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Original Article

Progressive Collapse Assessment Of Framed RC Building According To GSA Guidelines

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ABSTRACT

Today development in the RC structure has reached a new height. Due to advancements in technology, designers can design to stand strong against natural lateral forces like wind and earthquakes. Some of the extreme human-made events or natural events damage part of the structure or can lead to the destruction of the structure. These human-made events can include bomb explosions, gas-pipe explosions, etc. The failure of a primary vertical supporting structural member-like column will force adjoining structural members to resist the load previously taken by the column. This transmission and resisting process will continue till equilibrium is attended in the structure. If equilibrium is not attended then it will end up with the collapse of the structure or major part of it. This is called 'Progressive Collapse'. Rising incidents of blast events as an act of terrorism, to harm the national important structures or to harm a large number of people have increased throughout the world. The purpose of this project was to assess the progressive collapse of RC-framed structures, according to the General Service Administration (GSA 2003) guidelines. GSA provides the procedure to perform different analyses to determine the capability of a structure against progressive collapse. In the current study, Linear Static Analysis (LSA) and Non-Linear Static Analysis (NLSA) were performed to calculate the Demand-Capacity Ratio (DCR) at critical beam sections and Plastic Hinge Formation respectively. Both analyses were performed on the model of existing structure viz. GST Bhavan located in Pune city. Model was created in SAP2000 software. The demand-capacity ratio are compared with plastic hinge formation. Comparison of linear static analysis and non-linear analysis reveals that the hinge formation starts at the location having the maximum demand capacity ratio calculated from static analysis. Also, from current scenario the middle column on the longer side has been identified as the critical column.

Keywords: RC Structure, GSA guidelines, Progressive collapse, DCR, Plastic Hinge, SAP2000.

INTRODUCTION

1.1 General

In this 21st century, framed RC structures are one of the most common structures built throughout the world. Due to the lack of ability to build more stories by limiting the width of the wall in load-bearing structures, framed RC structures started gaining importance. In framed structures, the gravity loads viz. dead load and live load, coming on the slab are transferred to the adjoining beams which next transfer the load on the columns then down to the foundation, and ultimately to the ground present below it. This is called 'Gravity Load Path'. In tall structures, the loads associated with wind are greater than dead or live loads, the lateral wind load imposed on these structures is usually the governing factor in structural design. Shear walls are built within the building to resist these lateral forces. The shear walls are positioned symmetrically in the plan to avoid lateral-torsional motions. They are positioned at the perimeter of the structure or can be constructed as a shear core. Examples are encasing a lift shaft or stairwell. The way through which the lateral loads are transferred through a building to the ground is called the 'Lateral Load Path'. The primary elements of this path include vertical and horizontal components. Vertical components are shear walls and frames while horizontal components are roofs, floors, and foundations. The lateral forces acting on roofs and floors are transferred to shear walls and then to the foundation. Shear walls transmit both lateral as well as gravity forces to the foundation, which ultimately transfers to the ground.

Relevant conflicts of interest/financial disclosures: The authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

Historically, structural engineers have always tended to avoid designing for extremely unlikely loads and relied on built-in redundancy in traditional structural systems as well as material safety factors to cover extreme cases. The high attention given to progressive collapse analysis has materialized into explicit requirements for redundancy in building codes all over the world. As per the American Association of State Highway and Transportation Officials (AASHTO), redundancy can be defined as the capability of a structural system to carry loads after damage to or the failure of one or more of its members. There are three types of redundancy

1. Load path redundancy
2. Structural redundancy
3. Internal redundancy

A member is considered as a load path redundant if an alternative and sufficient load path are determined to exist. The alternative load paths must have sufficient capacity to carry the load redistributed to them from an adjacent failed member. A member is considered structurally redundant if its boundary conditions or supports are such that failure of the member merely changes the boundary or support conditions but does not result in the collapse of the superstructure. And internal redundancy is when a structural member has alternative and sufficient load paths existing within the member itself. For example, a riveted steel member connection is considered internally redundant if it has multiple plies.

1.2 Definitions of Progressive Collapse

a) UFC defines progressive collapse as mentioned in the commentary of the American Society of Civil Engineers Standard 7-05 "Minimum design loads for buildings and other structures" (2005) as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it is called Progressive collapse".

b) According to GSA Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance (2013), "progressive collapse is defined as an extent of damage or collapse that is disproportionate to the magnitude of the initiating event."

1.3 Past Events of Progressive Collapse Around the World

Fig. 1.1 The collapse of one corner of the Ronan building (Wikipedia)

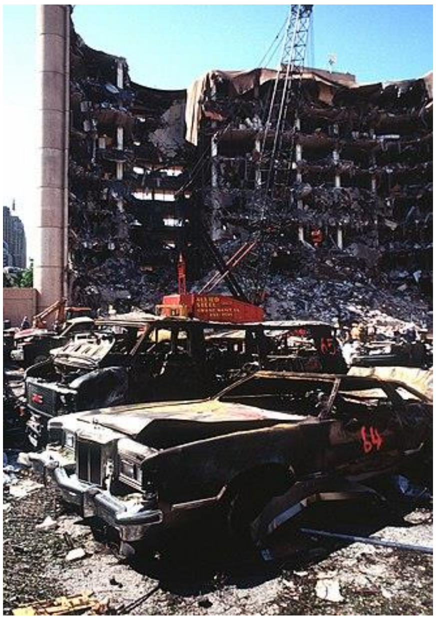

Fig. 1.2 One-third of the Murrah Federal Office damaged. (Wikipedia)

Fig. 1.3 Top view of damaged federal building. (Civil Engineering)

Past events of progressive collapse created fear in people to stay in high-rise structures. Events like the collapse of the 22-story Ronan building (Fig. 1.1) in 1968, due to a gas explosion led to the collapse of one entire corner on the 18th floor. The explosion blew the load-bearing walls, leaving four floors above it in unsupported condition and this caused the progressive collapse of the structure. This event led to having major changes in building regulations of the UK country. In 1995, there was a terrorist act of truck bomb explosion near the Murrah Federal Office Building in Oklahoma City, US (Fig.1.2). The explosion of the bomb first destroyed the column near to it. The lower floors were pushed upwards by the explosion's shockwave before the fourth and fifth floor collapsed onto the third floor, which housed a transfer beam that stretched along the length of the structure and was supporting the pillars that were above it. Due to the increased weight, the third floor and the transfer beam broke away, resulting in the structure collapsing. An aerial view of the damaged portion of the building can be seen in Fig. 1.3. Despite the fact that the structure complied with all code requirements, research conducted after the tragedy revealed that changes to the building design, such as different reinforcement detailing and the addition of some reinforcement, could have prevented the collapse without significantly increasing construction costs. Another case of progressive collapse is the failure of the World Trade Centre, US (Fig. 1.4); due to a terrorist attack in 2001. The investigation was carried out by the National Institute of Standards and Technology (NIST). The report submitted by this institute in 2005 states that the fire to be the main cause of the collapse. Perimeter columns and floors were deteriorated by the heat of the fire. Sagging of floors pulled the perimeter columns inward leading them to buckle. Impact of the airplanes with the high-speed, damaged majority of top floor portions which fell upon the lower undamaged structure. The collapse began with the drop of the upper floor on the lower floor through the height of one story which released the necessary energy to begin a progressive collapse.

Fig. 1.4 World Trade Centre, US (Los Angeles Times)

1.1 GSA Guidelines

The General Service Administration (GSA) progressive collapse guideline provides a detailed methodology and performance criteria needed to assess the vulnerability of new and existing buildings to progressive collapse. For framed structures the following analysis cases should be considered (GSA 2003).

1.4.1 Exterior Considerations:

The following exterior analysis cases should be considered:

a) Analyse the building for the instantaneous loss of a column for one floor above grade (1st story) located at or near the middle of the long side of building. This scenario is shown as case 1 (see Blue Colour Circle in Fig.1.5).

b) Case 2 in which analysis for the instantaneous loss of a column for one floor above grade (1st story) located at or near the middle of the short side of the building is carried out, i.e. case 2 (see Black Colour Circle in Fig.1.5).

c) Analyse for the instantaneous loss of a column for one floor above grade (1st story) located at the corner of the building. This scenario is shown as case 3 (see Red Colour Circle in Fig.1.5).

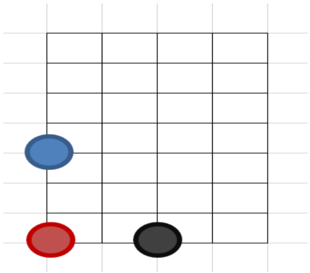

Fig. 1.5 Plan view of a typical building

1.4.2 Interior Considerations:

Buildings that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case. Analyse the building for the instantaneous loss of one column that extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (1st story). The column considered should be interior to the perimeter column lines. In the present study interior column removed condition is shown as case 4 (see Figure 1).

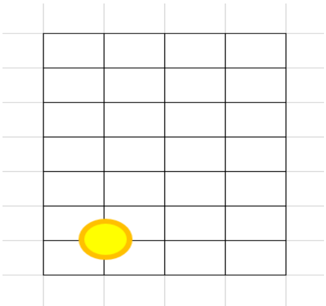

Fig. 1.6 Plan view of a typical building

Problem Statement

Progressive collapse occurs when local damage in a structural component initiates a chain of failures, leading to partial or total building collapse. Many reinforced concrete (RC) framed buildings, especially older ones, were not designed to resist such abnormal events—such as sudden column removal, blasts, or impacts—making them potentially vulnerable.

The General Services Administration (GSA) Guidelines offer a standardized method for evaluating building robustness under component removal scenarios. However, the progressive collapse performance of many existing RC buildings remains uncertain.

This study aims to assess the progressive collapse potential of a selected RC framed building by modeling and analyzing various column removal scenarios as per GSA Guidelines. The investigation focuses on deformation patterns, force redistribution, and demand–capacity ratios to identify structural weaknesses and evaluate the building's ability to develop alternate load paths.

Objectives

Following are the objectives of the project

1. To evaluate the resistance of RC framed structure to progressive collapse, designed according to IS 456-2016.
2. To perform linear and non-linear analysis on an existing RC framed structure.
3. To determine the critical column removal location.

Methodology

Objectives will be accomplished by exploring various past literature available on the progressive collapse. :

1. Building Selection and Description: A typical RC framed building is selected, with detailed information on its geometry, material properties, and loading.
2. Modeling and Analysis: A 3D finite element model of the building will be created using software SAP2000.
3. Progressive Collapse Simulation: Perform linear static or nonlinear static analysis to obtain structural response. Evaluate deformation, bending moments, and DCR values.
4. Assessment of Structural Response: A comparative study among various column removal cases will lead to critical column location.

MODELLING OF AN EXISTING STRUCTURE

4.1 General

In this chapter, an existing structure is analyzed to study the targeted objectives. The existing structure is 'GST Bhavan', a RC framed structure located in Pune city. Linear Static Analysis (LSA) and Non-Linear Static Analysis (NLSA) were performed on the structure in SAP2000. Both the analysis methods are discussed in detail later in this chapter along with their step-by-step procedure to perform them in the software. Details of existing structure and modelling & designing parameters are discussed further in this chapter.

4.2 Analysis Procedure and Acceptance Criteria

A progressive collapse analysis is performed to check the ability of structure to resist the extreme and unlikely loads coming on structure. The analysis methods used are threat independent, which means considering the removal of vertical support i.e. column was result of some short duration abnormal loading. This short duration abnormal loading can be blast load or result of car-accident or any other cause. When a RC framed structure witnesses any sudden removal of column then structure behaves dynamic, characterized by the significant material and geometric nonlinearity. Methods of analysis used to determine the possible damage to structure due to progressive collapse includes simple 2D linear procedure to complex 3D non-linear dynamic analysis.

4.2.1 Linear Static Analysis (LSA)

Linear static analysis is the simplest method of analysis. In this method, intended column is removed from its location and the analysis with following load combination is carried out.

Load Combination = 2(DL+0.25LL) …(4.1)

Where,

DL = Dead load

LL = Live load

Here in static analysis, the applied gravity load is amplified by factor '2' to approximately account for both nonlinear and dynamic effects. After performing analysis, Demand Capacity Ratio (DCR) at critical locations are computed. DCR is demand by capacity ratio. Demand will be obtained from analysis result with load combination given in equation 4.1 and capacity of the section of the member will be obtained from original design of structure. These DCR will be checked against their acceptance criteria given by GSA guidelines. DCR calculated from linear static analysis will help to determine the potential for progressive collapse of structure.

4.2.2 Non-Linear Static Analysis (NLSA)

Non-Linear Static Analysis well known as 'Pushover Analysis' is used to analyze a building for lateral load. This method takes into consideration both material and geometric nonlinearity. In pushover analysis the lateral load on the building is increased step-wise until the maximum load is attained. For progressive collapse analysis, vertical pushover analysis is performed in which the amplified gravity load will be applied step-wise until the complete load attains. In software this step-wise load increment will be simulated by creating a non-linear load case. In this method, plastic hinge formation is studied. For that purpose automatic hinge properties or user-defined hinge properties can be assigned in the software. In case of automatic hinge properties, program will automatically generate hinge property at given specified location.

There are five default hinge properties available viz., Shear (V2 or V3), Axial (P), Torsion (T), Flexural Hinges i.e. Moment (M2 or M3) and Interacting Hinge (P-M2-M3). Preliminary studies indicated that collapse of the RC building under column removed case is governed by the flexural mode of failure for a beam element. For current study only moment hinges in beam were considered. SAP2000 program will use Table 10-7 of ASCE 41-17 code for default hinge properties. A graphical representation of moment-theta curve of hinge is shown in Fig. 4.1. This curve is drawn w.r.t. generalized force vs deformation curve given in FEMA 356 (Chapter 2). Moment and rotation values are normalized by dividing the actual values by scale factors. Moment values are normalized by dividing them by yield moment of respective member. Initial ascending part of curve i.e. segment AB is straight with x-coordinate zero because in this region beam will behave elastically and return to its original form without any permanent rotation. The portion before yielding is calculated by the program automatically and need not to be provided by the user. Point B is where hinge starts to yield. Point C represents the ultimate capacity of hinge after which the hinge capacity immediately drops to point D. Point D represents the residual strength of the hinge thereafter goes upto point E where total failure of hinge is reached. Approximately residual capacity is taken as 20% of ultimate capacity of hinge. Using Table 10-7 of ASCE 41-17, modelling parameters and numerical acceptance criteria for RC beams can be specified. When 'User-Defined Hinge Properties' option is to be used at that time this table is used or else just use 'Automatic Hinge Properties'. Numerical Acceptance Criteria in Table 10-7 will include the limits for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) level.

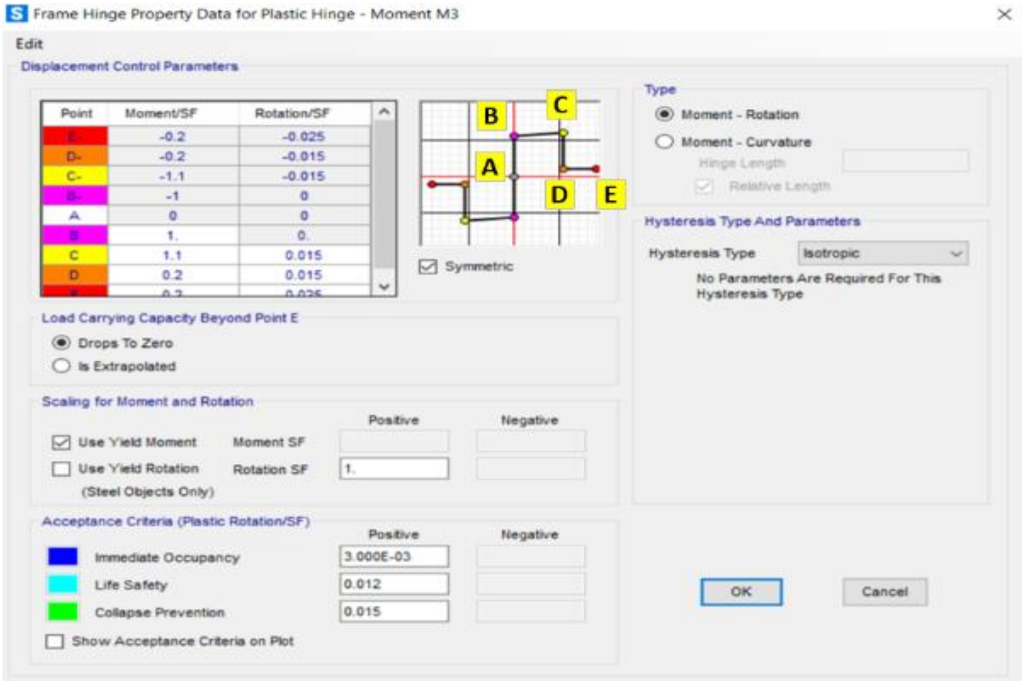

Fig. 4.1 Moment-theta curve of a moment hinge

4.2.3 Acceptance Criteria for Progressive Collapse

The results obtained from the linear static analysis are to be studied to determine the extent of distribution of the potential demands on other structural members. On removing the column form the building, there is need to check which members have exceeded their respective maximum demands. The magnitude and distribution of demands will be indicated by DCR. It is calculated as follows:

$$\mathrm{DCR} = \frac{\text{Demand}}{\text{Capacity}} = \frac{Q_{UD}}{Q_{CE}} \qquad \ldots(4.2)$$

Where,

Q_{UD} = Acting force (demand) determined in a member (moment, axial force, shear)

Q_{CE} = Expected ultimate capacity (capacity) of the member (moment, axial force, shear)

In case of beam, both the demand and capacity of member will be calculated in terms of moment. DCR of structural member should be less than 2 to avoid flexure failure and less than 1 to avoid the shear failure.

4.3 Details of Existing Structure

Fig. 4.2 Front view of GST Bhavan

Fig. 4.3 Side view of building

4.4 Modelling and Analysis of Existing Structure using SAP2000

The existing structure, GST Bhavan is a 6 storey RC framed structure located in Pune city. It was modelled and analyzed in SAP2000 program. Details of modelling and analysis carried out in software are discussed below.

4.4.1 Modelling of Structure in SAP2000

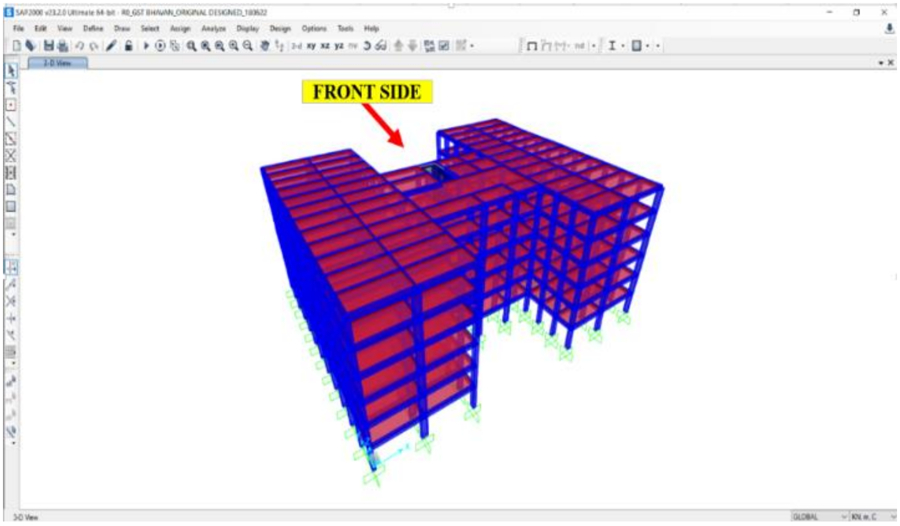

Fig. 4.4 3D view of model in SAP2000

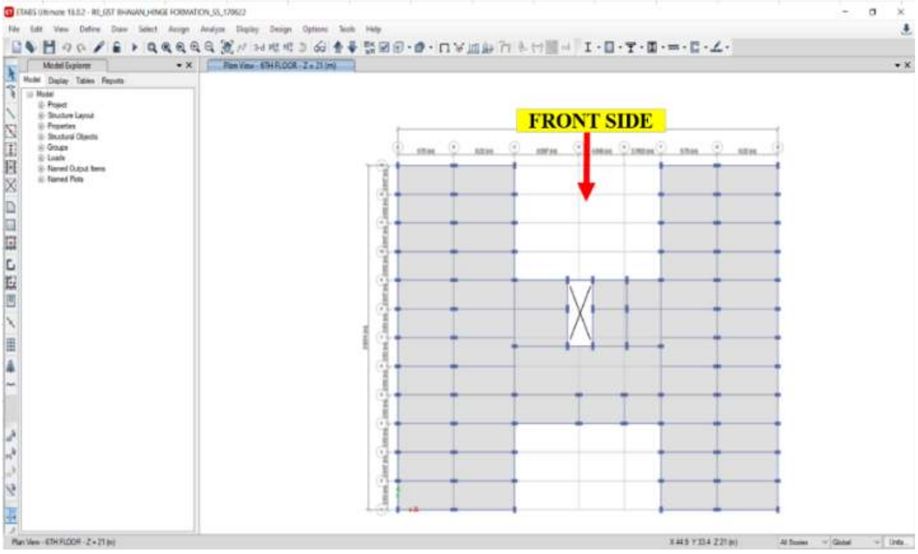

Fig. 4.5 Plan view of model in ETABS

4.4.2 Details of Structure

Structural details of GST Bhavan are mentioned in Table 4.1 below.

Name of Building	GST Bhavan
Location	Pune
Storey	G + 6
Concrete	M25
Steel	Fe415 as main steel Fe250 used for stirrups and ties
Beam Section	300 mm x 500 mm (all beams are of same size)

Column Section	350 mm x 600 mm (External Columns) (Fig. 4.6) 350 mm x 750 mm (Internal Columns) (Fig. 4.7)
Slab Section	125 mm Shell-thin type (all slabs) (Fig. 4.8)

Table 4.1 Structural Details of GST Bhavan

4.4.3 Details of Load Applied on the Building

Different loads acting on the structure are Self-weight, Dead load (DL), Super-Dead Load (SDL), Live Load (LL), Earthquake Load (EQ) and Wind Load (WL). These loads are defined as per IS 875 Part 2 (Fig. 4.9). Wall load is applied on beams, LL and Floor Finish are applied on slab elements. LL applied on the slab elements are taken as per Table 1 of IS 875 Part 2 (Business and Office Buildings). Parapet wall on terrace is of height 1.5 m. Load details for building are given in Table 4.2, 4.3, 4.4 and 4.5 below.

Load Type	**Magnitude**
Dead Load (DL)	Self-weight of member
SDL (Wall Load) Wall thickness 230 mm Wall thickness 115 mm Parapet wall 230 mm	 11.50 kN/m 5.75 kN/m 6.9 kN/m
Live Load (LL)	4.0 kN/m^2 – Passage area slabs 2.0 kN/m^2 – WC slabs 2.5 kN/m^2 – all other slabs
Floor Finish (SDL)	7.65 kN/m^2 – WC slabs 1 kN/m^2 – all other slabs

Table 4.2 Load Details for Ground to 5th floor

Load Type	**Magnitude**
DL	Self-weight of members
SDL - Parapet wall	6.9 kN/m
LL	1.5 kN/m2
FF	1.0 kN/m2

Table 4.3 Load Details for Terrace level

Building Location	Pune
Seismic Zone	3
Seismic Zone Factor (Z)	0.16
Soil Type	Medium
Importance Factor (I)	1
Response Reduction Factor (R)	5

Table 4.4 Earthquake Load Details

Time period was calculated manually using following formula mentioned in Cl. 7.6.2 (c) of IS 1893 (Part 1)-2016.

$$T = \frac{0.09*h}{\sqrt{d}} \quad \ldots(4.2)$$

where,

T = time period (sec)

h = height of building (m)

d = base dimension of building in considered earthquake direction (m)

Location	Pune
Basic wind speed (V_b)	39 m/s
Terrain Category	4
Importance Factor (I)	1
k1	1
k3	1

Table 4.5 Wind Load Details

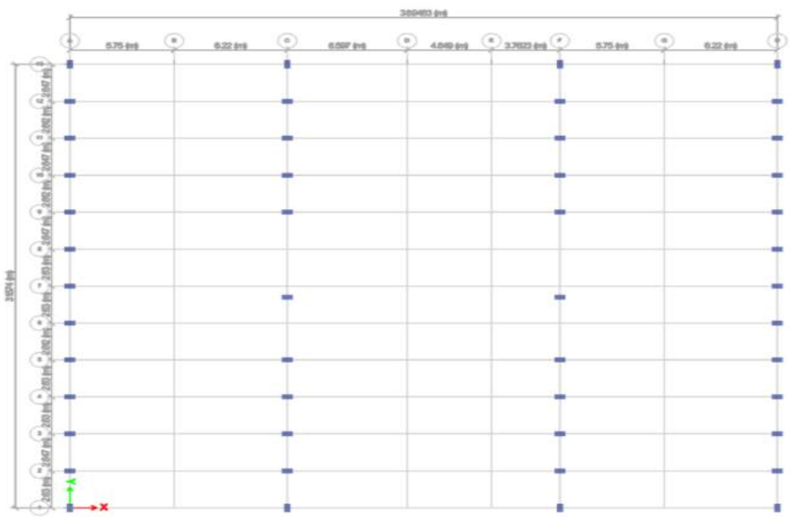

Fig. 4.6 Column positioning & orientation of 350 mm x 600 mm in ETABS

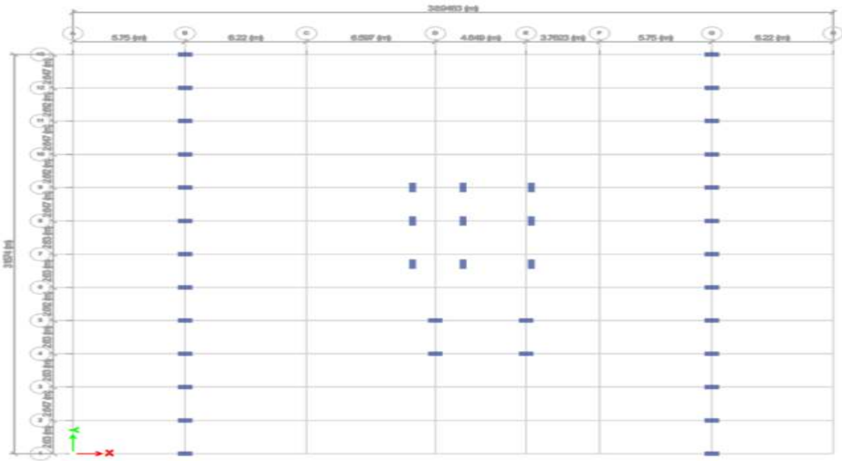

Fig. 4.7 Column positioning & orientation of 350 mm x 750 mm in ETABS

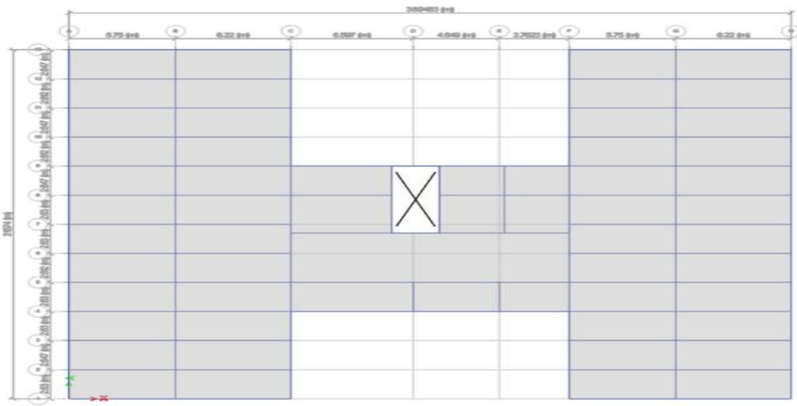

Fig. 4.8 Slab provided seen in Plan view in ETABS

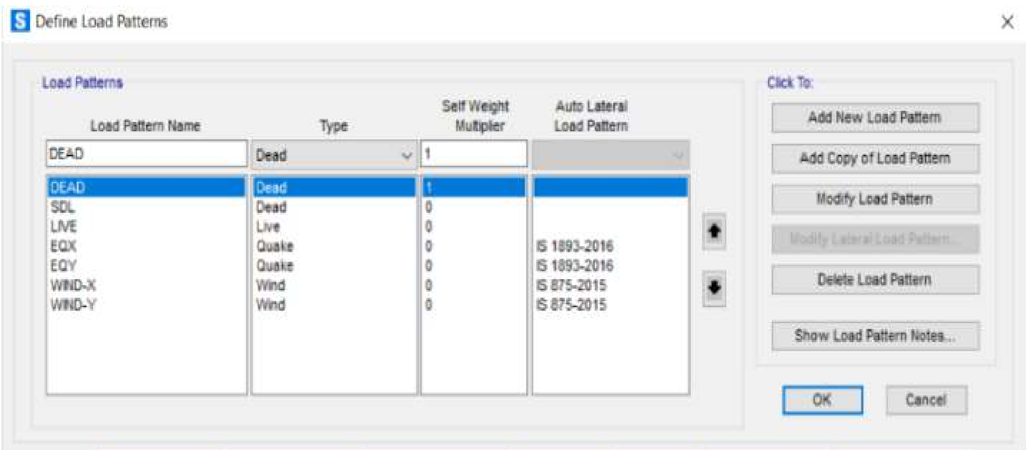

Fig. 4.9 Load patterns defined in SAP2000

4.4.4 Load Combination and Some Additional Things

The building is analyzed and then designed for the load combinations as per IS 456:2000. Once model is created, define a 'Rigid Diaphragm' separately for all floors and assign it. Also, define a mass source as shown in Fig. 4.10 below. Mass source needs to be defined for seismic design of structure as during earthquake not 100% LL will be present. So, according to IS 1893-2016, consider 25% of LL while earthquake analysis.

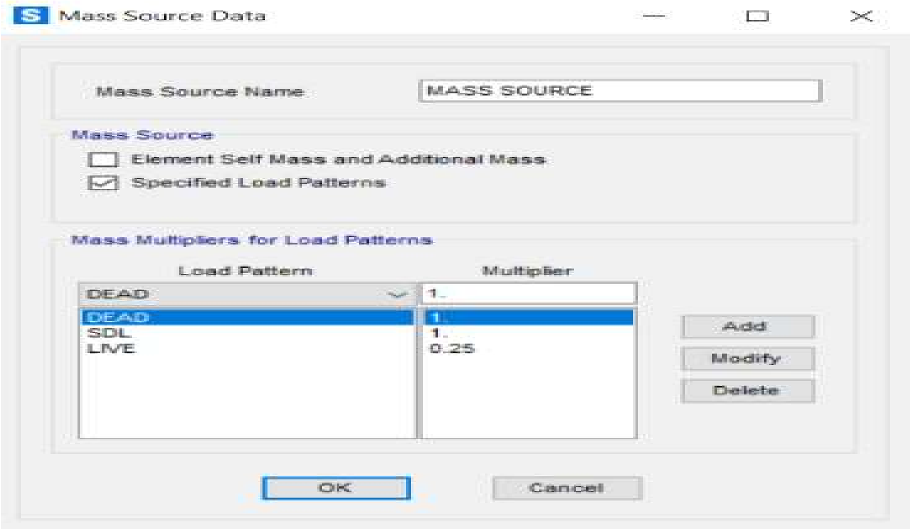

Fig. 4.10 Mass source defined in SAP2000.

4.4.5 Column Removal Cases

As per GSA 2003 guidelines, different column removal cases are to be considered and separate analysis needs to be performed for these cases. For existing structure following column removal cases were studied (Fig. 4.11).

a. Case 1: Removal of middle column on longer side (Blue circle)

b. Case 2: Removal of middle column on shorter side (Black circle)

c. Case 3: Removal of corner column (Red circle)

d. Case 4: Removal of interior column (Yellow circle)

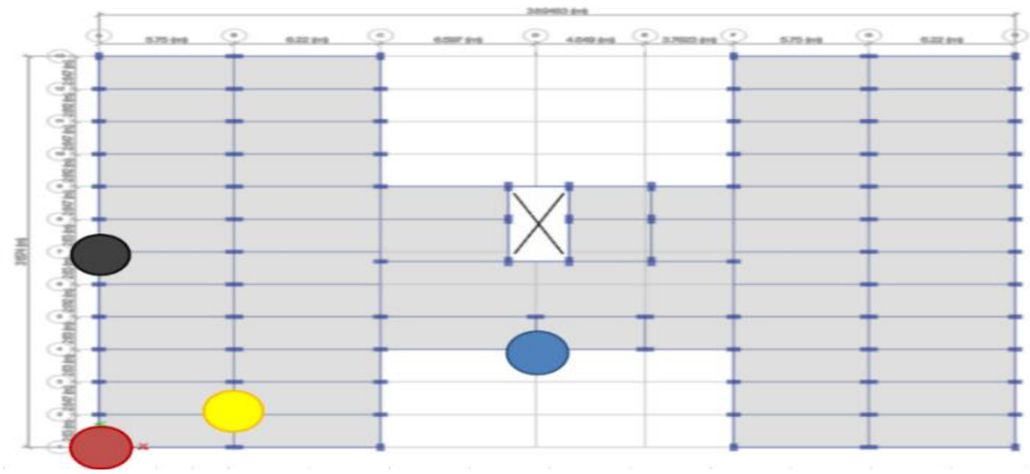

Fig. 4.11 Location of column removal cases in ETABS

4.5 Step-by-Step Procedures

Following are various procedures discussed to perform the respective actions.

4.5.1 Procedure to Perform Linear Static Analysis

Following are the steps to perform Linear Static Analysis in SAP2000 program and to calculate DCR.

1. Create a model of G+6 storey building in SAP2000 using beams, columns and slabs of their respective sizes.
2. Define load patterns and apply loads on the building as discussed above.
3. Define diaphragm and mass source.

4. Analyze and design the building for IS 456-2000 load combinations.
5. On completion of design, reinforcement detailing for each member will be obtained. Save the results of reinforcement detailing for DCR calculation.
6. Removal the desired column from its position (directly delete it).
7. Create a new load combination as mentioned in Equation 4.1.
8. Again analyze the model and determine BMD for moment 3-3, to determine the 'demand' coming on building members.
9. DCR can be calculated, knowing demand from latter analysis and obtaining the capacity from former design of the building.
10. Sample calculation of DCR is shown below.

4.5.2 Procedure for Calculating Demand Capacity Ratio (DCR)

To determine the expected material strength (for determining capacity), design material strength may be increased by strength increase factor (Table 4.6). Capacity of section is calculated as per IS 456-2000.

Material	Strength Increase Factor
Concrete	1.25
Steel	1.25

Table 4.6 Material strength increase factor

Illustration for a DCR of a beam for case 1: removal of middle column on longer side of building. Location of that column is shown in Fig. 4.12.

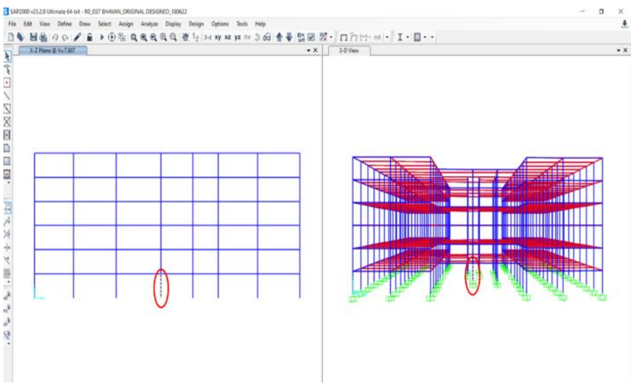

Fig. 4.12 Removal of middle column on longer side

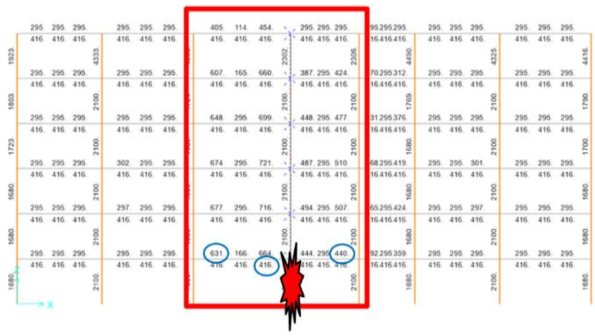

Fig. 4.13 Reinforcement detailing of building

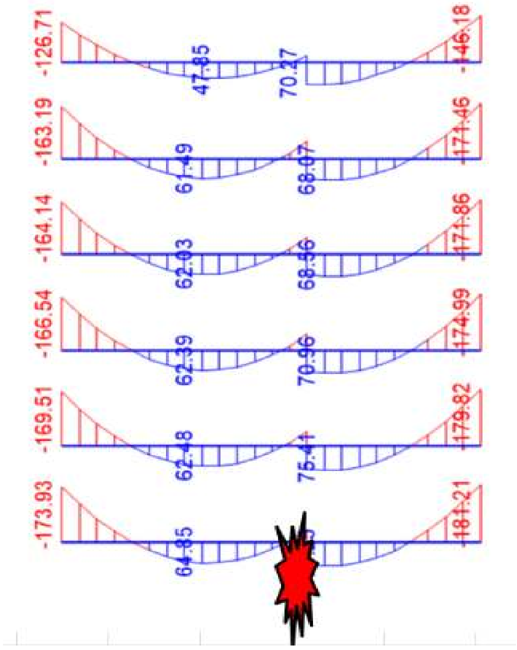

Fig. 4.14 BMD for beam corresponding to load combination = 2(DL + 0.25LL)

Beam on right side of 'removed column'

A_{st} = 440 mm^2 (Fig. 4.13)

fy = 1.25 x 415 MPa = 518.75 MPa

fck = 1.25 x 25 MPa = 31.25 MPa

Here, b=300, D=500, d=470 mm

$$x_u = \frac{0.87*fy*Ast}{0.36*fck*b} \quad \ldots(4.3)$$

MR = $0.87*fy*A_{st}*(d-0.42x_u)$

= 0.87*(1.25*415)*440*(470-0.42*58.838)

MR = 88.424 KNm = Capacity

Demand = 181.21 KNm

$$DCR = \frac{181.21}{88.424} = 2.049$$

Similarly, for section at left end of beam (left beam of column removed) and right section of beam (left beam of column removed)

$DCR_{left\ section}$ = 1.405 …corresponding to A_{st} = 631 mm^2

$DCR_{right\ section}$ = 0.885 …corresponding to A_{st} = 416 mm^2

4.5.3 Procedure to Perform Non-Linear Static Analysis

Following are the steps to perform Non-Linear Static Analysis in SAP2000 program and to assign plastic hinge.

1. Model of G+6 building is analyzed and designed as per IS 456-2000.
2. Unlock the model and create a new load combination as mentioned in Equation 4.1.
3. Analyze and design building for new load combination.

(This step is required to get value of 'Design Shear (V)' for new load combination for every beam)

4. Unlock the model and assign auto-hinge properties to all beams.
5. Remove the required column.
6. Run analysis again without Earthquake, Wind and Model cases.
7. Determine maximum displacement at joint of removed column.
8. Create non-linear load case i.e. force-based non-linear case.

9. Again run analysis without Earthquake, Wind and Model cases.
10. Check damage level in hinges and determine the hinge sequence.
11. Determine the displacement of joint (top joint of removed column) by just hovering the cursor over the particular joint.

4.5.4 Procedure to Define Auto-Hinge Property for Beam in SAP2000

Following is the procedure to assign 'Auto-hinge properties' to beams in SAP2000.

1. Select all the beams of the building.
2. Go to 'Assign' option, then to 'Frame' and select on 'Hinges' (Fig. 4.15).

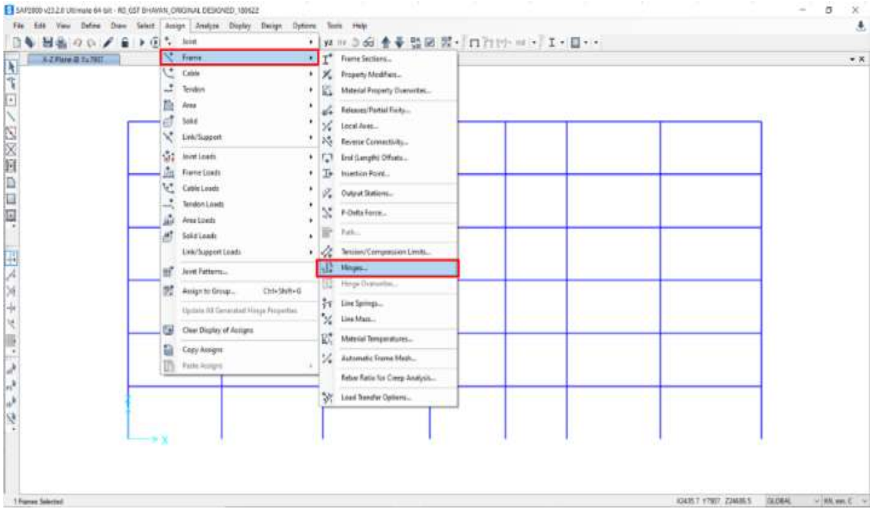

Fig. 4.15 Hinge option in SAP2000

3. Add hinges at relative distance of '0'. For relative distance of '0', click on 'Add Hinge' option (Fig. 4.16).

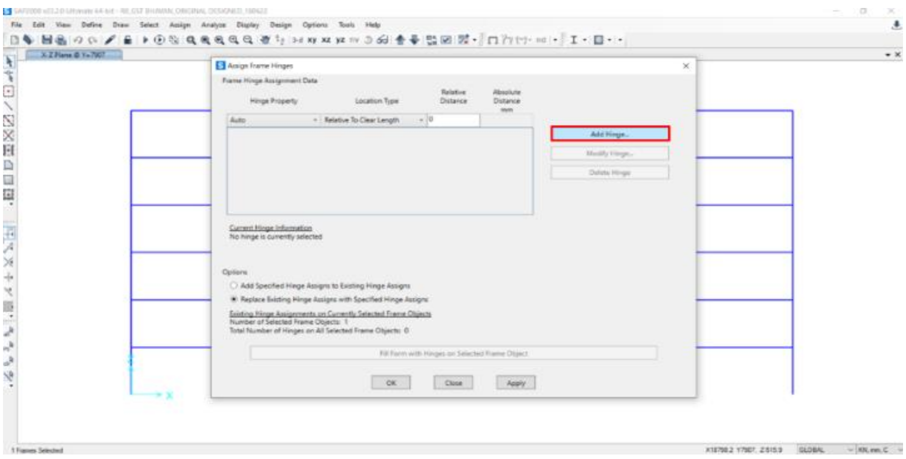

Fig. 4.16 'Add Hinge' option

4. As shown in Fig. 4.17, select 'Tables in ASCE 41-17' so that program considers auto-hinge properties from ASCE 41-17 code. Select 'Table 10-7 for Concrete beam', as hinge is assigned to beam element. DOF as M3, moment about vertical axis will govern the beam flexural failure. Design Shear (V) should be calculated from new load combination mentioned in Eq. 4.1. Assume transverse reinforcement as 'Non-conforming' to get safer side results. Click 'Ok' to complete the process.

5. Similarly, assign plastic hinge (PH) at relative distance '1'.

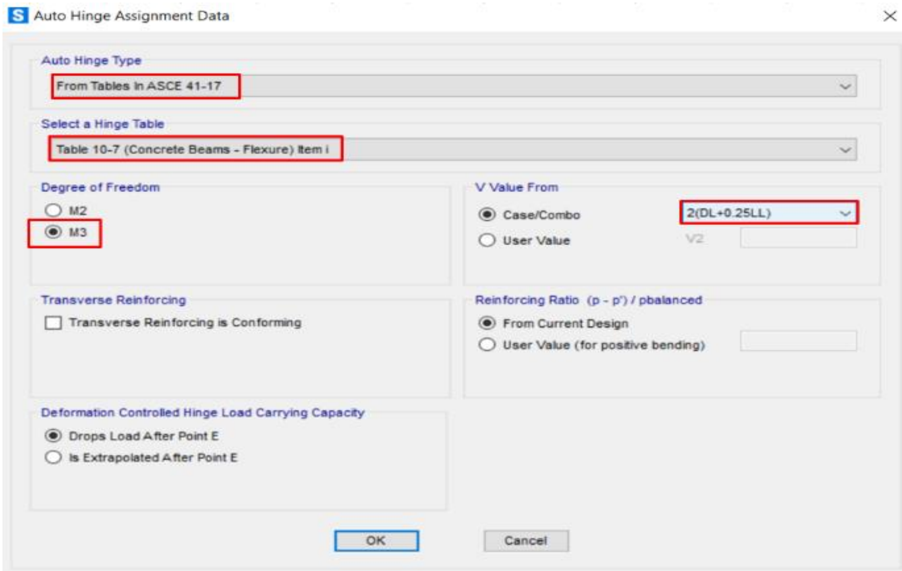

Fig. 4.17 Options to be selected while assigning auto-hinge property to a beam element

4.5.5 Procedure to Define Non-Linear Load Case in SAP2000 and to View Plastic Hinge Formation

In order to get the PH formation in the structure, non-linear load case has to be defined. It can either be force-based case or deformation-based case. For this project as number of loads are more than 1, force-based load case was created. Following is the procedure to form a non-linear load case.

1. Load patterns should be defined before defining the non-linear load case.

2. Go to 'Define' option and select 'Load Cases' (Fig. 4.18).

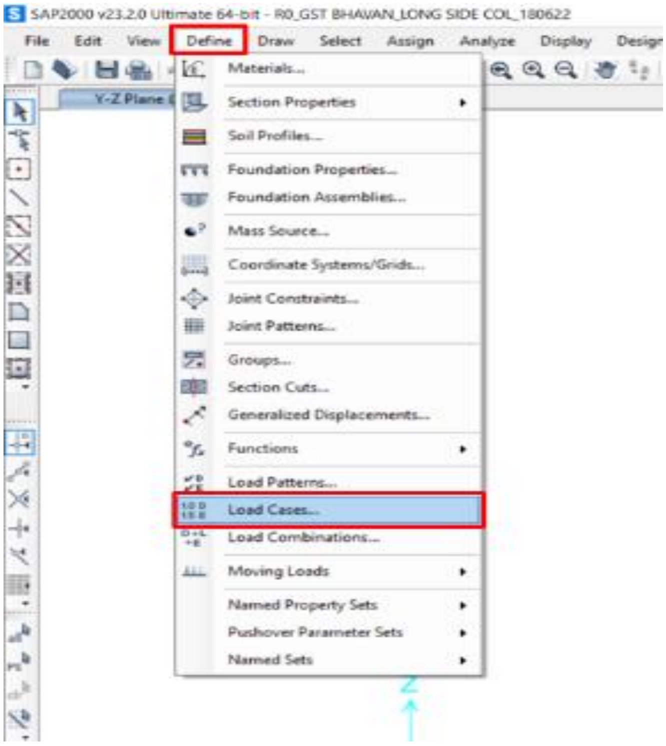

Fig. 4.18 Load Cases option in SAP2000

3. Create a new load case by name ‘Force’.
4. As shown in Fig. 4.19, select ‘Static’ load case type. Analysis to be performed is ‘Non-linear’. As only material non-linearity is to be considered, select ‘None’ for ‘Geometric Non-Linearity’. Add the loads to be applied step-wise on the building. As per new load combination, DL and SDL is applied 2 times and LL is applied 0.5 times on building.

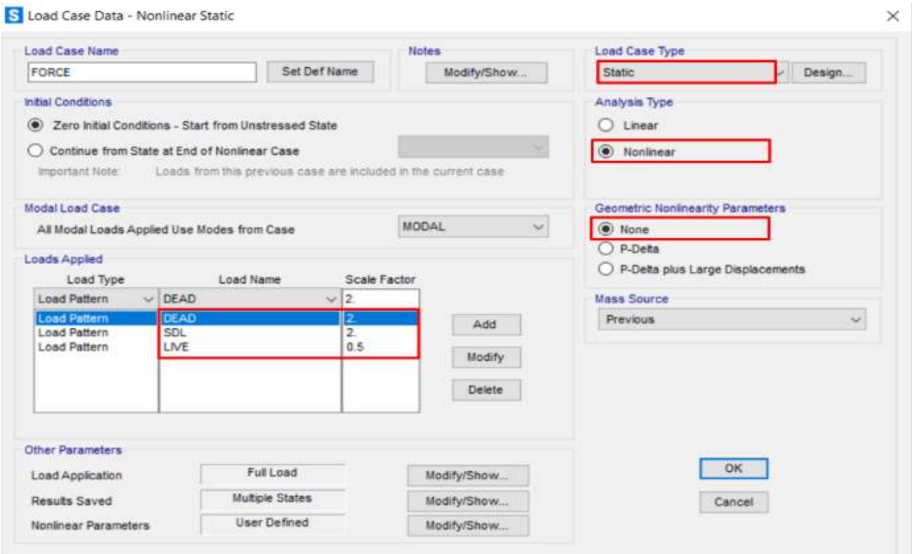

Fig. 4.19 Non-linear load case details

5. In other parameters option, details of ‘load application’ is shown in Fig. 4.20. Select displacement (DoF) in vertical direction i.e. U3 at joint corresponding to column removal location (top joint of column removed).

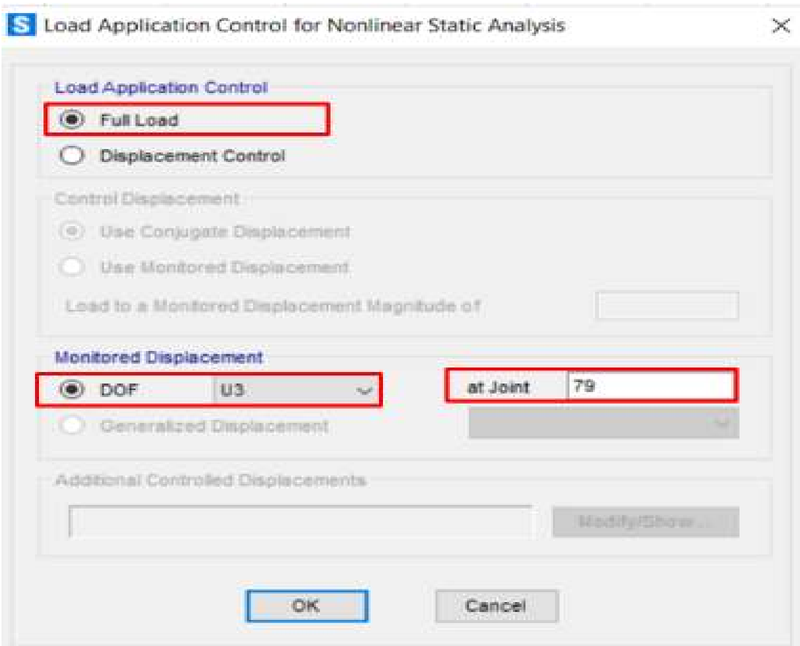

Fig. 4.20 Load application details

6. Details of ‘Result Saved’ are shown in Fig. 4.21. To view hinges, divide the whole load application in no. of steps. Results are to be saved for each steps so select ‘Multiple States’ option and un-tick ‘Save positive displacements increments only’ option.

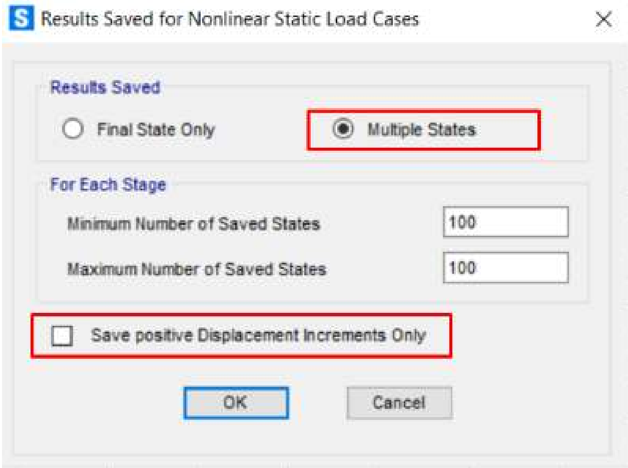

Fig. 4.21 Details of 'Result Saved' option

7. Details of 'Nonlinear Parameters' are shown in Fig. 4.22. Change no. of steps to 100, steps in which the load has to be divided.

Fig. 4.22 Details of 'Nonlinear Parameters'

8. Click 'Ok' to finish the procedure.

9. To view the results of non-linear load case created, run analysis without Earthquake, wind and model case.

10. PH formation can be viewed in deformed shape of building. Select 'Show Deformed Shape' from 'Display' option (Fig. 4.23)

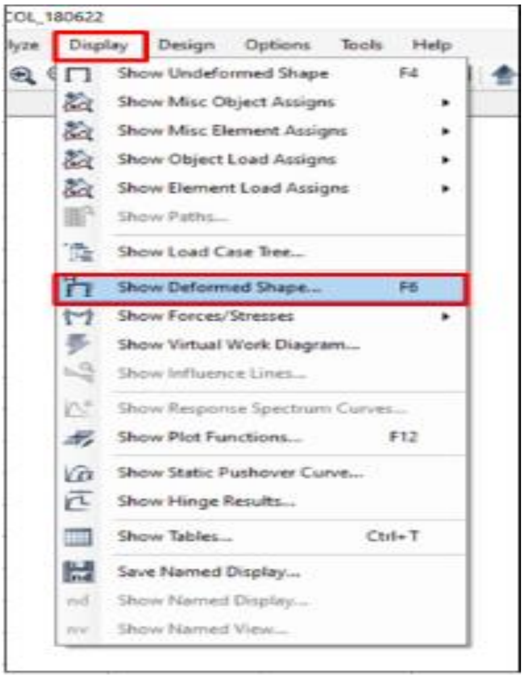

Fig. 4.23 Deform shape option

11. In 'Deform Shape' option, select the non-linear load case 'Force' previously defined. Proceed from step '1' to last step, to determine the hinge sequence formed in the building after removal of column. There are two options to view PH state. 1st option is to see the region in which PH lies on M-θ curve. 2nd option is view damage level of PH.

12. For 1st option, select 'B, C, D and E Points' option (Fig. 4.24). Color of hinge will define the region in which the PH has reached for particular step (Table 4.7). As seen in Fig. 4.25, all PH's are in region BC i.e. all of them have crossed yield moment.

PH Color	Region of M-θ curve	Comments
Grey	AB	PH still not yielded
Green	BC	PH crossed yield point
Sky Blue	CD	PH cross ultimate-strength
Pink	DE	PH in its residual strength
Red	After E	Failure of PH

Table 4.7 Defining PH State

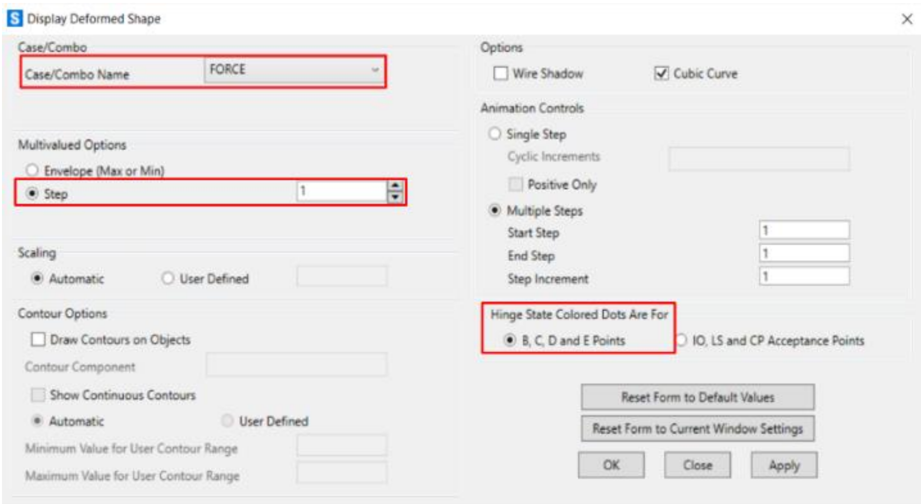

Fig. 4.24 Deformed Shape option to view PH

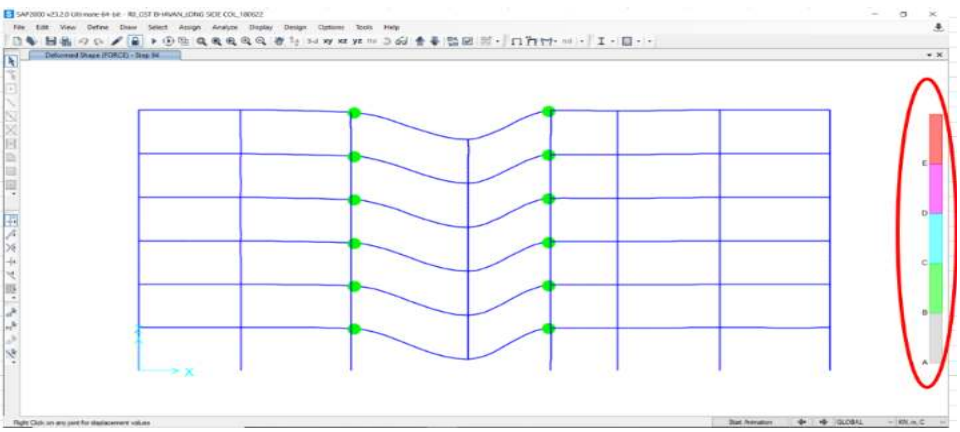

Fig. 4.25 PH state in M vs θ curve

13. For 2nd option, choose 'IO, LS and CP Acceptance Points' (Fig. 4.24). Table 4.8 will show the color corresponding to PH damage level. In Fig. 4.26, all PH's are seen in IO level.

PH Color	Damage Level
Grey	Immediate Occupancy (IO)
Green	Life Safety (LS)
Sky Blue	Collapse Prevention (CP)
Red	Failure

Table 4.8 PH Damage Levels

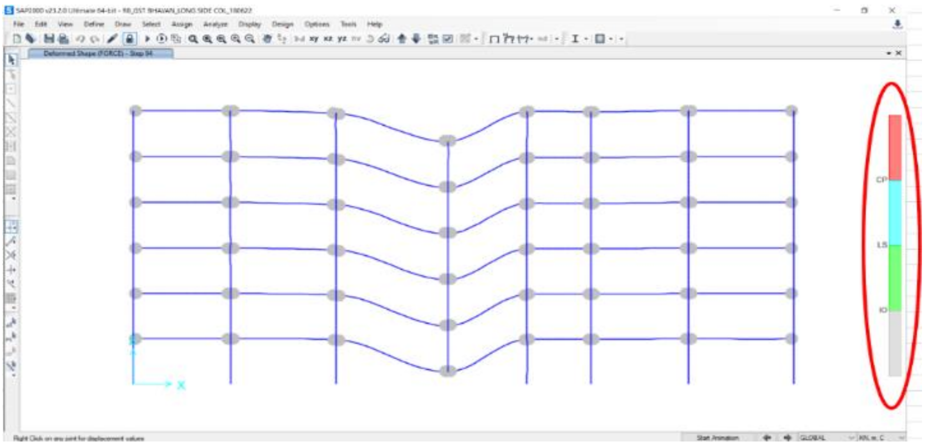

Fig. 4.26 PH damage levels

4.6 Closure

In this chapter, procedure to perform both the analyses method are explained along with their procedure to be followed to perform them in SAP2000 program. Procedure to calculate DCR and assigning PH are also explained. Results obtained for both of them are discussed in next chapter.

RESULTS AND DISCUSSION

5.1 General

In this chapter, results obtained from linear and nonlinear static analysis for different column removal cases are discussed. DCR obtained from LSA and PH formed after performing NLSA are shown. Overall performance of building under different column removal scenarios were studied.

5.2 Results of Linear Static Analysis

Linear Static Analysis is the simplest method of analysis, which is used in current study to determine Demand-Capacity Ratio (DCR) in every structural member. In present work, DCRs were calculated only for beams present in bay adjacent the column removal bay. Location of different column removal cases are shown in Fig. 5.1.

Case 1: Removal of middle column on longer side (Blue circle)

Case 2: Removal of middle column on shorter side (Black circle)

Case 3: Removal of corner column (Red circle)

Case 4: Removal of interior column (Yellow circle)

5.2.1 Case 1: Removal of Middle Column on Longer Side

Frame number 239 (C239) (Fig. 5.1) was removed from its position to determine the potential of progressive collapse. DCRs obtained in X-Z plane are shown in Fig. 5.2. As per GSA (2003) guidelines, flexure DCR of right far end sections of two beams on right side of C239 has crossed flexure acceptance criteria, which means there is high possibility that these members have failed after removal of column. DCR value of far end sections from column removal location on either side are higher than that of near end sections in this case.

DCRs of beams in Y-Z plane are shown in Fig. 5.3. DCR of all the critical sections except one on the last floor had crossed the DCR flexure limit of 2. So, all beams in Y-Z plane have high possibility of being failed after removing column. DCR value of far end sections from column removal location are higher than that of near end sections in this case also.

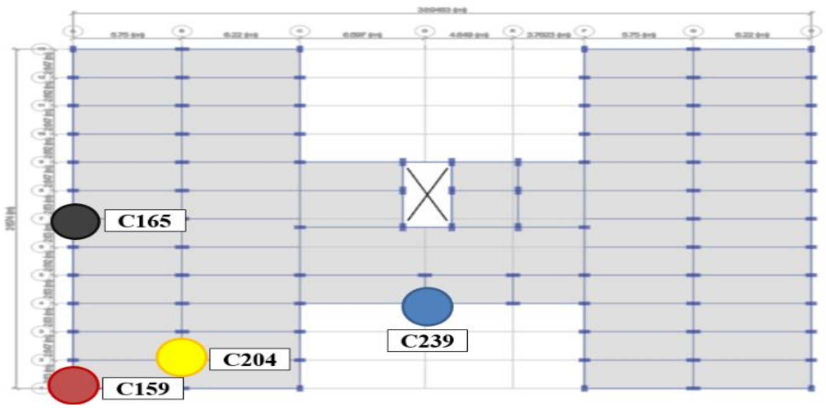

Fig. 5.1 Location of column removal cases considered for analysis

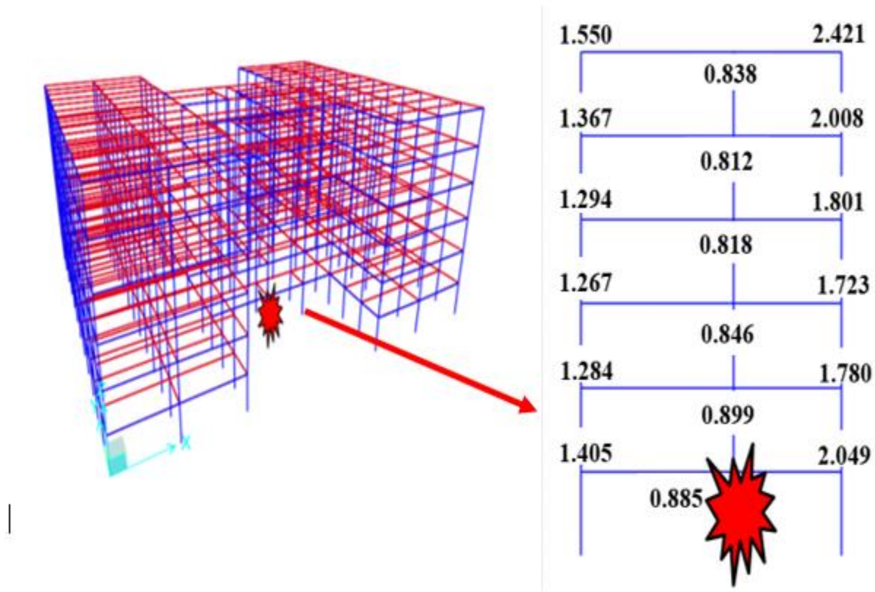

Fig. 5.2 DCR of beam sections in X-Z plane after removing Middle Column on Longer Side C239 (case 1)

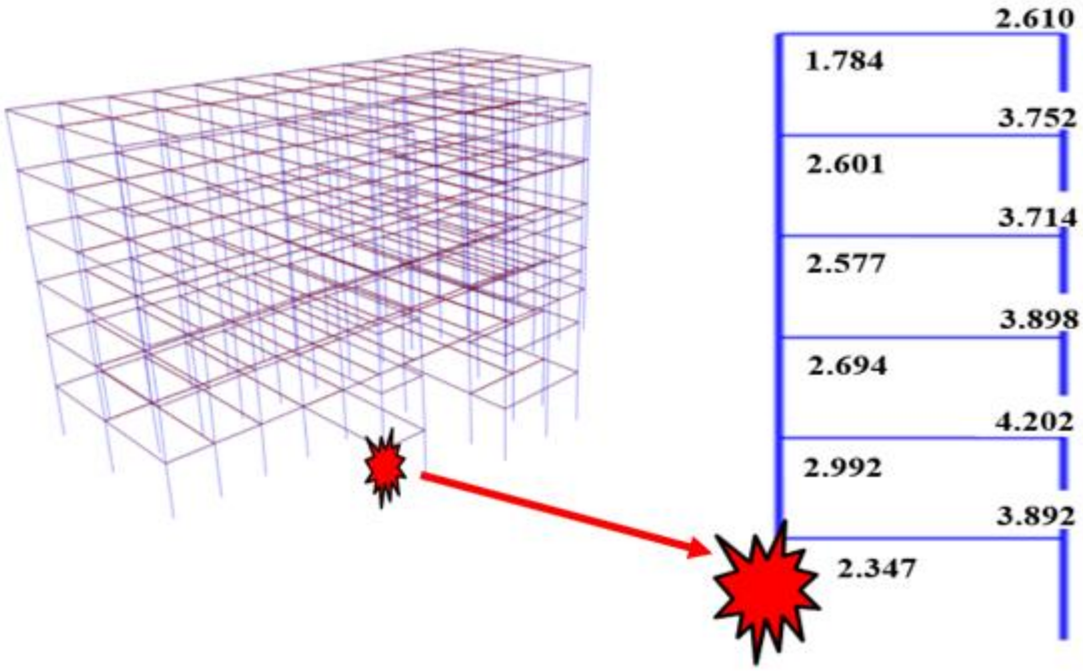

Fig. 5.3 DCR of beam sections in Y-Z plane after removing Middle Column on Longer Side C239 (case 1)

5.2.2 Case 2: Removal of Middle Column on Shorter Side

For case 2, frame number 165 (C165) (Fig. 5.1) was removed from its original position. After performing LSA, following DCR were obtained in X-Z plane (Fig. 5.4). No DCR had crossed flexure failure limit. So as per GSA (2003) guidelines, all beam members in X-Z plane are safe.

DCRs in Y-Z plane are shown in Fig. 5.5. DCR of beam sections located at far end from removed column had crossed flexure limit on 1st and 2nd floor, while ratios of all other section on remaining floors are still within acceptance limit. Beams on first two floors have high probability of being failed. It can be seen in this case too, that DCR value of far end sections from column removal location on either side are higher than that of near end sections.

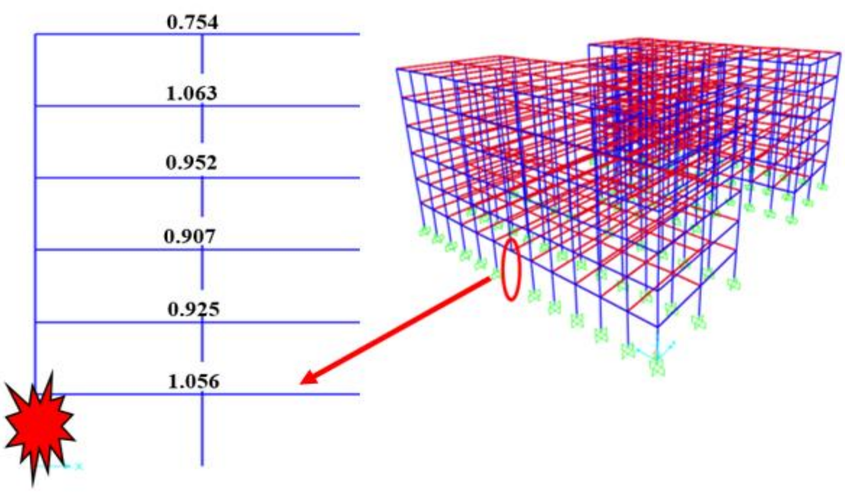

Fig. 5.4 DCR of beam sections in X-Z plane after removing Middle Column on Shorter Side C165 (case 2)

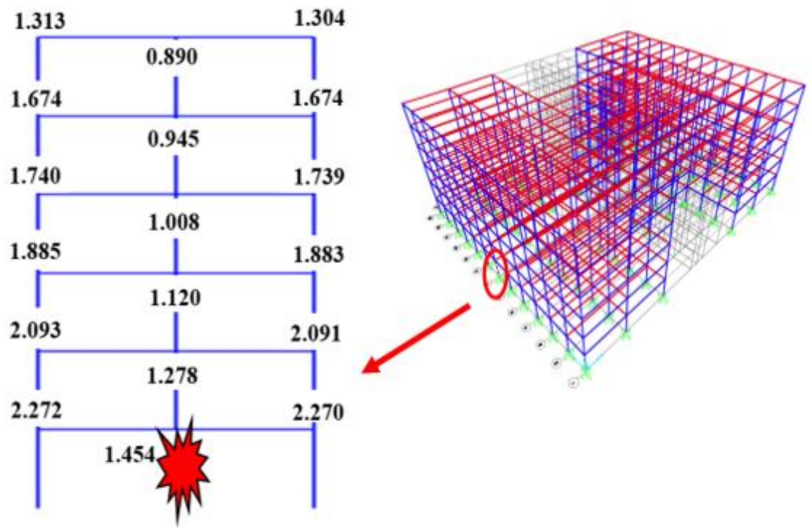

Fig. 5.5 DCR of beam sections in Y-Z plane after removing Middle Column on Shorter Side C165 (case 2)

5.2.3 Case 3: Removal of Corner Column

For corner column removal case, frame number 159 (C159) (Fig. 5.1) was removed from its position and LSA was performed. DCRs in X-Z plane are shown in Fig. 5.6 below. None of the beam section's DCR had crossed the flexure limit. So, all beams in X-Z plane are safe as per GSA guideline. DCR value of far end sections from column removal location are higher than that of near end sections

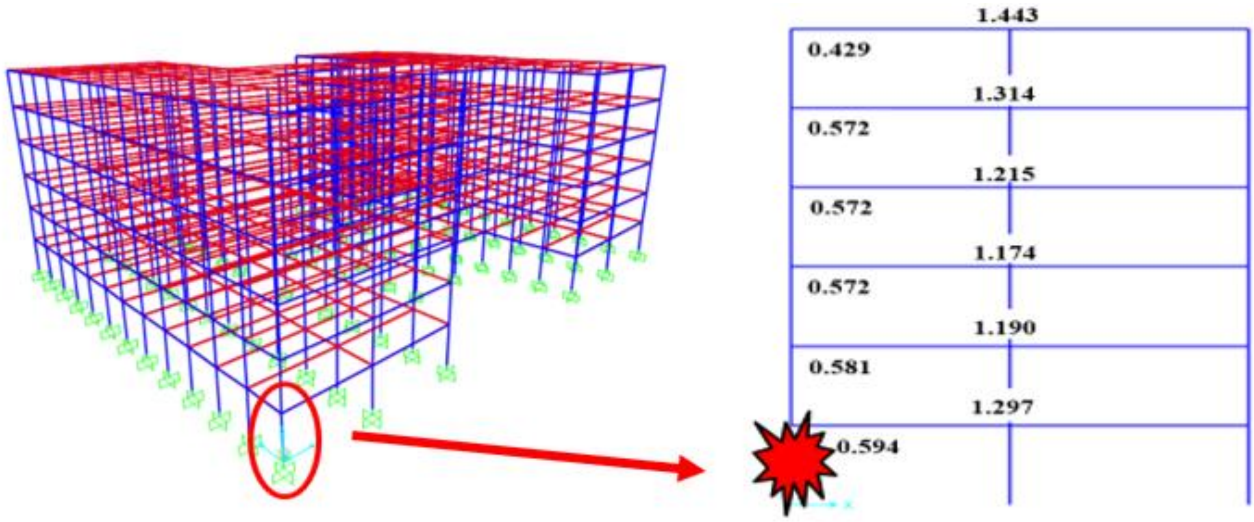

Fig. 5.6 DCR of beam sections in X-Z plane after removing Corner Column C159 (case 3)

DCR in Y-Z plane is shown in Fig. 5.7 below. All the sections on right far end of the beams have crossed the flexure failure limit. There is high possibility of these all beams to be failed on removing the corner column. Similarly, for this case too, DCR value of far end sections from column removal location are higher than that of near end sections.

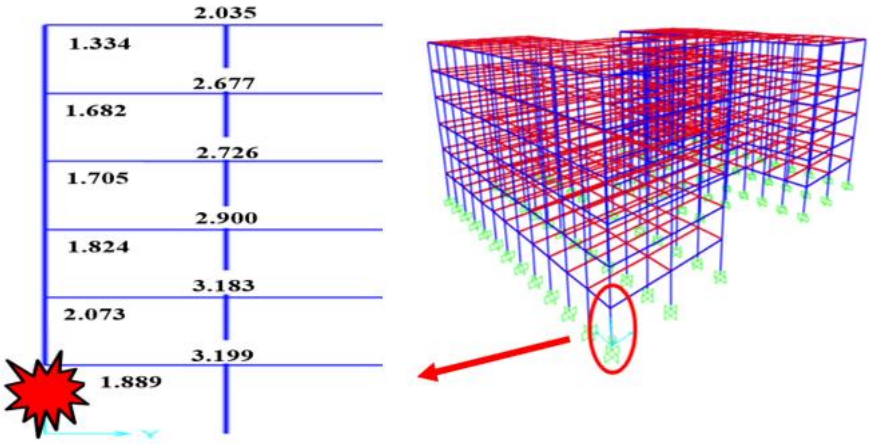

Fig. 5.7 DCR of beam sections in Y-Z plane after removing Corner Column C159 (case 3)

5.2.4 Case 4: Removal of Interior Column

For case 4, an interior column i.e. frame number 204 (C204) was removed. DCR of beams in X-Z plane is shown in Fig. 5.5. DCR of all critical section was found to be within permissible limit specified by GSA guidelines. So, chances are that all beams in this plane are safe even after removal of column because the load has been safely re-distributed among these structural members.

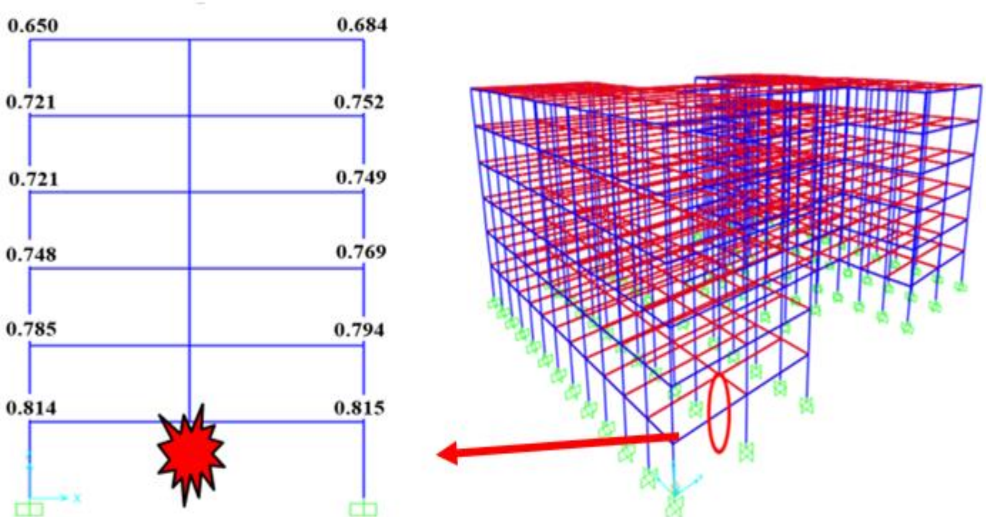

Fig. 5.8 DCR of beam sections in X-Z plane after removing Interior Column C204 (case 4)

DCR of beam sections in Y-Z plane is shown in Fig. 5.6 below. No any DCR has crossed the permissible value of flexure limit. So, as per GSA guidelines, chances are that all beam members are safe even after failure of interior column. The load coming on the column has been re-distributed safely.

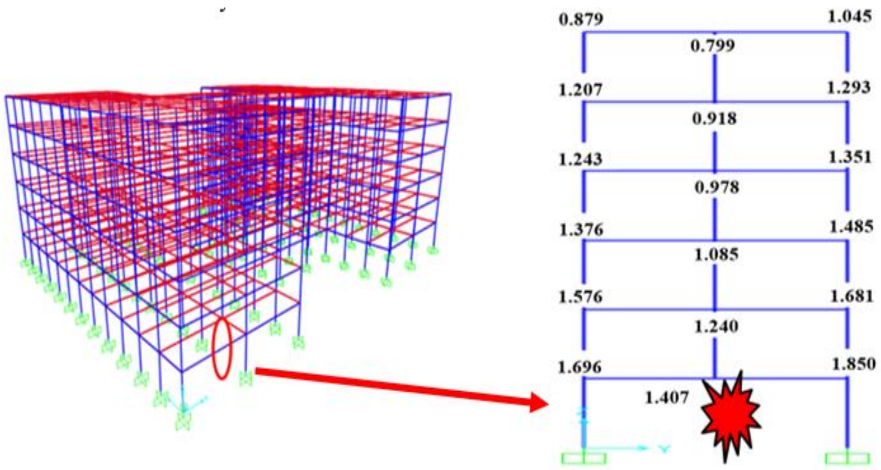

Fig. 5.9 DCR of beam sections in Y-Z plane after removing Interior Column C204 (case 4)

5.3 Results of Non-Linear Static Analysis

The non-linear static analysis was performed to determine the formation of hinge in the beams. Load coming on the building was increased step-wise to determine the hinge sequence corresponding to a particular displacement. Results of PH formation on removing a column is discussed for each case.

5.3.1 Case 1: Removal of Middle Column on Longer Side

On removal of C239, middle column on longer side, NLSA was performed and PH formation pattern under non-linear load case was studied. Final state of PH formation is shown in Fig. 5.10. It can be seen that, PHs are formed only in beams adjacent to column removal bay.

In Fig. 5.11, it can be seen that as load increases, plastic hinges starts forming at various different critical sections of the beam. It can be stated that, PH initially formed in shorter length (right-side) beam and then in longer length (left-side) beam.

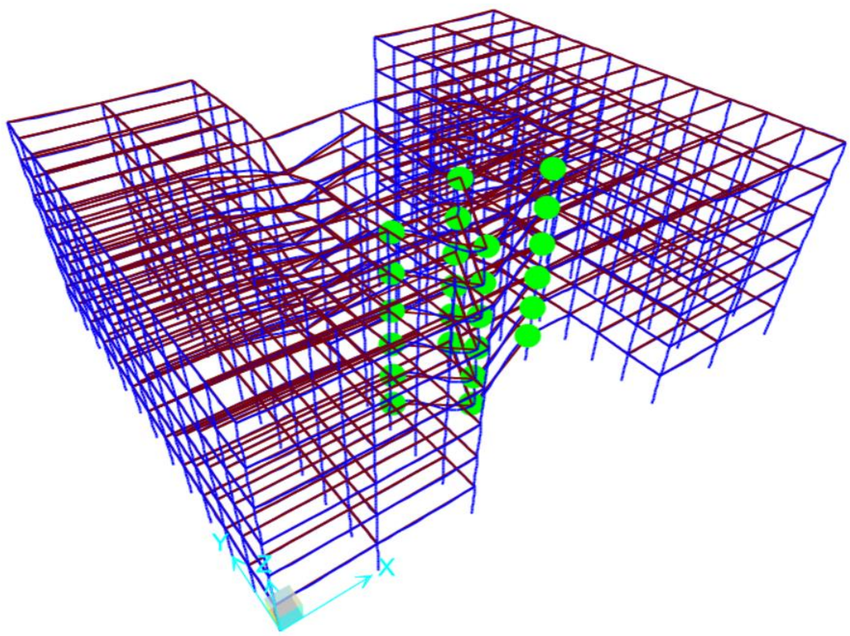

Fig. 5.10 Final state of PH formation for Removal of Middle Column on Longer Side

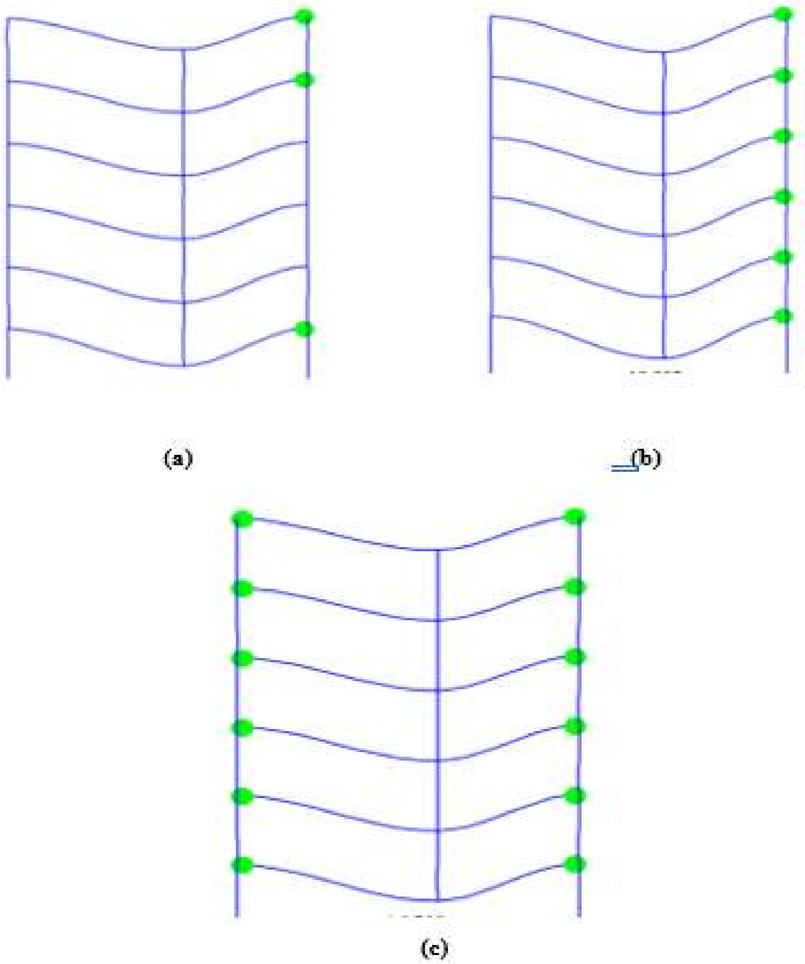

Fig. 5.11 PH formation in X-Z plane beams for Removal of Middle Column on Longer Side

PH formation in Y-Z plane can be seen in Fig. 5.12. As loading progresses, PHs are formed initially at far end of the beam from column removed location. Later formed at near end of the beam.

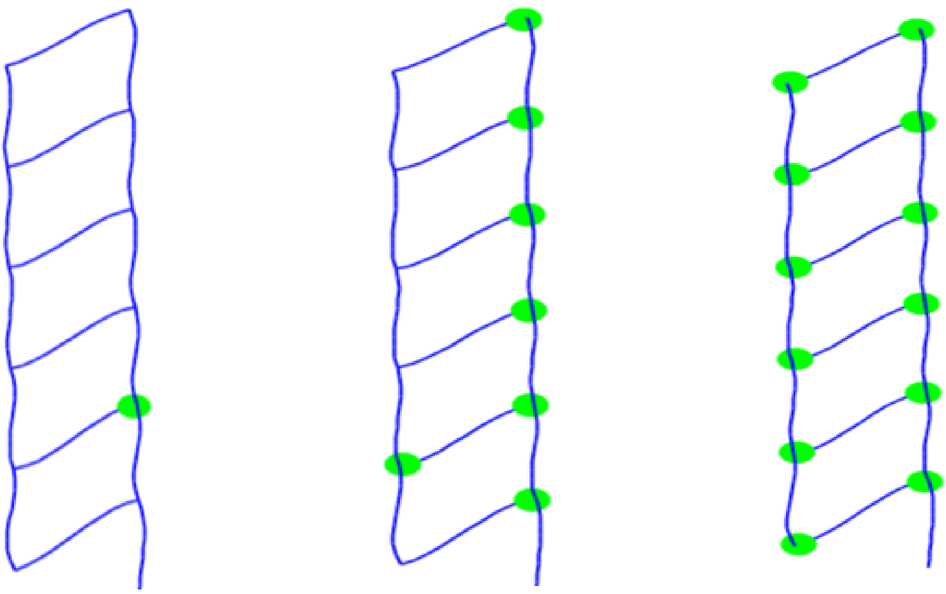

Fig. 5.12 PH formation in Y-Z plane beams for Removal of Middle Column on Longer Side

5.3.2 Case 2: Removal of Middle Column on Shorter Side

For case 2, C165 was removed from its location. Final state of PH formation after column removal is shown in Fig. 5.13. In this case also, PHs were formed in beams in bay orthogonal to column removal bay.

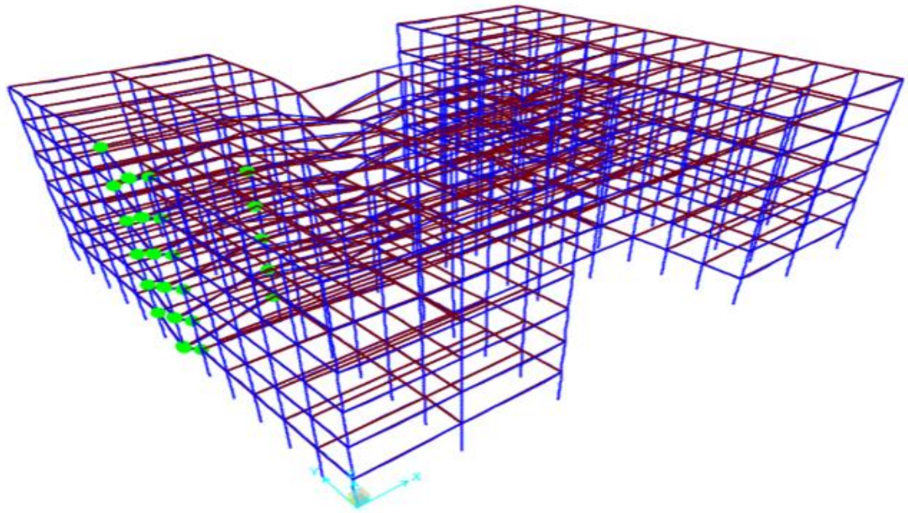

Fig. 5.13 Final state of PH formation for Removal of Middle Column on Shorter Side

PH formation for Y-Z plane are shown in Fig. 5.14. As load increases, PH starts forming initially in bottom floors beams and progresses to top floor beams. Till it reached final state, PH was formed at both critical sections of all the beams, initially forming at the far end of beam. As beam length on either side of this column are less than that in orthogonal direction, PH were formed at both the end of beam.

PH formation in X-Z plane are shown in Fig. 5.15. As loading progresses, PH starts forming from top floor beam. As beam length in this plane is more than that of Y-Z plane, PHs were formed only at the far end of the beam.

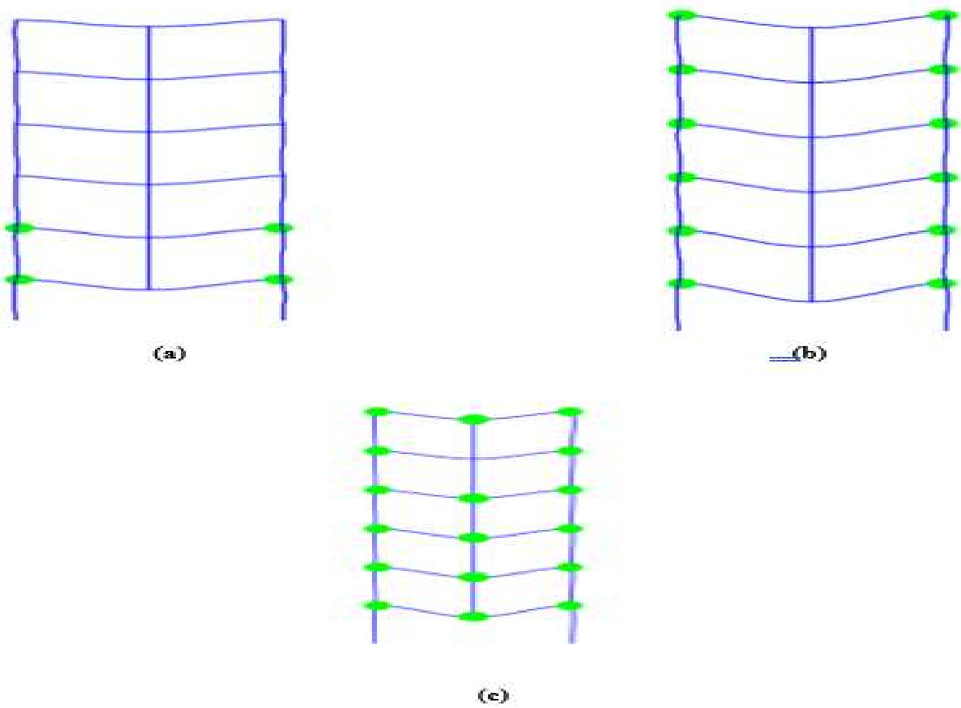

Fig. 5.14 PH formation in Y-Z plane beams for Removal of Middle Column on Shorter Side

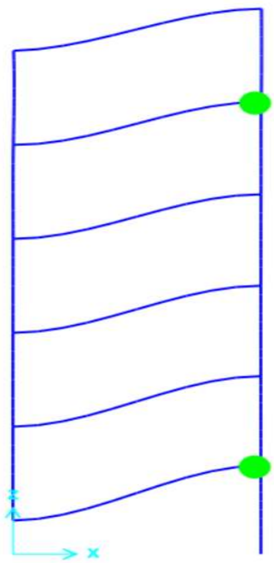

Fig. 5.15 PH formation in X-Z plane beams for Removal of Middle Column on Shorter Side

5.3.3 Case 3: Removal of Corner Column

For corner column removal case, C159 was removed and NLSA was performed. Final state of PH formation (Fig. 5.16) showed that PHs were formed only in orthogonal planes connected to column removal location.

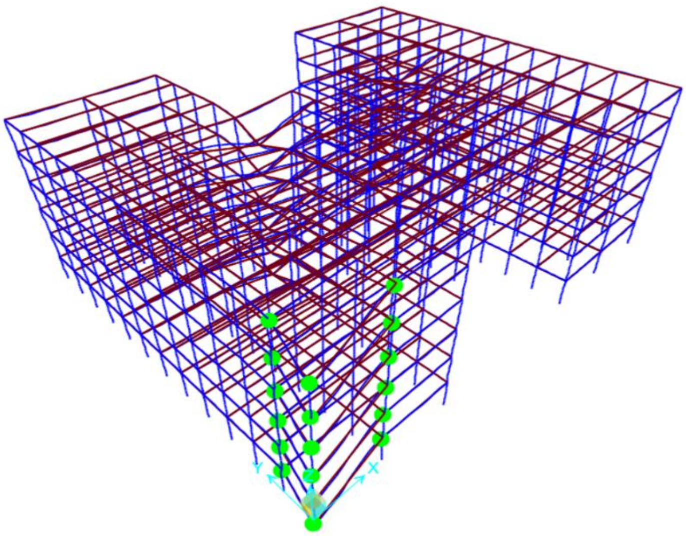

Fig. 5.16 Final state of PH formation for Removal of Corner Column

PH formation in X-Z plane is shown in Fig. 5.17. On removal of column, PH first started at top beam and as final state of loading was reached, PHs were formed only at far end on beam as beams in X-Z plane are longer than that of orthogonal beams in Y-Z plane.

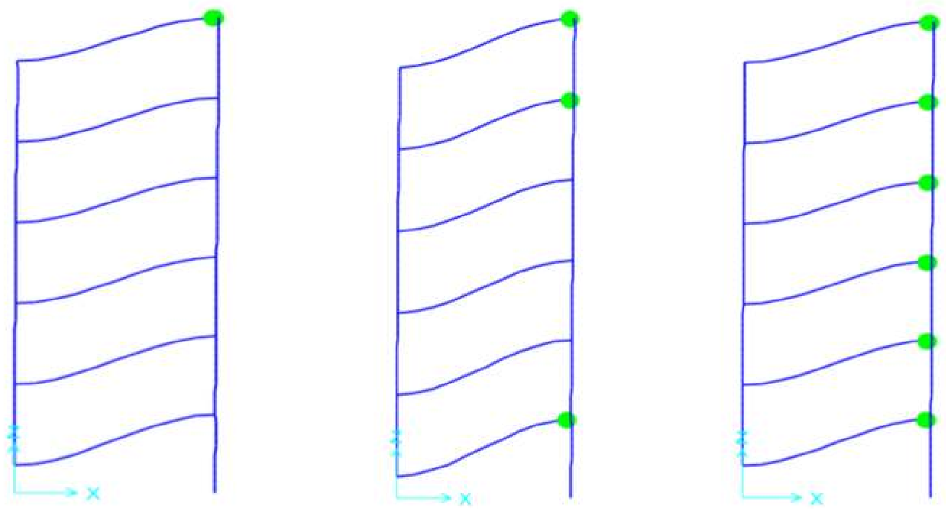

Fig. 5.17 PH formation in X-Z plane beams for Removal of Corner Column

PH formation in Y-Z plane is shown in Fig. 5.18. As load increased, PH progressed from bottom floor beams to top floor beams. In this plane, beam length was shorter than that of beams in X-Z plane. So, PHs were formed at both the end section of all the beams. First PHs were formed at far end of all beams and then started to form at near end of beams.

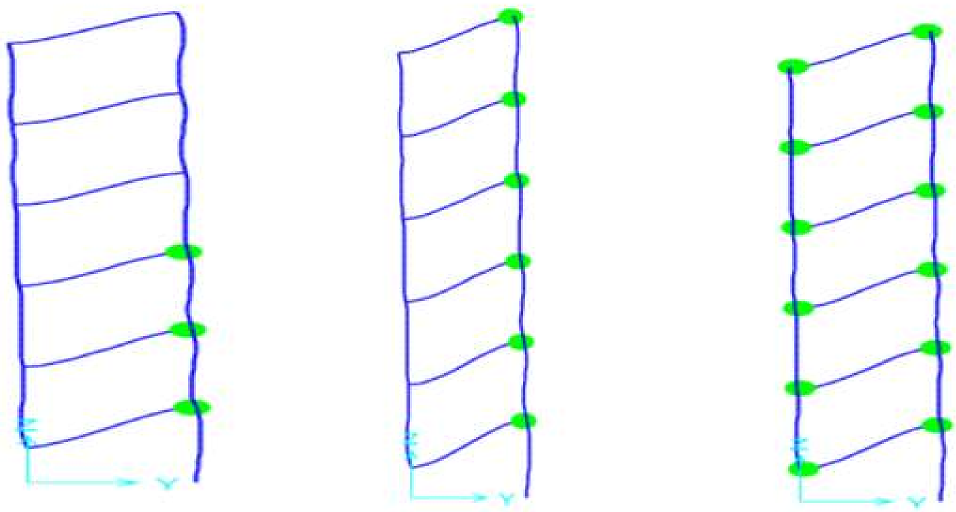

Fig. 5.18 PH formation in Y-Z plane beams for Removal of Corner Column

5.3.4 Case 4: Removal of Interior Column

PH final state after removing the interior column is shown in Fig. 5.19 . PHs were formed only in beams present in Y-Z plane.

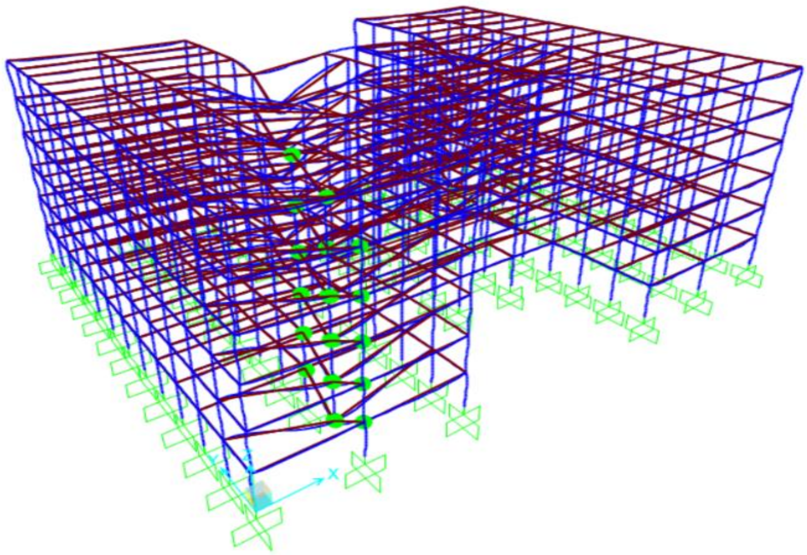

Fig. 5.19 Final state of PH formation for Removal of Interior Column

No PH were formed in X-Z plane. Length of beams in X-Z plane are more than that of beams in Y-Z plane of the building. It can be assumed that, major part of extra load of the column is transferred to beams in Y-Z plane which increased moment demand thereby forming the PH in them.

PH formation in Y-Z plane is shown in Fig. 5.20. As loading was increased the PH started from bottom beams and progressed towards top beams. Once PH were formed at far end of all beams on either side, PH starts to form at near end of beams.

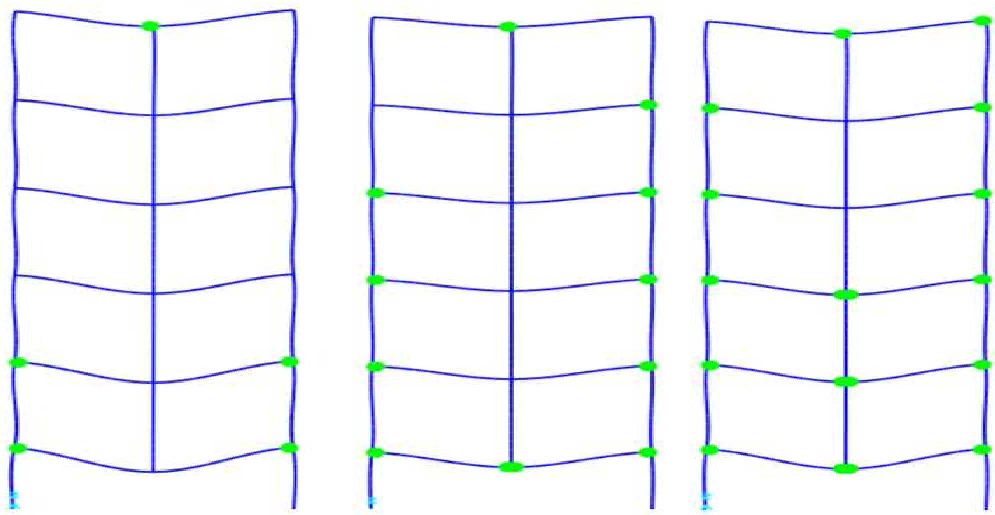

Fig. 5.20 PH formation in Y-Z plane beams for Removal of Interior Column

5.4 Closure

After removing particular column from its position, LSA and NLSA were performed and results obtained for each case has been discussed above. Common observations from each case for DCR and PH formation are commented in next chapter.

CONCLUSION

Past events of progressive collapse around the world lead to the formation of codes and guidelines to resist the structure and reduce the damage caused by the phenomenon. One of them is General Service Administration (GSA) 2003 guidelines. GSA provides the guidelines to reduce the potential for progressive collapse in new and existing buildings. As per these guidelines, four column removal cases were decided to study. By modelling and analyzing the structure considered by Joshi et al. in their study, validation was done in SAP2000 software.

In current study, GST Bhavan (Pune) building is considered. The building is modelled and analyzed in SAP2000 software. For this study, Demand-Capacity Ratio (DCR) from Linear Static Analysis (LSA) and Plastic Hinge formation from Non-Linear Static Analysis were determined for all four column removal cases. The results obtained from both the analysis were discussed in above chapters and the conclusions derived from them are discussed below.

A. Conclusions regarding DCR are as follows

1. DCR of beam sections exceeding the permissible flexure limit of 2 as per GSA 2003 guidelines are more in Y-Z plane than in X-Z plane. Number of beams exceeding the limit in Y-Z plane for case 1, case 2 and case 3 are 6 beams, 4 beams and 6 beams respectively. Almost all beams in Y-Z plane had crossed the limit which concludes that they have high possibility of being failed on removal of column.

2. While in interior column removal case, no DCR is greater than flexure limit of 2. But still DCR values of beam sections in Y-Z plane is more than that of beam sections in X-Z plane.

3. From above two statements it can be inferred that, moment-demand increases more in shorter length beam than in longer length beam on removal of column in any case.

4. 1st objective of the project has been checked. It is found that, the building designed as per IS 456-2000 is vulnerable to progressive collapse in every column removal case, as beam sections have high possibility of been failed on removal of vertical support which can further lead to failure of more structural members.

5. Also, here research gap was addressed on different spans. DCR in smaller spans is more than larger spans.

B. Common observations regarding PH formation are as follows

1. Plastic Hinge (PH) starts forming at location having higher DCR in all four cases studied.

2. It was observed in all 4 cases that, PHs were formed only in orthogonal planes to that of column removal location. This indicated that the column load is re-distributed among these adjacent beams which have high possibility that their elastic state has been crossed.

3. In all cases, PHs first formed at far end of beam and then at near end of beam.

4. On removal of column, PHs starts forming from lower floor beam and progresses towards upper floor beams. This indicated that load is majorly attracted by lower floor beams.

From above study it can be commented that, removal of middle column on longer side is critical in current study due to presence of structural members attached to it. More number of shorter spans beams are connected to this column bay, which resulted in forming plastic hinges in every beam in shorter span, ultimately exceeding their flexure limit of 2 as per GSA guidelines (which infers that they have very high probability of being failed on removal of column). The displacement of column removal joint was also relative more than all other cases.

So the overall results obtained and conclusions derived from them are found satisfactory and satisfy the preliminary objectives of the study.

FUTURE SCOPE

The base for progressive collapse has been made in current project by explaining the procedure to perform them step-by-step in software, illustration of some calculations, validating the past study, etc. Future scope of the project involves the more research that can be made in continuation with this as an extension of this study. Currently the RC framed structure was analyzed only statically as per GSA 2003 guidelines. Future scope of the project can involve:

1. The RC framed structure can be analyzed by 'Dynamic Method of Analysis', which can be more realistic and more accurate.

2. In current study, Plastic hinge formation was restricted to beams only. So in future Plastic hinges in column can be studied.

3. Advance software like 'Perform 3D' can be used to perform the Dynamic Analysis for better results.

4. Currently study was bound only upto analysis, further the RC framed structure can be designed to resist the progressive collapse.

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