Linear and Nonlinear Static Analysis for Assessment of Progressive Collapse Potential of Multistoried Building

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ABSTRACT

Explosions almost instantaneously damage the structures. The direct action of the high intensity blast on the exposed surfaces of the building may causes damage to the primary structural components like columns and structural walls. Damage can be in form of loss of non-structural element, damage to structural components, and collapse of structural element leading to progressive failure of part or whole building. The failure of a member in the primary load resisting system leads to redistribution of forces to the adjoining members and if redistributed load exceeds member capacity it fails. This process continues in the structure and eventually the building collapses. This phenomenon is referred as progressive collapse of the structure. When a multi storey building is subjected to sudden column failure, the resulting structural response is dynamic, typically characterized by significant geometric and material nonlinearity. Analysis methods used to evaluate the potential of progressive collapse varies widely; ranging from the simple two dimensional linear elastic static procedures to complex three dimensional nonlinear dynamic analyses.

In the present study the demand capacity ratios of reinforced concrete four storey and ten storey frame structure are evaluated as per GSA guidelines. The linear static and nonlinear static analyses are carried out using software SAP2000. For progressive collapse analysis, a nonlinear static analysis method employs a stepwise increment of amplified vertical loads which can be referred as vertical pushover analysis. The demand capacity ratios found using linear static analysis at critical locations are compared with the hinge formation obtained from nonlinear static analysis. Comparison of linear static and nonlinear static analysis reveals that hinge formation starts from the location having maximum demand capacity ratio calculated from static analysis.

INTRODUCTION

The direct action of the high-intensity air-blast on the exposed surfaces of the building causes damage to individual non-structural element like exterior infill walls, windows etc, and structural components of the building like slab, girders, columns and load-bearing or structural walls. Local damage is the primary damage mechanism under blast loading. Buildings are designed usually for loads that are smaller than that imposed by the blast overpressures and reflection effects. The failure of a member in the primary load resisting system leads to redistribution of force to the adjoining members, this action continues in the structure and eventually the building collapses. This event is considered as progressive collapse**.** The collapse of a single structural element or few structural elements may lead to progressive collapse of a part or the whole building. Murrah Federal Building and 23-storey Ronan Point in East London are well known examples of progressive collapse.

In the paper one four storey and one ten storey building is considered, to study the effect of single column failure on low rise as well as high rise buildings. Symmetrical building configuration is chosen for better understanding of the behavior. Evaluation of progressive collapse potential of building designed for seismic loading is carried out.

Progressive collapse analysis is performed using SAP2000. In progressive collapse analysis, guidelines of the U.S. General Services Administration (GSA) are followed. Generally, a prime location for a vehicle bomb is in a basement parking or near an exterior parking area, and structural protection against detonation in a basement parking area is rarely feasible. Hence it is necessary to study the interior consideration along with exterior considerations.

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 typical or symmetrical framed structures the following analysis cases should be considered (GSA 2003).

Exterior considerations. The following exterior analysis cases should be considered:

1. Analyze 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 Figure 1).
2. 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 Figure 1).
3. Analyze 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 Figure 1).

Interior considerations. Buildings that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case.

Analyze 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).

A separate analysis must be performed for each case. While performing a static linear analysis, the vertical load case applied to the structure is as:

$$\text{Load} = 2(DL + 0.25LL) \quad (1)$$

Where DL = Dead Load, and LL = Live Load.

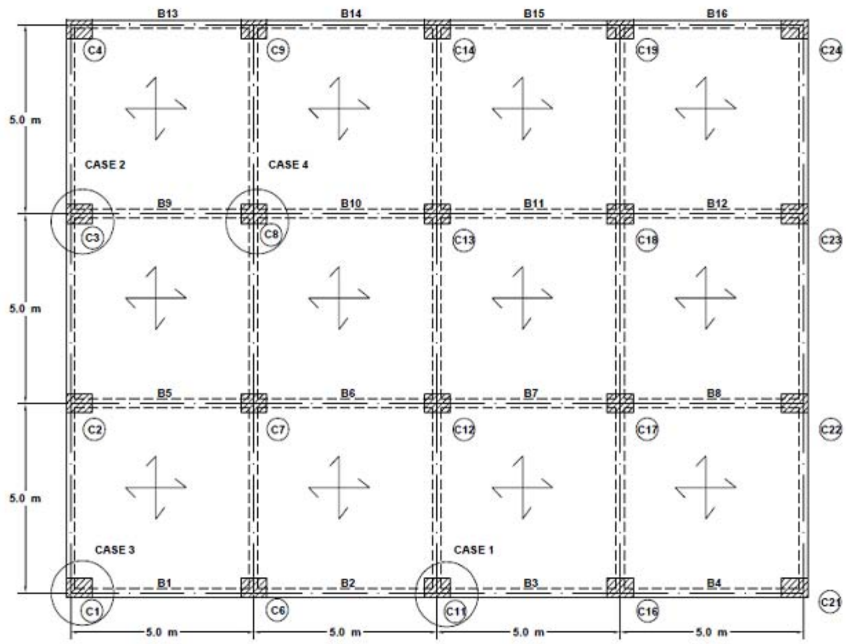


Figure 1. Plan dimension of the building.

ANALYSIS PROCEDURE AND ACCEPTANCE CRITERIA

A progressive collapse analysis is required to determine the capability of a structure to resist abnormal loadings. The proposed progressive failure analysis method is threat independent, in the sense that it is initially assumed that some type of short duration abnormal loading has caused local damage represented by the removal of one or more critical members. When a multi storey building is subjected to sudden column failure, the resulting structural response is dynamic, typically characterized by significant geometric and material nonlinearity. Analysis methods used to evaluate the possibility of progressive collapse widely varies; it is ranging from the simple two dimensional linear elastic static procedures to complex three dimensional nonlinear dynamic analyses.

Linear static analysis. In the linear static analysis column is removed from the location being considered and linear static analysis with the gravity load given by Eq.1 imposed on the structure has been carried out.

From the analysis results demand at critical locations are obtained and from the original seismically designed section the capacity of the member is determined. Check for the DCR in each structural member is carried out. If the DCR of a member exceeds the acceptance criteria in shear and flexure, the member is considered as failed. The demand capacity ratio calculated from linear static procedure helps to determine the potential for progressive collapse of building.

Nonlinear static analysis. Nonlinear Procedure implies the use of static or dynamic finite element analysis methods that takes into consideration, both material and geometric nonlinearity. Special attention should be given to facilities that contain atypical structural configurations and high rise buildings that may exhibit complex response modes for the case where a primary vertical element is instantaneously removed. Nonlinear static analysis can be used for a wide variety of purposes, including analyzing a structure for material and geometric nonlinearity, to include the P-delta or large displacement effects and to perform buckling analysis etc. Nonlinear static analysis is widely used to analyze a building for a lateral load and is known as “pushover analysis”. In this method loads is applied step by step until maximum load is attained, and structural members are allowed to undergo in to nonlinear behavior. For progressive collapse analysis, a nonlinear static analysis method implies a stepwise increase of amplified vertical loads, until maximum amplified loads are attained or until the structure collapses it can be referred as vertical pushover analysis (Marjanishvili 2004).

Nonlinear static analysis procedure is carried out in the following steps using SAP2000 (Marjanishvili 2006).
1. Build a finite-element computer model.
2. Define and assign nonlinear plastic hinge properties, to beams and columns.
3. Apply static load combination 2(DL+0.25LL).
4. Perform nonlinear static analysis.
5. Verify and validate the results based on hinge formation.

Acceptance criterion for progressive collapse. The GSA proposed the use of the Demand–Capacity Ratio (DCR), the ratio of the member force and the member strength, as a criterion to determine the failure of main structural members by the linear analysis procedure (GSA 2003).

$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

Where,
Q_{UD} = Acting force (demand) determined in member or connection (moment, axial force, shear, and possible combined forces)
Q_{CE} = Expected ultimate, un-factored capacity of the member and connection (moment, axial force, shear and possible combined forces)

For the building having a typical structural configuration, the DCR of the primary structural components should be less than 2 to avoid failure in flexure and 1 to avoid shear failure.

MODELLING OF BUILDING

The building for the study is four and ten storey symmetrical R.C. building. The structure consists of four bays of 5 m in the longitudinal direction and three bays of 5 m in the transverse direction. Typical floor-to-floor height is 3.1 m and for the first story it is 3.4 m. Wall having 115 mm thickness is considered on all the beams. Slab thickness considered is 150 mm.

Beam size is taken same for four and ten storey as 300 × 550 mm. Column size of 350 × 600 mm is considered for four storey building. Column size of 500 × 700 mm is considered for ten storey building. Loading considered on the building for the study are as follows.

Dead load
Self weight of the structural elements
Floor finish = 1.5 kN/m^2 and
Wall load on all beams is 7.13 kN/m
Live load
On roof 1.5 kN/m^2, and
On floors 3.0 kN/m^2
Seismic loading as per IS:1893
Zone V,
Soil type II
Importance factor 1

The characteristic compressive strength of concrete (f_{ck}) is 25 N/mm^2 and yield strength of reinforcing steel (f_y) is 415 N/mm^2. Analysis and design of building for the loading is performed in the SAP2000.

One four storey and ten storey building is designed for seismic loading in SAP2000 according to the IS 456:2000. Based on the reinforcement Demand capacity ratio is calculated at L, C and R locations (see Figure 2).

LINEAR STATIC PROGRESSIVE COLLAPSE ANALYSIS

To evaluate the potential for progressive collapse of a four storey symmetrical reinforced concrete building using the linear static analysis four column removal conditions is considered (see Figure 1). First building is designed in SAP2000 for the IS 1893 load combinations. Then separate linear static analysis is performed for each case of column removal. Deflection at the point above the removed column has been observed for the GSA loading for all the four cases of column loss. Member forces at critical locations are considered for the load combination given in Equation.1. Demand capacity ratio for flexure at all storey is calculated for all four cases of column failure.

The removal of a column at the middle of the long side of the building Case 1 doubles the beam span from 5 m to 10 m. The new 10 m beams must be capable of providing an alternate load path into the adjacent columns. A positive moment is now developed just over the removed column. Bending moment diagram of the four storey building after column failure for linear static analysis is presented (see Figure 2).

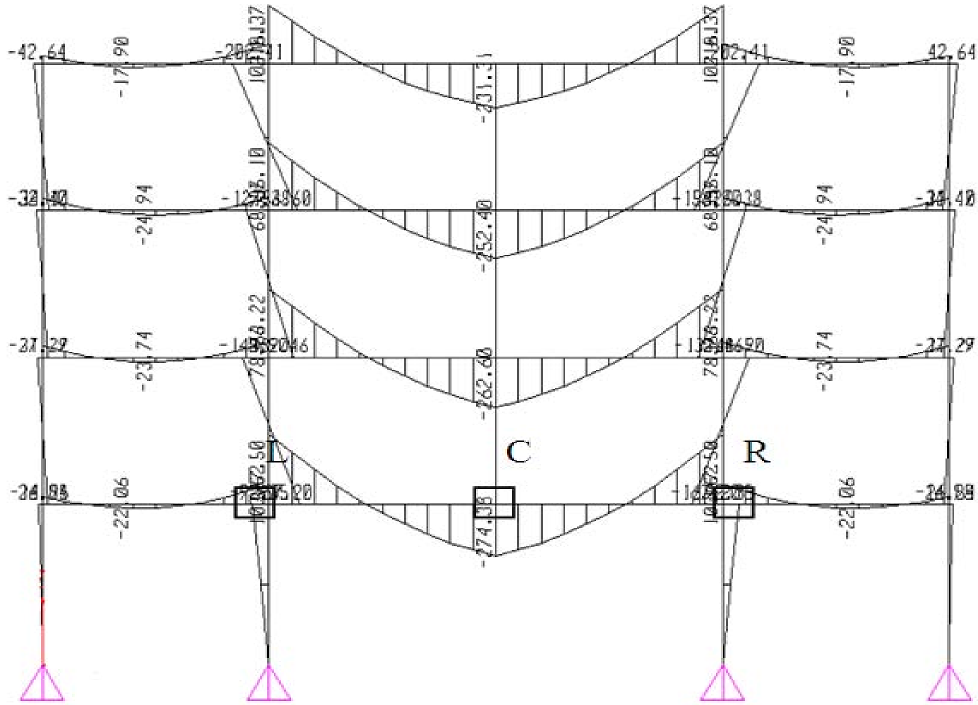


Figure 2. Bending moment diagram for case 1.

Calculation of demand capacity ratio. Capacity of the member at any section is calculated as per IS 456:2000 at critical sections using increased material strength (see Table 1).

Table 1. Strength-increase factors for reinforced concrete.

Construction Material	Strength Increase Factor
Concrete Compressive Strength	1.25
Reinforcing Steel(tensile and yield strength)	1.25

Demand capacity ratio for flexure. Moment capacity of section above the removed column can be found with reference to IS 456:2000 for Case 1 is illustrated below.

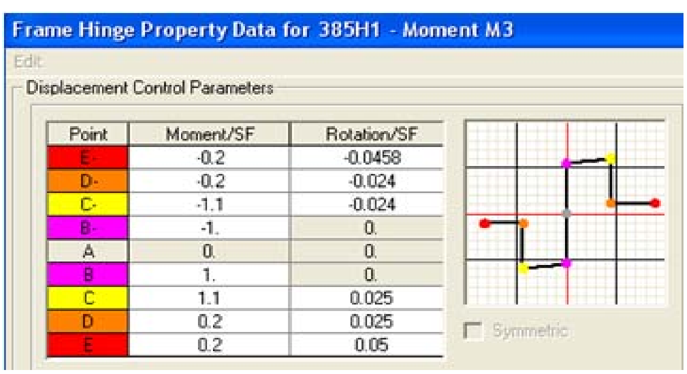
Ast = 1345 mm^2
fck = 1.25 × 25 = 31.25 N/mm^2 (1.25 is the strength increase factor)
fy = 1.25 × 415 = 518.75 N/mm^2
Hence for b = 300 mm and d = 520 mm,
Moment of resistance point above column removed,
Mu = 270 kN m.

Demand capacity ratio after removal of column is found out considering demand as the member force for the load combination mentioned in Equation 1 after removal of column. Capacity is calculated as mentioned above for the designed reinforcement at that section. The demand capacity ratio for flexure calculated at all the four storey level for four cases of column failure is shown (see Figure 4) where Case 4 is shown for interior column failure consideration. The demand capacity ratio for flexure calculated for ten storey building designed for seismic loading for all four cases of column failure (see Figure 5).

NONLINEAR STATIC PROGRESSIVE COLLAPSE ANALYSIS

For nonlinear analysis automatic hinge properties and user-defined hinge properties can be assigned to frame elements. When automatic or user-defined hinge properties are assigned to a frame element, the program automatically creates a generated hinge property for each and every hinge. There are five default hinge options are available, Axial (P), Torsion (T), Moment (M2 or M3), Shear (V2 or V3), and Coupled (P-M2-M3). The hinge properties are calculated by the program for the cross section and reinforcement details provided. For default moment hinges, SAP2000 uses Tables 6-7 and 6-8 of FEMA 356. A graphical representation of the moment hinge property is shown (see Figure 3).

The behavior and response of a building has been observed in SAP2000 for moment-rotation relationship is shown (see Figure 3). There is no plastic deformation occurs until point B, where the hinge yields. This is followed by a point C, which represents the ultimate capacity of the hinge. After point C, the force capacity of hinge immediately drops to point D which corresponds to the residual strength of the hinge. Point E represents the ultimate displacement capacity of the hinge after which total failure of the hinge is reached. Hinge property is defined to the beam members by selecting them and from the assign menu auto M3 hinge property from FEMA 356 has been assigned. Preliminary studies indicated that collapse of the R.C. building under column removed conditions is governed by the flexural failure mode of beam elements.



Point	Moment/SF	Rotation/SF
E-	-0.2	-0.0458
D-	-0.2	-0.024
C-	-1.1	-0.024
B-	-1.	0.
A	0.	0.
B	1.	0.
C	1.1	0.025
D	0.2	0.025
E	0.2	0.05

Figure 3. Moment (M3) hinge property.

RESULTS AND DISCUSSION

Four storey and ten storey reinforced concrete symmetrical building (see Figure 1) is studied for linear static and nonlinear static analysis to assess the potential for progressive collapse. Demand capacity ratio for flexure are shown (see Figure 4 and 5), which indicates that DCR for flexure exceeds permissible value specified by GSA guidelines only at some of the upper storey for seismically designed building.

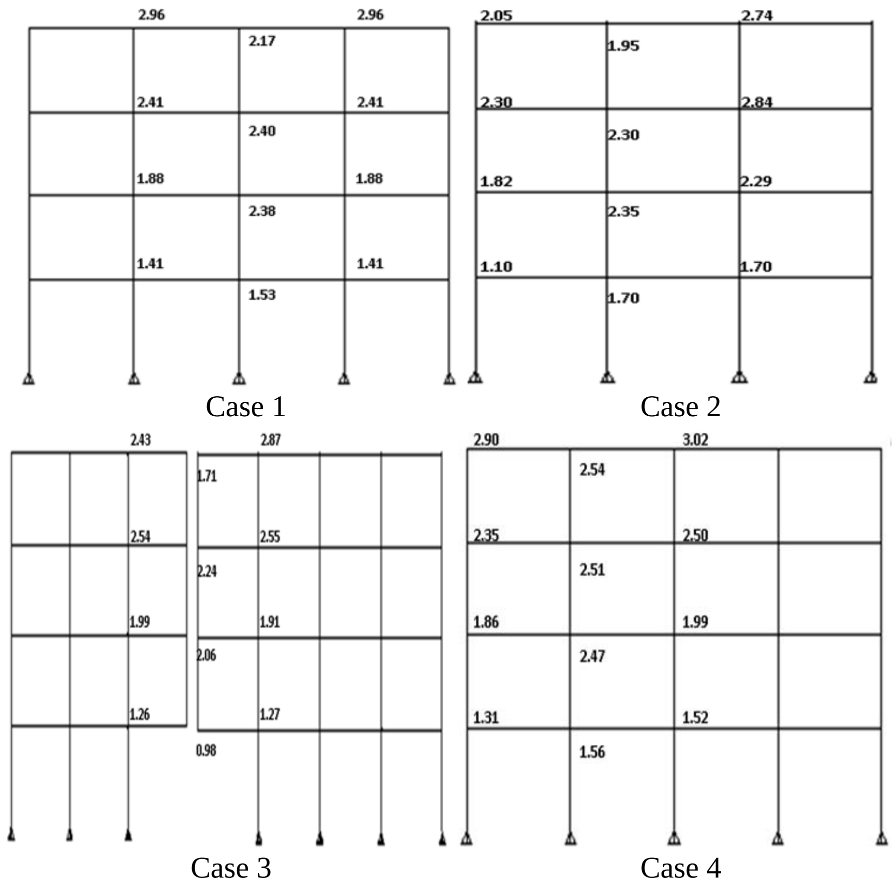


Figure 4. Demand Capacity Ratio for four storey building.

To consider material and geometrical nonlinearity model of structure is prepared, the loads are magnified by a dynamic increase factor that accounts for dynamic effects and the resulting load is applied to the model with the removed vertical load-bearing element.

Results obtained from linear static and nonlinear static analysis are compared for four storey as well as for ten storey building. Four column loss scenarios which include three external column loss scenario and one internal column loss have been considered. A separate analysis is performed for each case of column failure. After analysis has been performed the hinge formation pattern for various displacement levels are observed for all the four cases of column removal in the building designed for seismic loading. Hinge formation is compared with DCR obtained from linear static analysis. Steps of the hinges formation at some of the displacement levels for seismically designed building are shown (see Figure 6 and 7).

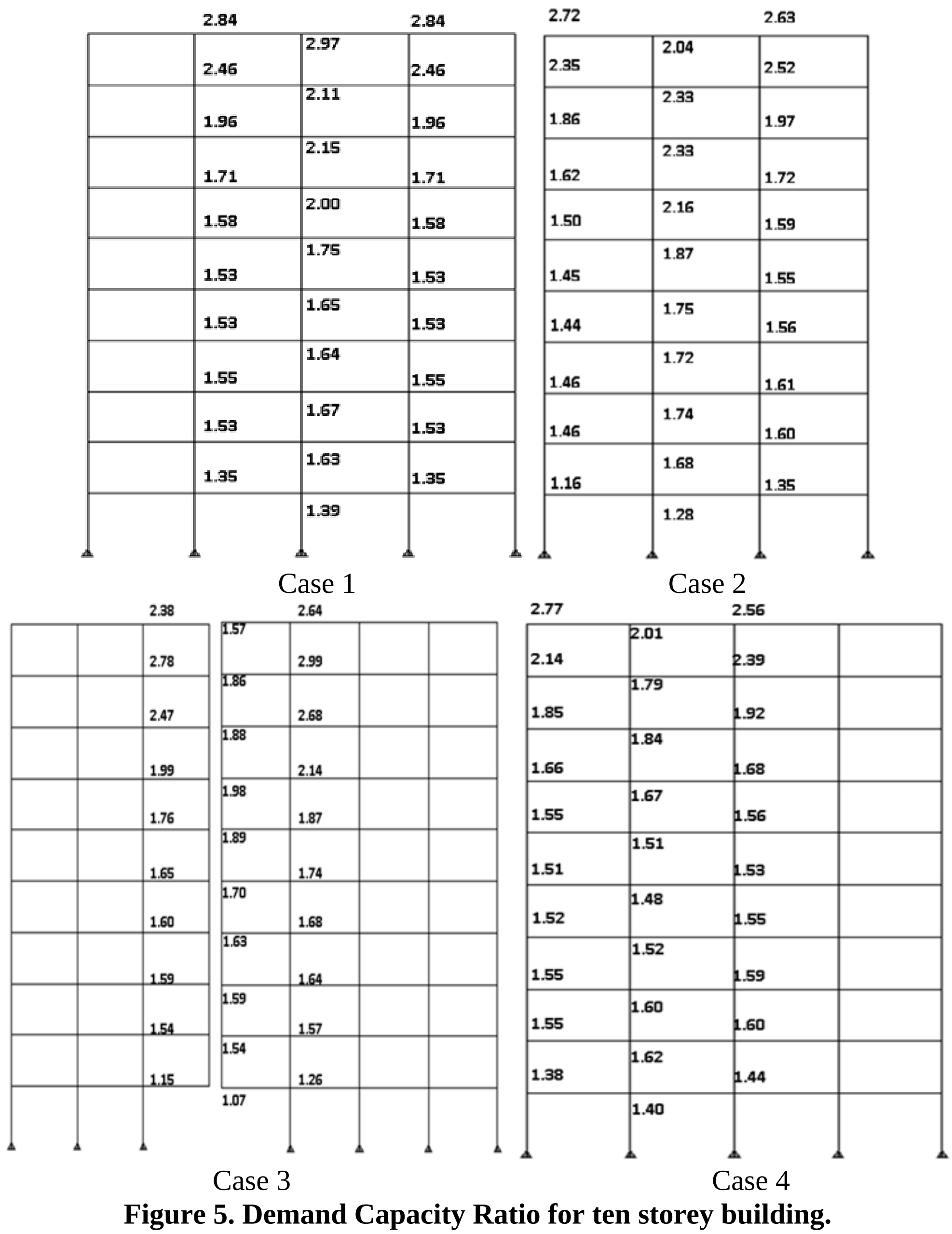
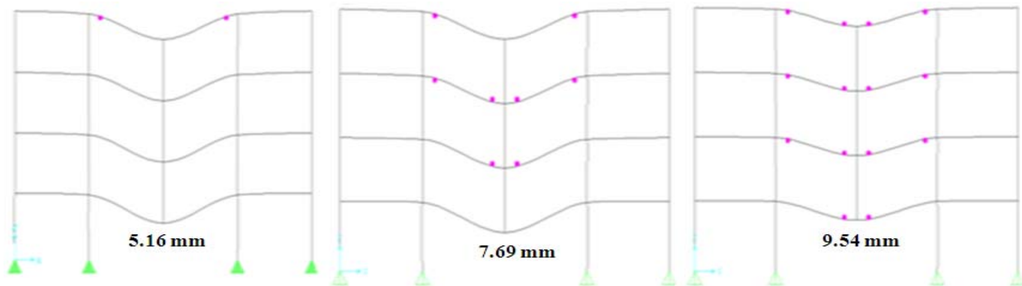
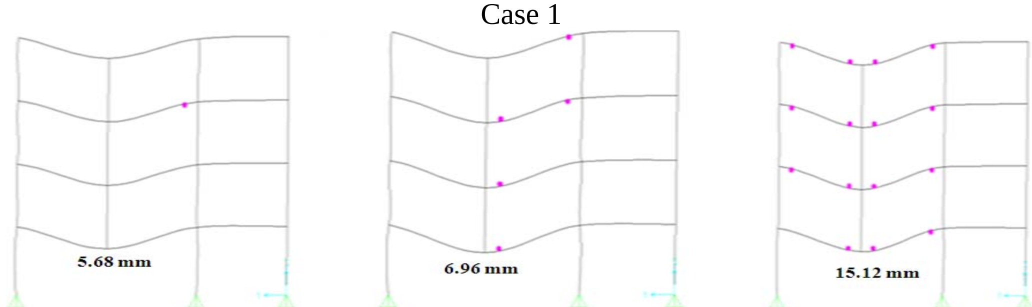


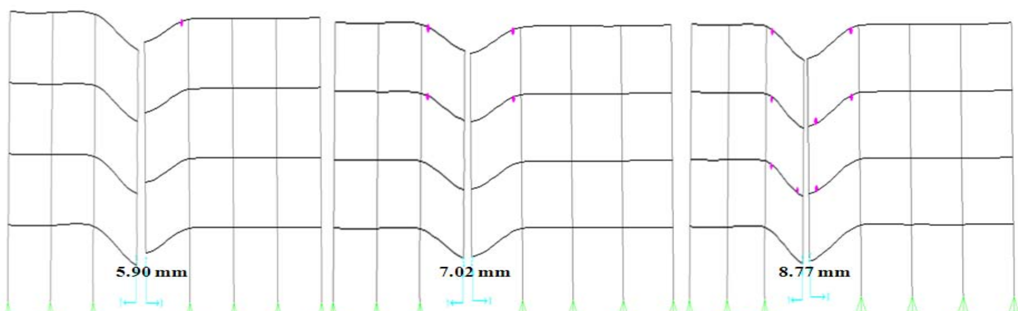
Figure 5. Demand Capacity Ratio for ten storey building.



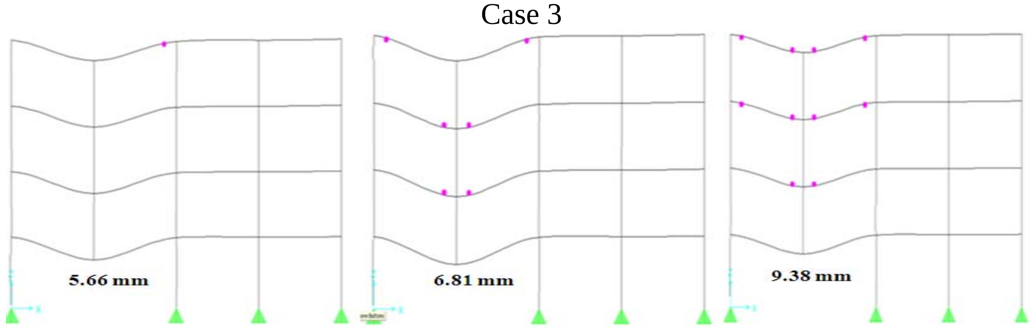
Case 1



Case 2

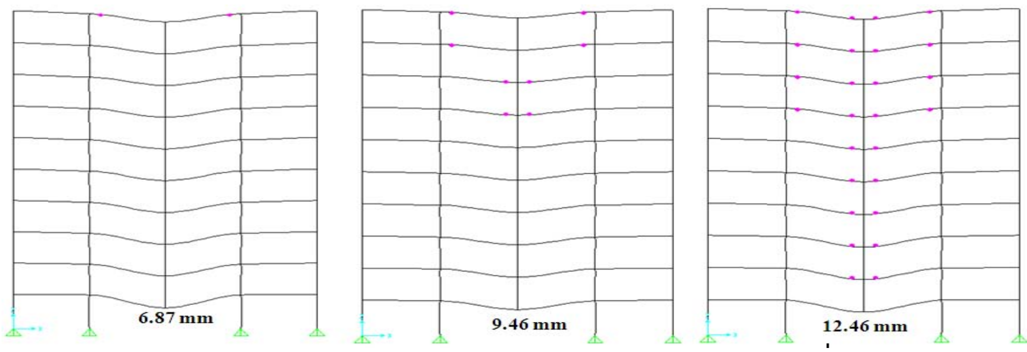


Case 3

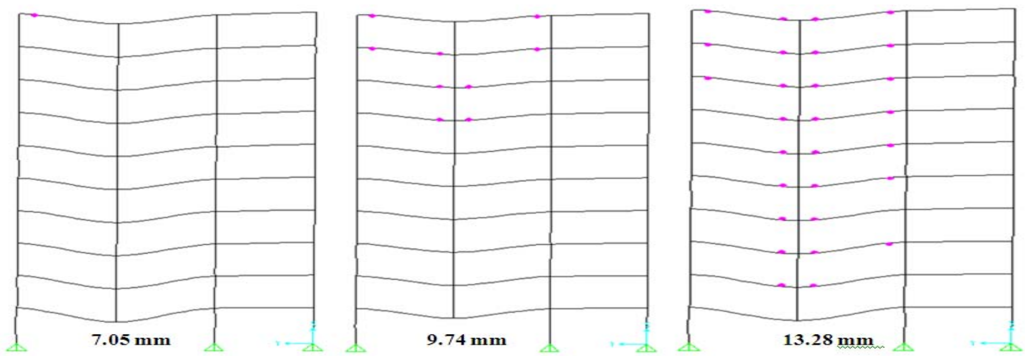


Case 4

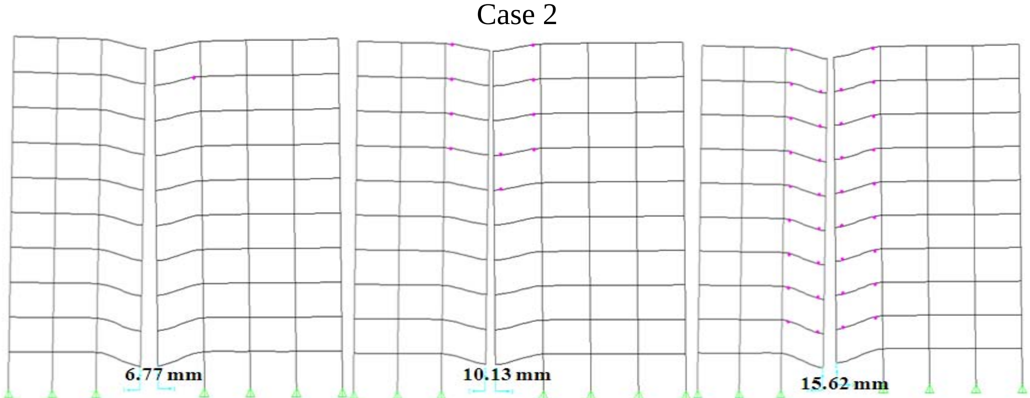
Figure 6. Steps of hinge formation in four storey seismically designed building.



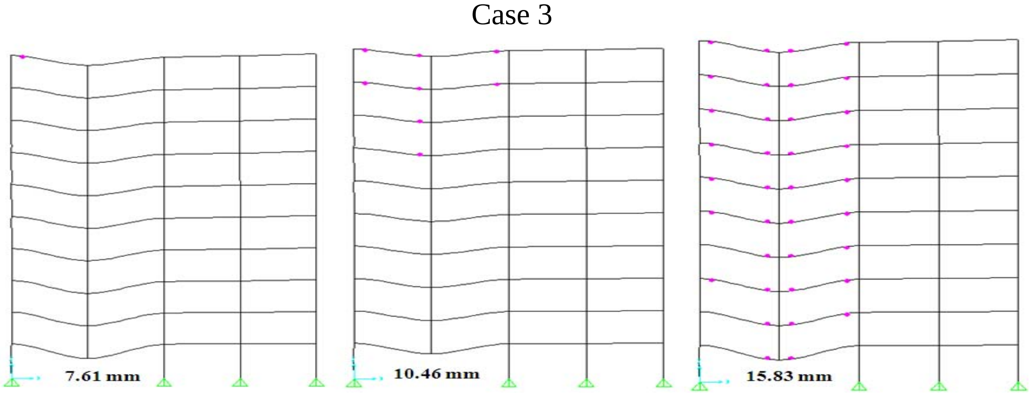
Case 1



Case 2



Case 3



Case 4

Figure 7. Steps of hinge formation in ten storey seismically designed building.

Displacement in mm at the point above the column failure is also shown (see Figure 6 and 7) for the hinge formation. It can be clearly observed that first hinge forms at the location where demand capacity ratio is maximum. Further in next step sections having higher values of demand capacity ratio shows hinge formation.

CONCLUDING REMARKS

When a single base column fails, only those two orthogonal planar frames to which the failed column belongs share the released forces.

Nonlinear static analysis reveals that hinge formation starts from the location having maximum demand capacity ratio. Then formation of hinge continues through the locations having higher DCR in various displacement levels. Hence locations where the demand capacity ratio exceeds the permissible values in linear static analysis, there is a high possibility that the member components exceeded its elastic limits during column failure scenario.

From this study it is observed that to avoid the progressive failure of beams and columns, after failure of particular column due to extreme loading from blast, adequate reinforcement to limit the DCR within the acceptance criteria and adequate detailing can be useful. In general, structures designed and detailed with an adequate level of continuity, redundancy, and ductility can develop alternative load paths following the loss of an individual member and prevent progressive collapse.

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