International Journal for Research in Engineering Application & Management (IJREAM)
ISSN : 2454-9150 Vol-04, Issue-02, May 2018

Progressive collapse of RC framed structure Due to column loss scenario

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Abstract: Progressive collapse is the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it. The slew of high profile engineering calamities in the past decades has demonstrated the disastrous consequences of progressive collapse. This study is focused on investigating the response of the structures after the sudden column loss design scenarios. Critical columns are identified based on Demand Capacity Ratios (DCR). A reinforced concrete G+12 framed structure is analyzed for linear dynamic analysis according to General Services Administration (2016) guidelines and the critical columns are re-designed. Based on the response of the structure after the sudden column loss design scenario, various design parameters such as bending moment, axial force and DCRs are determined. The analytical results shows that following the imposed initial damage, the bending moments in columns increased 78 times whereas the axial force increased by 5 times in linear dynamic analysis. The entire G+12 RC framed structure is made resistant to progressive collapse by increasing the size of columns and the percentage increment of area for critical columns in linear dynamic analysis varies between56.25% to 86.5% of before change of section respectively.

Keywords* — *Demand capacity ratios, ETABS, column loss scenarios, progressive collapse, capacity curve, axial, GSA guidelines

I. INTRODUCTION

Progressive collapse is the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or disproportionately large part of it. Failure of one or more primary load carrying members cause overloading of nearby other structural member due to change of load pattern which ultimately leads to failure of the members. In design of the buildings mostly the dead load and live load are considered, depending on the location of the structure seismic, wind and snow loads may also be considered. In most of the cases the designed buildings experience only the types of loads mentioned above and some of them may be subjected to abnormal loadings such as pressure loads and impact loads for which the building is not designed for. These may include internal gas explosions, blast, vehicular collision and aircraft impact. Most of the buildings may be vulnerable to these abnormal loadings. Considering these aspects The General Services Administration and Department of Defense Unified Facilities Criteria have developed design guidelines to resist and prevent progressive collapse.

In this paper a reinforced concrete G+12 framed structure is analyzed to study the effect of column failures at various locations considering GSA guidelines [9]. The method of analysis will be linear dynamic analysis and column removal case will be as per GSA corner column removal, interior column removal and exterior column removal. Alternative path method is a method of transferring the forces through the loss of a load-bearing element. In this approach it restricts the acceptability of the abnormal loading conditions that would cause the provided level of damage. The advantage of this method is that it supports structural systems with ductility, continuity and energy consuming properties that are suitable in preventing the progressive collapse.

II. LITERATURE REVIEWS

Rupa Purasinghe *et al (2010)* **[1]** has presented various design procedures for progressive collapse analysis, both linear and non-linear procedures with respect to GSA and DoD guidelines and analyzed a nine storey building with rigid moment connections and pre-Northridge connections for linear and non-linear analyses respectively. He compared how the loadings vary for various procedures in consideration with column removal scenarios and observed that in linear procedures the maximum DCR value of the model didn't exceed allowable DCR value 2 as per GSA guidelines, Where as in Nonlinear procedures maximum plastic rotation did exceed allowable value 1.5 as per DoD guidelines [10] and stated that the structure is strong enough to resist rotational deformations, so that member hinges do not approach strength-degrading levels even

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ISSN : 2454-9150 Vol-04, Issue-02, May 2018

though the structure has modelled with pre-Northridge connections. He concluded that nine-story building has large column and beam sections which makes it resilient to progressive collapse.

S. M. Marjanishvili *et al* (2010) [2] has presented available design methods such as linear-elastic static; nonlinear static; linear-elastic dynamic; and nonlinear dynamic procedures with respect to GSA and DoD guidelines, as systematic analysis for progressive collapse is not available. He discussed how various loading conditions to be incorporated for analyzing progressive collapse in a structure with respect to GSA and DoD guidelines and compared the advantages & disadvantages of the available analysis methods. He evaluated a new analysis method which progresses from simple linear elastic static analysis to complex nonlinear time history analysis known as Progressive Analysis Method by incorporating the advantageous parts of all the four procedures by systematically applying increasingly comprehensive analysis procedures to confirm that the occurrence of progressive collapse is high.

III. MODELLING

A. Modelling in ETABS

Analyses have been performed using ETABS, which is a structural analysis program used for static and dynamic analyses of building structures. In this study, ETABS 2016 Version 16.2.0 has been used. A description of the modelling details is provided in the below sections. A three-dimensional model of the building structure is created in ETABS [12] to carry out linear dynamic analysis. Beam and column elements are modelled as rectangular framed elements with material properties and section properties as mentioned in section III –*B*. And slab section is considered as membrane section with 150 mm thickness. The structure is loaded as mentioned in III-C accordingly and load combinations are predefined for carrying analysis

The structure is analysed for linear dynamic analysis by creating response spectrum function. Response-spectrum analysis (RSA) is a method in which the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure is measured. Response-spectrum analysis provides insight into dynamic behavior by measuring acceleration. The structure is subjected for loadings such as gravity loadings, seismic loadings, wind loadings and response spectrum loadings as mentioned in below sections and carried out the analysis.

The concrete frame design is performed for the structure and all the elements are checked to observe whether all the structural elements are below the failure limit (Demand Capacity Ratios of the elements have been checked and all the DCR's are less than 1).

B. Building Configuration

The Details of the building model (G + 12) is shown in figure 1 and figure 2, with individual story height of 3m is considered for the study. The total height of the building considered is 39m. The building considered also has vertical irregularity. All column and beam sections are modelled as rectangular shape elements using frame elements and the slab section is modelled as membrane type. Section properties and material properties are as mentioned as below.

(a) Material Properties

- Concrete

  Grade of Concrete, f_{ck} = M25
  Poisons ratio = 0.2
  Density =25 kN/m^3
  Modulus of Elasticity = 25000 MPa

- Steel

  Yield Stress, f_y = 500 MPa
  Modulus of Elasticity = 2 x 10^5 MPa

(b) Section Properties

- Different column sizes

15"X30"	12"X30"
24"X24"	12"X36"
15"X36"	18"X30"

- Different beam sizes

12"X30"	12"X24"
9"X18"	9"X12"

(c) Thickness of slab = 150 mm

C. Loadings

Primary loading considered on the building for the study are as:

(a) Gravity loading**:**

Dead load: Self weight of the structural elements
Live load at typical floor: 3.0 (kN/m^2)
Live load at terrace floor: 2.0 (kN/m^2)
Floor finish floor: 1 (kN/m^2)
Wall load: 8 (kN/m)

(b) Seismic Loading:

As per IS 1893(Part 1):2002 the structure is located in Zone III with site type II (Medium soil).
Zone factor, Z = 0.16
Importance Factor, I = 1
Response Reduction Factor, R = 5

(c) Wind loading**:**

The lateral wind loads are considered as per IS875:1987 with Structure class C and terrain category 3
Basic Wind Speed, V_b = 55 m/s
Windward Coefficient, $C_{p,wind}$ =0.8

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Leeward Coefficient, $C_{p,lee}$ =0.5
Risk Coefficient, k_1 =1
Topography Factor, k_3 =1

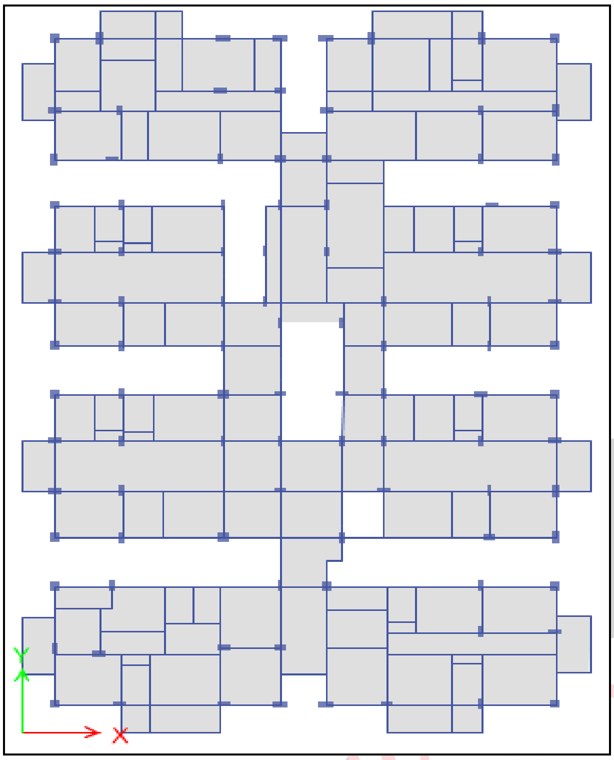

Figure 1: Plan showing beam layout of the building in ETABS

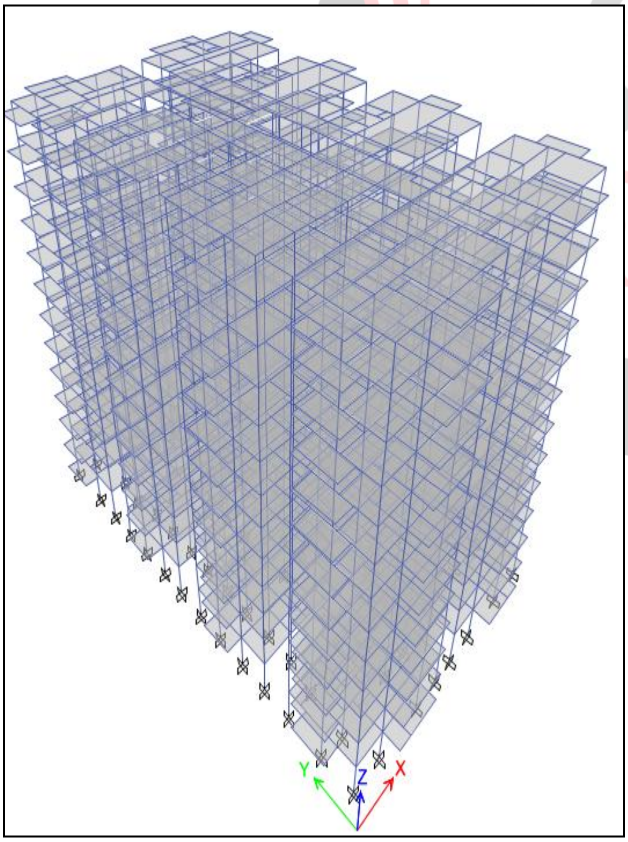

Figure 2: 3-D view of the structure in ETABS

IV. METHODOLOGY

The structure is designed to resist progressive collapse using dynamic analysis according to GSA guidelines by using ETABS software. The analyses are carried out to determine the potential for progressive collapse when it is subjected to the instantaneous removal of a primary vertical element. The assessment of the potential for progressive collapse using the results of analysis is achieved by using the acceptance criteria in the form of appropriate Demand-Capacity Ratios. DCR ratio are determined for all the columns and according to GSA guidelines columns with DCR ratio's greater than 2 are identified as critical columns. The critical columns are redesigned using enhanced local resistance method by increasing flexural and shear capacity of the columns.

A. Linear dynamic progressive collapse analysis:

Analyze G+12 model as shown in figure 1 using ETABS software considering lateral forces and response spectrum function. Perform concrete design and determine the reinforcement to be provided in members. Create column loss scenario by removing ground floor column from the specified location one at a time as shown in figure 3.Apply the dynamic load combinations as per GSA 2016 guidelines [9].Perform response spectrum analysis considering acceleration as load type and response spectrum function. Evaluate the results based on demand-to-capacity ratio (DCR), where demand is taken as the peak value of response from the calculated response spectrum analysis.

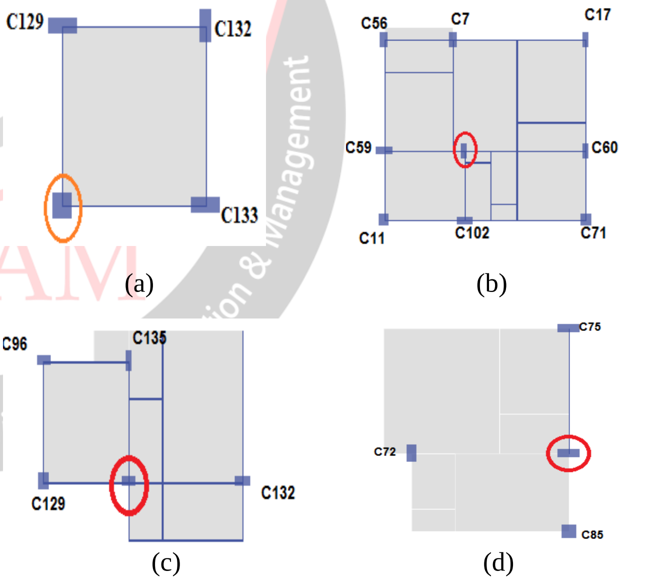

Figure 3: Column Removal Scenarios for LDC, LDI, LDES and LDEL models respectively

B. Analysis Loading

Increased Gravity Loads for Floor Areas Above Removed Column or Wall.

GLF = ΩLF [1.2 D + (0.5 L or 0.2 S)] Equ - (1)

GLF = Increased gravity loads for deformation- controlled actions for Linear Static Analysis

D = Dead load including façade loads (lb/ft² or kN/m²)

L = Live load including live load reduction per Section (lb/ft² or kN/m²)

S = Snow load (lb/ft² or kN/m²)

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ISSN : 2454-9150 Vol-04, Issue-02, May 2018

ΩLF = Load increase factor for calculating force-controlled actions for Linear Static analysis=2

For other structural elements in the static analyses, the load combinations are

Gravity Loads for Floor Areas Away From Removed Column or Wall.

$G = [1.2\ D + (0.5\ L \text{ or } 0.2\ S)]$ Equ - (2)

Whereas in case of Linear Dynamic Analysis apply the loading as mentioned in equation 2 to the entire structure.

C. DCR Limitation

The magnitude and distribution of of potential demands on both the primary and secondary structural elements will be indicated by Demand-Capacity Ratios (DCR). Acceptance criteria for the primary and secondary structural components shall be determined as:

DCR = QUD / QCE

QUD = Acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces),

QCE = Expected ultimate, unfactored capacity of the component.

According to GSA 2016 guidelines the allowable DCR values for primary and secondary structural elements should not exceed 2.0 for both regular and irregular geometrical structures

V. RESULTS AND DISCUSSIONS

Increase in bending moment for beams after column removal and increase in axial force for critical columns for Different column removal scenarios are observed in table 1. For resisting progressive collapse critical columns are redesigned by increasing the section size and strength of concrete for some columns have increased from M25 to M35 as shown in Table 2. The entire G+12 RC framed structure is made resistant to progressive collapse by increasing the size of columns and the percentage increment of area of critical columns.

Table 1: Change in Bending Moment & Axial Force for LD Models

Column Removal scenario	Parameter	Element	Before removal	After removal	Increased by
LDC	Bending moment, kNm	B548	46.24	1113.97	24.1
	Axial Force, kN	C133	5607.90	13261.42	2.37
		C132	3962.09	10166.79	2.57
LDI	Bending moment, kNm	B630	166.70	987.63	5.93
		B638	341.14	1251.30	3.67
	Axial Force, kN	C56	4863.08	13973.43	2.88
		C7	5712.81	18683.24	3.28
		C60	3627.87	12225.00	3.37
		C71	4190.89	14641.63	3.5

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Column Removal scenario	Parameter	Element	Before removal	After removal	Increased by
LDEL	Bending moment, kNm	B247	64.92	2044.19	31.49
		B645	22.01	1712.76	77.82
	Axial Force, kN	C75	4337.34	16004.19	3.69
		C85	5911.83	14797.12	2.51
LDES	Bending moment, kNm	B548	46.62	1576.22	33.81
		B547	128.99	1277.55	9.91
	Axial Force, Kn	C132	5884.57	14873.31	2.53
		C96	4575.29	13272.93	2.91

Table 2: Increased Critical Column Sizes for LD Model

Column Removal scenario	Element	Section size		DCR	
		Before change	After change	Before change	After change
LDC	C133	36”X15”	36”X24”	2.074	1.391
	C132	15”X30”	24”X30”	2.206	1.424
LDI	C56	36"X15"	36"X24"	2.105	1.388
	C7	15"X30"	36"X24"	3.171	1.859
	C60	15"X30"	18"X30"	2.205	1.615
	C71	24"X24"	30"X30"	2.199	1.433
LDEL	C75	24"X24"	30"X30"	2.444	1.869
	C85	36"X15"	30"X36"	2.325	1.359
LDES	C132	15"x30"	24"X30"	2.543	1.953
	C96	24"x24"	30"X30"	2.018	1.664

To mitigate progressive collapse of beams and columns caused by failure of critical column an alternate load path is provided. An alternate load path can be adopted by increasing size of critical columns, increment of reinforcement in critical columns and even by increasing strength of concrete to avoid the progressive failure. So the critical columns whose DCR values are more than acceptance criteria value according to GSA guidelines [9] have been redesigned such that the load of the critical column is redistributed to the adjacent columns and DCR’s of those columns are less than 2.

A. Capacity Curves

Beam and column sections with DCRs larger than the 2.0 are replaced with inserted hinges to simulate the inelastic response of the column-removed building under vertical downward loadings. Apparently, elastic-perfectly plastic models are assumed for the inserted hinges in the GSA linear procedure [9]. Progressive collapse resistance of the column-removed building is obtained by performing a series of LS analysis with gradually increased multiplier of (1.2DL + 0·25 LL). The progressive collapse resistance of the building subjected to sudden column loss is approximated to 2.5, 3.0, 3.25, 3.5 and 4.0 times of (1.2DL + 0·25 LL).

When the applied load exceeds the collapse resistance, a local flexural failure mechanism is formed and the following analyses result in progressive collapse. The number of inserted hinge usually increases with the applied Loading and the load-displacement responses obtained from the incremental LD analyses for the four conditions. The abscissa is the displacement of the column-removed

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ISSN : 2454-9150 Vol-04, Issue-02, May 2018

point. The ordinate is the loading magnitude expressed in terms of (DL + 0•25 LL) as shown in figure 4.

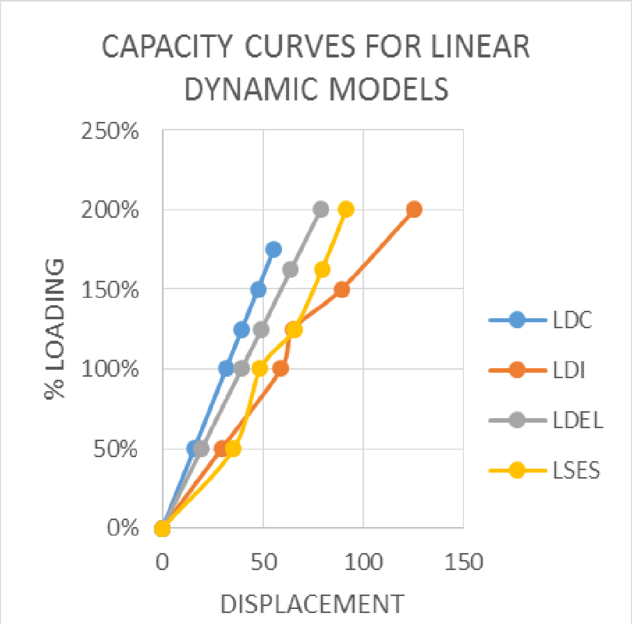

Figure 4: Combined Capacity Curves for Linear Dynamic model

VI. CONCLUSIONS

In Linear dynamic analysis two critical columns are observed for LDC, LDEL and LDES models where as in case of LDI model four critical columns are observed. So in case of linear dynamic analysis the worst case is observed in LDI scenario in which four columns are being identified as critical in which the DCR ratios are exceeding 2.0.In linear dynamic analysis the maximum increase in bending moments and axial force for columns after column removal scenario is 78 times and 5 times respectively. Critical condition observed is interior column removal scenario.

So the G+12 RC framed structure is made resistant to progressive collapse by increasing the size of columns and the percentage increment of area of critical columns varies between56.25% to 86.5% with respect to linear dynamic analysis. The DCR values of the critical columns after redesigning, are between0.389 to 1.901 which is less than '2'. Hence the flexural and shear capacity of columns have been increased to mitigate progressive collapse of beams and columns caused by failure of particular column an alternate load path is provided by increasing size of critical columns, increment of reinforcement in critical columns and by increasing strength of concrete to avoid the progressive failure. So the critical columns whose DCR values are more than acceptance criteria value suggested by GSA have been redesigned such that the load of the failure column is redistributed to the adjacent columns and DCR's of those columns are less than 2.

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