Research Article
Economic Design Issues of RC Structures against Progressive Collapse

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Abstract: Progressive collapse of structures is defined as the damage incurred on the structure as a result of abnormal and occasional loads which result in initiation of a chain reaction mechanism ending with progressive and disastrous failure. In this study, the economic cost of designing RC structures located in non-earthquake hazardous regions to meeting progressive collapse design requirements is estimated. Therefore, a typical reinforced concrete has been modeled to achieve the above objective. The Occupancy Category II per UFC 3-301-01 has been chosen for this structure. Tie Forces for the whole structure and Enhanced Local Resistance for the corner and penultimate columns at the first story are the design requirements for this OC. It was concluded that the cost of reinforcement for this OC level in order to meet UFC criteria and maintain safety requirements was around 15% compared to reinforcement requirements of ACI 318-08/IBC 2009 for non-seismic region. The importance of progressive collapse as a major concern has led to addition of explicit requirements regarding the redundancy of the structures in seismic codes throughout the world. Specific codes related to progressive collapse analysis of structures have been published by the Department of Defense (UFC 4-23-03) and the General Services Administration (GSA 2003). Most of the countries located in non-seismic regions are not designed properly against earthquakes according to the relevant seismic codes. Hence, the majority of typical RC buildings are exposed to progressive collapse in such regions.

Keywords: Enhanced local resistant, progressive collapse, RC structures, Tie Force (TF) method

INTRODUCTION

After the partial collapse of the London's Ronan Point apartment tower in 1968, investigations on progressive collapse resistance has begun by many nations and a series of design codes, guidelines and specifications including the ACI318-08 (ACI Committee 318, 2008), General Services Administration (David N. Bilow *et al*., 2003) and Unified Facilities Criteria 4-023-03 (Unified Facilities Criteria, 2010) have been published. General recommendations to prevent progressive collapse on the basis of provision of redundancy, continuity, integrity, ductility and path redistribution have been provided by current codes. A quick review on the previous literature indicates that the majority of projects have been conducted on reinforced concrete frame structures. Hence, for the purpose of assessing the progressive collapse of a multi-story structure, the instantaneous loss of a column is considered during the design phase. Others studied the sudden removal of columns in different conditions based on the methodology suggested by General Services Administration (David N. Bilow *et al*., 2003) in high-rise concrete structures.

Different software was used to determine the ability of seismic designed frame structures to withstand progressive collapse. Furthermore, most actual design codes are published to design the structures for conventional loads during their service life. Hence, the effect of abnormal loads such as blasts due to events like gas explosion and terrorist attacks or vehicle impact are not considered in the design of structures and only general recommendations are offered by several codes to mitigate the devastating effects of progressive collapse. As a result, appropriate load resisting systems must be selected by engineers to take into account the effect of lateral, gravity and progressive collapse loads simultaneously. Redistribution of loads and therefore, prevention of progressive collapse cannot be provided by structures containing interior core walls and ordinary moment frames to resist lateral forces or flat-plate systems for gravity loads. This is because the design of gravity-load systems does not provide alternative load paths once a primary vertical support has been removed. Figure 1a indicates a brittle failure of a two-bay beam resulted from the removal of a column caused by lack of continuous bottom reinforcement in the beam.

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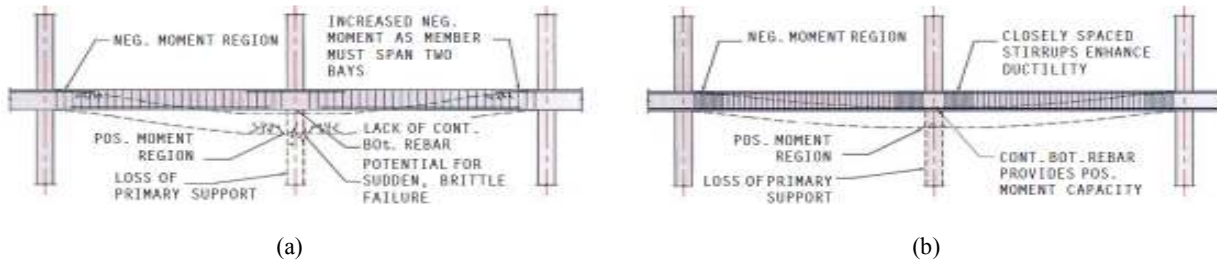(a)   (b)

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

However, incorporation of Special Moment Resisting Frames (SMRF) that provide the required ductility and capacity by their special lateral systems could lead to progressive collapse prevention (Fig. 1b). Cost estimation of the redesign of structures located in non-seismic regions based on Unified Facilities Criteria4-023-03 (Unified Facilities Criteria, 2010) is the objective of this study. In order to achieve this, the Tie Force Method and Enhanced Local Resistance Method are incorporated as the design requirement for OC II.

METHODOLOGY

One of the two following options may be selected for OC II structures. The Tie Force requirement for the entire structure and Enhanced Local Resistance for the first story corner and penultimate columns (a penultimate column is the closest column to the corner) shall be incorporated by the designer in the first option. Also, the analysis and design of the building using the Alternate Path method for indicating the structure's ability to bridge, once a column or a load-bearing wall or a wall at certain locations are removed, will be done by the designer in the second option. In this study, the first option is considered as design requirement. In the TF method, the assumption is that the structural elements are mechanically tied together and hence, the continuity, ductility and alternate load paths development will enhance consequently. The provision of Tie Forces is typically done through the existing structural elements and connections designed by incorporation of conventional procedures to support the loads the structure is subjected to. Ties are categorized into four types based on their location and function namely viz internal, peripheral, ties to columns and walls, as well as vertical ties (Fig. 2). Various types of ties possess load paths which have to be the shortest. Simultaneously, the continuity needs to be maintained and the required tie strength need to be satisfied.

Tie force design procedures: The design of tie strength based according to Load and Resistance Factor Design (LRFD)approach his done through multiplying the nominal tie strength, Rn and the strength reduction factor, Φ in conformance with the specifications and requirements of applicable material specific codes Eq. (1):
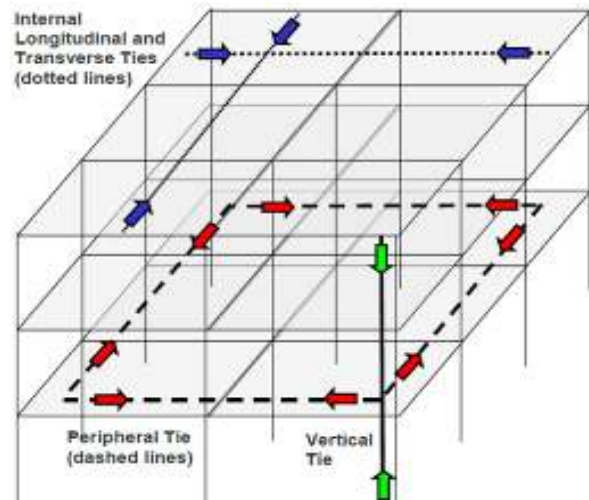

Fig. 2: Tie forces in a frame structure

$$\Phi\ R_n \geq \Sigma\ \gamma_i\ Q_i \qquad (1)$$

where,
$\Phi\ R_n$ = Design tie strength
$\Sigma \gamma_i\ Q_i$ = Required tie strength

Longitudinal and transverse ties: The provision of the required transverse and longitudinal tie resistance is done through the use of the floor and roof system. Some or all of the required tie forces are provided by the structural members such as girders, beams, spandrels and so on once the structural members and their connections are able to carry the total internal tie force which acts over the structural member spacing while a 0.20-rad (11.3-deg) rotation is in progress. The tie strength F_i (*lb/ft* or *kN*/m) required in the transverse of longitudinal direction is as follows Eq. (2):

$$F_i = 3\ WF\ L1 \qquad (2)$$

where,
WF = Floor load, determined per (1.2D+0.5L) (*lb/ft²* or kN/m²)
$L1$ = Greater of the distances between the centers of the columns

Table 1: Material properties

	P Kg/m³	Ƴ Kgf/m³	V	E MPa	F_y MPa	F_{ys} MPa	f'_c MPa
Concrete material properties	245	2400	0.2	$\gamma^{1.5} \times 0.043\sqrt{fc'}$	420	300	21

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

$$Fp = 6 \, WF \, L1 \, Lp \qquad (3)$$

where,
WF = Floor load, determined per (1.2D + 0.5L) (*lb*/*ft²* or *kN*/m²)
$L1$ = Greater of the distances between the centers of the columns
Lp = 3-*ft* (0.91-m)

Verticals ties: The required vertical tie strength is carried through the use of the columns and load-bearing walls. The tying of each column is done in a continuous manner form the foundation to the roof level. The design strength of the vertical tie in tension equals to the maximum vertical load received by the column from any one story, incorporating the floor load WF and the tributary area as demonstrated in above section.

Enhanced local resistance design procedures: One of the requirements for enhanced local resistance provisions for Occupancy Category II is the shear capacity and demand of the columns at the first floor above grade reach the enhanced flexural resistance of the columns. Once the conventional design process is finished, the flexural resistance is determined which must be equal to the enhanced flexural resistance for Occupancy Category II. The required shear resistance is determined by incorporation of Eq. (4) as follows:

$$Vu = 7.5 \, MP/L \qquad (4)$$

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

CONFIGURATION AND ANALYTICAL MODELING OF PREFERRED STRUCTURES

The design process against progressive collapse is illustrated by modeling a typical reinforced concrete structure. The occupancy of the structure is less than 500 people and therefore, is categorized as Occupancy

Table 2: Gravity loads

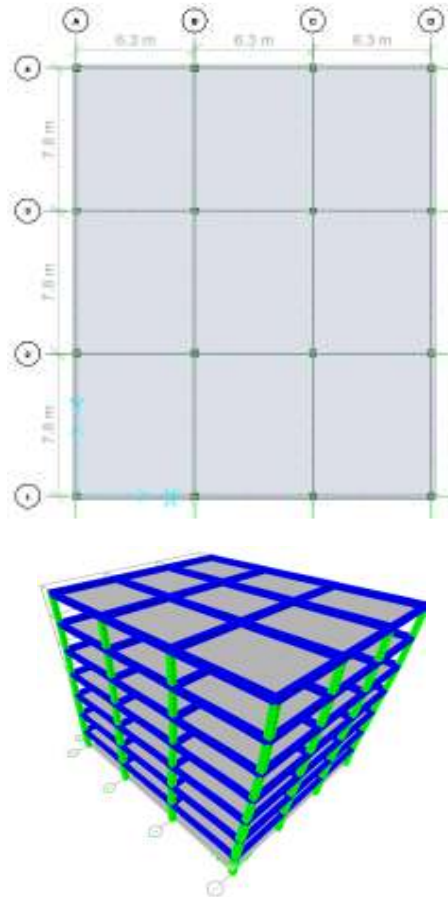| Load type	Weight KN/m²
Slab	0.17×24 = 4.08
Flooring dead load	1.9
Live load	5
The total load	10

Fig. 3: Proposed RC structure

Category II according to Unified Facilities Criteria 3-301-01 (Unified Facilities Criteria, 2012). The number of stories is seven and its function is office use. Furthermore, the lateral load bearing system is moment resisting frame.

Modeling assumptions:

- **Systems of gravity:** Two Way Slab
- **Vertical support:** Columns
- **Lateral:** Ordinary Moment Resistant Frames (OMRF)
- **Foundation:** Shallow footings
- Wind Load (W) was determined per Minimum Design Loads for Buildings and Other Structures
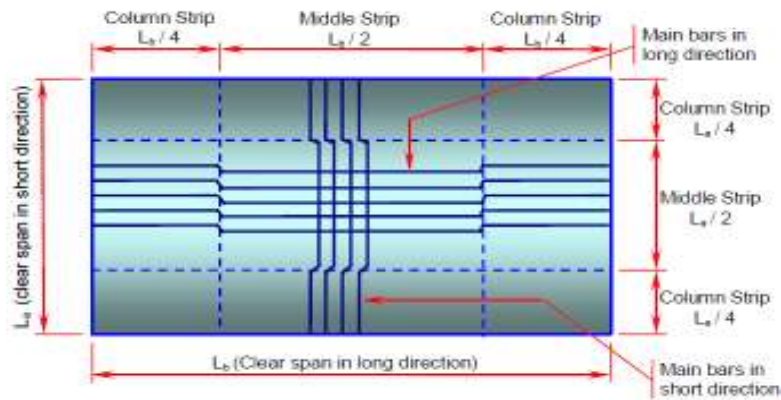
Fig. 4: Middle and column strips

"ASCE 7-05" (ASCE Members, 2006) using 110 mph with exposure = B and importance factor = 1.0

• Earthquake Load (E) is assumed not to control the design because the building is in a non-seismic region

The concrete material properties and Loading Assumptions are illustrated in Table 1 and 2, respectively. Figure 3 demonstrates the plan view for proposed RC structure.

ANALYSIS AND DESIGN OF THE MODELED STRUCTURE

Design requirement by ACI 318-08/IBC 2009: Design of two-way slabs for gravity loading: The plate bending theory which is a complex extension of beam bending describes the behavior of two-way slab. Based on the Length-to-Breadth (L/B) ratio of a slab in a framed building, it could either be one or two-way. The behavior of two-way slabs are similar to mat (raft) foundations. A series of two dimensional equivalent frames represent the slab system for each spanning direction. An equivalent frame along a column line is a slice of the building bound by the center-lines of the bays adjacent to the column line. A column strip and two middle strips are derived from the division of the width of the equivalent frame. Each Middle Strip (MS) is composed of the remains of two adjacent equivalent frames (Fig. 4). Computation of the thickness of the slab is done using the minimum thickness, Eq. (5):

$$T \, min = \frac{P}{160} > 75mm \qquad (5)$$

where,
P = Perimeter of the panel

The thickness of the slab was chosen to be 170 mm based on the above equation. Also, the minimum reinforcement required for the slab is determined via
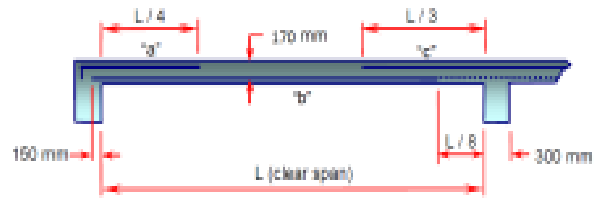

Fig. 5: Slab section detail based on ACI 318-08

Eq. (6), ACI 318-08 (ACI Committee 318, 2008), which is 306 mm^2:

$$Ast = 0.0018 \, b.t \qquad (6)$$

Based on reinforcement requirements, Fig. 5 demonstrates slab section details and Fig. 6 demonstrates the required reinforcement schedule for each direction.

Design of columns: Since the structure is located in non-seismic region, the seismic design detailing of columns are not required. The required reinforcement based on gravity gravitational loads are presented in Fig. 7.

Design requirements for occupancy categories II (by UFC 4-023-03):
Tie force requirement: The following formulas calculate the longitudinal, transverse, vertical and peripheral ties according to above sections and (Table 3):

• **Longitudinal and transverse ties:**

$$WF = 1.2 \, D + 0.5L \rightarrow$$
$$WF = 1.2 * (4.08 + 1.9) + 0.5 * (5)$$
$$= 9.676 \, KN/m^2$$
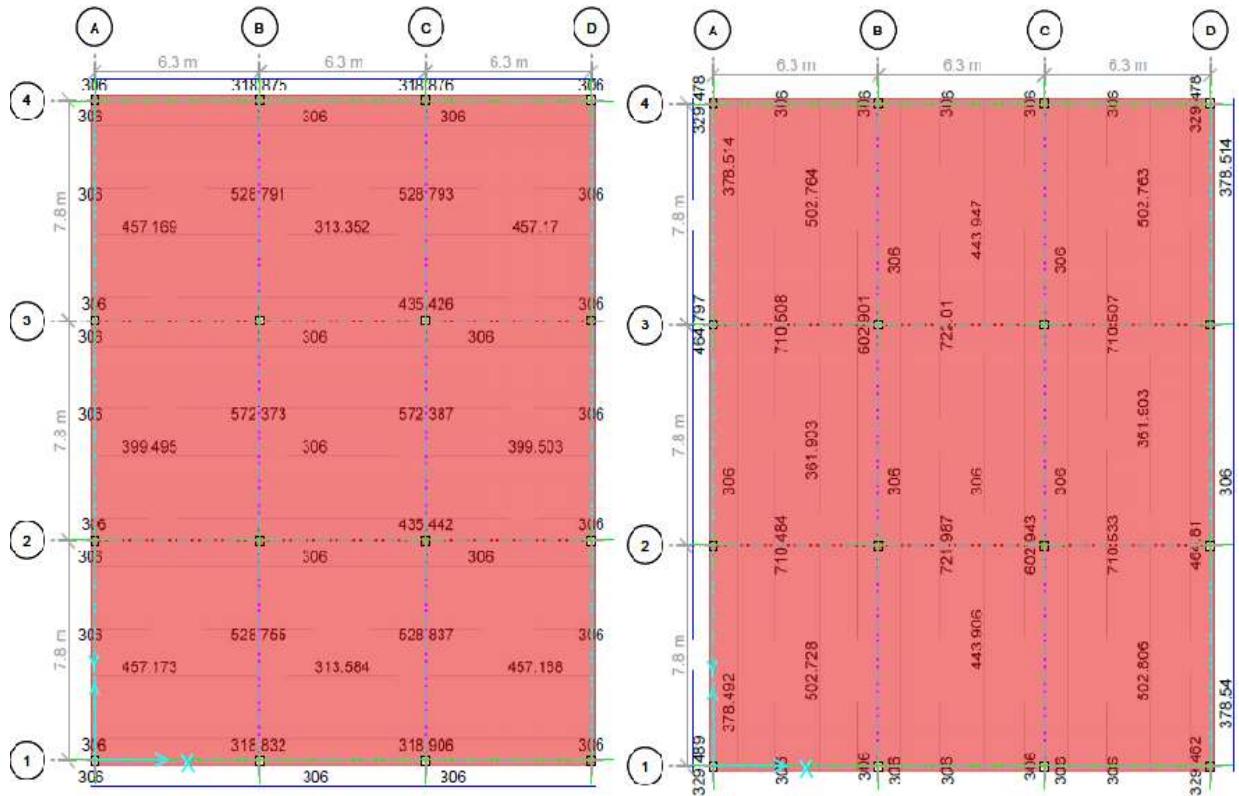
Longitudinal Ties $\rightarrow$

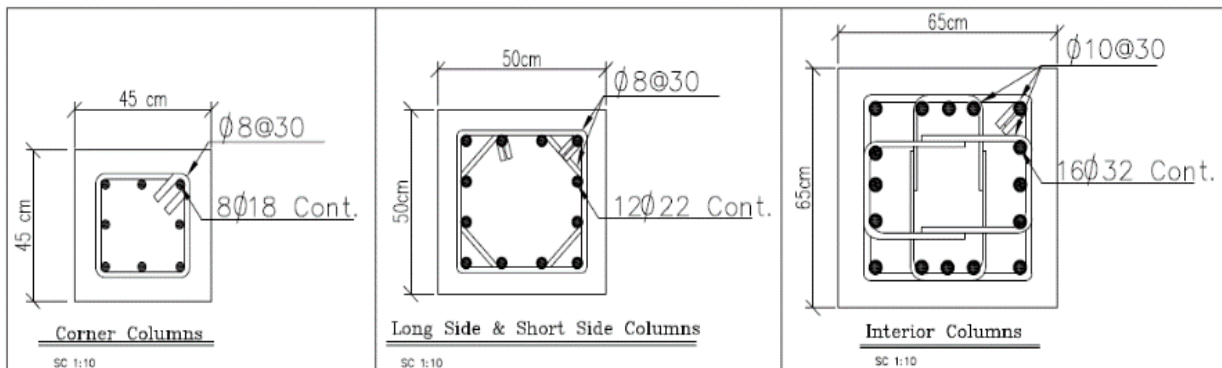Fig. 6: Top and bottom reinforcement (mm²/m) longitudinal direction and transverse direction

Fig. 7: Column reinforcement at first elevated level

Table 3: Slab section detail based on ACI 318-08

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

Table 4: Typical slab section detail based on UFC 4-023-03

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

Table 5: Tie force calculation

Tie type	Location	Length m	wFKN/m²	FKN/m	As req'd mm²/m	Reinforcement
Longitudinal (short length)	Distributed	6.30	9.676	182.87	469	φ12@20
Transverse (Long length)	Distributed	7.80	9.676	226.41	580	φ12@15
Tie type	Location	Length m	wFKN/m²	FKN	As req'd mm²	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 m²	wFKN/m²	FKN	As req'd mm²	Reinforcement
Vertical	Corner	12.30	9.676*7	832	2130	2φ10
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 6: Enhanced local resistance calculation

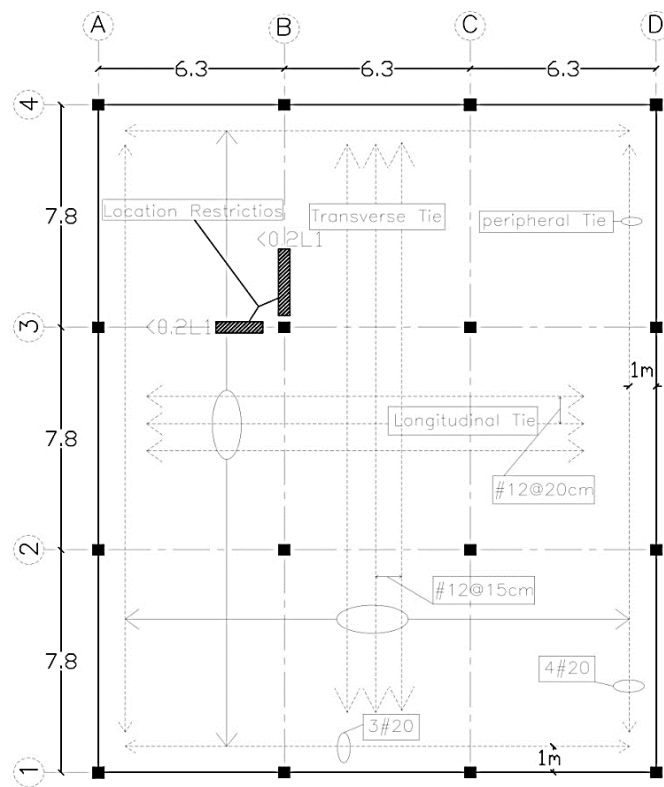| Location	Moment capacity (Mₚ) KN.m	Axial load KN	Required shear strength (Vᵤ)	Reinforcement
Corner	21.6	556	52.3	No additional
Penultimate	94.3	3119.7	228	105 mm²

Fig. 8: Slab reinforcement based on UFC 4-023-03

$Fi = 3 * 9.676 * 6.30 = 182.87 \ KN/m$

Transverse Ties →

$Fi = 3 * 9.676 * 7.80 = 226.41 KN/m$

• **Peripheral ties:**

$WF = 1.2 \ D + 0.5L$ →

$WF = 1.2 * (4.08 + 1.9) + 0.5 * (5) = 9.676 \ KN/m^2$

PeripheralLongitudinal Ties →

$Fi = 6 * 9.676 * 6.30 * 0.91 = \ 332.83 \ KN$

Peripheral Transverse Ties →

$Fi = 6 * 9.676 * 7.80 * 0.91 = \ 412.08 \ KN$

• **Vertical ties:** The reinforcement calculated in conformance with ACI 318-08 (ACI Committee 318, 2008) is capable of being incorporated as vertical tie strength without the need for extra reinforcement. The design strength of the vertical ties in tension must be equal to the largest amount of vertical load subjected to the column from any story.

The slab section and reinforcement plan for the purpose of meeting the requirements of progressive collapse are demonstrated in Fig. 8 and Table 4, respectively. The tie force calculations and the required reinforcement are tabulated in Table 5.

Enhanced local resistance: The assumption for ELR evaluation would be 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. The Enhanced Local Calculation is presented in Table 6 According to shear calculations; the shear associated with this moment capacity obliges the replacement of stirrups from #8@30 cm to #8@25 cm for Penultimate columns.

RESULTS AND DISCUSSION

Based on the calculations done conforming to the specifications of ACI 318-08 ACI Committee 318 (2008) and International Building Code "IBC" (International Code Council, 2009), the designed structure lacks minimum required strength against progressive collapse and hence, retrofitting is inevitable. There are no specific design requirements for the design of roof in seismic areas in ACI 318-08 (ACI Committee 318, 2008).

In other words, a single procedure is incorporated for the design of roofs in all regions. Obviously, the structural elements designed in conformance with Specific design criteria for earthquake design have a more satisfactory performance against progressive collapse. Therefore, it is evident that for structures designed with Specific design criteria for earthquake (SMRF), the roofs need to be reanalyzed according to Unified Facilities Criteria 4-23-03 (Unified Facilities Criteria, 2010). The results of this study indicated that roofs are the most vulnerable structural elements to earthquake in non-seismic regions and hence, require retrofitting immediately. After provision of Tie Forces and additional stirrups as specified in Enhanced Local Resistance for OC II and design requirements option 1 according to UFC 4-23-03 (Unified Facilities Criteria, 2010), 15% additional reinforcement is required to meet the progressive collapse provisions compared to ACI 318-08 (ACI Committe 318,318) and International Building Code "IBC" (International Code Council, 2009).

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