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Research Article

Investigation of Progressive Collapse Resistance for a Seismically Designed RC Building

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Abstract: The objective of this study is the vulnerability phenomenon to progressive collapse of a 7-storey Reinforced Concrete (RC) building in conformance with ACI 318-11/IBC 2009. Progressive collapse is defined as a disastrous structural phenomenon due to human-made and natural hazards. The progressive collapse mechanism involves a single local failure which could lead to major deformations ending up with the total collapse of the structure. The building was designed as a Special Moment Resisting Frame (SMRF) and assumed to have an Occupancy Category II per UFC 3-301-01. Tie Forces (TF) and Enhanced Local Resistance (ELR) methods according to Unified Facilities Criteria (UFC 4-023-03) and Demand Capacity Ratio (DCR) method based on General Services Administration (GSA) were incorporated to investigate the probability of progressive collapse. According to GSA, the study illustrated that the building's columns did not require additional reinforcement to prevent progressive collapse. Also, the study revealed that additional reinforcement was required for all the three column removal cases to meet the progressive collapse requirements based on GSA. Furthermore, it was concluded that two-way slabs required additional reinforcement to meet UFC requirements.

Keywords: Alternate path method, progressive collapse, reinforced concrete structures, seismic designed, tie forces method

INTRODUCTION

Countless natural hazards such as earthquakes and flood plus manmade ones such as blasts or fire my influence the performance of structures during their lifespan. The design of structures is done to prevent the occurrence of unexpected hazards during their lifespan. Yet, there exists an extreme possibility of occurrence of unexpected events that may lead to disastrous failure of structures. Recently, the unexpected events like the bombing of Murrah Federal building in 1995 and the 9/11 terrorist attacks on World Trade Centre have demonstrated the vulnerability of manmade structures to such events. The consistent occurrence of unexpected incidents such as terrorist attacks necessitates the consideration of progressive collapse requirements in building design and analysis to mitigate its devastating consequences. As shown in Fig. 1a, once the column removal occurs, a brittle failure occurs in the resulting two-bay beam due to lack of continuous bottom reinforcement. The ductility and capacity requirements, however, could be met by incorporating Special Moment Resisting Frames (SMRF) as the lateral load resisting system to prevent progressive collapse (Fig. 1b). Various methods have been introduced to mitigate the effects of progressive collapse and many standards, building codes and design guidelines have been published to discuss this issue. Two of the most popular codes in the fields of progressive collapse are General Services Administration (The U.S. General Services Administration, 2003) and Unified Facilities Criteria (2010) (UFC 4-023-03) that have been used to design and analyze the structures against progressive collapse. In this study, a 7-story RC moment frame building was investigated in conformance with ACI 318-11 (ACI Committee 318, 2008) /IBC 2009 (International Code Council, 2009) codes to design the structure in high level seismic zones. Yet, the possibility of progressive collapse occurrence cannot be investigated using such codes. Hence, investigation of resistance of seismic designed structures against progressive collapse has been studied in accordance with Unified Facilities Criteria "UFC 4-023-03" (Unified Facilities Criteria, 2010) and Demand Capacity Ratio in accordance with General Services Administration (The U.S. General Services Administration, 2003) incorporating Tie Forces to achieve the above mentioned objective.

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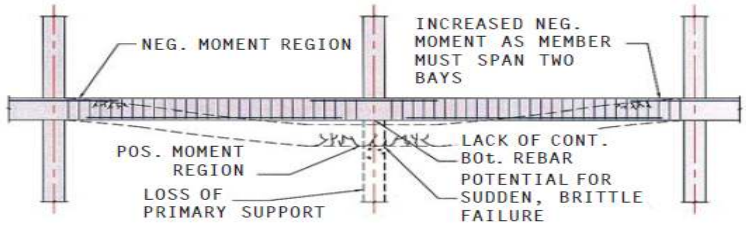
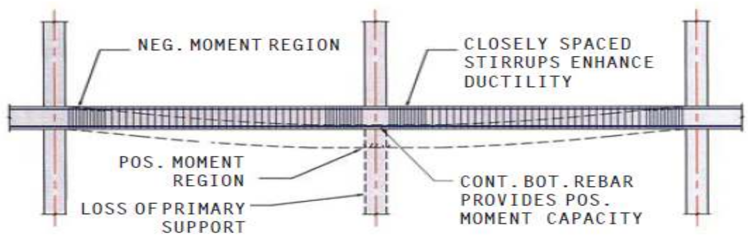


(a)



(b)

Fig. 1: Response of beam for “missing column” scenario, (a) gravity-load designed beam, (b) seismically designed beam

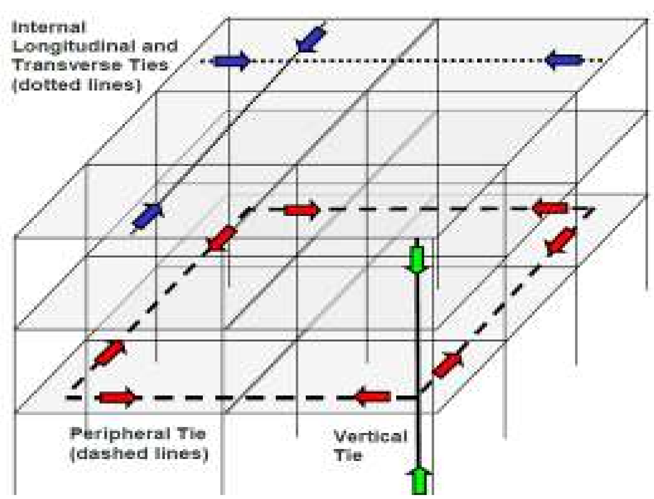


Fig. 2: Tie forces in a frame structure

MATERIALS AND METHODS

In this research, the following two options were incorporated to investigate probability of progressive collapse. The first option would include the incorporation of Tie Force requirements according to Unified Facilities Criteria (UFC 4-023-03) (Department of Defense 2010) for the entire structure and Enhanced Local Resistance for the first story corner and penultimate columns (a penultimate column is the closest column the corner). The second option would include the analysis of the building with the Alternate Path method as recommended in General Services Administration (The U.S. General Services Administration, 2003) to indicate the ability of the structure to bridge over the removal of columns at specified locations.

Tie force design procedures: The assumption behind the TF method is the existence of mechanical ties between the structural elements which enhances the ductility, continuity and development of alternate load paths. The provision of Tie Forces by the existing structural elements and connections designed applying the conventional procedures to carry the standard loads imposed upon the structure. Four types of ties namely viz internal, peripheral, ties to columns and walls, as well as vertical ties exist based on the location and function (Fig. 2). Being as short as possible would be the main requirement for the load paths of various types of ties while continuity and the required tie strength need to be maintained Eq. (1):

$$\Phi\ Rn \geq \Sigma\gamma i\ Qi \quad (1)$$

where,
Φ Rn = Design tie strength
$\Sigma\gamma i$ Qi = Required tie strength

Longitudinal and transverse ties: In order to provide the required longitudinal and transverse tie resistance, the floor and roof system is supposed to be used. The tie strength Fi (lb/ft or kN/m) required in the longitudinal or transverse direction is as follows Eq. (2):

$$Fi = 3\ WF\ L1 \quad (2)$$

where,
WF = Floor load, determined $(1.2D + 0.5L)$
L1 = Greater of the distances between the centers of the columns

Peripheral ties: The peripheral ties are placed within 3-ft (0.91-m) of the edge of a floor or roof and adequate development or anchors at corners, re-entrant corners or changes of construction is provided. The peripheral tie strength Fp (lb or kN) required is as follows Eq. (3):

$$Fp = 6\ WF\ L1\ Lp \quad (3)$$

where,
WF = Floor load, determined as $(1.2D + 0.5L)$
L1 = For exterior peripheral ties, the greater of the distances between the centers of the columns, frames, or walls at the perimeter of the building in the direction under consideration (m or ft). For peripheral ties at openings, the length of the bay in which the opening is located, in the direction under consideration:

Lp = 3-ft (0.91-m)

Verticals ties: The columns and load-bearing walls are incorporated to carry the demanded vertical tie strength. Columns are tied in a continuous manner from the foundation to the roof level. The vertical tie's design strength in tension is equal to the maximum vertical load absorbed by the column from any one story, applying the floor load WF and the tributary area as indicated in above section.

Enhanced local resistance design procedures: Shear capacity achievement of corner and penultimate perimeter columns at the first floor above grade along with the required shear demand for columns to achieve enhanced flexural resistance are the necessary provisions for Occupancy Category II. By the time the conventional design process is done, the determination of flexural resistance, which is equal to the enhanced flexural resistance for Occupancy Category II, is performed. The determination of the required shear resistance is executed using Eq. (4):

$$Vu = 7.5\ MP/L \quad (4)$$

where,
Vu = Required shear strength
MP = Column moment capacity accounting for axial load
L = Column height

Alternate path: The alternate path method is utilized by the GSA criterion to guarantee that the progressive collapse will not occur. The linear elastic static analyses or non-linear dynamic analysis are incorporated by designers to control the capacity of structural members in the alternate path structure, i.e., the behavior of structural member once a single column is removed. This study is based on linear elastic static analysis. The static analysis procedure and its relevant limitations regarding its use are provided in the upcoming sub-sections.

Loading: Each structural member of the alternate path structure is subjected to the following gravity load for static analysis purposes Eq. (5):

$$Load = 2\ (DL + 0.25LL) \quad (5)$$

where,
DL = Dead load
LL = Floor live load

Analysis procedure and acceptance criteria: The progressive collapse potential can be estimated incorporating the following procedure:

Step 1: The analysis of the components of the primary structural elements will be done once an instantaneous loss in primary vertical support occurs. A consistency needs to exist between the applied downward loading and the one presented in above section.

Step 2: An evaluation on the results from the analyses in step 1 needs to be done incorporating the analysis criteria introduced in the following Eq. (6):

$$DCR = QUD/QCE \quad (6)$$

where,
QUD = Acting force determined in the structural element

Table 1: Material properties

	P Kg/m^3	ϒ Kgf/m^3	ν	E MPa	F_y MPa	F_{ys} MPa	f'_c MPa
Concrete material properties	245	2400	0.2	$\Upsilon^{1.5} \times 0.043\sqrt{fc'}$	420	300	21

QCE = Expected ultimate, un-factored capacity of the structural element

It could be noted that determination of the ultimate capacity of the structural component is done through an allowed material strength increase of 25% for concrete and reinforcement bars.

Step 3: Prevention of the collapse of the alternate path structure is guaranteed once the values of DCR for each structural element is equal or less than the upcoming Eq. (7):

$$DCR \leq 2.0 \quad (7)$$

Once the DCR values of the structural elements surpass the above limits, there will be no additional capacity for effective redistribution of loads in structural members and hence they will be considered as failed. Consequently, this will eventually lead to the collapse of the entire structure. The DCR methodology mentioned above is in conformance with NEHRP (FEMA, 1997) Guidelines for the Seismic Rehabilitation of Buildings issued by FEMA (1997).

Configuration and analytical modeling of progressive collapse design: In order to demonstrate the process of design against progressive collapse, a typical reinforced concrete structure has been taken into account. The occupancy of the structure is less than 500 people and therefore, is categorized as Occupancy Category II per UFC 3-301-01 (Unified Facilities Criteria, 2012). The structure under investigation is a seven-story SMRF and its intended function would be for office use.

Modeling assumptions:

- **Systems of gravity:** Two Way Slab
- **Vertical support:** Columns
- **Lateral:** Special Moment Resistant Frames (SMRF)
- **Foundation:** Shallow footings
- Wind Load (W) was calculated per ASCE 7-05 (ASCE Members, 2006) incorporating 110 mph with exposure B and importance factor equal to 1.0
- Earthquake Load (E) is assumed based on IBC 2009 (International Code Council, 2009) using Response Modification Factor R = 10, Site Class = B, Occupancy Importance = 1 and Design Spectral Accelerations (S_{DS} and S_{D1}) = 2.29 and 0.869 based on this data the Seismic Design Category (SDC) determine D

Table 2: Gravity loads

Load type	Weight KN/m^2
Slab	0.17×24 = 4.08
Flooring dead load	1.9
Live load	5
The total load	10

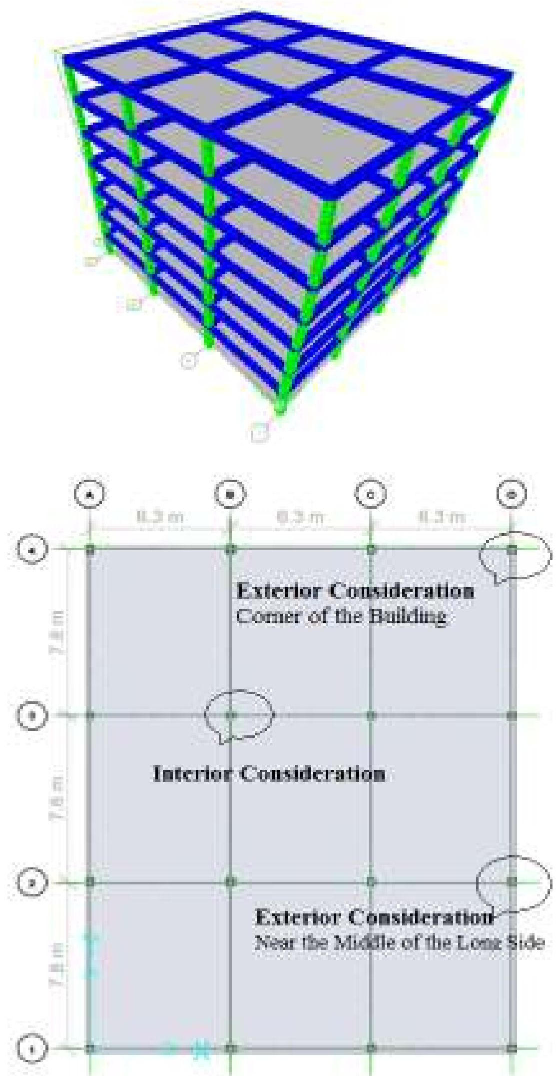
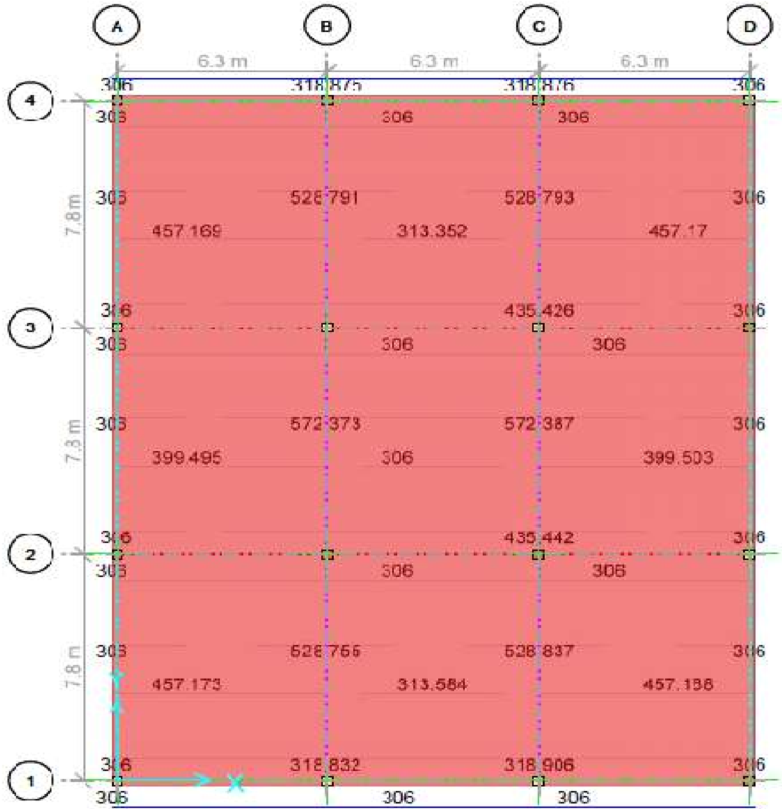


Fig. 3: Proposed RC structure

Table 1 and 2 demonstrate the concrete material properties and loading assumptions, respectively. The plan view of the proposed RC structure as well as removal column location is indicated in Fig. 3.

RESULTS AND DISCUSSION

Design requirement by ACI 318-08/IBC 2009:
Design of two-way slabs for gravity loading: Plate bending theory, which is a complex extension



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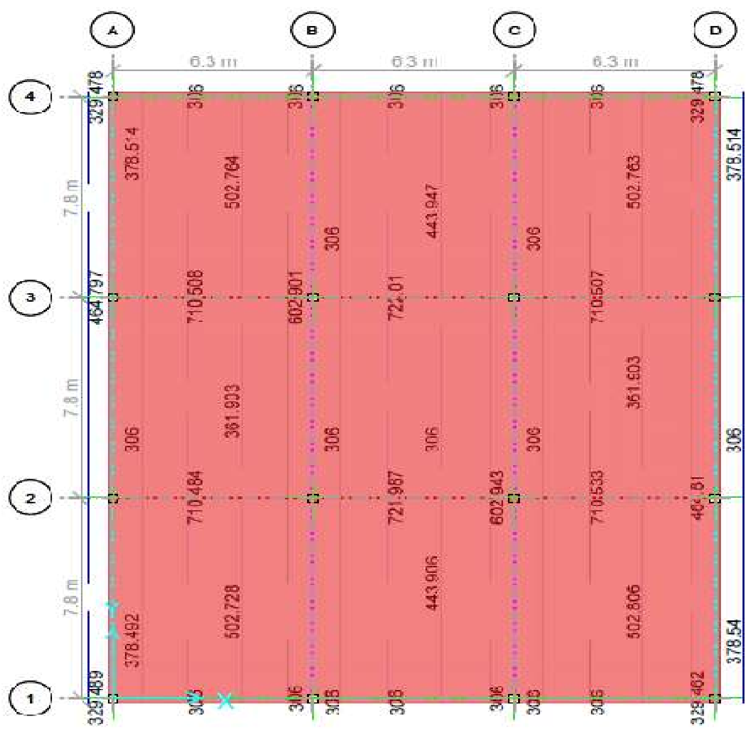


(b)

Fig. 4: Top and bottom reinforcement (mm^2/m), a) longitudinal direction, b) transverse direction

Mark	t (mm)	Spacing of 12 mm Ø bars (mm)						Remarks
		Longitudinal Direction			Transvers Direction			
		"a"	"b"	"c"	"a"	"b"	"c"	
S1	175	300	200	200	300	200	150	Two-way

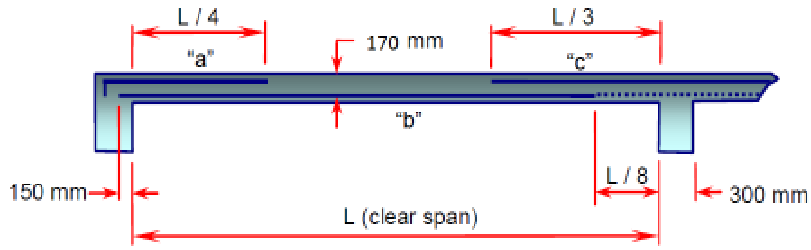


Fig. 5: Slab section detail based on ACI 318-08

Table 3: Frames elements dimension

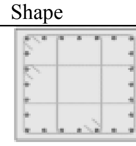
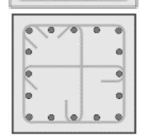
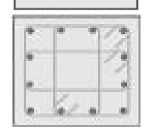
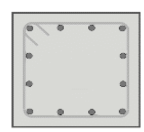
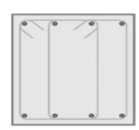
Element type	Level	Location	Dimension mm	Longitudinal reinforcement	Transverse reinforcement	Shape
Column	1 and 2	Interior	650×650	24 Ø 28	Ø 10 @ 150 mm 4 each direction	
		Long side and short side	650×650	16 Ø 28	Ø 10 @ 150 mm 3 each direction	
		Corner	500×500	16 Ø 22	Ø 10 @ 150 mm 3 each direction	
	3 and 4	Interior	500×500	12 Ø 32	Ø 10 @ 200 mm 4 each direction	
		Long side and short side	500×500	12 Ø 28	Ø 10 @ 200 mm 4 each direction	
		Corner	500×500	12 Ø 22	Ø 10 @ 200 mm 4 each direction	
	5 and 7	Interior	450×450	12 Ø 25	Ø 10 @ 150 mm 2 each direction	
		Long side and short side	450×450	12 Ø 20	Ø 10 @ 150 mm 2 each direction	
		Corner	450×450	12 Ø 18	Ø 10 @ 150 mm 2 each direction	
Beam	1 and 2	Transverse and longitudinal (interior)	300×600	Top = 5 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
		Transverse and longitudinal (exterior)	300×600	Top = 4 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
	3 and 4	Transverse and longitudinal (interior)	300×600	Top = 6 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
		Transverse and longitudinal (exterior)	300×600	Top = 5 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
	5 and 7	Transverse and longitudinal (interior)	300×600	Top = 5 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
		Transverse and longitudinal (exterior)	300×600	Top = 4 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	

Table 4: Tie force calculation

Tie type	Location	Length m	wF KN/m^2	F KN/m	As req'd mm^2/m	Reinforcement
Longitudinal (short length)	Distributed	6.30	9.676	182.87	469	φ12@200
Transverse (long length)	Distributed	7.80	9.676	226.41	580	φ12@150
Tie type	Location	Length m	wF KN/m^2	F KN	As req'd mm^2	Reinforcement
Peripheral	Longitudinal (short length)	6.30	9.676	332.83	853	3φ20
Peripheral	Transverse (long length)	7.80	9.676	412.08	1057	4φ20
Tie type	Location	Area m^2	wF KN/m^2	F KN	As req'd mm^2	Reinforcement
Vertical	Corner	12.30	9.676*7	832	2130	No additional
Vertical	Long and short side	24.57	9.676*7	1664	4260	No additional
Vertical	Interior	49.14	9.676*7	3328	8521	No additional

Table 5: Enhanced local calculations

Location	Moment capacity (M_p) KN.m	Axial load KN	Required shear strength (V_u)	Shear resistant capacity (V_r)	Stirrup reinforcement require
Corner	241	1713	583	450	Ø 10 @ 120 mm 4 each direction
Penultimate	324	3129	783	450	Ø 10 @ 100 mm 4 each direction

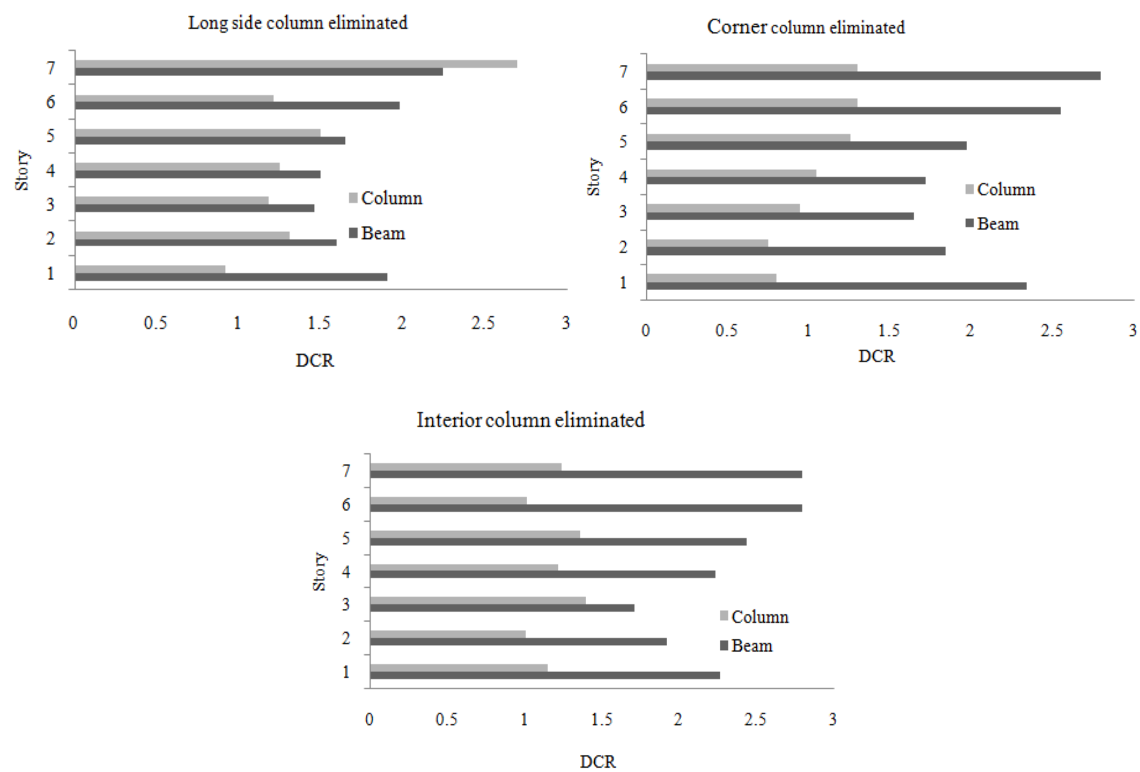


Fig. 6: DCR vs. stories

of beam bending, describes the two-way slab behavior. Depending on the Length-to-Breadth (L/B) ratio of the slab in a framed building, it was a two-way slab. A series of two dimensional equivalent frames for each spanning direction were representatives of the slab system.

Computation of the slab thickness was done according to its minimum thickness and Eq. (6). Based on the calculations, the thickness of slab was chosen as 170 mm:

$$\text{T min} = \frac{P}{160} > 75mm \quad (8)$$

where,

P = The perimeter of the slab

The required reinforcement schedule for each direction is demonstrated in Fig. 4 based on reinforcement requirements. Also, the slab section details are illustrated in Fig. 5.

Design of frame elements: The column and beam dimensions along with the details of arrangement of longitudinal reinforcement are shown in Table 3.

Design requirements for progressive collapse (by UFC 4-023-03):

Tie force requirement: The following table calculates the longitudinal, transverse, vertical and peripheral ties according to above section (Table 4).

The assumption for ELR evaluation includes having fixed columns at the first level and pinned ones at the base. All the corner and penultimate columns in this model possess shear and flexural strength which is a function of axial load. Equation (4) clearly defines the required shear resistance. Furthermore, Enhanced Local calculations based on shear calculations are presented in Table 5.

Alternate path requirement: In this study, evaluation of the progressive collapse potential of a seven-story reinforced concrete structure was done incorporating the linear static analysis considering the removal of a column in three different cases including exterior corner column, exterior long side column and interior column. First, the structure was designed in conformance with ACI 318-11/IBC 2009. Then, for each case of the column removal, the linear static analysis was performed. Next, calculation of the demand capacity ratio for flexure at all story levels was performed for the three independent cases of column removal. For instance, once the column in the middle of the long side was removed, the beam span was doubled from 7.8 to 15.6 m.

As a result, the new 15.6-m beam must be able to provide an alternate load path into the adjacent columns. Therefore, a positive moment was created over the removed column. Once the column was removed in each separate case, calculation of all the DCR values for critical cases was done and DCR values were plotted against stories as shown in Fig. 6.

CONCLUSION

Examining the probability of progressive collapse in SMRF reinforced concrete structures in conformance with the 2009 International Building and ACI 318-11 (ACI Committee 318, 2008) Code was the main objective of this study. An inherent ability to resist progressive collapse exists in seismically designed structures compared to Ordinary Moment Resistant Frames (OMRF). Provision of adequate reinforcement to prevent the progressive failure of beams and columns as a result of removing a particular column is necessary to put some limits on the DCR within the acceptance criteria. In general, development of alternate load paths resulting from the removal an individual member is possible once the structures are designed with adequate reinforcement. Conclusions for the 7-story building used in this study are as follows:

- Since the value of column DCR for almost all removal cases investigated in this study were less than 2, the columns will not fail and do not require changing in order to meet the GSA criteria for buildings designed for SDC D.
- For interior column removal case, more beams require additional reinforcement to satisfy the GSA criteria compared to long side and corner column removal cases. In other words, in case of interior column removal, the structure is more susceptible to experience progressive collapse.
- It is evident that for structures designed in conformance with 2009 International Building (International Code Council, 2009) and ACI 318-11 (ACI Committee 318, 2008) Code as SMRF, the roofs need to be reanalyzed according to UFC 4-23-03 (Unified Facilities Criteria, 2010) to avoid progressive collapse. The results of this study indicated that roofs are the most vulnerable structural elements in case of a sudden removal of any structural element and hence, require retrofitting immediately.

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