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ANALYSIS OF REINFORCED CONCRETE BUILDING UNDER PROGRESSIVE COLLAPSE

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Abstract- Progressive collapse is a chain reaction of failures propagating throughout a portion of the structure disproportionate to the original local failure occurring when a sudden loss of a critical load-bearing element initiates a structural element failure, eventually resulting in partial or full collapse of the structure. General Services Administration (GSA) provides the guidelines for progressive collapse analysis. The focus of the project is to study progressive collapse behavior of building. In this project it is proposed to carry out progressive collapse analysis of 15 storey RC framed building by removing columns from different positions one at a time as per the GSA guidelines. Loads are applied as per GSA guidelines. As per GSA guidelines, the columns are removed at different positions one case at a time namely corner column, exterior column and interior column at ground floor. For all three cases both linear and non-linear analysis is done. In a linear static analysis for all beams DCR values for flexure are greater than two which shows that they are going to fail under sudden column loss conditions. These beams need to be redesigned to improve reinforcement to arrest the progressive collapse. It is observed that the shear in beams is notcritical in any case. But by linear static and non-linear static analysis, it is observed that the beams are going to fail in flexure. In non-linear static analysis for a corner, exterior and interior removal case, the building carries a maximum base shear were observed. It shows that the most dangerous case is interior column removal case and least dangerous is corner column removal case.
Keywords -Linear static analysis; Non-linear static analysis; GSA Guidelines; DCR values; Reinforced Concrete Beam; Reinforced Concrete Column; Etabs.

1. INTRODUCTION

Buildings are generally designed according to the design standards which usually consider dead, imposed, wind and earthquake loads etc. and their combinations. There are allowances for other loads, such as impact and explosion. Civil engineering structures can be subjected to loads due to natural disasters like earthquakes, hurricanes, tornadoes, floods, fires and man-made and artificial disasters, such as explosion and impact, during their lifetime. However, there are still circumstances that are unforeseeable at design stage. On the other hand, every project has a budget and engineers should meet the design requirements while producing an economical design within the allocated budget. The building will be collapse if one of the important structural members gets fail. Due to the failure of one structural member (local failure), the load on the other members in the vicinity of it increases and that member is going to fail if an increased load goes beyond the capacity of the member. Likewise, failure will transfer from one member to another which leads to collapse of the whole structure. Such type of failure of structure is known as progressive collapse or cascade failure.Xinzheng Lu and Kaiqi Lin (2016) [19] has studied that increasing the slab reinforcement by expanding the slab thickness marginally improved the collapse resistance under the catenary mechanism but contributed little to that under the beam mechanism. J M Russel, J S Owen and I Hajira (2015) [8] has studied that ultimate failure is punching shear failure of the corner columns. The dynamic response of the system is altered by force distribution and damage. Meng et al (2012)[11] has studied Demand to capacity ratios are calculated to check out the susceptibility of structure for progressive collapse. Finally, it is concluded that the GSA linear static analysis of the building has a low potential for progressive collapse. SewerynKokat et al (2012)[15]hasstudied the behavior of RC flat slab frame building under a progressive collapse. From the results of SAP linear dynamic analysis, it is observed that the structure would still be susceptible to the progressive collapse of the central column removal case but not necessarily for exterior column removal case. MengHao Tsai et al (2011)[12] has observed that if any one neglect panel type walls, then collapse resistance will be overestimated and wing type walls may bring less adverse effect on the building response under column losses.

1.1 The objectives for this dissertation work are

To understand the process of progressive collapse of 15 storey RC framed building due to the sudden column loss scenario.
To understand the progressive collapse analysis of building using linear static analysis and non-linear static analysis.
To recommend solutions for reducing or preventing progressive collapse of a RC framed structure, if it is necessary.

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To check whether a RC building (special moment resistance frame) designed and detailed by Indian codes for seismic loads provides any resistance to progressive collapse or not.

2. METHODOLOGY

The GSA guidelines are valuable guidance for reducing the potential for progressive collapse in the design of new and upgraded buildings, as well as for estimating the potential for progressive collapse in existing buildings. In this study, the progressive collapse potential of RC buildings evaluated by GSA guidelines. The detailed GSA suggestions and loading combination for a computer model and the column removal procedure used in this study.

2.1 Load case for deformation-controlled actions Qudfor linear static analysis

To calculate the deformation-controlled actions, simultaneously apply the following combination of gravity loads, increased gravity loads for floor areas above removed column or wall. Apply the increased gravity load combination in equation 3.1 to those bays immediately adjacent to the removed element and at all floors above the removed element as shown in Fig.1.

$GLD = \Omega LD\,[1.2DL + 0.5LL]$ (3.1)

Where, GLD = Increased gravity loads for deformation-controlled actions for linear static analysis

DL = Dead load, including façade loads (kN/m2); LL = Live load (kN/m2)

ΩLD = Load increase factor for calculating deformation-controlled actions for linear static analysis is given in Table.1

Gravity loads for floor areas away from removed column or wall: Apply the gravity load combination in equation 3.2 to those bays not loaded with GLD as shown in Fig.1.

$G = 1.2DL + 0.5\,LL$ (3.2)

Where, G = Gravity loads

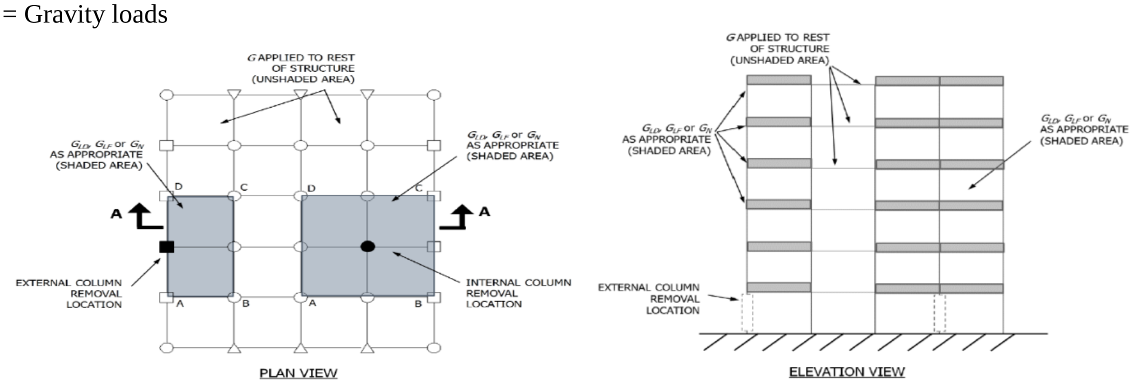


Fig. 1 Plan and elevation views of external and internal column removal

Table 1 load increase factors for linear static analysis

Material	Structure type	ΩLD Deformation-controlled	ΩLF Force-controlled
Steel	Framed	0.9 mLIF+ 1.1	2.0
Reinforced concrete	Framed	1.2 mLIF+ 0.80	2.0
Load-bearing Wall		2.0 mLIF	2.0

2.2 Load case for deformation-controlled actionsQudfor non-linear static analysis

To calculate the deformation-controlled and force-controlled actions, simultaneously apply the following combination of gravity loads.

Increased gravity loads for floor areas above removed column or wall: Apply the increased gravity load combination in equation 3.3 to those bays immediately adjacent to the removed element and at all floors above the removed element as shown in Fig.1.

$GN = \Omega N\,[1.2DL + 0.5LL]$ (3.3)

Where, GN = Increased gravity loads for non-linear static analysis

DL = Dead load, including façade loads (kN/m2)

LL = Live load (kN/m2)

ΩN= Dynamic increase factor for calculating deformation controlled and force-controlled actions for non-linear static analysis is given in Table .2.

Table.2 Dynamic increase factors ΩN for non-linear static analysis

Material	Structure type	ΩN
Steel	Framed	$1.08+0.76/(\theta pra/\theta y+0.83)$
Reinforced concrete	Framed	$1.04+0.45/(\theta pra/\theta y+0.48)$

Gravity loads for floor areas away from removing column or wall: Apply the gravity load combination in equation 3.4 to those bays not loaded with GN as shown in Fig.1.

$$G = 1.2\ DL + (0.5\ LL) \tag{3.4}$$

Where, G = Gravity loads

Limitations of Demand Capacity Ratio

To calculate the DCRs for either framed or load-bearing structures, create a linear model of the building. The model will have all primary components except for the removed wall or column. The deformation-controlled load case shall be applied, with gravity dead and live loads increased by the load increase factor ΩLD. The resulting actions (internal forces and moments) are defined as QUDLim.UseQUDLim to calculate the DCRs for the deformation-controlled actions as:

DCR =QUDLim/QCE ……. Where,

QCE = Expected strength of the component; QUDLim = Expected demand on the component

2.3 Model Description

Table 3 Geometric and Loading Data of BuildingTable 4 column schedule

1.	Plan of Building	30 m × 25 m
2.	No. of Bays	5 in both direction
3.	Height of each Floor	3 m
4.	Live load	3 kN/m2
5.	Floor finish load	1.5 kN/m2
6.	Slab thickness	150mm
7.	Grade of Concrete	M40
8.	Grade of Steel	Fe500
Peripheral columns: C1 to C7, C12, C13, C18, C19, C24, C25, C30 to C36		
Size(mm)	600 x 600	500 x 500
Main steel	4Ø20 + 8Ø16	12Ø16
Internal columns: C8 to C11, C14 to C17, C20 to C23, C26 to C29		
Size(mm)	700 x 700	600 x 600
Main steel	4Ø25 + 8Ø20	12Ø20

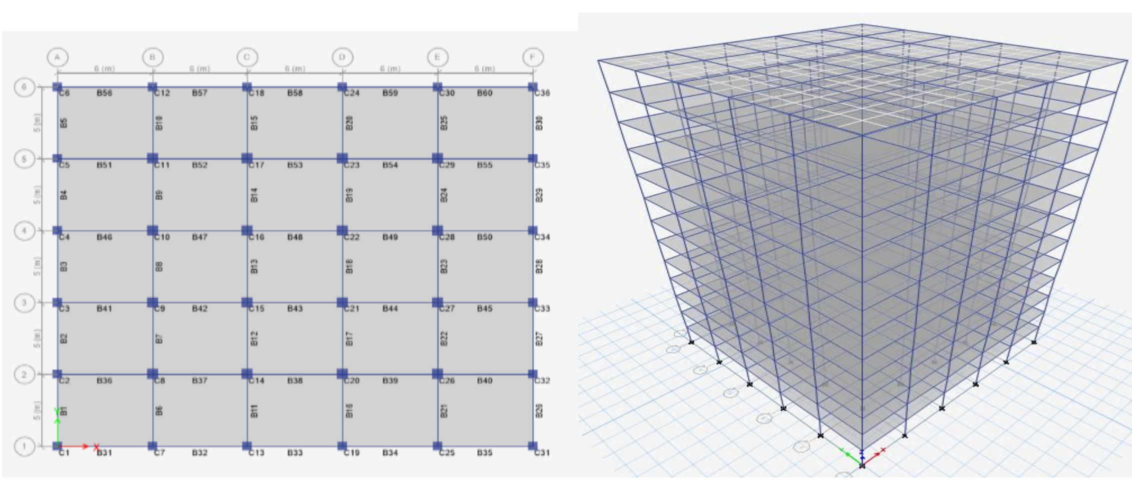


Fig. 2 Typical floor plan and structural member labels and 3D view

3. RESULTS AND DISCUSSIONS

3.1 Corner Column Removal Case for linear static analysis

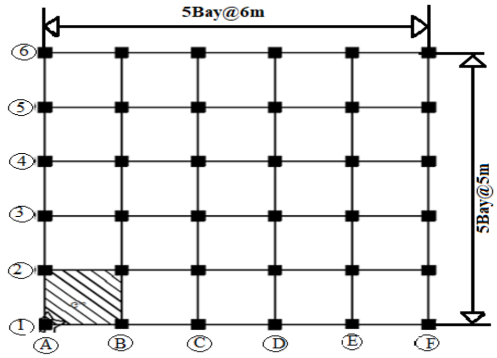


Fig. 3 plan view of corner column removal

For the corner column removal model, the load GLDshould be applied in vicinity of removed column (shaded area in Fig.3) and in the remaining area G load should be applied.DCR for beam is calculated by dividing the bending moment value to the capacity values of respective beams.

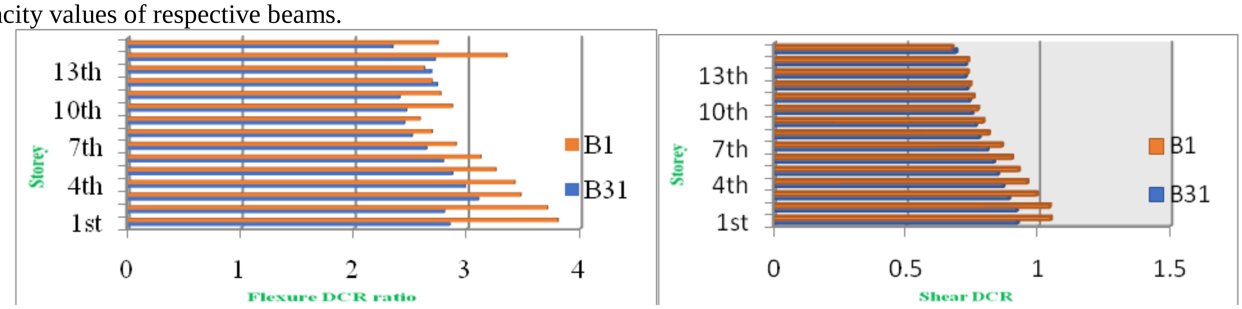


Fig.4 Flexure and Shear DCR of B31 and B1 beams when corner column is removed

From Fig.4 it is observed that the all the DCR values are more for all the transverse beams compared to that of all the longitudinal beams. For all the beams in both directions, the DCR values exceed a limit of 2 as given by ASCE 41. Hence, the beams are critical in corner column removal case. it is also observed that the beams shear DCR values are more for all the transverse beams compared to that of all the longitudinal beams.

3.2 Exterior Column Removal Case for linear static analysis

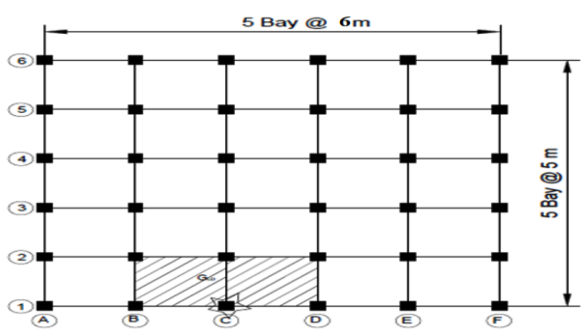


Fig. 5 plan view of edge column removal

For the edge column removal model, the load GLD should be applied in vicinity of removed column (shaded area in Fig. 5) and in the remaining area G load should be applied. DCR for beam is calculated by dividing the bending moment value to the capacity values of respective beams

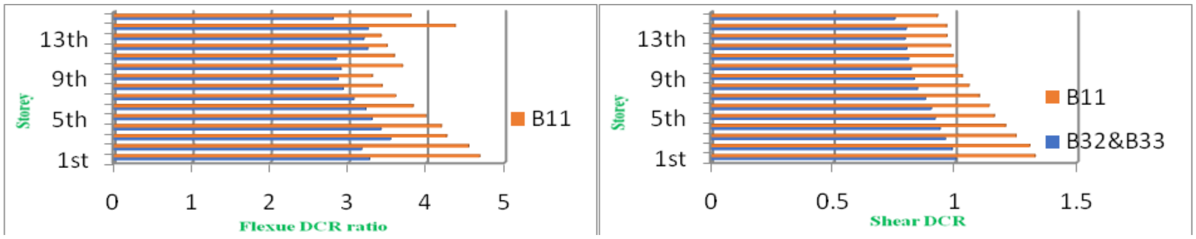


Fig. 6 Flexure and shear DCR of B11 and B32, B33 beams when edge column is removed

From Fig. 6 it is observed that the DCR values are more for all the transverse beams compared to that of all the longitudinal beams. For all the beams in both directions, the DCR values exceed 2. Hence, all the beams are critical in edge column removal case. The graphs indicate the collapse behavior of beams and the beams DCRs which exceed the DCR limitations given by GSA guidelines. it is also observed that the shear DCR values are more for all the transverse beams compared to that of all the longitudinal beams

3.3 Interior Column Removal Case

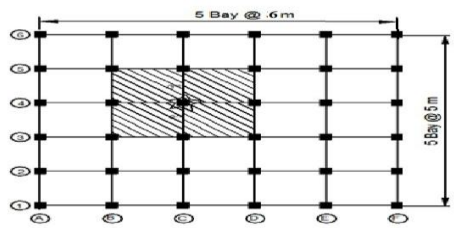


Fig.7 plan view of interior column removal

For the interior column removal model, the load GLDshould be applied in vicinity of removed column (shaded area in Fig.7) and in the remaining area G load should be applied. DCR for beam is calculated by dividing the bending moment value to the capacity values of respective beams.

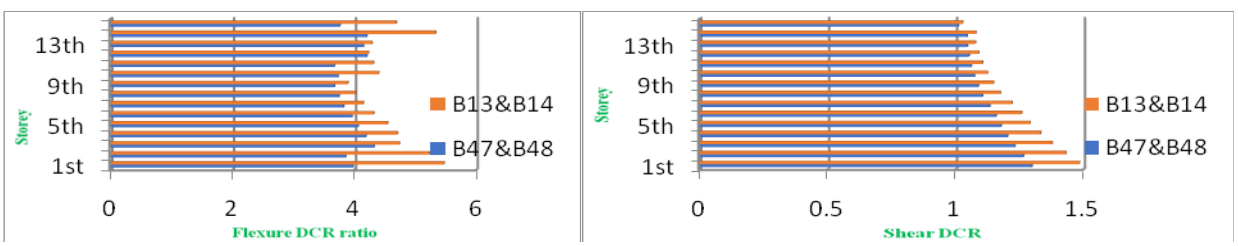


Fig.8 Flexure and Shear DCR of beams when interior column is removed

From Fig. 8 it is observed that the DCR values are more for all the transverse beams compared to that of all the longitudinal beams. For all the beams in both directions, the DCR values exceed 2. Hence, all the beams are critical in interior column removal case. The graphs indicate the collapse behavior of beams and the beams DCRs which exceeds the DCR limitations given by GSA guidelines.it is also observed that the shear DCR values are more for all the transverse beams compared to that of all the longitudinal beams.

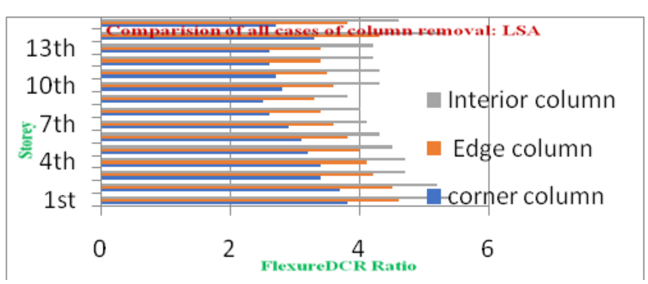
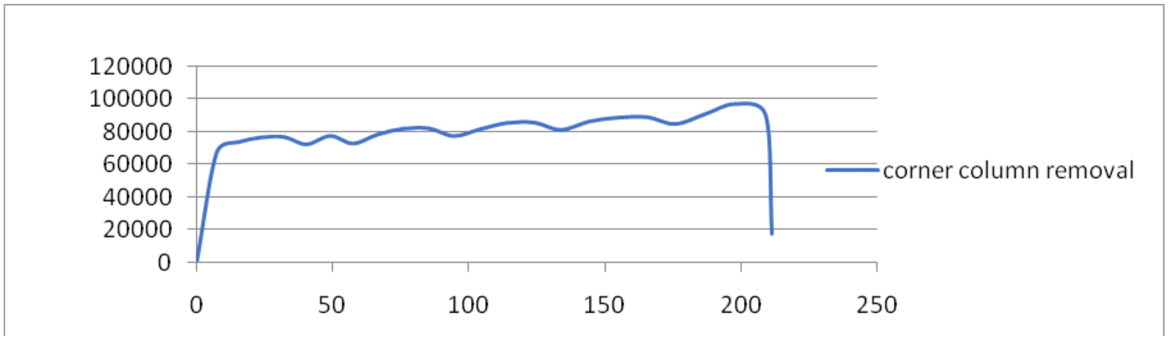


Fig.9 Flexural DCR for beams in all three column removal cases

The DCR of transverse beams in each case is noted separately and compared. For such three critical beams, one from each case, a combined bar chart is plotted as shown in Fig.9.

3.4 Corner Column Removal Case-Pushdown Analysis

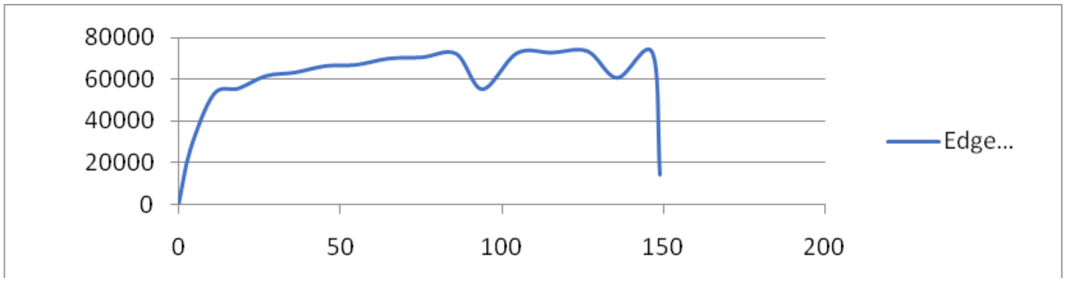
The push down (non-linear static) analysis carried out for load combination of GN = ΩN[1.2DL + 0.5LL] in column removal region (hatched portion in Fig.3) and for other region G = 1.2DL + 0.5LL on all storeys.The dynamic load increase factor in the corner column removal case is calculated for each beam.These values are used in the non-linear static analysis.The minimum dynamic increase factor obtained is (ΩN=1.191) is used and a total UDL of 22.6 kN/m2 in hatched region and 19.01 kN/m2 in other region is applied.The total load in ETAB 2009 software is taken in 23 steps and vertical deflection at column removal location and base shear is observed for each step.



Graph.1 Base shear vs. monitored deflection at corner column removal point

3.5 Exterior Edge Column Removal Case-Pushdown Analysis

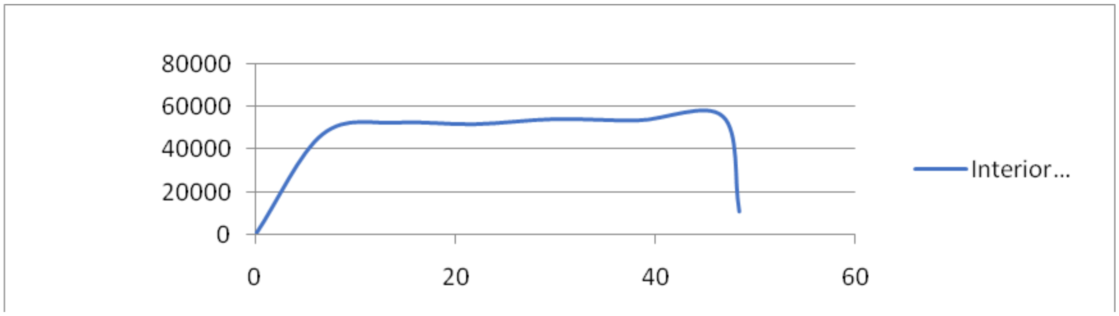
The push down (non-linear static) analysis carried out for load combination of GN= ΩN[1.2DL + 0.5LL] in column removal region (hatched portion in Fig.5) and for other region G = 1.2DL + 0.5LL on all storeys. The minimum dynamic increase factor obtained from calculations (ΩN =1.191) is used and a total UDL of 22.6 kN/m2 in hatched region and 19.01 kN/m2 in other region is applied. Default non-linear hinges are assigned to column and beams and after removal of corner column structure is pushed down and then the total load in ETABS 2009 software is taken in 17 steps and vertical deflection at column removal location and base shear is observed for each step.



Graph.2 Base shear vs. monitored deflection at edge column removed point

3.6 Interior Column Removal Case-Pushdown Analysis

The push down (non-linear static) analysis carried out for load combination of GN = ΩN[1.2DL + 0.5LL] in column removal region (hatched portion in Fig.7) and for other region G = 1.2DL + 0.5LL on all storeys.The dynamic load increase factor for interior column removal case is calculated. These values are used in non-linear static analysis.The minimum dynamic increase factor obtained from calculations (ΩN =1.191) is used and a total UDL of 22.6 kN/m2 in hatched region and 19.01 kN/m2 in other region is applied. The total load is taken 7 steps and vertical deflection at column removal location and base shear is observed for each step.



Graph.3 Base shear vs. monitored deflection at interior column removal case

A graph between vertical deflection at column removal point and base shear is shown in Graph. 3.which shows that this interior column removal case is very severe when compare to remaining cases.

By observing the base shear vs. displacement graph of all the three the following conclusions of pushdown corner column removal case are arrived.

•For corner column removal case the hinge formation sequence shows that the most critical beams are presented upto the 6th storey and these beams have reached maximum deformation capacity (performance level E).

• For edge column removal case the hinge formation sequence shows that the most critical beams are present upto third storey beams long grid 1 and these beams have reached maximum deformation capacity (performance level E).

• The hinge formation sequence shows that the most critical beams are present upto 7th storey beams long grid C and these beams have reached maximum deformation capacity (performance level E).

•This is the least resistance to progressive collapse among three removal cases. As soon as the column is removed and the structure is pushed down structure loses its stability.

•No hinges are formed in columns. Therefore columns are not critical as compared to beams.

4. CONCLUSIONS

The following conclusions are made.

In a linear static analysis for all beams DCR value for flexure are greater than two which shows that they are going to fail under sudden column loss conditions. These beams need to be redesigned to improve reinforcement to arrest the progressive collapse.

In a linear static analysis for all beams DCR value for shear are less than two which shows that all beams are safe under progressive collapse. The shear capacity of beams is too high compared with shear demand.

In non-linear static analysis for a corner, exterior and interior removal case, the building carries a maximum base shear 91115kN,73403kN and 55741 kN respectively before failure. It shows that the most dangerous case is interior column removal and least dangerous is corner removal case.

Observing the hinge formation pattern in all the three cases of column removal of nonlinear static analysis, it is found that non-linear hinges in the lower storey beams have reached maximum deformation capacity (performance level E). It means that lower storey beams are more critical than upper storey beams.

A special moment resistant frame (SMRF) designed by IS 456 and detailed by IS 13920 does not provide resistance to progressive collapse. This is because of that SMRF is designed for lateral loads and in progressive collapse the failure loads are gravity loads.

5. FUTURE SCOPE

The present study is done using linear static and non-linear static methods with symmetrical building. The buildings can be analyzed using non-linear dynamic method with an irregular structure providing a shear wall in the building. The stiffness contributions of shear walls can be assessed.

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