Progressive Collapse Analysis of Performance-Based Designed RC Building

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Abstract. Progressive collapse begins with a local failure in any one or more load-bearing structural members that spreads from one element to another element. The event is due to a change in the original load-sharing path of the structural member. The whole process leads ultimately to the collapse of the complete structure or a considerable portion. Any abnormal loading or events like vehicle collision, bomb blast, gas explosion, or earthquakes can trigger this situation. Most structural systems are reinforced concrete, which behaves on accidental action according to its inherent strength and stiffness characteristics. Redundancy plays a significant role in saving it before the collapse. Engineers analyse and suggest member sizes of the buildings through documented guidelines of the code applicable in that country. After construction, the building performance does not remain the same and deteriorates after prolonged use. In such a case, it is impossible to regulate the buildings' performance with age. The building may change to a lower performance level and be more vulnerable to collapse. Hence, the structure's behaviour is observed differently according to its robustness and redundancy. In the present study, a sample building was designed considering IO, LS and CP performance levels and examined by Push Over Analysis. Then We conducted collapse analysis using SAP 2000 on three different models in an exterior and interior column loss situation. The collapsed state has been critically examined. The IO stage building designed in Zone V of the Indian subcontinent has performed well and shows inherent collapse potential.

Keywords. Progressive Collapse**,** Performance Level**,** Drift, Life Safety,

1. Introduction

Progressive collapse begins with a local failure in any one or more load-bearing structural members that spreads from one element to another element. The event is due to a change in the original load-sharing path of the structural member. The whole process leads ultimately to the collapse of the complete structure or a considerable portion. Any abnormal loading or events like vehicle collision, bomb blast, gas explosion, or earthquakes can trigger this situation. It is a dynamic behaviour of civil engineering structure influenced by material and geometrical nonlinearities. Most structural systems are reinforced concrete, which behaves on accidental action according to its inherent strength and stiffness characteristics. Redundancy plays a significant role in saving it before the collapse. Engineers analyse and suggest member sizes of the buildings through documented guidelines of the code applicable in that country. After construction, the building performance does not remain the same and deteriorates after prolonged use. In such a case, it is impossible to regulate the buildings' performance with age. The building may change to a lower performance level and be more vulnerable to collapse. Hence, the structure's behaviour is observed differently according to its robustness and redundancy. In the present study, a sample building was designed considering IO, LS and CP performance levels and examined by Push Over Analysis. Then We conducted collapse analysis using SAP 2000 on three different models in an exterior and interior column loss situation. The collapsed state has been critically examined. The IO stage building designed in Zone V of the Indian subcontinent has performed well and shows inherent collapse potential.

2. Performance-Based Design of RC Structure

The ASCE 43-14 and FEMA 356 categorised performance level of building under three category:Immediate occupancy level (IO), Life Safety (LS)& Collapse prevention (CP). These levels are equilibrium state of structure to describe the building's expected performance. It is measured in terms of damage/ plastic joint rotation, storey drift or visible damage on structures. It is described as the limiting damage state of the structural systems. Performance objectives consist of specifications of the design event in which the building is designed to resist a permissible level of damage. Table 1 describes the approximate limiting levels of structural and non-structural damage that might be expected of buildings evaluated under seismic performance as per ASCE 41-13 (2014).

Table 1. Building Performance Levels

Performance Level	Damage Description	Downtime
Immediate Occupancy	Negligible structural damage, essential system operational, minor overall damage.	Immediate
Life Safety	Possible structural damages, no collapse, minimal falling hazards, adequate emergencies ingress.	Probable Total Loss
Collapse Prevention	Severe structural damage, minor residual stiffness and strength, incipient collapse; possible falling hazards, possible restricted access.	Probable Total Loss

Figure 2 below illustrates the qualitative performance levels of ASCE 43-13 (2014) superimposed on a global force-displacement relationship for a sample building.

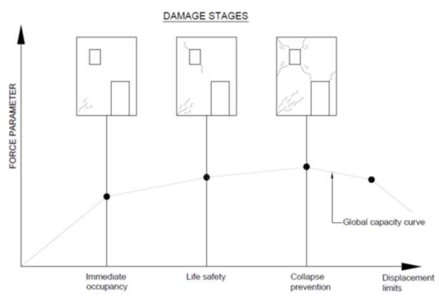


Figure 1. Performance levels as per ASCE 43-13 (2014)

Performance point (PP) is a discrete point on Base Shear ~nonlinear displacement diagram. It Indicates the performance state for which the building is designed. To determine the performance of the building, we need to examine the performance state under any unforeseen action expected during design life of structure. In this paper seismic force has considered to examine performance of building and is site specific for Zone V as laid down in IS 1893: 2016.

If $\Delta_{pp} < \Delta_{IO}$, it implies IO Performance level building.

$\Delta_{pp} > \Delta_{IO}$ & $< \Delta_{LS}$, LS Performance level building.

$\Delta_{pp} > \Delta_{LS}$ & $< \Delta_{CP}$, CP Performance level building.

$\boldsymbol{\Delta i}$ = amount of sway on the floor,

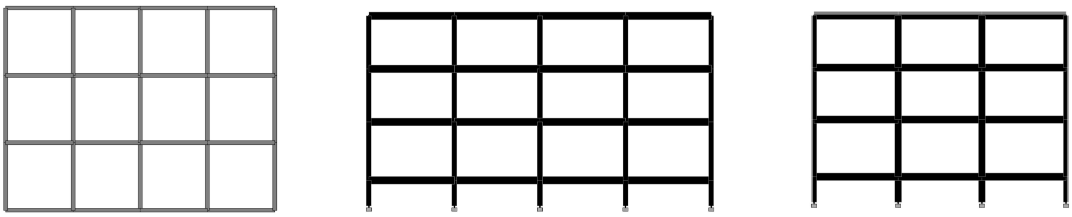
$\boldsymbol{\Delta_{i+1} - \Delta_i}$ = storey drift

$(\Delta_{i+1} - \Delta_i)/h$= Storey drift ratio (Generally expressed in percentage)

3. Selection of Building Models Meeting Target Performance:

3.1. Geometrical Configuration

A three storey Reinforced Concrete building having plan dimensions 20 m x 15 m. shown in Figure 2, located in seismic zone V of Indian subcontinent on soft soil, is considered. It is proposed to design a building meeting target performance. The sectional elevations of RC frames are shown in Figures 2 (a),(b), and (c). The sizes of the beams and columns are given in Tables 2,3, and 4. The relevant data are as follows: Grade of concrete: M30, Grade of steel: Fe 500, Live load on the roof 1.5 kN/m2, Live load on floors 3 kN/m2, Roof finish 1.5 kN/m2, Brick wall on beams 230 mm thick. Density of concrete is 25 kN/m3, Density of brick wall including plaster 20 kN/m3. The slab thickness considered was 150mm thick.



(a) **(b)** **(c)**

Figure 2. Building **p**lan, front elevation and side elevation

3.2. Stress and Strain Diagram of Material.

Nonlinear Material properties has considered for concrete M30 Grade concrete shown in Figure 3 and Steel fy 500 Grade Shown in figure 4. The stress strain curve for nonlinear concrete material and nonlinear steel material are in SAP 2000 to develop appropriate interaction state for selected cross section of frame.

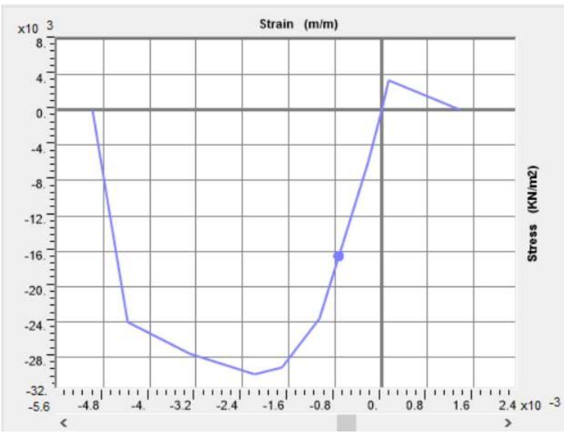


Figure 3. Stress-Strain Curve of M30 **Figure 4**. Stress-Strain Curve of Fy500

3.3. Selection and Verification of CP Stage Building

Collapse Prevention means the building is in a partial or total collapse state. It experiences severe damage, degrading the stiffness and strength of columns, sizeable storey drift and degradation in vertical-load-carrying capacity. The gravity load resisting system may continue to carry its gravity load demands, but the risk of falling debris may exist. The structure may not be practical to repair and is unsafe for re-occupancy, as aftershock activity could induce collapse. The CP stage building has lesser reinforcement and lesser size. It experiences seismic loading of severe earthquake load and resists it up to the desired performance level before the collapse. The linear analysis was performed, and section and reinforcement were provided tentatively to storey drift H/50 instead of H/250 permissible.

Reinforcement of the column and beam was kept up to 75%. The design base shear calculated is 2118.69 KN for seismic Zone V, and the response reduction considered is 5. The trial section and reinforcement details are shown in Table 2. The performance analysis is carried out using SAP 2000, introducing the hinge parameter

Table 2. Member Sizes and reinforcement details for CP stage Building Model (CP-RCBM)

	Column Details			Beam Details	
	Size	Reinforcement		Size	Reinforcement
C1	300X300	3-20# on each face Stirrup 8# 100 c/cThree-legged in each direction	PB	250X450	4-20# Top 4-20# Bottom Two-legged Stirrup 8# 120 c/c (M3= -194 KNm)
C2	300X375	3-20# on each face Stirrup 8# 100 c/c Three-legged in each direction	FB	250X450	4-20# Top 4-20# Bottom Two-legged Stirrup 8# 120 c/c (M3= -321 KNm)
C3	325X400	5-16# on each face Stirrup 8# 100 c/c five-legged in each direction	RB	250X450	4-20# Top 4-20# Bottom Two-legged Stirrup 8# 120 (M3= -209 KNm)

at both ends of columns and beams.P-M2-M3 hinge parameters are used in columns, and P-M3 parameters are used in beams. The pushover curve is shown in Figure 5. The target displacement to resist the base shear of 2118.69 KN is 154 mm between the LS and CP stages. This building model is CP performance-level building.

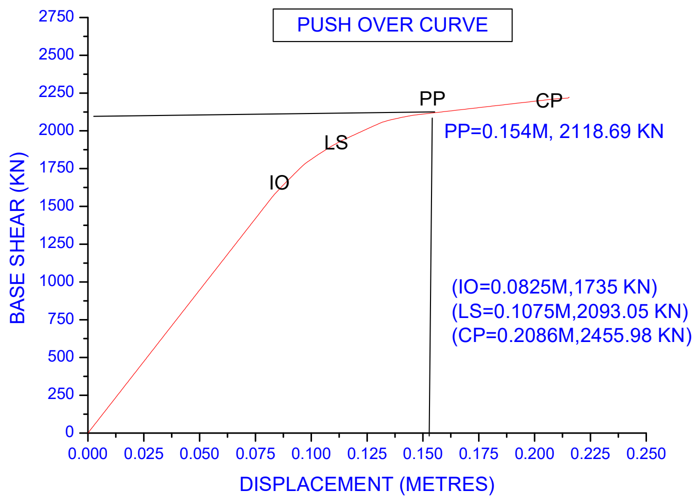


Figure 5. Push-Over Curve (CP Building Model)

3.4. Selection and Verification of LS Stage Building

Life Safety is a state of a building in which considerable damage has occurred, but some margins are available against building collapse. Some beams /columns are severely damaged, but there is no scene of falling debris inside or outside the building. Limited casualty may occur during the accidental actions; however, it is expected that the overall risk of life due to structural damage is low. Repairing the damaged state can be possible; however, this may not be practical for economic reasons. The selection of a building model to meet this performance level is made by Linear analysis. Frame sections and reinforcement are provided tentatively to storey drift H/190 instead of H/250 permissible. Reinforcement of the column and beam was kept up to 85%. The design base shear is nearly the same as before and is 2268 KN for seismic Zone V and response reduction 5. The Top storey deflection is 143 mm. The trial section and reinforcement details are shown in Table 3.

Table 3. Member Sizes and reinforcement for LS stage Building Model (LS-RCBM)

Column			Beam		
		Reinforcement			Reinforcement
C1	400X400	**4-16# on each face Stirrup 8# 100 c/c Four-legged in each direction**	PB	300X500	**4-20# Top4-20# Bottom 2 legged Stirrup 8# 120 c/c(M3= -193.22 KNm)**
C2	400X450	**5-16# on each face Stirrup 8# 100 c/c Five-legged in each direction**	FB	300X500	**4-20# Top 4-20# Bottom 2 legged Stirrup 8# 120 c/c(M3= -322.93 KNm)**
C3	400X500	**6-16# on each face, Stirrup 8# 100 c/c Six-legged in each direction**	RB	300X500	**4-20# Top4-20# Bottom 2 legged Stirrup 8# 120 c/c(M3= -214.76 KNm)**

The performance analysis is carried out in the form of pushover analysis. The pushover curve shows in figure 6. Structure performance has a ductility of 2.7 up to the CP stage. The base Shear resisted by structure is 2268 KN up to storey drift considered in the design. The structure is capable of resisting more base shear.

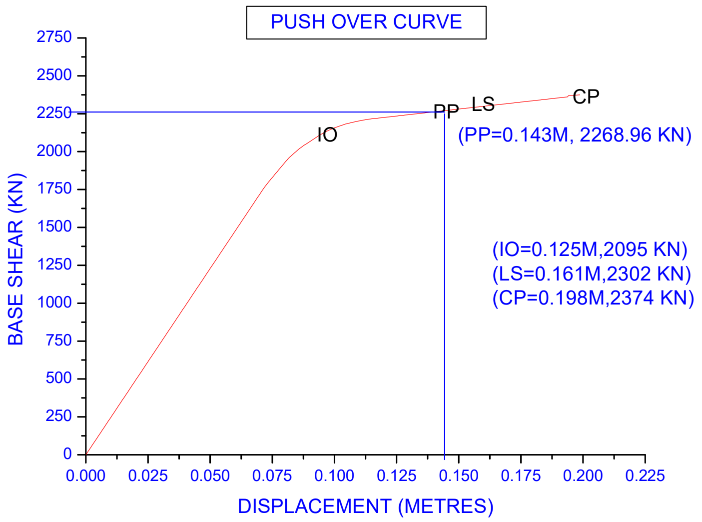


Figure 6. Push-Over Curve(LS Building Model)

3.5. Selection and Verification of IO Stage Building Model (IO-RCBM)

The immediate occupancy Performance level is a damaged state of a building where only minimal structural damage has occurred. The building's vertical and lateral force-resisting systems retain nearly all pre-earthquake strength and stiffness. The risk of threatening injury due to structural damage is very low. Linear analysis and RCC design were performed, and Sizes of section and reinforcement in structural members were provided tentatively, maintaining storey drift H/250. Table 4 shows details. The design base shear calculated was 2441 KN for seismic Zone V, and the response reduction considered was 5. The Top storey deflection was 45mm. The trial section and reinforcement details are shown in Table 4. The performance analysis is carried out in the form of pushover analysis. The pushover curve shows in figure 7. The base Shear resisted by structure is 2441 KN up to storey drift considered in the design. The structure can resist more base shear up to the IO stage.

Table 4. Member Sizes and reinforcement for IO stage Building Model (IO-RCBM)

Column			Beam		
		Reinforcement			Reinforcement
C1	450X450	**3-20# on each face Stirrup 8# 100 c/c Three-legged in each direction**	PB	350X500	**4-20# Top 4-20# Bottom 2 legged Stirrup 8# 120 c/c(M3= -185.26 KNm)**
C2	450X500	**3-20# on each face Stirrup 8# 100 c/cThree-legged in each direction**	FB	350X500	**4-20# Top 4-20# Bottom 2 legged Stirrup 8# 120 c/c(M3= -318.77 KNm)**
C3	450X600	**5-16# on each face Stirrup 8# 100 c/c Five-legged in each direction**	RB	350X500	**4-20# Top 4-20# Bottom 2 legged Stirrup 8# 120 c/c(M3= -225.75 KNm)**

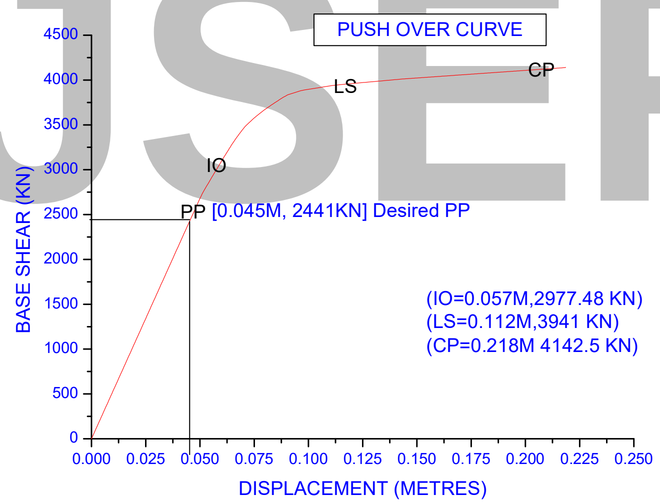


Figure 7. Push Over Curve(IO Building Model)

Table 5. Schedule of Target Performance and Achieved Performance

Building Name	Target Performance objectives		Achieved performance	
	Performance Level	Drift	Performance Level	Drift
CP-RCBM	CP	1.5%	CP	1.57%
LS-RCBM	LS	1.0%	LO	0.85%
IO-RCBM	IO	0.4%	IO	0.38%

4. Methodology of Collapse Analysis

4.1 Alternative Path Method For Collapse Analysis

The Alternative Path approach described in detail by GSA 2016 and DOD guidelines of 2009. The structure is examined under exterior and interior column removal scenario of ground floor. This method is used as tool in a missing column scenario to assess the potential for progressive collapse. It is used to check whether a building can successfully withstand the loss of a critical member. The method can be used to design new buildings or check adequacy of existing structures against progressive collapse resistance. The major advantage of APM is that it supports structural systems with ductility, continuity, and energy-consuming properties that are suitable in preventing progressive collapse. This method is consistent with the seismic design approach promoting stable structures well tied together. It is a hazard-independent methodology. It does not consider the triggering event type or the reasons for damage conditions but concentrates on the structure's response after removing critical structural members.

4.2. Dynamic Modelling of Column Removal

For dynamic column removal, the load 1.2DL+0.5LL was uniformly applied as vertical gravity load on the entire span of frames (GSA, 2016). Using the alternate path method, time-history of vertical displacement of column removal point is calculated. For this purpose, the reaction forces acting on a column are determined before its removal. Then, the column is removed and replaced by concentrated loads equivalent to its forces as shown in Figure 8. More details can be found in Tavakoli et al. (2013). In this study, the loads increased linearly for five seconds until they reached their full amounts, were kept unchanged for two seconds, and the concentrated forces were rapidly removed at seven seconds to simulate column failure (Kim et al., 2009).

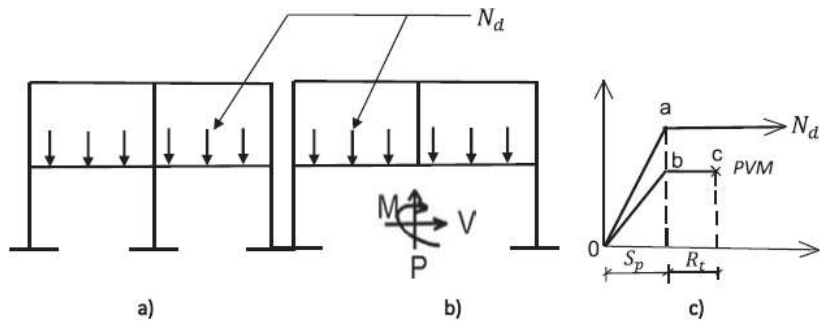


Figure 8. Loading for dynamic column removal.

4.3. Non Linear Dynamic Analysis by Hilber-Hughes-Taylor Method

The time history anlysis performed in SAP 2000 by using the Hilber-Hughes-Taylor method (also called α- method). It is an extension to the Newmark method. With the Hilber-Hughes-Taylor method, it is possible to introduce numerical dissipation without degrading the order of accuracy. The Hilber-Hughes-Taylor method uses the same finite difference formulas as the Newmark method with fixed γ and β

$$\gamma = \frac{1}{2}(1\text{-}2\alpha) \tag{1}$$

$$\beta = \frac{1}{4}(1\text{-}\alpha)^2 \tag{2}$$

The time-discrete equation of motion is modified as follows:

$$\mathbf{M}\,^{t+\Delta t}\ddot{\mathbf{u}} + (1+\alpha)\mathbf{C}\,^{t+\Delta t}\dot{\mathbf{u}} - \alpha\mathbf{C}\,^{t}\dot{\mathbf{u}} + (1+\alpha)\,^{t+\Delta t}\mathbf{f}_{\text{int}} - \alpha\,^{t}\mathbf{f}_{\text{int}} = \,^{t+(1+\alpha)\Delta t}\mathbf{f}_{\text{ext}} \tag{3}$$

For $\alpha = 0$ the method reduces to the Newmark method.

For $-\frac{1}{3} \leq \alpha \leq 0, \gamma = \frac{1}{2}(1\text{-}2\alpha)$ and $\beta = \frac{1}{4}(1\text{-}\alpha\ )^2$ the scheme is second order accurate and unconditionally stable. Decreasing $\boldsymbol{\alpha}$ means increasing the numerical damping. This damping is low for low-frequency modes and high for the high-frequency modes.

5. Result and Discussion

The building model was critically examined by pushover analysis, and its configuration and property were selected strictly complying with a target performance. Base shear and drift are two essential criteria used in selecting building configuration. After ascertaining the building configuration, collapse analysis was performed on all three individual building models. Critical Column Loss cases considered are Corner (CCR), Middle (MCR), Middle of long side (LSCR) and Middle of the short side (SSCR). The collapse analysis of corner column loss in CP-RCBM shows the total collapse in the Column loss area, whereas other sites remain intact. The deformation is not stable after 2.5 seconds of time history. It is shown in Fig 10. No plastic hinge formation was noticed in the area other than column loss location and is shown in Fig 9.

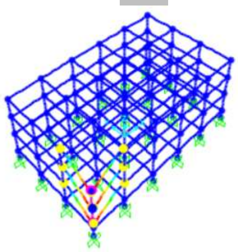


Figure 9 . Deformed Shape of CCR case of CP-RCBM

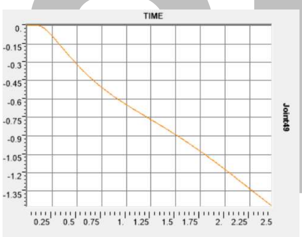


Figure 10. Deflction at column loss point in storey 1 of Figure 9

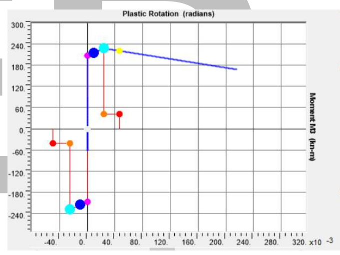


Figure 11. Hinge Rotation of Beam at storey 1 in Figure 9.

In the middle column removal case, the result is not converging after 1.17 seconds of time history analysis. The beams at the column loss location have reached complete collapse, whereas plastic hinge formation is also seen in other places as shown in Fig 12. Many columns have gone beyond the CP stage of plastic rotation.

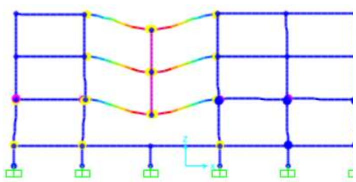


Figure 12. Deformed Shape of LSCR case of CP-RCBM

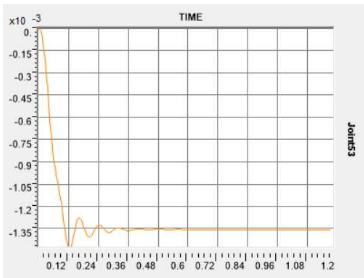


Figure 13. Deflction at column loss point in storey 1 of Figure 12.

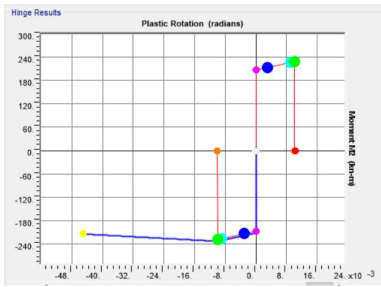


Figure 14. Hinge Rotation of Beam in storey 1 in Figure 12.

A similar situation was observed in the side column removal case. In collapse analysis of corner column loss, LS-RCBM also shows the total collapse in the Column loss area, whereas other sites remain intact. The deformation is not stable after 2.5 seconds of time history.

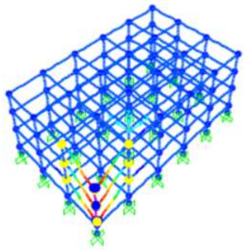


Figure 15. Deformed Shape of CCR case of LS-RCBM

Figure 16. Deflction at column loss point in storey 1 of Figure 15

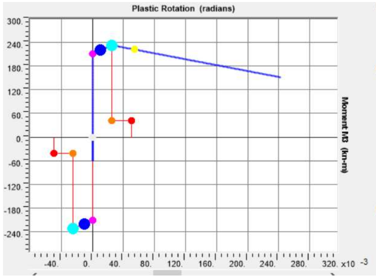


Figure 17. Hinge Rotation of beam in storey 1 in Figure 15

In collapse analysis of corner column loss, IO-RCBM shows the total collapse in the Column loss area where other areas remain intact. Figure 18 shows the formation of plastic hinge. The deformation is stable after 0.5 seconds of time history as shown in figure 19. It has been noticed that the hinge formation is confined to column removal only.

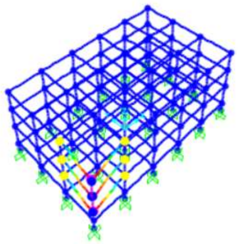


Figure 18. Deformed Shape of CCR case of IO-RCBM

Figure 19. Deflction at column loss point in storey 1 of Figure 18

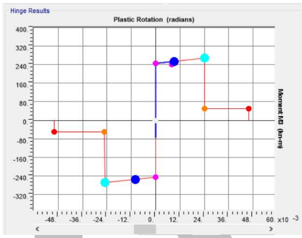


Figure 20. Hinge Rotation of beam in storey 1 in Figure 19

In collapse analysis of middle column loss, IO-RCBM shows the total collapse in the Column loss area where other areas remain intact. The deformation is stable after 0.52 seconds of time history. It has been noticed that the hinge formation is confined to column removal only. A similar situation was observed in the side column removal case but analytical solution are converging. The final deflection at column loss location tabulated in table 6. IO-RCMB has relatively better stable result after column loss.

Table 6. Maximum Deflection at column Loss Location

Building Name	CCR	MCR	LSCR	SSCR
CP-RCBM	1350 mm	1442 mm	1355 mm	1335 mm
LS-RCBM	1325 mm	1318 mm	1309 mm	1305 mm
IO-RCBM	178 mm	162 mm	152 mm	145 mm

6. Conclusions

Performance based design is widely used in connection with seismic load. In this study three RC frames are considered designed using STAAD Pro and examined in SAP 2000 for their performance. Ascertaining their performance, same building were examined for their collapse potential in SAP 2000. The progressive collapse analysis of Buildings are performed as per GSA guideline 2016. Automatic hinge parameter were assigned in beam and column. The main conclusions are as follows:

- CP and LS performance level building have not resisted well and beam deflection has gone beyond acceptable limit. When the Performance level increases, progressive collapse resistance will be increased as well.
- IO performance level building shows wide acceptability and performed well in column loss scenario.

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