Progressive Collapse Analysis of Multi-Story Space Frames.

Tharwat Sakr, Atef Eraky, Jala Esmat

Abstract— The progressive collapse of building structures is initiated when one or more vertical load carrying members (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 weight (gravity load) transfers to neighboring elements in the structure. If these elements are not properly designed to resist and redistribute the additional gravity load the vertical load carrying elements of the structure continue to fail until the additional loading is stabilized causing greater damage to the structure than the initial cause. This paper aims to study the progressive collapse of multi-story buildings and how it can be affected by the statical system of the building. The alternate path method is applied to an analytical model of space frames five-story, five -bays RC frame with different statical systems using linear and non-linear static analysis. The study focuses on the behavior of space frames due to three scenarios of column removals representing inner column, edge column, and corner column.Four different structural systems are investigated including the column-beam framed system, the core wall system, the bracing system, and the hat-trigger system. The linear static analysis models indicate that the structure would be susceptible or not susceptible to progressive collapse for all analysis cases. Investigating the effects of column removal, the original structural system, shear wall, and bracing systems proved to be more affected by inner column removal. This can be attributed to the huge load supported by the removed columns which suddenly seeks another path. Removing the edge column then the corner column has less effect due to the relatively less loads need to be relocated. At the other hands, the hat-rigger system shows better behavior as it provides alternate path through tension in the upper floors to the upper truss and then to other column as compression.

Index Terms—sensitivity of building, progressive collapse, multistory space frames, load redistribution, alternate load path, loss of columns, structural systems.

1 INTRODUCTION

Progressive collapse is generally a rare accident, but its effects on buildings are very dangerous and costly. Without significant consideration of adequate continuity, ductility and redundancy during design, the progressive collapse cannot be prevented. Progressive collapse has been documented historically and became an important issue for structural design following the collapse of London's Ronan Point Apartment building, a 22-story; precast concrete panel building, in May 1968 (Griffiths et al., 1968) [1]. In this event, a small gas explosion on the 18th floor resulted in the loss of a supporting wall panel, which led to the collapse of the upper floors. Thus, the collapse progressed towards the ground due to the impact of the failed upper floors on the floors below as shown in (Fig.1). The progress of consecutive damage during the progressive collapse of Alfred P Murrah building in Oklahoma City in 1995 and the collapse of twin towers of the World Trade Center during the suicide attacks in New York City in 2001 constitutes very clear examples [2]. In a progressive collapse two stages can be identified [3]. In the first one, the structure undergoes a traumatic event and suffers some local Damage which is a direct effect of the traumatic event. In the second stage, the damage progresses up to a final wider extent, which is an indirect effect of the traumatic event.

Causes can be subdivided in categories: (a) during the design phase (wrong design or ignoring specified loads), (b) during the construction phase (bad workmanship, low quality mate rials and design), and (c) during use (loads unforeseen in the design phase, materials deterioration, bad maintenance in addition to natural and man-made hazards).

Due to its great effects on life safety and economic losses, codes were developed (ASCE 2010, ACI 2005, GSA 2003, and DOD 2005) [4], [5], [6], [7] addressing progressive collapse in the design of buildings. There are two general methods for structural design of buildings to mitigate damage due to progressive collapse: indirect and direct design methods. Indirect design incorporates implicit consideration of resistance to progressive collapse through the provision of minimum levels of strength, continuity, and ductility. Direct design incorporates explicit consideration of resistance to progressive collapse through two methods. One is the Alternative Path Method (APM) in which local failure is allowed to occur, but seeks to provide alternative load paths so that the damage is absorbed and major collapse is averted. The other method is the Specific Local Resistance Method that seeks to provide strength to resist failure. While as direct design is utilized in the design provisions specifically developed for progressive collapse analysis of structures (GSA 2003, DOD 2005), general building codes and standards (ACI 2005, ASCE/ SEI 2005) use indirect design by increasing overall integrity of structures. The General Services Administration (GSA) published guidelines for progressive collapse analysis and design of structures in 2000 and 2003. The GSA (2003) guidelines are primarily based on APM and mandates instantaneous removal of one load-bearing element with different scenarios as the initiation of damage. GSA (2003) also recommends application of a nonlinear analysis, particularly for buildings having more than 10 stories above the grade. DOD (2005) provides two design methods: one employs the Tie Force Method (indirect design), and the other employs the Alternative Path Method (direct design).

- *Jala Esmat: Master student, Faculty of Engineering, Zagazig University-Egypt. E-mail: jala_3010_eng@yahoo.com*
- *Tharwat Sakr: Professor of Structural Engineering, Faculty of Engineering, Zagazig University, Egypt.*
- *Atef Eraky: Professor of Structural Engineering, Faculty of Engineering, Zagazig University,Egypt .*

Distinguishing between ductile and brittle modes of failure, acceptance criteria consist of strength requirements and deformation limits. ACI 2005 requirements for structural integrity are to improve the redundancy and ductility in structures, which are primarily based on providing some continuous reinforcement in beams and floor systems to bridge a damaged support. These include horizontal and vertical ties throughout the structure, continuous reinforcement in perimeter elements, a specified amount of splicing, and connections that do not rely on gravity.

There were many researches dealing with experimental and analytical techniques that have been used in progressive collapse analysis of structures: Izzuddin .B. et al. [8] proposed a multi-level framework for progressive collapse assessment of building structures subject to sudden column loss. Izzuddin discussed the assessment framework employing three stages, namely (i) determination of the nonlinear static response, (ii) simplified dynamic assessment, and (iii) ductility assessment. The application of the proposed progressive collapse assessment framework to steel-framed composite buildings with simple/ partial-strength connections was demonstrated. Shi. Y. et al. [9] proposed a new method for progressive collapse analysis of reinforced concrete (RC) frame structures; it is based on the alternative load path method in the GSA and DOD guidelines, but with modifications by including the inevitable non-zero initial conditions and damage in the structural members caused by the direct blast load. A three-story two-bay RC frame is analyzed to demonstrate the efficiency and reliability of the proposed method. It is found that the proposed method gives a similar prediction of the frame collapse process to that of the direct simulation of the structure response to blast load. STINGER. S. [10] presented three laboratory tests (discontinuous reinforcement frame, continuous reinforcement frame and infill wall frame) involving the comparison between their results.

In this paper, the sensitivity of different structural statical systems used in buildings to progressive collapse is investigated. Space models of different building examples are carried out using the alternate path method. Three scenarios of column removals from ground floor representing edge, corner, and inner columns are included as per the related standards. Four different structural systems are investigated including the column-beam framed system, the core wall system, the bracing system, and the hat-trigger system. Investigations of the different design parameters such as deformation, moment, shear and damage ratio values are carried out by applying the alternate path method.

2 Numerical Modeling

Building example is selected to represent the majority of buildings encountered in the common construction industry. The selected building is five story concrete framed building with rectangular plan containing five spans in each direction as shown in Fig. (2).

Figure 1: Ronan Point Apartment Building (Open University, 2003)

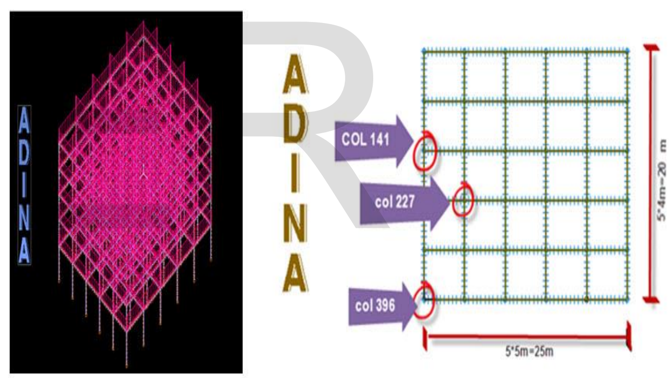

Figure2: Example building
(Space frame model and removed ground floor columns)

The building is modeled as linear elastic with elasticity modulus (E) = 2.09E10 pa, poison's ratio (ν) =.25 and Density of concrete (γ) =2500Kg/ m^3. Columns and beams of the selected buildings are designed as per the common design practice and their cross sections and reinforcements are shown in Table 1.

Also this table shows the moment of resistance value (Mr.) which is calculated from equation (1) [11]

$$M_r=.85*A_s*F_Y*d \qquad (1)$$

where M_r: moment of resistance KN.m, A_s: reinforcement area, F_Y: yield stress (KN/ m^2) and d: cover of the concrete section (m)

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linear models	cross section		Moment of resistance (Mr)		Reinforcement (As)	
	beams	columns	beams	columns	beams	columns
4m frame	30*50cm	40*50cm	166.12	276.86	6ϕ16	10ϕ16
6m frame	30*60cm	40*60cm	256.96	406.06	6ϕ18	12ϕ16
8m frame	30*80cm	40*80cm	432.598	738.3	6ϕ20	16ϕ16
10m frame	30*100cm	40*100cm	730.61	1186.97	8ϕ20	20ϕ16

Table 1: Cross sections and reinforcement details for all models.

According to DOD and GSA the results which obtained from these static computations are compared with the structural resistance using the so called “demand resistance ratios ” (DRR). A local DRR is defined as;

$$\text{DRR}=\begin{cases} M_{max} / M_r \Longrightarrow \text{ in beams (B.M only).} \\ N_{max} / N_r \Longrightarrow \text{ in bars (axial force only).} \\ M_{max} / M_r(N) \Longrightarrow \text{in columns (combined B.M and axial force).} \end{cases}$$

where:
M_{max} and N_{max} are the maximum moment and axial force acting on the section while M_r and N_r are the bending moment and axial resistances of the section, respectively. The global DRR is taken as the maximum local DRR over the structure i.e. DRR max. For reinforced concrete structures values of 200% for the demand-resistance ratio should not be exceeded, otherwise the structure is deemed as prone to progressive collapse [11].

3 Results of the Study

Analysis was conducted to complete space frame at different statical systems for 4m span model for all analysis cases (original system, the removal of edge col., the removal of inner col. and the removal of corner col. case).

3.1. Results of the 4m span Building.

From all used systems for original case the z- displacement values are almost the same, with little differences for which sample deformed shape is shown in fig.3 for the case of frame system. The maximum bending moment at building frame system is 56.14 KN.m, while the shear wall system, the bracing system and trigger system recorded 58.52, 56.38 and 54.93 KN.m, respectively, which are also very similar. The small difference in deformation and moments between different statical systems can be attributed to the fact in all cases, loads follow approximately the same load path through beams and columns/ walls. As trigger system gives another load path through column tension and hat-truss, its moment is slightly smaller than other systems.

The fixation of floor beams at shear walls also leads to the increase of maximum moment in beams at the wall connection. Maximum shear forces show the same trend such that for frame, bracing, shear wall and trigger systems, the observed maximum shear values are 57.2, 57.31, 57.48 and 56.52 KN, respectively. For edge column removal, Z - Displacement Value increased locally at the position of the removed column as shown in fig.4. The maximum deflection reached 28.98mm at the frame system and gradually reduced at other statical systems to reach 17.88mm at the trigger system. Removing the edge column also increased the maximum bending moment of the building to 281.2, 279.15, 227.8, and 189.59 KN.m for the cases of frame, shear wall, bracing, and hat-trigger systems, respectively, as compared to the approximately 56.4 KN.m in case of no column removal. Removing the edge column increased the maximum shear force to 178.395, 182.97, 182.22 and 127.66 KN for the cases of building frame, shear wall, the bracing and trigger systems, respectively, as compared to the approximately 57.2 KN in case of no column removal.
As stated in the related regulations, the second case is by considering the removal of corner column for which deformed shapes are shown in Fig 5. For different structural systems. The maximum deflection for this case recorded the values of 26.00, 24.95, 25.04, and 11.60 mm for the cases of frame building, shear wall, bracing, and hat-trigger systems, respectively at the removal of corner column, as compared to the approximately 5 mm in case of the original building. Removing the corner column increased the maximum bending moment of the building from 56.4 in case of the original building to 221.439, 211.354, 212.961 and 122.081 KN.m for the previously-mentioned systems. These values are smaller than the previous case because of the smaller area of load duplicating beside the removed column. Maximum shear forces observed are 122.25, 119.24, 119.76 and 105.13KN for the frame, shear wall, bracing and trigger system, respectively.
The case of inner column removal is then considered for which the displacement plots are shown in Fig.6. The Z-displacement for the frame system is observed to be 31.69 mm and reduced in the other statical systems (shear wall, bracing and trigger) to 29.54, 31.05 and 12.97 mm, respectively. The enhancement of deflection in case of hat-trigger system by about 59% compared to the building frame case is due to the existence of clear alternate path for loads to hat-truss through tension in upper columns and then to the other building columns. Removing the inner column also increased the maximum bending moment of the building to 321.352, 338.661, 324.956 and 164.190 KN.m for the cases of frame, shear wall, bracing, and hat-trigger systems, respectively, as compared to the approximately 56.4 KN.m in case of no column removal. These values are greater than previous cases because of duplication of loads and the sudden increase of the length of four panel’s spans attached to the removed column. Removing the inner column increased the maximum shear force to 226.416, 338.069, 227.586 and 126.536 KN for the cases of frame, shear wall, the bracing and trigger systems, respectively.

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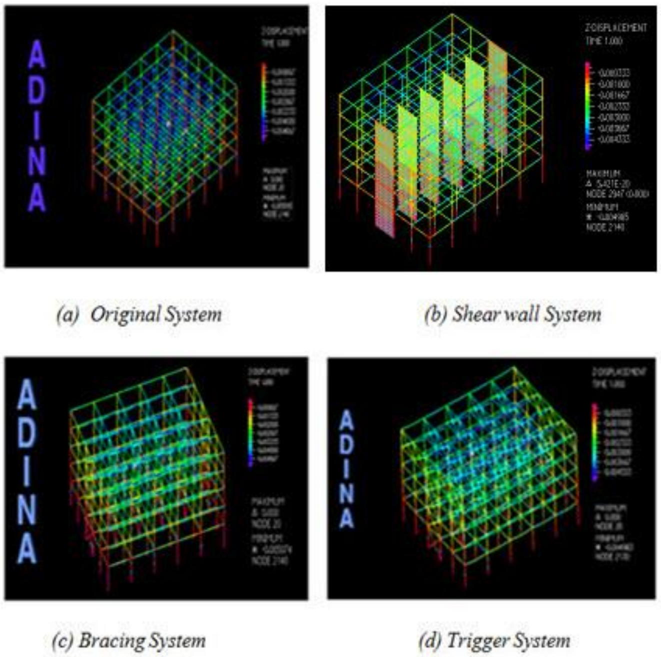

Figure 3: Deformed shapes for the 4m span building with different statical systems - Original Case.

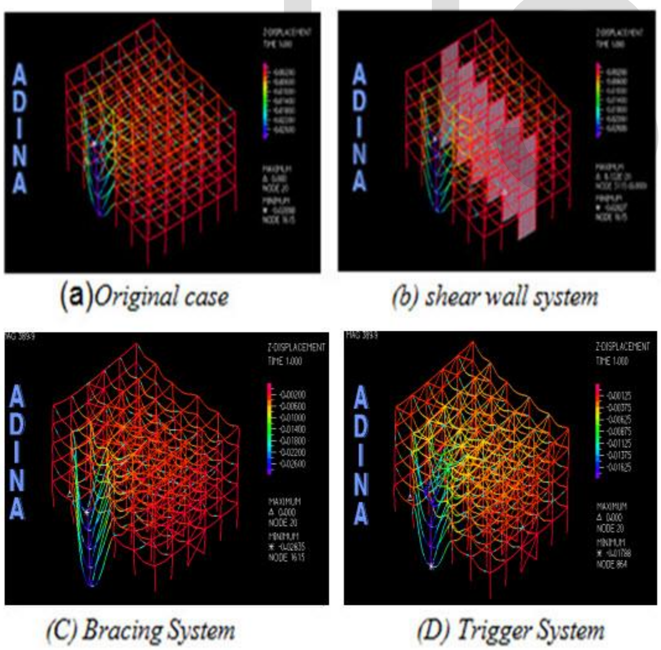

Figure 4: Deformed shape of the 4m frame with different statical system –edge column Removal

3.2 Summarized results of all models

In this section, the results of the 4, 6, 8, and 10m span buildings are summarized and discussed. Fig. 7 shows the relation between moment ratio and the main span of the building in case of edge column removal.

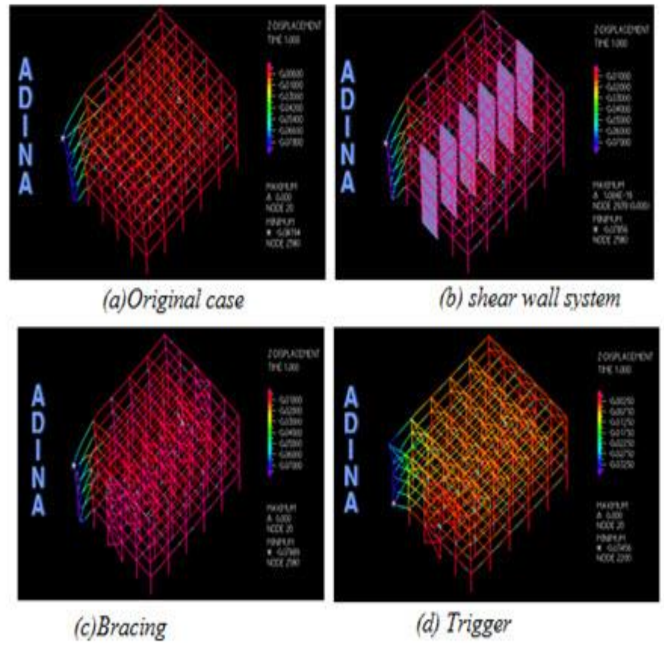

Figure 5: Deformed shape of the 4m frame with different statical system –corner column removal

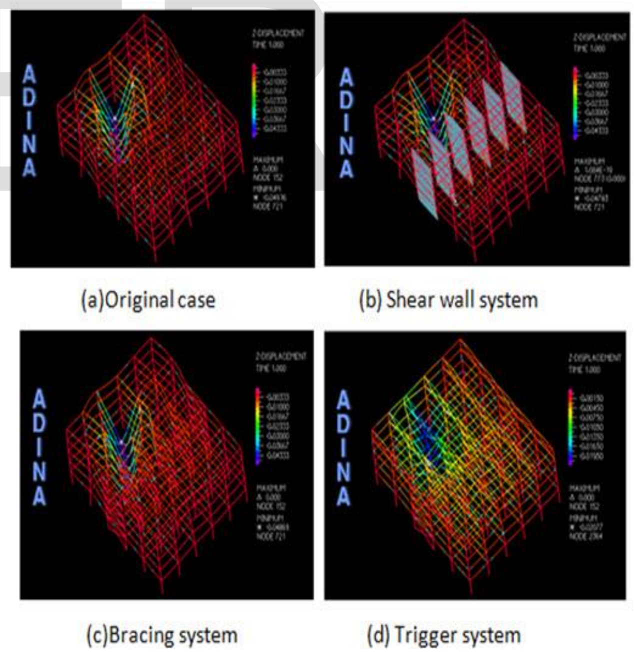

Figure 6: Deformed shape of the 4m frame with different statical system –inner colunm removal.

The moment ratio increases for the 6 m span building, and then, at general, decreases with increasing the main building span. This can be due to using the same spacing for buildings with different main spans. Beams in the transverse direction (5m) then share supporting the missed column loads with other longitudinal beams in the main directions (4m,6m, 8m and 10m).

It is also observed from the figure that the moment ratios are very similar for the frame, shear wall, and bracing systems. Such that removing the edge column increased the maximum moment at the 4m frame model for frame building, shear wall, and bracing systems to 5.0, 4.95, 4.04 times that incase of no column removal. And increased to 5.16, 5.27 and 5.08 times that in case of no column removal for the 6m frame model. These ratios reach 3.42, 3.46 and 3.45 times for the 8 m frame model and 3.4, 3.44, 3.42 for the 10 m span building for the same statical systems. This increase of moment is due to duplication of loads and the sudden increase of the length of spans attached to the removed column. The case of hat-trigger system shows different behavior such that the increase of moment above the original case is relatively smaller reaching 3.36, 3.64, 2.87 and 3.04 for 4m, 6m, 8m and 10m frame models. This enhanced behavior can be attributed to clear load path provided through the above column tension to the hat truss above.

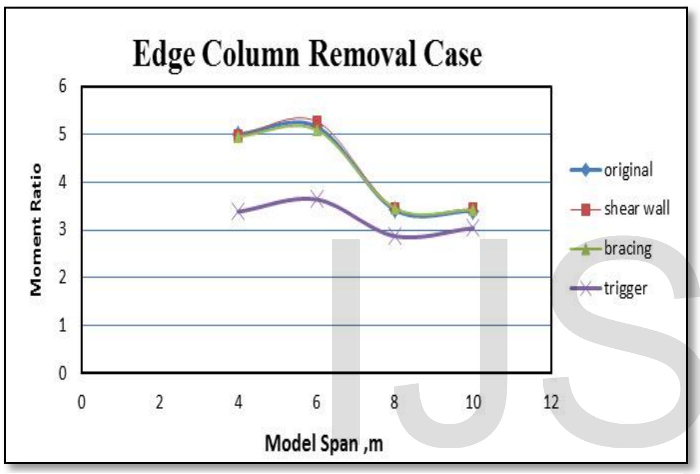

Figure 7: Relation between of the maximum moment ratio and model span for different statical systems –The edge column removal case.

To investigate the effect of changing the building span on the safety of the building, Figure 8 shows the relation between the DRR and the building main span for the case of edge column removal. The DRR value is observed to increase with increasing the building main span. DRR values in the edge column removal case at the 4 and 6m frame model for all statical systems are acceptable. They exceed 100% but still below 200% such that at the 4m frame system is equal 169%, at shear wall syatem 168%, at bracing system 167 %and trigger system 114%. And for the 6m frame model at the frame system DRR is equal 157%, at shear wall system 160%, at bracing system 155% and trigger system DDR equals 111%. These values are acceptable according to the criteria of common regulations [7]. Normally, these values indicate that all systems are able to survive in case of progressive collapse due to of edge column removal. The trigger system is observed to give better behavior in this case in terms of DRR. The results of the 8m frame model for original, shear wall and bracing system indicate that these systems perhaps affected by progressive collapse.

The DRR for the same cases are not exceeded 200% that equal 181%, 182% and 182%, respectively. But it indicates minor yielding at the trigger system and also demand resistance ratio is 151% which still below 200%, so the structure with trigger system is not susceptible to collapse. For the large span model (10m frame) DRR values at all used systems (original, shear wall, bracing and trigger system) are 193%, 195%, 194% and 173%, respectively. DRR values are very close to 200%. Normally, these values indicate that all systems may be affected by progressive collapse and not suitable for resisting progressive collapse in the edge column removal case and also trigger system is the most effective system in this case.
Also previous figures show that the behavior differs from moment ratio than DRR curve. This because of the different of DRR for original case at all models.

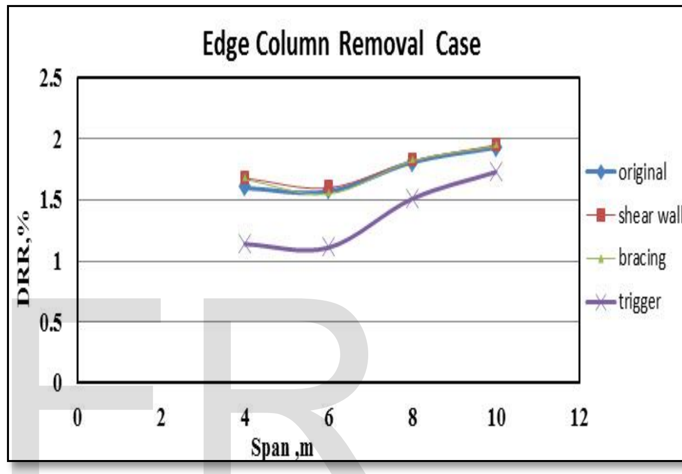

Figure 8: Relations between DRR values and model span for different statical systems for four models (4m, 6m, 8m and 10 m span)-Edge Column Removal Case.

In case of corner column removal, the maximum moment values are observed to be smaller than the previous cases as shown in fig 9, which plots the moment ratio against the building main span for this case. It shows the relation between moment ratio and main building span as the same trained before. For the 4m frame model the maximum moment increase is observed to be 3.94, 3.61, and 3.77 for the frame, shear walls, bracing systems, respectively. The maximum moment at the 6m frame model for frame building, shear wall system, bracing system reach 5.13, 5.02 and 5.03 times, respectively. And also at the 8m frame model reached to 2.92, 2.95 and 2.96 times. For the 10m frame model the maximum moment reach to 2.88, 2.89 and 2.89 times that in case of no column removal. This can be attributed to the huge load supported by the removed columns which suddenly seeks another path. But, the hat-rigger system shows different behavior such that the increase of moment reaches 2.22, 2.81, 1.91 and 2.12 for the 4m, 6m, 8m and 10m frame model, respectively which is attributed to the cantilever-like behavior of the hat-truss behaves and the relatively high load of the removed column.

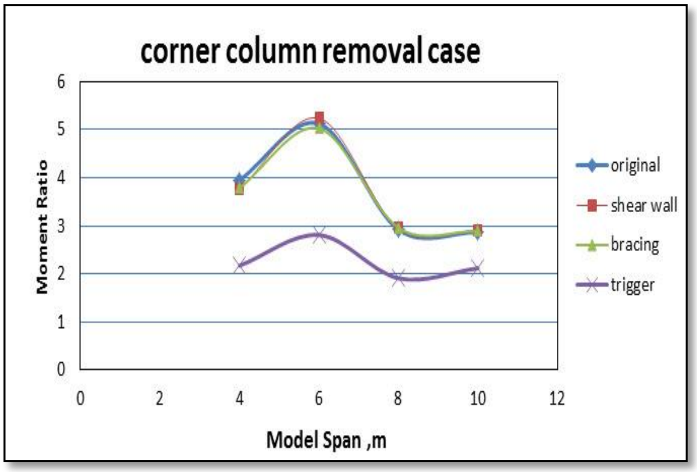

Figure 9: Relation between of the maximum moment ratio and model span for different statical systems –The corner column removal case.

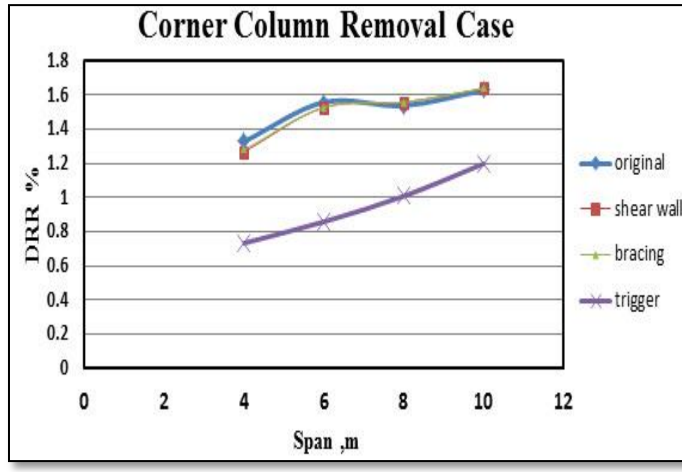

Figure 10 : Relations between DRR values and model span for different statical systems for four models (4m, 6m, 8m and 10 m span)-Corner Column Removal Case.

To investigate the building behavior in case of corner column removal, figure 10 shows the relation between the DDR and the building main span for corner column removal case. At this case for the 4m and 6m frame model the result of trigger system indicates no yielding understood from the demand resistance ratio of 73.5% and 86%,respectively which is enough below 100%, so the structure with trigger system is considered to be far away from progressive collapse for corner column removal case.all other systems (frame, shear wall and bracing system) DRR values recorded 133%, 127%, 128%,respectively .And at the 6m frame model DRR values recorded 156%, 153% and 153%, respectively. The DRR values of these systems are below 200% which means that they are affected by progressive collapse, but still accepted as per the commonly used criteria. Also at 8m frame model, results of trigger system indicates minor yielding with demand resistance ratio 101% which far below 200%, so the structure with trigger system is not susceptible to collapse. All other systems (original, shear wall and bracing system) when the permanent loads are multiplied by a factor of 2 are 154%, 156% and 156%, respectively. The DRR values of these systems are still below 200%. Finally, for the 10m frame model, DRR values are also below 200% for all used systems (original, shear wall, bracing and trigger system) recording 163%, 164%, 164% and 120%, respectively. According to the failure criteria, the DDR is considered accepted which means that the system is safe if subjected to corner column removal. These values indicate that all systems are suitable for resisting progressive collapse in the corner column removal case with the trigger system having the most effective behavior in this case.

Fig. 11 shows the relation between moment ratio and the main span of the building in case of inner column removal for building frame, shear wall, bracing and trigger systems. These values are greater than previous cases because of duplication of loads and the sudden increase of the length of four panel's spans attached to the removed column.

It is also observed from the figure that the moment ratios are very similar for the frame, shear wall, and bracing systems. Such that removing the inner column increased the maximum moment at the 4m frame model to 5.72, 5.79 and 5.76 times that for the original building for frame building, shear wall system, bracing system, respectively. At the 6m frame model, the maximum moment for the same systems reaches to 6.82, 6.95 and 6.8. These ratios reach to 4.93, 5.13 and 5.0 times for the 8m frame model and 4.83, 5.00, 4.88 for the 10 m span building for the same statical systems. This can be attributed to the huge load supported by the removed columns which suddenly seeks another path. The case of hat-trigger system shows different behavior such that the increase of moment above the original case is relatively smaller reaching 2.99, 3.23, 2.97 and 3.4 for the 4m, 6m, 8m and 10m frame model, respectively. This enhanced behavior can be attributed to clear load path provided through the above column tension to the hat truss above.

Figure 12 shows the relation between the DRR and the building main span for the case of edge column removal. The DRR value is observed to increase with increasing the building main span. The 4m and 6m frame model, for the case of trigger system indicates no yielding such that the demand resistance ratio is 99% and 98%, respectively which still below 100% which means that the structure with trigger system is not susceptible to collapse. But at other systems (original, shear wall and bracing system) when the permanent loads are multiplied by a factor of 2 (at four bays beside removed column). For the 4m frame model DRR values record 194%, 204% and 196%, respectively. And DRR for the 6m frame model exceeded 200% to record 208%, 212% and 207%, respectively. So other systems would be deemed as susceptible to progressive collapse. For the 8m frame model result of DRR values exceed 200% reaching 260%, 271% and 264% for the same structural systems. For the trigger system, DRR value still below 200% which record 157%, respectively.

Also for the large span model (10m frame) DRR values exceed 200% at the same systems which are 275%, 284%, 277% and 194%, respectively. Even for the 10 m span, according to relevant standards, the trigger system still acceptable for progressive collapse resistance.

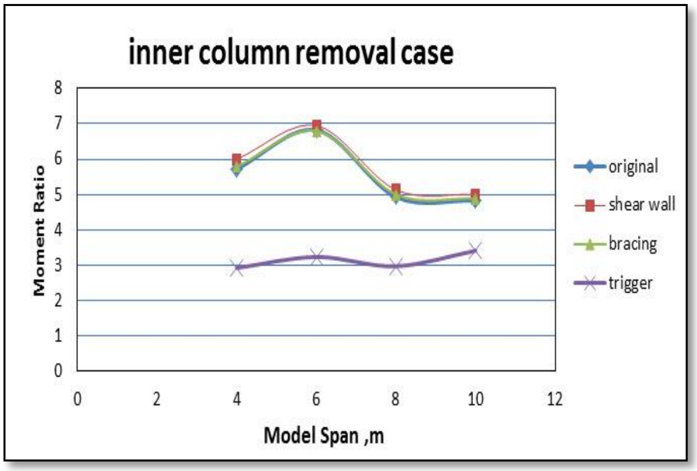

Figure 11: Relation between of the maximum moment ratio and model span for different statical systems –The inner column removal case.

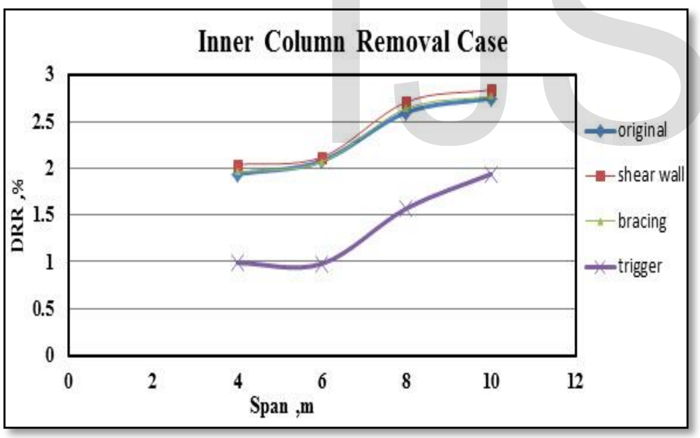

Figure 12: Relations between DRR values and model span for different statical systems for four models (4m, 6m, 8m and 10 m span)-Inner Column Removal Case.

4 Conclusions

This paper presents the progressive collapse analysis of buildings with different structural systems. These systems include the column-beam framed system, the central core wall system, the bracing system, and the hat-trigger system as a progressive collapse mitigation scheme. As recommended by relevant regulations, three scenarios of column removals representing inner column, edge column, and corner column using linear static analysis. The most important conclusions can be summarized as follows:

- For all cases, the removal of Edge, inner, or corner column from the ground floor increases dramatically the vertical deflections, shear forces, and bending moments of the building for all used statical systems.
- For the studied cases, and based on the maximum DDR values, the acceptance of different statical systems for progressive collapse resistance were as follows:
  - In case of edge column removal, the 4m and 6m span buildings are acceptable while the 8m and 10m span buildings are sussibtible to progressive collapse.
  - All studied examples proved to be progressive collapse resisting in case of corner column removal.
  - The trigger system proved to be effective such that it provides progressive collapse resistance for buildings with different spans and removal scenarios.
  - Buildings with all spans and statical systems except hat-trigger system are observed to be insufficient for progressive collapse due to inner column removal.
- For all column removal scenarios, the greater the span model, the less the ability to withstand collapse at all column removal scenarios.
- The inner column removal case is the most serious cases, which caused an early collapse of the building. The removal of edge column has less effect then corner column which have relatively minor damaging especially for large spans buildings.
- The use of central shear wall or bracing systems is ineffective for progressive collapse resistance as they are usually far away from the local failure zone of the building.
- Trigger system proved to be effective for progressive collapse resistance as it provides clear alternate path for the removed column loads.

5 References

[1] Griengsak Kaewkulchai, B.Eng., M.S., Dynamic Progressive Collapse Of Frame Structures, The University of Texas at Austin,2003

[2] Allen D.E., Schriever W.R. (1973) - Progressive collapse, abnormal loads and buildingcodes. Structural failures: modes, causes, responsibilities - American Society of Civil Engineers, New York, NY.

[3] Francesco Barni, A Method to Assess the Progressive Collapse Vulnerability of Frame Structures, University of Florence, 2011.

[4] Wright R.N., Carino N.J., Gross J.G., Sansalone M. (1989) - Assessment: U.S. office building In Moscow - Journal of Performance of Constructed Facilities, ASCE 3(1) 1989 pp. 2-15.

[5] ACI 318 (2005). "Building Code Requirement for Structural Concrete and Commentary," American Concrete Institute, Farmington Hills, MI.

[6] GSA, Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. General Service Administration (GSA), Washington, D.C., 2003.

[7] DOD. Unified facilities criteria (UFC) 4-023-03—design of buildings to resist progressive collapse, Department of Defense, 2005,

[8] B.A. Izzuddin, A.G. Vlassis, A.Y. Elghazouli and D.A. Nethercot, Progressive collapse of multi-story buildings due to sudden column loss — Part I: Simplified assessment framework, science direct 30 (2008) 1308–1318.

[9] Yanchao Shi, Zhong-Xian Li, Hong Haoa, A new method for progressive collapse analysis of RC frames under blast loading, science direct 32 (2010) 16911703.

[10] STEPHEN MARTIN STINGER, EVALUATION OF ALTERNATIVE RESISTANCE MECHANISMS FOR PROGRESSIVE COLLAPSE, University of Missouri-Columbia, 2011.

[11] Seweryn Kokot, Armelle Anthoine, Paolo Negro and George Solomos, Static and dynamic analysis of a reinforced concrete flat slab frame building for progressive collapse, JRC, 2010.