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An analysis of a high-rise building's progressive collapse if four of its exterior columns were removed

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Abstract— *Progressive failure is the partial or complete failure of a structure as a result of a local failure occurring in the load-bearing element, resulting in failure propagating from one element to another. A progressive collapse study of a real ten-story telecommunications building was performed using ETABS (V.23) software, and RC ACI Code was used to design the building. The demand-to-capacity ratio is analyzed according to GSA acceptance standards to evaluate the ability of the structure to transfer loads to nearby members. Four central columns on the sixth and seventh floors were removed. The examination included a failure scenario resulting from the removal of an explosion on the sixth and seventh floors of the building. The results of the study found that removing four supports on the two floors did not cause gradual collapse and that the redistribution after removing the column was equal. The shear and bending DCR values for column loss are lower than 1, according to GSA 2016. As a result of load redistribution, the nearby column receives compressive strains as the supports above the deleted one lose axial compressive pressures. The weight on the sixth level was moved to columns C1A and C4A at grid A, while columns C2A and C3A were removed. This transfer was twice as great as the load communicated before the columns were removed. This implies that the adjacent columns were large enough to support additional loads. Since DCR readings were less than the permitted limits at 1, the beams were acceptable in flexure, shear, and DCR readings for column axial load.*

Keywords— ***Progressive Collapse Analysis, Sequential Column Removal, Reinforced Concrete Frames, Alternate Path Analysis, Column Failure.***

I. INTRODUCTION

In recent years, growing attention has been paid to structural failure mechanisms, particularly progressive collapse. Progressive collapse refers to the situation whereby a structure can experience partial or complete collapse due to the failure of a load-bearing element. This failure in the final condition is much more severe than it would be in the starting condition, which is what makes it progressive (Ellingwood et al., 2007). The progressive collapse of a building structure can begin when one or more of the vertical load-bearing members are removed. This can happen due to man-made or natural hazards. The 1968 partial collapse of the 22-story Ronan Point apartment complex in London, United Kingdom, brought this phenomenon to the attention of scientists. A gas explosion in the 18th-floor kitchen led to the collapse of part of the building. Since then, the roots of progressive failure have been studied more comprehensively after the terrorist acts of September 11, 2001, and new standards incorporating these concepts were created (Joshi et al, 2010).

progressive collapse of a structure can be caused by a variety of abnormalities, including aircraft collisions, design or construction errors, fires, gas explosions, unintended overloads, hazardous materials, automobile accidents, bomb explosions, and others (Burnett, 1975). To prevent catastrophic failure, one of the widely used methods is the alternate load path method, which involves designing backup load paths in case a component of the structure fails. This approach takes into account the structural reaction after the initial failure rather than the cause of the initial local failure, making it a threat-independent methodology. It has been observed that the alternate load path method is used in the majority of published tests on progressive collapse, especially in cases involving sudden loss of columns (Usmani et al, (2003), Fu, (2009), Tavakoli and Kiakojouri, (2013)), SAP2000 (Marjanishvili, (2004), Bae et al, (2008), Kim and Lee, (2010)), and Opensees (Kim and Kim, (2009), Talaat and Mosalam, (2009), Kim et al, (2010)). Nonlinear FE packages like Abaqus are widely used in numerical analyses conducted for progressive collapse. Several references offer a thorough example of 3D finite element modeling. (Tsai and Huang, 2011) studied the impact of three various types of walls on resistance to progressive collapse. They investigated the structure with ten floors developed to GSA specifications. If the center column is on the transverse side, the middle column is on the longitudinal side, or the edge column is on the short side, they are removed from the building. The research included both linear and nonlinear static analyses. This study only looks at flexural capacity. According to the study, walls increase the building's stiffness and strength and increase its resistance to progressive collapse. (Jeyanthi and Kumar, 2016) evaluated the multi-story RC structure with column removal to study progressive collapse. The structure consisted of an eight-story educational building designed according to the Indian Building Code. The crucial columns were identified and removed as per GSA, initiating the progressive collapse. A comparison between the parameters before and after the progressive collapse was carried out. Based on the analysis conducted using the ETABS software, the findings of the study revealed that if the corner column breaks, the probability of progressive collapse increases. It was also observed that the beams adjacent to the missing column had the highest bending moment in comparison to the beams farther from the damaged column junction.

(Raghavendra et al, 2014) analyzed a symmetric ten-story RC structure using linear analysis by ETABS software. To assess progressive collapse according to the guidelines of the GSA, four critical columns have been removed: a long-side exterior column, a short-side external column, a corner column, and an inside column. It was found that the structure had a low potential to resist progressive collapse. Therefore, two approaches were proposed to mitigate the collapse of DCR values. Providing a bracing on the top story of the structure and providing a bracing on the side face of the structure. In this paper, a ten-story building is considered, to study the effect of multi-column failure on the building. The evaluation of progressive collapse for a typical designed building is carried out. Progressive collapse analysis is performed using ETABS (v.23), and the linear analysis procedure of the GSA guidelines is followed.

II. ANALYTICAL WORK

A. Analysis procedure and acceptance criteria as per GSA guidelines

To determine the capability of a structure to resist abnormal loadings and to evaluate the potential for progressive collapse. Firstly, the reinforced framed structure is designed using ETABS for dead and live loads, according to ACI 318-19. For progressive collapse analysis, column C2 is removed. One case has been considered: Analyze the loss of a column for the sixth story located near the corner of the building (interior removal), as shown in Figure 1.

The requirements adopt a technique that involves estimating demand capacity ratios for building components, comparing DCR values to design strengths, and estimating the threat of gradual collapse. The outcomes of the linear analysis are presented to predict the locations of future failures by figuring out the amount as well as the distribution of expected loads on main and minor structural components. The following formula is used to compute DCR values:

DCR = QUD /QUF

QUD stands for active force (demand), such as moment, shear, and axial force. QCE signifies the expected ultimate moment, shear, and axial force. DCR (demand-capacity ratios) will show the volume and distribution of these demands. To demonstrate if structural elements and connections have sustained significant damage or collapsed, the linear elastic approach is employed. In standard structural configurations, DCR values exceeding 1 indicate that the structure is destroyed or broken down.

B. Load Combinations

There are two load combinations for the linear analysis sort out, two cases: the first instance, which is specified in equation (1) to compute the QUD for the deformation controlled, and the second case, which is defined in equation (2) to calculate the force controlled according to GSA (GSA,2016) recommendations:

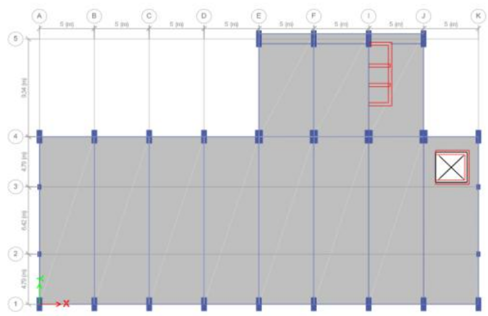

Fig.1: Building plan with column removal locations on the sixth floor.

GLD=ΩLD [1.2D+0.5 L] …(1)

GLF=ΩLF [1.2D+0.5 L] … (2)

G=1.2D+0.5L …………… (3)

The third equation (3) is used in both cases for members not exposed to the elimination effect, as indicated in Figure (1).

Where the dead and live factors remain the same and consistent for both load combinations in Equations (1) and (2).

Table (1) illustrates how the method of determining the load of these load increase factors, ΩLD and ΩLF. ΩLF is 2.0 for RC-framed buildings, and ΩLD is determined from a formula accounting for the structure's mLIF factor, which is the small value of m.

Where m is calculated according to GSA (GSA,2016). There ΩLD and GLD are applied to the load combination on Equations (1) and (2) around the tributary areas of the removed columns. Then gravity loads G in Equation (3) was applied to the slab regions that weren't loaded with ΩLD and ΩLD that weren't near the removed columns.

Table (5-1) Factors of loading increase in linear analysis (GSA,2016)

Material	Structure Type	Ω_{LD}, Deformation-controlled	Ω_{LF}, Force-controlled
Steel	Framed	0.9 m_{LIF} + 1.1	2.0
Reinforced Concrete	Framed[A]	1.2 m_{LIF} + 0.80	2.0
	Load-bearing Wall	2.0 m_{LIF}	2.0
Masonry	Load-bearing Wall	2.0 m_{LIF}	2.0
Wood	Load-bearing Wall	2.0 m_{LIF}	2.0
Cold-formed Steel	Load-bearing Wall	2.0 m_{LIF}	2.0

Verify that, deformation is controlled in all components:

Φ m QCE > QUD …………(4)

where, from the static linear model, M is part or component demand modifier of the values defined in Table (7) of GSA (GSA,2016)

Φ = strength reduction factor, for flexural(0.9), shear (0.75), and axial force(0.65).

QCE is expected for component or element strength.

Verify the following to force-controlled movements in all components, check that:

Φ QCL > QUF (5)

where QUF is force controlled

QCL is the required strength of an elements

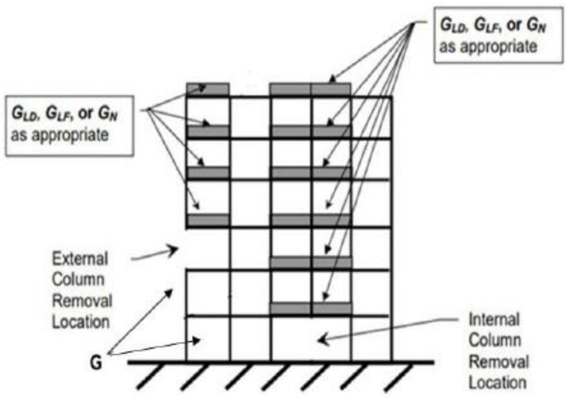

Fig.2. Sites of load for Linear Static Analysis (GSA,2016)

III. MODELING OF BUILDING

A. Building Description

The ten-story reinforced concrete building with a basement was tested using the ETABS (v.23) program. The floors of the structure vary in height, as shown in Table 2. The slabs are built of 160-mm-thick boards, and Figure 3 depicts a cross-section of the beam and column. depicts in Table 3 the dimensions of the columns. All supports were planned as fixed supports. the study was done using linear static analysis. Concrete's compressive strength (fc') is 30 $\frac{N}{mm^2}$ whereas steel's yield strength (fy) is 420 $\frac{N}{mm^2}$. The structure's shape and gravity loads were tested by ACI Code 318-19. The load of the wall on the beams was different according to the height of the floor, as shown in Table 4.Live Load: Obtained from ACI 318-19 on the roof 1 $\frac{KN}{m^2}$.; on floor 4.79 $\frac{KN}{m^2}$ [19].

Table 2 stories' height.

Story	**Height** (m)
the basement	4.35
1^{st} and 2^{nd} story	3.50
3^{rd} to the 7^{th} stories	5.7
8^{th} to the 10^{th} *stories*	4.7

Table 3 Displays the Column dimensions.

Columns number	**Dimensions (mm)**
C1	1300 x 500
C2	350 x 500

Table 4 Characteristics of the wall.

Material	Thickness (cm)	Clear Storey height (m)	Unit weight $\frac{kN}{m^3}$	Partitions load $\frac{kN}{m}$
Brick	25	3.5	18	15. 75
		5.7		25.65
		4.7		21.15

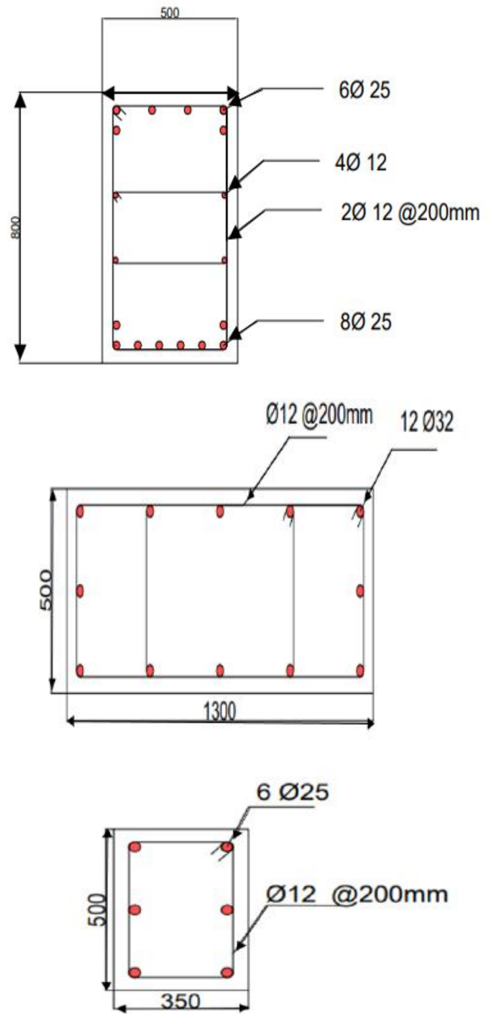

Fig.3: Details of beam and columns reinforcement.

IV. RESULTS AND DISCUSSION

In this scenario, the edge columns towards the center of the short side, represented by C2A and C3A on the sixth and seventh floors, as well as the brick wall in grid A of these floors, have been removed. After removing an edge column, the internal forces (i.e., demands) in each beam were calculated using three-dimensional analysis. In this case, perimeter beams along grid line A (between grid lines A1 and A4) provide the flexure mechanism for beams over the removed column. Figure (4) shows moment diagrams for the beams along grid line A throughout the whole height of the construction. Maximum demands are compared to the available design strengths of the beams. Figure F indicates the maximum bending moments of the beams and shows the DCR values, which are calculated by dividing the moment demands by the design moment strengths. DCR values were calculated only for the ends of the side beams because of their connection to the joint subjected to column removal.

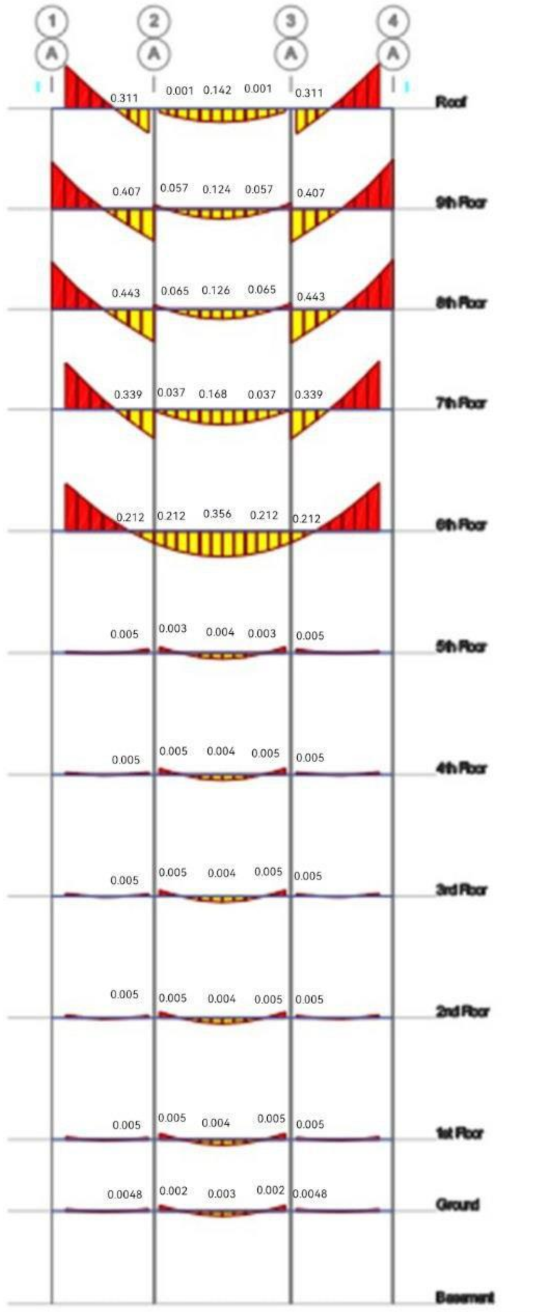

Fig.4: Moment diagram of beams and DCR values at grid A

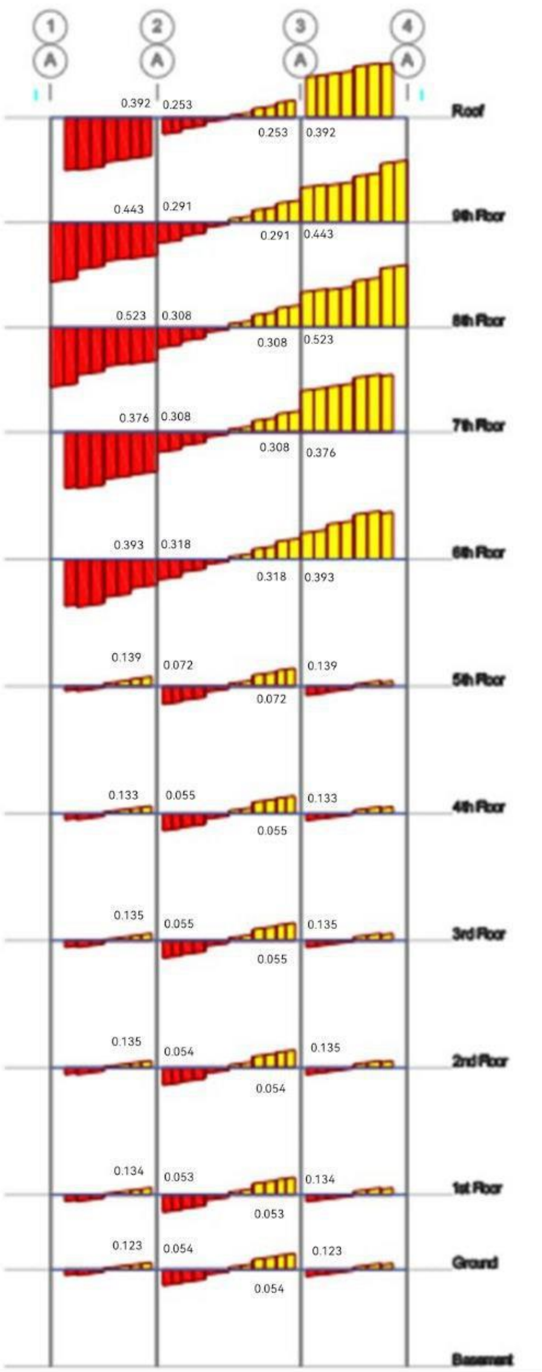

Fig.5: Shear diagram of the beam and DCR values at grid lines A

After columns C2 A and C3 A were removed, the A2 and A3 ends of beams B 1-2 A and B 3-4 A began moving downward due to tension force. This deformation added a positive moment to joint A3 and A2 ends, whereas the other end remained relatively steady at joint A1 and A4 ends. Bending moment distributions along grid line A vary across grid lines A3 and A2. This is due to the variation in beam span lengths between beams B 2-3A and B 3-4A. Note that the lower floors are subjected to a load less than the upper floors due to the different load combinations that were applied to them, so the load on the first six floors is less compared to the upper floors. and because the structure complies with the acceptance criteria restrictions of linear static analysis, it is not prone to progressive collapse failure. The results of the maximum shear DCR value for beams B 1-2 A, B 3-4 A, and B 2-3 A do not exceed the limits set by linear statics; they reach 0.739 on the eighth floor and 0.318 on the sixth floor. Based on these results, one can conclude that the structure does not pose a risk of gradual collapse failure in the second case because the DCR doesn't exceed the restrictions of the acceptance criteria for linear stability. Max shear distribution and DCR values at grid A are shown in Figure 5.

Fig.6: Axial load for columns after columns were removed and DCR values at grid line A

The results of the maximum shear DCR value for columns and the axial load distribution of the columns that remained on the A-axis are depicted in Figure 6. Which was provided by calculating the demand and capacity of each structural member. The maximum DCR for all members who meet the acceptance criteria is less than one. The results of DCR values for the axially loaded columns C2A and C3A on the ground floor do not exceed the limits set by linear statics; they reach 0.442, and on the sixth floor, C1A and C4A reach 0.36. It can be seen that the loads transferred from C2A and C3A to the two side columns increased compared to the load before removal. This is due to the dynamic influence factor present in the load equation, meaning that the load has doubled in addition to the load that was transferred to columns C1A and C4A from the upper middle.

V. CONCLUSION

The possibility of the progressive collapse of the Telecommunications Building, which was built and determined by the ACI Code (318-1965)[19], was studied. The probability of progressive collapse was studied in this research using the ETAB 2023 computer program and linear static analysis methods described in the 2016 GSA recommendations for RC structures. Evaluating the DCR at each step requires analyzing the structure such that the DCR of any member does not exceed a specified limit state. The results of the study may lead to the following conclusions:

1. The largest axial DCR value in columns in this case (the elimination of four exterior columns on the sixth and seventh floors, on edge columns C2A and C3 A was 0.436 on the ground floor. In contrast, the DCR value for edge columns C1A and C4A was 0.36 on the sixth floor. This explains why the neighboring columns' size was big enough to carry extra loads. The study concluded that eliminating four critical supports at levels six and seven does not lead to progressive collapse.
2. Because the re-distribution following column removal is significantly more equal in the central column scenario, the largest DCR values for shear and flexure for the center column lost in the remove scenario were equal. The largest Flexure DCR values for beams B 1-2 A and B 3-4 A in edge were (0.4437 and 0.4432) on the eighth floor. and the largest Flexure DCR value for B 2-3 A was (0.356) in mid-span on the sixth floor.
3. At the same time, the largest shear DCR values for B 1-2 A and B 3-4 A in edge were (0.739) on the eighth floor. and the largest shear DCR value for B 2-3 A was (0.318) on the sixth floor in the second case.
4. As a result of load redistribution, the nearby column receives compressive strains as the supports above the deleted one lose axial compressive pressures. The weight on the sixth level was moved to columns C1 and C4 at grid A, while columns C2 and C3 were removed. This transfer was twice as great as the load communicated before the columns were removed. This implies that the adjacent columns were large enough to support additional loads.
5. Because the DCR values were less than 1, the spanned beams above the removed column did not shear in every instance. Since DCR readings were less than the permitted limits at 1, the beams were acceptable in flexure , shear, and DCR readings for column axial load.

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