Progressive Collapse of Reinforced Concrete Buildings: A Study on GF Column Elimination

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ABSTRACT

Progressive collapse arises when local failure in a building member spreads to the adjacent members, this may promote further failure. In general, structures are designed to bear the normal expected loads like dead load, live load and lateral load (wind and seismic). However, some structures rarely are being exposed to sudden loads due to natural, man-made, intentional or unintentional reasons. These unexpected loads induce progressive collapse event. Therefore, many studies have been conducted to improve the building performance against extreme load hazards and progressive collapse phenomenon.

In this work various position of GF column is selected and removed floor wise. As a result, the prime objective of this study is to find out the most critical location of the removed vertical-support element. Additionally, Linear static analysis of the three-dimensional (3-D) computer models of each selected building was carried out by using STAAD. Pro program. Ultimately, observations from this research demonstrate that the increase in the height of the structure and the removal of column from the bottom or near the bottom of the short side of the building is more significant to progressive collapse event.

KEYWORDS: Progressive collapse, local, loads, column, building.

1. INTRODUCTION

The progressive collapse of building frame is being when one or more vertical load carrying elements (typically columns) is removed. Once a column is removed due to a vehicle impact, fire, earthquake, or other man-made or natural hazards, the building's mass (gravity load) transfers to neighbouring columns in the structure. If these columns are not well designed to resist and share the additional gravity load, that part of the structure fails. The vertical load carrying elements of the structure continue to fail until the additional loading is stabilized. As a result, a generous part of the building frame may collapse, causing greater harm to the building frame than the initial impact.

Column was removed, analyzed and compared with the analysis results from the computer program STAAD. Pro V8i. The Structural Analysis Program STAAD. Pro V8i is a powerful computer program used to design and analyze various structures. The program analyzes two dimensional linear static models to three dimensional nonlinear dynamic models. The load distributions and bending moments generated from each column removal are calculated and compared in the STAAD. Pro V8i computer simulation. This research study analyzes the data collected in the field and compares it to the STAAD. Pro V8i simulation results.

The focus of this research is to determine if a structure is susceptible to progressive collapse. The hypothetical conditions are generated and analyzed, compared with the results from STAAD.Pro V8i computer model of the building. Using STAAD. Pro V8i, the structure's potential for progressive collapse was determined.

Progressive collapse of structures refers to local damage due to occasional and abnormal events such as gas explosions, bomb attacks and vehicular collisions. The local damage causes a succeeding chain reaction mechanism spreading throughout the entire structure, which in turn leads to a terrible collapse. In general, the size of resulting collapse is disproportionate with the triggering event. Progressive collapse may be concluded in 2 outcomes either partial collapse or global collapse. Moreover, the ratio of total destroyed volume or area to the volume or area damaged by the originated event could be defined as the degree of progressivity in a collapse.

1.2 Progressive collapse

Progressive collapse is a situation in which a local failure in a structure leads to load redistribution, resulting in an overall damage to an extent disproportionate to the initial triggering event. While the disproportionate collapse is associated with local failure of a structural component leading to the total failure of the entire structure or a significant portion of the structure, that is, the extent of final failure is not proportional to the original local failure. An example for this sort of collapse, the failure of a single column in a frame system due to an abnormal event leads to a chain reaction of subsequent failures for the adjoining components resulting in the entire collapse of the building.

1.3 Ronan point apartment

The collapse of the Ronan Point apartment could be considered as the first well-known and the most publicized example of progressive collapse. The Ronan Point tower was a multi-story residential building consisted of 22 stories located in Newham, East London, United Kingdom constructed between July, 1966 and March, 1968. The overall dimensions of the plan were 24.4m by 18.3m and the total height of the apartment was 64m. It was easy to be built since the structural flat plate floor system contained precast concrete for the walls, floors and staircases. The walls and floors were bolted together and the connections were filled with dry packed mortar. This means that the floors did not have a high potential to withstand bending, especially if overhanged, so that each floor was supported directly by the walls in the lower story.

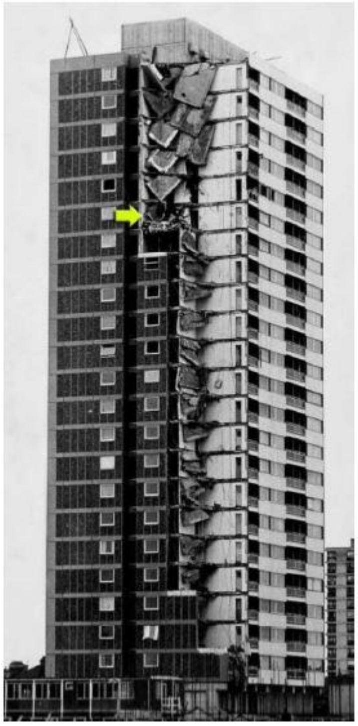

Fig. 1.1. The Ronan Point Apartment, London, UK

It is obvious in (Fig. 1.2) that the eventual result of the very moderate gas explosion was the collapse of the corner bay for the full height of the tower (entire collapse of the southwest corner). The consequences of the partial collapse of the 22-storey precast Ronan Point partment were a building bereft of one of its corners besides four dead residents and seventeen injured but the tenant of the flat number eighteen Mrs. Hodge who triggered the incident survived.

Despite the truth that the partial collapse of the Ronan Point tower in London, England in 1968 was not categorized as one of the biggest buildings disasters of recent years, it was such a shocking accident because the extent of the failure was absolutely out of case was of the order of 20.

It should be stated that the wall system was designed only to withstand the extreme wind pressure; hence the continuity in the vertical load path was lost for the upper floors. The collapse was attributed to the lack of structural integrity, mainly in terms of redundancy and local resistance. In other words, the structural system was not designed to provide alternate load path to redistribute the stresses. Another reason of this disproportionate collapse was the building had been constructed with very poor workmanship, and thus its overall structural robustness was considerably compromised.

Further investigations in this collapse reported that stronger interconnection amongst the structural elements is the key for such kind of facilities where this improvement in the connections between the wall panels and floors is likely to have great reduction in the damage scale of the Ronan Point apartment.

Ultimately, the building was demolished in 1986 in the last century due to safety concern.

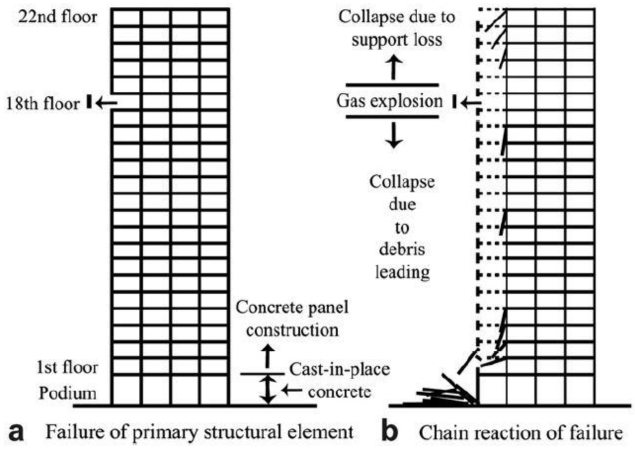

Fig. 1.2: Progressive Collapse of Ronan Apartment

2. LITERATURE REIVEW

Bibiana Luccioni et. al. (2002) analysed the structural failure of a reinforced concrete building caused by a blast load is presented in this paper. All the process from the detonation of the explosive charge to the complete demolition, including the propagation of the blast wave andits interaction with the structure is reproduced. The analysis was carried out with a hydrocode. The problem analysed corresponds to an actual building that has suffered a terrorist attack. The paper includes comparisons with photographs of the real damage produced by the explosive charge that validates all the simulation procedure.

G.N.Narule et. al . (2015) explained a building undergoes progressive collapse when a primary structural element fails, resulting in the failure damage is disproportionate to the original cause, so the term disproportionate collapse is also used to describe this collapse type. Progressive collapse can be triggered by manmade, natural, intentional, or unintentional causes. Explosion, fires, earthquakes creates large amounts of stresses and the failure of supporting structural members can lead to a progressive collapse failure. Progressive collapse is a complicated dynamic process where the collapsing system redistributes the loads in order to prevent the loss of critical structural members. For this reason beams, columns, and frame connections must be designed in a way to handle the potential redistribution of large loads. This research provides insight into the structural configuration to achieve a demand to capacity ratio of appropriate quantity and prevent collapse in the event of a single column loss. Several relationships developed between various analysis procedure against shear forces. Ultimately, all this information can be used in design codes where there are currently very limited or no specific rules or guidelines.

Alexander M Remennikov (2003) describe the explosive devices have become the weapon of choice for the majority of terrorist attacks. Such factors as the accessibility of information on the construction of bomb devices, relative ease of manufacturing, mobility and portability, coupled with significant property damage and injuries, are responsible for significant increase in bomb attacks all over the world. In most of cases, structural damage and the glass hazard have been major contributors to death and injury for the targeted buildings. Following the events of September 11, 2001, the so-called “icon buildings” are perceived to be attractive targets for possible terrorist attacks. Research into methods for protecting buildings against such bomb attacks is required. Several analysis methods available to predict the loads from a high explosive blast on buildings are examined. Analytical and numerical techniques are presented and the results obtained by different methods are compared. A number of examples are given.

Aditya Kumar Singh et. al. (2014) acknowledged that in today’s world terrorists’ attacks are common and not a single country is completely safe. High-explosive detonations propagate blast energy in all directions, causing extensive damage to both the target structure and nearby buildings. Structural damage and the glass exposure have been major contributors to death and injury for the targeted buildings. If the structures are properly designed for these abnormal loads damage can be controlled. Within the Indian Standard Codes these types of situations are not dealt with and they need further explanation as the engineers have no guidelines on how to design or evaluate structures for the blast phenomenon for which a detailed understanding of structural behavior as well as effects of different kinds of blast load is required. In this paper, an attempt has been made to review various loading which can occur during a blasti.e, the dynamic impact loading, varying rate concentrated loading &transverse blast loading and the methods applied to analyze those loading phenomenai.e Single

Degree of Freedom (SDOF) model, Finite Element Model (FEM) & non-linear dynamic analysis. Based on the results obtained by different methodsa comparison has been made and the suitability is discussed.

G. P. Deshmukh et. al. (2016) Explosives are very severe problem all over the world due to terrorist's activities. Their aim is to destroy the place where there is large amount of rush. As the shopping malls are having large amount of rush daily so such type of the building should be blast resistant. We are going to study the behavior of shopping mall against blast load. A comparative study is carried out using ETABS software with different weight of TNT explosive using time history calculations and then finally it is concluded that what is the exact difference in the effect of two different weights of TNT explosives. Different parameters are compared using graphical and tabular form.

Ganainy and El Naggar (2009) studied the seismic performance of moment-resisting frame steel structure with multiple underground stories resting on shallow footings. A parametric study that involved evaluating the nonlinear seismic response of five, ten and fifteen storey moment-resisting frame steel structures resting on flexible ground surface, and buildings having one, three and five underground stories was performed.

Mehmet Inelet. al (2008) analysed that over the past two decades, Turkey has been hit by several moderate to large earthquakes that effect in significant loss of life and property. A remarkable number of casualties and heavily damaged or collapsed structures has emphasized inadequate seismic performance of multistorey reinforced concrete buildings, typically three to seven stories in height. This study focus to evaluate seismic performance of the most common reinforced concrete structure stock in Turkey considering nonlinear behavior of the components. A sample building set is selected to reflect existing project construction practice; regular structures and structures with irregularities such as soft storey, short columns, heavy overhangs and soft storey with heavy overhangs. Ductile and non-ductile details are taken into account by transverse reinforcement amount. The capacity curves of the investigated building set are determined by pushover analysis conducted in two principal directions. Inelastic dynamic characteristics are represented by equivalent single-degree-of-freedom (SDOF) systems

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evaluation was accomplish in accordance with the recently published Turkish Earthquake Code (2006) that has equality with FEMA-356 guidelines. Analytical damage evaluation in this study has shown that the seismic impact of earthquakes experienced in Turkey are significant and some of the earthquakes impose excessive displacement demands. For that reason, a considerable portion of existing building stock may not be safe enough in Turkey or similar countries. Also, it was observed that structural irregularities affect seismic performance of buildings. Soft storey with heavy overhangs and short columns have the most negative effect.

Karavasilis et. al. (2008) presented extensive parametric study on the inelastic seismic response of plane steel moment-resisting frames with vertical mass irregularity. A family of 135 such frames, designed according to the structural and European seismic codes, are subjected to an ensemble of 30 ordinarily (i.e., without near-fault effects) earthquake earth motions scaled to varying intensities in order to drive the structures to different limit states. The statistical analysis of created response databank indicates that the number of storey, beam-to-column strength ratio and the location (soft storey at top, mid and bottom height) of the heavier mass influence the height-wise distribution and amplitude of inelastic deformation demands, whereas the response did not seem to be influenced by the mass ratio. Nonlinear regression analysis was employed in order to derive simple formulae which reflect aforementioned influences and offer, for a given strength reduction (or behaviour) factor, three important response quantities, i.e., maximum roof displacement, maximum inter-storey drift ratio and maximum rotation ductility along the height of the structure.

A. Plumier, et. al (2005) the aim of the study was to promote safety without too much changing the constructional use of reinforced concrete structures. A test program was realized on cruciform beam-to-column nodes with a column inserted between infills. Composite solution increases the ductility significantly. The most frequent failure mode of reinforced concrete moment–frame buildings is the so called "soft storey" mechanism. It consists in the localization of buildings' seismic deformations and rupture in bottom storeys of the buildings.

Mo and Chang (1995) studied a practical system combining a elastic first storey with sliding frictional interfaces. The system utilizes Teflon sliders at the top of the first storey RC framed shear walls to carry a portion of the superstructure.

Chen and Constantinou (1990). In this studythe practical system deliberately introduces flexibility to the first storey of buildings was described. The system utilizes Teflon sliders to carry a portion of the superstructure. The Energy dissipation is provided by first storey ductile columns and by the Teflon sliders. Utilizing this concept the seismic response features of a multistorey frame are analyzed and discussed. The results show that it is feasible to provide effective preservation to the superstructure while maintaining the stability of the first storey.

Wen et. al. (2002) concluded that worldwide experience repeatedly show that damages in structures caused by earthquakes are highly dependent on site condition. In this paper, a 21-storey shear wall-structure built in the 1960s in Hong Kong was selected as an case to calibrate these two effects. Under various design earthquake intensities and for various site circumstances, the fragility curves or damage probability matrix of such building was quantified in terms of the ductility factor, which is evaluated from the ratio of storey yield shear to inter storey seismic shear. For high-rise buildings, a higher probability of damage was obtained for a soft storey condition, and damage was more severe for far field earthquakes than for near field earthquakes.

3. METHODOLOGY AND SELECTION OF PROBLEMS

Significant analytical and experimental research is conceded out since few decades, which tries to recognize the behaviour of reinforced concrete frames with progressive collapse under seismic loading. Several types of analytical models based on the physical understanding of the overall behaviour of progressive collapse frame were developed over the years to simulate the behaviour of frames. And methodology of work is explain as follows-

3.1 Material and geomerical properties

Following material properties have been considered in the modelling -
Unit weight of RCC: 25 kN/m^3
Poisson's ratio of cement brick: 0.17

Young's modulus of cement brick: 2.17185×10^7
Unit weight of clay brick: 20 kN/m^3
The depth of foundation is considered at 3.0 m below ground level and the floor height is 3.0 m.

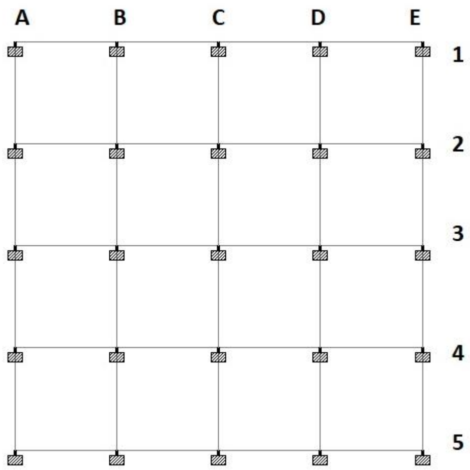

Fig. 3.1: Typical diagram of column plan

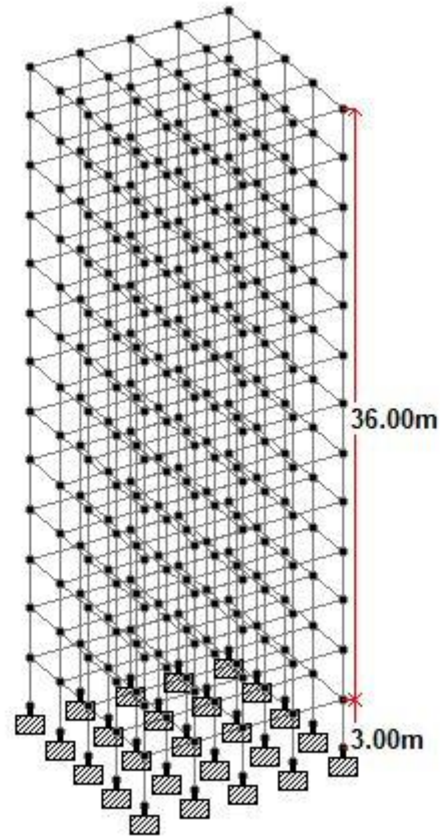

Fig. 3.2: Isometric view of building

3.3 Loading conditions
Following load are calculated and considered for analysis -

(a) **Dead Loads**: As per IS: 875 (part-1) 1987
Self weight of slab
Floor load = 0.14 x 25 = 3.75 kN/m^2 (Floor thickness = 140 mm assumed)
Floor Finish load = 1 kN/m^2
Total floor load = 3.75 + 1 = 4.75 kN/m^2
Wall height = 2.5 m (3-0.5)
External wall thickness including plaster = 0.22 m
Internal wall thickness including plaster = 0.11 m
Clay masonry wall Load (external) = 0.22 m x 2.5 m x 20 kN/m^3 = 11 kN/m
Clay masonry wall Load (internal) = 0.11 m x 2.5 m x 20 kN/m^3 = 5.5 kN/m

(b) **Live Loads**: As per IS: 875 (part-2) 1987
Live Load = 3kN/m^2
Live Load at seismic calculation = 0.75 kN/m^2

(c) **Earthquake Loads**: The earthquake calculation are as per IS: 1893 (part 1) 2002
a. Earthquake Zone-II (Table - 2)
b. Importance Factor: 1 (Table - 6)
c. Response Reduction Factor: 5(Table - 7)
d. Damping: 0.05 (5 percent) (Table - 3)
e. Soil Type: Medium Soil (Assumed)
f. Period in X direction (P_X):$\frac{0.09*h}{\sqrt{dx}}$seconds Clause 7.6.2
Period in X direction (PX) = 0.09x36/sq. root 12 = 0.936

g. Period in Z direction (P_z):$\frac{0.09*h}{\sqrt{dz}}$seconds   Clause 7.6.2 [21]

Period in X direction (P_z) = 0.09x36/ sq. root 12 = 0.936

Where, h = height of the building

d_x= length of building in x direction

And d_z= length of building in z direction

Ah_X = (Z/2 x I/R x Sa/g)

3.4 Steps of methodolgy is as follows

Step-1 Selection of building

Step-2 Selection of different progressive collapse conditions

Step-3 Selecting progressive collapse conditions are as follows:

Case 1: No column is collapse

Case 2: Column collapse at GF(bottom)

Step-4 Selection of seismic zones (II)

Step-5 Formation of load combination (13 load combinations.

Step-6 Modelling of building frames STAAD.Pro V8i is used.

Step-7 Analyses all the STAAD.Pro models

Step-8 A study comparing the results in terms of maximum moment, shear force, maximum displacement and storey displacement.

4. ANALYSIS & RESULT

4.1 Max. Bending Moment

Table 4.1: Maximum bending moment Mz (kNm) at various stories

Maximum bending moment Mz (kNm) at various stories	
Floor	Bottom
No Collapse	100.17
A1	202.326
A5	202.326
C3	203.93

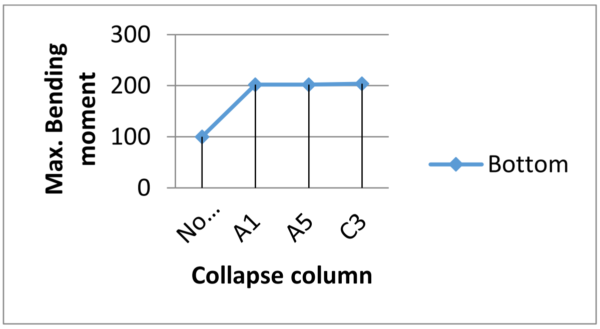

Fig. 4.1: Maximum bending moment Mz (kNm) at various stories

4.2 Max. Shear Force

Table 4.2: Maximum Shear Force (kN) at various stories

Maximum Shear Force (kN) at various stories

Floor	Bottom
No Collapse	90.541
A1	104.039
A5	152.824
C3	156.308

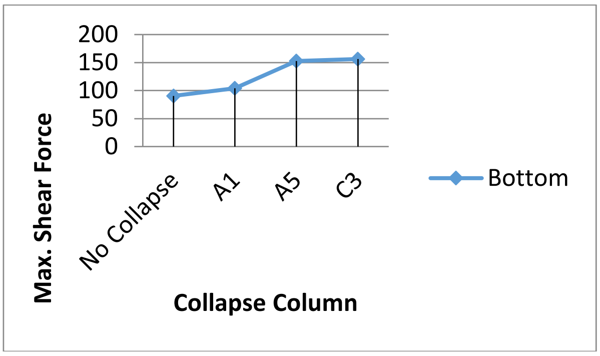

Fig. 4.2: Maximum Shear Force (kN) at various stories

4.3 Maximum Displacement

Table 4.3: Maximum Displacement (mm) in X direction at various stories

Maximum Displacement (mm) in X direction at various stories	
Floor	Bottom
No Collapse	47.909
A1	56.711
A5	56.711
C3	48.072

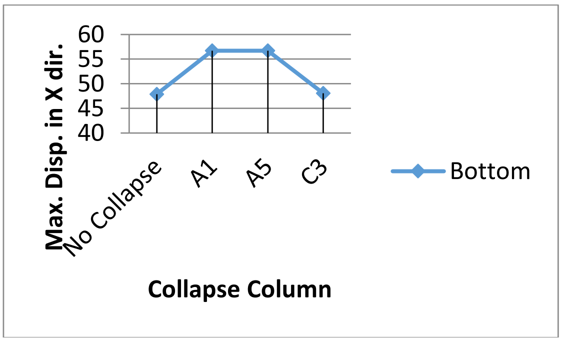

Fig. 4.3: Maximum Displacement (mm) in X direction at various stories

Table 4.4: Maximum Displacement (mm) in Z direction at various stories

Maximum Displacement (mm) in Z direction at various stories	
Floor	Bottom
No Collapse	47.909
A1	56.711

A5	56.711
C3	48.072

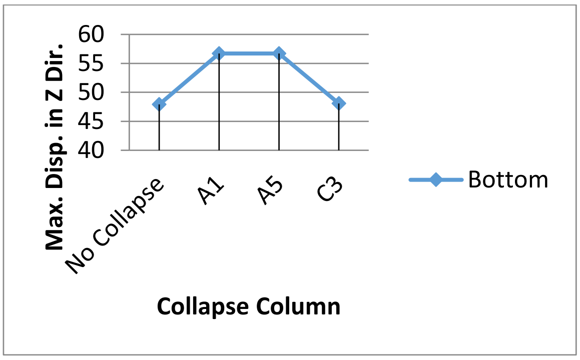

Fig. 4.4: Maximum Displacement (mm) in Z direction at various stories

4.4 Conclusion

In this study progressive collapse of building is analysed while targeting various vulnerable points (columns remove) to excaudate and determine results with the help of various above parameters. conclusion of above work is as follows:

1. While removing bottom column it is found that Column C3 is more critical means Centre of building is directly affect the progressive collapse
2. After removing column parameters such as BM, SF & Disp. are increased due to which steel, shear reinforcement and size of sections are increased respectively
3. Removing column also reduce the stability and strength of structure

In this study while removing column, located at GF of building causes instability of structure so GF column is critical. And to prevent structure from collapse, strengthen the vulnerable elements of structure while providing perfect detailing of reinforcement.

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