.4221	-1162.4221	-1162.4221
3	-2370.7491	-774.7057	-774.7057	-774.7057
4	-2661.1036	-386.9893	-386.9893	-386.9893
5	-1752.3770	0	0	0
6	-1443.5687	-11.6407	-11.6419	-11.6405
7	-1135.3963	-37.4759	-37.4752	-37.4761
8	-823.2353	-62.521	-62.521	-62.5204
9	-521.5664	-87.1728	-87.1714	-87.1714
10	-215.3916	-111.8438	-11.8443	-111.8443

Table 4.13 Axial Force of column- C60

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	-2991.8246	-3162.5610	-3160.7394	-2162.561
2	-2680.3946	-2879.217	-2879.217	-2879.217
3	-2370.7491	-2597.3006	-2597.3006	-2597.3006
4	-2661.1036	-2315.3842	-2315.3842	-2315.3842
5	-1752.3770	-2034.1949	-2034.1949	-2037.1949
6	-1443.5687	-1677.3485	-1677.2485	-1677.2485
7	-1135.3963	-3224.4574	-1324.4574	-1324.4574
8	-823.2353	-972.4723	-972.4723	-972.4723
9	-521.5664	-620.8825	-620.8825	-620.8825
10	-215.3916	-269.6879	-269.6879	-269.6879

Table 4.14 Maximum Bending Moments of Beam-B55

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	78.0415	90.0328	83.5049	92.0328
2	78.0415	92.0328	82.0328	92.0328
3	78.0415	92.0328	92.0328	92.0328
4	78.0415	92.0328	90.9017	92.0328
5	80.0351	101.2802	101.2802	101.2802
6	81.8833	96.7298	96.7298	96.7298
7	81.7979	96.6614	96.6614	96.6614
8	83.6553	98.975	98.975	98.975
9	85.5229	101.3081	101.3081	101.3081
10	63.9933	72.1602	72.1602	72.1602

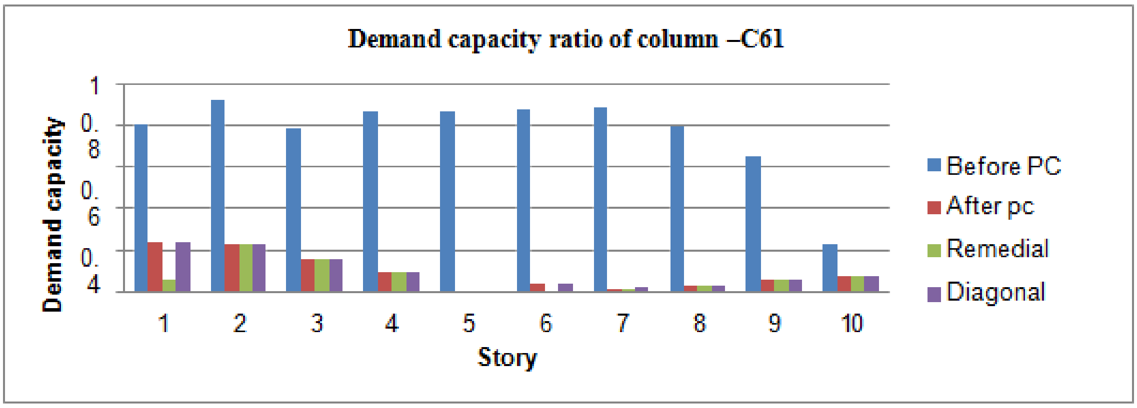
Table 4.15 Shear Force of Beam –B55

Story	Before PC	After pc	Remedial	Diagonal Bracing
1	73.0370	90.4789	90.1722	90.4789
2	73.0370	90.4789	90.4889	90.4789
3	73.0370	90.4789	90.4749	90.4789
4	73.0370	90.4789	90.4789	90.4789
5	73.6920	95.0789	95.0789	95.0789

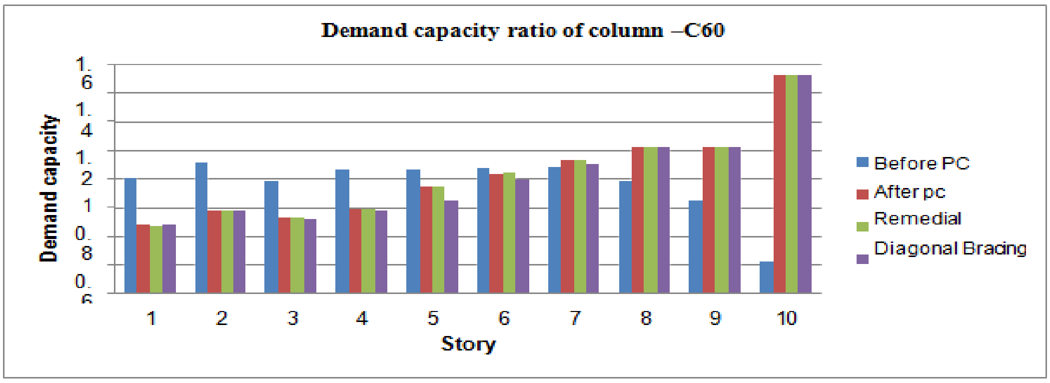
6	74.1606	92.203	92.223	92.223
7	74.0794	92.1501	92.1501	92.1501
8	74.5227	92.9347	92.9347	92.9347
9	74.9619	93.7076	93.7076	93.7076
10	53.0557	61.3736	61.3736	61.3736

Table 4.16 Story Drifts

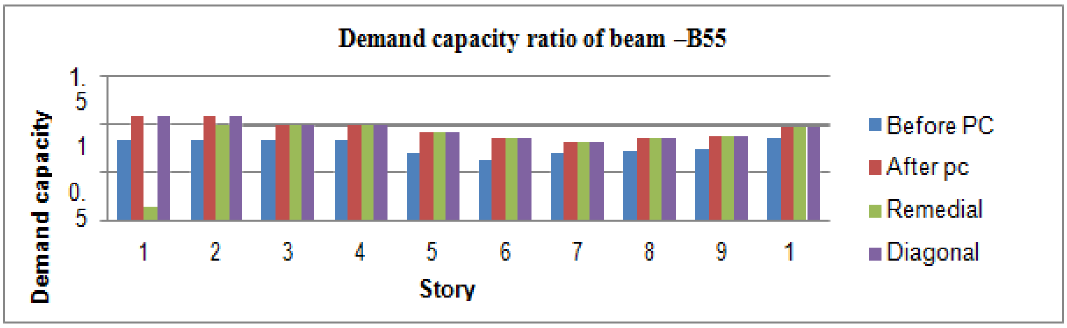
Story	**Before PC**	**After pc**	**Remedial**	**Diagonal Bracing**
1	0.006394	0.000001	0.000001	0.000041
2	0.019369	0.000001	0.000002	0.000032
3	0.031735	0.000002	0.000002	0.0000014
4	0.042882	0.000004	0.000004	0.000003
5	0.055461	0.000004	0.000005	0.000006
6	0.071067	0.000005	0.000006	0.000008
7	0.087048	0.000008	0.000008	0.000011
8	0.102665	0.000009	0.000009	0.000001
9	0.117916	0.000011	0.000011	0.000012
10	0.125788	0.000014	0.000014	0.000016



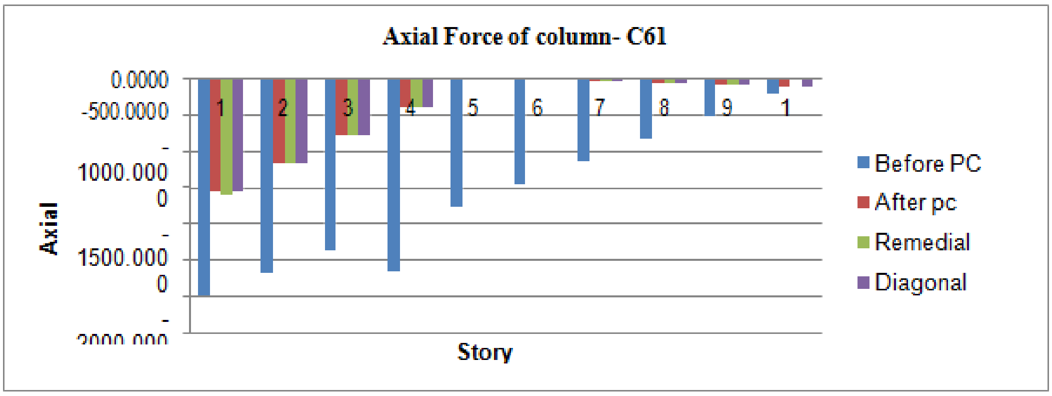
a . Demand-Capacity Ratio of Column C61



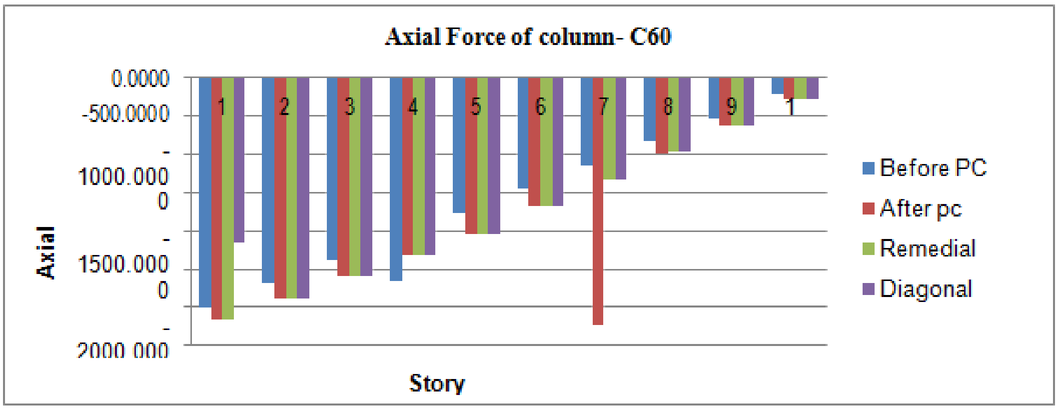
b. Demand-Capacity Ratio Of Column C60



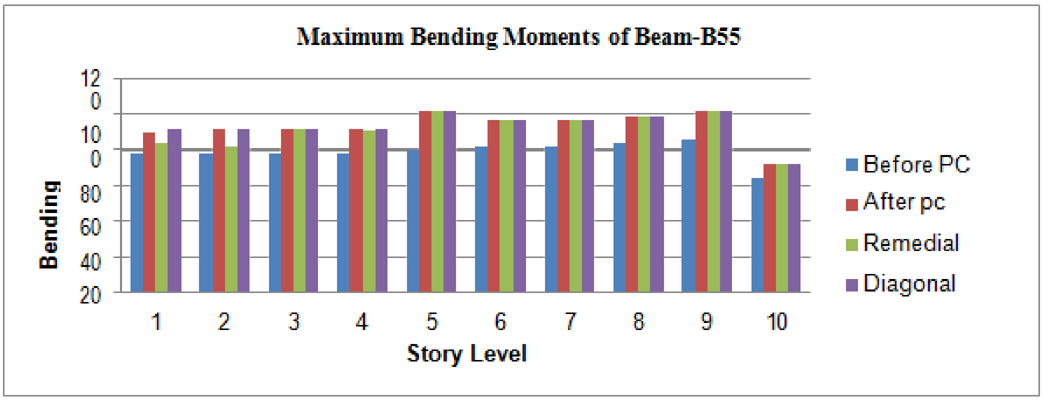
C. Demand-Capacity Ratio Of Beam B55



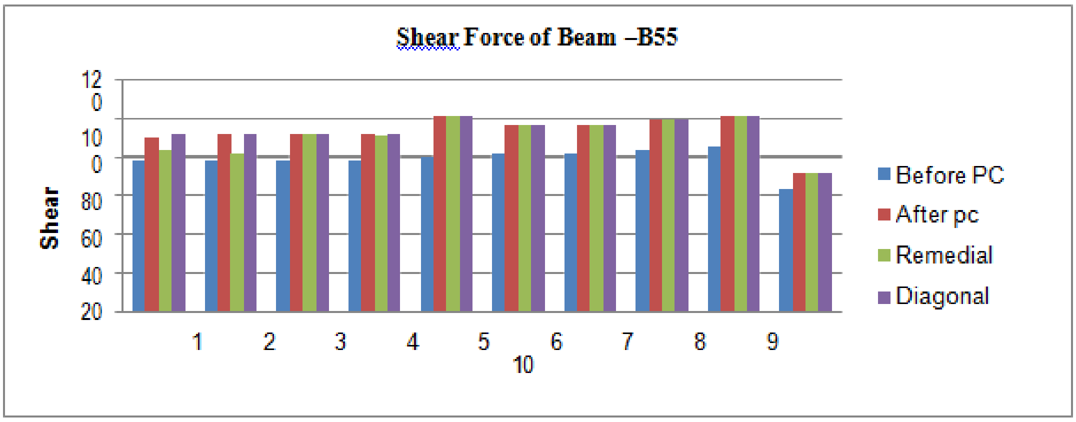
d. Axial Force of Column C61



e. Axial Force of Column C60

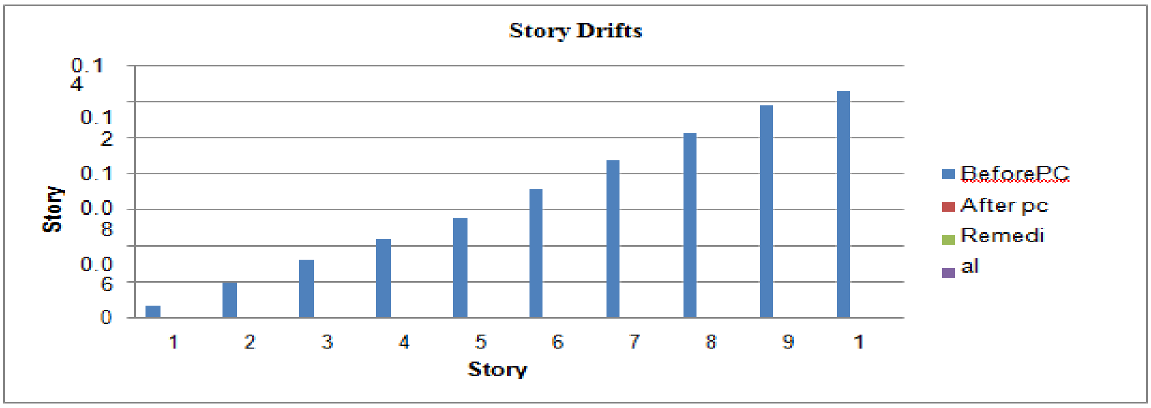


f. Maximum Bending Moments of Beam B55



g. Shear Force of Beam B55

h.



h. Story Drift

Graph 4.2 (a-h) comparisons of various parameters for removal of column C61 at fifth floor

In this case we perform the progressive collapse analysis using GSA-2013 guidelines for this the sudden removal of any column due to abnormal load, in this particular case the corner column of the first story was failed then the failure patterns of structure from local failure to global failure i.e. complete failure was investigated. After failure structure we are using same remedial to increases failure time i.e. decreases the DCR i.e. .complete failure was invested. We can use linear static analysis. The DCR increases when remove column C61 at 5st story for linear static which means structure fails at column and beam position. The DCR is the ratio of load coming on the element to the ultimate capacity of the element. The structure member is safe if the DCR is below 1. And it said assumed failed when the ratio exceeds the limit of unity. Extent of damage can be quantifiable by observing the DCR values of member s. DCR of adjoining structural members to removed column can be column are calculated using linear static method for column strength loss cases considered as per GSA guideline. Using remedial after removal of column C61 column the DCR of critical column is changed in case of LSA analysis. This means after using remedial frames and diagonal bracing are capable of taking load up to certain limit before collapse. So it is concluded that remedial frame and diagonal bracing are stronger as compared to normal frame. But as compared to the remedial and diagonal bracing system is stronger. Also from the graph 4.2 (a, b & c) it is observed that effect of column strength loss on the beam go on decreasing for beam at upper level DCR values for exterior column strength loss scenario are less because of the fact that external beam contribute to less slab area as compare to internal beam. The change in bending moments of beams observed helps to conclude the above statements. The bending moment of beams go on decreasing at higher levels for three column strength loss cases considered. Hence the DCR values of beams go on decreasing. Graph 4.2 (f) shows the comparison of bending moments when column strength loss takes place at ground level. Comparison of magnitudes of the bending moment of beam immediate above removal column is summarized in table no.4.12 It is observed that at near starting point of the beam, the shear force changes its nature and increases in magnitude whereas the shear force increases considerably after column strength loss suddenly but does not changes its nature. Though shear force changes its nature of increases considerably after column strength loss suddenly it does not lead to failure of the member, because sections used have sufficient capacity to resist shear force increased. It has been observed that there is no effect on shear force of beams for column strength loss at different level. for comparison of all the parameters Table5.77 The axial force and bending moment in the aforementioned graph 4.2(d & e) both changed as a result of the removal of the crucial column, with the axial force at the critical column drastically decreasing while increasing in the other columns. Whereas in the case of bending moments, all adjacent beams close to the critical column experience an increase in moment in a circular manner. static linear analysis

3. Conclusions

1. Based on the findings, it is concluded that gradual collapse in diagonal braced systems has a better impact than increasing the size of beams and columns at crucial locations.
2. When the number of stories rises, the influence of progressive collapse lessens because there are more members to carry the dispersed load. As a result, the DCR values of the beams go down for the higher levels, indicating that more failures occur in the vicinity of the removed column.
3. Although DCR values for columns continue to rise as they approach higher levels, DCR values for beams continue to decline.
4. It has been noted that the impact of progressive collapse was greater when a corner column was abruptly removed; however, as the number of stories rises, the effect of progressive collapse diminishes since there are more members available to bear the dispersed weight.
5. Failure may be partial or complete, although shear fore is not the cause; rather, it is the result of an increase in the beam's bending moment caused by the redistribution of loads on the deleted region location (strong column & weak beam)
6. The strain on the neighbouring column increases due to the removal of the column, but the same column loses strength on subsequent levels, making the impact more dangerous when it happens suddenly on higher levels.
7. With any multi-story high rise building, stiffness and strength are more crucial, making it feasible to add bracing to increase this property of the structure.

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