*- Technical Paper -*

# Progressive Collapse Assessment of Multi-Story Flat Slab Reinforced Concrete Structures under Gravity Loads

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ABSTRACT

The resistance of multi-story RC flat slab structures to progressive collapse is numerically evaluated using Applied Element Method. The case study was designed according to ACI 318 and its progressive collapse was assessed in accordance to the UFC guidelines. The analytical results showed that the case study generally satisfies the limits of UFC guidelines except for the cases of near-corner interior column loss from tenth floor and edge shear wall loss from ground, fifth, eighth and tenth floors. It experienced a partial collapse due to edge shear wall removal from eighth and tenth floors.

Keywords: progressive collapse, alternative path method, flat slab

## 1. INTRODUCTION

Since the partial progressive collapse of Ranon point building in 1968 in England, attention on progressive collapse has been increased in the civil engineering community. The Ranon point building, a 22-story building, failed due to a gas explosion in the 18th floor. In 1995, Sampoong Store in South Korea collapsed due unconsidered increase in dead loads. Finally, the ‘World Trade Center’ towers totally collapsed in 2001 due to plane crashing into the towers at a high speed. As a result of those collapses, many countries adopted explicit progressive collapse measures into their building codes such as General Services Administration (GSA, 2003) and the Unified Facilities Criteria (UFC, 2013). The objective of this research is to assess the progressive collapse resistance of a ten-story reinforced concrete flat slab structure designed according to (ACI-318, 2008). The assessment is carried out in accordance to the UFC guidelines for Alternative Path Method, where the structure ability to bridge over a missing vertical structural element is checked with the resulting extent of damage being localized

The choice of the numerical method to do this investigation was very crucial because of the significant need to simulate the progressive collapse of different parts of the structure. Although the FEM is a robust and well established structural analysis method, it is not the optimum solution for the current study scope. Many drawbacks are associated with the FEM progressive collapse analysis; the element damage separation, falling and collision with other elements are very difficult). Hartmann *et al.* (2008) showed that the computations associated with the simulation of collapses of real world structures based on conventional FEM are very costly. Therefore, in the current study, the numerical analysis was carried out using the Applied Element Method. The Applied Element Method is based on discrete crack approach and is capable of following the structure's behavior to its total collapse (Tagel-Din and Meguro, 2000, Meguro and Tagel-Din, 2001, Tagel-Din, 2002, Meguro and Tagel-Din, 2003, Sasani and Asgitoglu, 2008, Salem et al., 2011, Park et al., 2009, Helmy et al., 2009, Helmy et al., 2012, Helmy et al., 2013, Sasani, 2008, Wibowo, 2009, Salem 2011, Salem and Helmy, 2014, Salem et al., 2014).

## 2. UNIFIED FACILITIES CRITERIA GUIDELINES(UFC)

The UFC guidelines provide three design approaches; Alternate Path (AP), Enhanced Local Resistance (ELR) and Tie Forces approaches. Using of these approaches depends on the occupancy category level (O.C) of the building which is a function of level of occupancy and criticality of the building. The design approach used for this study is The Alternate Path method. The AP requires that the structure be capable of bridging over a missing structural element, with the resulting extent of damage being localized. In AP method type of model and load combinations depend on type of analysis, static or dynamic. For nonlinear dynamic analysis, only three-dimensional model is allowed with load combination [1.2 D + 0.5 L], where D and L denotes for dead load and live load, respectively. The UFC states that the beam-to-beam and/or slab-to-slab continuity is assumed to be maintained across a removed vertical support. As for load-bearing wall, the removed length of wall is twice the clear story height. For shear wall has a C-shaped cross-section in plan, only the flange or only the web are removed, but not both. Removal location of external supporting element should be near the middle of the short side, near the middle of the long side, and at the corner of the building. For structures with uncontrolled public access, removal location of supporting element should be near the middle of the short side, near the middle of the long side, the corner of the uncontrolled space and where the plan geometry of the structure changes significantly. For each plan location defined for element removal, the AP analysis is performed for first story above grade, story directly below roof, story at mid-height, story above the

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location of a change in column size. The structure considered safe if after support removal slab, beam, column and slab-column joint rotation satisfy the UFC acceptance criteria.

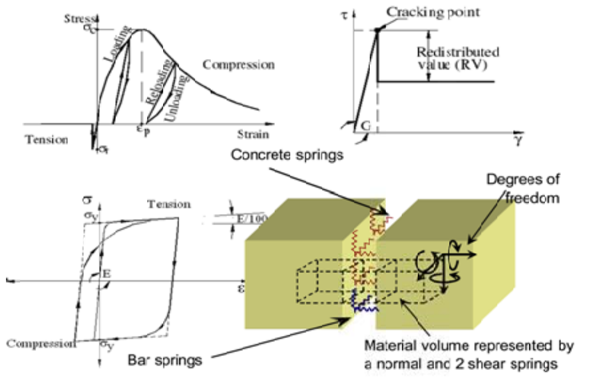



Fig.1 Modeling of a structure using AEM

## 3. THE APPLIED ELEMENT METHOD (AEM)

The AEM is an innovative modeling method adopting the concept of discrete cracking. In AEM, structures are modeled with elements assembly as shown in Fig.1. The elements are connected together along their surfaces through a set of normal and shear springs. Those springs are responsible for transfer of normal and shear stresses among adjacent elements. Each spring represents stresses and deformations of a certain volume of the material as shown in Fig.1. Each two adjacent elements can be completely separated once the springs connecting them are ruptured.

Fully nonlinear cyclic constitutive models are adopted in the AEM as shown in Fig.1. For concrete in compression, elasto-plastic and fracture model is adopted (Maekawa and Okamura, 1983). When concrete is subjected to tension, linear stress-strain relationship is adopted until cracking, where the stresses drop to zero. Since the method adopts discrete crack approach, the reinforcing bars are modeled as bare bars for the envelope (Okamura and Maekawa, 1991) while the model of Ristic *et al.* (1986) is used for the interior loops. Reinforcing bars are assumed to remain elastic in shear. Full bond between reinforcing bars and concrete is assumed since for deformed bars physical slip does not occur at the bar surface (Okamura and Maekawa, 1991). Reinforcing bars rupture when reaching their ultimate strength.

The AEM is a stiffness-based method, in which an overall stiffness matrix is formulated and the equilibrium equations including each of stiffness, mass and damping matrices are nonlinearly solved for the structural deformations (displacements and rotations). The solution for equilibrium equations is an implicit one that adopts a dynamic step-by-step integration (Newmark-beta time integration procedure) (Bathe, 1982 and Chopra, 1994).

In the AEM, two adjacent elements can separate from each other if the springs connecting them are ruptured. Elements may automatically separate, re-contact or contact other elements. In this study, the Extreme Loading for Structures (ELS) software (www.appliedscienceint.com), which is based on the AEM, is used.

The AEM was proven to be capable of following the deformations of a structure subjected to extreme loads to its total collapse.

## 4. CASE STUDY

A ten-story reinforced concrete structure with footprint dimensions of 42 × 42 m and a total height of 31 m is investigated. The structure is composed of equal bays of 6 m span as shown in Fig.2. The ground floor is a public access space (uncontrolled area) with total height of 4 m, while all the other floors are 3m high. The structure has two edge shear walls and one interior core. The structural system of the floors is a flat slab with edge (marginal) beams. The slab thickness is 250mm reinforced with top and bottom meshes of D12@165 in both directions, in addition to additional top reinforcement of D12@333 mm in both directions at interior columns locations. Concrete dimensions and reinforcement detailing for beams and columns are shown in Fig.3. The structure was designed according to ACI 318 (2008) including the effect of both gravity loads and seismic loads. In addition to the self-weight of the structure, uniformly distributed loads of 20 $kN/m^2$ were considered on slabs for each of the live loads, the finishes and the partitions. Fig.4 shows the structural model of the case study and material properties are shown in Table 1.

According to the UFC specifications, the analysis cases are as follows;

1. Removal of a corner column
2. Removal of an edge column
3. Removal of an internal column
4. Removal of an internal column near corner.
5. Removal of an edged shear wall.
6. Removal of an internal shear wall.

For each case, the element removal is carried out, independently, at different floors; ground floor, fifth floor, eighth floor (floors at which column sizes change) and tenth floor (floor just below the roof). Fig.2 shows the locations of the removed columns, walls and core. The support removal is instantaneous, at t=0.00 sec. The removal didn't impede into the connection at the floor level to ensure beam-to-beam and slab-to-slab continuity according to UFC specifications.

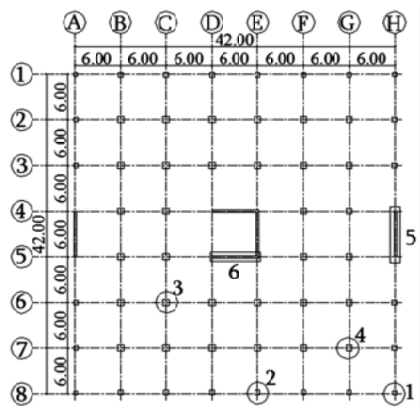
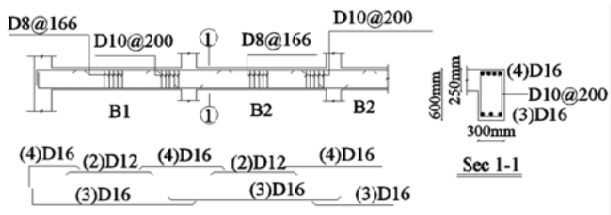
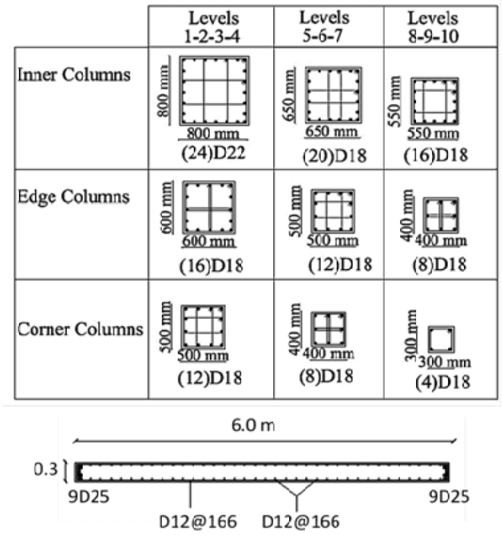



Fig.2 Structure layout




(a) Beams reinforcement




(b) Columns and walls reinforcement

Fig.3 Reinforcement details of the case study

Table 1 Material properties for case study

| Material | Young's Modulus (MPa) | Comp. Strength (MPa) | Tensile Strength (MPa) | Yield stress/ Ultimate Strength (MPa) |
|---|---|---|---|---|
| Concrete | 21019 | 20 | 2 | ------ |
| Re-bars | $2x10^5$ | ------ | ------ | 360/520 |

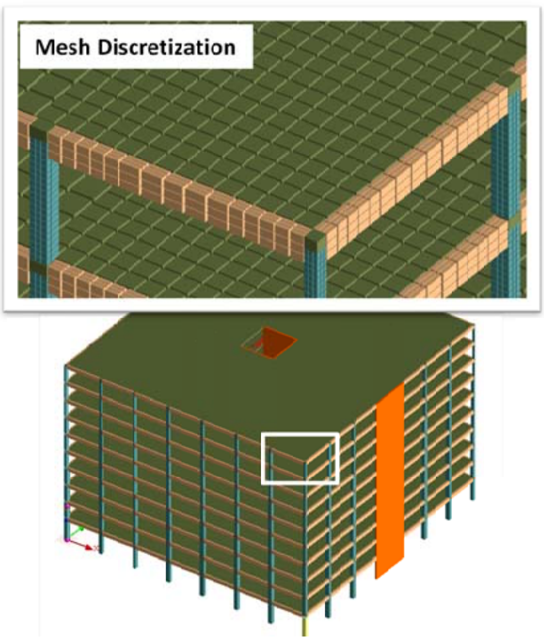



Fig.4 AEM model of the case study

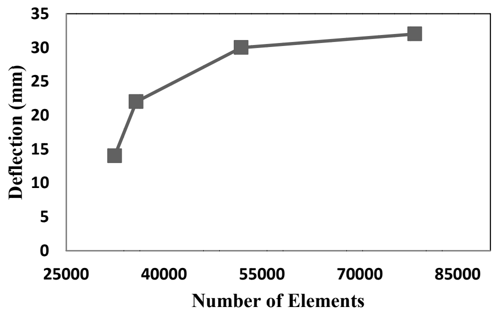



Fig.5 Mesh sensitivity analysis

## 5. ANALYSIS RESULTS AND DISCUSSIONS

A mesh sensitivity analysis was carried out for the case of corner column removal. Fig.5 shows the mesh sensitivity for the deflection for an element just above the removed column. The change in the deflection from mesh No.3 to No.4 is very small; therefore, mesh No.3 was used in the analysis.

### 5.1 Satisfaction of the UFC requirements

In all analysis cases, the structure did not collapse and could bridge over the removed support except for the case of edge shear wall removal from the eighth and tenth floor, where the structure experienced a partial collapse as shown in Fig.6. Those collapsed cases are not accepted by the UFC specifications. As for the uncollapsed cases, the UFC puts limits for the rotations in slabs, beams, columns and slab-column connections. The Factor of safety (F.O.S) for rotations is calculated by dividing the UFC limit by the maximum rotation obtained from the analysis. Table 2 summarizes the F.O.S. for all study cases. In all cases, the structure was safe according to UFC guidelines except for the case of near-corner interior column removal from the $10^{th}$ floor and cases of edge shear wall removal from ground, fifth, eighth, and tenth floors. It is obvious the case of edge shear wall removal was the most critical case. This could be attributed to the relatively large unsupported area resulting from the wall loss.

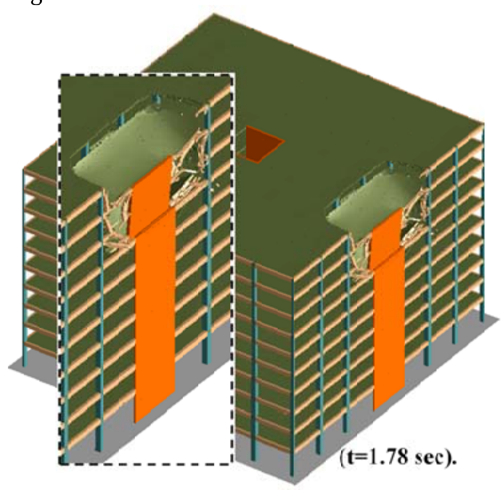



Fig.6 Partial collapse of edge shear wall removal from the $8^{th}$ floor

Table 2 Summary of the analysis results according to UFC requirements

| Case | Level | Collapse | Maximum deflection (mm) | | Slab (2.86°)[1] | Beam (3.6°)[2] | Joint (2.86°)[3] | Column (0.86°)[4] | Status |
|---|---|---|---|---|---|---|---|---|---|
| Corner Column | Ground | X | 21.2 | Rotation | 0.46° | 0.4° | 0.31° | 0.26° | Safe |
| | | | | F.O.S | 6.2 | 9 | 9.2 | 3.3 | |
| | 5th | X | 25.0 | Rotation | 0.44° | 0.41° | 0.32° | 0.25° | Safe |
| | | | | F.O.S | 6.5 | 8.7 | 8.9 | 3.4 | |
| | 8th | X | 27.25 | Rotation | 0.44° | 0.43° | 0.33° | 0.25° | Safe |
| | | | | F.O.S | 6.5 | 8.3 | 8.6 | 3.4 | |
| | 10th | X | 41.0 | Rotation | 0.56° | 0.56° | 0.48° | 0.51 | Safe |
| | | | | F.O.S | 5.1 | 6.4 | 5.9 | 1.6 | |
| Edge Column | Ground | X | 28.7 | Rotation | 0.91° | 0.48° | 0.31° | 0.31° | Safe |
| | | | | F.O.S | 3.1 | 7.5 | 9.2 | 2.7 | |
| | 5th | X | 32.1 | Rotation | 0.86° | 0.52° | 0.33° | 0.3° | Safe |
| | | | | F.O.S | 3.3 | 6.9 | 8.6 | 2.8 | |
| | 8th | X | 34.9 | Rotation | 0.91° | 0.6° | 0.35° | 0.3° | Safe |
| | | | | F.O.S | 3.1 | 5.9 | 8.1 | 2.8 | |
| | 10th | X | 41.5 | Rotation | 1.0° | 0.7° | 0.36° | 0.28° | Safe |
| | | | | F.O.S | 2.8 | 5.1 | 7.9 | 2.9 | |
| Interior Column | Ground | X | 40 | Rotation | 0.79° | 0.28° | 0.33° | 0.27° | Safe |
| | | | | F.O.S | 3.6 | 12.8 | 8.6 | 3.1 | |
| | 5th | X | 44.7 | Rotation | 0.85° | 0.28° | 0.34° | 0.26° | Safe |
| | | | | F.O.S | 3.3 | 12.8 | 8.4 | 3.2 | |
| | 8th | X | 48.9 | Rotation | 0.92° | 0.27° | 0.34° | 0.25° | Safe |
| | | | | F.O.S | 3.1 | 13.3 | 8.4 | 3.3 | |
| | 10th | X | 67.8 | Rotation | 1.18° | 0.32° | 0.35° | 0.3° | Safe |
| | | | | F.O.S | 2.4 | 11.2 | 8.1 | 2.8 | |
| Near Corner Interior Column | Ground | X | 69 | Rotation | 1.67° | 0.44° | 0.75° | 0.73° | Safe |
| | | | | F.O.S | 1.7 | 8.1 | 3.8 | 1.17 | |
| | 5th | X | 75.4 | Rotation | 1.79° | 0.48° | 0.77° | 0.75° | Safe |
| | | | | F.O.S | 1.59 | 7.5 | 3.7 | 1.14 | |
| | 8th | X | 89.3 | Rotation | 1.98° | 0.43° | 0.89° | 0.85° | Safe |
| | | | | F.O.S | 1.44 | 8.3 | 3.2 | 1.01 | |
| | 10th | X | 135.9 | Rotation | 2.98° | 0.45° | 1.0° | 0.75° | Unsafe |
| | | | | F.O.S | ------- | 7.9 | 2.86 | 1.14 | |
| Edge Shear Wall | Ground | X | 187.7 | Rotation | 4.42° | 8.15° | 0.67° | 0.28° | Unsafe |
| | | | | F.O.S | ------- | ------- | 4.2 | 3.0 | |
| | 5th | X | 210.9 | Rotation | 5.16° | 9.42° | 0.8° | 0.28° | Unsafe |
| | | | | F.O.S | ------- | ------- | 3.56 | 3.0 | |
| | 8th | √ | Partial Collapse | | | | | | Unsafe |
| | 10th | √ | Partial Collapse | | | | | | Unsafe |
| Interior Shear Wall | Ground | X | 3.7 | Rotation | 0.41° | 0.35° | 0.3° | 0.33° | Safe |
| | | | | F.O.S | 6.9 | 10.2 | 9.5 | 2.6 | |
| | 5th | X | 4.6 | Rotation | 0.44° | 0.32° | 0.33° | 0.3° | Safe |
| | | | | F.O.S | 6.5 | 11.2 | 8.6 | 2.8 | |
| | 8th | X | 4.8 | Rotation | 0.44° | 0.32° | 0.33° | 0.3° | Safe |
| | | | | F.O.S | 6.5 | 11.25 | 8.6 | 2.8 | |
| | 10th | X | 74.8 | Rotation | 2.2° | 0.25° | 0.9° | 0.23° | Safe |
| | | | | F.O.S | 1.3 | 414. | 3.1 | 3.7 | |

[1,2,3,4] UFC rotation limits for slabs, beams, joints and columns, respectively

## 5.2 Investigation of the collapsed cases

The main resisting alternative load paths after support loss are the slab catenary action, as well as the vierndeel action composed by the slabs and columns above the removed support. It is clear that, the more floors above the level of removed support, the stronger the vierendeel action will be, resulting in higher redistribution of forces to the elements adjacent to the removed wall. The collapsed cases were those cases where the wall was removed in the upper floors. In other words, they are the cases where the virendeel action is a minimum. After the wall removal, the slabs could not span for three unsupported bays and collapse took place.

## 5.3 Effect of removed support location on structural behavior

The rotation values for slabs, columns and slab-column connections are generally higher for support loss in upper floors. Fig.7 shows the relation between removed support location and maximum slab rotation. All cases satisfied the UFC limits for slab rotation (2.86°) except for the case of the near-corner interior column removal from the $10^{th}$ floor and cases of edge shear wall removal from ground, $5^{th}$, $8^{th}$ and $10^{th}$ floors. The maximum slab rotation was in case of edge shear wall removal from $5^{th}$ floor. That is explained that the shear wall carries larger area of slabs compared to columns, and therefore, the unsupported bays after shear wall removal are relatively huge leading to higher rotation. However, although the interior shear wall have the same unsupported bays as the edge shear wall removal case, the interior shear wall removal showed less slab rotations. That can be explained that, after the interior wall was removed, its vertical load transferred to the remained parts of the core in addition to the adjacent columns. The existence of the remained core parts supported the slabs and helped decreasing the slab rotation. Also, the existence of the core in all the upper floors with its huge section helped spanning the three unsupported bays and prevented the structure collapse. The near-corner interior column removal case showed the highest slab rotation among cases of column removal followed by the interior column, the edge column, then the corner column. The rotation for case of near-corner interior column removal is higher than that of case of interior column due to the fact that the slab at the edge usually has higher deflections and rotations than that at interior supports.

As shown in Fig.8, the maximum column rotation did not exceed 0.85° which satisfies the UFC limits (0.86°). Maximum column rotation was in the cases of the near-corner interior column removal. The variation in column rotation with floor level showed a maximum value of 11.6%, except for the case of corner-column removal where the variation reached 30.2%.

All cases satisfied the UFC limits for joint rotation at slab- column connection (2.86°) as shown in Fig.9. Case of near-corner column removal at tenth floor showed the highest joint rotation among columns.

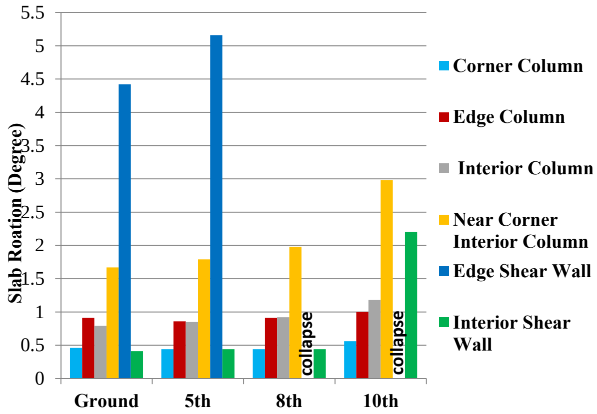



Fig.7 Effect of location of removed support on slab rotation

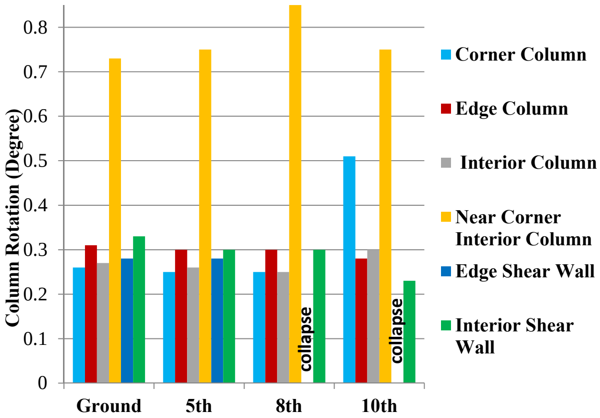



Fig.8 Effect of location of removed support on column rotation

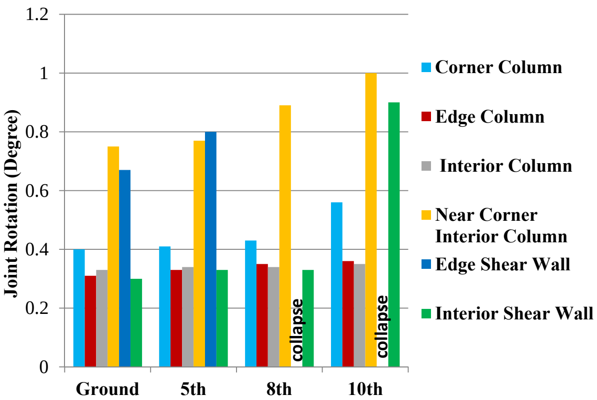



Fig.9 Effect of location of removed support on joint rotation

# 6. CONCLUSIONS

Based on the analytical results, the following conclusions can be obtained:

1-The flat slab structural system designed according to the ACI-318 (2008) can be generally considered appropriate system to resist the Progressive Collapse. All analysis cases satisfy the limits of (UFC) guidelines except for the case of near-corner interior column loss from the tenth floor and cases of edge shear wall loss from ground, fifth, eighth and tenth floors.

2-Losing a vertical support in a flat slab system is more critical in the upper floors than the lower ones and leads to higher deflections and higher probability of partial collapse.

3- Numerical results showed that the edge shear wall loss was the most critical case. The structure experienced a partial collapse due to its removal from 8th and 10th floors. Also it showed the highest values of deflection, slab rotation, and joint rotation when it was removed from ground and 5th floors.

4- Analysis showed that the loss of interior column near the edge of the building is also critical and gave high values of rotations exceeding the (UFC) limits inspite of non occurrence of collapse.

## REFERENCES


1) ACI 318-08, *Building code requirements for structural concrete and commentary*, American Concrete Institute; 2008
2) Applied Science International, LLC (ASI) www.appliedscienceint.com
3) Bathe, K., *Solution of Equilibrium Equations in Dynamic Analysis*. Prentice Hall, Englewoods Cliffs, N.J., 1995
4) Chopra, A., *Dynamics of Structures: Theory and Applications to Earthquake Engineering*. Prentice Hall, Englewoods Cliffs, N.J., 1995
5) Department of Defense, DoD, *Design of buildings to resist progressive collapse, Unified Facilities Criteria* (UFC, 4-023-03). USA; 2013.
6) General Service Administration, GSA, *Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects*; 2003.
7) Hartmann, D., Breidt, M., Nguyen, V., Stangenberg, F., Hohler, S., Schweizerhof, K., Mattern, S., Blankenhorn, G., Moller, B., and Liebscher, M. (2008), Structural Collapse Simulation under Consideration of Uncertainty –Fundamental Concept and Results, *Computers and Structures*, 86, 2064–2078.
8) Helmy H., Salem H., Tageldin H., Numerical simulation of Charlotte Coliseum demolition using the Applied Element Method. *USNCCM-10 conference-Ohio-USA*, 2009.
9) Helmy, H., Salem, H. and Mourad, S., Progressive collapse assessment of framed reinforced concrete structures according to UFC guidelines for alternative path method, *Engineering Structures*, 2012, 42, 127–141.
10) Helmy, H., Salem, H. and Mourad, S., Computer-aided assessment of progressive collapse of reinforced concrete structures according to GSA Code, *Journal of performance of constructed facilities, ASCE*, October 2013 (In Press).
11) Maekawa, K. and Okamura, H., The Deformational Behavior and Constitutive Equation of Concrete using the Elasto-Plastic and Fracture Model, *Journal of the Faculty of Engineering, The University of Tokyo (B)*,1983, 37(2): 253-328.
12) Meguro, K., and Tagel-Din, H., Applied Element Simulation of RC Structures under Cyclic Loading, *ASCE*, 2001, 127(11), 1295-1305.
13) Meguro, K., and Tagel-Din, H., AEM used for Large Displacement Structure Analysis, *Journal of Natural Disaster Science*, 2003, 24(1), 25-34.
14) Meguro K, Tagel-Din H., Applied element simulation of RC structures under cyclic loading, *ASCE*, 2001, 127(11), 1295-1305.
15) Meguro K, Tagel-Din H., Applied element method for structural analysis: theory and application for linear materials, *Structural Eng./Earthquake Eng., International Journal of the Japan Society of Civil Engineers (JSCE);* 2000; 17(1); 21s-35s.
16) Okamura, H. and Maekawa, K., *Nonlinear Analysis and Constitutive Models of Reinforced Concrete*. Gihodo, Tokyo, 1991.
17) Park H, Suk C, Kim S., Collapse modeling of model RC structures using the Applied Element Method, *Journal of Korean Society for Roc Mechanics, Tunnel& Underground Space*, 2009, 19 (1), 43-51.
18) Ristic, D., Yamada, Y., and Iemura, H., Stress-Strain Based Modeling of Hysteretic Structures under Earthquake Induced Bending and Varying Axial Loads, *Research report No. 86-ST-01, 1986, School of Civil Engineering, Kyoto University*, Kyoto, Japan.
19) Salem H., Computer-aided design of framed reinforced concrete structures subjected to flood scouring, *Journal of American Science*, 2011, 7(10), 191–200.
20) Salem H, El-Fouly A, Tagel-Din, H., Toward an economic design of reinforced concrete structures against progressive collapse, *Engineering Structures*, 2011, 33, 3341-3350.
21) Salem, H. and Helmy, H. “Numerical investigation of collapse of the Minnesota I-35W bridge”, Engineering Structures, 2014, 59, 635–645
22) Salem, H, Mohssen, S, Kosa, K, and Hosoda, A, “Collapse Analysis of Utatsu Ohashi Bridge Damaged by Tohuku Tsunami using Applied Element Method”, Journal of Advanced Concrete Technology, 2014, Vol. 12(10), 388-402.
23) Sasani M, Sagiroglu S., Progressive collapse resistance of hotel San Diego. Journal of Structural Engineering, 2008, 134(3), 478-488.
24) Sasani M., Response of a reinforced concrete infilled-frame structure to removal of two adjacent columns, *Engineering Structures*, 2008, 30, 2478–2491.
25) Tagel-Din, H., and Meguro, K., Applied Element Method for dynamic large deformation analysis of structures, *Structural Engineering and Earthquake Engineering. Journal of the Japan Society of Civil Engineers (JSCE)*, 2000, 17(2): 215-224.
26) Tagel-Din, H., Collision of structures during earthquakes", *Proceedings of the 12th European Conference on Earthquake Engineering*, 2002, London, UK.
27) Tagel-Din H, Meguro K., Applied Element Method for simulation of nonlinear materials: theory and application for RC structures, *Structural Eng./Earthquake Eng., International Journal of the Japan Society of Civil Engineers(JSCE)*, 2000, 17(2), 123s-148s.
28) Tagel-Din H, Meguro K., Applied Element Method for dynamic large deformations analysis of structures, *Structural Eng./Earthquake Eng., International Journal of the Japan Society of Civil Engineers (JSCE)*, 2000, 17(2), 215s-224s.
29) Wibowo H, Reshotkina S, Lau D., Modelling progressive collapse of RC bridges during earthquakes, *CSCE Annual General Conference*, 2009, GC-176-1-11.