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Progressive Collapse Analysis and design of Steel Structure

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Abstract - Progressive collapse is defined as total or remarkable partial collapse of structure following local damage at a small portion of the building. Progressive collapse of structures is due to explosion, vehicle impact, fire, or other man-made hazards etc. The main aim of the present study is to assess the behaviour of steel structure under accidental load which may lead to progressive collapse of complete structure .Performance of steel structure will be evaluated for sudden column loss as per the present guideline available for critical column removal like GSA or DOD. In order to study the behaviour of steel building structure on the special moment resting frame (SMRF) under the progressive collapse G+10 structure is modelled in E-Tab (2018). in order to know about progressive collapse and to obtain reliable results, linear static (LS) analyses procedure for single column removal have been implemented in this study for better understanding factors considered in the study. For demand capacity ration (DCR), displacement of removal location, axial load in the column specially columns adjacent to removed column

Key Words: ***PROGRESSIVE COLLAPSE, APM , DCR ,OMRF ,LSA ,GSA,FEM ,ETABS***

1. INTRODUCTION

The term progressive collapse has been used to describe the spread of an initial local failure in a manner analogous to a chain reaction that leads to partial or total collapse of a building. The underlying characteristic of progressive collapse is that the state of failure is disproportionately greater than the initial failure.

1.1 Types of progressive collapse

Even though progressive collapse is managed in the design rules and norms as one event it can be divided into several parts depending on the reason for the progressivity. The reason that causes the progressive collapse depends on the type of structure and the initiating event. Below five types of progressive collapse will be described. The presented collapse modes are pancake, zipper, domino, instability, and section-type destruction.

1.2 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 ,demand capacity ratio ,axial load 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

In this methodology, neither missing members nor threat is considered in the design. It actually places implicit considerations to mitigate progressive collapse by stipulating minimum requirements of strength, continuity and ductility to key structural members. Therefore, theoretically, if these "minimum requirements" are fulfilled, the structural system is considered to be able to withstand a presumed abnormal loading. Also, if a key structural element happens to fail, alternate paths should be possible for the system to redistribute its gravity loads. The intent of this method is to create superfluous structure that can withstand any presumed loadings, which induced many building codes and specifications to integrate this approach, as it is believed to improve overall structural response. However, some researchers have criticized this approach since it does not provide a special consideration on the behavior of a structure when a key structural element is removed, which is not conducive to a clear idea on progressive collapse prevention. The principle feature of this methodology requires the identification of tie forces. It consists of tying the structural elements of the building, which is known as the Tie Force (TF) method.

Method enhances the continuity, ductility, and structural redundancy by requiring ties to keep the components of the structure together in the event of an abnormal loading. This requires several horizontal ties, including: internal ties, peripheral ties and ties to edge columns, corner columns and walls. As well, vertical ties must be provided in columns and load-bearing walls. The location and direction of ties that are required to hold structural elements together when they are subjected to localized damage are illustrated in Fig. 2. It should be mentioned here that, as a number of Assumptions are involved in this method; the empirical factors need to be carefully checked, in Order to assure the method's safety

2.1 Probable analysis and loading criteria for assessment of PC

The possible scenarios to be considered of potential for progressive collapse of building should be sufficient in member and includes all unique structural differences. that could probably affect the outcome of decision regarding low or high potential of building for PC For framed structural facilities that have a relatively simple, uniform and repetitive layout with no atypical structure configurations the following analysis scenarios be used

2.1.1Exterior consideration

1. Analysis for the instantaneous loss of a column for one floor above grade located at or near the middle of the short and long side of the building.

2. Analysis for the instantaneous loss of a column for one floor above grade located at the corner of the building.

Analyze for the instantaneous loss of a column For one floor above grade located at or near the middle of the short side of the building. Analyze for the instantaneous loss of a Column For one floor above grade located at or near the Middle of the building long lading .Analyze for the instantaneous loss of a column for one floor above grade located at the corner of the building.

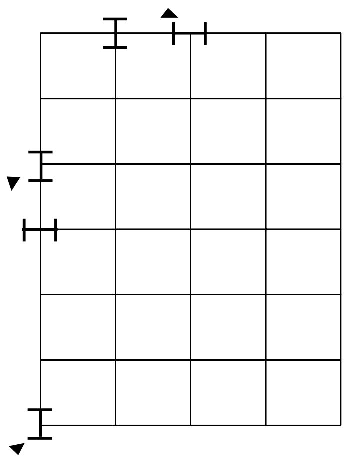


Figure 2.1 Plan view columns to be removed for assessment

2.1.2 Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case(s) in the procedure outlined in Analyze for the instantaneous loss of 1 column That extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (1 story). The column considered should be interior to the perimeter column lines.

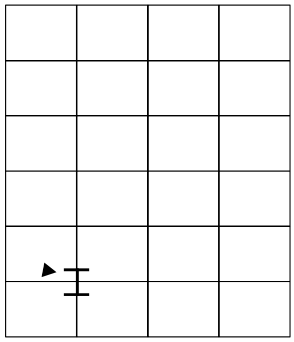


Figure 2.2 Plan view showing column to be removed interiorly

3. Modeling & its details

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

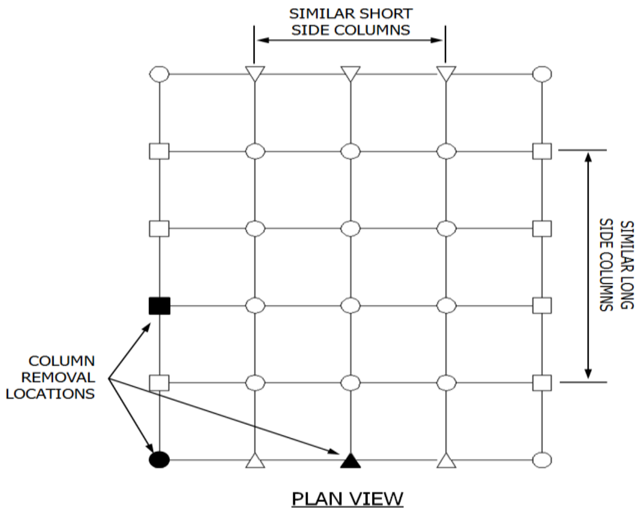
Table 3.1 Detail of Model Structures.

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**

3.2 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.

Generally and as a minimum external columns must be removal near the middle of the short near the middle of the long side and at the corner of the building .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 ,centre column of the building at different floor location .for this 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. For structure with underground parking or other uncontrolled public ground floor areas, it is also recommended that internal column be removed near the middle of short side, near 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 figures show the possible location for the column and the areas to be loaded with increased load as per GSA guideline



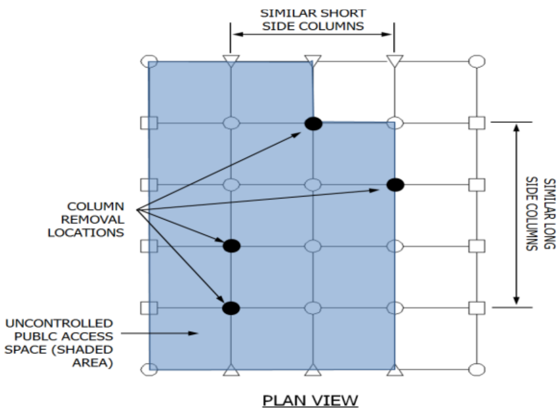


Figure 3.2 Possible locations for internal column removal case

4.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 this 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 1 :(C1) analysis for the sudden loss of a column situated at the corner of building

Case 1.a: Column C1 Remove at ground floor , Case 1.b: Column C1 Remove at fifth floor
Case 1.c: Column C1 Remove at ninth floor

Case 2: (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 2.a: Column C6 Remove at ground floor , Case 2.b: Column C6 Remove at fifth floor
Case 2.c: Column C6 Remove at ninth floor

Case 3: (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 3.a: Column C61 Remove at ground floor , Case 3.b: Column C61 Remove at fifth floor
Case 3.c: Column C61 Remove at ninth floor

Case 1.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

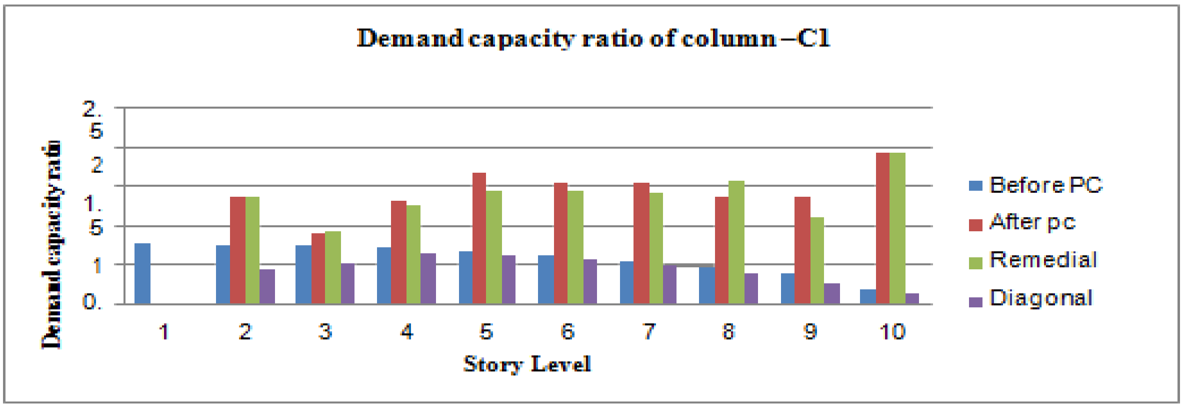
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	54.4072	73.9029	75.2777	73.9029
2	54.4072	73.9029	73.9029	73.9029
3	55.0499	74.8208	74.8208	74.8208
4	55.6918	75.7407	75.7407	75.7404
5	56.4632	79.5636	79.5636	79.5636
6	56.4632	76.7669	76.7669	76.7669
7	57.0263	77.6292	77.6292	76.6292
8	57.0263	77.6292	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

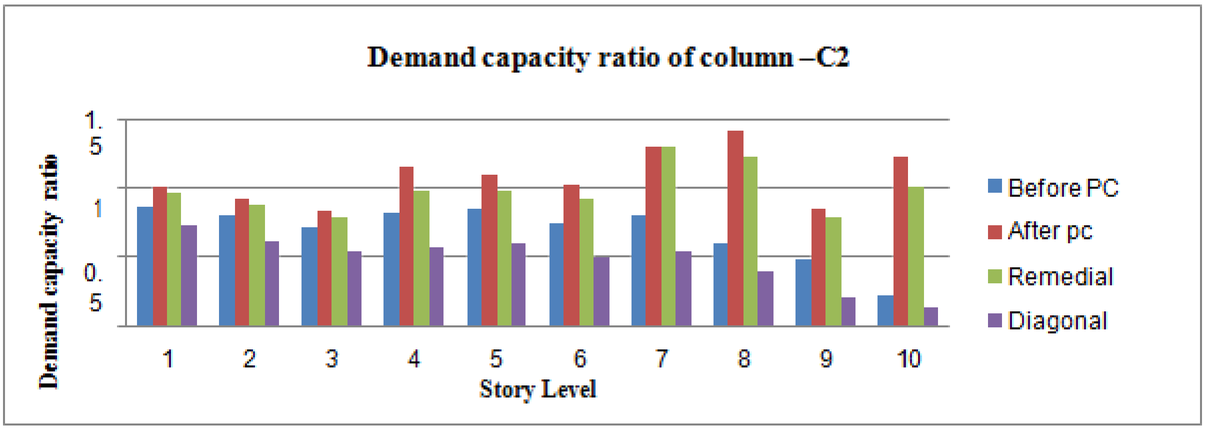
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	51.4435	73.352	74.8784	73.352
2	51.4435	73.352	73.352	73.352
3	51.7856	73.8403	73.8403	73.8403
4	52.1259	74.3771	74.3271	74.3271
5	52.3247	76.1461	76.1461	76.1461
6	52.3247	74.6937	74.6926	74.6926
7	52.3011	74.8799	74.8799	74.8799
8	52.3011	74.8799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

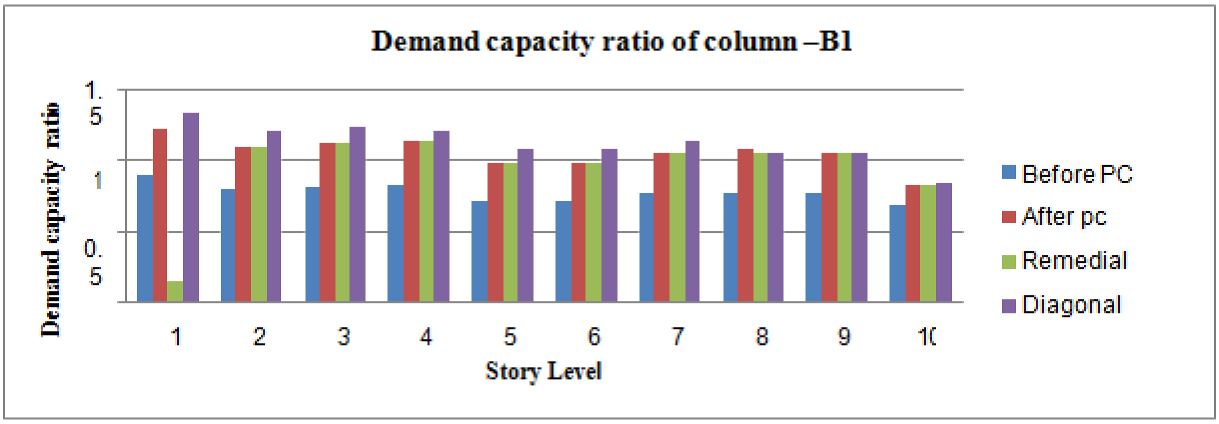
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.006394	0.000331	0.000298	0.000065
2	0.019369	0.001333	0.00116	0.000056
3	0.031735	0.002364	0.002639	0.000028
4	0.042882	0.003489	0.003309	0.000061
5	0.055461	0.005115	0.004929	0.000267
6	0.071067	0.007516	0.007402	0.000187
7	0.087048	0.010396	0.010204	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



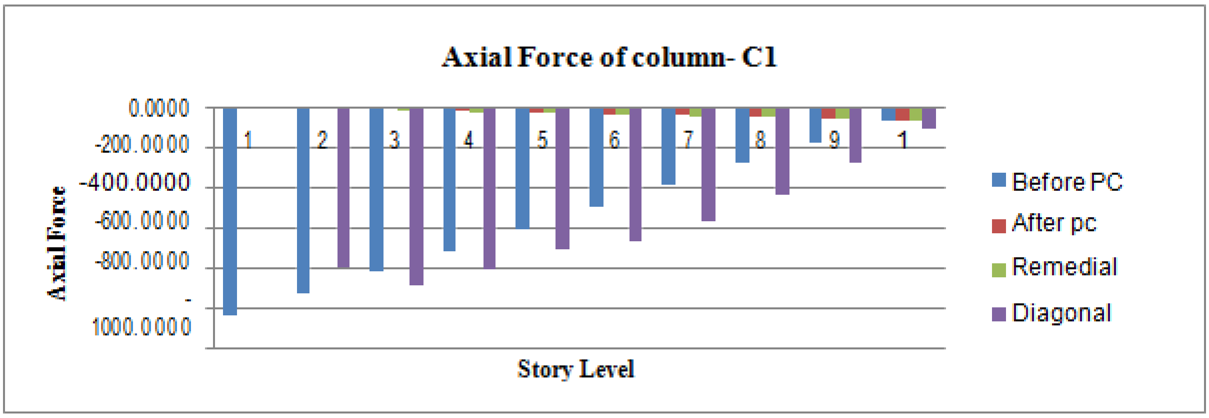
a. Demand capacity ratio of column –C1



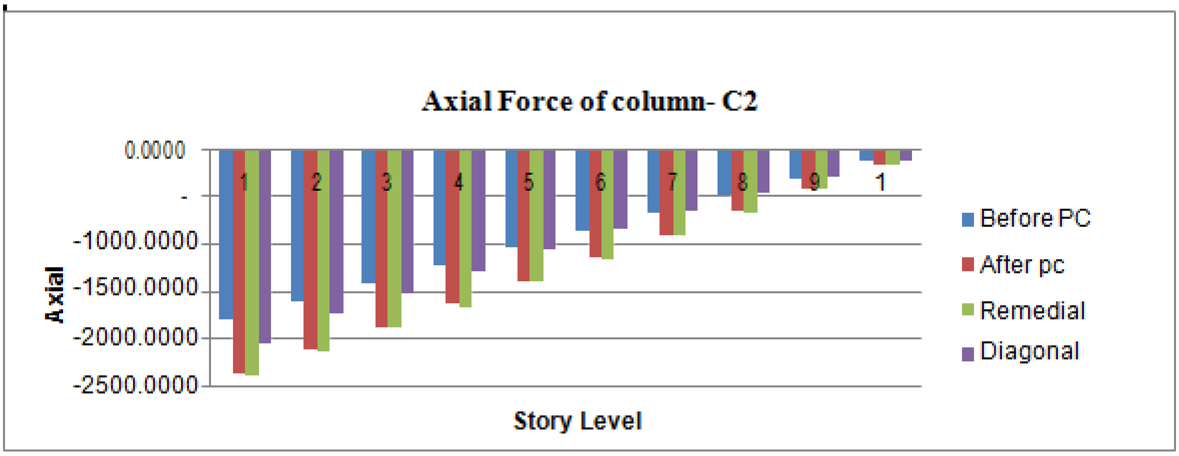
b. Demand capacity ratio of column –C2



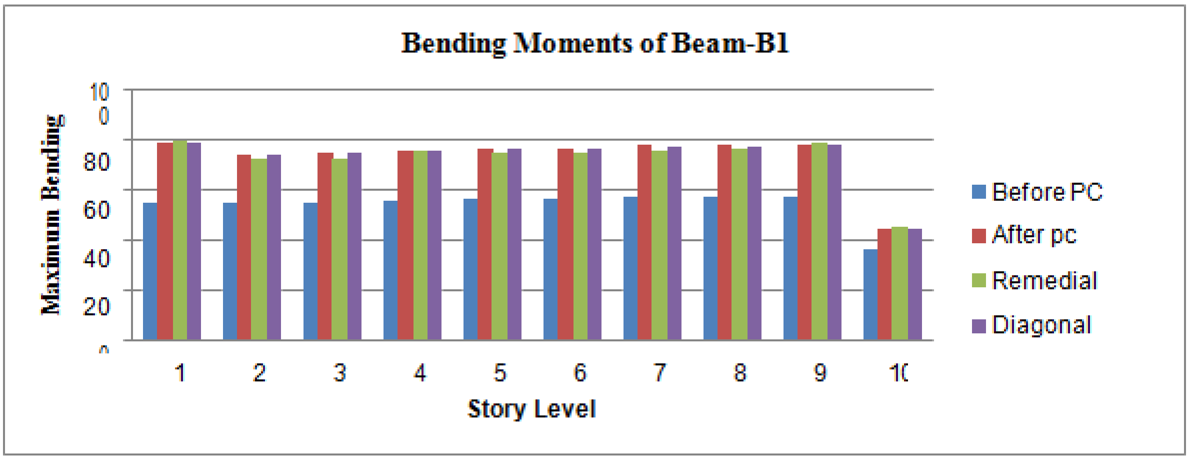
c. Demand capacity ratio of column –B



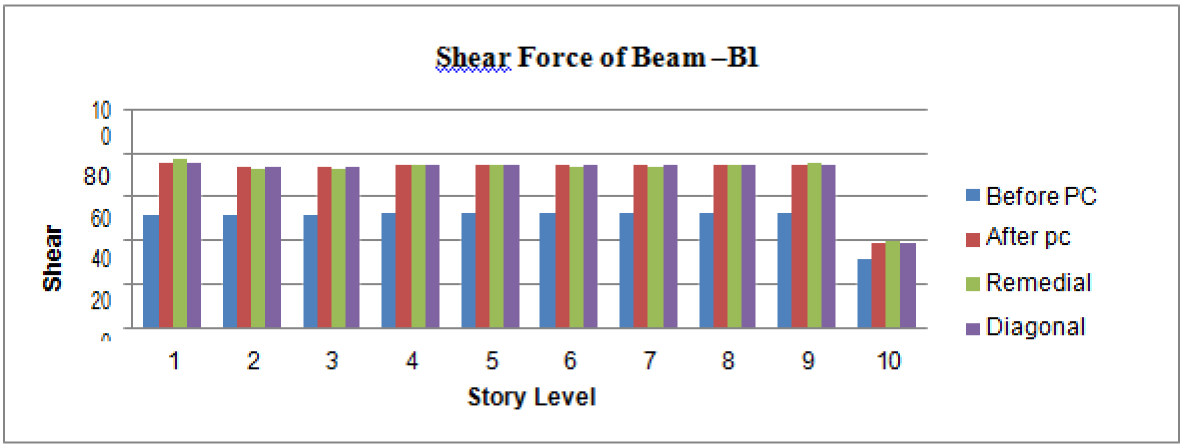
d. Axial Force Column C1



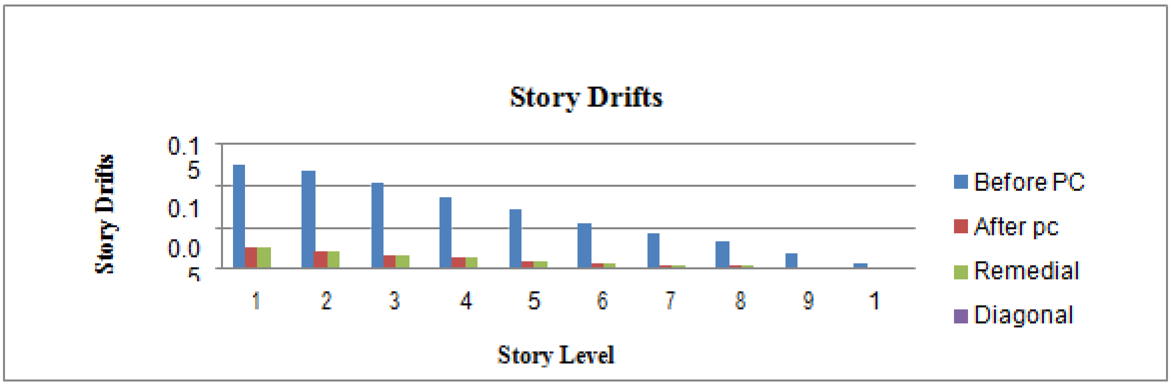
E. Axial Force Column C2



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 3.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

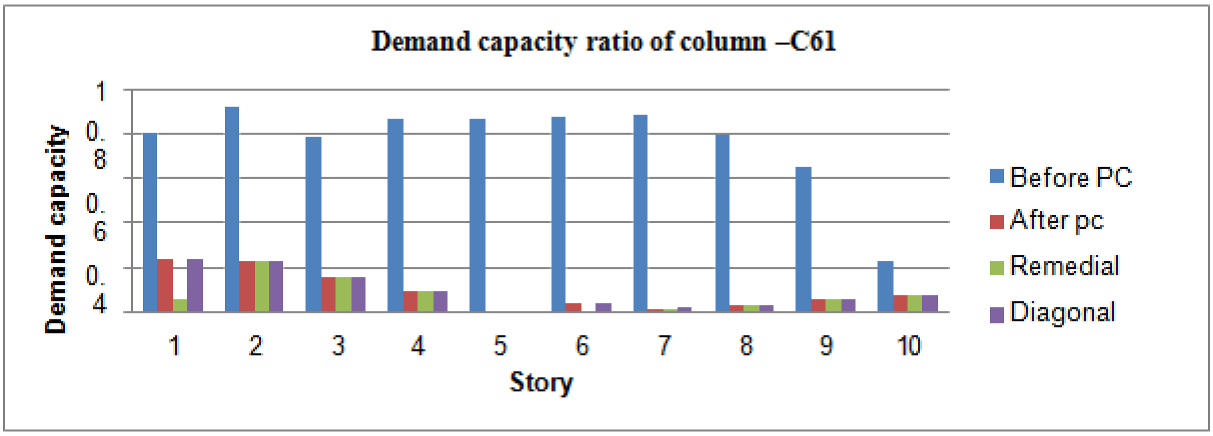
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	78.0415	90.0328	83.5049	92.0328
2	78.0415	92.0328	82.0328	92.0328
3	78.0415	92.0328	92.0328	92.0328
4	78.0415	92.0328	90.9017	92.0328
5	80.0351	101.2802	101.2802	101.2802
6	81.8833	96.7298	96.7298	96.7298
7	81.7979	96.6614	96.6614	96.6614
8	83.6553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

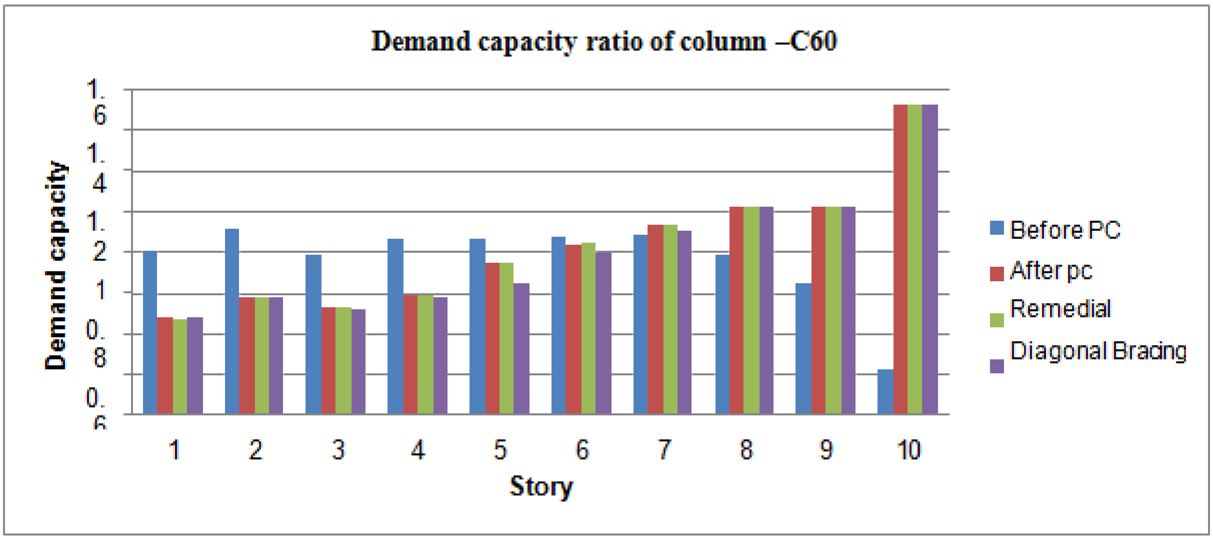
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	73.0370	90.4789	90.1722	90.4789
2	73.0370	90.4789	90.4889	90.4789
3	73.0370	90.4789	90.4749	90.4789
4	73.0370	90.4789	90.4789	90.4789
5	73.6920	95.0789	95.0789	95.0789
6	74.1606	92.203	92.223	92.223
7	74.0794	92.1501	92.1501	92.1501
8	74.5227	92.9347	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

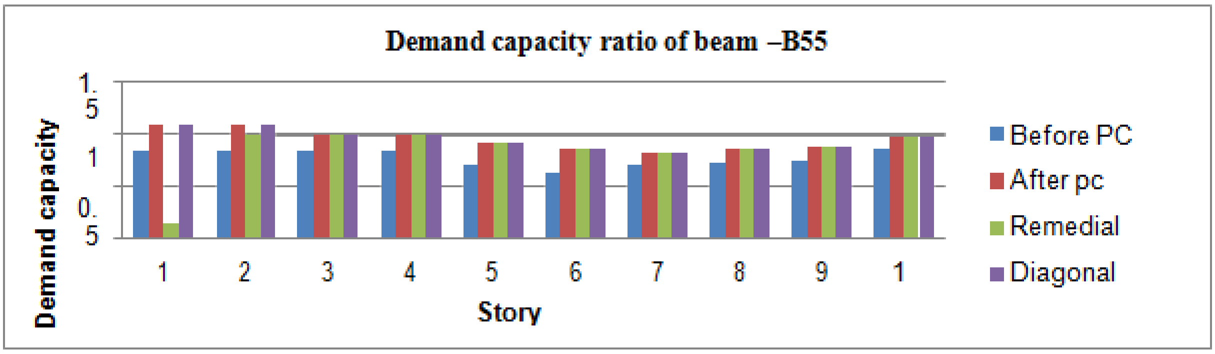
Story	Before PC	After pc	Remedial	Diagonal Bracing
1	0.006394	0.000001	0.000001	0.000041
2	0.019369	0.000001	0.000002	0.000032
3	0.031735	0.000002	0.000002	0.0000014
4	0.042882	0.000004	0.000004	0.000003
5	0.055461	0.000004	0.000005	0.000006
6	0.071067	0.000005	0.000006	0.000008
7	0.087048	0.000008	0.000008	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



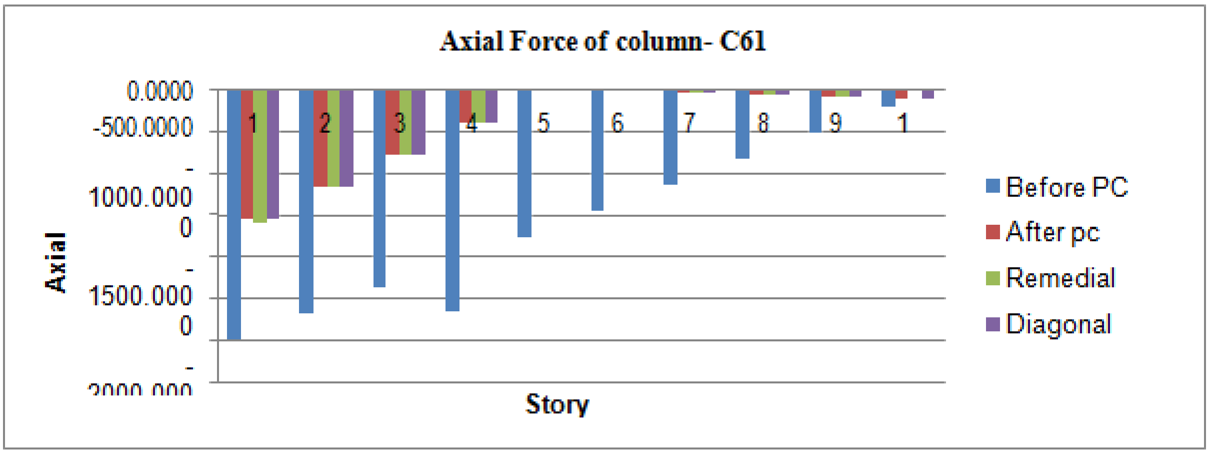
a . Demand Capacity Ratio Of Column C61



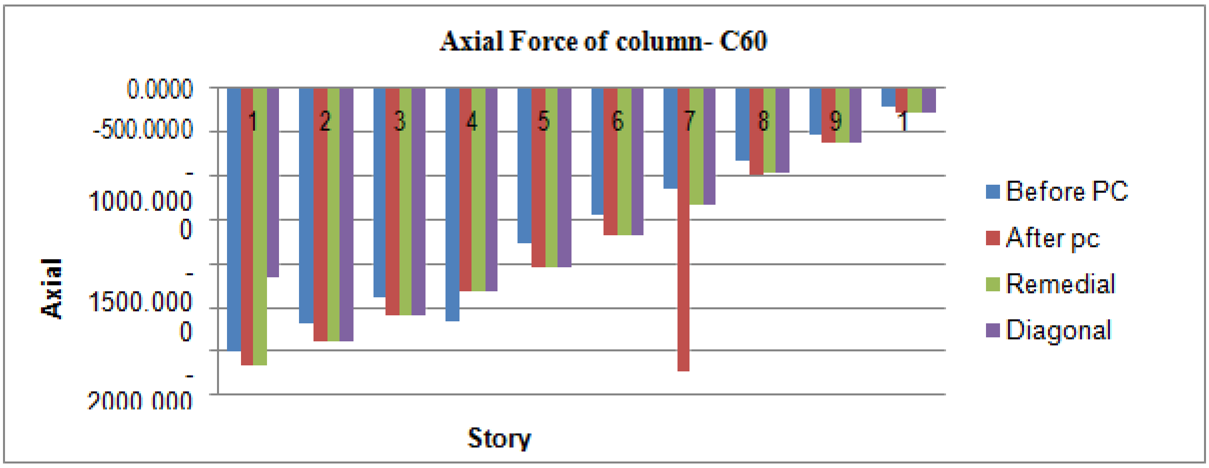
b. Demand Capacity Ratio Of Column C60



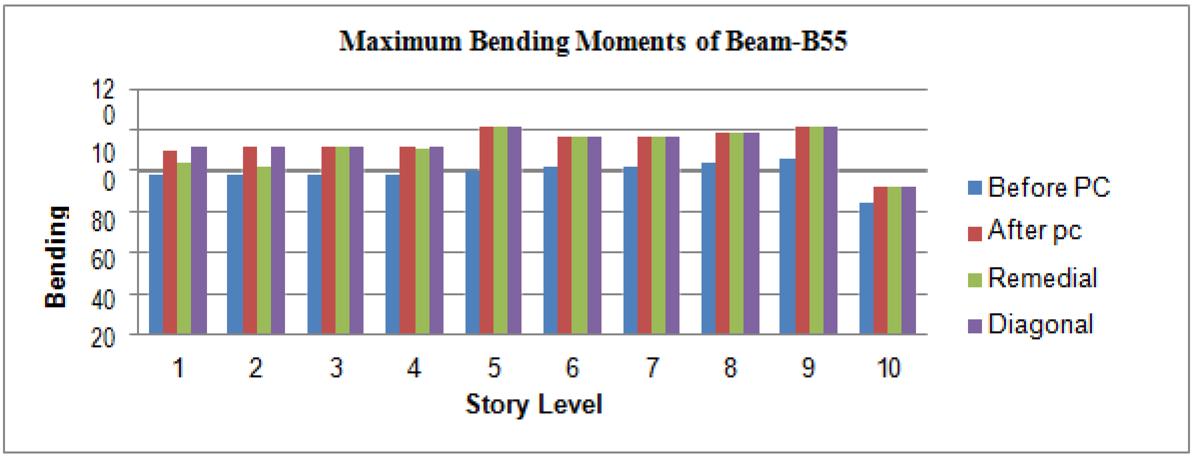
C. Demand Capacity Ratio Of Beam B55



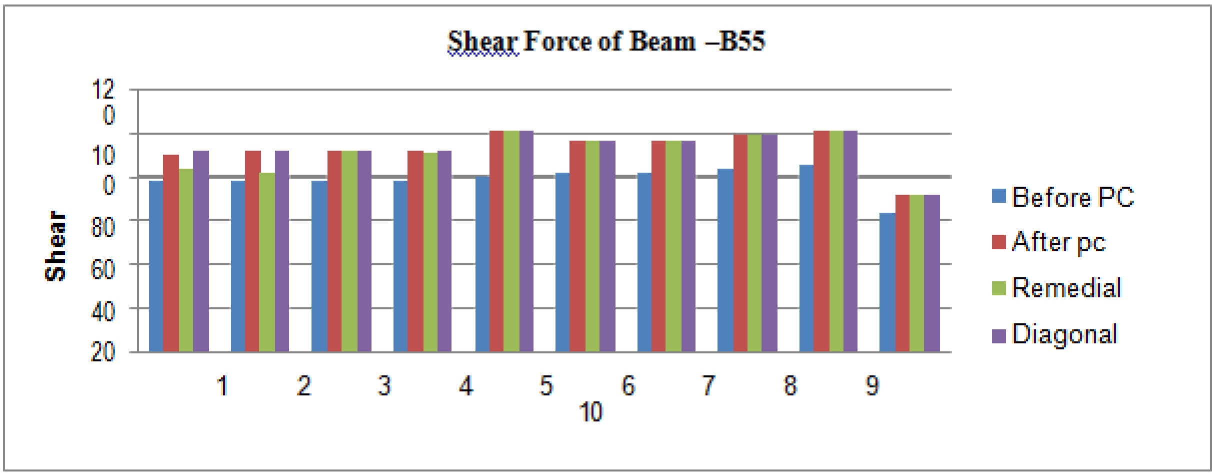
d. Axial Force Of Column C61



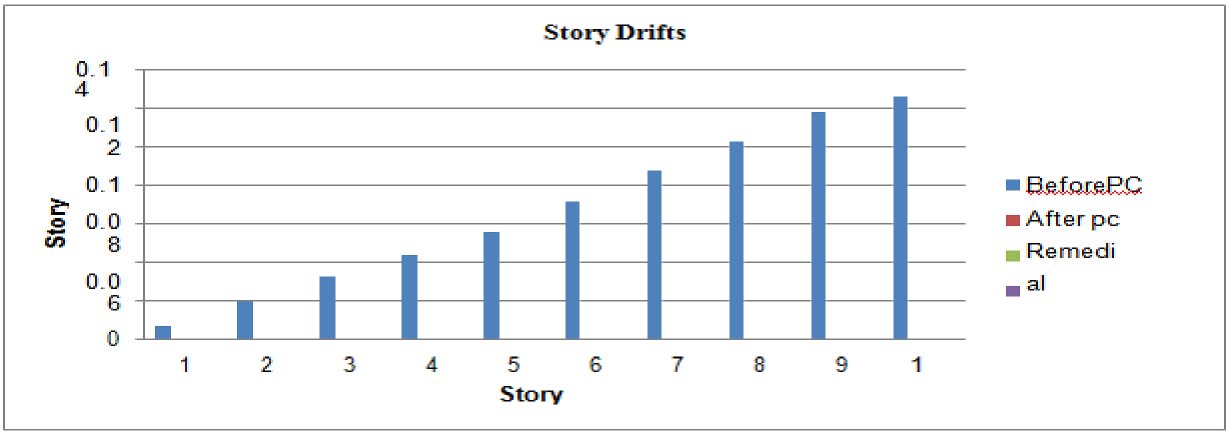
e. Axial Force Of Column C60



f. Maximum Bending Moments Of Beam B55



g. Shear Force Of Beam B55



h. 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

Also we concluded in the above graph 4.2(d &e) 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

5. CONCLUSIONS

1. From results it is conclude that the effect of progressive collapse in diagonal braced system is best as compared to increase the beam and column size at critical location system .

2. Number of story increases effect of progressive collapse decreases since the numbers to members for taking distributed load are more and hence DCR values of beams go decreasing for upper levels beam which shows the more failure occurs in nearby area of removed column.

3. DCR values of beam go on decreasing towards upper levels but DCR values of column go on increasing towards upper level.

4. It is observed that effect of progressive collapse was more when corner column was suddenly removed, as the number of story increases effect of progressive collapse decreases since the number of members for taking distributed load is more.

5. It is the increase in bending moment of beam due to redistribution of loading on removed area location which leads to failure may be partial or fully but not shear fore (strong column &weak beam)

6. Because of removal of column there is increase in load on the nearby column but loss of strength of same column on succeeding levels and same effect is more hazardous when sudden column loss occurs on higher levels

7. In any multi story high rise building stiffness and strength are more important so to stiffness and strength are more important so to improve this characteristic of the structure it is possible to provide bracing.

REFERENCES

1. Engineering Research and Applications. Vol. 2, Issue 4, June July 2012[2]

2. Jinkoo Kim† and Junhee Park “Design of steel moment frames considering progressive collapse”, Steel and Composite Structures, Vol. 8, No. 1 (2008) 85-98[3].

3. Mojtaba Hosseini , Nader Fanaie and Amir Mohammad Yousefi “Studying the Vulnerability of Steel Moment Resistant Frames Subjected to Progressive Collapse”, Indian Journal of Science and Technology, Vol 7(3), 335–342, March2014.

4. M. A. Hadianfard , M. Wassegh M. and Sultana Mohammed. “Linear and nonlinear analysis of progressive collapse for seismic designed steel moment frames.”14 th international conference on computing in civil and building Engineering. Moscow, Russia 27- 29 june2012.

5. Lawver, D., H. Levine, and D. Tennant Response of Reinforced Concrete Roof and Floor Slabs to Internal Explosion for Ladeburg Bunker Test 10 10th International Symposium on Interaction of the Effects of Munitions with Structures, May2001.

6. Progressive Collapse Design Guidelines, Phase 1: Literature Search Department of State/Office of Foreign Buildings Operations, Vols 1-3, August 8,1996.

7. American Society of Civil Engineers (ASCE), Minimum Design Loads for Buildings and Other Structures, ASCE7-9

8. U.S. Army Technical Manual TM5-1300 Structures to Resist the Effects of Accidental Explosions”.

9. Vaughan, D. K. FLEX User's Guide Report UG8298, Weidlinger Associates, Los Altos, CA, May 1983 plus updates through1997.

10. ANSYS, Inc. ANSYS User’s Guide, Release 5.5 Canonsburg, PA,1994.

11. Hobbit, Carlson& Sorensen, Inc. ABAQUS/Explicit, Version 6.1Pawtucket, RI,2000.

12. LARSA, Inc., “LARSA Windows Series,” New York, NY,1997.

13. Mould, J. C., Jr. and H. S. Levine (1993), "A Rate-Dependent Three Invariant Softening Model for Concrete," published in Studies in Applied Mech., Mech. and Materials and Structures, Ed. by G.Voyiadjis, L. Bank and L. Jacobs, Elsevier,1994.